Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering

CHALLENGES AND INNOVATIONS IN GEOTECHNICS

Actes du 18e Congrès International de Mécanique des sols et de Géotechnique

DÉFIS ET INNOVATIONS EN GÉOTECHNIQUE

Paris 2013

Edited by / Sous la direction de
Pierre Delage, Jacques Desrues, Roger Frank, Alain Puech, François Schlosser

VOLUME 1
Technical Committee 307

Sustainability in Geotechnical Engineering

Comité technique 307

Construction durable en géotechnique
General Report of TC 307
Sustainability in Geotechnical Engineering

Rapport général du TC 307
Durabilité en géotechnique

Basu D.
University of Waterloo, Canada

Puppala A.J., Chittoori B.
University of Texas at Arlington, U.S.A.

ABSTRACT: Sustainable geotechnics is an emerging sub-discipline within geotechnical engineering that covers a wide range of topics related to the sustainable development of civil infrastructure and society. In this general report, a brief overview of this new sub-discipline is provided with an aim to connect the broader scope of sustainability to geotechnical engineering research and practice. In addition, the papers allocated to the sustainability session (TC 307) of 18th ICSMGE are reviewed in the context of the big picture. Most of the papers deal with material recycling, reuse, and use of alternate materials in geotechnical engineering constructions. These apart, the topics covered in the allocated papers include use of geosynthetics, sustainable foundation engineering, subsurface remediation and site redevelopment, and sustainability assessment. Some of the important topics related to sustainable geotechnics not covered by the allocated papers are also mentioned.

RÉSUMÉ: La géotechnique soutenable est une nouvelle sous-disciplines de la géotechnique qui couvre un large éventail de sujets liés au développement durable des infrastructures et de la société. On présente dans ce rapport général un bref aperçu de cette nouvelle sous-discipline avec l'objectif de relier le champ plus large du développement durable avec la recherche et la pratique en géotechnique. En outre, les articles de la session dédiée à la géotechnique soutenable (TC 307) de la 18e ICSMGE sont examinés dans un contexte plus large. La plupart des articles traitent de recyclage de matériaux, de réutilisation, et de l'utilisation de matériaux alternatifs dans les constructions géotechniques. En plus de cela, les sujets abordés dans les articles incluent l'utilisation des géosynthétiques, les travaux de fondation durable, la décontamination et le réaménagement de sites et l'évaluation de la durabilité. Certains sujets importants liés à la géotechnique soutenable non couverts par les articles considérés sont également mentionnés.

KEYWORDS: sustainable geotechnics, geosustainability, geohazard, resilience, life cycle assessment, recycling, reuse, remediation.

1 INTRODUCTION

Sustainability is a multi-scale, multi-disciplinary and multi-dimensional paradigm that aims at ensuring the well being of the world for the current and future generations. With its origin in the environmentalism of the nineteenth and early twentieth century, sustainability has come a long way since its inception in the later half of the twentieth century and is now widely recognized as a principle that advocates a balanced development maintaining harmony between the three Es — environment, economy and equity (Edwards 2005). The environmental aspect has mostly been the driver of sustainability movement because of global concerns regarding the rise in atmospheric carbon dioxide and temperature, rapid depletion of natural resources, and other similar environmental and ecological hazards. The construction industry accounts for about 40% of the global energy consumption, depletes large amounts of sand, gravel and stone reserves every year, contributes to desertification, deforestation and soil erosion, and causes land, water and air pollution (Dixit et al. 2010, Kibert 2008, Puppala et al. 2012, Saride et al. 2010). Therefore, green practices within the civil engineering industry can reduce the impact of construction on the environment. Geotechnical design and construction, being placed early in a typical civil engineering project, can significantly contribute to sustainable development by adopting environment-friendly, cost-effective and socially-acceptable choices and setting a precedent for the remainder of the project. The role of geotechnical engineering in sustainable development is being increasingly recognized, as evidenced by the formation of the ISSMGE technical committee “Sustainability in Geotechnical Engineering” (TC307) in 2012. In fact, the 18th ICSMGE has set a precedent by devoting a technical paper discussion session to the sustainability theme.

The purpose of this general report is (i) to provide a perspective on the new area of sustainable geotechnics, and (ii) to review the papers allocated to the sustainability session of the 18th ICSMGE in the context of the big picture.

2 SUSTAINABILITY AND GEOTECHNOLOGY

Engineered systems serve human societies by developing cost-effective products and, in the process, draw resources from natural systems and generate emissions and wastes that nature has to absorb. Thus, engineered systems are intrinsically connected to the social, environmental, and economic systems. Because humankind is heavily dependent on engineered systems, sustainability of the physical world cannot be achieved without contributions from the engineered systems. This has been summed up by Basu et al. as the four Es of sustainability — engineering design, economy, environment and equity, as described in Figure 1.

Geo-structures and geo-operations often form important interfaces between the built and natural environments, and interact with and affect a wide variety of externalities. For example, dams and levees buffer the fluctuations in hydrologic cycles and affect water movement across regional and political boundaries; extraction of petroleum resources from the subsurface affects the natural environment and global economy; and landfill systems prevent contaminants from reaching groundwater across regional scales. Further, geotechnical engineering has a very important role in mitigating and containing disasters, and failure to do so is often catastrophic to the society. Breach of a levee system during a hurricane or tsunami, breakdown of underground water pipeline network during an earthquake, landslides triggered by rainfall or
earthquake, disruptions and distress in an underground transit system due to terror attack are examples of disasters related to geotechnical systems. Thus, geotechnical engineering has a wide gamut and a global reach, and can influence the sustainable development of infrastructure and civil societies in a significant way. According to Long et al. (2009), there are seven categories where geotechnical engineering can contribute to improve the sustainability of the societal system. These include (i) waste management, (ii) infrastructure development and rehabilitation, (iii) construction efficiency and innovation, (iv) national security, (v) resource discovery and recovery, (vi) mitigation of natural hazards, and (vii) frontier exploration and development. A similar set of sustainability objectives for geotechnical engineering was also identified by Pantelidou et al. (2012): (i) energy efficiency and carbon reduction, (ii) materials and waste reduction, (iii) maintaining natural water cycle and enhancing natural watershed, (iv) climate change adaptation and resilience, (v) effective land use and management, (vi) economic viability and whole life cost, and (vii) positive contribution to society.

3 THEMES COVERED IN SUSTAINABILITY SESSION

There are 28 papers allocated to the sustainability session with authors from 20 countries representing all the ISSMGE regions. These papers cover a wide range of topics that can be broadly grouped into five areas.

3.1 Use of recycled and alternate materials

According to Vaniček et al., a variety of waste products are generated in the society that can be utilized in geotechnical constructions. These waste products can be categorized into industrial wastes (e.g., ash and slag), construction and demolition wastes (e.g., used bricks, concrete, and asphalt), mining wastes (mine tailings), and other wastes (e.g., tires, plastics, glass, and dredged material). Basu et al. provided an overview of the different waste utilization methods in geotechnical constructions and discussed about chemical soil treatment. Waste utilization and use of alternative material is one of the most widely researched areas in geotechnical engineering and it is not surprising that, out of the 28 papers allocated to this session, 20 papers contribute to this topic.

The papers on industrial waste recycling deal with a variety of geotechnical applications. Baykal investigated the use of silt-sized fly ash in manufacturing artificial, sand-sized pellets for use in construction projects (Figure 2). He reviewed the cold bonding pelletization technique, and studied the index and mechanical properties of the fly-ash pellets. The manufactured pellets behave like calcareous sands found in the nature.

Figure 2. Manufactured fly ash pellets (Figure 2 of Baykal).

In another example of recycling of fly ash, Vukičević et al. investigated the reusability of a class-F fly ash (KFA) from a Serbian thermal power plant as a stabilizer in low plasticity silt and in high plasticity expansive clay. Several geotechnical engineering properties including grain size distribution, Atterberg limits, unconfined compression strength, moisture-density relationship, swell potential, and California bearing ratio (CBR) were determined for the control and treated soils. Based on the study, the authors concluded that the particular fly ash in question can be used as a stabilizer, and advocated a case-by-case approach.
case approach with proper investigations for making decisions regarding the suitability of fly ash as a construction material. Kikuchi and Mizutani proposed the use of granulated blast furnace slag (GBFS) as an alternative construction material for port structures because GBFS can reduce liquefaction potential and earth pressure when used as a backfill material for quay walls. The inherent ability of GBFS to solidify upon contact with seawater was explored and methods were proposed for its standardized application in the field. As GBFS solidification is a lengthy process and often the solidification is not uniform, Kikuchi and Mizutani proposed the use of powdered blast furnace slag (PBFS) in conjunction with prior homogeneous mixing treatment (PHMT) to accelerate the GBFS solidification process. In their experimental investigation, Kikuchi and Mizutani considered several issues, e.g., material separation after construction due to water flow, solidification of GBFS underground with flowing water, and the effect of the change in pore fluid chemistry due to a change from sea to fresh water on GBFS solidification, in determining the most appropriate mixture of GBFS and PBFS for accelerating the GBFS solidification. The authors found that PHMT treated GBFS-PBFS mixture is effective in reducing the amount of material separation in the GBFS-PBFS mixture and produced sufficient unconfined compression strength after about 2 months of curing in the seawater because of which it can be used to prevent liquefaction. Nawagamuwa et al. investigated the properties of waste copper slag for use in vertical sand drains and sand piles as a substitute for sand. Geotechnical properties such as particle size distribution, hydraulic conductivity, shear strength, and stiffness were studied for the sand-sized waste copper slag particles mixed with poorly graded sand. It was observed that the particle size distribution, shear strength and hydraulic conductivity were not significantly affected due to the addition of the slag. However, the stiffness of the slag-sand mixture increased significantly. Based on the study, Nawagamuwa et al. concluded that waste copper slag can be safely and effectively used as a replacement for sand in vertical drains.

Vizcarra et al. (2013) investigated the applicability of municipal solid waste (MSW) incineration ash mixed with non-lateritic clay in pavement base layers. Chemical, physical, index, and mechanical tests were performed on the ash-soil mixture with 20% and 40% ash content, and the mechanistic-empirical design (Figure 3) for a typical pavement structure were carried out. The mechanical tests included modified Proctor test, resilient modulus test, and permanent deformation test. The addition of 20% fly ash to the non-lateritic clay soil improved the mechanical behavior and reduced the expansion of the clay. The fly ash mixed soil had a mechanical behavior compatible with the requirements for a low traffic volume. Edil also focused on pavement geotechnics and provided an overview of different recycled waste products used in pavement construction. He discussed about the rapid characterization of industrial wastes like fly ash and bottom ash, and construction and demolition wastes (CDW) like recycled asphalt pavement and concrete aggregates with respect to their physical characteristics, geomechanical behavior, durability, material control, and environmental impact.

In another study related to pavements, Cameron et al. proposed the use of recycled concrete aggregates (RCA) blended with recycled clay masonry (RCM), obtained after demolition, in unbound granular pavements. The CDW were obtained from two local producers in South Australia, and conventional classification tests for soils and aggregates, Los Angeles abrasion test, Micro-Deval test, falling head permeability test, drying shrinkage test, undrained triaxial and repeated loading triaxial tests, and permanent strain rate modeling were performed. The test results were compared with the specifications from road authorities both within and outside Australia, and the RCA products were classified as Class 1 or base and the blended products as Class 2 or subbase materials.

Farias et al. also studied the feasibility of using CDW in paving of a shopping-center site in Recife, Pernambuco, Brazil. They performed a series of physical, chemical and mechanical tests with mixtures of different proportions of CDW obtained from the site and in situ excavated soil, and concluded that the recycled residues of civil construction (RRCC) alone and RRCC mixed with soil meet all the criteria of the local standard NBR 15.116:2004. Farias et al. (2013) also performed an economic analysis of different construction alternatives with the RRCC, which is described in section 3.5. The study by Santos et al. also involves CDW. They presented a laboratory-scale experimental investigation on the performance of instrumented wrapped-faced retaining walls constructed using recycled construction and demolition wastes (RCDW) consisting of soil, bricks, and small particles of concrete. CDW is abundantly available in Brazil and approximately 70% by mass of municipal solid waste consist of CDW. CDW was found to have excellent mechanical and chemical properties for use as a back-fill material in geosynthetics reinforced walls. Consequently, two 3.6-m high, wrapped-faced retaining walls with facing batter angle of 13° were constructed at the University of Brasilia (UnB) Retaining Walls Test Facility. One retaining wall was constructed with geogrid and the other with geotextile with identical reinforcement lengths and spacings of 2.52 m and 0.6 m, respectively, using RCDW as the compacted backfill (Figure 4). The walls were instrumented along their central sections to measure strains, displacements, and earth pressures. The walls performed well during and after construction with the maximum horizontal displacement at the wall face being 150 mm. The only downside was the creation of uneven surfaces near the face due the presence of coarse particles. The use of a selected RCDW near the face for better aesthetic appeal was recommended.

Vaníček et al. presented an example of waste recycling in which a new construction material consisting of brick, fiber and concrete was used to reinforce dykes for flood protection and erosion control.

Winter discussed the use of lightweight tire bales (Figure 5) as a potential alternative for pavement foundation on soil soils. Tire bales comprise of 100 to 115 tires of light-goods vehicles and cars compressed into a lightweight block with a mass of about 800 kg and density of approximately 0.5 Mg/m³. The bales measure approximately 1.3 m × 1.55 m × 0.8 m and are secured by five galvanized steel tie-wires running around the length and depth of the bale. The key advantage of tire bales is their modular nature which leads to potential savings in plant, labor, and time. These bales have been used in pavement construction, slope protection, river bank erosion control, and lightweight embankment constructions. Winter described the different construction techniques and provided information regarding the measurement of properties, engineering properties
and behavior associated with tire-bale use in construction, example applications, and end-of-service-life options.

Abdelhaleem et al. considered the use of recycled rubber and rubber-sand mixtures (RSM) as replacement soils in seismic areas due to the increased damping capacity of RSM. They performed site response analysis using the two-dimensional finite element method with equivalent-linear constitutive models for the geo-materials. Three earthquake ground motions of comparable magnitude and varying frequency content were applied to a deposit of sand with replacement soil and with different configurations of RSM. A parametric study was performed for investigating the effect of depth and thickness of the RSM layer and of the relative magnitudes of the natural period of the site and predominant period of earthquake on the sand-replacement soil-RSM system.

Kalumba and Chebet investigated the possibility of using discarded polyethylene shopping bags as soil reinforcement, and performed direct shear tests on Klipheuwel and Cape Flats sands mixed with perforated and non-perforated polyethylene strips of different lengths and of widths (Figure 6). Direct shear tests were performed with sand-polyethylene mixture and it was observed that there was an overall increase in the friction angle due to addition of the strips and that the increase in the friction angle depends on the length and width of the strips, perforations present in the strip, and percent weight of the strips (see, for example, Figure 7). Based on their results, Kalumba and Chebet suggested that the polyethylene strips can be used to increase the shear resistance of sandy soils.

Abdelrehman et al. performed a laboratory-scale study to investigate the efficacy of expanded polystyrene (EPS), a cellular polymeric material commonly used in the packaging industry, in reducing the heave in footings placed on expansive clay (Figure 8). They studied the compaction characteristics of EPS of different size and bead density mixed with silica sand. Subsequently, Abdelrehman et al. studied the response of circular footings of different diameters resting on a layer of sodium bentonite by replacing a part of the bentonite layer with the EPS-sand mixture. They performed a parametric study of the footing heave-settlement response as a function of different proportions of EPS-sand mixture, different replacement soil-layer thickness, footing size, and bead density. Abdelrehman et al. found that the swelling deformation of the footing decreases as the replacement-layer thickness increases.

In another example of EPS recycling, Teymur et al. compared the performance of glass foam and EPS geo-foam as components of controlled low strength material (CLSM) often used as compacted backfill. They performed index tests, unconfined compression tests, and CBR tests, and found that glass mixtures have greater unit weight and strength than those of EPS foam mixtures. They concluded that glass foam CLSM can be used as pavement subbase, as fill for slopes and retaining structures, and to increase the strength and stiffness of soft clay deposits.

Drinking water sludge (DWS) discharged during water purification has potential use as a road infrastructure material (Watanabe and Komini). However, decomposition of the organic matter present in DWS decreases its shear strength because of which it is important to determine its durability for reuse. Watanabe and Komini collected DWS samples from Ibaraki, Japan that contains aluminum and organic matters in the solid phase, and performed triaxial tests on the samples after subjecting them to aluminum leaching and biodegradation. They found that the shear strength of DWS decreases due to loss of organic matter and aluminum (Figure 9). Watanabe and Komini further quantified the effect of aluminum leaching and biodegradation.
organic loss on the shear strength by modeling the leaching as a diffusion process and the organic loss as an exponential decay process. The study shows that DWS can be used in geotechnical applications.

![Figure 9. Friction angle of drinking water sludge as a function of decomposition rate of organic matter (Figure 7 of Watanabe and Komini).](image)

Di Emidio et al. investigated the possibility of reusing dredged materials in landfill cover as a low-cost alternative. Enormous amounts of dredged material are generated from maintenance, construction, and remedial works related to water systems, and these materials are usually disposed of in landfills. Therefore, the reuse of dredged materials is important all over the world. For use in landfill cover, the dredged material must have low hydraulic conductivity and must retain the contaminants already present in it. In their study, Di Emidio et al. used dredged sediment obtained from Kluizenendok in Ghent, Belgium and commercially processed kaolin Rotoclay® HB clay, and treated both with an anionic polymer Sodium CarboxyMethylCellulose (Na-CMC). Polymerization is particularly useful for dredged materials contaminated with metallic wastes. The authors investigated the mechanisms through which polymers can improve the efficiency of dredged sediments in waste containment impermeable barriers. Di Emidio et al. also conducted hydraulic conductivity and batch sorption tests to study the barrier performance and transport parameters of the treated dredged material and clay. The results showed that polymer treatment maintained low hydraulic conductivity of soil in electrolyte solutions and helped the material contain the spread of pollution. The results indicated that dredged sediments can be reused as alternative low-cost impermeable landfill cover.

Nakano and Sakai performed consolidation and triaxial tests on cement treated dredged soil samples collected from Nagoya Bay, Japan and modeled their elemental behavior using the SYS Cam-clay model. About 1.3 million m³ of dredged soil is produced annually in Nagoya Bay, which has limited storage capacity because of which there is a pressing need for using the dredged soil as a geo-material. However, the clayey soil has low shear strength and high water content because of which cement is used as a stabilizer to improve its mechanical properties. The constitutive model of Nakano and Sakai reproduced the elemental test results reasonably well and the authors also performed finite element analysis using the software GEOASIA in order to capture the nonuniform deformation of triaxial test samples.

Air-foam treated lightweight soil, known as Super Geo-Material (SGM), is an example of an alternate material that is useful in harbor and airport constructions because of its light weight, safety features, and recyclability. Kataoka et al. mixed six different types of soils from Japan with seawater, blast furnace cement, and animal-protein hydrolyzed air-foam to prepare SGM specimens. They measured the unconfined compressive strength and small-strain shear modulus of the specimens, and studied their microstructure using a scanning electron microscope. Kataoka et al. observed that the strength and stiffness of the SGM samples increased with increase in the number of curing days, and attributed this increase to the growth and bonding of needle-like ettringite crystals within the SGM sample pores caused by the curing process.

Jeffers and Lam discussed the use of polymers as an alternative to bentonite in geotechnical construction fluids ( slurries). Polymers have several advantages over bentonite in that polymer fluids require smaller preparation plants that can access congested urban areas, require shorter preparation time, and are environmentally less hazardous. In addition, constructions made with polymers have better performance than their bentonite counterparts. However, there are some limitations of polymers like reduction of fluid properties due to continued shear in recirculation systems and potential for loss of properties in saline soils. Therefore, Jeffers and Lam recommended that polymers should be used carefully with proper monitoring.

In order to investigate the reusability of in situ excavated soil with poor mechanical properties, Blanck et al. studied the effect of three non-traditional additives, an acid solution, an enzymatic solution, and a lignosulfonate, on the compaction characteristics and strength of silt. The test results showed that the acid solution did not improve the compaction characteristics and that adequate soil compaction can be achieved with low water content using the enzymatic solution and lignosulfonate. Blanck et al. concluded that enzymatic and lignosulfonate treatments would reduce water usage in constructions.

### 3.2 Efficient use of geosynthetics

The use of geosynthetics can reduce resource consumption and environmental impacts of geotechnical constructions, and can prevent soil erosion (Herten et al., 2012). Frischknecht et al. showed through life cycle assessment of pavement drainage systems that constructions using geosynthetics have less environmental impact.

Herten et al. presented a general discussion on the use of geosynthetics, particularly geogrids, and pointed out that constructions with geosynthetics is more economical and environment friendly than traditional alternatives. According to Herten et al., political reasons and population density and distribution often dictate the construction choices related to the national and international traffic routes within the European Union (EU), and geosynthetics can be used to advantage in many such constructions. The authors discussed about the use of geosynthetics in slope stabilization, reinforced earth walls, sound barrier walls, and embankments on soft clay, and pointed out the beneficial features of geosynthetics. They also discussed about the provisions given in Eurocode-7, German standards and British standards regarding constructions related to geosynthetics. Based on Herten et al., it can be concluded that efficient, economic, aesthetically pleasing, and environment friendly constructions with minimal monitoring requirement are possible using geosynthetics.

### 3.3 Sustainable foundation engineering

Foundations form an integral part of geotechnical constructions, and sustainable design and construction of foundations are very important for overall sustainable development (Basu et al.). As part of sustainable foundation engineering, Basu et al. advocated the use of proper constitutive models and appropriate numerical analyses, adoption of reliability based design approach (e.g., LRFD), incorporation of spatial heterogeneity of soil in analysis and design, adoption of economical and environment friendly construction practices, reuse and retrofitting of existing foundations, and use of foundations in harvesting wind and geothermal energy.

Bourne-Webb et al. presented a case study of piled raft construction for a shopping center in Cambridge, UK as a cost-effective, time-saving and resource-efficient alternative to conventional pile foundations. Based on detailed site
characterization that consisted of collection of data from adjacent sites, stress-path tests on bore hole samples, suction tests, and estimation of coefficient of earth pressure at rest, on monitoring of an instrumented basement of an adjacent hotel structure, and on linear and pseudo-nonlinear soil structure interaction analysis considering plate-on-spring approach, the piled raft foundation was designed with the piles used as settlement reducers. The design also ensured that there was minimal disturbance to the adjacent structures.

Reuse of foundations is often preferred over new foundations because reuse reduces waste disposal and environmental impact. Guillon et al. presented three case studies of foundation reuse projects in Paris and Pantin. Figure 10 shows a cross section of one of the rehabilitation projects in Paris in which the existing pile foundations were strengthened by jet grouting and additional support was provided by newly installed micropiles. Guillon et al. concluded that a site specific approach involving proper site characterization, condition assessment of existing foundations, delineation of existing and new foundation geometry, accurate estimation of changes in load, deformation and capacity during the construction process, consideration of different possible construction alternatives, proper choice of reinforcement technique, and proper monitoring is required for successful reuse of foundations. Vaniček et al. also advocated reuse of foundations particularly in the context of brownfield redevelopment.

Figure 10. A cross section of foundation reuse project of Calberson warehouses, MacDonald Boulevard, Paris (Figure 1 of Guillon et al.).

3.4 Subsurface remediation and site redevelopment

Redevelopment of contaminated sites including brownfield sites and landfills is an important part of sustainable geotechnics. According to Vaniček et al., brownfield redevelopment involves an initial reconnaissance study involving site characterization and economic feasibility study, followed by detailed site investigation, site remediation, and construction of new facilities at the site. Site remediation involves ground improvement by physical means (e.g., compaction and clay injection) and chemical treatment using encapsulation, permeable reactive barrier (Figure 11) and chemical stabilization. Vaniček et al. also provided an example of use of coal mine sites in Czech Republic where clayey overlays covering coal seams were excavated during mining activities and subsequently backfilled, and constructions were made on the mine sites using the mine-spoil heaps.

Figure 11. Site remediation by permeable reactive barrier (adapted from Figure 4 of Vaniček et al.).

McIntosh and Barthelmess provided a case study of reuse of a derelict (puticible waste) landfill site at Unanderra, NSW, Australia. Different geotechnical and geoenvironmental studies were conducted to assess the potential of the landfill site for construction. Environmental monitoring included testing of groundwater for contaminants and metals, and of gas monitoring wells for methane, hydrogen sulphate, and carbon dioxide. Monitoring for vibrations, noise and dust produced during site preparation was also conducted. The landfill density was increased by dynamic compaction, and environmentally neutral coal was rejected in the landfill site were used as cheap backfill material. A leachate control pond was constructed to receive the leachate during compaction and also to manage storm water on a long-term basis. Future civil and building services have been designed such that they do not penetrate the capping layer of the landfill. Driven steel piles bearing on underlying latite bedrock will be designed as building foundations. Flexible aprons will be provided between buildings and adjacent car parks, walkways and recreation areas, and raft slabs may be feasible for some lightweight, single-story buildings. The environmental design includes capping consisting of HDPE, GCL, geotextile fabric, 300 mm gravel drainage layer with a reinforcing geotextile, a gas drainage layer forming part of the cap, and leachate collection drains.

3.5 Sustainability assessment

The foregoing studies show that geotechnical engineering can contribute significantly to solutions of sustainability problems. Most studies are based on the common notions of sustainability—like recycling, reuse, and use of alternate materials, technologies and resources. However, whether such new approaches are actually sustainable or not cannot be ascertained without proper assessment using, for example, whole life cost analysis and risk based performance analysis. Thus, a sustainability assessment framework is necessary for geotechnical projects to ascertain the relative merits of different options available for a project.

Frischknecht et al. performed a comparative life cycle assessment (LCA) of a pavement filtration system by comparing the performance of a gravel filter and a geosynthetics-based filter drain with the same hydraulic conductivity of 0.1 mm/s or more and with the same design life of 30 years. Life cycle inventory (LCI) of gravel and geosynthetics filter for a m² functional unit was performed — Table 1 shows some key figures of LCI. Polypropylene granules was used as the basic material for the geosynthetics filter, and the LCI of geosynthetics manufacturing was performed using the ecoinvent data v2.2 based on the categories of raw materials, water, lubricating oil, electricity, thermal energy, fuel for forklifts and factory building. The environmental impact assessment (EIA) was performed considering eight impact indicators: cumulative energy demand, global warming potential, photochemical ozone formation, particulate formation, acidification, eutrophication, land competition, and water use. Based on the study, Frischknecht et al. found that the geosynthetics based filter layer causes lower environmental impact than the conventional gravel-based drain and that the environmental impact of
geosynthetics manufacturing is mostly controlled by the impacts of raw-material production and electricity consumption during manufacturing.

Table 1. Selected key figures describing the constructions of one square meter of gravel and geosynthetics filter (Table 2 of Frischknecht et al.).

<table>
<thead>
<tr>
<th>Material/Process</th>
<th>Unit</th>
<th>Gravel filter</th>
<th>Geosynthetics filter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>t/m²</td>
<td>0.69</td>
<td>0</td>
</tr>
<tr>
<td>Geosynthetics layer</td>
<td>m³/m²</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Diesel used in building machines</td>
<td>MJ/m²</td>
<td>2.04</td>
<td>1.04</td>
</tr>
<tr>
<td>Transport, lorry</td>
<td>tkm/m²</td>
<td>34.5</td>
<td>0.035</td>
</tr>
<tr>
<td>Transport, freight, rail</td>
<td>tkm/m²</td>
<td>0</td>
<td>0.07</td>
</tr>
<tr>
<td>Particulates &gt; 10 µm</td>
<td>g/m²</td>
<td>4.8</td>
<td>0</td>
</tr>
<tr>
<td>Particulates &gt; 2.5 µm and &lt; 10 µm</td>
<td>g/m²</td>
<td>1.3</td>
<td>0</td>
</tr>
</tbody>
</table>

Holm et al. developed an assessment and decision making tool for sustainable management of contaminated sediments in the Baltic sea, which included an emerging technology of solidification/stabilization. They presented the results of three case studies based on the ports of Oxelösund (Sweden), Gävle (Sweden) and Hamburg (Germany). Different management scenarios were considered at each port, and LCA were performed to choose the best options. Recycling of sediments, disposal in river and sea, energy use, and environmental impact were considered in the LCA. Holm et al. further developed a multicriteria decision analysis (MCDA) to integrate the three Es of sustainability in their decision making tool following a structured and balanced way.

Basu et al. also developed a multicriteria based sustainability assessment framework and applied it to pile foundation projects. The framework considers a life-cycle view of the pile construction process, and combines resource consumption, environmental impact, and socio-economic benefits of a pile-foundation project over its entire life span to develop a sustainability index (Figure 12).

Edil provided an overview of the sustainability assessment tools used in pavement construction projects. He mentioned that LCA and life cycle cost analysis (LCCA) can be successfully used to assess the sustainability of pavement constructions. He further described a rating system for sustainable highway constructions known as Building Environmentally and Economically Sustainable Transportation-Infrastructures-Highways (BE’ST-in-Highways™), which evaluates the sustainability of a highway project in terms of quantitative difference between a reference design and proposed alternative designs.

Farias et al. performed an economic analysis of different construction alternatives for their CDW paving project described in section 3.1. Although this is not a complete sustainability analysis, the environmental and social benefits are inherently present in the project. They considered two alternatives, first in which the CDW is completely disposed of in landfills and second in which the CDW and in situ soil mixture is used in paving the construction site. The high cost of disposal (Table 2) made the first option the most viable one with a direct cost savings of US$ 1.9 million, which does not even include the indirect cost-saving benefits from the reduced environmental impact that the project ensures.

Table 2. Costs for final disposition of wastes in licensed places (Table 6 of Farais et al.).

<table>
<thead>
<tr>
<th>Disposition place</th>
<th>Unit</th>
<th>Unitary cost (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inert landfill</td>
<td>m³</td>
<td>47.30</td>
</tr>
<tr>
<td>Processing plant</td>
<td>m³</td>
<td>18.36</td>
</tr>
</tbody>
</table>

In their study on the use of enzymatic solution and lignosulfonate as additives in silt (described in section 3.1), Blanck et al. performed LCA-EIA using 10 impact categories proposed in the NF P 01-010 standard. The analysis showed that the use of enzymatic solution reduces impacts in seven out of ten categories (Figure 13). The use of lignosulfate, however, did not produce sufficient environmental benefits.

Figure 13. Results of environmental impact analysis for the use of enzymatic solution (Figure 3 of Blanck et al.).

4 IMPORTANT THEMES NOT COVERED

As sustainable geotechnics covers a wide range of topics, it is natural that the papers allocated to the sustainability session do not cover all the areas related to geosustainability. Some of the important topics not covered in details include sustainable site characterization, geovalorization mitigation, reliability- and resilience-based analysis and design, geothermal energy foundations, geo-structures for wind and solar energy, sustainable ground improvement techniques, sustainable use of underground space, carbon sequestration, and ethical practices in geotechnical engineering. The other sessions and workshops of 18th ICMSGE cover some of these themes.

5 CONCLUSIONS

Sustainable geotechnics is a new sub-discipline focusing on geotechnical engineering practices that reduce the detrimental effects of geotechnical constructions and ensure the well being of the society and natural environment at all times. It not only includes environment-friendly practices that are cost effective and cause minimal financial burden to the present and future generations, but also promote reliability- and resilience-based design and adaptive management strategies so that social vulnerability is minimized and overall well being is upheld.

This general report provided an overview of this emerging area of geosustainability and reviewed the twenty eight papers allocated to the sustainability session of 18th ICMSGE. The authors of these papers represent 20 countries covering all the ISSMGE regions. Most of the papers emphasized the
environmental aspect of sustainability, while some papers on sustainability assessment focused on the economic and social aspects. The topics covered by these papers can be broadly classified into five sub-themes: material recycling, reuse and use of alternate materials, efficient use of geosynthetics, sustainable foundation engineering, subsurface remediation and site redevelopment, and sustainability assessment. Although these papers deal with a variety of topics that contribute to the sustainable development of civil infrastructure and society, they do not cover all the important topics that are parts of sustainable geotechnics.

5 ACKNOWLEDGEMENT
Alain Corfdir helped us in interpreting the papers written in French for which we are grateful.

6 REFERENCES
Blanc G., Cuisinier O. and Masroui F. 2013. Méthodes non traditionnelles de traitement des sols : apports techniques et impact sur le bilan environnemental d’un ouvrage en terre. Proc.18th ICSMGE.
Edil T. B. 2013. Characterization of recycled materials for sustainable construction. Proc. 18th ICSMGE.
Guilloix A., Le Bissonnais H., Sausac L. and Perini T. 2013. La réutilisation des fondations existantes dans les projets de réhabilitation de constructions anciennes. Proc. 18th ICSMGE.
Jeffers S. A. and Lam C. 2013. Polymer support fluids: use and misuse of innovative fluids in geotechnical works. Proc. 18th ICSMGE.
McIntosh G. W. and Barthelness A. J. 2013. Building on an old landfill: design and construction. Proc.18th ICSMGE.
Nakano M. and Sakai T. 2013. Interpretation of mechanical behavior of cement-treated dredged soil based on soil skeleton structure. Proc. 18th ICSMGE.
Teymur B., Tuncel E. Y. and Ahmedov, R. 2013. Comparing the properties of EPS and glass foam mixed with cement and sand. Proc. 18th ICSMGE.
Viczarr I., Szeliga L., Casagrande M. and Motta L. 2013. Applicability of municipal solid waste (MSW) incineration ash in road pavements. Proc.18th ICSMGE.
Vukčević M., Marać-Dragojević S., Joković S., Marjanović M. and Pujević V. 2013. Research results of fine-grained soil stabilization using fly ash from serbian electric power plants. Proc.18th ICSMGE.
Winter M. G. 2013. Road foundation construction using lightweight tyre bales. Proc. 18th ICSMGE.
Evaluation of Rubber/Sand Mixtures as Replacement Soils to Mitigate Earthquake Induced Ground Motions

Abdelhaleem A.M.
Construction Research Institute, National Water Research Center, Egypt

El-Sherbiny R.M., Lotfy H.
Cairo University, Egypt

Al-Ashaal A.A.
Construction Research Institute, National Water Research Center, Egypt

ABSTRACT: Use of recycled rubber and rubber/sand mixtures (RSM) as lightweight material has been widely growing over the past decade. The increased damping capacity of RSM leads to considering its use as replacement soils in seismic areas to reduce the amplitude of earthquake induced ground motions. This paper presents a study on the effect of utilizing a layer of RSM within a replacement soil on the ground response during an earthquake. Site response analyses were performed using 2D finite element analyses applying an equivalent-linear constitutive model. Three earthquake ground motions of varying frequency content were applied to a deposit of sand with replacement soil having different configurations of RSM. Placing a layer of RSM within the replacement soil resulted in increasing the site natural period causing damping of spectral accelerations at low periods and amplification of spectral accelerations at high periods. Using a thin layer of RSM at deeper depths was more effective in than using thick but shallow RSM layers. The results indicate that RSM layers may be effective when the predominant period of the earthquake is lower than the site natural period, while the configuration is subject to the natural period of the intended structure.

RÉSUMÉ: Le caoutchouc recyclé et les mélanges sable-caoutchouc (MSC) en tant que matériaux légers ont eu une utilisation accrue au cours de la dernière décennie. L'augmentation de la capacité d'amortissement de MSC conduit à considérer son utilisation en tant que remplacement des sols dans les zones sismiques afin de réduire l'amplitude des secousses observées pendant les tremblements de terre. Cet article présente les résultats d'une étude sur l'influence des couches de MSC comme sol de remplacement sur la réponse du sol au cours d'un tremblement de terre. L'analyses de réponses du site ont été réalisées en utilisant la méthode d'éléments finis 2D appliquée sur un modèle constitutif du type « linéaire équivalent ». Trois régimes de tremblements de terre de fréquence variable ont été appliqués à un dépôt de sable avec terres de remplacement ayant différentes formulation de MSC. Placer une couche de MSC dans le sol de remplacement a eu pour effet d'augmenter la période naturelle du site; et ceci provoque une atténuation des accélérations spectrales à des périodes faibles et une amplification de la même accélération aux périodes fortes. L'application d'une mince couche de MSC à des profondeurs importantes a été plus efficace que d'utiliser des couches épaisses peu profondes. Les résultats indiquent que les couches MSC ne peuvent être efficaces que si la période dominante du tremblement de terre est inférieure à la période naturelle du site, et ceci en maintenant la configuration soumise à la même période naturelle de celle de la structure en question.

KEYWORDS: Recycled Material, Rubber-Sand Mixture, Replacement Soil, Earthquake Mitigation

1 INTRODUCTION

The use of recycled rubber and rubber/sand mixtures (RSM) as lightweight material in civil engineering applications has been widely growing over the past decade. Processed waste tires mixed with soils have been introduced as lightweight fills for slopes, retaining walls, and embankments. The mechanical properties of the mixture were discussed by (Edil and Bosscher, 1994; Ghazavi, 2004; Zornberg et al., 2004; and Mavroulidou et al., 2009), while dynamic properties of granulated rubber-sand mixtures were studied by (Feng et al., 2000; and Anastasiadis et al., 2012). Xu et al. (2009) performed numerical studies on protecting buildings from earthquakes hazards by RSM.

The utilization of RSM as replacement soils in seismic areas to reduce the amplitude of earthquake induced ground motions is addressed in this paper. The effect of changing the depth and thickness of the RSM layer will be investigated in this study. The results will be compared for a range of medium amplitude ground motions.

Table 1. Properties of sand and granulated rubber

<table>
<thead>
<tr>
<th>Material</th>
<th>Sand</th>
<th>Granulated rubber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight, $\gamma$ (kN/m$^3$)</td>
<td>16.50</td>
<td>6.50</td>
</tr>
<tr>
<td>Specific gravity, Gs</td>
<td>2.67</td>
<td>1.10</td>
</tr>
<tr>
<td>Max. particle size, $D_{50}$ (mm)</td>
<td>0.85–2.00</td>
<td>4.75–6.35</td>
</tr>
<tr>
<td>50% passing size, $D_{10}$ (mm)</td>
<td>0.56</td>
<td>2.8</td>
</tr>
<tr>
<td>Coefficient of uniformity, Cu</td>
<td>2.76</td>
<td>2.29</td>
</tr>
<tr>
<td>Coefficient of curvature, Cc</td>
<td>1.23</td>
<td>1.18</td>
</tr>
</tbody>
</table>

The RSM used in the analyses herein was assumed to contain 35% rubber content (by weight) and a dry unit weight of 12.5 kN/m$^3$. The modulus reduction and damping curves of dry RSM (C3D06-R) for different confining pressures ($\sigma'$) were generated according to Senetakis et al. (2012). The modulus...
reduction and damping curves of dry rubber-sand mixture (C3D06-R3), sand (C3D06), and the replacement soil at confining pressures ($\sigma'_m = 50$ kPa) are shown in Figure 1. The small strain shear moduli for the sand, RSM, and replacement soil are 65.6 MPa, 10.4 MPa, and 234 MPa, respectively.

3 NUMERICAL MODEL

A number of two-dimensional finite element models were built in QUAKE/W to evaluate the site response during an earthquake. The soil was modeled using an equivalent linear constitutive model. The baseline case representing the untreated site condition constitutes a 20 m thick layer of sand above bedrock. Two additional layers were inserted into the original model to simulate replacement soil and RSM layers in the different numerical analyses, as shown in Figure (2). The width of the RSM layers was assumed 20m.

![Figure 1. The modulus reduction and damping curves at ($\sigma'_m = 50$ kPa)](image1)

![Figure 2. The FEM model used in the numerical study](image2)

The influence of two parameters on the site response was studied, namely the depth of the rubber-sand mixture layer ($Y$) and the thickness of the rubber-sand mixture layer ($h$).

4 EARTHQUAKE GROUND MOTIONS

Three earthquake ground motions of comparable magnitude and different frequency content were used to investigate the ground surface layer response in case of pure sand deposit (baseline case) and in cases of the existence of the RSM layer. The earthquake ground motions data were obtained from the ground motion database of the Pacific Earthquake Engineering Research Center (PEER). The ground motion database includes a very large set of ground motions recorded in worldwide shallow crustal earthquakes in active tectonic regimes. Figure (3) shows the response spectrum of earthquake input ground motions, and Table (2) summarizes their characteristics. The predominant period of the selected input ground motions varies between lower than the site natural period ($T_{site} = 0.5$ sec) such as in Lytel Creek ($T_p = 0.08$ sec) and San Francisco ($T_p = 0.26$ sec), and greater than the site natural period such as in Mammoth Lake earthquake ($T_p = 0.925$ sec) in order to cover a range of frequency contents for intermediate earthquakes.

![Figure 3. Earthquake ground motions](image3)

<table>
<thead>
<tr>
<th>Event Name</th>
<th>Magnitude (M)</th>
<th>Peak Grnd. Accel. (PGA g)</th>
<th>Predominant Period, $T_p$ (sec)</th>
<th>Station</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lytel Creek (1970)</td>
<td>5.33</td>
<td>0.070</td>
<td>0.130</td>
<td>Cedar Springs, Allen Ranch</td>
</tr>
<tr>
<td>San Francisco (1957)</td>
<td>5.28</td>
<td>0.095</td>
<td>0.260</td>
<td>Golden Gate Park</td>
</tr>
<tr>
<td>Mammoth Lake (1980)</td>
<td>4.73</td>
<td>0.031</td>
<td>0.925</td>
<td>USC Cash</td>
</tr>
</tbody>
</table>

5 RESULTS AND DISCUSSION

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).

The response spectra at ground surface were plotted to investigate the effect of changing the depth of the RSM layer on top of the RSM layer. The numerical simulations were performed for the baseline model (Pure sand) and for another three models in which a 1 m thick layer of RSM were placed at depths of 2m, 4m, and 6m. The three different input ground motions were applied to each model. The replacement soil on top of the RSM layer was modeled as a well compacted sand-gravel mixture layer with a unit weight of 20 kN/m$^2$. The modulus reduction and damping curves for the replacement soil are shown in Figure 1. The thickness of the replacement soil is the same as the depth of the RSM layer from ground surface ($Y$).
Because the predominant period of the ground motion is less than the site natural period, amplification may occur at the fundamental or at secondary order periods of the site. The amplification factor between the spectral acceleration (Sa) at bedrock layer and at the surface layer has a maximum value at the fundamental period of the site. This value is reduced strongly in the successive secondary order periods.

Placing a layer of RSM resulted in increasing the site natural period causing shifting in the fundamental and secondary site periods to higher periods. Shifting of the site periods leads to maximum amplification at higher – more damped periods. This resulted in damping of spectral accelerations at lower periods and amplification of spectral accelerations at higher periods when using RSM compared to the baseline case (pure sand).

Increasing the depth of the rubber-sand mixture layer resulted in increasing the shifting towards the higher periods that leads to a higher order matching between site periods and ground motion predominant period (T_p). Thus, increasing the depth of the rubber-sand mixture layer resulted in highly damped response spectra of the surface layer at lower periods and amplification at higher periods (Figure 5). Increasing the RSM depth to 4m resulted in reduction in the maximum spectral acceleration reaching up to 56% to 45% in case of Lytel Creek and San Francisco earthquakes, respectively, compared to the baseline case (Pure sand). Placing the RSM layer at 6m depth resulted in a small reduction in the maximum spectral acceleration equals to 16 % compared to the baseline case (Pure sand), and this percentage may be slightly increased if RSM layer is put in deeper level.

**B. Thickness of the rubber-sand mixture layer (h).**

In this section, the numerical simulations were performed for the baseline model (pure sand) and for another three different models including RSM layers of varying thickness. Different values for the thickness of the RSM layer (h) were chosen to be 2m, 4m and 6m in the models, while the depth of the top of the RSM layer was constant at (Y=2 m) below ground surface in all simulations. The three different input ground motions were applied to each model. In case of Mammoth Lake earthquake, an additional simulation was performed with an RSM layer thickness of 9m.

The result of simulations was plotted in terms of the response spectrum of the ground surface layer to investigate its response to the change in the thickness of the RSM layer. Figure (6) shows the simulation results for the three different earthquake ground motions. Similar to the previous section, the results can be divided into two groups as follow:

**Group (1) - (T_p < T_{site}).**

Increasing the thickness of the RSM layer caused decreasing of low period spectral acceleration and also caused considerable increasing in the high period spectral acceleration comparing by the baseline case (balancing). This is evident from the amplification factors plotted in Figure (7) against different RSM thicknesses for different periods. Comparing to the baseline case (pure sand), at thickness of RSM layer equals to 6m a reduction in the maximum spectral acceleration in cases of Lytel Creek and San Francisco earthquakes ranged from 47% to 36%, respectively.
Increasing the thickness of the RSM layer from 1m to 4m caused increasing in the maximum spectral acceleration compared to the baseline case. Increasing the thickness of the RSM layer from 4m to 6m resulted in a reduction in the maximum spectral acceleration decreased but still higher than the maximum spectral acceleration in the base line case. Increasing the RSM thickness to 9m resulted in reduction in the maximum spectral acceleration to 38% compared to the baseline case. However, further increase in the thickness up to 9m resulted in a reduction in the amplification factor below the baseline case.

6 CONCLUSIONS

The following main points may be concluded based on the analyses presented herein:

- The effect of using RSM layer is dependant on the site natural period and the frequency content of the ground motion, while the effective configuration of the RSM layer is subject to the natural period of the intended structure.
- Placing a layer of RSM resulted in increasing the site natural period and damping of spectral accelerations at low periods and amplification of spectral accelerations at higher periods compared to the baseline case.
- The deeper the RSM layer, the larger the shift in site natural period resulting in more effective damping and lower response spectrum at ground surface for a wider range of periods. Thus, the higher the natural period of the structure, the deeper the sand/rubber layer needed to achieve damping.
- For the same excavation depth, using a thin layer of RSM at the bottom of the excavation is more effective in damping the spectral accelerations at ground surface than using a thick layer of RSM.
- Settlements and creep in RSM layer should be studied in case of large thickness.
- Further investigation is needed to confirm the observation through physical and numerical modeling for earthquakes of different magnitude, amplitude, and frequency content.
- Soil structure interaction needs to be further investigated to examine the effect of the overlaying structure on the response.

7 REFERENCES


New Replacement Formations on Expansive Soils Using Recycled EPS Beads

Remplacement sur les sols expansifs en utilisant des perles EPS

Abdelrahman G.E.
Civil Engineering Department, Faculty of Engineering, Fayoum University, Egypt

Mohamed H.K.
Soil Mechanics and Geotechnical Engineering Institute, Housing and Building National Research Center (HBRC), Egypt

Ahmed H.M.
Civil Engineering Department, Faculty of Engineering Cairo University, Egypt

ABSTRACT: One of the main problems encountered in constructing foundations on clays is volume change independent of loading caused by swelling of the soil. When the swelling is obstructed, large swelling pressures arise and that can cause damage to structures. This study examines the role of recycled expanded polystyrene (EPS) beads which is mixing with replaced soil in accommodating soil expansion and hence reducing swelling pressures on structures foundaation. Laboratory tests are presented on the formation of expansive soil using Bentonite clay. Laboratory model was used to measure the decrease of the swelling, using replacement material which formed of blending sandy soil with recycled (Expanded Poly-Styrene) EPS-beads. The effect of different compositions and different ratios between EPS-beads, and sand as a replacement soil on the expansive soil (Bentonite powder, PI = 95.4%, and Gs = 2.55) which had free swell equal to 96.7% were studied. Results so far show that the EPS beads mixed with sand significantly reduces the volumetric change of the expansive soils. The parametric study showed that increasing EPS beads percentage in the replacement soil decreases bearing capacity and dry density γd and increases OMC while for the Bentonite free swell decreases and settlement increases. Increasing footing breadth increases swelling and settlement. With increasing replacement layer thickness and beads density, the swelling and settlement decrease.

RÉSUMÉ : Un des principaux problèmes rencontrés dans la construction des fondations sur des argiles est le changement de volume indépendant de chargement provoqué par le gonflement du sol. Lorsque le gonflement est obstrué, les grandes pressions de gonflement surviennent et peuvent causer des dommages aux structures. Cette étude examine le rôle du polystyrène expansé recyclé (EPS) des perles qui est le mélange avec le sol remplacé en accusant l'expansion du sol et donc la réduction des pressions sur le gonflement de fondation des structures. Des essais en laboratoire sont présentés sur la formation des sols expansifs avec de l'argile bentonite. Modèle de laboratoire a été utilisé pour mesurer la diminution de l'enflure, l'utilisation du matériel de remplacement qui a formé de l'assemblage avec des sols sablonneux recyclés (Expanded Poly-Styrene) EPS-perles. L'effet de différentes compositions et différents ratios entre les EPS-perles, et le sable du sol comme un remplacement sur le sol expansif (bentonite en poudre, PI = 95.4%, et GS = 2,55), qui avait sans égale gonfler à 96.7% ont été étudiés. Les résultats obtenus jusqu'à présent montrait que les perles EPS mélangés avec du sable réduisent considérablement le changement volumétrique des sols gonflants. L'étude paramétrique a montré que l'augmentation des EPS perles de pourcentage dans le sol de remplacement diminue la capacité portante et la densité sèche γd, et augmente OMC alors que pour la bentonite diminue la houle libres et augmente de règlement. L'augmentation de la largeur de fondation augmente l'enflure et de règlement. Avec une épaisseur de remplacement couche augmente et la densité des perles, l'enflure et la diminution de règlement.

KEYWORDS: Recycled expanded polystyrene, beads, expansive soils, swelling, sand, Bentonite.

1 INTRODUCTION

Problems related to expansive soils exist worldwide. Many buildings, light structures, highways, railways, airport slabs, water channels, pipelines, earth retaining walls, dams and bridges are damaged by expansive soils. One of the main problems encountered in constructing foundations on clays is volume change independent of loading caused by swelling and shrinkage of the soil. When the swelling is obstructed, large swelling pressures arise and that can cause damage to structures. There are many conventional treatments available for control of these problems. These include soil replacement with compaction control, moisture control, surcharge loading and thermal methods (Chen, 1988; Nelson and Miller, 1992). However, these methods have their own limitations with regards to their effectiveness and costs.

Expanded polystyrene (EPS) is a cellular polymeric material commonly used as a packaging medium for a variety of consumer appliances and electronic equipment. It is a lightweight material with a very low density (0.10 -0.20 kN/m³). Due to its convenience and low cost, EPS usage is increasing in the consumer market. That in turn results in a continuing increase in the availability of waste EPS products. Because of their lightweight and bulk nature, the waste EPS products occupy a substantial area of the landfill. Unlike other organic materials, EPS is not decomposable or biodegradable. Because of these problems, the European Union has restricted the disposal of EPS into landfills and set recycling targets (PPW Directive, 2005; UNEP, 2000). These impositions have forced manufacturers to look for alternative reuse and recycle options. There are many recycling options available like thermal and compression methods. However, possible contamination of the products while in transportation and their limited usage make some of the products unsuitable for recycling. Hence there is a need to try other innovative applications for the bulk utilisation of waste EPS.

Since its inception, EPS composite soil has attracted the interests of many researchers. A few papers have been published regarding using EPS composite soil in reducing swelling pressures on structures foundation and behind retaining walls. Illuri & Nataatmadja (2007) and Illuri (2007) investigated the use of recycled EPS as a partial soil replacement and swell modifier for expansive soils. Artificially prepared expansive soils were manufactured in the laboratory by mixing fine sand with sodium bentonite of various proportions. Recycled EPS beads were mixed with these soils
and the effects of varying the amount were investigated. The proposed soil improvement technique is thus showing great promise in sustainable construction. Nataatmadja and Illuri (2009) prepared an artificially reconstituted soils of different plasticity values by mixing fine sand and sodium bentonite. It has been found that the addition of EPS granules into these soils results in light-weight backfill materials, suitable for reducing swelling pressure behind domestic retaining walls. The current research was conducted to investigate the recyclability of EPS packaging products in reducing swelling pressures on structures foundation by using recycled EPS beads as a mechanical admixer in replaced soils at their optimum-moisture contents. Mixing recycled EPS beads with soil replacement is introduced an environment-friendly geometrics. The applications of recycled EPS as a swell shrink modifier as well as desiccation controller of expansive soils were considered in this study. The quantitative evaluation also whether recycled EPS beads provides significant benefits for use in soil replacement to reduce swelling pressures was done through an extensive experimental program.

2 MATERIALS

2.1 Replacement Soil

The replacement soil was sub-angular silica sand and classified according to the unified soil classification (USCS) system as a poorly-graded clean medium to fine sand (SP) with coefficient of curvature (C_C)= 1.73, coefficient of uniformity (C_u)=3.6 , max dry density (γ_dmax)=19.2kN/m³ and optimum moisture content (OMC)= 9%.

2.2 Expansive Soil

The expansive soil was a sodium Bentonite. The physical properties for the used Bentonite are summarized in Table 1.

Table 1. Physical properties of the used Bentonite

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit (LL)%</td>
<td>143</td>
</tr>
<tr>
<td>Plastic Limit (PL)%</td>
<td>47.6</td>
</tr>
<tr>
<td>Plasticity Index (PI)%</td>
<td>95.4</td>
</tr>
<tr>
<td>Free Swell (FS)%</td>
<td>96.7</td>
</tr>
<tr>
<td>Specific Gravity (Gs)</td>
<td>2.55</td>
</tr>
<tr>
<td>Max Dry Density (γ_dmax) kN/m³</td>
<td>14</td>
</tr>
<tr>
<td>Optimum Moisture Content (OMC)%</td>
<td>24</td>
</tr>
</tbody>
</table>

2.3 Recycled EPS Beads

For the present study, waste EPS beads were collected with three different beads densities and particle sizes. Photo 1 shows the beads's size compared to sand particles. The beads densities and particle sizes are summarized in Table 2.

Table 2. Properties of EPS Beads

<table>
<thead>
<tr>
<th>Property</th>
<th>400</th>
<th>500</th>
<th>600</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (K/m³)</td>
<td>0.10</td>
<td>0.16</td>
<td>0.20</td>
</tr>
<tr>
<td>Particle Size (mm)</td>
<td>5-6</td>
<td>4-5</td>
<td>1-3</td>
</tr>
</tbody>
</table>

3 COMPACATION CHARACTERISTICS OF SAND EPS MIX

To study the compaction characteristics, standard Proctor compaction tests were performed on a number of sand EPS mixes. With the addition of EPS beads, the density of the resulting composite is much lower than the original soils. The EPS beads were added to the moist soil at a certain percentage of the soil’s dry mass. Compaction tests of the sand with EPS (SWEPS) composite were subsequently carried out immediately after mixing the sand and EPS. Compaction curves for mixes of sand with different percentages of EPS beads are shown in Figure 1. From this figure, it can be observed that with the addition of EPS beads the dry density of the resulting mix varies considerably, it decrease with increasing the beads content but there is no significant variation in the optimum moisture content. This can be attributed to the low bulk density and very low moisture absorbency of the EPS beads. Since the beads are bulk in volume but very low in mass, the mass of the soil-EPS composite is generally controlled by the mass of the soil in the mix. Furthermore, as the moisture is held within the soil particles, the optimum moisture content of the mix is controlled by the optimum moisture content of the sand. From previous stud it is found that the increase of EPS beads density increases the maximum dry density at the same beads ratio, Abdelrahman, (2009)

![Figure 1. Compaction curves for mixes with sand and EPS contents at density of t EPS beads = 0.16kN/m³](image)

4 EXPERIMENTAL WORK

4.1 Test Model

Experimental model consists of cylindrical soil sample container with diameter is 15 cm and height = 18cm, two vertical dial gages to measure the settlement and swelling, circular footing with different diameters. Vertical stress equal to 30 kN/m² was applied on the footing which represented the applied stress of three stories building.

4.2 Test Program

A series of tests were performed on circular footing with different diameters rested on sand EPS mix replacement layer with different ratios of EPS beads and layer thickness above Bentonite layer both sand EPS mix replacement layer and Bentonite layer were compacted at their optimum moisture content (OMC). Mixing EPS beads with sand replacement layer leads to settlement under loading condition before adding water to the swelling layer cause EPS beads are compressible material and this explain why swelling and settlement are discussed together in the test results. The studied parameters are summarized in Table 3.

Table 3. The studied parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of footing (d) cm</td>
<td>5, 7, 10, and 12</td>
</tr>
<tr>
<td>Beads Density (γ_b) kN/m³</td>
<td>0.1, 0.16 and 0.2</td>
</tr>
<tr>
<td>Beads Content (B) %</td>
<td>0, 0.3, 0.6, 0.9, and 1.2</td>
</tr>
<tr>
<td>Normalized Replacement Thickness (t/a) %</td>
<td>0, 12.5, 20, 25 and 33</td>
</tr>
</tbody>
</table>

4.3 Test Results and Analysis

4.3.1 Test results presentation

Swelling and settlement on surface soil and circular footing are presented in a set of curves with the different studied parameters.
4.3.2 Effect of footing diameter \( (d) \) cm
Increasing rigid footing breadth causes increasing in settlement in soil. In case of swelling soil increasing footing breadth also causes increasing in swelling deformation even with existing of beads. But EPS beads may leads to decrease the percent of the swelling and the settlement deformation as shown in Figure 2.

4.3.3 Effect of beads density \( (\gamma_B) \) kN/m\(^3\)
EPS density appears to be the main parameter that correlates with most of its mechanical properties. Compression strength, shear strength, tension strength, flexural strength, stiffness, creep behavior and other mechanical properties depend on the density. Higher density leads to improve its effect. As shown in Figure 3 increase beads density \( (\gamma_B) \) kN/m\(^3\) leads to decrease each of the swelling and settlement on surface soil and circular footing.

4.3.4 Effect of beads content \( (B) \) %
The EPS beads are highly compressible, with soft elastic nature, having only about 1% of the density of a typical soil.
As shown in Figure 4 increase beads content \( (B) \) % leads to decrease swelling pressure on the footing and decreases also the swelling settlement. EPS beads particles swelling energy absorption because of its compressible nature.

4.3.5 Effect of normalized replacement thickness \( (t_r/t_s) \) %
Replacement layer thinness \( t_r \) was chosen as a percentage of soil layer thickness \( t_s \). As shown in Figure 5 increasing the replaced layer improve the resistance to the swelling pressure and decrease the swelling deformation. But economic point of view also is important taking in to account the swelling layer thickness. Increasing the normalized replacement thickness \( (t_r/t_s) \) % leads to decrease each of the swelling and settlement on surface soil and circular footing.
These parameters included the effect of footing diameter (d) cm where increasing rigid footing breadth caused increasing in settlement in soil but the presence of EPS beads lead to decrease the percent of the swelling and the settlement deformation. The second parameter was the effect of beads density (γ_B) kN/m³ where increase beads density lead to decrease each of the swelling and settlement on surface soil and circular footing. The third parameter the effect of beads content in the sand replacement layer (B)% which was the most important parameter in this study where increasing this content lead to significant decrease in swelling. The fourth parameter was the effect of normalized replacement thickness (t_r/t_s) % where increasing the replaced layer improved the resistance to the swelling pressure and decrease the swelling deformation. The innovative application of the recycled EPS beads mixed with sand replacement layer at optimum moisture content, so as to make a beneficial use of the waste EPS products, will offer a sustainable solution for both the housing and EPS industries.

5 CONCLUSIONS

The results of a study on the potential use of sand –EPS mix as soil replacement layer to reduce swelling of expansive soils below structure foundation have been presented. Different parameters affecting the swelling of structure foundation have been studied.
ABSTRACT: This paper presents an overview of the different research studies performed in geotechnical engineering related to sustainable development. The philosophies of sustainability as applicable in geotechnical engineering are discussed. A review of the research and case studies performed in geotechnical engineering and how they can impact sustainable development is presented with particular emphasis on foundation engineering and ground improvement.

RÉSUMÉ : Cet article présente une vue d'ensemble des différentes recherches effectuées en géotechnique liée au développement durable. Les philosophies de la durabilité comme applicable en géotechnique sont discutées. Un examen des études de recherche et de cas réalisées en géotechnique et comment ils peuvent influer sur le développement durable est présenté avec un accent particulier sur les travaux de fondation et de l'amélioration du sol.

KEYWORDS: sustainability, waste recycling, life cycle assessment, multicriteria analysis, risk, resilience, carbon footprint.

1 INTRODUCTION

Civil engineering processes are both resource and fuel intensive. According to Dixit et al. (2010), the construction industry accounts for about 40% of the global energy consumption and depletes about two fifth of the sand, gravel and stone reserves every year. Construction activities also add to the problems of climate change, ozone depletion, desertification, deforestation, soil erosion, and land, water and air pollution (Kibert 2008). A geotechnical construction project not only has the above detrimental effects on earth’s resources and environment but also changes the land use pattern that persists for centuries and affects the social and ethical values of a community. Thus, geotechnical projects interfere with many social, environmental and economic issues, and improving the sustainability of geotechnical processes is extremely important in achieving overall sustainable development.

This paper attempts to connect the broader scope of sustainable development with geotechnical engineering and presents a review of the research done on different aspects of sustainability in geotechnical engineering with particular emphasis on foundation engineering and ground improvement.

2 SUSTAINABILITY AND GEOTECHNOLOGY

Sustainability of a system is its ability to survive and retain its functionality over time. For an engineered system to be sustainable, it should be efficient, reliable, resilient, and adaptive. Efficiency requires that the resource use, cost and environmental impacts of the engineering system are minimal. Reliability ensures that the system is sufficiently far away from its predictable failure states. A resilient system has the ability to return to its original functioning state within an acceptable period of time when subjected to unpredictable disruptions. An adaptive system is responsive to gradual and natural changes within itself and in its environment, and is flexible to modifications and alterations required to cope with such changes. Together, these characteristics help in deciding whether an engineered system is capable of surviving in a complex and evolving socio-economic environment without losing its own character and function, and without violating the limits of the carrying capacity of the natural systems. Thus, the objective of sustainable engineering is to ensure the integration of an engineered system into the natural and man-made environment without compromising the functionality of either the engineered system or that of the ecosystem and society, and this harmony between the natural and built environments must be maintained at the local, regional and global scales. Therefore, in the engineering domain, sustainability can be looked upon as a dynamic equilibrium between four E’s — engineering design, economy, environment and equity, as described in Figure 1.

Figure 1. The four E’s of sustainability in engineering projects.

In view of the four E’s approach of sustainable engineering, the sustainability objectives that may be incorporated in geotechnical projects are: (i) involving all the stakeholders at the planning stage of the project so that a consensus is reached on the sustainability goals of the project (such as reduction in pollution, use of environment friendly alternative materials, etc.), (ii) reliable and resilient design and construction that involves minimal financial burden and inconvenience to all the stakeholders, (iii) minimal use of resources and energy in planning, design, construction and maintenance of geotechnical facilities, (iv) use of materials and methods that cause minimal negative impact on the ecology and environment, and (v) as
much reuse of existing geotechnical facilities as possible to minimize waste. This approach aims at reaching a dynamic equilibrium between engineering integrity, economic efficiency, environmental effectiveness, and social acceptability and equity.

In an endeavor to incorporate sustainability in geotechnical design, three new trends have been identified (Iai 2011): (i) geo-structures are now designed for performance rather than for ease of construction, (ii) designs are now more responsive to site specific requirements, and (iii) the designs consider soil-structure interaction rather than just analysis of structural or foundation parts.

3 SUMMARY OF SUSTAINABLE GEOTECHNOLOGY RESEARCH

Several research studies have been performed that aim at making geotechnical engineering practice sustainable. The areas in which research has progressed include (1) the use of alternate, environment friendly materials in geotechnical constructions, and reuse of waste materials, (2) innovative and energy efficient ground improvement techniques, (3) bio-slope engineering, (4) efficient use of geosynthetics, (5) sustainable foundation engineering that includes retrofitting and reuse of foundations, and foundations for energy extraction, (6) use of underground space for beneficial purposes including storage of energy, (7) mining of shallow and deep geothermal energy, (8) preservation of geodiversity, and (9) incorporation of geothecics in practice.

Geohazards mitigation is another important aspect of sustainable geotechnical engineering — related studies include studies on the effects of global climate change and of multi-hazards on geo-structures. In this context, it is important to note that sustainable geotechnical engineering should not only focus on minimization of ecological footprints but also on making geo-structures reliable and resilient so that the effects of hazards, both natural and man-made, can be minimized. The aspect of reliability and resilience is particularly important for critical infrastructures (e.g., lifeline systems like transportation and power supply network without which other systems like cities cannot function) of which geo-structures like dams, embankments, slopes and bridge foundations are important components.

The recent research studies on geosustainability are mostly based on the common notions of sustainability like recycling, reuse and use of alternate materials, technologies and resources. However, other such new approaches are actually sustainable or not cannot be ascertained without proper assessment using, for example, whole life cost analysis and risk based performance analysis. Therefore, a complete sustainability assessment framework is necessary for geotechnical projects to ascertain the relative merits of different options available for a project.

Any geosustainability assessment framework should have a life cycle view of geotechnical processes and products and should (i) ensure societal sustainability by promoting resource budgeting and restricting the shift of the environmental burden of a particular phase to areas downstream of that phase, (ii) ensure financial health of the stakeholders, and (iii) enforce sound engineering design. As the uncertainties associated with geotechnical systems are often much greater than those with other engineered systems, sustainability framework for geotechnical engineering should include an assessment of the reliability and resilience of the geo-system, and offer flexibility to the user to identify site specific needs.

From the environmental impact point of view, quantitative environmental metrics like global warming potential (Storesund et al. 2008), carbon footprint (Spaulding et al. 2008), embodied carbon dioxide (Egan et al. 2010), embodied energy (Chau et al. 2006) and a combination of embodied energy and emissions (carbon dioxide, methane, nitrous oxide, sulphur oxides and nitrogen oxides) (Iinui et al. 2011) have been used to compare competing alternatives in geotechnical engineering. But, assessing the sustainability of a project based solely on metrics like embodied carbon dioxide or global warming potential involves ad hoc assumptions, puts excess emphasis on the environmental aspects and fails to consider a holistic view that must also involve technical, economic and social aspects (Holt et al. 2010, Steedman 2011).

Among the sustainability assessment tools that address the multidimensional character of sustainability, some are qualitative and represent the performance of a project on different sustainability related sectors pictorially (e.g., GeoSpeAR) (Holt 2011). The second category of multidimensional assessment frameworks consist of quantitative and life cycle based tools. Life cycle costing (LCC), life cycle assessment (LCA), multicriteria analysis and combinations of LCC and LCA have been used for purpose. These frameworks and metrics like Green Airport Pavement Index, BE2ST-in-Highways and Environmental Sustainability Index fall under this category (Pittenger 2011, Lee et al. 2010b, Torres and Gama 2006).

The third approach to sustainability assessment is based on point based rating systems that provide a measure of sustainability of projects based on points scored in the different relevant categories. Rating systems like GreenLites (McVoy et al. 2010), I-LAST (Knuth and Fortman 2010), Greenroads (Muench and Anderson 2009), MTO–Green Pavement Rating System (Chan and Tighe 2010) and Environmental Geotechnics Indicators (Jefferson et al. 2007) fall under this category.

4 SUSTAINABLE GROUND IMPROVEMENT

A major part of the sustainability related research in geotechnical engineering has focused on ground improvement through the introduction of novel, environment friendly materials with particular emphasis on the use of waste materials. Puppala et al. (2009) proposed the use of alternate materials for soil stabilization including the use of recycled materials in geotechnical constructions. Other examples include the use of recycled glass-crushed rock blends for pavement sub-base and recycling of shredded scrap tires as a light-weight fill material.

Reuse of old pavements including asphalt and concrete pavements has been on the rise (Gnanendran and Woodburn, 2003). The old pavements are recycled into full and partial depth reclamation bases with cement or other additive treatment. Sometimes these pavements are recycled into aggregate materials which are termed as reclaimed asphalt pavement (RAP) materials. RAP materials have been used as bases with chemical stabilization, and several state DOT agencies in the USA has been using them in the new pavement construction projects. Puppala et al. (2009) performed a series of resilient modulus tests on cement and cement-fiber treated RAP for use as pavement base material. They reported that the structural coefficients increase with an increase in the confining pressure and these values are higher for cement and cement-fiber treated aggregates. The significant increase of structural coefficients with cement-fiber treatment (30%) was attributed to the tensile strength and interlocking properties offered by the fiber content.

Investments made on transportation and processing is reduced when native material after stabilization is used as a base or backfill material. This saves money that might otherwise be spent on fuels for transportation. The old pavement material if cannot be reused has to be landfilled, which increases the costs associated with the landfilling practices. Therefore, the use of old pavement materials as stabilized bases reduces the space used for landfills, which, in turn, reduces the overall carbon footprint of the project by not using aggregates from quarries.
The Integrated Pipeline (IPL) project which involves a long pipe line installation is a joint effort between the Tarrant Regional Water District (TRWD) and Dallas Water Utilities (DWU) that is aimed at bringing additional water supplies to the Dallas/Fort Worth metropolis. As a part of the pipeline layout and construction, large amounts of soil need to be excavated during the pipeline installation. Also, large amounts of material need to be imported for bedding and backfilling of the trenches. Both importing new fill material and exporting excavated trench material for landfilling will have serious implications on the economic and environmental aspects of the construction project.

As a result, a research study was initiated at the University of Texas at Arlington to identify chemical treatment of in-situ soil material that can be reused as either bedding, zone or backfill materials for the pipeline installation. Based on the comprehensive laboratory studies, the soils along the pipeline alignment are identified for potential reuse as backfill, bedding and zone materials after chemical amendment, and more details can be found in Chittoori et al. (2012). The cost and environmental benefits as well as emissions reductions of using in-situ native material versus imported fill materials are also explained.

5 SUSTAINABLE FOUNDATION ENGINEERING

Foundations form an integral part of geotechnical construction, and sustainable design and construction of foundations are very important for overall sustainable development. Sustainable foundation engineering entails robust analysis and design, economical and environment friendly construction that cause minimal disruptions to life and damage to adjacent properties, reuse and retrofitting of existing foundations as much as possible, and use of foundations in harvesting geothermal energy.

Robust design of foundations essentially involves a rigorous analysis (e.g., use of proper constitutive equations and analytical or numerical modeling of appropriate boundary value problems) and choice and execution of an appropriate design methodology (e.g., identification of all possible limit states and moving the design state sufficiently away from the limit states by either using a reliability based method or by applying load and resistance factor design (LRFD) methodology). The recent trend in geotechnical engineering to incorporate LRFD is encouraging and several research studies have been conducted to rigorously develop resistance factors based on reliability analysis (e.g., Basu and Salgado 2012). Further, the incorporation of random fields to characterize spatial heterogeneity of soil in the probabilistic analysis of foundations and related soil structure interaction problems significantly contributes to sustainable foundation engineering (Haldar and Basu 2011, 2012).

Misra and Basu (2011, 2012) recently developed a multicriteria based sustainability assessment framework for pile foundation projects. The framework considers a life-cycle view of the pile construction process (Figure 2), and combines resource consumption, environmental impact and socio-economic benefits of a pile-foundation project over its entire life span to develop a sustainability index (Figure 3). The use of resources is taken into account based on the embodied energy of the materials used, the impact of the process emissions is assessed using environmental impact assessment and the socio-economic impact of the project is assessed through a cost benefit analysis. Three indicators are derived from the three aspects and are combined through weights to calculate the sustainability index (SI) for the different alternatives available for the project (Figure 3).

Figure 2. Flow chart showing the inputs, outputs, processes and impact categories in pile construction.

Figure 3. Multicriteria based sustainability assessment framework.

Reuse and retrofitting of foundations is a traditional practice for almost all refurbishment projects, but recently the concept has been extended for redevelopment projects as well (Butcher et al. 2006a). Reuse of foundations is an attractive option because the cost of removal of an old foundation is about four times that of construction of a new pile, disturbance to adjacent structures caused by foundation removal can be avoided, and backfilling of voids created by the removed foundation is not required. At the same time, the embodied energy consumed in reusing foundations is nearly half of that consumed in installing new foundations. Consequently, several case studies demonstrating the benefits of reuse of foundations have been documented (Anderson et al. 2006, Butcher et al. 2006b).

Foundation engineering has a prominent role in the alternative energy sectors like geothermal and wind energy. Case studies show that deep foundations can be used as energy storage and transmitting elements (Quick et al. 2005) while concrete surfaces in contact with the ground (e.g., basement walls) can act as heat exchangers (Brandl 2006). Research is in progress to develop proper characterization, analysis and design of energy related geo-structures like energy piles (Laloui 2011), wind turbine foundations (Doherty et al. 2010) and foundations for oil and gas drilling operations (Yu et al. 2011).

6 CONCLUSIONS

In recent times, a concerted effort is noted within the civil engineering industry in delivering built facilities that are eco-friendly and sustainable. Geotechnical construction, being resource intensive and by virtue of its early position in civil engineering projects, has a great potential to influence the sustainability of such projects. Incorporating sustainability in geotechnical engineering requires an understanding of the ideological conflicts that characterize sustainability and of the approaches that can make engineering processes sustainable. Philosophically, engineering sustainability can be looked upon as the balance between engineering design, economy, social equity and the environment (4 E’s).

Sustainability related research studies in geotechnology essentially belong to two categories: those that contribute to global sustainability through the use of alternative materials and innovative engineering and those that develop sustainability assessment frameworks. A summary of these research studies is provided with emphasis on two particular areas, ground...
improvement and foundation engineering. The focus of these studies is mostly on the environmental and economic aspects. It is recommended that a more holistic approach considering environmental, social, economic, reliability and resilience aspects (the 4 E’s) should be developed for sustainable geotechnical practices.

7 REFERENCES


ABSTRACT: Powder wastes like fly ash are produced in large volumes. They have handling, disposal problems and poor engineering performance due to their silt size. Manufacturing artificial sand and gravel from these silt sized powder wastes in large quantities will solve the associated problems of having silt size. Disc pelletizers with manufacturing capacities reaching one million ton a year makes this process economically feasible and practical for geotechnical applications. Fly ash is one of these powder wastes having silt size and easily available in many countries where they create huge disposal problems. Cold bonding pelletization technique is used to produce fly ash pellets of sand and gravel size and their mechanical properties are determined. The manufactured pellets are lightweight materials with adequate strength and can be used in many geotechnical projects. The fly ash pellets show similar behavior to that of calcareous sands. In addition to utilization of pellets as manufactured, it is also possible to manufacture soil to the desired specification by adding additives or apply surface treatment.

RÉSUMÉ : Déchets de poudre comme les cendres volantes sont produits en grandes quantités. Ils ont la manipulation, et l’élimination des problèmes de performance d’ingénierie pauvres en raison de leur taille limon. Fabrication de sable et de gravier artificielle à partir de déchets de limon ces poudres de taille en grandes quantités permettra de résoudre les problèmes associés ayant une taille de limon. Granulateurs à disques avec des capacités de production pour atteindre un million de tonnes par an rend ce processus économiquement faisable et pratique pour des applications géotechniques. Les cendres volantes sont un de ces déchets en poudre ayant une taille de limon et facilement disponible dans de nombreux pays où ils créent de grandes problèmes d’élimination. Technique de granulation à froid de liaison est utilisé pour produire des boulettes de cendres volantes de sable et de gravier taille et leurs propriétés mécaniques sont déterminées. Les pellets sont fabriqués avec des matériaux légers résistance suffisante et peut être utilisé dans de nombreux projets en géotechnique. Les granulés de cendres volantes présentent un comportement similaire à celui des sables calcaires.

KEYWORDS: Powder wastes, cold bonding pelletisation, silt size, fly ash, calcereous sands, grain crushing.

1 INTRODUCTION

With increasing disposal costs, and growing ecological concerns, waste materials are utilized in geotechnical applications more and more each year. Due to its silt size, powder materials are hard to handle, transport, compact and dispose. Increasing the size of the powder wastes from silt size to sand and gravel size has a lot of benefits. Powder wastes like coal burning thermal power plant fly ash are used in many geotechnical applications. The pelletization cost for fly ash is around one to two Euros per ton and the capacity of one pelletizer can be as high as one million tons per year, making this approach a feasible and practical application in geotechnical engineering. Annual fly ash production for many countries is in the range of 1 to 100 million tons. This paper summarizes a series of research work about manufacturing sand and gravel from powder fly ash by cold bonding pelletization technique. The pelletization mechanism is explained and physical and engineering properties of the produced pellets are given.

The manufactured pellets behave like calcareous sands found in the nature. The source and shape difference of the natural calcareous sands do not exist in the manufactured pellets having nearly perfect sphericity and roundness. The crushing behavior of the manufactured soil is studied in detail. For potential applications like backfill for retaining walls, fill under the footings, pile installation in existing manufactured soil embankment, anchor installation in manufactured fills, the interface behavior and the influence of crushability on the interface behavior is also studied. Finally odometer tests, direct shear tests are conducted and the results are summarized.

1.1 Mechanism of pellet formation

Pelletization process is the agglomeration of moisturized fines in a rotating drum or disc. The product at the end of the process is called the “fresh pellet”. The crushing strength of the fresh pellet must be enough for hauling and stockpiling purposes. The pelletization technology is widely used in powder metallurgy engineering, and medicine industry.

The pelletization theory was developed in 1940’s. The performance of the pelletization process is a function of; i) the engineering properties of the material pelletized; ii) the amount of moisture in the medium; iii) the mechanical process parameters such as the angle of balling drum or disc to the normal and the revolution speed. Observations and analysis performed on these parameters with respect to mechanic and kinetic laws formed the theory of pelletization process (Baykal and Doven 2000).

When a fine grained material is moisturized, a thin liquid film forms on the surface of the grains, which forms meniscus between the grains. With the rotation in a balling drum or disc, they form ball shape structures with enhanced bonding forces between grains due to centrifugal and gravitational forces. The mechanism of pellet formation is presented in Figure 1. In the pendular state water is present only at point of contact of the grains. With more water addition some of the pores are filled with water in the funicular state. All intergranular space is filled.
with water in the capillary state. The most suitable state for pellet formation is the capillary state.

Figure 1. Mechanism of pellet formation; a) the pendular state; b) the funicular state; c) the capillary state.

The formation of capillary force between two grains is presented in Figure 2. The grain diameter of the powder material influences the magnitude of the surface tension force; small grain diameter is necessary to create enough pulling force to initiate agglomeration. Agglomeration can be achieved by drum or disc pelletizers. A typical disc pelletizer designed and manufactured for this study is presented in Figure 3 (Doven 1998).

Figure 2. Surface tension force created by water bridge between two particles.

Figure 3. The sketch of disc pelletizer (back view).

The revolution speed of the disc can be controlled between 0 and 70 rpm and the angle of the disc plane to the normal can be adjusted between 0 and 90 degrees. The diameter of the disc is 0.40 meters and scraping blades are placed from center to one edge at 0.06 m intervals. During the revolution of the disc the grains pulled by surface tension are compacted further. The agglomerated grains hit to the scraping blades, falling free to the bottom section of the disc. This free fall action compacts the agglomerated product more. This repeated revolving and free fall action densifies and makes the agglomerated product stronger for handling. The motion of the grains in the disc is presented in Figure 4. The forces applied to the grains during pellet formation are presented in Figure 5. To achieve the most suitable pelletization process; the revolution speed and the angle of disc plane to the normal should be set in a manner to avoid the dominancy of gravitational or centrifugal forces (Figure 6).

2 PHYSICAL AND ENGINEERING PROPERTIES OF THE MANUFACTURED PELLETS

Turkey produces more than 17 million tons of fly ash annually. The fly ash used in the presented studies is obtained from Soma Coal Burning Thermal Power Plant in the west part of Turkey. The typical chemical composition of Soma fly ash is given in Table 1. The physical properties of manufactured fly ash pellets are presented in Table 2. The water absorption of the produced pellets is high.
3 CRUSHING BEHAVIOR OF MANUFACTURED PELLETS

To demonstrate the effect of aggregate crushing sieve analyses were performed before and after direct shear testing of fly ash pellets at 50, 100 and 200 kPa normal stress. The change in grain size distributions before and after execution of the direct shear tests are given in Figure 8 (Danyildiz 2007).

The fly ash pellets crushing behavior is similar to calcareous sands. The measured crushing behavior does not pose a threat for the engineering performance of the fly ash pellets for most geotechnical applications.

4 SHEAR STRENGTH OF FLY ASH PELLETS

Direct shear tests are conducted on manufactured fly ash pellet aggregates under 50, 100 and 200 kPa normal stress applications. Interface tests are conducted on split samples of fly ash pellets and concrete. The internal friction angle and interface friction angle plots are presented in Figure 9.

Tables 1 through 3 show that fly ash pellets formed with cold bonding technique have similar engineering properties to that of soils. With no additional binder like lime or cement, self cementitious fly ash pellets have acceptable engineering properties. The soundness tests were conducted using sodium sulphate. Less than 12 percent weight loss after sodium sulphate treatment is allowable for concrete applications. The durability performance of the manufactured aggregates is adequate even for more demanding applications like concrete production.

From geotechnical point of view, the manufactured pellet aggregates have properties similar to those of granular soils except high water absorption value.

Table 1. The chemical composition of Soma Fly Ash.

<table>
<thead>
<tr>
<th>Element</th>
<th>Per cent</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>50.5</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>23.7</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>5.8</td>
</tr>
<tr>
<td>CaO</td>
<td>9.3</td>
</tr>
<tr>
<td>MgO</td>
<td>2.6</td>
</tr>
<tr>
<td>SO₃</td>
<td>1.4</td>
</tr>
<tr>
<td>Loss on Ignition</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Soma fly ash is self cementitious and it will harden without the need of another binder. The physical properties of the manufactured fly ash pellets are given in Table 2. The typical fly ash pellets are given in Figure 7.

Table 2. Physical properties of the fly ash pellets.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight</td>
<td>9.6 kN/m³</td>
</tr>
<tr>
<td>Water absorption</td>
<td>31.4 %</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.17</td>
</tr>
<tr>
<td>Bulk specific gravity</td>
<td>1.29</td>
</tr>
</tbody>
</table>

Figure 7. Manufactured fly ash pellets.

Table 3. Engineering properties of the fly ash pellets.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimum moisture content</td>
<td>34.4 %</td>
</tr>
<tr>
<td>Dry unit weight (Standard Proct)</td>
<td>11.96 kN/m³</td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>29.4°</td>
</tr>
<tr>
<td>California Bearing Ratio</td>
<td>58 %</td>
</tr>
<tr>
<td>Soundness loss of weight 9.5-4.75 mm</td>
<td>9.0 %</td>
</tr>
<tr>
<td>Soundness loss of weight 19-9.5 mm</td>
<td>7.9 %</td>
</tr>
</tbody>
</table>

Tables 1 through 3 show that fly ash pellets formed with cold bonding technique have similar engineering properties to that of soils. With no additional binder like lime or cement, self cementitious fly ash pellets have acceptable engineering properties. The soundness tests were conducted using sodium sulphate. Less than 12 percent weight loss after sodium sulphate treatment is allowable for concrete applications. The durability performance of the manufactured aggregates is adequate even for more demanding applications like concrete production.

From geotechnical point of view, the manufactured pellet aggregates have properties similar to those of granular soils except high water absorption value.

Figure 8. The grain size distribution of fly ash pellets before and after the conduction of direct shear test at a normal stress of 50, 100 and 200 kPa.

The fly ash pellets crushing behavior is similar to calcareous sands. The measured crushing behavior does not pose a threat for the engineering performance of the fly ash pellets for most geotechnical applications.

Figure 9. Internal and interface friction angles of fly ash pellets and pellet concrete interface.
The size of the silt sized powder wastes can be increased to sand and gravel size by pelletization technique in large volumes and low cost making the technique suitable for geotechnical applications. In this study experience with self cementing fly ash is presented, however the technique is applicable to other silt sized powder wastes provided that adequate capillary forces develop between the grains. For non self cementing fly ash, binders like hydrated lime or cement can be used for manufacturing. For higher performance needs the crushing strength of the fly ash pellets can be improved by using lime or cement additives. Surface treatment of pellets is possible using water glass at reasonable cost. High water absorption values (30 -35 per cent) place the manufactured granular material in an unique place in classical soil classification. The durability of the aggregates is also satisfactory. With low unit weigh, free draining behavior, and ease of compaction, the manufactured soil has good potential for large volume utilization in geotechnical applications. The geotechnical properties of the manufactured soil ensures high stability. In addition to its potential for utilization in large volumes, the manufactured soil is a great tool for experimental research on crushable soils. It is possible to control the size, shape, surface texture, roundness, sphericity, crushing strength, unit weight, water absorption properties of the powder materials to produce soil with target engineering properties. This way by fixing one parameter at a time it will be possible to study the effects of each parameter on the engineering behaviour of natural soils. The cold bonding pelletization technology is a low tech technology which requires minimum capital investment and low operational costs. The whole process can be automated. If the manufactured aggregates are not used in a geotechnical application, a major reduction in disposal costs is achieved by improving handling, transportation, compaction and disposal in a dump site. Free draining property, high stability and potential for reuse when needed are other benefits of utilization.

ACKNOWLEDGEMENTS

The studies presented above are conducted by PhD student Ata Gurhan Doven; MSc students; Haydar Arslan, Egemen Danyildiz and Murat Cenk Erdurak with financial support provided by Bogazici University Scientific Research Fund and Turkish Scientific Research Council (TUBİTAK).

REFERENCES


Erdurak M.C.2011. Artificial sand production for laboratory tests. MSc Thesis Bogazici University, Istanbul


Méthodes non traditionnelles de traitement des sols : apports techniques et impact sur le bilan environnemental d’un ouvrage en terre.

Soil treatment with non traditional additives in earthworks: evaluation of the technical and environmental improvements.

Blanc G.
Agence de l’environnement et de la Maîtrise de l’Énergie, Angers, France
LEMTA (CNRS, UMR 7563), Université de Lorraine, France.

Cuisinier O., Masroufi F.
LEMTA (CNRS, UMR 7563), Université de Lorraine, France.

RÉSUMÉ : Dans le contexte actuel de fort développement des problématiques environnementales, la mise au point de techniques permettant de valoriser au mieux les matériaux de terrassement tout en limitant l’impact environnemental des chantiers est devenue un enjeu majeur. L’une des solutions innovantes proposées est d’utiliser des sous-produits industriels organiques en traitement des sols. Ainsi, l’objectif de cette étude est d’évaluer les apports de trois produits non traditionnels (un produit acide, un produit enzymatique et un lignosulfonate) sur le compactage et la portance d’un limon et d’en évaluer les effets sur le bilan environnemental d’un ouvrage en terre. Dans un premier temps, les résultats expérimentaux ont montré que les traitements enzymatique et au lignosulfonate permettaient d’augmenter les masses volumiques sèches atteintes et d’économiser l’eau lors de la mise en œuvre du limon à condition que sa teneur en eau initiale soit faible. Dans un second temps, la comparaison des impacts environnementaux des deux traitements effectuée suivant la méthode d’analyse du cycle de vie, a permis d’identifier la variante ayant l’impact environnemental le plus restreint.

ABSTRACT: Sustainable development principles lead earthworks companies to use all natural materials on the construction site and to reduce the environmental impact of their activities. In this context, the use of industrial organic products has been proposed. The aim of this study is to characterize the modification of compaction and bearing capacity of a silt when treated with three non-traditional additives (an acid solution, an enzymatic solution and a lignosulfonate). Observed modifications of the compaction properties showed interesting applications for the compaction of the silt when its natural water content is low. For the enzymatic and lignosulfonate treatments, savings of water could be expected during the construction stage. For these two treatments, a comparison of the global environmental impact was made thanks to a life cycle assessment study.

MOTS-CLÉS : Terrassement, traitements non traditionnels, compactage, analyse du cycle de vie, impact environnemental.

KEYWORDS: Earthworks, nontraditional treatments, compaction, life cycle assessment, environmental impact.

1 INTRODUCTION


Au-delà des aspects uniquement techniques, il est également nécessaire d’aborder les traitements sous l’angle de leur bilan environnemental pour y intégrer plus amplement les aspects du développement durable. L’une des approches possibles est fondée sur les principes de l’analyse du cycle de vie (AFNOR 2006). La seconde phase de cette étude consiste à calculer et à comparer les bilans environnementaux des différentes variantes de traitement sur l’ensemble des phases du cycle de vie de la construction d’un ouvrage en terre. En effet, l’utilisation des produits non traditionnels se heurte à un verrou technique...
supplémentaire du fait de l’absence d’études ayant porté sur le bilan environnemental des opérations de traitement des sols.

2 MATÉRIEL ET MÉTHODES

Trois traitements non traditionnels ont été testés sur un limon. Suivant leur nature, une procédure spécifique est suivie.

2.1 Produits de traitement

Le produit classé dans la famille des produits acides est une solution aqueuse d’acide sulfurique contenant du limonène sulfonaté, sous-produit de l’industrie de la transformation des agrumes. Les résultats expérimentaux obtenus au laboratoire et ceux issus de la littérature (par ex. Tingle et Santoni 2003, Rauch et al. 2003) n’ayant pas montré de sensibilité des résultats mécaniques par rapport au dosage, un dosage classique de 0,01 % est utilisé.

Le produit enzymatique est une solution aqueuse organique dérivée de la transformation des mélasses, un sous-produit de l’industrie sucrière. Comme les propriétés mécaniques des sols traités ne semblent pas affectées par des modifications du dosage (Tingle et Santoni 2003, Velasquez et al. 2006), un dosage courant de 0,002 % est utilisé pour ce traitement.

Le troisième produit est une poudre de lignosulfonate de calcium, polymères organiques dérivant des lignines, sous-produit des industries papetières. Certaines études ont montré que les valeurs de résistance à la compression simple sont maximales pour un dosage de 5 % (Tingle et Santoni 2003, Santoni et al. 2002). Dans cette étude, les dosages massiques de 0,5 ; 2,0 et 5,0 % sont donc testés.

2.2 Caractéristiques du limon étudié

Le sol testé est un limon fin (87 % de passant à 80 µm), peu plastique (wL = 34 % ; IP = 14) fréquemment rencontré lors de travaux de terrassement en France, en région parisienne.

2.3 Procédure de traitement

Pour les traitements au produit acide et à la solution enzymatique, la première étape de préparation consiste à humidifier le sol avec de l’eau distillée pour atteindre une teneur en eau de 3 % inférieure à la teneur en eau finale souhaitée. Le mélangé est effectué à l’aide d’un malaxeur à couteaux puis est laissé reposé 24 heures en sacs hermétiquement fermés. Le produit de traitement est alors dilué dans la quantité d’eau requise pour atteindre la teneur en eau souhaitée puis ajouté progressivement au sol lors de l’opération de malaxage. Pour le traitement au lignosulfonate, le produit est directement ajouté au sol préalablement humidifié. Indépendamment du traitement, un temps de cure d’une heure est respecté avant compactage.

3 RÉSULTATS EXPÉRIMENTAUX

Les résultats expérimentaux présentés portent sur les caractéristiques de compactage et de portance du limon traité.

3.1 Propriétés de compactage

Les essais de compactage ont été menés conformément à la norme NF P 94-093 dans des moulés CBR. Pour chaque teneur en eau, un essai de poignonnement selon NF P 94-078 permet de déterminer la valeur de l’Indice Portant Immédiat (IPI) du sol. Pour le traitement au lignosulfonate, seul le dosage de 2,0 % est représenté sur la figure 1. Il s’agit du dosage pour lequel l’effet sur la courbe de compactage est maximal.

La figure 1 montre que le traitement à 0,01 % de produit acide n’entraîne pas de changement significatif. En revanche, après ajout de 0,002 % de solution enzymatique et de 2,0 % de lignosulfonate, l’optimum Proctor est atteint pour des teneurs en eau optimales (wopt) plus faibles et des masses volumiques sèches maximales (ρmax) plus élevées. Ainsi, pour le traitement enzymatique, ρmax est de 1,86 Mg/m³ au lieu de 1,82 Mg/m³ et wopt de 14,5 % au lieu de 15,5 %. Du côté sec de l’optimum, les traitements au produit enzymatique et au lignosulfonate contribuent à augmenter les masses volumiques sèches dans une gamme de teneurs en eau allant de 8 à 15 %.

Figure 1. Courbes de compactage et de portance du limon traité avec trois produits non traditionnels.

3.2 Exemple d’application au traitement d’un sol sec

Les résultats expérimentaux montrent que certains traitements affectent la courbe de compactage en déplaçant l’optimum vers le côté sec et en augmentant les densités sèches obtenues. Cet effet peut trouver des applications intéressantes lors de la mise en œuvre de sols dont la teneur en eau initiale est située du côté sec de l’optimum. Ainsi, en supposant que la teneur en eau initiale du sol soit de 9,0 % (état très sec) et un objectif de compactage de 1,78 Mg/m³, trois variétés de mise en œuvre peuvent être considérées pour un compactage à l’énergie Proctor normale (Figure 2) :

- le sol non traité est humidifié jusqu’à une teneur en eau de 14,0 % puis compacté,
- le sol est traité à 0,002 % de produit enzymatique, la teneur en eau de compactage requise est alors de 11,5 %,
- le sol est traité au lignosulfonate qui est épandu sous forme de poudre, puis mélangé au sol. Ensuite, la teneur en eau du sol est augmentée pour atteindre la valeur de 11,5 % puis le sol est compacté.

Figure 2. Comparaison des variantes de mise en œuvre du limon considérant une humidification puis un compactage à l’énergie Proctor normale.
Dans le cas du traitement au produit enzymatique et à 2,0 % de lignosulfonate, le compactage peut avoir lieu à une teneur en eau de 11,5 % au lieu de 14,0 % ce qui représente une économie d’eau de 44,5 m³ pour 1000 m³ de sol compacté.

Les résultats expérimentaux ont montré qu’il était possible, grâce aux traitements, d’atteindre un objectif de compactage donné pour une teneur en eau moindre ce qui permet de réaliser une économie d’eau. Toutefois, les étapes de production des substances utilisées, leur transport et leur mise en œuvre génèrent des impacts environnementaux qu’il est nécessaire d’évaluer sur l’ensemble des étapes du cycle de vie de l’ouvrage. L’objectif principal de la partie suivante est ainsi de définir dans quelle mesure les variantes traitées induisent une réduction de l’impact environnemental global par rapport à la mise en œuvre du sol non traité.

4 ANALYSE EN CYCLE DE VIE D’UN REMBLAI TRAITÉ

La démarche appliquée est celle définie dans les normes régissant l’analyse du cycle de vie. Le principe consiste à quantifier les intrants pour un système donné puis à calculer l’impact environnemental associé grâce aux données d’Inventaire du Cycle de Vie de ces intrants (iCV). L’impact environnemental est alors évalué à l’aide d’une méthode de calcul de l’impact. Au cours de cette étude, la méthode utilisée par la norme NF P 01-010 (AFNOR, 2004) est appliquée.

4.1 Définition du système

Le système étudié est un remblai dont la masse volumique sèche visée est de 1,78 Mg/m³. La teneur en eau initiale du sol est supposée être de 9,0 % ce qui permet de se situer dans un contexte où les traitements présentent les meilleurs avantages (Figure 2). L’IPI minimal requis est fixé à 10 pour assurer une bonne traficabilité des engins de chantier. L’unité fonctionnelle (Figure 2) est définie comme le volume compacté de 1000 m³.

Définir le système revient à différencier les processus qui sont pris en compte dans l’étude et ceux qui en sont exclus. Au cours de cette étude, une démarche comparative a été adoptée ce qui a permis de réaliser un certain nombre de simplifications en ne considérant que les étapes qui diffèrent entre les variantes. Par exemple, l’ensemble des étapes préparatoires au chantier, les étapes d’extraction ou encore de transport du sol n’ont pas été prises en compte dans le calcul des impacts environnementaux car ces étapes sont identiques pour toutes les variantes.

4.2 Calcul des intrants

Les intrants considérés sont l’eau, les produits de traitement et les carburants. Les quantités des autres intrants (volume de sol par exemple) sont identiques pour toutes les variantes ce qui permet de les retirer du système.

La quantité d’eau requise est directement calculée à partir de la différence entre la teneur en eau initiale et finale. L’ICV de la production de l’eau dépend principalement de son origine. Par exemple, l’eau peut être prélevée sur des réseaux d’eau potable, dans des cours d’eau, ou encore être pompée dans un forage. Il est également possible d’anticiper les besoins en eau du chantier en créant des bassins pour y stocker les eaux de pluie. Compte tenu de la diversité des approvisionnements possibles et du manque de données statistiques relatives aux prélevements d’eau sur chantiers, l’impact environnemental associé à l’étape de prélevement de l’eau ne sera pas pris en compte et devra être discuté dans une étude de sensibilité.


L’ICV du lignosulfonate est issu d’une étude réalisée par Modahl et Vold (2011) à partir de données obtenues pour une usine située à Sarpsborg en Norvège.


Pour chacune des variantes, les intrants calculés sont résumés dans le tableau 1. Les résultats mettent par exemple en évidence que les variantes traitées consomment moins d’eau par rapport à la variante non traitée (44 500 L au lieu de 89 000 L). Pour le traitement au produit enzymatique, la consommation du pulvimiréxer est deux fois moindre en comparaison avec la variante non traitée car une seule passe suffit pour effectuer le traitement contrairement à la variante traitée où l’humidification doit être réalisée en deux passées. Le traitement au lignosulfonate nécessite quant à lui l’apport d’une masse de 35 600 kg de lignosulfonate dont le transport représente une consommation de carburant estimée à 379 L.

Tableau 1. Comparaison des intrants du système pour les trois variantes étudiées.

<table>
<thead>
<tr>
<th>Intrant</th>
<th>Non traité</th>
<th>0,002 % Produit enzymatique</th>
<th>2,0 % Lignosulf.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eau (L)</td>
<td>89 000</td>
<td>44 500</td>
<td>44 500</td>
</tr>
<tr>
<td>Produit (kg)</td>
<td>-</td>
<td>35 600</td>
<td>-</td>
</tr>
<tr>
<td>Camion</td>
<td>-</td>
<td>2 379</td>
<td>2 9</td>
</tr>
<tr>
<td>Arroseuse</td>
<td>5 2</td>
<td>2 9</td>
<td>-</td>
</tr>
<tr>
<td>Pulvimiréxer</td>
<td>456</td>
<td>228</td>
<td>456</td>
</tr>
<tr>
<td>Compacteur</td>
<td>30</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Épandeur</td>
<td>-</td>
<td>-</td>
<td>2,9</td>
</tr>
<tr>
<td>Fuel lourd (kg)</td>
<td>-</td>
<td>0,7</td>
<td>-</td>
</tr>
</tbody>
</table>

4.3 Calcul des impacts environnementaux

Pour le traitement au produit enzymatique (Figure 3), le calcul des indicateurs des 10 catégories d’impact proposés dans la norme NF P 01-010 montre que la variante traitée présente des impacts réduits dans 7 catégories sur 10 (consommation de ressources énergétiques, épuisement de ressources naturelles, consommation d’eau, changement climatique, formation d’ozone atmosphérique, pollution de l’air, pollution de l’eau). La consommation d’énergie est par exemple réduite de plus de 40 % et passe de 18,8 10^6 MJ à 10,7 10^6 MJ si le traitement au produit enzymatique est mis en œuvre. Au contraire, pour trois catégories, l’impact est augmenté, la production de déchets et la destruction de l’ozone stratosphérique en tête, suivie de l’acidification atmosphérique. Au-delà des valeurs calculées, il est nécessaire de se poser la question du caractère significatif des écarts observés. En effet, pour la production de déchets par...
exemple, la variante traitée présente un impact 4,5 fois plus élevé, cependant, l'augmentation de la production de déchet ne représente en valeur que 2,8 kg pour 1000 m3 de sol compacté ce qui correspond à un volume très faible en comparaison aux 5200 kg/personne/an produite en moyenne en Europe (Eurostat 2008).

5 CONCLUSION

L'étude présentée a l'originalité d'aborder des méthodes non traditionnelles de traitement à la fois sous l'angle des aspects géotechniques et sous celui des aspects environnementaux grâce à une étude d'analyse du cycle de vie. Les essais de compactage et de portance ont permis d'identifier des variantes permettant de faciliter la mise en œuvre du limon et de réaliser des économies d'eau. Pour ces variantes, l'analyse du cycle de vie a montré que le traitement au produit enzymatique induisait une réduction de l'impact environnemental dans l’essentiel des catégories. Dans la situation étudiée, le traitement permet ainsi de combiner intérêt technique et environnemental. Au contraire, le traitement au lignosulfonate génère une forte augmentation de l’impact environnemental ce qui limite l’intérêt du traitement. Afin de confirmer la robustesse de l'étude environnementale, celle-ci devra être complétée par une étude de sensibilité portant sur les hypothèses et données d’entrées du système.

6 REMERCIEMENTS

Cette étude est financée par l’Agence de l’Environnement et de la Maîtrise de l’Énergie (ADEME) sous forme d’une thèse en partenariat avec Égis Géotechnique et DTP Terrassement. Les auteurs tiennent à remercier l’ensemble des personnes impliquées dans le comité de pilotage de la thèse pour leur participation et leur soutien.

7 REFERENCES


ABSTRACT: A piled raft solution was proposed as an alternative to a conventional fully piled foundation for a new shopping development in Cambridge, UK. This paper demonstrates how the use of precedent knowledge, appropriately targeted investigation and modelling can provide cost effective and resource efficient foundation solutions.

KEYWORDS: Gault Clay, piled raft foundation, nonlinear stiffness, site characterisation.

1 INTRODUCTION.

1.1 Piled rafts and settlement reducing piles


In essence, where the soil underlying a structure is sufficiently stiff it is often the case that the use of a plain or piled raft solution will lead to economies when compared to the costs associated with a fully piled foundation system.

Effectively, load from the superstructure is first distributed through a plain raft to the subsoil and if the analysis predicts settlement in excess of that deemed to be acceptable, settlement reducing piles can be introduced at strategic points in order to stiffen the support to the raft and bring the expected settlements down to an acceptable level.

However, this solution is not often examined due to the sophisticated nature of the soil-structure interaction that needs to be analysed – the work involved in delivering the solution is too ‘complicated’ and the structural engineer prefers the ease, risk transfer and robustness of a fully piled option.

1.2 Gault Clay and geological setting

The Gault Clay is an over-consolidated clay that was laid down towards the end of the Lower Cretaceous period (c. 100 Ma), and its engineering behaviour lies between that of a soil and weak rock (Marsh & Greenwood 1995). In Cambridge, UK the Upper Gault sub-division predominates.

In the Cambridge area, the strata over-lying the Gault, most significantly the Chalk of the Upper Cretaceous, have been largely removed by erosion, and the area was also subjected to a number of glaciations.

The removal of an estimated 150 m of overburden (Samuels 1975), in addition to the ice cover, has subjected the clay to significant stress relief with associated intense fissuring and softening and in addition, the stratum has experienced moderate levels of tectonic activity.

In engineering terms, the Upper Gault is a high plasticity clay (LL ~ 70 - 80%; PI ~ 45 - 55%) with moisture contents close to its Plastic Limit and a significant calcite content (30 – 40%).

At the site, the soil profile (Table 1), especially the distribution of the superficial soils and the upper surface of the Gault Clay, has been modified by historic construction that had reduced the original ground level.

Groundwater was found to be perched within the superficial soils at a level between 0.5 m and 1 m above the surface of the clay (noting that the surface of the clay undulated significantly), and water levels in the Lower Greensand were found to be in hydraulic continuity (Nash et al. 1996).

<table>
<thead>
<tr>
<th>Top of layer: m OD</th>
<th>Thickness: m</th>
<th>Soil description</th>
</tr>
</thead>
<tbody>
<tr>
<td>+5.8 to +11</td>
<td>0.3 to 3.3</td>
<td>Made Ground</td>
</tr>
<tr>
<td>0.3 to 1.7</td>
<td>Bricearth</td>
<td></td>
</tr>
<tr>
<td>0.1 to 2.4</td>
<td>Sand &amp; Gravel</td>
<td></td>
</tr>
<tr>
<td>+3.2 to +8.5</td>
<td>0 to 3.6</td>
<td>Gault Clay (weathered)</td>
</tr>
<tr>
<td>&gt;34.7</td>
<td>Gault Clay</td>
<td></td>
</tr>
<tr>
<td>-30 approx.</td>
<td>Not proven</td>
<td>Lower Greensand</td>
</tr>
</tbody>
</table>

Table 1. Site specific soil profile

1.3 Development at adjacent sites, Grand Arcade and foundation options

Located in central Cambridge (Fig. 1), the Grand Arcade site is immediately adjacent to the Lion Yard shopping centre and the Crown Plaza Hotel (Lings et al. 1991, Ng & Nash 1995). The development covers a total area of about 1.4 hectares.

The excellent field and laboratory research work undertaken in relation to the deep basement excavation at the latter site, informed the decisions made during the development of the ground model for this project.

In addition to these relatively modern buildings, other structures of significance to the project were the Post Office (PO) and Telephone Exchange (BT) building(s) in the northeast corner of the site, and retained facades along the eastern boundary, facing St. Andrews Street; all of which had to be protected from damage during the works.
The re-development at the site involved the demolition of a number of existing buildings and because of the long history of occupation in the area, archaeological investigation was undertaken prior to the main construction works starting.

To form the basement substructure, zero sheet-pile and secant bored pile perimeter retaining walls were installed where needed, and ground levels were reduced generally by between 4 m and 7 m across the site.

The gross weight of the new structure was equivalent to a uniform loading of about 120 kPa which with an average unloading due to demolition and excavation of 80 kPa (ranging between 30 and 100 kPa), equates to a net loading of 40 kPa. However, this varied somewhat across the site and as a result in some areas, e.g. under core structures and some columns, net contact pressures locally were as high as 220 kPa.

During tendering, a value engineering exercise was undertaken and the option of replacing the conforming fully piled foundation with a piled raft was investigated, and this preliminary assessment suggested that significant savings were possible using this alternative.

2 SOIL CHARACTERISATION

2.1 Site specific investigation and testing

In order to develop the ground model for the proposed raft foundation it was necessary to supplement the site investigation and laboratory testing previously completed with further, high quality sampling and testing. Therefore, three additional 30 m deep boreholes were completed using rotary coring techniques and sub-samples were taken from the cores for stress path testing.

Prior to the stress path testing, the suction in each sample was measured using the IC suction probe, in order to be able to understand whether the samples may have been unduly disturbed when recovered or when in transit.

These measurements proved to be very interesting (Fig. 2) and while generally consistent they were significantly lower than what would be expected based on experience in for example the London Clay Formation, and imply much lower values of “at-rest” earth pressure coefficient, K₀ than might be expected based on a simple one-dimensional depositional and erosion environment.

It is thought that the post-depositional processes alluded to earlier and especially the lateral changes resulting from historic tectonics may have led to lateral stress relief and generally lower K₀ values in the Gault Clay. There is also the possibility that the clay simply cannot maintain suctions high enough to reliably represent the in situ stress conditions, although this is considered less likely - similar results in terms of suction measurement from large diameter (300 mm) samples of Gault Clay recovered from a site near Cambridge were seen by the Authors.

Stress path testing was undertaken on six samples to examine the changes in soil stiffness, during stress paths representative of the expected unloading during demolition and excavation, and reloading as the raft is constructed and loaded.

2.2 Ground response at adjacent site

When developing the ground model for design of the piled raft, it was recognised that next door there was effectively a full scale element test available in the form of the instrumented 10 m deep basement excavation created for the Crown Plaza Hotel (Fig. 3).

The instrumented basement construction provides very useful information in terms of the vertical ground movements associated with unloading during excavation and in the long-term (Nash et al. 1996, Lings et al. 1991). And while acknowledging that the piled foundations will have influenced the observations to a degree, the data (Fig. 4) was able to be used to make an independent assessment of the non-linear stiffness of the soil mass with depth and strain level. In particular the excavation to Level 2 was of interest as this represented the level of excavation in the new development.
During the basement excavation, block samples of the clay were recovered for laboratory characterization of the soil. These studies examined the strength and deformation characteristics of the clay (Ng & Nash, 1995), Table 2 and its stiffness anisotropy (Pennington et al. 1997, Lings et al. 200). As with the field observations, this data informed the development of the ground model for this project as described in the following.

Table 2. Typical Gault Clay geotechnical parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural moisture content: %</td>
<td>25 - 32</td>
</tr>
<tr>
<td>Liquid Limit, LL: %</td>
<td>70 - 80</td>
</tr>
<tr>
<td>Plasticity Index, PI: %</td>
<td>40 - 50</td>
</tr>
<tr>
<td>Liquidity Index, LI: %</td>
<td>0 ± 0.1</td>
</tr>
<tr>
<td>Undrained shear strength, c_u: kPa</td>
<td>75 ± 5z (1)</td>
</tr>
<tr>
<td>Critical State angle of resistance, ( \phi'_{CS} ): deg</td>
<td>24° - 28°</td>
</tr>
<tr>
<td>Peak apparent cohesion, c'PK: kPa</td>
<td>2 - 3</td>
</tr>
<tr>
<td>Peak angle of shearing resistance, ( \phi'_{PK} ): deg</td>
<td>32° - 34°</td>
</tr>
<tr>
<td>Initial shear modulus, G0: MPa</td>
<td>80 - 120</td>
</tr>
<tr>
<td>Young’s modulus at 0.2% strain, E_u,0.2%: MPa</td>
<td>c. 10</td>
</tr>
</tbody>
</table>

(1) z = depth below top of clay

2.3 Integrated ground model

In order to undertake the geotechnical calculations for the design of the piled raft, a ground model in terms of vertical stiffness, E was developed based on back-analysis of the observed heave response in the adjacent basement, Fig. 4 & 5 and stress path testing undertaken on high quality core samples from the site, Fig. 6. Data from the latter are summarised in Table 3.

The stiffness data from these two sources are compared in Fig. 6 – the comparison is remarkably good given the quite different sources and gave confidence in the use of the ground model for the design of the foundation system.

Table 3. Summary of stress path testing

<table>
<thead>
<tr>
<th>Sample</th>
<th>1R-C09</th>
<th>1R-C17</th>
<th>2R-C05</th>
<th>2R-C17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth: m</td>
<td>11.2</td>
<td>25.6</td>
<td>10.3</td>
<td>19.6</td>
</tr>
<tr>
<td>G_max: MPa</td>
<td>37.5</td>
<td>180</td>
<td>45.5</td>
<td>104</td>
</tr>
<tr>
<td>E_u,0.01/c_u</td>
<td>1000</td>
<td>2200</td>
<td>1400</td>
<td>1200</td>
</tr>
<tr>
<td>E_u,0.1/c_u</td>
<td>475</td>
<td>860</td>
<td>300</td>
<td>560</td>
</tr>
<tr>
<td>E_u,0.5/c_u</td>
<td>190</td>
<td>220</td>
<td>100</td>
<td>280</td>
</tr>
</tbody>
</table>

The stiffness profile chosen for the design calculations is shown in right-hand side of Fig. 5 where it is compared to the ‘average’ line from the field data (dashed line in both sides of Fig. 5) and various values of constant E_u/c_u ratio ranging from 250 to 1500.

In the preliminary design at tender stage, anticipating strains in the region of 0.2% to 0.3%, a uniform modulus ratio E_u/c_u = 300 was used. It is clear from the figures below that this assumption quickly becomes unrepresentative of the response of the clay at depth and therefore any calculations are likely to overstate the movements that might be expected.

In the design calculations, use of the linear model yielded settlement predictions about one-third larger than that of the pseudo-nonlinear model. Thus peak settlements were reduced from 90 mm to 60 mm, and the area where settlements were considered excessive (i.e. greater than 40 mm), was greatly reduced.

3 FOUNDATION DESIGN

3.1 Calculation method

A number of simplified methods have been proposed for the evaluation of the load-settlement response of piled rafts but often these are difficult to apply to cases where the raft shape and/or load patterns are complex – as was the case here.

For this project, a plate-on-springs type structural analysis was undertaken in parallel with geotechnical analysis of ground movement using the pseudo-nonlinear elastic ground model described above. The analyses were iterated to achieve comparable movement predictions, the latter calculation providing subgrade stiffness values for use in the former that
were consistent with the expected load changes and associated ground response across the raft footprint.

When including settlement reducing piles (SRP) in the calculation, rather than modelling them as a spring, it was assumed that they acted as a constant load. This was deemed acceptable on the basis that they would be settling 25 mm to 40 mm, at which point the shaft resistance would be fully mobilised.

Calculations were undertaken for three stages of loading / soil response:

1. **Undrained excavation** using the undrained stiffness profile in Fig; 5; this analysis suggested heave up to 30 mm might occur.

2. **Semi-drained net loading** using a stiffness profile based on \( E' = 0.75E_u \) which was considered to be a reasonable estimate for the situation at the end of construction when it was estimated that 40±10% consolidation might have been achieved.

3. **Drained net loading including uplift due to groundwater** using a stiffness profile based on \( E' = 0.50E_u \) which was assessed using elastic theory and the degree of anisotropy in stiffness suggested by Pennington et al. (1997), i.e. \( C_{30/h}\!/C_{0(hv)} \approx 2 \).

### 3.2 Settlement control and role of SRP

In the design, an overall factor-of-safety with respect to bearing capacity failure in excess of three was demonstrated for the raft. However, mitigation measures were required in order to:

- Reduce excessive contact pressures and limit raft settlements to less than 40 mm, as a plain raft was expected to settle 40–60 mm with local maxima of 60–90 mm under heavily loaded columns and building cores.

- Limit raft settlements along sensitive boundaries to less than 25 mm; in the absence of SRP where a similar range of settlement values as indicated above were expected.

- Minimise net load changes adjacent to the diaphragm wall on the hotel boundary which was achieved by introducing SRP to limit the contact pressures on this boundary to approximately the same values present prior to the redevelopment.

- Minimise movements of bearing piles supporting the hotel which with the measures introduced were estimated to be less than 1.5 mm.

This was achieved by the introduction of SRP at the required locations. Their use in the first instance is well documented however it is thought that this is the first such application in terms of mitigating potential impacts outside the site boundary.

### 4 CONCLUSIONS

Use of precedent knowledge of the ground’s response to load change has allowed a calculation model to be derived that is better conditioned to predict ground movements in Gault Clay.

In this case, the use of a piled raft, though more complex to design, provided a clear cost and time advantage to a fully piled solution. Furthermore, the use of a well-conditioned pseudo-nonlinear elastic soil model allowed further savings by reducing the number of SRPs needed to achieve the performance criteria.

SRP have been employed in a novel way in order to limit vertical ground movements off-site by constraining those within the site boundary, and thus protect neighbouring buildings from potentially damaging movement.

### 5 ACKNOWLEDGEMENTS

The authors would like to thank Bovis Lend Lease and the Grand Arcades Partnership for allowing us to report on our contribution to this project, and Dr. David Nash of Bristol University with whom the first author spent a very interesting and productive afternoon discussing the Gault Clay.

The work presented here was undertaken while the authors worked at Card Geotechnics Ltd., UK.

### 6 REFERENCES


The Use of Recycled Aggregates in Unbound Road Pavements

L'utilisation d'Agrégats Recyclés en Revêtements de Chaussée sans Liant

Cameron D.A., Rahman H.H., Azam A.M.
University of South Australia (UniSA), School of Natural and Built Environments, Adelaide, Australia

Gabr A.G.
Mansoura University, Public Works Department, Faculty of Engineering, Egypt

Andrews R.
ARRB Group (South Australia), Adelaide, Australia

Michell P.W.
Aurecon Australia Pty Ltd, Adelaide, Australia

ABSTRACT: This paper argues for the acceptability for use in unbound granular pavements of recycled concrete aggregates (RCA) and recycled clay masonry (RCM) derived from demolition. Specifications from road authorities both within and outside Australia are considered, and results of tests carried out on specimens of RCA and RCM are compared with these specifications. The tests included conventional classification tests for soils and aggregates, Los Angeles abrasion value, Micro-Deval, falling head permeability, drying shrinkage, undrained triaxial tests and repeated loading triaxial testing for resilient modulus and permanent strain rate. The influence of matric suction on resilient modulus of the granular pavement materials is presented. Both RCA and RCM blended with RCM (20% by mass maximum) were found to meet existing specifications and therefore can be incorporated in road pavements. RCA was found to be suitable for use as a base course. In the case of RCA blended with RCM, as the proportion of RCM increases, the permanent strain rate increases and resilient modulus decreases, thereby compromising use of the blends as base material. However, RCA with up to 20% RCM is suitable for use as sub-base of a road pavement.

RÉSUMÉ : Ce document plaide pour l'acceptabilité, pour les revêtements de chaussée granulaires sans liant, des agrégats de bétons recyclés (ABR) et de la maçonnerie recyclée d'argile (MRA) provenant de la démolition. Les spécifications des autorités routières d'Australie et d'ailleurs sont considérées, et des résultats d'essais effectués sur des spécimens de ABR et de MRA sont comparés à ces spécifications. Les essais comprennent des essais conventionnels de classification pour sols et agrégats, valeur d'abrasion Los Angeles, Micro-Deval, perméabilité à charge variable, séchage et rétraction, essais triaxiaux non drainé et essais triaxiaux répétés pour le module résilient et la vitesse de déformation constante. L'influence de la succion matricielle sur le module résilient des matériaux granulaires de revêtement de chaussée est présentée. L'ABR et l'ABR mélangé avec le MRA (20% maximum en masse) se sont avérés satisfaire les spécifications existantes et peuvent donc être incorporés en revêtement de chaussée. L'ABR n'est pas approprié pour l'usage comme couche de base. Dans le cas de l'ABR mélangé avec le MRA, à mesure que la proportion de MRA augmente, le taux de déformation permanente augmente et le module résilient diminue, compromettant de ce fait l'utilisation des mélanges en tant que matériau de couche de base. Cependant, l'ABR avec jusqu'à 20% RCM convient pour l'usage comme sous-couche de chaussée routière.

KEYWORDS: recycled aggregate, C&D waste, resilient modulus, permanent strain, matric suction, prediction

1 INTRODUCTION

Recycling of construction and demolition wastes can produce acceptable aggregates for civil engineering applications such as unbound granular pavements. Australian practice is well behind countries such as Japan, the United Kingdom, France, Germany and the Netherlands, but the recycling aggregate industry, which has emerged over the last decade, is growing. Quarry industries seem to feel challenged by recycling but should realise that even in Europe with its long history of recycling, recycled aggregate supply is unlikely to exceed 10 to 15% of total demand (Meininger and Stokowski 2011). Much of the research in Australia to date has focussed on Recycled Concrete Aggregates (RCA), Recycled Clay Masonry (RCM), recycled glass and waste rock. Much can be learned from the European experience, but this experience cannot be simply adopted as it is based on the range of climates, pavement construction practices and geology throughout Europe. Furthermore, the great majority of pavements in Australia are thinly surfaced, resulting in higher stresses being applied to the aggregates by passes of traffic.

Much of the work to date has been limited to the laboratory (e.g. Nataatmadja and Tan 2001, Athershan et al. 2009, 2010, Arulrajah et al. 2011, 2012a, 2012b and 2012c, Gabr et al. 2012, Gabr and Cameron 2012a, Azam and Cameron 2012, Azam et al. 2012, Jitsangiam et al. 2009, Leek and Siripun 2010). Gabr 2012 developed an empirical model for predicting permanent strain from testing of South Australian RCA, which he incorporated into Finite Element Analysis (FEA) to predict pavement life, similar to the approach of Huvstig et al. 2008. However the validation of this approach has not been made. Nevertheless a few field trials of roads constructed with C&D waste have been conducted (Ecocycle 1997 and Bowman & Associates 2009a and 2009b). The combination of laboratory and field data with FEA has much potential to improve pavement design generally.

This paper summarizes the work undertaken at the University of South Australia (UniSA) to evaluate aggregate produced from two local producers of recycled C&D waste, which consisted of either crushed concrete or RCA blended with fired clay masonry (RCM). All products were nominally 20 mm sized maximum aggregate. Variations in moulding moisture levels have been investigated, leading to some interesting findings relating soil suction to resilient modulus.

The South Australian Department for Transport, Energy and Infrastructure (DPTI, formerly DTEI) stipulates a range of material properties (DPTI, 2011), but includes minimum resilient modulus and maximum rate of permanent strain for Class 1 bases, based on a simplified, single stress stage Repeated Load Triaxial Test (RLTT). These performance based specifications are unique in Australia if not worldwide. In parts of Scandinavia, specifications require back-calculated resilient
modulus from a falling weight deflectometer or Young's modulus from a plate bearing test (Gabr and Cameron 2011).

2 MATERIALS AND RANGE OF TESTS

Two RCA basecourse products, A and B, were tested, along with a comparable product (A20) of RCA with 20% by mass of RCM. Further materials were made at UniSA by blending product B with RCM to 10%, 20% and 30% (B10, B20 and B30). DPTI permit up to 20% by mass in RCA of "foreign material" consisting of clay brick tile, crushed rock and masonry for base course and subbase applications. Finally a virgin quartzite aggregate (Q) was evaluated, which is commonly used in Adelaide for construction of Class 1 bases.

The particle size distributions of the materials fell within DPTI specifications for Class 1 base. All the materials were well-graded gravel and sand mixtures with silty gravel; GW-GM according to the Unified Soil Classification System (USCS). Material A lay close to the coarse specification limit, while Material B20 crossed between the limits and had a fairly high proportion of fine sand-sized particles. The fines content of the two RCA products, A and B, were just 5% and 7% respectively, while the quartzite base material (Q) had 11%.

Tests were conducted in line with the requirements of current Australian specifications. These included plasticity of fines, aggregate strength tests, Los Angeles abrasion, CBR tests on 4 day soaked samples and RLTT to the DTEI 2008 protocol. In addition, falling head permeability tests and shrinkage tests were conducted. Some concern has been expressed relating to the propensity of RCA to exhibit some cementation upon wetting and compaction. This self-cementation of RCA materials can produce increase of strength with time, but also the possibility of reduced permeability (AASHO 2002) and shrinkage. Therefore shrinkage was investigated.

3 MATERIAL PREPARATION

All materials were compacted to a target Dry Density Ratio (DDR) of 98% of maximum dry density under Modified Proctor compactive effort. Static compaction, which is advocated by DTEI for unbound granular material, was used for compaction, largely because of the consistency of preparation. Moulding moisture variations are indicated for particular tests as follows. In South Australia, materials are commonly compacted at 80% of OMC and are allowed to dry back to 60% of OMC.

3.1 Falling Head Permeability

For the falling head permeability tests, the moulding moisture contents were 100 and 80% of OMC. Blended materials were tested; A20, B10, B20 and B30.

3.2 Drying Shrinkage

Samples were 200mm high by 100 mm diameter. Triplicate samples of materials A and B, and duplicate samples of A20 and B20, were prepared OMC. The target moisture content was reduced to 90% OMC if the material was found to be too fragile upon extrusion (e.g. samples B & B20). After compaction, the samples were extruded from the mould and sealed in plastic bags to cure for 7 days; the samples were then stored in a curing room (temperature 23±2°C and relative humidity 55±5%).

3.3 Undrained Triaxial and RLTT Testing

Duplicate samples were prepared. The resilient modulus and permanent deformation behaviour of RCA mixtures were investigated at different levels of moulding moisture contents, as was the undrained shear strength. Generally just one day of curing occurred before de-moulding and testing.

4 RLTT TEST METHODS

In Australia, there are two standard approaches to Repeated Load Triaxial Testing (RLTT); multi-stage stress testing and single-stage stress testing, e.g. the DTEI approach. DTEI 2008 specified application of a constant confining stress of 196 kPa and a vertical deviator stress of 460 kPa, pulsed over 50,000 loading cycles. AUSTROADS established a multi-stage stress RLTT under drained conditions to determine the permanent deformation and resilient modulus properties. Both these test protocols have been applied. In the RLTT program, deformations were measured with two pairs of inductance coils ("Emu" coils) mounted on the sample (Gabr et al. 2012).

5 INDEX VALUES AND OTHER PROPERTIES

The plasticity of fines of the various materials is indicated in table 1. The DPTI (2011) specifications call for a maximum Liquid Limit of 25% for Class 1 and 28% Class 2, and so A20 falls into Class 2, while all other materials would be acceptable for Class 1 applications.

Los Angeles Abrasion Value (LAA) and Micro Deval help to evaluate the abrasion resistance/toughness under traffic loading. The LAA values of the South Australian RCA examples ranged between 37% and 39%, which failed to meet the maximum of 30% proposed by DPTI. The Ile de France specifications (2003) for LAA allowed 35% for roads with the greatest traffic (GR4), increasing to 40% for GR3 and 45% for GR2. The values for RCM blends were in the range of 40 to 45%. The French Micro Deval limit of 30% for GR4 was met by both RCA products, as the Micro-Deval values were 30% and 28% for products A and B, respectively. There is however a further requirement that the sum of LAA and Micro Deval must not exceed 55 for GR4 and 65 for GR3. Product B was on the limit for GR3, while product A, exceeded it (69). GR3 corresponds to a road with daily annual traffic of 85.

Average shrinkage curves with time are provided in Figure 1 for the four materials that were tested. Interestingly, shrinkage strains were similar for the RCA products, as they were for the blends (20% RCM); however, the addition of crushed masonry resulted in an appreciable drop in shrinkage. In the case of product B, a reduction of almost 60% was observed.

The permeability of blended recycled material when prepared at OMC was approximately 2 x 10-8 m/sec for blends based on RCA product B, but it was observed that A20 was ten times more permeable. Compaction to the same density but at just 80% OMC increased the permeability of all materials generally by a factor of approximately three.

<table>
<thead>
<tr>
<th>Material</th>
<th>A</th>
<th>A20</th>
<th>B</th>
<th>B10</th>
<th>B20</th>
<th>B30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit (%)</td>
<td>26</td>
<td>27</td>
<td>23</td>
<td>24</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>Plastic Index (%)</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>3</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>LAA (%)</td>
<td>39</td>
<td>42</td>
<td>37</td>
<td>42</td>
<td>43</td>
<td>45</td>
</tr>
<tr>
<td>Micro Deval (%)</td>
<td>30</td>
<td>-</td>
<td>28</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

All materials met the specification of a minimum CBR of 80%, despite masonry content reducing the CBR significantly. Similarly the requirement of a maximum unconfined compression strength of 1 MPa after curing was met.

A study was undertaken of the matric suction-moisture content relationship, or soil water characteristic curve (SWCC) of the materials to enable estimation of matric suctions of RLTT samples from measured moisture contents. Initial matric suction was determined by the hanging column method for low suctions and the contact filter paper method (refer Azam and Cameron).
2012 for details of the filter paper method) for suctions greater than 20 kPa. The SWCC plots of gravimetric moisture content against matric suction are provided in Figure 2 for the 2 blends with 20% RCM. Air entry values ($u_{ae}$) were the same for these two samples but the residual suction ($u_r$) differed (15 and 30 kPa). The air entry values were within the range reported for RCA by Rahardjo et al. 2010.

The DPTI specification for Class 1 material requires a minimum rate of permanent strain over the last 30,000 load cycles of $-1 \times 10^{-8}$% per cycle. RCA performance was generally acceptable for Class 1 (refer Figure 4) although the blends with RCM were more appropriate for Class 2 applications. The quartzite material, Q, failed to make the DPTI Class 1 limit when prepared at 80% OMC or wetter.

### Table 2. Undrained shear strength (80% OMC)

<table>
<thead>
<tr>
<th>Material</th>
<th>A20</th>
<th>B10</th>
<th>B20</th>
<th>B30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion (kPa)</td>
<td>163</td>
<td>134</td>
<td>41</td>
<td>9</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>48</td>
<td>55</td>
<td>60</td>
<td>53</td>
</tr>
<tr>
<td>Nom. strength (kPa)</td>
<td>277</td>
<td>274</td>
<td>214</td>
<td>142</td>
</tr>
</tbody>
</table>

Undrained shear strength data are indicated in Table 2 for material prepared at a target of relative moisture content of 80% OMC. The last row contains nominal shear strength values based on the shear strength parameters and a normal stress of 100 kPa. The nominal shear strength decreased generally with masonry content for the B samples, but the A samples seemed unaffected.

Shear strength increased with matric suction or decrease of relative moisture content. Between target moisture contents of 90 and 60% OMC, on average the nominal shear strength of the recycled materials increased by 13%, while the quartzite strength was improved by 9%.

The resilient modulus of the RCA products (materials A and B) varied between 500 and 950 MPa, clearly surpassing the DPTI 2011 requirement of 300 MPa. Generally resilient modulus decreased with moisture content although product A had a fairly constant modulus of approximately 600 MPa. The materials blended with crushed masonry performed well also. Even material B30 had a minimum modulus greater than 400 MPa. A relationship between initial matric suction and resilient modulus from the single stress stage of the DTEI test protocol, was developed and is illustrated in Figure 3. A simple power function is consistent with the findings of Gupta et al. 2007.

Further work is underway on predicting the resilient modulus from multi-stage triaxial stress testing (stress dependent model).
generate stresses for application of the permanent strain model and therefore predict pavement life. The life of a pavement constructed with material B was predicted to improve 100 fold when the moulding moisture content was 60%, not 90% OMC.

8 CONCLUSION

From the evidence presented from RLTT data, the RCA products could be used as Class 1 base, while the blended products are more suited to Class 2 applications, or sub-base. Other specification systems depend on basic engineering properties, such as Los Angeles Abrasion and Liquid Limit may restrict the application of recycled materials to lesser applications. This paper has highlighted current research on recycled aggregates in South Australia, including the development of models for predicting both resilient modulus and permanent strain. Improvements to the models are being sought; for example matric suction should replace moisture content in the permanent strain model.

9 ACKNOWLEDGEMENTS

This research has been supported by industry partners ResourceCo and Adelaide Resource Recovery, as well as ZeroWaste and the Australian Road Research Board.

10 REFERENCES


EcoRecycle 1997. Investigation into the use of recycled crushed concrete for road base use. VicRoads, Alex Fraser, CSR Readymix Quarries and Independent Cement and Lime, Melbourne, Australia.

Gabr, A.G. 2012. Repeated load testing for primary evaluation of recycled concrete aggregate in pavements. PhD dissertation, University of South Australia, School of NBE.


Reuse of dredged sediments for hydraulic barriers: adsorption and hydraulic conductivity improvement through polymers

La réutilisation des sédiments dragués pour barrières hydrauliques: l'adsorption et l'amélioration de la conductivité hydraulique avec des polymères

Di Emidio G., Verastegui Flores R.D., Bezuijen A.
Ghent University, Zwijnaarde, Belgium

ABSTRACT: Environmental management and handling of dredged sediments is important worldwide, as enormous amounts of dredged material emerge from maintenance, construction and remedial works within water systems. Usually these materials, after temporary upland disposal in lagoons, are disposed in landfills. The aim of this study is to analyse the possible reuse of these sediments as a low-cost alternative material for landfill covers. The mechanisms through which polymers can improve the efficiency of dredged sediments for waste containment impermeable barriers were investigated. An anionic polymer was adsorbed to the surface of a dredged sediment. Hydraulic conductivity and batch sorption tests were executed to study the barrier performance and the transport parameters of this treated soil. Polymer treatment maintained low hydraulic conductivity of the soil to electrolyte solutions in the long term. The polymer treatment helped the soil to retain the spread of pollution.

RÉSUMÉ: La gestion de l'environnement et des sédiments dragués est important partout, parce que énormes quantités de matériaux de dragage sortent de l'entretien, la construction et les travaux de réparation dans les systèmes d'eau. Habituellement, ces matériaux, après le stockage dans les lagunes temporaire, sont déplacé dans les décharges. Le but de cette étude est d'analyser la possibilité de réutiliser ces sédiments en tant que matériau alternative à faible coût pour les couvertures d'enfouissement. Les mécanismes par lesquels les polymères peuvent améliorer l'efficacité des sédiments dragués pour les barrières de confinement des déchets imperméables ont été examiné. Un polymère anionique a été adsorbé à la surface des sédiments de dragage. Des essais de conductivité hydraulique et de sorption ont été exécutés pour étudier la performance de barrière et les propriétés de transport de cette terre traitée. Le traitement de polymère maintient une faible conductivité hydraulique du sol à le solutions électrolyte à long terme. Le traitement polymère contribué à résister la propagation de la pollution dans le sol.

KEYWORDS: reuse of dredged sediments, polymer treatment, low permeable hydraulic barriers.

INTRODUCTION

Soil contamination by heavy metals has been a long-term and worldwide environmental problem generated by anthropogenic activities of the past several decades. Heavy metals present in soils could find their way into human and animal populations through direct exposure or food chain/web, posing a serious risk to human health (Garcia-Sanchez et al. 1999; Gao et al. 2003; Ling et al. 2007). Heavy metals may be retained in clay soils by several soil phases or mechanisms, such as exchangeable, carbonate, hydroxide and organic phases (Griffin et al. 1976; Plassard et al. 2000; Sharma and Reddy 2004). The factors affecting the sorption of contaminants in soils are: (1) contaminant characteristics, such as water solubility, polar-ionic character, octanol-water partition coefficient; (2) soil characteristics such as mineralogy, permeability, porosity, texture, homogeneity, organic carbon content, surface charge, and surface area; and (3) fluid media characteristics, such as pH, salt content, dissolved carbon content.

Landfill sites for both chemical and industrial waste might be a serious threat for the environment, when not properly designed. To avoid pollution of the ground and groundwater, landfill sites are sealed with compacted clay liners (CCLs), geomembranes and Geosynthetic Clay Liners (GCLs). Suitable barriers must have a low permeability. To meet this property, the soil contained in CCLs and GCLs must fulfill some well-known physical and hydraulic criteria (Daniel 1993, Mitchell 1993).

Next to standard CCL and GCLs, there are emerging innovative barrier materials and systems, more efficient and/or less costly. Alternative evapotranspirative barriers (Malusis and Benson, 2006, Zornberg and McCartney 2010, Kison et al. 2012) or alternative barrier materials (such as, among others, paper sludge (Rajasekaran et al. 2000) dredged sediments (Di Emidio et al. 2006) can be necessary when: (1) high costs are associated with prescriptive materials and methods, (2) prescribed materials are not readily available, (Shackelford, 2005) and (3) when alternative materials are available in large quantities. In this regard, Di Emidio et al. (2006) studied the suitability of dredged materials to be used as alternative cover liner material for landfills. Different dredged materials were analyzed by means of laboratory tests, focusing on physical properties and hydraulic conductivity performance. As a result, acceptable zones (Daniel, 1993) based on hydraulic conductivity were established. Test results showed suitability of dredged sediments as hydraulic barrier alternative materials. Therefore, the use of dredged materials for cover liners
represents an interesting opportunity for the future reuse of non-contaminated dredged materials.

On the other hand, dredged sediments are often polluted with contaminants, such as heavy metals (Singh et al. 2000, Mulligan et al. 2001, Peng et al. 2009). Methods to remove said metals from dredged sediment (Mulligan et al. 2001, Meegoda and Ruvini 2001, Bradl 2005, Peng et al. 2009) might be cumbersome and expensive. Therefore, alternative methods resulting in dredged sediments that retain heavy metals are highly needed.

Mazzieri et al. (2010) compared a polymer amended GCL with a conventional GCL permeated with a synthetic metal-rich acidic solution in order to compare the hydraulic, buffering and contaminant retention properties of the GCL materials. The breakthrough of metals occurred much earlier in the untreated GCL than in the polymer treated GCL, which was able to retain metals more effectively. Further insights are required to better understand the mobility of heavy metals in polymer treated clays and the ability of such clays to retain the heavy metals in the long-term.

This study involves the treatment of kaolin clay (as reference material) and dredged sediments with different percentages of an anionic polymer, Na-CMC (Sodium CarboxyMethyl Cellulose). This treatment is meant to improve their hydraulic performance as a lining material. This paper shows preliminary results (using MgCl$_2$ and sea water as reference solutions) to study the effects on both the hydraulic conductivity and the adsorption characteristics of polymer treated clays, such as kaolin and dredged sediments. The adsorption on polymer treated clays of heavy metals such as Zn, Cu and Pb is currently under investigation.

2 MATERIALS

A commercial processed kaolin Rotoclay® HB (Goonvean, St. Austell, UK) and a dredged sediment (DS) were used in this investigation. The kaolin was chosen as reference material because it has been largely used in previous laboratory research. The dredged sediment was obtained from Kluizendok in Ghent, Belgium. Table 1 shows some properties of the base materials used in this research. Both materials were treated with an anionic polymer, Sodium CarboxyMethylCellulose (Na-CMC) using different polymer dosages (2% and 8%) by dry weight of soil. The treatment consists of pouring a soil in a polymeric solution using a mechanical stirrer. The slurries obtained are then oven dried. After drying, the soils are ground using a mortar grinder (Di Emidio, 2010, 2012).

Deionised water, produced using a water purification system, was used as reference solution. A reference electrolyte compound, MgCl$_2$, was used for preliminary batch sorption tests on the treated and untreated soils. The electrolyte solutions were prepared by dissolving salts in deionised water. Moreover, natural seawater from the North Sea (near Oostende in Belgium) was used as permeant solution for the hydraulic conductivity tests on the treated and untreated soils. Table 2 and 3 show the chemical characteristics of deionized water and seawater.

3 METHODS

3.1 Batch sorption test

To study the adsorption of MgCl$_2$ on the treated and untreated soils, batch sorption tests were performed following the ASTM D4646. Different concentrations of MgCl$_2$ were used to prepare the equilibrium solutions for the batch sorption test. The untreated and treated soils were mixed for 24 hours in a rotatory table with MgCl$_2$ solutions of different concentrations (100 mg/l, 600 mg/l, 2000 mg/l, 6000 mg/l), using a soil-to-solution ratio 1:4. Then the slurries were separated by centrifugation. A centrifugation speed of 3000 rpm was sufficient to separate untreated soils from the solution, whereas a centrifugation speed of 10000 rpm was necessary to separate the treated soils from the solution. The sorption isotherms were obtained by plotting the sorbed mass of Mg$^{2+}$ and Cl$^-$ (meq/100g of soil, measured with a Spectroquant Photometer) vs. the equilibrium MgCl$_2$ concentration.

Table 1. Properties of the materials analyzed

<table>
<thead>
<tr>
<th>Parameter</th>
<th>kaolin</th>
<th>DS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type / source</td>
<td>Rotoclay®/Austell</td>
<td>Kluizendok</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.64</td>
<td>2.75</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>59.0</td>
<td>44.1</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>38.0</td>
<td>27.1</td>
</tr>
<tr>
<td>Swell index (ml/g)</td>
<td>3.71</td>
<td>2.29</td>
</tr>
<tr>
<td>Silt content (%)</td>
<td>62.4</td>
<td>49.3</td>
</tr>
<tr>
<td>Clay content (%)</td>
<td>35.3</td>
<td>5.0</td>
</tr>
<tr>
<td>Sand content (%)</td>
<td>0.0</td>
<td>45.7</td>
</tr>
</tbody>
</table>

Table 2. Chemical analysis of the solutions used

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Deionized water</th>
<th>Seawater</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC (mS/cm)</td>
<td>0.0039</td>
<td>49.9</td>
</tr>
<tr>
<td>Salinity (-)</td>
<td>0.0</td>
<td>32.4</td>
</tr>
<tr>
<td>pH (-)</td>
<td>7.57</td>
<td>7.78</td>
</tr>
<tr>
<td>Na$^+$ (M)</td>
<td>-</td>
<td>0.455</td>
</tr>
<tr>
<td>K$^+$ (M)</td>
<td>-</td>
<td>0.012</td>
</tr>
<tr>
<td>Mg$^{2+}$ (M)</td>
<td>-</td>
<td>0.053</td>
</tr>
<tr>
<td>Ca$^{2+}$ (M)</td>
<td>-</td>
<td>0.012</td>
</tr>
<tr>
<td>Cl$^-$ (M)</td>
<td>-</td>
<td>0.561</td>
</tr>
<tr>
<td>SO$_4^{2-}$ (M)</td>
<td>-</td>
<td>0.024</td>
</tr>
<tr>
<td>HCO$_3^-$ (M)</td>
<td>-</td>
<td>0.003</td>
</tr>
<tr>
<td>CO$_3^{2-}$ (M)</td>
<td>-</td>
<td>0.0003</td>
</tr>
<tr>
<td>NO$_3^-$ (M)</td>
<td>-</td>
<td>0.0007</td>
</tr>
</tbody>
</table>

Table 3. Chemical properties of the MgCl₂ solutions used for the Batch Sorption test

<table>
<thead>
<tr>
<th>MgCl₂ (mg/L)</th>
<th>EC (mS/cm)</th>
<th>Salinity (-)</th>
<th>pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.301</td>
<td>0.0</td>
<td>6.87</td>
</tr>
<tr>
<td>200</td>
<td>1.382</td>
<td>0.5</td>
<td>7.04</td>
</tr>
<tr>
<td>2000</td>
<td>4.16</td>
<td>2.1</td>
<td>7.3</td>
</tr>
<tr>
<td>6000</td>
<td>12.13</td>
<td>6.46</td>
<td>7.7</td>
</tr>
</tbody>
</table>

3.2 Hydraulic conductivity test

Flexible wall hydraulic conductivity tests were conducted in order to investigate the impact of 8% of polymer addition on the hydraulic performance of the soils to a high concentrated electrolyte solution (natural seawater). The hydraulic conductivity tests were performed with an average effective stress of 30 kPa on 10 cm diameter samples with an initial porosity of \( n \approx 0.718 \) (kaolin) and \( n \approx 0.542 \) (dredged sediment). To prepare the kaolin samples, the untreated and treated soil were poured dry in a stainless steel ring (0.45 g/cm², as a standard GCL, 10 cm diameter) with a fixed height between two porous stones and submerged with seawater, with a sitting weight on top, for about one week. The dredged sediment sample was prepared by standard proctor compaction (ASTM D0698) with a water content two points higher than the optimum, to simulate a Compacted Clay Liner.

4 RESULTS AND DISCUSSION

4.1 Batch sorption test results

Figure 1 shows the sorption isotherms of the treated and untreated kaolin (a) and of the treated and untreated dredged sediment (b). The adsorbed mass of ions is plotted here vs. the equilibrium concentration of the MgCl₂ solutions. As shown in the graphs, batch sorption test results demonstrate that the sorbed mass of magnesium cations is higher onto the polymer treated soil compared to the untreated soil.

4.2 Hydraulic conductivity test results

Figure 2 shows the hydraulic conductivity test results of the treated and untreated soils. As shown in Figure 2.a, the hydraulic conductivity to seawater of the kaolin treated with 8% of the polymer was lower compared to that of the kaolin treated with 2% of the polymer. This result demonstrates that the hydraulic performance of a kaolin clay as barrier increases with increasing polymer dosage.

Figure 2.b shows that the hydraulic conductivity to seawater of the dredged sediment treated with 8% of the polymer is nearly two orders of magnitude lower compared to that of the untreated dredged sediment.
5 CONCLUSION

The sorption isotherms of Mg$^{2+}$ and Cl$^-$ on the treated and untreated kaolin and on the treated and untreated dredged sediment were analyzed. The adsorbed mass of ions was plotted vs. the equilibrium concentrations. Batch sorption test results demonstrated that the sorbed mass of magnesium cations was higher onto the polymer treated soils compared to the untreated soils. These results are promising in view of metals retention in polymer treated dredged sediments. To further demonstrate the higher retention ability of polymer treated clays, the adsorption of heavy metals on kaolin, bentonite and dredged sediments is currently under investigation.

Hydraulic conductivity test results of treated and untreated soils were shown. The hydraulic conductivity to seawater of the kaolin treated with 8% of the polymer was lower compared to that of the kaolin treated with 2% of the polymer. This result demonstrates that the hydraulic performance of a kaolin clay increases (the hydraulic conductivity decreases) with increasing polymer dosage. The hydraulic conductivity to seawater of the dredged sediment treated with 8% of the polymer was significantly lower compared to that of the untreated dredged sediment. These results suggest the possible reuse of dredged sediments as alternative low cost impermeable barrier materials to isolate polluted sites and landfills.

6 ACKNOWLEDGEMENTS

The authors would like to acknowledge former thesis students (S. Lerno and R. Van den Steen) and technical staff (J. Van der Perre) for helping with the experiments.

7 REFERENCES


Characterization of recycled materials for sustainable construction

Caractérisation des matériaux recyclés pour la Construction durable

Edil T.B.

University of Wisconsin-Madison, USA

ABSTRACT: Recyclable materials and industrial byproducts provide an environmentally and economical alternative to natural earthen materials when used safely and wisely in geotechnical construction. In particular, the construction of various elements of transportation systems requires large quantities of materials and locally available recyclable materials can be used extensively enhancing sustainability of construction. This paper addresses the rapid characterization required for this new class of materials such as recycled asphalt pavement, concrete aggregate, and coal combustion residues (fly ash, bottom ash). Recycled materials and industrial byproducts require an assessment of their environmental suitability in terms of potential impacts on surface and ground water quality for their acceptance. Finally, their field behavior need to be evaluated and their contribution to sustainability to be assessed.

RÉSUMÉ : Les matériaux recyclables et les sous-produits industriels fournissent un environnement économe aux matériaux de terre naturels une fois utilisés sans risque et sont largement utilisés dans la construction géotechnique. En particulier, la construction de divers éléments des systèmes de transport exige de grandes quantités de matériaux et localement des matériaux recyclables disponibles peuvent être employés intensivement augmentant la durabilité de la construction. Ce document adresse la caractérisation rapide exigée pour cette nouvelle classe des matériaux tels que le trottoir réutilisé d'asphalte, l'agrégat concret, et les résidus de combustion de charbon (cendres volantes, cendre inférieure). Étant les matériaux réutilisés ou les sous-produits industriels, leur acceptation exige une évaluation de l'aptitude environnementale en termes d'impacts potentiels sur la qualité extérieure et d'eaux souterraines. En conclusion, leur besoin de comportement de champ d'être évalué et leur contribution à la durabilité à évaluer.

KEYWORDS: recycled materials, sustainable construction, recycled asphalt pavement, recycled concrete aggregate, coal ash.

1 INTRODUCTION

Development and growth need to be sustainable, in other words, integrate environmental, economic, and social dimensions towards global stewardship and responsible management of resources. Strategies need to evolve for sustainable development. Large quantities of natural and processed materials are used in construction activities such as buildings, transportation facilities, infrastructure, and environmental applications. These materials use natural resources and consume large quantities of energy to extract or process with associated green house gas emissions. Transportation facilities, such as highways, in particular use large quantities of materials in initial reconstruction and also during periodic rehabilitation. Recycling industrial byproducts and construction materials in highway construction can generate “green highways” where use of virgin materials and large amounts of energy and generation of green house gas emissions are minimized.

The necessary steps for characterization of widely used recycled materials (i.e., recycled asphalt pavement and recycled concrete aggregate) and industrial byproducts (i.e., coal combustion products such as bottom ash and fly ash) in highway construction are presented and discussed. The approaches for determining their physical characteristics, geomechanical behavior (i.e., resilient modulus), durability (i.e., freeze-thaw cycling, temperature effects, wet-dry cycling), constructability (i.e., compaction), material control, and their environmental suitability (i.e., leaching characteristics) for alternative beneficial uses are presented. Life cycle assessment (LCA) of the environmental benefits and the life cycle cost analysis (LCCA) for use of these materials are also discussed.

2 CHARACTERIZATION

We have been characterizing natural earthen materials systematically for nearly a century. Widespread use of these recycled materials is relatively new and time window to characterize them is short because of economical and environmental drivers. Testing methods developed for soils and construction specifications for natural aggregates and soils can be adapted to this new class of recycled materials and the existing pavement design procedures can be followed.

Material control in terms of variability of composition, grain size characteristics, inclusion of impurities are issues that need to be assessed for recycled materials, as they are products of anthropogenic processes rather than geological processes. There may be differences arising from basic material characteristics that may impact constructability in terms of compaction control. Modern pavement design requires resilient or elastic modulus as the primary geomechanical property. On the basis of this property layer thicknesses in a pavement and service life of a pavement can be determined. Durability under climatic effects, i.e., freeze-thaw and wet-dry cycles, is a critical quality for pavement materials because the pavements are surficial and directly influenced by the climate. Some of these materials have sensitivity to temperature in ways we are unaccustomed dealing with soils. Finally, while we do not question the environmental suitability of traditional materials like crushed aggregate, concrete and asphalt used in highway construction, use of recycled materials and industrial byproducts requires evaluation of environmental suitability, i.e., the leaching characteristics.

3 MATERIALS, APPLICATIONS & CRITICAL CHARACTERISTICS

The outstanding characteristics of a range of recycled materials widely used in highway construction are described along with typical applications.
3.1 Recycled Asphalt Pavement (RAP)

RAP (Figure 1) is produced by removing and reprocessing the hot mix asphalt (HMA) layer of existing asphalt pavement (Guthrie et al., 2007; FHWA, 2008). There is some ambiguity regarding the nomenclature involved in the production of RAP. Full depth reclamation (FDR) refers to the removal and reuse of the HMA and the entire base course layer; and recycled pavement material (RPM) refers to the removal and reuse of either the HMA and part of the base course layer or the HMA, the entire base course layer and part of the underlying subgrade, implying a mixture of pavement layer materials (Guthrie et al., 2007, Edil et al. 2012). Unless specified, these three distinct recycled asphalt materials are collectively referred to as RAP.

RAP is typically produced through milling operations, which involves the grinding and collection of the existing HMA, and FDR and RPM are typically excavated using full-size reclaimers or portable asphalt recycling machines (FHWA, 2008, Guthrie et al. 2007). RAP can be stockpiled, but is most frequently reused immediately after processing at the site. Typical aggregate gradations of RAP are achieved through pulverization of the material, which is typically performed with a rubber-tired grinder.

RAP particles are coated with asphalt and its most value added use is in production of hot mix asphalt (HMA) with the benefit of reducing the fresh asphalt content. Seven RAP and 2 RPM samples collected from geographically diverse 7 states in the U.S.A. indicated a range of 5-7% asphalt content. RAP and RPM are widely used as unbound base material and the most common test used for specification is Grain Size Analysis. The most distinguishing physical characteristics are the grain size with some samples coarser and others finer. D50 of the 9 samples ranged 1.6 to 5.8 mm and the fines content was less than 2%. These materials all classified as A-1-a or A-1-b according to the AASHTO soil classification system. These samples had an average impurity (geotextiles, pavement markings, etc.) content of 0.2% for RAP, indicating that recycling industry has developed sufficient controls.

The compaction characteristics using the modified Proctor test indicated that the maximum dry unit weight (MDU) varies within a narrow range (19.4 to 21.5 kN/m³) for RAP and the optimum moisture contents (OMC) (5.2 to 8.8%). OMC correlates significantly with the uniformity coefficient and percent moisture absorption and MDU correlates with OMC for RAP (Bozyurt et al. 2012).

Summary resilient modulus (SRM) calculated at a bulk stress of 208 kPa, typical of base course layer) of the 9 RAP and RPM samples measured at OMC and 95% modified Proctor MDU, indicated that RAP/RPM has higher SRM (168 to 266 MPA) than natural crushed aggregate (152 MPA) and is significantly correlated with grain size characteristics (percent fines, D50), asphalt content, specific gravity, and percent absorption (Bozyurt et al. 2012).

Application of freeze-thaw cycles indicated that SRM decreased in a range of 28 to 53% up to 20 cycles. However, RAP still had a higher stiffness than natural crushed rock aggregate regardless of the number of freeze-thaw cycles (Edil et al. 2012). Because of its asphalt content RAP can be expected to be sensitive to temperature changes. A decrease of approximately 30% in SRM was observed in RAP between the 23 and 35 °C. These temperature effects were absent in control specimens containing no asphalt. Micro-Deval and particle size distribution tests conducted on RAP after 5, 10, and 30 wet/dry cycles showed no apparent particle degradation (Edil et al. 2012).

RAP has excellent drainage capacity due to the hydrophobic nature of the asphalt coating and does not retain moisture (Nokkaew et al. 2012). Field leachate samples collected indicated that the concentrations of As, Se and Sb for RAP were slightly higher than the corresponding USEPA groundwater maximum contaminant level (MCL) but decreased rapidly after the first flush (Edil et al. 2012). Falling Weight Deflectometer (FWD) tests were conducted at a test facility (MNROAD) on pavement with base course material of RAP indicated relatively small variation in stiffness and resilient modulus seasonally and indicated no deterioration over 4 years.

The investigations undertaken on RAP indicate that it is a suitable material for unbound base course applications and shows equal or superior performance characteristics compared to natural aggregates in terms of stiffness, freeze-thaw and wet-dry durability, and toughness. Their compositional and mechanical properties vary in relatively small range. The relative differences of RAP from natural aggregate such as temperature sensitivity, plastic deformations, and water absorption and retention characteristics are also well established. To determine the various properties of RAP (e.g., compositional characteristics, grain size distribution, compaction, resilient modulus), existing standard test methods employed for natural crushed aggregate can be used with added consideration for temperature control. There are no established standards for freeze-thaw and wet-dry cycling but published research methods can be adopted (Edil et al. 2012).

3.2 Recycled Concrete Aggregate (RCA)

The production of RCA (Figure 1) involves crushing structural or pavement concrete to a predetermined gradation. Fresh RCA typically contains a high amount of debris and reinforcing steel, and it must be processed to remove this debris prior to reuse (FHWA 2008). One of the value-added applications is use of RCA as a base course material although it can be used in constructing working platforms over soft subgrade and drainage medium. Depending on the crushing methods, the particle size distribution of an RCA can have a wide variability; with a lower particle density and greater angularity than would normally be found in more traditional virgin base course aggregates. Residual mortar and cement paste are typically found on the surface of the RCA, as well as contaminants associated with construction and demolition debris. The self-cementing capabilities of RCA are an interesting secondary property. The crushed material exposes un-hydrated concrete that can react with water, potentially increasing the materials strength and durability when used as unbound base course for new roadway construction. It follows that service life could also be extended as a result of these properties.

Seven RCA samples collected from geographically diverse 7 states in the U.S.A. indicated a range of 5-6.5% mortar content. The most distinguishing physical characteristics are the grain size with some samples coarser and others finer. D50 of the 7 samples ranged 1.6 to 5.8 mm and the fines content was less than 2%. These materials all classified as A-1-a or A-1-b according to the AASHTO soil classification system. These samples had an average impurity (geotextiles, pavement markings, etc.) content of 1% for RCA, indicating that recycling industry has developed sufficient controls. The most predominant impurities for RCA were asphalt aggregate, aggregate with plastic fibers, brick, and wood chips. RCA derived from structures tend to have brick content. The effect of brick content up to 30% indicated no adverse effect on resilient modulus of RCA (Shedivy 2012).

The compaction characteristics using the modified Proctor test indicated that the maximum dry unit weight (MDU) varies within a narrow range (19.4 to 20.9 kN/m³) for RCA and optimum moisture contents (OMC) (8.7 to 11.8%). OMC is greater than RAP’s due the higher absorption capacity due to the porous nature of the mortar portion of RCA. OMC of RCA correlates significantly with the uniformity coefficient and...
percent moisture absorption and MDU correlates with OMC for RCA (Bozyurt et al. 2012)  

<table>
<thead>
<tr>
<th>RAP</th>
<th>RCA</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA)" /></td>
<td></td>
</tr>
</tbody>
</table>

Summary resilient modulus (SRM calculated at a bulk stress of 208 kPa, typical of base course layer) of the 7 RCA samples measured at OMC and 95% modified Proctor MDU, indicated that RCA has higher SRM (163 to 208 MPA) than natural crushed aggregate (152 MPA) and lower than RAP/RPM. SRM is significantly correlated with $D_{90}$ and OMC (Bozyurt et al. 2012).

Application of freeze-thaw cycles indicated that SRM decreased 10-18% during the first five freeze-thaw cycles, but then an increased 30-38% above the initial SRM after 20 freeze-thaw cycles. The self-cementing properties of RCA and fines content generation over time could explain why an increase in stiffness after five freeze-thaw cycles occurred. (Bozyurt et al. 2011). Micro-Deval and particle size distribution tests were conducted on RCA after 5, 10, and 30 wet/dry cycles and no apparent trend was found between particle degradation and wet/dry cycling of the material.

RCA has high drainage capacity but retains moisture more than RAP and natural aggregate base because of its hydrophilic cement mortar (Nokkacew et al. 2012). Laboratory batch and column leach tests and field leachate samples collected indicated that RCA base course has high alkalinity (pH = 10.8 to 12.9). As, Cr, Pb, and Se exceeded the maximum contaminant levels (MCLs) for the USEPA drinking water standard both at the field sites and in the laboratory column leaching tests. The concentrations of As, Pb, and Se for RCA exceeded the corresponding MCL only once or twice and the leaching behaviors were similar to that of the control natural crushed aggregate base course. As and Cr appear to be mainly sourced from the cement mortar based on the acid digestion results (Edil et al. 2012). Falling Weight Deflectometer (FWD) tests that were conducted at the MnRoad test facility on pavement with base course material of RCA indicated relatively small seasonal variation in modulus and no deterioration over 4 years.

The investigation undertaken on RCA indicate that it is a suitable material for unbound base course applications and shows equal or superior performance characteristics compared to natural aggregates in terms of stiffness, freeze-thaw and wet-dry durability, and toughness. Their compositional and mechanical properties vary in relatively small range. The relative difference of RCA from RAP and natural aggregate is its water absorption and retention characteristics. RCA displays high alkalinity thus oxyanions (As, Se, and Cr) should be given more attention to as they demonstrate enhanced leaching in a highly alkaline environment.

To determine the various properties of RCA (e.g., compositional characteristics, grain size distribution, compaction, resilient modulus), existing standard test methods employed for natural crushed aggregate can be used. There are no established standards for freeze-thaw and wet-dry cycling but published research methods can be adopted (Edil et al. 2012).

### 3.3 Coal Combustion Products (CCP)

CCPs of interest to highway construction include fly ash and bottom ash. When pulverized coal is burned in a dry bottom boiler, about 80 percent of the unburned material or ash is entrained in the flue gas and is captured and recovered as fly ash (Figure 2). The remaining 20 percent of the unburned material is dry bottom ash, a porous, glassy, dark gray material with a grain size similar to that of sand or gravelly sand (Figure 3). Although similar to natural fine aggregate, bottom ash is lighter and more brittle and has a greater resemblance to cement clinker. Beneficial use of bottom ash in highway applications, which is less than 50% of the material produced in the U.S.A., include structural fill (nearly half of all use), road base material, working platform material for construction of pavements over soft subgrade, fine aggregate in wearing surface in pavements and flowable fills, and as snow and ice control products. Bottom ash is predominantly well-graded sand-sized, usually with 50 to 90 percent passing a 4.75 mm (No. 4) sieve and 0 to 10 percent passing a 0.075 mm (No. 200) sieve (http://rmrc.wisc.edu).

Bottom ash has MDU of 11.8 to 15.7 kN/m² and OMC of 12-24%. Its internal friction angle varies from 32° to 45°. California bearing ratio (CBR) is typically 20 to 40. Summary resilient modulus (SRM calculated at a bulk stress of 208 kPa, typical of base course layer) for properly compacted bottom ash can be taken as 100 MPa. Bottom ash has similar drainage characteristic as sand with a hydraulic conductivity of 1x10⁻² mm/s. All standard tests used to characterize natural granular materials like sand can be directly used for bottom ash.

The fly ash is a fine-grained, powdery particulate material that is carried off in the flue gas and usually collected by means of electrostatic precipitators, baghouses, or mechanical collection devices such as cyclones. Beneficial use of fly ash in highway applications, which is less than 50% of the material produced in the U.S.A., include cement replacement/additive in concrete (nearly half of all use), structural fill, stabilization agent for road subgrade and base material, working platform material for construction of pavements over soft subgrade, flow agent in flowable fills, and mineral filler in asphalt layers (http://rmrc.wisc.edu). Self-cementing coal fly ashes are suitable materials for the stabilization of subgrade soils, recycled pavement materials, and road surface gravel. Fly ash stabilization can result in improved properties, including increased stiffness, strength and freeze-thaw durability; reduced hydraulic conductivity, plasticity, and swelling; and increased control of soil compressibility and moisture. Fly ash stabilized materials may be used in roadway construction, such as working platforms during construction, stabilized subgrade, subbase, and base layers (Edil et al. 2006). Recently published ASTM standard practice provides guidance for testing and designing of stabilization of soil and soil-like materials with self-cementing fly (ASTM D7762 2011).

The possibility of groundwater contamination by trace elements that are commonly associated with coal combustion by-products is a concern. Areas with sandy soils possessing high hydraulic conductivities and areas near shallow groundwater should be given careful consideration especially.

![Fly Ash (different colors)](image)
for large volume uses like structural fills. An evaluation of groundwater conditions, applicable state test procedures, water quality standards, and proper construction are all necessary considerations in ensuring a safe final product. There are several leaching tests and currently U.S. Environmental Protection Agency is nearing publication of new leaching standard appropriate for beneficial use application of CCPs and other similar materials (Kosson et al. 2002). U.S. EPA is currently reviewing its rules regarding beneficial use of CCPs.

4 ASSESSMENT OF SUSTAINABILITY

Assessment of sustainability involves the life cycle assessment (LCA) of the environmental benefits and the life cycle cost analysis (LCCA). LCA involves determining a variety of sustainability metrics (e.g., energy consumption, GHG emissions, water use, hazardous waste generation, etc.) associated with production of construction materials, their transportation to the construction site, and construction itself. These determinations can be made using available database programs such as the PaLATE model (Horvath, 2004). LCCA evaluates life cycle cost of design alternatives including the initial construction and maintenance based on service life. A convenient computer code named RealCost can be used for this purpose (FHWA 2004). Service life is a crucial part of this analysis and materials with higher modulus typically result in a longer service life for the same thickness of pavement layers. Examples of LCA and LCCA are available (Lee et al. 2011).

A rating system for sustainable highway construction, named Building Environmentally and Economically Sustainable Transportation-Infrastructure-Highways, BE2ST-IN-HIGHWAYS™ was developed to provide a quantitative methodology for rating the benefits of sustainable highway construction (Lee et al. 2011). The methodology is grounded in quantitative and auditable metrics so that a transparent linkage exists between the project rating and the sustainable practices employed in construction. This rating system can be employed by the highway construction industry and agencies to quantitatively evaluate sustainable practices and to incorporate sustainable elements into projects.

The BE2ST-IN-Highways™ system evaluates sustainability of a highway project in terms of a quantitative difference between a reference design and proposed alternative design(s). Thus, the reference highway design must be defined realistically. A conventional design approach in which sustainability concepts are not incorporated explicitly can be used as a reference design. The analysis assumes that the service life of conventional and alternative designs can be based on an international roughness index (IRI) prediction made with the Mechanistic Empirical Pavement Design Guide (M-EPDG) program (NCHRP) and the rehabilitation occurs at the end of the predicted service life.

5 CONCLUSIONS

1. It is imperative that industry-wide sustainable construction practices be adopted and recycled materials play a significant role in earthen construction where large quantities of materials are used such as in roadway construction.
2. Benefits of recycled materials include reduction in greenhouse gas emissions, energy, natural resources, and cost.
3. Wise use of recycled materials may create longer lasting structures and reduction in cost.
4. Conducting quantitative analyses using appropriate sustainability metrics to assess alternatives involving recycled materials is imperative.

6 ACKNOWLEDGEMENTS

The information and ideas presented were developed through numerous projects involving many associates and students, too long to list here. Prof. Craig H. Benson and the Recycled Materials Resource Center are acknowledged.

7 REFERENCES


Technical and Economic Analysis of Construction and Demolition Waste Used in Paving Project

Analyse technique et économique des déchets dans la construction de pavage

Farias A., Fucale S., Gusmão A.
University of Pernambuco, Recife, Brazil

Maia G.
Gusmão Engenheiros Associados, Recife, Brazil

ABSTRACT: In this research, the technical and economic feasibility was analyzed as to the use of wastes originated from the deep excavation activity (continuous helical piles) and by demolition of old constructions for the application in layers of subgrade, sub-base and base in paving project. For such, laboratory tests were conducted for the verification of granulometry, real density, limits of consistence, compaction with intermediate energy, California Bearing Ratio (CBR) with measuring of expansion, which exposed the quality of the materials and its potentials. A comparative analysis was carried out between recycled material costs and the aggregate commonly used in paving project, discovering, besides the technical advantage, also the economic advantage of this alternative material.

RÉSUMÉ: Dans cette étude, nous avons analysé la faisabilité technique et économique de l'utilisation des déchets générés par l'activité des fondations profondes (type pieux CFA) et la démolition des anciens bâtiments pour une utilisation dans des couches de renforcement de la couche de forme, couche de fondation et le revêtement de base. Par conséquent, les tests de laboratoire ont été effectués afin de vérifier la taille des particules, les limites de densité réelles, avec compactage d'énergie intermédiaire, de soutien Californie Index pour mesurer l'expansion, qui a exposé la qualité des matériaux et de leur potentiel. Toujours en détenant une analyse comparative entre le coût des matériaux recyclés et des agrégats couramment utilisé dans le revêtement, encontando, outre l'avantage technique, l'avantage de ce matériau alternative économique.

KEYWORDS: Construction and Demolition Waste, Recycling, Paving.

1 INTRODUCTION

The civil construction chain is one of the most important economic sectors in Brazil, between the years of 2004 and 2010, this sector grew 42.41%, representing an annual average of 5.18%. In 2011, between January and September, there was an increment of 3.8% over the same period last year, with the creation of 309,425 formal jobs during the first ten months of this year (CBIC, 2011). Moreover, constructive activities still have great social importance for countries, since they employ, direct or indirectly, a large percentage of manpower.

However, despite the economic importance, the construction industry has significant negative impacts to society, such as large waste generation, since the Construction and Demolition Waste - CDW, as it is in the city of Recife, capital of the state of Pernambuco, Brazil, represents 41% of all municipal solid waste (Gusmão, 2008).

Current Brazilian legislation, through the CONAMA Resolution 307/2002 predicts the principle of the polluter-payer for the civil construction sector, in other words, all the CDW is responsibility of the own sector, involving all the responsibility for waste management, including the final disposition only in places duly licensed.

In this scenario, it is essential that construction practices sustainable principles, with construction technologies that emphasize prevention, reduction, reuse and recycling of materials, besides the collection and disposal of waste committed.

In order to encourage the reuse and recycle of CDW in the construction itself, there are already technical rules to standardize and regulate the use of these materials, and one of them is the 15.116:2004 NBR, which deals with recycled aggregates from CDW – the use in preparation of concrete with no structural function and paving projects.

The paper presents a case of construction work to build a shopping center in Recife, where the waste from excavation (soil) of continuous helical displacement piles and the recycled CDW were used in an innovative way in the paving projects of the worksite itself, obtaining at the end of the construction, a significant economy of resources and materials.

1.1 Continuous helical displacement pile

The continuous helical displacement piles were introduced in Recife in the 1990s. At the time the equipment was brought from the Southeast region, with high mobilization costs. Its more frequent use in the construction of buildings started in the city in 2001 and is currently the most widely used type of pile in the construction of buildings in Recife. In 2010, it is estimated that the helical piles accounted for about three quarters of the pile market for buildings in the city (Gusmão, 2011).

The continuous helical piles have some peculiarities that popularized their use in the urban environment, especially the fact of not causing vibrations and for having great productivity compared to other solutions, such as pre-molded, metal and excavated piles.

However, there is one particular aspect which can be a limiting factor to its use: the production of excavation waste (soil). According to CONAMA Resolution 307/2002, the excavated soil is a CDW, and as such, it should be tracked throughout the whole building process: separation, storage, transportation, recycling and final disposal. Current legislation does not allow the soil to be disposed without any control.

Even in licensed areas, landfills of CDW occupy large areas, which could have a nobler purpose in the urban environment.
2 CHARACTERIZATION OF THE ENTERPRISE

It consists in the construction of a horizontal shopping center in Recife, Pernambuco, Brazil. The edification is formed by an pre-molded arched concrete structure with a total construction area (shopping areas and garages) of 255,500 m².

From the topographic point of view, the natural terrain did not possess pronounced leveling differences. As for the geological point of view, the land is located in the fluvial-marine plain, within the undifferentiated marine terrace.

On the terrain of the enterprise, there were 07 major warehouses and 10 smaller deposits, totaling an area of 20,949 m² of demolition, responsible for generating approximately 18,900 tons or 13,500 m³ of CDW. Considering also the temporary installations and concrete slabs of the service paths, it is estimated that globally 23,560 tons or 16,830 m³ of CDW were generated.

Given the geotechnical characteristics of the land and construction schedule, continuous helical piles were designed and implemented for the foundation of all edifications. Table 1 shows the total quantitative at the end of the work, as well as the production of excavated soil. A total of 25,013 m³ or 42,522 tons of soil (admitting an apparent specific weight of 17 kN/m³).

There is, therefore, a total waste (demolition + soil deriving from piling) of 66.082 tons. If all this material was taken to a licensed landfill, the cost of provision would be of US$ 28,00/ton, or a total of US$1.9 million, besides environmental costs.

Having this in mind and the large area of paving of the site, a technical and economic study for the reuse of waste from the excavation residues in the pavement layers of the work was proposed. The excavated material was then separated and stored in an area of the own work site.

Table 1 – Quantitative of piles.

<table>
<thead>
<tr>
<th>Diameter of Pile</th>
<th>Quantity</th>
<th>Length Medium (m)</th>
<th>Excavated Soil (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>504</td>
<td>21.68</td>
<td>1,713</td>
</tr>
<tr>
<td>500</td>
<td>2,965</td>
<td>23.30</td>
<td>17,506</td>
</tr>
<tr>
<td>600</td>
<td>692</td>
<td>22.67</td>
<td>5,794</td>
</tr>
</tbody>
</table>

3 PAVING OF THE ENTERPRISE

The total paved area of the work was of 96,463 m² and is in accordance with the specifications of the paving project, the sub-base layer in the circulation of the exterior parking lot (flexible pavement) should be stabilized granulometrically with sandy material, and have a thickness of 0.20 m and minimum California Bearing Ration (CBR) of 20%.

In the parking spaces outside (semi-rigid pavement), the sub-base layer should consist of improved soil with 4% cement, 0.10 m thick and minimum CBR of 20%.

Data showed that the paving and land leveling work would require 22,631 m³ of natural noble aggregates, in other words, there was a potential for the use of much of the residues in the work site.

4 METHODOLOGY

In order to enable the use of its own wastes in the paving projects of the work, the following actions were established:

i) Processing of waste from demolition of the warehouses through a mobile unit installed at the construction site. These wastes are in this paper called recycled residues of civil construction - RRCC;

ii) Separation and storage of excavated soil in the implementation of continuous helical piles;

iii) Conduction of laboratory tests to characterize the RRCC and the excavated soil.

For the study of the technical viability of the use of excavated soil in the paving process, several laboratory tests in four distinct phases were conducted, whose tests are summarized in Table 2.

Table 2 – Summary of the characterization tests and assayed samples.

<table>
<thead>
<tr>
<th>Phase</th>
<th>RRCC or Mixture</th>
<th>Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Soil, RRCC, R30S70 and R60S40</td>
<td>Granulometry, CBR and Limits of Consistency</td>
</tr>
<tr>
<td>2</td>
<td>Soil, RRCC and R60S40</td>
<td>Granulometry, Limits of Consistency, Real Density of grains, Compaction, CBR, shape Index, Abrasion “Los Angeles” e Level of sulphates</td>
</tr>
<tr>
<td>3</td>
<td>R40S60 (samples collected in experimental field)</td>
<td>Granulometry, Compaction and CBR</td>
</tr>
<tr>
<td>4</td>
<td>RRCC, R60S40, R40S60 e R30S70 (samples collected in experimental field)</td>
<td>Compaction, Humidity in situ, Density in situ (Sand Flask)</td>
</tr>
</tbody>
</table>

RRCCR – recycled residues of civil construction; R30S70 – 30% RCCR + 70% pile soil; R40S60 – 40% RCCR + 60% pile soil; R60S40 – 60% RCCR + 40% pile soil

5 TECHNOLOGICAL CHARACTERIZATION OF THE MATERIAL

5.1 Composition of RRCC

Figure 1 shows the gravimetric composition of RRCC. It is observed that is predominant the concrete, since warehouses had large areas of concrete floor. In the small material, with diameter less than 4,8 mm, it was not possible to differentiate the waste just by sight.

Figure 1 – Gravimetric composition of RRCC.

5.2 Composition of the excavation soil

For the development of the design of the foundations of the project, a total of 65 reconnaissance assays for percussion were performed. Initially, it was thought that the land would present deposits of soft soils, which are typical for this region of the city, but tests showed a basement formed by predominantly sandy soils.

Figure 2 shows the prediction of the type of excavated soil obtained from surveys conducted initially in the terrain. It is observed that sandy soils represent 85% of total soils present in the subsoil up to an average depth of 23m. The major difficulty in reusing soil from the excavation of a pile is that the excavated material is the result of full depth of the pile, and there are no means to segregate it. For a stratified profile, a completely heterogeneous material will be found.
5.3 Granulometry

Figure 3 shows the granulometric curve of soil samples and of RRCC. It is observed that the RCCR is classified as gravel with thick and medium-sized sand. On the other hand, the soil is medium-sized and thin sand, which coincides with the prior prediction made by the assays.

5.4 Compaction curves

Figure 4 shows some compaction tests with intermediate energy. It is observed that the optimum moisture varies between 4.1 and 9.1%, which are typical values for granular soils. It is also observed that the mixture soil + RRCC presented higher densities than the materials isolated, probably due to the fact that a higher degree of the group was reached.

5.5 California Bearing Ratio (CBR)

Table 3 shows the CBR values obtained for the soil, RRCC and mixture of 60% RRCC and 40% soil (R60S40). The average values were equal to 39, 189 and 115%, respectively. The expansion values ranged from 0 to 0.2%.

5.6 Shape Index

As the shape index approaches only the coarse aggregate with maximum characteristic dimension superior to 9.5 mm, only the RRCC samples were assayed. According to NBR 7809:1983, the maximum limit in the relation length/thickness is 3.0. This condition was met in both samples.

5.7 “Los Angeles” Abrasion

Just as in shape index, the “Los Angeles” abrasion test refers only to the coarse aggregate. For the two RRCC samples, the values were equal to 26.8 and 26.3% of depreciation, which are below the maximum limit of 50% set in the standard.

5.8 Sulphate levels

The maximum level of sulphate in relation to the mass of the recycled aggregate is 2%, according to NBR 15.116:2004. The values obtained in the assays of the soil and RRCC ranged between 0.04 and 0.09%, in other words, they were below the maximum limit set in the standard.

5.9 Technical Feasibility for using CDW in Paving Projects

Table 4 summarizes the results of some samples of soil, RRCC and the mixture soil-RRCC, comparing the values with the recommendation of NBR 15.116:2004. It was observed that only the soil did not meet the requirements for the use in paving layers. However, both the isolated RRCC as well as mixed with soil from the excavation of piles, meet all the criteria of the standard.

5.10 Economic feasibility of the use of CDW in paving projects

For the study of the economic feasibility of using CDW in paving layers in the project, an inquiry of the unitary cost of the acquisition of the aggregates specified in the paving project was initially performed, whose values are shown in Table 5. The costs of implementing the layers were not considered in the comparison, as it would be the same with the use of natural aggregates or CDW.

Table 6 shows the costs for the disposal at two sites licensed by the environmental agencies, which is an inert landfill or a processing plant for CDW. Both are located in the Metropolitan area of Recife.
Table 4 – Samples that did not meet the requirements of NBR 15.116:2004.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Samples</th>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniformity Coefficient (%)</td>
<td>NBR 15.116</td>
<td>10</td>
<td>4.88</td>
<td>3.62</td>
</tr>
<tr>
<td>Material through strainer N* (%)</td>
<td>Between</td>
<td>10 e</td>
<td>53.47</td>
<td>45.76</td>
</tr>
<tr>
<td>CBR (%) - Subgrade</td>
<td>≥ 12</td>
<td>10.20</td>
<td>18.70</td>
<td>190.5</td>
</tr>
<tr>
<td>CBR (%) - Sub-base</td>
<td>≥ 20</td>
<td>10.20</td>
<td>18.70</td>
<td>190.5</td>
</tr>
<tr>
<td>CBR (%) - Base</td>
<td>≥ 60</td>
<td>10.20</td>
<td>18.70</td>
<td>190.5</td>
</tr>
<tr>
<td>Subgrade Expansion (%)</td>
<td>≤ 1</td>
<td>0.20</td>
<td>0.10</td>
<td>0</td>
</tr>
<tr>
<td>Sub-base Expansion (%)</td>
<td>≤ 1</td>
<td>0.20</td>
<td>0.10</td>
<td>0</td>
</tr>
<tr>
<td>Expansion (%) - Base</td>
<td>≤ 0.5</td>
<td>0.20</td>
<td>0.10</td>
<td>0</td>
</tr>
<tr>
<td>Maximum dimension of grains (mm)</td>
<td></td>
<td>63.5</td>
<td>19.10</td>
<td>19.10</td>
</tr>
<tr>
<td>Shape Index</td>
<td>&lt; 3.0</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Depreciation</td>
<td>&lt; 50</td>
<td>*</td>
<td>*</td>
<td>26.76</td>
</tr>
<tr>
<td>Sulphate Content (%)</td>
<td>&lt; 2.0</td>
<td>0</td>
<td>0</td>
<td>0.05</td>
</tr>
</tbody>
</table>

1 – Soil; ii – RCCR; iii – R60S40

* Not determined

Table 5– Costs of the acquisition of materials for paving project.

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit</th>
<th>Unitary cost of the acquisition (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand for landfill</td>
<td>m³</td>
<td>14.00</td>
</tr>
<tr>
<td>RCCR obtained with mobile plant onsite</td>
<td>m³</td>
<td>14.00</td>
</tr>
<tr>
<td>RCCR obtained from processing plant outside the site</td>
<td>m³</td>
<td>9.11</td>
</tr>
<tr>
<td>Simple graduated gravel (SGG)</td>
<td>m³</td>
<td>20.41</td>
</tr>
<tr>
<td>Soil of the continuous helical pile</td>
<td>m³</td>
<td>0*</td>
</tr>
</tbody>
</table>

* cost of transportation onsite disconsidered

Table 6 – Costs for the final disposition of wastes in licensed places.

<table>
<thead>
<tr>
<th>Disposition place</th>
<th>Unit</th>
<th>Unitary cost (US$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inert Landfill</td>
<td>m³</td>
<td>47.30</td>
</tr>
<tr>
<td>Processing plant</td>
<td>m³</td>
<td>18.36</td>
</tr>
</tbody>
</table>

* cost of transportation onsite disconsidered

For the calculation of the financial impact of the use of the investigated materials in paving project, two scenarios for reuse of waste were considered:

- Scenario 1: all the brute RRCC is taken to the processing mill for recycling, and the pile soil is deposited in the inert landfill. The layers of paving are executed with natural aggregates;
- Scenario 2: use of mixture of pile soil with RRCC in the regularization of the terrain and the sub-base layer. The base layer is performed with the remaining available RRCC and another portion of natural aggregate (SGG).

In simulations, the volumes were calculated from the paving projects (flexible and semi-rigid), land leveling and gabion wall containment. In all cases, a bulking of 12% was admitted (project value) and apparent specific weight of RRCC and the soil is equal to 14 and 17 KN/m³, respectively.

The estimation of Scenario 1 presented a cost of US$ 2.2 million, while Scenario 2 showed a total cost of US$ 320,498.21. In other words, the use of residues represent a direct saving of about US$ 1.9 million continuing to meet the technical requirements of the paving project, and allowing a very significant reduction of environmental impacts that were not valued.

In the implementation of the paving, the base layer only contained simple graduated gravel (SGG), as an option of the designer. Still, the direct saving obtained was almost the same as in scenario 2.

In summary, demolition residues and the soil from the excavation of helical piles were transformed into the “worksite quarry”. The sustainable construction was cheaper than conventional work, showing the potential of the "green economy".

6 CONCLUSION

The article presents a case of building a shopping center in Recife where the excavation residues of the 4,000 helical foundation piles, in other words, about 25,000 m³ of soil were used in the layers of the paving work (regularization of the terrain and sub-base). Demolition residues were also used from old existing warehouses on the land, which were transformed into recycled aggregates with a mobile plant installed at the worksite.

The demolition residues and soil from the excavation of the helical piles were transformed into the "worksite quarry". The sustainable work was cheaper than the conventional construction, showing the potential of the "green economy".

7 ACKNOWLEDGMENTS

The authors would like to thank the JCPM group for their trust throughout the development of the study, especially Dr. Francisco Bacelar. We also thank the Post-Graduation Program in Civil Engineering of the University of Pernambuco, CAPES (Brazilian funding agency) and FACEPE (funding agency of the state Pernambuco).

8 REFERENCES


Comparative Life Cycle Assessment of Geosynthetics versus Conventional filter layer

Analyse de cycle de vie comparative d’une couche de filtre géotextile et conventionnelle

Frischknecht R., Büsser-Knöpfel S., Itten R.
Treeze Ltd., Kanzleistrasse 4, 8610 Uster, Switzerland

Stucki M.
Zurich University of Applied Sciences, Institute of Natural Resource Sciences, Campus Grüental, 8820 Wädenswil Switzerland

Wallbaum H.
Chalmers University of Technology, Civil and Environmental Engineering, 412 96 Göteborg, Sweden

ABSTRACT: Geosynthetics made from plastics can replace filter layers made of gravel. In this article goal and scope, basic data and the results of a comparative life cycle assessment of gravel and geosynthetics based filter layers are described. The filter layers of a road made of 30 cm gravel and a filter geosynthetics, respectively, form the basis for the comparison. The filter layers have the same technical performance and the same life time of 30 years. The product system includes the supply of the raw materials, the manufacture of the geotextiles and the extraction of mineral resources, the construction of the road filter, its use and its end of life phase. The life cycle assessment reveals that the geosynthetics based filter layer causes lower environmental impacts per square metre. The cumulative greenhouse gas emissions amount to 7.8 kg CO₂-eq (mineral filter) and to 0.81 kg CO₂-eq (geosynthetic filter). The variation of the thickness of the gravel based filter layer confirms the lower environmental impacts of a geosynthetics based filter layer. Environmental impacts of the geosynthetic production are dominated by the raw material provision (plastic granulate) and electricity consumption during manufacturing.

RÉSUMÉ : Les géotextiles sont utilisés pour remplacer le gravier dans les couches de filtres. Cet article contient une description de la définition de l’objectif et du champ d’étude, de l’analyse de l’inventaire et des résultats d’un analyse de cycle de vie comparative d’une couche de filtre géotextile et conventionnelle. La couche de filtre d’une rue est construite avec 30 cm de gravier ou avec une couche géotextile. Les deux couches de filtres ont les mêmes propriétés techniques et la même durée de vie de 30 ans. Les systèmes contiennent la provision des matériaux, la fabrication des filtres géotextiles et l’extraction du gravier, la construction, l’utilisation et l’évacuation de la couche de filtre. L’analyse de cycle de vie démontre qu’un mètre carré d’une couche de filtre géotextile cause moins d’impacts environnementaux qu’un mètre carré d’une couche de filtre gravier. Une couche de filtre gravier entraîne 7,8 kg CO₂-eq, celle de filtre géotextile 0,81 kg CO₂-eq des émissions des gaz à effet de serre par mètre carré. La variance de l’épaisseur de la couche de gravier n’influence pas sur la séquence environnementale des deux couches. La provision des matériaux et l’électricité utilisé dans la fabrication de la couche de filtre géotextile sont des facteurs primordiaux en ce qui concerne les impacts environnementaux de la couche de filtre géotextile.

KEYWORDS: filter layer, geosynthetics, gravel, life cycle assessment, LCA

MOTS-CLÉS : couche de filtre, géotextile, gravier, analyse de cycle de vie, ACV

1 INTRODUCTION

Geosynthetic materials are used in many different applications in civil and underground engineering, such as in road construction, in foundation stabilisation, in landfill construction and in slope retention. In most cases they are used instead of minerals based materials such as concrete, gravel or lime.

Environmental aspects get more and more relevant in the construction sector. That is why the environmental performance of technical solutions in the civil and underground engineering sector gets more and more attention. The European Association for Geosynthetic Manufacturers (E.A.G.M.) commissioned ETH Zürich and Rolf Frischknecht (formerly working at ESU-services Ltd.) to quantify the environmental performance of commonly applied construction materials (such as concrete, cement, lime or gravel) versus geosynthetics (Stucki et al. 2011).

In this article, the results of a comparative Life Cycle Assessment (LCA) of a filter function in road construction are described. The filtration function is either provided by a gravel or a geosynthetic filter layer.

The environmental performance is assessed with eight impact category indicators. These are Cumulative Energy Demand (CED, Frischknecht et al. 2007), Climate Change (Global Warming Potential, GWP 100, Solomon et al. 2007), Photochemical Ozone Formation (Guinée et al. 2001a; b), Particulate Formation (Goedkoop et al. 2009), Acidification (Guinée et al. 2001a; b), Eutrophication (effects of nitrate and phosphate accumulation on aquatic systems, Guinée et al. 2001a; b), Land competition (Guinée et al. 2001a; b), and Water use (indicator developed by the authors). The calculations are performed with the software SimaPro (PRé Consultants 2012).

2 GEOSYNTHETIC FILTER VERSUS MINERAL FILTER

Filters systems in road construction assure that the base soil is retained with unimpeded water flow. In this article, the case of a geosynthetic filter layer is compared to the case of a mineral filter layer.

Polypropylene granules are used as basic material for the geosynthetic layer. They need to be UV stabilised to meet the requirements. The average weight of the polymer is 175 g/m².
The way of the construction of the filter depends on several factors. The basic conditions are shown in Tab. 1 and Fig. 1. The two alternative cases compare the environmental impacts of one square meter of the filter area below the road. The additional excavation needed at the boundary area of the mineral filter is not considered in the comparison.

Table 1. Design criteria of the two filter systems.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Gravel filter</th>
<th>Geosynthetic filter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Filter size</td>
<td>m²</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Filtration geosynthetic</td>
<td>g/m²</td>
<td>0</td>
<td>175</td>
</tr>
<tr>
<td>Gravel</td>
<td>cm</td>
<td>30</td>
<td>0</td>
</tr>
</tbody>
</table>

From these parameters it is calculated that the required thickness D of the mineral filter is 300 mm and the one with the geosynthetic filter layer is 1-2 mm. Fig. 1 shows a cross section of the filter profile as modelled in this LCA. In a sensitivity analysis the thickness of the gravel filter is varied by ±10 cm.

Figure 1. Cross section of the mineral filter (top) and geosynthetic filter system (bottom)

The functional unit in the comparative LCA is the provision of 1 m² of filter with a hydraulic conductivity (k-value) of 0.1 mm/s or more and an equal life time of 30 years.

The difference between the two cases lies in the amount of primary gravel used, the energy consumption that is related to the filter material used (material transportation, excavation etc.), and the use of geosynthetics. Recycled gravel is not considered for the filter system since no onsite recycled gravel is available when building a filter for the first time.

Some important key figures of the construction of the filter systems are summarized in Tab. 2. The information refers to one square meter filter and a life time of 30 years. The figures shown regarding the particulate emissions refer to emissions from mechanical processes (e.g., pouring, compacting of gravel). Direct land use is not included in this LCI because the type of land use under which the filter is being built is not known.

Table 2. Selected key figures describing the two constructions of one square meter of filter

<table>
<thead>
<tr>
<th>Material/Process</th>
<th>Unit</th>
<th>Gravel filter</th>
<th>Geosynthetic filter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>t/m²</td>
<td>0.69</td>
<td>0</td>
</tr>
<tr>
<td>Geosynthetic layer</td>
<td>m³/m²</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Diesel used in building machines</td>
<td>MJ/m²</td>
<td>2.04</td>
<td>1.04</td>
</tr>
<tr>
<td>Transport, lorry</td>
<td>tkm/m²</td>
<td>34.5</td>
<td>0.035</td>
</tr>
<tr>
<td>Transport, freight, rail</td>
<td>tkm/m²</td>
<td>0</td>
<td>0.07</td>
</tr>
<tr>
<td>Particulates, &gt;10 µm</td>
<td>g/m²</td>
<td>4.8</td>
<td>0</td>
</tr>
<tr>
<td>Particulates, &gt;2.5 µm &amp; &lt;10 µm</td>
<td>g/m²</td>
<td>1.3</td>
<td>0</td>
</tr>
</tbody>
</table>

3 MANUFACTURING OF THE GEOSYNTHETIC LAYER

Data about geosynthetic material production are gathered at the numerous companies participating in the project using pre-designed questionnaires. The company specific life cycle inventories are used to establish average life cycle inventories of geosynthetic material.

The data collected include qualitative information of system relevant products and processes from the producer, information from suppliers of the producer (where possible) as well as data from technical reference documents (e.g. related studies, product declarations, etc.). Average LCI are established on the basis of equally weighted averages of the environmental performance of the products manufactured by the participating companies.

The primary source of background inventory data used in this study is the ecoinvent data v2.2 (ecoinvent Centre 2010), which contains inventory data of many basic materials and services.

In total, data from 13 questionnaires concerning the production of geosynthetic layers used in filter applications are included. The quality of the data received is considered to be accurate. The level of detail is balanced in a few cases before modelling an average geosynthetic layer.

Tab. 3 shows important key figures of the production of an average geosynthetic layer.

Table 3. Selected key figures referring to the production of 1 kg geosynthetic layer used in filter applications

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw materials</td>
<td>kg/kg</td>
<td>1.05</td>
</tr>
<tr>
<td>Water</td>
<td>kg/kg</td>
<td>2.16</td>
</tr>
<tr>
<td>Lubricating oil</td>
<td>kg/kg</td>
<td>0.0026</td>
</tr>
<tr>
<td>Electricity</td>
<td>kWh/kg</td>
<td>1.14</td>
</tr>
<tr>
<td>Thermal energy</td>
<td>MJ/kg</td>
<td>1.49</td>
</tr>
<tr>
<td>Fuel for forklifts</td>
<td>MJ/kg</td>
<td>0.09</td>
</tr>
<tr>
<td>Factory building</td>
<td>m³/kg</td>
<td>2.51E-5</td>
</tr>
</tbody>
</table>
4 LIFE CYCLE IMPACT ASSESSMENT

In this section the environmental impacts of 1 square meter filter over the full life cycle are evaluated. The life cycle includes the provision of raw materials as well as the construction and disposal phases.

In Fig. 2 the environmental impacts of the full life cycle of the filter are shown. The environmental impacts of the case with highest environmental impacts (mineral filter 1AS1) are scaled to 100%. The total impacts are subdivided into the sections filter system, raw materials (gravel, geosynthetic layer), building machine (includes construction requirements), transports (of raw materials to construction site) and disposal (includes transports from the construction site to the disposal site and impacts of the disposal of the different materials).

Fig. 2 shows that the average geosynthetic filter system (1A) causes lower environmental impacts compared to the mineral filter system with regard to all indicators investigated. For all indicators the average filter with geosynthetics (1A) causes less than 25% of the environmental impacts of a conventional gravel based filter (1B). The geosynthetic filter (1B) layer causes between 0.2% and 14.3% of the environmental impacts of the mineral filter layer (1A, water use, CED non-renewable). The greenhouse gas emissions caused by the geosynthetic filter (1B) are 10.4% of the greenhouse gas emissions caused by the mineral filter (1A).

The non-renewable cumulative energy demand of the construction and disposal of 1 square meter filter with a life time of 30 years is 131 MJ-eq in case of the mineral filter and 19 MJ-eq in case of the geosynthetic filter. The cumulative greenhouse gas emissions amount to 7.8 kg CO₂-eq (mineral filter) and to 0.81 kg CO₂-eq (geosynthetic filter). The main source of difference is the use and transportation of gravel. Hence, the use of geosynthetics may contribute to reduced environmental impacts of filter layers, because it substitutes the use of gravel.

4.1 Sensitivity analysis

In a sensitivity analysis, it is analysed how the results of the gravel filter layer change, if the thickness of the mineral filter is increased by 10 cm to a total thickness of 40 cm (1AS1) or if the thickness of the mineral filter is decreased by 10 cm to a total thickness of 20 cm (1AS2).

Fig. 2 reveals that, if a thicker filter layer is constructed, the environmental impacts of the gravel based filter increase by 33% and if a thinner filter layer is constructed, the environmental impacts of the gravel based filter are decreased by 33%. Nevertheless, in all cases the environmental impacts of a filter with geosynthetics (1B) are considerably lower than the environmental impacts of a gravel based filter (1A, 1AS1, 1AS2).

4.2 Contribution Analysis Geosynthetic Production

In this section the environmental impacts of 1 kg geosynthetic layer are evaluated. The life cycle includes the provision and use of raw materials, working materials, energy carriers, infrastructure and disposal processes. The category geosynthetic in Fig. 3 comprises the direct burdens of the geosynthetic production. This includes land occupied by the factory producing the geosynthetic as well as process emissions (e.g. NMVOC, particulate and COD emissions) from the production process but not emissions from electricity and fuel combustion.

The environmental impacts of the geosynthetic filter are shown in Fig. 3. The cumulative greenhouse gas emissions amount to 3.2 kg CO₂-eq per kg.

Environmental impacts are mostly dominated by the raw material provision and electricity consumption. Raw material includes plastics, chemicals, printing colours, and other additives. Plastic raw materials are responsible for between 4% (land competition) and 80% (CED non-renewable) of the overall impacts, printing colours, chemical and additives for between 2% and 10%.

Country-specific electricity mixes are modelled for each company and thus impacts of electricity consumption depend not only on the amount of electricity needed but also on its mix. The high share of electricity in CED renewable can be explained by the use of hydroelectric power plants in the electricity mixes of several factories.

Heating energy and fuel consumption for forklifts are of minor importance. With regard to land competition the geosynthetic production plays an important role (92% of
overall impacts). The impacts are dominated by the direct land use, i.e. land which is occupied by the manufacturer in which the geosynthetic is produced. Indirect land use, i.e. land occupation stemming from upstream processes, is significantly lower because no land occupation is reported in the inventories of plastic feedstock and no land intensive products such as wood are used in considerable amounts.

Water consumption (tap water, deionised water, decarbonised water) is included in the working materials. As a consequence, this category bears about 15% of the total amount of water used.

5 DISCUSSION AND CONCLUSION

A filter using a geosynthetic layer causes lower environmental impacts compared to a conventional gravel based filter layer with regard to all impact category indicators investigated. If 30 cm of gravel are saved, the specific climate change impact of the construction of 1 square meter filter using geosynthetics is about 7 kg CO₂-eq lower compared to the impacts from the construction of an equivalent gravel based filter.

The difference is considerable for all indicators (more than 85 %) and reliable. The difference in the environmental impacts arises mainly because the applied geosynthetic substitutes gravel, which causes considerably higher impacts when extracted and transported to the place of use. At least a layer of 8 cm of gravel must be replaced by geosynthetics used as a filter in order to cause the same or lower environmental impacts regarding all indicators.

The environmental impacts of the gravel based filter are significantly reduced, when constructing smaller filters (20 cm instead of 30 cm). Nevertheless, the sequence of the two cases does not change and the difference is still significant between the sensitivity cases of the mineral filter and the geosynthetic filter.

6 REFERENCES


Figure 3. Environmental impacts of the life cycle of 1 kg geosynthetic layer. Geosynthetic includes direct burdens of the geosynthetic production. Raw materials include plastic, extrusion if necessary, and additives, working materials include water (tap and deionised) and lubricating oil, other energy includes thermal energy and fuels, infrastructure covers the construction of the production plant and disposal comprises wastewater treatment and disposal of different types of waste.

3206
La réutilisation des fondations existantes dans les projets de réhabilitation de constructions anciennes

Reuse of existing foundations for the rehabilitation of old buildings

Guilloux A., Le Bissonnais H., Saussac L.*, Perini T.
Terrasol, Paris, France
(*à la date de publication : Geos, Paris, France)

RÉSUMÉ : Lors de la réhabilitation de bâtiments existants, il est bien évidemment souhaitable de conserver dans la mesure du possible les fondations existantes de l’ancienne structure, quitte à les renforcer ou à créer des fondations supplémentaires si celles qui existent ne permettent pas de garantir la bonne tenue de la structure nouvelle. L’expérience, et notamment les trois exemples décrits dans cet article, montre que cette préoccupation conduit à développer une démarche de conception géotechnique originale : il convient de bien connaître l’état des fondations existantes, de bien évaluer les variations de charges à tous les stades (depuis l’ancienne construction jusqu’à la nouvelle, en passant par les phases de chantier), de décider quelles sont les fondations à renforcer, puis d’étudier des techniques de renforcement adaptées aux conditions de chantier, dans des espaces souvent restreints et sans créer de désordres sur les parties d’ouvrages conservées, et enfin de s’assurer de la compatibilité des déformations et reports de charges.

ABSTRACT: For projects of rehabilitation of an existing building, it is obviously preferable to use as much as possible the existing foundations, including by reinforcing them or by creating additional ones when the existing ones cannot guarantee the safety of the new structure. Based on three case histories described in this paper, it is shown that a “new” geotechnical approach is required, including: deep knowledge of the state of the existing foundations, careful analysis of the load variations (from the old buildings to the new structure, together with the temporary stages), selection of the foundation to be reinforced, choice of reinforcement techniques, often to be implemented in narrow spaces and without any disorders on the structure left in place, and finally detailed analysis of load transfers and deformations compatibility.

KEYWORDS: Réhabilitation, renforcement, transferts de charges, compatibilité des déformations. Rehabilitation, reinforcement, load transfer, deformation compatibility.

1 PRESENTATION

Les enjeux du Développement Durable sont à l’évidence très présents dans tous les projets de réhabilitation des constructions, qu’il s’agisse de bâtiments ou d’ouvrages d’art. Du point de vue du géotechnicien, il s’agit avant tout de rechercher une réutilisation maximale des parties de fondations existantes, afin de limiter les travaux souvent lourds de démolition et de reconstruction, avec consommation de matériaux « neufs ». Ainsi, du point de vue du géotechnicien, cette préoccupation de Développement Durable conduit à se poser un certain nombre de questions, qui sortent quelque peu des problématiques géotechniques habituelles :
- pour réutiliser les fondations existantes, il est essentiel de bien les connaître : quels sont les moyens d’investigation et de contrôle utilisables pour s’assurer de la géométrie de pieux par exemple, de la qualité du béton etc. alors que les plans de construction ne sont pas toujours disponibles ?
- lorsque le projet de réhabilitation conduit à une augmentation des descentes de charges, il faut soit renforcer les fondations existantes pour en augmenter la capacité portante soit en créer de nouvelles pour repandre les charges additionnelles : comment alors évaluer la redistribution des charges, en intégrant les phasages de construction initiale, démolition et reconstruction ?
- même lorsque le nouveau projet ne conduit pas à une réelle modification des charges, on reste parfois confronté à une question d’évolution de la réglementation, qui est souvent devenu plus contraignante : ainsi, en l’absence de charges complémentaires, faut-il renforcer des fondations d’un ouvrage qui s’est toujours bien comporté, uniquement parce qu’il n’est plus conforme à la réglementation actuelle ?
- enfin, dans le cas où des renforcements de fondations existantes ou de nouvelles fondations sont rendus nécessaires, quelles sont les techniques de réalisation permettant à la fois d’intervenir dans des espaces souvent restreints et de minimiser les démolitions mêmes partielles sur l’existant ?

Ces différentes questions sont illustrées par trois projets en région parisienne, dont on présente les résultats des reconnaissances de l’existant, les études de conception des fondations et les méthodes de réalisation :
- La réhabilitation des entrepôts Calberson, boulevard Mac-Donald à Paris : la construction de nouveaux étages de superstructures et la détection d’anomalies géologiques en base des pieux existants ont conduit à la fois à renforcer les pieux existants par un traitement en jet-grouting sous leur base et à mettre en œuvre des fondations nouvelles par micropieux ;
- La réhabilitation du secteur Est de l’Université de Jussieu (Paris Vème) : le projet de réhabilitation ne conduisait en général pas à une augmentation des charges, mais les auscultations des pieux existants ont révélé des défauts localisés, en particulier en termes de longueur, qu’il a fallu traiter soit en reprenant une partie des charges par des micropieux nouveaux, soit par injections de terrain sous les fondations existantes. Un plot d’essai de traitement de sol a été réalisé avec essais de chargement statique axial sur des pieux existants pour quantifier l’effet du traitement ;
- La réhabilitation d’un ancien centre de tri postal à Pantin : sa transformation en vue de son réaménagement en Data Center a conduit à une augmentation importante des charges sur les pieux existants. Après une rédéfinition des paramètres de sol pour s’approcher au mieux des conditions
de tassement observé depuis la construction de l’immeuble (phase calage), les pieux ont été traités toute hauteur par 2 colonnes de jet-grouting.

2 ENTREPÔTS MAC-DONALD

2.1 La phase de conception

Ce projet consiste à réhabiliter d’anciens entrepôts Calberson, construits dans les années 1970 sur une emprise de 600 x 60 m² située boulevard Mac-Donald à Paris, pour les transformer en bureaux, logements et équipements sociaux, conduisant à surélever la structure existante par la construction de nouveaux étages de superstructures. Le bâtiment existant est fondé sur pieux d’environ 10 m de profondeur et 1,0 à 1,6 m de diamètre.

L’ouvrage, comporte des sous-sols sur 6 m de profondeur environ, repose sur 10 m de calcaires de Saint-Ouen surmontant les sables de Beauchamp.

Figure 1 : coupe type du projet

Les études préalables se sont attachées à faire un diagnostic des pieux existants et à conduire des reconnaissances de terrain, qui ont révélé des anomalies géologiques en base des pieux existants.

Il était donc nécessaire :
- d’une part de renforcer les pieux existants pour optimiser leur portance, c’est-à-dire les faire travailler au maximum de la contrainte admissible dans le béton, fixée à 4,8 MPa après carottages et essais de compression sur le béton, et en respectant les tassements admissibles. Après examen de différentes solutions, la technique retenue a consisté à prolonger les pieux par des colonnes de jet-grouting sous leur pointe ;
- d’autre part de prévoir des fondations nouvelles par micropieux pour des structures nouvelles descendues en infrastructures, les noyaux.

En phase études, des essais de chargement statique des pieux existants ont également permis de définir des paramètres optimisés pour l’effort en pointe et le frottement latéral, mais également de mesurer en vraie grandeur la raideur des pieux, paramètre essentiel pour vérifier la distribution entre les pieux existants et les micropieux nouveaux des charges additionnelles, ainsi que la compatibilité des déformations.

2.2 Suivi du chantier de renforcement

Pour cet ouvrage, dont les pieux existants présentaient des défauts de portance vis-à-vis des futures charges, plus importantes que celles connues antérieurement, la solution retenue de renforcement des fondations consistait à prolonger la base des pieux par une colonne de jet-grouting de 1,3 m de diamètre et 3 m de longueur en général (deux colonnes pour les pieux de 1,6 m).

Sur la base de reconnaissances géotechniques approfondies, des essais de chargement des pieux existants et des nouvelles descentes de charges, nous avons pu déterminer les pieux nécessitant un renforcement et dimensionner ce dernier. C’est au total environ les ¾ des 500 pieux qui ont dû être renforcés.

Le chantier de jet-grouting a fait l’objet d’un suivi avec contrôle renforcé, pour la réalisation des forages inclinés traversant les pieux existants et permettant de réaliser les colonnes de jet-grouting sous leur base. En outre, s’agissant de travaux de reprise en sous-œuvre d’un ouvrage existant, il a fallu s’assurer-que les travaux de jet ne conduisaient pas à des désordres sur les structures conservées, notamment vérification des soulèvements lors des phases d’injection (35 à 40 MPa) puis des tassements avant que la colonne ne fasse prise. De ce point de vue le chantier s’est déroulé sans désordres majeurs, avec des mouvements des poteaux ne dépassant pas quelques millimètres, ce qui a confirmé que la technique du jet-grouting, bien contrôlée et avec un phasage adapté, permettait d’intervenir en sous-œuvre sans créer de mouvements significatifs.

2.3 Dimensionnement des fondations nouvelles

Pour les parties de structures nouvelles (noyaux), nécessitant leurs propres fondations, compte tenu du contexte et de l’espace de travail contraint, c’est une solution sur micropieux qui a été retenue.

Le dimensionnement en termes de capacité portante de ces micropieux a conduit à prévoir des micropieux de longueur réduite, à 8 m en général après essais de chargement. Mais plus que la capacité portante, c’est en fait sur les redistributions des charges entre les pieux existants, de grande section, et les micropieux nouveaux, a priori plus souples, que l’attention a été portée. Cette problématique a nécessité une approche en déformations, pour s’assurer de la compatibilité des tassements sous charges entre les anciennes et nouvelles fondations.

Ainsi, après les essais de chargement des pieux existants qui avaient permis de définir les paramètres de raideur, des essais de chargement de micropieux ont été également réalisés, et il a même été réalisé un essai de chargement en vraie grandeur sur un groupe de 4 micropieux, pour mesurer l’incidence des effets de groupe sur la raideur. Un dispositif de chargement a été mis en place sur le site, dans l’intention d’évaluer un potentiel effet de groupe comportant quatre vérins, montés en parallèle et permettant d’appliquer un effort de 1000 kN sur chaque micropieux. L’essai a été concluant et n’a pas mis en évidence d’interaction significative entre les micropieux.

Puis les calculs ont été conduits avec une approche de plaque sur appuis élastiques, pour chacun des noyaux, les raideurs des appuis ayant été définies sur la base des essais de chargement. La Figure 2 montre un exemple de résultat pour l’un des noyaux avec charges concentrées, avec des tassements compris entre 1 et 4,5 mm, tout à fait compatibles avec ceux des pieux voisins.

C’est au total plus de 2700 micropieux qui ont été réalisés en complément du renforcement des pieux par Jet Grouting, et qui ont fait l’objet d’un suivi dans le cadre d’une mission géotechnique G4 selon la norme NF P94-500.
3 RÉHABILITATION DU SECTEUR EST DU CAMPUS DE JUSSEU

Pour ce projet, la problématique était assez différente, dans la mesure où la construction nouvelle n’apportait pas de suppléments de charges sur les fondations par rapport à la construction ancienne. Les bâtiments datant des années 1970 se sont avérés fondés sur des pieux de 0.5 à 0.8 m de diamètre et d’environ 8 à 9 m de profondeur.

La Figure 3 montre que les pieux traversent 4 à 7 m de remblais et alluvions récentes limoneuses, puis 3 à 4 m d’alluvions anciennes sablo-graveleuses, de façon à venir « s’ancrer » sur l’horizon sous-jacent de calcaire grossier. Mais en pratique cet ancrage n’est pas toujours assuré, et la portance des pieux est donc variable.

Ainsi dans le cadre de la réhabilitation de ce bâtiment, il a fallu « remettre à niveau » les niveaux de sécurité des fondations des différents appuis, ce qui a conduit à développer une méthodologie de projet suivant les différentes phases suivantes :

1. Des investigations des pieux existants, par exploitation des données d’archives et des puits de reconnaissance pour en déterminer le diamètre, complétées par méthodes géophysiques (impédance mécanique et sismique parallèle) pour en déterminer la longueur. Ces investigations ont montré que les pieux descendaient « plus ou moins » jusqu’au calcaire grossier, mais pas toujours avec un ancrage suffisant.

2. Des reconnaissances géotechniques et une réévaluation de la portance des pieux, avec les méthodes et moyens « modernes », afin de déterminer quels étaient les pieux à renforcer : essais pressiométriques de qualité, plots d’essais de traitement de terrain par injection sous la pointe, essais de chargements statiques de pieux avant et après injection, puis enfin calculs de pieux avec une approche en déformations et pas seulement en capacité portante, et enfin comparaison entre les résultats des modélisations et ceux des essais en vraie grandeur.

Ces premières comparaisons montraient en général que le comportement réel des pieux avant injection était beaucoup plus favorable que prévu, ce qui a conduit à faire de nombreuses retro-analyses pour finalement conclure à la nécessité de majorer les hypothèses géotechniques de frottement latéral par rapport aux règles usuelles.

Par ailleurs lors des essais de chargement après injection en pointe de pieu, l’un des pieux d’essai s’est rompu, correspondant à un dépassement de la contrainte admissible du béton du pieu, tandis que l’autre a montré un comportement largement amélioré par rapport à celui avant injection.

Cet ensemble d’essais et modélisations a permis de préciser quels étaient les pieux à renforcer, à valider la méthode de renforcement par injection sous la pointe, et à définir des hypothèses de calculs réalistes pour le projet final.

3. Une revue des différentes méthodes de renforcement des pieux existants, micropieux, jet-groutting, injection en masse. Une analyse des avantages et inconvénients de chaque procédé a été conduite selon une approche multicritère, intégrant les conditions de mise œuvre, les risques de désordres sur la structure existante lors de leur mise en œuvre, la fiabilité du résultat, et bien sûr les coûts et délais.

Ces approches ont conduit à retenir finalement la solution de traitement par injection en masse (Figure 3), validée par des plots d’essai qui ont été rigoureusement suivis, et complétée par des essais de chargement des pieux avant et après injection.

L’ensemble de la démarche, avec notamment des approches du comportement des pieux en déformations, a permis de réduire au stade projet d’environ 30 % le nombre total de pieux à traiter (environ 480). Les études détaillées d’exécution devraient encore conduire à une réduction très importante de fondations traitées.

4 RÉHABILITATION EN DATA CENTER D’UN ANCIEN CENTRE DE TRI POSTAL À PANTIN

Il s’agit toujours de la même problématique que pour les projets précédents : réhabiliter un bâtiment existant R + 5 avec un niveau de sous-sol, construit en 1973 et d’une emprise de 160 x 50 m, fondé sur 250 pieux environ, et qui devait être transformé en Data Center, avec une forte augmentation des descentes de charges.

Le contexte géologique comporte environ 6 m de remblais et limons, surmontant 3 à 4 m de marnes infra-gypseuses puis le calcaire de Saint-Ouen, les pieux étant ancrés dans l’un ou l’autre de ces deux derniers horizons.

Comme pour les cas précédents, la démarche suivie a conduit à identifier la géométrie des pieux existants et leur capacité portante. En l’absence de données d’archives, il a été procédé à des investigations des pieux existants par forages et méthodes géophysiques, qui ont permis d’établir la base de toute l’approche de conception et réutilisation des pieux existants, à savoir leur géométrie : il s’avère que leur diamètre est de 1.2 m et que leur profondeur est variable entre 9 et 11 m environ, c’est-à-dire que leur pointe se situe au voisinage de l’interface Calcaire de Saint-Ouen / sables de Beauchamp.

La vérification de la résistance à la compression simple Rc du béton des pieux a été faite par prélèvement de carottes de béton et essais sur échantillons. On a ainsi pu vérifier que le béton des pieux était de très bonne qualité avec des valeurs moyennes de Rc de l’ordre de 40 MPa.

Enfin les analyses de la capacité portante des pieux existants ont été faites à partir de nouvelles reconnaissances géotechniques : il s’est avéré qu’une proportion importante des pieux ne présentait pas la sécurité réglementaire. En effet les premiers calculs montraient que, selon les normes actuelles, et avec les hypothèses géotechniques déduites des essais, le
bâtiment existant n’aurait pas du tenir pendant 40 ans sans déformations significatives, ce qui n’était à l’évidence pas le cas.

Nous avons donc été conduits à développer plusieurs approches en rétro-analyse (comparaisons entre les charges appliquées au cours de la vie de l’ouvrage et les capacités portantes théoriques, estimation du tassement réel subi par la construction...). Ces approches ont permis de réévaluer les propriétés géotechniques des terrains, et en pratique de les majorer par rapport aux résultats déduits des essais, ce qui a conduit à limiter le nombre de pieux existants à renforcer de façon lourde : la proportion de pieux qu’il a ainsi été décidé de renforcer est passée de 90% à 40 % environ.

Enfin nous avons examiné différentes conceptions de fondations pour la nouvelle structure, incluant soit des renforcement de pieux existants soit des fondations nouvelles, lorsqu’il avait un déficit de portance. Toutes ces solutions ont été étudiées en s’assurant de la compatibilité des déformations entre les différents types de fondations, et de l’incidence des phases provisoires (notamment la perte de portance provisoire lors de réalisation de jet-grouting, avant prise du coulis).

Le projet de base prévoyait de réaliser 4 à 6 micropieux autour de chacun des pieux présentant une portance insuffisante, avec une semelle de répartition liée aux pieux. Ces micropieux étaient destinés à reprendre tout ou partie de la différence de charge ramenée par la superstructure aux poteaux supportés par la fondation profonde. Cette solution bien que classique présentait néanmoins le désavantage :
- de multiplier les forages ;
- de nécessiter une semelle de forte épaisseur avec un ferrailage lourd afin de répartir correctement les charges et pouvoir ainsi solliciter les micropieux de rigidité bien moindre que celle des pieux.

Par ailleurs, elle ne permettait pas de tenir du planning imposé et dépassait le budget prévu initialement.

Ainsi, forts de l’expérience du chantier des entrepôts Calberson, Boulevard Mac-Donald à Paris, il a été décidé d’examiner une solution de confortement des pieux par jet-grouting. Après analyse de diverses solutions (solution mixte « micropieux + traitement de la pointe des pieux » - et solution de simple traitement en pointe), l’entreprise générale en charge de l’opération a opté pour la réalisation de deux colonnes de jet de renforcement de part et d’autres des fondations et de diamètre variable (entre 80 cm et 120 cm selon la profondeur) en prenant soin de rester au-dessus de la pointe des pieux existants selon le schéma de la Figure 4.

**Figure 4 : principe du confortement par jet-grouting**

Cette solution permettait de conserver la résistance en pointe pendant les travaux. Seule une perte de frottement a été prise en compte dans les calculs pendant la phase de jet-grouting proprement dite.

Le calcul de la fondation définitive a été mené en considérant que l’ensemble « pieu + jet » formait un monolithe permettant de recalculer ainsi le pieu renforcé avec une surface en pointe et un périmètre frottant majoré tout en vérifiant la part de charge passant dans le jet et la part de charge passant dans le pieu pour chaque section, à partir des raideurs relatives de chacun des matériaux.

La réussite de cette solution tenait principalement dans le bon accrochage entre pieux et colonnes de jet. Le planning ne permettant pas de mettre en œuvre un essai de chargement préalable, des colonnes d’essais de jet ont été néanmoins réalisées sur des pieux abandonnés puis dégagées sur 3,00 m de hauteur pour un constat visuel du contact pieu-jet et du diamètre des colonnes, qui s’est avéré tout à fait satisfaisant. Le frottement pieu-jet adopté était au final de qu, = 500 kPa.

Cette solution novatrice a permis de réutiliser les fondations existantes après traitement en vérifiant à la fois la portance et la compatibilité des tassements avec les impératifs de déplacement imposés par la structure.

5 SYNTHÈSE ET MÉTHODOLOGIE DE CONCEPTION

Le retour d’expérience de ces trois projets met en évidence une approche géotechnique spécifique, qui doit impérativement passer par les différents stades suivants :
- État des lieux et diagnostic des fondations existantes : type, géométrie, résistance intrinsèque, à partir de l’exploitation de données d’archives, d’investigations géophysiques (non destructives) et de reconnaissances destructives. Cette analyse doit intégrer également un diagnostic de l’état de l’ouvrage pour apprécier s’il a subi des dommages passés ;
- Analyse des desentes de charges sur les fondations, depuis l’état ancien jusqu’à la construction future, sans oublier les phases provisoires de chantier de réhabilitation ;
- Analyses géotechniques de la portance admissible des fondations existantes (intégrant les réglementations en vigueur), et donc des éventuels déficits de charge à reprendre pour l’état futur, ainsi que des déformations qu’elles ont pu subir par le passé ;
- Étude de différents scénarios pour la reprise de ces déficits, en envisageant plusieurs solutions combinant les fondations existantes, éventuellement à renforcer, et des fondations nouvelles à créer (pieux, micropieux …) ; des plots d’essais en vraie grandeur sont souvent nécessaires à ce stade pour valider le choix des solutions et préciser le comportement en terme de raideur notamment ;
- Choix final des solutions, avec dimensionnement détaillé des différents systèmes : fondations anciennes conservées, renforcements et fondations nouvelles. Ce choix doit impérativement prendre en considération les conditions pratiques de mise en œuvre (travail en sous sols notamment), les compatibilités de déformations entre ces différents systèmes et l’histoire du chargement, y compris les phases intermédiaires de chantier.
- Enfin un suivi géotechnique rigoureux du chantier, avec en particulier une attention toute particulière apportée aux mesures de déplacements de la structure conservée pendant l’exécution des fondations nouvelles et les traitements de terrain, afin de s’assurer que les travaux de fondations n’engendrent pas de désordres sur la structure existante.
Modern geotechnical construction methods for important infrastructure buildings

Méthodes de construction modernes des ouvrages géotechniques dans les grands projets d’infrastructures

Heerten G.
NAUE GmbH & Co. KG, Gewerbestraße 2, 32339 Espelkamp, Germany

Vollmert L.
BBG Bauberatung Geokunststoffe GmbH & Co. KG, Gewerbestraße 2, 32339 Espelkamp, Germany.

Herold A.
IBH – Herold & Partner Ingenieure, Humboldtstr. 58 b, 99425 Weimar, Germany

Thompson, Dupond J.
Same affiliation

Alcazar G.
Other affiliation

ABSTRACT: Efficient traffic routes form the basis for a brisk economic trade and a close economic and social relationship of the European countries. Thus, the massive development of the national and international traffic routes is correspondingly important. Especially in densely populated areas the routes have to be designed and constructed in such a way that emissions remain compatible for the direct surroundings. In this case the modern construction management requires solutions which are well-engineered and which can be carried out economically. The visible parts of the constructions should correspondingly suit the landscape in an ideal way. Thus, geosynthetics are used for the static design of high dams and noise protection barriers, in case of a foundation on extremely weak subsoils and for ground water protection measures. Large-scale projects actually realized as well as constructions monitored since years stand representatively for the possibilities and the importance which geosynthetics provide for the use in important infrastructural projects.

RÉSUMÉ : L’efficacité d’un réseau routier est la base d’échanges économiques rapides et de relations étroites en matière économique et sociale entre les pays européens. Ainsi, le développement massif des infrastructures routières nationales et internationales est par conséquent important. En particulier dans les zones fortement peuplées, les routes doivent être conçues et construites de sorte que les nuisances restent acceptables par le milieu environnant. Dans ce cas, la programmation de construction moderne exige des solutions qui soient bien conçues et réalisables de façon économique. Les parties visibles d’ouvrages devraient, également, s’adapter au paysage environnant de manière la plus naturelle possible. Pour ce faire, les géosynthétiques entrent dans la conception statique de grands ouvrages de retenue et des barrières anti-bruit, dans la construction de remblais sur des sols d’extrême faible portance et dans les ouvrages de protection des eaux souterraines. De grands projets aujourd’hui achevés ainsi que des ouvrages sous monitoring depuis plusieurs années sont représentatifs de l’étendue des possibilités qu’offrent les géosynthétiques dans les grands projets d’infrastructure.

KEYWORDS: geosynthetic reinforced soil, geogrid, design, regulation, Eurocode 7, infrastructure, costs, carbon footprint

1 INTRODUCTION

The reconstruction of existing and the design of new roads has to accept the geographic situation and has to take political and social based decisions into consideration. So not only bridges and tunnels, but also dams and cuttings are required in areas where the subsoil is not well suited and noise barriers have to be built using local soils.

Geosynthetics as modern construction material are relatively new in terms of understanding and are still not part of the standard education. Problems of understanding synthetics are often linked to the fact, that synthetics behave different compared to well-known materials as concrete and are not ideal elastic as e.g. steel. On the other hand, synthetics and wood, one of the eldest construction material ever used by civil engineers, are both polymers and comparable in many aspects. Additionally, synthetics are already used in many applications where concrete is not suitable and has to be protected against chemicals e.g. in pipelines.

It will be shown that geosynthetics have already become an important construction material in infrastructure applications and allow for modern and economic constructions, saving costs by lean structures, combining local soils, concrete and steel.

The significant reduction of the carbon footprint in many cases seems to be a future topic, but it has started right now. Several studies have been worked out actually, comparing classical solutions and structures using geosynthetics, as e.g. by Corney et al. (2009). Not only the economic effect becomes clear in this study by requiring less energy, but also the reduction of CO₂ during the whole process. Egloffstein (2009) shows an example for an executed steep slope with an inclination of 60° for a road in a hilly region of Germany, using geosynthetics (Fig. 1). This construction had been planned with conventional cantilever walls, but had to be redesigned using a fully greened facing due to political reasons. Nevertheless, the budget for the construction has been reduced by factor 1.6 (Wessling & Vollmert). The carbon footprint - not being a topic in 2004 - has been calculated for the well documented wall respectively slope by Egloffstein (2009) as given in Fig. 2. The result 6:1 stands for its own.

2 EUROPEAN REGULATIONS – ACTUAL STATUS AND LINK TO GEOSYNTHETICS

Actually, Eurocode 7 (EC7) has become the decisive regulation in geotechnical works for all European countries linked to EU law, but has not become well established up to now. All
national standards have to be reduced to additional regulations, not being in any conflict to EC7. For Germany, the actual status for the normative range of regulations is given in Fig. 3. The

![Figure 1. Steep slope in Idstein, under construction 2001 (left) and in service 2004 (right)](image)

![Figure 2. Carbon footprint comparison, example Idstein; cantilever wall (left) and executed geogrid reinforced steep slope (right)](image)

current national standard DIN 1054 will be used as supplementary rule, but being reduced to fragments. The three parts of the rules will be combined to a normative handbook with blended text for practice aspects. DIN 1054:2010 refers to recommendations published by the German Geotechnical Society (DGGT).

The latest recommendation EBGEO dealing with geosynthetics is directly linked to DIN 1054:2010 and therefore also according to EC7; special hints are given how to use EBGEO in the EC7 concept. This recommendation is available as English translation (EBGEO, 2011). Substantial design instructions which meet the requirements of numerous practical applications are offered. These reflect the state-of-the-art considering the proven scientific findings.

In Great Britain, the comparable recommendation is called BS 8006. BS 8006 can also be read as a supplementary annex to the European Regulation and also hints are given how to use it in the EC7 context. So actually two finalized recommendations dealing with the use of geosynthetics are available and allow for design in accordance to EC7.

As it is well known, that EC7 allows for three different design approaches DA1, DA2 and DA3, used in different countries of the EU, it is of general interest whether the British and German recommendations lead to comparable results. Fig. 4 gives an example for a typical steep slope reinforced by geogrids. In both calculations the full set of partial factors as given by EC7 added by specific partial factors for geosynthetics as given by EBGEO and BS 8006 in addition to EC7 and the supplementary national regulations are used (Klompmaker & Werth, 2011).

It has to be stated here, that EC7 gives no full set of partial factors for the usage of geosynthetics. Therefore the authors strongly recommend the use of EBGEO in combination with DA3 and BS 8006 in combination with DA1 as long as no national regulations or recommendations exist in the other countries of the EU. Disregarding additional partial factors as given by EBGEO or BS 8006 can lead to significant excess of the ultimate and serviceability limit state.

For EBGEO and BS 8006 it can be stated that the outcome indicates that the codes are validated against practice, current scientific experience and result in well comparable utilization ratio.

3 LONG-TERM EXPERIENCE ON GEOSYNTHETICS REINFORCED WALLS AND CONCLUSIONS FOR DESIGN

Reinforced soil as one of the eldest construction techniques already used 1400ac in Iraq (Tower of Babylonia) has become popular with the availability of high strength wovens and the today’s use of geogrids with perfect geosynthetic-soil-interaction properties. Herold (2007) documented seven high loaded structures that are under continuous supervision. The documented strain within the geosynthetic reinforcement is measured within the range of 0.05 % … 0.4 %. Various measurements from literature (Pachomow et al., 2007) show
also low strains less than 1.5%. The strain therefore is less as expected by Ultimate Limit State Design (ULS), but in accordance to scientific approaches and actual understanding of compound material (Heerten et al., 2009).

Ruiken et al. (2010) managed to visualize the shear rotation of granular material in the front of a reinforced wall. The required deformation of the facing is very low and is depending on the degree of reinforcement respectively the vertical layer distance of the reinforcement. Secondary shear planes develop during deformation, showing significant differences as to be expected by active earth pressure theory. EBGEO has already used these findings on the basis of the publication by Pachomow (2007), allowing for a reduced earth pressure on the facing of a reinforced earth wall.

From back analysis of the constructions documented by Herold (2007), taking the actual design codes into consideration, general conclusions can be drawn and recommendations for further design are given concerning the expected deformation of a construction, see Table 1.

4 WORKED EXAMPLES FOR INFRASTRUCTURE DESIGN USING HIGH STRENGTH GEOGRIDS

The high ductility of reinforced soil structures and its economic benefit have raised a certain increase of usage in the last decade in Europe. Vollmert et al. (2010) document a structure, using approx. 6 ha of geosynthetics for noise barrier walls and embankments on weak subsoil in the Netherlands. Fig. 5 gives an partial overview of the construction with a total length of 2 km. The costs of the geosynthetics used are less than 1% of the total budget.

The visible and to environmental influences exposed part of the construction is the facing. Several facing types as gabions, wrap-around method as well as concrete blocks and panels are commonly used.
Table 1. Deformations to be expected for reinforced walls (according to Herold, 2007)

<table>
<thead>
<tr>
<th>Deformation Type</th>
<th>Total</th>
<th>Post-construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. horizontal</td>
<td>$h_{\text{total}} = 0.005 \ldots 0.01 \times H$</td>
<td>$h_{\text{post}} = 0.15 \ldots 0.3 \times h_{\text{total}} = 0.00075 \ldots 0.003 \times H$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. vertical</td>
<td>$v_{\text{total}} = 0.01 \ldots 0.02 \times H$</td>
<td>$v_{\text{post}} = 0.15 \ldots 0.4 \times v_{\text{total}} = 0.0015 \ldots 0.008 \times H$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- $H$: max. height of construction;
- $h$: horizontal deformation;
- $v$: vertical deformation;
- Total: all deformations within the construction;
- To be added by subsoil settlements;
- Lower boarder for walls without surcharge;
- Upper boarder with surcharge.

The long-term serviceability depends therefore on usual and well-known construction materials as steel and concrete. Nevertheless, the stability of the construction shall be ensured even in case of partial damage, e.g. following accidents, fire or vandalism or the construction has to be designed properly and protected by crash barriers.

It is the unique characteristic of reinforced walls that the internal stability can nearly never collapse by stress on the facing. Providing facing systems that can be repaired easily therefore give additional benefit for the maintenance of the structures.

An intelligent and lean facing system, combining the aspects of economic design and easy to maintain, has been realised at the project Wien (Fig. 6), differentiating the static support system on the soil side and the galvanised steel grid on the facing side. The gap is filled with natural stone in analogy to gabions. In case of a damage of the outer steel grid, the structure remains stable and just the filling and outer steel grids have to be reconstructed.

5 FINAL REMARK

Extending and reconstructing the infrastructure within the European countries as well as the international routes, structures for noise barriers and embankment foundations are required, fulfilling the economic requirements. Geosynthetics, namely geogrids, allow to use local soils and can be combined with steel elements as well as concrete and wooden elements.

The experience and scientific findings gained in the last two decades ensure a sophisticated level of engineering as documented in the design codes EBGEO and BS 8006, both in accordance with Eurocode 7. Nevertheless it should be noted that Eurocode 7 alone does not provide full sets of partial factors, so additional factors provided by EBGEO and BS 8006 are strongly recommended to be used depending on the specific national used design approaches.

6 REFERENCES


Sustainable Management of Contaminated Sediments

Gestion durable des sédiments contaminés

Holm G.
Swedish Geotechnical Institute, Linköping, Sweden
Lundberg K., Svedberg B.
Luleå University of Technology, Luleå, Sweden

ABSTRACT: Increasing sea transport volumes require expansion of ports and due to longer, wider and more deep-draught ships considerable dredging of sediments often contaminated with heavy metals and organic contaminants have to be implemented. Handling options have to be identified in each actual case and the choice of option should be based on a sustainability approach, considering economy, environment and social aspects. In the EU-funded project “Sustainable Management of Contaminated Sediments in the Baltic Sea” (SMOCS; www.smocs.eu) within the Baltic Sea Region Programme 2007-2013, tools for assessment and decision making are developed, i.e. Life Cycle Analysis (LCA), Risk Assessment (RA) and Multi Criteria Decision Analysis (MCDA). To develop the tools a series of case studies have been performed comparing different handling options such as land disposal, sea disposal, confined disposal and beneficial use in port constructions utilizing the stabilization/solidification technology.

RÉSUMÉ : L'augmentation des volumes de transport maritime nécessitent l'extension des ports, et l’arrivée de navires plus longs, plus larges et à plus fort tirant oblige le dragage considérable de sédiments souvent contaminés par des métaux lourds et des polluants organiques. Différentes options de gestion doivent être identifiées dans chaque cas concret et le choix de l'option doit être fondé sur une approche de durabilité, compte tenu de l'économie, l'environnement et les aspects sociaux. Dans l'UE, un projet financé par «La gestion durable des sédiments contaminées de la mer Baltique ”(SMOCS; www.smocs.eu) au sein du programme régional de la mer Baltique 2007-2013, des outils d'évaluation et de prise de décision sont développés, i. e. Analyse de Cycle de Vie (ACV), l'évaluation des risques (RA) et la multi-analyse des critères de décision (MCDA). Pour développer ces outils, une série d'études de cas ont été réalisées comparant diverses méthodes de manutention telles que l'enfouissement, l'immersion en mer, l'immersion confinée aquatique et l’utilisation bénéfique dans les constructions portuaires en utilisant les technologies de stabilisation / solidification.

KEYWORDS: Contaminant, sediment, mangement, sustainability, stabilisation, solidification.

1 INTRODUCTION.

Sea transport is increasing due to its environmental and economic benefits. Ports are therefore a key part of the multi-module transport system in the society. In Sweden, for example, the ports manage more than 90% of the tonnage in the trade. To enable to manage this amount of trade, ports have to run and operate a port infrastructure that is robust, cost effective and environmentally sustainable. The increase in sea transport as well as longer, wider and more deep-draught ships cause huge needs of maintenance and development dredging of sediments in fairways and ports. In the coming years several million cubic meters of sediments need to be dredged in the Baltic Sea. A large volume of these sediments is contaminated with heavy metals and organic contaminants (HELCOM, 2009). This highlights a key issue for the society in the future - what is the sustainable management of these sediments? To approach sustainable management assessments should not be limited to site specific emissions but also include other categories such as use of energy, resources and climate impact (Arevalo, 2007). Handling options for dredged sediments have to be identified in each actual case and the choice of handling option should be based on a sustainability approach, considering economy, environment and social aspects, see Figure 1.

There are many possible options for managing contaminated sediments; the two major ones are either to take action or no action. In practice, action can include treatment in situ or ex-situ implying numerous options incl. capping of sea deposits, beneficial use of treated sediments as construction material or capping in-situ.

In the EU-funded project “Sustainable Management of Contaminated Sediments in the Baltic Sea” (SMOCS; www.smocs.eu) within the Baltic Sea Region Programme 2007-2013 tools for assessment and decision making are developed.

1.1 Aim and objectives

The aim of this paper is to presents results of case studies on assessment tools applicable in the sustainable management of contaminated sediments. The case studies include the stabilization/solidification (s/s) technology as one emerging sustainable handling option. Furthermore the authors present and invite stake holders to participate in networks to promote sustainable management in sediment handling and other port infrastructure.
2 TOOL-BOX FOR SUSTAINABLE MANAGEMENT

2.1 Overview of tools

There are several tools that could be used for sustainable assessment of contaminated sediments. The starting point for tools in decision making has to be based on an understanding of the nature of the decision that should be taken and the function and focus of the tool (de Ridder, et al., 2007). Therefore, the use of assessment tools depends on the decision level as well as the decision phase (see Figure 2).

Within the SMOCS project several case studies on assessment tools such tools have been performed. In this paper we present results from case studies using life cycle assessment (LCA) and multi criteria analysis (MCDA).

2.2 Life cycle analysis (LCA)

The purpose of a life cycle analysis (LCA) is to find out where in the life cycle the environmental load is the greatest and from what the impact is generated (ILCDA, 2011). This is mainly done through determining a material and substance flow. Thus is possible to assess the environmental impacts associated with all the stages of a product or a service/action life from-cradle-to-grave.

In the SMOCS project different management options of dredged contaminated sediments have been assessed by LCA (see Figure 3). The case studies have been based on sediment management in three ports: Port of Oxelösund, Sweden, Port of Gävle, Sweden and Port of Hamburg, Germany.

In the case studies of Port of Oxelösund and Port of Gävle, three possible management scenarios were compared. The scenarios were 1) utilization of sediment in quay construction by stabilization/solidification, 2) disposal in landfill and 3) disposal at sea. In the case study of Port of Hamburg, four management scenarios were compared. The scenarios were 1) disposal in river, 2) disposal at sea, 3) utilization of sediment in road construction or as landfill cover by METHA (Mechanical Treatment of Harbour Sediment), and 4) dewatering and disposal in landfill.

![Figure 2. Examples of when different types of assessment tools which could be implemented in a decision process (Lundberg, et al., 2011).](image)

![Figure 3. Port of Oxelösund, Port of Gävle, Sweden and Port of Hamburg, Germany (from top to bottom).](image)
The system boundaries were chosen with a comparative LCA approach, i.e. identical activities in all scenarios were excluded, aside from dredging activities. The system boundaries were expanded and included also the beneficial utilization of the sediment. Hence, the functional unit included the handling of the sediment and also the production of the service that the sediment would provide when utilized in quay, road, bricks etc. Disposal scenarios also included the fulfillment of the service but sediment material was substituted with the production and use of conventional material.

LCA could provide the decision maker with a good view on environmental impacts for either a certain activity or for a comparison of different activities. The LCA could thus be used in an early decision phase for comparing relative differences in energy use and climate impact between different handling options.

LCA could also be used for displaying the relation between the energy use and climate impact from production of material, transport of material and construction work and maintenance respectively. This could be made with a stand-alone approach. With the standalone LCA approach, it could be possible to describe significant activities in each sediment management alternative. Such approach demands a more extensive data inventory but could provide information on which measures should be taken in each management alternative to reduce the energy use and climate impact. The stand-alone approach has been tested in a SMOCS case study and the result is presented in Figure 4.

The data was collected from the cases of Oxelösund and Hamburg and completed with data on previously excluded activities such as dredging, transfers of dredged material.

The overall conclusion from these LCA cases studies are that the selection of handling alternative for sediment management has major significance on the overall energy use and climate impact. Furthermore, the energy use and climate impact from transportation of materials and dredged material is often significant in the context of sediment management.

2.3 Multi Criteria Decision Analysis (MCDA)

Multi criteria decision analysis (MCDA) is a tool that can integrate economic, environmental and social criteria and identifying the most sustainable handling alternative in a structured and rational way. Therefore, MCDA approach could be used at the project level for establishing the overall favorable handling alternative for management of contaminated sediments from a sustainability perspective. However, in order to be able to objectively score the performance of the different handling alternatives used in the MCDA assessment tools are needed.

Within SMOCS MCDA case studies has been performed for the Port of Gothenburg and the Port of Lübeck integrating economic, social and environmental criteria for decision.

Fundamental steps in a MCDA are to (Belton and Stewart, 2001)

1. Identify possible handling options
2. Identify decision criteria and their indicators
3. Weight decision criteria’s relative importance
4. Score the performance of the handling options in relation to each decision criterion
5. Calculate results

Results from the case study in the Port of Gothenburg are shown in Figure 5. A higher score should be interpreted as a better overall result, meaning that Rock chamber disposal and Solidification/stabilization are the most favorable options. A handling alternative scoring best on all decision criteria would result in the overall performance score 1.0. The bar colours show the contributions of environmental, social and economic criteria to the overall performance of each handling option. The impact of the port’s weighting can be seen clearly: economic and environmental criteria are given 2 and 1.5 times the weight of social criteria, and hence these contribute more to the overall performance.

3 CONCLUSION

MCDA provides a structured way of thinking through the whole range of decision criteria that should be taken into consideration.
when planning for handling contaminated sediments. MCDA can provide the transparency and documentation necessary for creating consensus between port owners and governmental organizations. This requires that a common opinion on decision criteria and weights can be established. It also requires that permit authorities embrace the concept of evaluating social, economic and environmental decision criteria together.

LCA is an appropriate tool for assessing energy and greenhouse gas emission, information that give important input to the MCDA. The LCA could be used both for comparing different handling alternatives as well as displaying the relation between the energy use and climate impact from production of material, transport of material, construction work, and maintenance.

The conclusion from the case studies using LCA and MCDA is that the selection of handling alternative for sediment management has major significance on the overall energy use and climate impact. Furthermore, it was shown that sediments utilized as construction material instead of disposed in landfill reduce energy use and climate impact significantly.

4 SMOCS DELIVERABLES AND A NETWORK ON SUSTAINABLE MANAGEMENT OF CONTAMINATED SEDIMENTS

The main deliverables of SMOCS consists of a guideline, tools for assessing sustainability and decision support, and a durable network. The guideline will address current and emerging technologies including verification of investigation and treatment technologies. The guideline will cover the whole process form planning to executing and control of treated sediments.

SMOCS has applied a highly participative approach. Therefore, the knowledge is compiled into the guideline in close cooperation with ports, maritime organizations, environmental authorities, construction industry as well as R&D performers. This approach is also a key starting point to establish a durable network.

The partners of the SMOCS project have agreed to establish a network for the period 2013-2017 on key issues and share experience, but also on development of further co-operation on identified issues. The topics covered are not limited to contaminated sediments thus including dredging and management of sediments in general as well as other port and authority issues if applicable.

The network is mainly based upon participants from the Baltic Sea Region. However as it exists in a European context, it is important that other regions can join and cooperate.

5 ACKNOWLEDGEMENTS

The SMOCS project is Part-financed by the European Union (European Regional Development Fund and European Neighbourhood and Partnership Instrument) the Baltic Sea Region Programme 2007-2013 and partly by the partners of the project being Swedish Geotechnical Institute, Luleå University of Technology, Sweden, Port of Gävle, Sweden, Lapprennannta University if Technology, Finland, Port of Kokkola, Finland, Maritime Institute, Gdansk, Poland, Port of Gdynia, Poland, Hamburg-Harburg University of Technology, Germany, Port of Klaipeda, Lithuania, and CORPI, Klaipeda, Lithuania

6 REFERENCES


Polymer support fluids: use and misuse of innovative fluids in geotechnical works
Les polymères: l'utilisation de nouveaux fluides de forage en travaux géotechniques

Jefferis S.A.
Environmental Geotechnics Ltd. and University of Oxford, United Kingdom

Lam C.
The University of Manchester, United Kingdom

ABSTRACT: Bentonite slurries have been used for over sixty years for the temporary support of excavations such as bored piles and diaphragm walls. At intervals over this time polymer products have been tried in place of bentonite but not always successfully. Recently it has become clear that, if used properly, polymer fluids offer many advantages over their bentonite counterparts, including improved foundation performance, lower environmental impacts, smaller site footprint and also simpler preparation, mixing and final disposal as they are used at much lower concentrations. They are also more easily managed than bentonite. However, successful use requires that the some specific characteristics of polymers are respected, in particular, it must be recognised that they are sorbed onto soils so that the polymer concentration in solution drops during use.

RÉSUMÉ: Les coulis de bentonite ont été utilisés depuis plus de soixante ans pour la mise en œuvre des pieux forés et parois moulées. Des boues polymères ont été testées pour remplacer ces suspensions d’argile mais les résultats n’ont pas toujours été conclusifs. Récemment, il est devenu évident que, si utilisées correctement, les polymères offrent de nombreux avantages, entre autre une amélioration de la performance de fondation et une réduction de l’impact sur l’environnement; les procédés de préparation et de mélange sont facilités ainsi que la disposition de déchets car la quantité de polymères utilisée est plus petite que la quantité de bentonite nécessaire dans les coulis de bentonite. Cependant le succès de l’utilisation des polymères est limité par certaines de leurs propriétés – en particulier, le fait qu’ils s’adsorbent aux sols diminue leur efficacité durant le forage.

KEYWORDS: bentonite, diaphragm walls, piles, polymer, support fluids.

1 INTRODUCTION

1.1 Background

Bentonite slurries have been used for over sixty years for the temporary support of excavations such as bored piles and structural diaphragm walls and somewhat more recently for slurry tunneling. At intervals over this time polymer alternatives have been tried but not always successfully, so that for some the word ‘polymer’ has become an anathema. However, recent developments have shown that, if used properly, polymer fluids offer many advantages over their bentonite counterparts, including improved foundation performance, smaller site footprint, reduced environmental impact and simpler mixing and final disposal as they are used at much lower concentrations than bentonite.

The many advantages polymer solutions offer can be achieved only if specifiers and users have a proper understanding of their properties and their in-situ behaviour and recognise that not all polymers are the same – the properties of the various polymers used in excavation works can vary very substantially. Unfortunately, it is still not unusual for users and/or specifiers to treat excavation support polymers as if they were a single material similar to bentonite. Polymer solutions are fundamentally different fluids to bentonite slurries and each type of polymer has distinct physical and chemical properties which must be respected to avoid misuse.

1.2 Natural and synthetic polymers

Early polymer fluids tended to be based on naturally derived products such as carboxymethyl cellulose, xanthan and guar gums but they had a limited range of properties, were easily biodegraded and thus short-lived unless treated with biocides which can have negative environmental impacts. Furthermore, like bentonite they could not inhibit the dispersion of fine soils such as clays into the excavation fluid and thus required cleaning before re-use.

In recent years, the advent of synthetic polymers has allowed the development of fluid systems with tailored properties. Systems can be designed to be bio-stable, environmentally benign and to inhibit clay dispersion so enabling repeated use without specialised soil-slurry separation plant such as hydrocyclones, dewatering screens and centrifuges. Today, with these benefits, synthetic polymers account for the vast majority of polymers used for foundation construction and in oil-well drilling (where bentonite free muds are regularly used). Natural polymers continue to be used for excavation projects where rapid biodegradation is useful such as the construction of permeable reactive barriers and deep drainage walls.

1.3 Objectives

To promote best practice in the use of polymer support fluids for the construction of deep foundations, this paper sets out the latest understanding of the behaviour of polymer fluids and also presents experience drawn from recent research and case histories from around the world.

2 SUCCESS THROUGH PROPER USE OF POLYMERS

2.1 Operational benefits

The operational benefits offered by polymer fluids traditionally have been one of the main reasons for contractors to switch from bentonite to polymers. For example, Lennon et al. (2006) note that the size and cost of the ancillary plant required for bentonite slurries make them relatively uneconomic for urban sites with restricted space and access such as those in city
centres. Figure 1 shows such a site in central Glasgow, UK which although measuring just 24 m by 40 m required sixty-two 750 mm diameter bored piles, i.e., approximately one pile every 4 m. The size of the site and the scope of the work meant that polymer fluids were the only feasible option because they do not require multiple holding tanks for slurry hydration nor do they require separation plant to recover the used slurry. Unlike bentonite slurries, polymer fluids require only a short swelling and hydration time prior to use and indeed emulsion polymers develop their properties almost instantaneously after mixing. Powered polymers, after wetting out, for example, with a Venturi eductor can be hydrated in an open-top tank gently agitated with a compressed air lance.

Figure 1. The small site in Glasgow where a polymer fluid was used.

Anonymous (2001) note that during the construction of the Channel Tunnel Rail Link (CTRL) East Kent-Ashford to Cheriton section, polymers were chosen because setting up a bentonite plant on some of the sites would have been almost impossible due to space restrictions. The saving of time for site set-up is an associated advantage. Compact polymer plant can be moved from site to site relatively quickly whereas mobilising bentonite plant on some of the sites would have been almost impossible due to space restrictions. The saving of time for site set-up is an associated advantage. Compact polymer plant can be moved from site to site relatively quickly whereas mobilising a bentonite set-up can absorb much valuable programme time.

2.2 Environmental benefits

Polymer fluids can offer significant environmental benefits when compared to their clay-based counterparts. For example, although used bentonite may be classified as a non-hazardous waste, it can be highly polluting if released into the aquatic environment. Polymer fluids can be disposed to sewer (with the undertaker’s consent) and the settled fines added to the excavation spoil – ideally for re-use. Thasnanipan et al. (2003) report that in Bangkok the primary reason for switching to polymers was, in most cases, to minimise the environmental issues associated with bentonite fluids. Caputo (2009) also expressed concerns regarding the potential environmental impacts associated with the use of bentonite for the bored piles for a bridge across the Tagus River in Portugal.

As outlined above, operational and environmental benefits are often cited as the main reasons for using polymers rather than bentonite. However, over the last two decades many field studies have been carried out to investigate the effects of polymer fluids and it is now appreciated that they can bring significantly improved load performance for piles, etc. The results of a recent UK case history are summarised below.

To assess the effects of different support fluids and of varying pile bore open times, Lam et al. (2010a) analysed the results from a full-scale field trial in East London, where the ground profile was a layer of made ground underlain by the Lambeth Group and then Thanet Sand. The trial involved the load testing of three instrumented piles, two of which were constructed under a polymer fluid and one under bentonite. The difference between the two polymer piles was the pile bore open time; one was concreted within 7.5 h of the completion of excavation (Pile P1) whilst the other was concreted at 26 h (Pile P2). The bentonite pile (Pile B1) was concreted at 7.5 h.

Figure 2 shows the load-settlement curves of the three piles; both polymer piles behaved similarly and significantly outperformed their bentonite counterpart at the maximum test load of 18 MN – and indeed the pile open for 26 h showed slightly better behaviour than that open to 7.5 h. Analysis of the data from the instrumentation on the piles and supporting laboratory tests demonstrated that the improvement in the load-settlement characteristics of the polymer piles was due to the higher shaft resistance and also the clean pile bases (Lam 2011). The effect of the polymer solution on concrete also was investigated. This showed that the polymer fluid had a similar effect on the strength and stiffness of hardened concrete as bentonite slurry – water from the fluids mixing with the surface concrete being the issue.

2.3 Improved foundation performance

Anonymous (2001) note that during the construction of the Channel Tunnel Rail Link (CTRL) East Kent-Ashford to Cheriton section, polymers were chosen because setting up a bentonite plant on some of the sites would have been almost impossible due to space restrictions. The saving of time for site set-up is an associated advantage. Compact polymer plant can be moved from site to site relatively quickly whereas mobilising a bentonite set-up can absorb much valuable programme time.

2.2 Environmental benefits

Polymer fluids can offer significant environmental benefits when compared to their clay-based counterparts. For example, although used bentonite may be classified as a non-hazardous waste, it can be highly polluting if released into the aquatic environment. Polymer fluids can be disposed to sewer (with the undertaker’s consent) and the settled fines added to the excavation spoil – ideally for re-use. Thasnanipan et al. (2003) report that in Bangkok the primary reason for switching to polymers was, in most cases, to minimise the environmental issues associated with bentonite fluids. Caputo (2009) also expressed concerns regarding the potential environmental impacts associated with the use of bentonite for the bored piles for a bridge across the Tagus River in Portugal.

As outlined above, operational and environmental benefits are often cited as the main reasons for using polymers rather than bentonite. However, over the last two decades many field studies have been carried out to investigate the effects of polymer fluids and it is now appreciated that they can bring significantly improved load performance for piles, etc. The results of a recent UK case history are summarised below.

To assess the effects of different support fluids and of varying pile bore open times, Lam et al. (2010a) analysed the results from a full-scale field trial in East London, where the ground profile was a layer of made ground underlain by the Lambeth Group and then Thanet Sand. The trial involved the load testing of three instrumented piles, two of which were constructed under a polymer fluid and one under bentonite. The difference between the two polymer piles was the pile bore open time; one was concreted within 7.5 h of the completion of excavation (Pile P1) whilst the other was concreted at 26 h (Pile P2). The bentonite pile (Pile B1) was concreted at 7.5 h.

Figure 2 shows the load-settlement curves of the three piles; both polymer piles behaved similarly and significantly outperformed their bentonite counterpart at the maximum test load of 18 MN – and indeed the pile open for 26 h showed slightly better behaviour than that open to 7.5 h. Analysis of the data from the instrumentation on the piles and supporting laboratory tests demonstrated that the improvement in the load-settlement characteristics of the polymer piles was due to the higher shaft resistance and also the clean pile bases (Lam 2011). The effect of the polymer solution on concrete also was investigated. This showed that the polymer fluid had a similar effect on the strength and stiffness of hardened concrete as bentonite slurry – water from the fluids mixing with the surface concrete being the issue.

2.3 Improved foundation performance

Anonymou...
viscosity of each of the fluids dropped and continued to do so working viscosities so leading to under-dosage of polymers. The effect of air bubbles on fluid viscosity escape of fine entrained air bubbles which were present in the in Figure 5. The overnight drop in viscosity was due to the of the fluid was measured at intervals and the results are shown through the pump system was then started with polymer drawn overnight to ensure stable fluid properties. Recirculation the storage tank may be at some distance from the excavation. This is an important aspect of plant operation as that the pump need not be repeatedly turned on and off during the excavation. Continuous circulation, although wasteful of energy, is generally regarded as beneficial for bentonite slurries as it prevents settlement and improves hydration.

Two commercially available polymer products based partially hydrolysed polyacrylamides (PHPAs) were used for the study. Each polymer fluid was prepared in accordance with the supplier’s recommended procedure and allowed to stand overnight to ensure stable fluid properties. Recirculation through the pump system was then started with polymer drawn off for use in pile bores as required. The Marsh funnel viscosity of the fluid was measured at intervals and the results are shown in Figure 5. The overnight drop in viscosity was due to the escape of fine entrained air bubbles which were present in the fluids after mixing. The effect of air bubbles on fluid viscosity is not well recognised and initial viscosities can be mistaken for working viscosities so leading to under-dosage of polymers.

From Figure 5 it can be seen that once pumping started the viscosity of each of the fluids dropped and continued to do so up to the end of the test. Both PHPAs were of high-molecular-weight (i.e. they were long-chain molecules – longer chain lengths tend to give higher viscosities) and it seems that the chains were undergoing scission as a result of continuing shear in the centrifugal pump and pipework so reducing the fluid viscosity. Indeed the damage was so severe for Fluid B that the initial 65 s viscosity (after overnight ageing) had reduced to 35 s at 22.5 h (after approximately 8 h recirculation) and was tending to that of pure water (28 s).

As the fluid was being used for pile excavations the viscosity was boosted by adding polymer directly to the pile bores to maintain stability and there were no collapses. However, had the monitoring programme not been in place, the contractor would not have been alerted to the problem and the pile bores might have collapsed due to the excessively low viscosity. To avoid viscosity reduction due to prolonged shear in centrifugal pumps, it is recommended that diaphragm pumps are used as they induce less shear and can be designed to stop automatically (so also saving energy) when the pressure rises as a result of closure of the delivery valve. If diaphragm pumps are not available, fluid recirculation should be minimised.

The viscosity and hence other properties of PHPA fluids can be damaged by salts present in mix waters and in the ground. To investigate the effect of salts in mix water, Lam (2011) measured the viscosity of several commercial polymer products over a range of sodium chloride concentrations in the mix water using an Ubbelohde capillary viscometer. Figure 6 shows some of the test results. It can be seen that above about 100 mg/litre sodium chloride, the PHPA fluid lost about 60% of its viscosity in deionised water whereas the blended polymer lost only about 40%. However, for both fluids there was little further effect up to 1000 mg/litre. The effects of salts in mix water are recognised by suppliers and are compensated by increasing the polymer concentration and raising the solution pH with caustic alkalis – though increase in pH may give limited benefit.

In saline soils there should be regular monitoring of fluid viscosity to check for viscosity loss; there are case histories of collapses. For example, on the Vasco da Gama Bridge in Portugal two of the piles had to be re-drilled following collapses which were possibly due to fluid contamination (Bustamante et al. 1998, KB Technologies Ltd. 2000). Schwarz & Lange (2004) also report a case history of pile bore collapse due to high concentrations of salts at a site in Benin. Although simple PHPAs can be adversely affected by salts, engineered polymer
sorption and thus a wholly insufficient use of polymer. The lack of appreciation of the need to replenish the polymer lost by sorption of polymers onto soils – which also can be a benefit as it reduces dispersion of fines into the fluid. To minimise the loss of fluid properties, fresh polymer must be regularly added to the system otherwise a significant degradation in performance of the fluid and potentially the foundation element will occur.

4 CONCLUSIONS
Through the use of case histories and recent research findings, this paper has outlined some strengths and limitations of polymer fluids as potential replacements for bentonite slurries particularly for small or congested sites. Strengths include improved foundation performance, simpler site operations and reduced environmental impact. Limitations include reduction of fluid properties due to continued shear in recirculation systems, potential for loss of properties in saline soils and importantly the results of a series of tests on a PHPA-bentonite mix with reduced environmental impact. Limitations include reduction of fluid properties due to continued shear in recirculation systems, potential for loss of properties in saline soils and importantly the results of a series of tests on a PHPA-bentonite mix with bentonite concentration in the fluid increases, the polymer concentration drops with use and unless the concentration is regularly re-established the fluid will become little more than muddy water, a condition which the authors have dubbed as ‘flipped’. The system has ceased to be polymer solution with some suspended soil and become a soil slurry with little polymer remaining in solution.

Recognition of the effects of sorption is absolutely key to the management of polymer slurries. With hindsight it is now clear that a number of past problems with polymers can be traced to a lack of appreciation of the need to replenish the polymer lost by sorption and thus a wholly insufficient use of polymer.

To illustrate the effect of polymer sorption, Figure 7 shows the results of a series of tests on a PHPA-bentonite mix with increasing concentration of bentonite; the latter was used as a soil as it strongly sorbs PHPAs. It can be seen that as the bentonite concentration in the fluid increases, the polymer concentration drops and ultimately approaches zero.

5 ACKNOWLEDGEMENTS
The work presented in this paper was undertaken as part of a research project jointly funded by the UK Engineering and Physical Sciences Research Council (EPSRC), Balfour Beatty Ground Engineering and KB International LLC. EPSRC grant reference nos.: EP/C537815/1 (Industrial CASE award) and EP/H50026X/1 (Knowledge Transfer Secondment). The authors would like to thank Messrs T. Suckling, V. Troughton, C. Martin, P. Martin and G. Goodhue for their help and advice throughout the project.

6 REFERENCES

Figure 6. Effect of added sodium chloride on the viscosity of a commercial blended polymer product and a pure PHPA both mixed in deionised water.

Figure 7. Reduction of polymer concentration by sorption.
Utilisation des déchets de sacs en polyéthylène (plastiques) pour l’amélioration des sols sableux

Kalumba D., Chebet F.C.
University of Cape Town, South Africa

ABSTRACT: This study investigated the possibility of utilising polyethylene shopping bags waste to reinforce soils to pave way for its use in civil engineering projects such as in road bases, embankments and slope stabilisation. A series of direct shear tests was undertaken on soil-plastic composites of two selected sandy soils: Klipheuwel and Cape Flats sands. Strips of shredded plastic material were used as reinforcement inclusions at concentrations of up to 0.3% by weight. The effect of varying dimensions of the strips was investigated by using strip lengths from 15 mm to 45 mm and strip widths from 6 mm to 18 mm. Shear strength parameters were obtained for composite specimen from which analyses were done to identify the extent of soil improvement. The testing programme involved addition of solid strips as well as perforated strips with varied diameter of perforations to examine the effect of the openings on the strips. Laboratory results obtained favourably suggest that inclusion of this material in sandy soils would be effective for ground improvement in geotechnical engineering.

RÉSUMÉ : Cette étude a examiné la possibilité de l’utilisation des déchets des sacs d’épicerie en polyéthylène pour renforcer les sols afin de promouvoir son intégration dans les projets de génie civil tels que les couches d’assise des routes, les remblais et la stabilité des pentes. Une série d’essais de cisaillement direct a été réalisée sur des composites plastique-sols sur deux sols sableux sélectionnés : sable de Klipheuwel et de Cape Flats. Des lamelles de matériau plastique déchiqueté ont été utilisées comme intrants de renforcement à des concentrations allant jusqu’à 0,3% du poids. L’effet de la variation des dimensions des lamelles a été apprécié en modifiant leurs longueurs de 15 à 45 mm et leurs largeurs de 6 à 18 mm. Les paramètres de la résistance au cisaillement obtenus pour les spécimens de composites ont servi à faire des analyses pour l’estimation du degré d’amélioration des sols. Le procédé scientifique a été fait avec des lamelles pleines et des lamelles perforées à divers diamètres afin d’observer l’effet des interstices dans les lamelles perforées. Les résultats de laboratoire obtenus confirment favorablement que l’ajout de ce matériau dans les sols sableux serait efficace pour l’amélioration des sols dans les applications d’ingénierie géotechnique.

KEYWORDS: Plastic bags, Polyethylene, Waste minimisation, Soil reinforcement, Ground improvement, Shear strength

1 INTRODUCTION

Increased use of plastics in day to day consumer applications has resulted in municipal solid waste containing an ever growing fraction of plastic material used for a short time and discarded. Ever since their invention over 60 years ago, plastics have taken centre stage in daily life due to favourable attributes such as low weight, durability and lower cost as compared to other material types (Thompson et al. 2009, Andrady and Neal 2009). These attributes make plastics convenient and therefore highly demanded by consumers with production increasing substantially from about 0.5 million tonnes in 1950 to over 260 million tonnes by 2008 with higher projections expected in the future (Thompson et al. 2009). A large percentage of plastics produced are used for disposable applications like packaging and therefore reach the waste stream more quickly since their usage life is shorter than that of plastics used in the construction or automotive industry (Azapagic et al. 2003). Consequently about 10% by weight and 20% by volume of the municipal waste stream is composed of plastics destined for landfills (Barnes et al. 2009, Azapagic et al., 2003). Of the plastic material discarded, 50% is from packaging, a third of which consists of plastic shopping bags (Nhamo 2008).

Plastic shopping bags are water resistant materials mostly made of polyethylene, a non-biodegradable polymer produced from non-renewable petroleum and natural gas resources. The linear consumption patterns of plastic bags involving single usage and then disposal has led to environmental challenges such as diminishing landfill space, marine and urban littering. There is therefore a growing need to find alternative uses of reclaimed plastic bag waste to lengthen the usage time of the plastic material. This is so as to tap into the abundant plastic resource that possesses a great extent of versatility and yet in the same vein poses a danger to the environment if not well managed in terms of responsible disposal that involves resource recovery vital in contributing to sustainable development.

Chen et al. (2011) maintain that new approaches on the reuse of plastic waste in cities as alternative materials for urban developmental programs, referred to as urban symbiosis, could help reduce green house gas emissions and fossil fuel consumption. This study explored the possibility of utilising reclaimed plastic material from polyethylene bags as tensile inclusions to reinforce soil for ground improvement schemes in geotechnical engineering applications such as retaining walls, road bases, embankments and slope stabilisation. Research into random inclusion of discrete polypropylene fibres in soil as reinforcement material have reported increases in peak shear strengths and reductions of post peak losses in soils (Zornberg 2002, Consoli et al., 2007, Falorca and Pinto 2011). Furthermore, these fibres have been found to improve compressive strength and ductility of soils (Maher and Ho 1994, Santoni et al., 2001, Miller and Rifai 2004). As a result, fibre reinforced soil consisting of polypropylene fibres have been successfully used on embankment slopes in the US (Gregory and Chill 1998) and in applications such as foundations for sport pitches, horse racing tracks and access for secondary roadways (Ibraim and Fourmont 2006).

The main objective of this study was therefore to investigate the effect of including plastic strips from polyethylene shopping
bags on the shear strength of two locally sourced sandy soils. Additionally, perforations were introduced on selected strips to examine if increased bonding and interlocking of soil in the soil-plastic composite through the openings in the plastic material provided an additional effect on the shear strength parameters of the soil-plastic composite.

2 MATERIALS AND METHODS

2.1 Soil Material

The soil types used in the study were Cape Flats sand and Klipheuwel sand, both predominant in the region of Cape Town, South Africa. Cape Flats sand is a medium dense, light grey, clean quartz sand with round shaped particles while Klipheuwel sand is a medium dense, reddish brown sand with angular particles. Table 1 gives a summary of the physical properties of the sands.

Table 1. Engineering properties of the selected soils.

<table>
<thead>
<tr>
<th>Soil Property</th>
<th>Cape Flats Sand</th>
<th>Klipheuwel Sands</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.66</td>
<td>2.64</td>
</tr>
<tr>
<td>Particle Range (mm)</td>
<td>0.075-1.18</td>
<td>0.075-2.36</td>
</tr>
<tr>
<td>Mean Grain Size, $D_{50}$</td>
<td>0.5</td>
<td>0.72</td>
</tr>
<tr>
<td>Coefficient of uniformity, $C_u$</td>
<td>3.0</td>
<td>4.21</td>
</tr>
<tr>
<td>Coefficient of curvature, $C_i$</td>
<td>0.85</td>
<td>1.05</td>
</tr>
<tr>
<td>Angle of friction ($\circ$)</td>
<td>38.5</td>
<td>41.6</td>
</tr>
</tbody>
</table>

2.2 Plastic Material

The plastic bags (Figure 1a) were sourced from a local supermarket and shredded into strips of varying lengths and widths using a laser cutting machine. The bags were labeled as high density polyethylene (HDPE) according to the plastics identification code by the American Society of the Plastics Industry (SPI). The density was measured as 743 kg/m³ with an average density of 743 kg/m³ and the tests performed for normal stresses of 25 kPa, 50 kPa, 100 kPa at a shear loading rate of 1.2 mm/min. The peak stress for each soil specimen was noted including the results obtained for the control experiment in which no strips were added to the soil.

2.3 Experimental Work

Soil samples for the tests were oven dried in order to eliminate any effects of moisture and the plastic strips mixed with the soil in a bowl to form a composite (Figure 2a). The plastic strips used were of lengths 15 mm, 30 mm, 45 mm, and widths of 6 mm, 12 mm, 18 mm. Perforations of diameter 1 mm and 2 mm were made on strips of width 6 mm while their lengths varied. The strips were added to the soil at concentrations of 0.1%, 0.2% and 0.3% by weight and the composite material placed into a 100 mm x 100 mm shear box for direct shear testing (Figure 2b). The specimen in the shear box was compacted to an average density of 1700 kg/m³ and the tests performed for normal stresses of 25 kPa, 50 kPa, 100 kPa at a shear loading rate of 1.2 mm/min. The peak stress for each soil specimen was noted including the results obtained for the control experiment in which no strips were added to the soil.

3 RESULTS AND DISCUSSION

The peak shear stresses obtained from the direct shear tests were recorded and plotted against the respective applied normal stresses to determine the friction angles for each soil specimen tested. The results revealed a general increase in peak friction angles for both Klipheuwel and the Cape Flats sands on addition of both the solid and perforated plastic strips. The plastic parameters yielded a distinct effect on the soils as they were varied with both soils showing a unique response to each parameter. The relationship between the peak friction angle and the different strip variables of length, width, concentration and perforation diameter are presented in Figures 3 and 4.

3.1 Solid Strips

The results indicate that the peak friction angle for both Cape Flats and Klipheuwel sand is enhanced on addition of solid plastic strips (Figure 3a). An increase in friction angle from 38.5° to 42.4° was observed for the Cape Flats sand and from 41.6° to 44° in Klipheuwel sand. The higher values obtained for Klipheuwel sand was due to the better grading and thus giving a higher initial shear strength. The results reveal that maximum friction angles were obtained with 15 mm strips for Klipheuwel and 45 mm strips for Cape Flats sand. Therefore, there could be a limiting plastic strip length for the soil-plastic composite beyond which further lengthening results in a decrease in the shear strength on addition of the solid strips.

Further testing indicated that beyond the reinforcement width of 6 mm, the peak friction angle decreased which suggested that the soil strength decreases as the reinforcement strips widen (Figure 3b). This may be due to an increased interaction between the plastic strips caused by more overlapping for the case of wider strips in the test specimen resulting in reduced soil-plastic interaction in the composite.

An almost linear increase in the initial friction angle was observed for Cape Flats sand on increasing the strip content with a progressive improvement from 38.5° at 0.1% to 42.4° at 0.3% concentration (Figure 3c). Klipheuwel sand on the other hand responded with an increase on addition of 0.1% plastic and a decrease at higher concentrations. Higher plastic content seemed to affect the particle interlocking in the more angular shaped Klipheuwel sand resulting in a lower friction angle at greater strip concentrations.
3.2 Perforated Strips

Introducing perforations on the strips achieved higher friction angles with additional increases of 3.5% and 18.0% on inclusion of the perforated strips in Klipheuwel and Cape Flats sand respectively. This was represented by improvements from 41.6° to 44° for Klipheuwel sand and 38.5° to 45.3° for the Cape Flats sand (Figure 4a). The effect of varying the length of the perforated plastic strips was more significant in Cape Flats sand. Addition of the reinforcement generally had a bigger impact as regards improvement of the shear strength parameters for the more round shaped Cape Flats sand than for angular shaped Klipheuwel grains. This indicates an inherent requirement for reinforcement of the more poorly graded Cape Flats sand so as to enhance its strength properties.

Varying the strip perforation diameter provided an increase of up to 14.7% in Klipheuwel sand compared to using solid strips and for Cape Flats sand a further increase of 8.5% was recorded (Figure 4b). This result indicated increases of 2° for every additional mm in perforation diameter.

The improvement in friction angle with perforation diameter may be attributed to interaction between the soil and the plastic in the composite as well as better bonding and interlocking between the soil particles through the perforations in the plastic strips.

An increase in the peak friction angle from 38.5° to 44.5° for Cape Flats sand was obtained when perforated plastic strips were added to the soil at a 0.1% concentration as shown in Figure 4c. A concentration of 0.2% resulted in a slight decrease and a further increase in concentration to 0.3% provided a higher friction angle of 45.0°. The pattern in Klipheuwel composites however indicated that addition of the perforated strips at a 0.1% concentration caused a slight improvement in friction angle but a decrease was observed for concentrations of 0.2% and 0.3%. This demonstrates that the influence of strip concentration could be soil specific due to the difference in soil properties like particle shape and size. Different soils would therefore require specific testing to determine the parameters particular to the soil type in order to obtain an optimal increase in the shear strength parameters on inclusion of the plastic strips.
3.3 Deformation of Strips during Shear

The plastic strips used in the composite specimen for the direct shear tests were assessed for physical deformations such as dents or rupture at the end of each experiment. The nature of deformations of the plastic strips was examined with respect to their location in the shear box. Visual inspections revealed that most of the elements that deformed were within or close to the shearing zone. The indentations on some of the strips may have been caused by soil particles as they pressed in to form surface attachments with the plastic strips (Figure 5a). This was mainly due to the particle shape and grading of the sandy soils used in the study that enhanced the frictional bonding between the soil and the plastic material. Other strips in the specimen were stretched and compressed due to the shearing action at or near the shear plane (Figure 5b). The stretched strips were located parallel to the shearing direction. This indicated that as the plastic strips were strained relative to the shearing direction, they improved the soil tensile strength by enabling transfer of forces arising from the loading conditions. None of the reinforcements in the composite were severely indented or ruptured during the shearing since the tensile strength of the plastic strips was greater than 15MPa compared to the generated shear forces in the test specimen under for a maximum applied normal stress of 100kPa.

The laboratory results presented in the study favourably suggest the possibility of utilizing plastic material as tensile inclusions in sandy soil to increase the resistance to shear. As demonstrated in Chebet et al. 2012, the plastic inclusions can also improve the load bearing capacity and settlement characteristics of the sand. Additionally, introduction of perforations on the plastic material further aids in the interaction between the soil and plastic thereby boosting the soil’s strength properties. However, a better understanding of the interaction mechanism in soils reinforced with the plastic material would be essential to properly document the engineering behaviour of the soil-plastic composite.

The influence of the soil physical properties, plastic properties and scale effects would need to be further investigated through more comprehensive testing in a wider range of stresses using larger scale tests to eliminate boundary effects. This could in turn contribute in the development of design methodologies for projects that may opt to incorporate this type of reinforcement material resulting in a reduction in project costs. Furthermore, successful application in the field could permit a reduction of plastic waste disposed of to landfills bringing along environmental benefits as a result of more efficient use of natural resources and reduction of CO₂ emissions.

4 CONCLUSION

A comprehensive laboratory direct shear testing programme was undertaken on composite specimens of sandy soils mixed with random inclusions of plastic strips obtained from high density polyethylene shopping bags. Two locally sourced soils, Klipheuwel and Cape Flats sands were selected for the research and the influence of plastic strips on the shear strength parameters of the sandy soils were studied. The effect of introducing perforations on the plastic strips was further examined. Parameters of the plastic strip inclusions such as length, width, concentration and diameter of perforations were varied to investigate the effect on the peak friction angle. The plastic strip lengths used in the study were 15 mm, 30 mm and 45 mm, strip widths 6 mm, 12 mm, 18 mm and perforation diameters of 1 mm and 2 mm made on the 6mm wide strips. The strips were added to the soil samples at concentrations of 0.1%, 0.2% and 0.3% by weight.

Results indicate an improvement in peak friction angle on addition of the solid strips and perforated strips of varied lengths and concentrations for the both sands. For the scope of experiments conducted, maximum values for the peak friction angles were obtained for strips of length 30 mm, concentration 0.1% and perforation diameter of 2 mm. Addition of perforations on the strips resulted in a further enhancement of the friction angle as compared to results obtained using specimens prepared with solid strips. An increase in the diameter of perforations resulted in higher values of friction angle at an average of 2° for each mm in perforation diameter.

5 REFERENCES


Effect of dredge soil on the strength development of air-foam treated lightweight soil

Kataoka S.
Hakodate National College of Technology, Hakodate, Japan
Horita T.
Kobe University, Kobe, Japan
Tanaka M.
Port and Airport Research Institute, Yokosuka, Japan
Tomita R., Nakajima M.
Koa Kaihatsu Corporation, Chiba, Japan

ABSTRACT: In this study, the unconfined compression strength, $q_u$, and the elastic shear modulus, $G$, from bender element test, were in detail examined in the laboratory by using the air-foam treated lightweight soil samples made from several kinds of soil. From the results of these tests, the values of the $q_u$ and the $G$ with curing period were different to the kinds of soils into the air-foam treated lightweight soil samples. In addition, it was found that the development of the $q_u$ and the $G$ were related to the increase / growth of the ettringite produced at the time of the reaction of hydration of the concrete. On the other hand, it has been shown that the relations between the $q_u$ and the $G$ are nearly proportional, and that this tendency remains always the same, whatever is the type of soil. Effets des sols de dragage sur le développement de la résistance des sols mélangés à de l’air

RÉSUMÉ : Dans cette étude, la résistance en compression simple, $q_u$, avec le module d'élasticité de cisaillement, $G$, du critère de l'élément bender, était en détail examinés en laboratoire à l'aide de l'air sous forme d'échantillons de sol traités légers fabriqués à partir de plusieurs types de sols. A partir des résultats de ces tests, les valeurs de la $q_u$ et le $G$ avec période de cure étaient différents des types de sols dans l'air sous forme d'échantillons de sol traité légers. En outre, il a été constaté que le développement de la $q_u$ et le $G$ sont liés à l'augmentation / la croissance de l'ettringite produite au moment de la réaction d'hydratation du béton. Dans le cas contraire, il a été montré que les relations entre la $q_u$ et le $G$ sont presque proportionnelle, et que cette tendance reste toujours le même, quel que soit le type de sol.

KEYWORDS: air-form treated lightweight soil, unconfined compression strength, elastic shear modulus.

1 INTRODUCTION

Air-foam treated lightweight soil is a ground material prepared by adding and mixing in a cement-type stabilizing agent and air foam made by a surfactant or animal-protein foaming agent to a source soil such as dredged soil and surplus construction soil. In recent years, there has been an increase in the number of construction projects using air-foam treated lightweight soil for the purpose of reducing earth pressure and containing land subsidence. This new type of ground material for harbor and airport construction that offers added value such as lightweight, safety, and recyclability is called Super Geo-Material (SGM) (e.g. Thuchida et al. 1996).

When SGM is employed for a construction site, the required strength of the mix proportion is obtained by multiplying the design strength by an overdesign factor, and a mix proportion test is conducted in advance to determine the amount of stabilizing agent and air foam to be added to the source soil of which the moisture content has been adjusted with water. In some cases where the physical properties of the source soil are expected to vary from one sampling location to another, a mix proportion test is conducted in advance to determine the amount of additives. Naturally, some variations are found in the strength of the soil samples (Nagatome et al. 2010). In fact, when the unconfined compression test and the bender element (BE) test were conducted on a large number of SGM samples taken from the same construction site and to which an equal amount of the stabilizing agent had been added, the unconfined compressive strength, $q_u$, varied from one sample location of the source dredged soil to another, albeit within the expected range of the design. It was confirmed that there is a high correlation between the shear wave velocity, $V_s$ (or shear modulus, $G$) and $q_u$ (Kataoka et al. 2011).

This study focused on the properties of the source soil within SGM. Six different types of source soil were used to prepare SGM in a room environment. The unconfined compression test and the BE test were conducted to examine the impact of curing time on the strength and stiffness of the soil. In addition, the microscopic structure of the specimens was observed using a Scanning Electron Microscope (SEM) in order to visually examine how the internal structure of SGM changed with the curing time.

2 SAMPLE PREPARATION AND TESTS PERFORMED

Table 1 shows the physical properties of the source soils used to prepare the SGM specimens. Six types of source soil were used: two types of dredged soil taken from the construction site of the Tokyo International Airport expansion project, one from the area where the odor of what was suspected to be hydrogen sulfide, biological decaying and the like (hereafter “Tokyo Bay A”) was relatively weak and the other from the area where the same odor was very strong (hereafter “Tokyo Bay B”); dredged soil taken from Kobe Port (hereafter “Kobe”); surface soil taken from a few meters below the seabed of the Sea of Okhotsk (hereafter “Okhotsk”); Kasaoka Clay; and Kuni-bond. The latter two are commercially available products. When liquid limit, $w_l$, a criterion used for the mix proportion design, was examined, the six types of source soil could be categorized as follows: Tokyo Bay A and B and Kobe, which had an approximately equal level of liquid limit; Kuni-bond, which had a higher $w_l$ than the aforementioned three types; and Okhotsk Seabed Sediment and Kasaoka Clay, which had a lower $w_l$. While there were almost no differences between Tokyo Bay A and B in physical properties such as $w_l$ and grain size composition, a major difference was observed in the pH level of pore water.
To prepare SGM specimens, seawater taken from Hakodate was used as the mixing water, blast furnace cement B as the stabilizing agent, and air foam (with a density of 0.05g/cm³) prepared with hydrolyzed animal protein using the pre-foaming method as the foaming agent. These ingredients were then mixed with each of the source soils that had been passed through a sieve of 425 lm and the resulting mixtures were put into plastic molds with a 5 cm diameter and a 10 cm height. With the top sealed by a plastic wrap, the mixtures were then cured in the air until the prescribed curing ages were attained. Table 2 shows the flow values of the specimens after the water content, w, of the source soils used to prepare the SGM specimens for this study was adjusted by sea water (hereafter “Adjusted Soil”) and after the stabilizing agent and the air foam were mixed in. The required wet density and the amount of stabilizing agent added were respectively kept constant at ρ=1.1 g/cm³ and 75 kg/m³, with the water content ratio of the Adjusted Soil at 285%, equivalent to 2.5 w₁ of Tokyo Bay A and B. While Tokyo Bay A, B and Kobe had similar microscopic structure. An internal observation of the SGM specimens was made using an SEM on each of the prescribed curing days to examine the results, vh and Gm, of the shearing modulus (Gm=ρL × VH, Gm=ρL × VH). An internal observation of the SGM specimens was made using an SEM on each of the prescribed curing days to examine the correlation between the strength development and the microscopic structure.

3 RESULTS AND DISCUSSIONS

3.1 Strength and Shear modules of the SGM specimens

Figure 1 shows the changes to the qₜ and Gₘ₀ levels of the prepared specimens with the elapse of curing days. From the results, qₜ and Gₘ₀ of all specimens showed a linear increase in the semi-log graph. However, a substantial degree of variability was observed from one source soil to another in the soil strength measured on the same curing day even though the mixing conditions, such as the cement quantity per unit volume and the w of the Adjusted Soil were identical. In particular, among the three types of dredged soil (Tokyo Bay A, Tokyo Bay B and Kobe) that shared almost identical physical properties such as w₁, the qₜ level of Tokyo Bay B was much lower than the other two. It is suspected that the composition of the pore water was suppressing the strength development of Tokyo Bay B, given that its pH level was lower than those of the other two specimens. On the other hands, the w₁ levels of Okhotsk and Kasaoka Clay were lower than those of the aforementioned three types of dredged soil. While Okhotsk showed large qₜ and Gₘ₀ levels, comparable to those of Tokyo Bay A, Kasaoka Clay had very low qₜ and Gₘ₀ values, similar to those of Tokyo Bay B. The factors causing the large qₜ and Gₘ₀ values of Okhotsk are suspected to be the large volume of silt in the soil. The small qₜ and Gₘ₀ values of Kasaoka Clay are believed to be caused by the material separation that occurred after the cement and air foam were mixed in because of the high w ratio of the Adjusted Soil of 285%, about 5 times greater than its w₁. The decrease in the strength of Kasaoka Clay is also likely to have resulted from its clay mineral components since the pore water composition.

Table 2. Water contents and flow values of the SGM specimens

<table>
<thead>
<tr>
<th>Samples</th>
<th>Tokyo Bay A</th>
<th>Tokyo Bay B</th>
<th>Kobe</th>
<th>Okhotsk</th>
<th>Kasaoka Clay</th>
<th>Kuni-bond</th>
</tr>
</thead>
<tbody>
<tr>
<td>ρ₀ (g/cm³)</td>
<td>2.62</td>
<td>2.70</td>
<td>2.64</td>
<td>2.56</td>
<td>2.71</td>
<td>2.70</td>
</tr>
<tr>
<td>w₁ (%)</td>
<td>114.7</td>
<td>112.4</td>
<td>108.2</td>
<td>85.6</td>
<td>55.4</td>
<td>133.1</td>
</tr>
<tr>
<td>Lₚ (%)</td>
<td>10.4</td>
<td>11.5</td>
<td>9.7</td>
<td>7.2</td>
<td>8.2</td>
<td>7.8</td>
</tr>
<tr>
<td>pH</td>
<td>7.7</td>
<td>3.4</td>
<td>7.9</td>
<td>7.6</td>
<td>7.5</td>
<td>-</td>
</tr>
</tbody>
</table>

Clay (2–5 μm)
| Clay (%)      | 33          | 39          | 36   | 24      | 44           | 12       |
| Clay (<2 μm)  | clay Qtz    | Qtz Qtz Qtz | Qtz Qtz Qtz |
| mineral       | PI          | PI          | PI   | PI      | PI           | PI       |

Kln

| (2.5w₁)        | (2.5w₁)     | (2.5w₁)     | (3.3w₁)| (5.1w₁) | (2.1w₁)      |

<table>
<thead>
<tr>
<th>Frow value (cm)</th>
<th>Adjusted soil</th>
<th>47.5</th>
<th>58.0</th>
<th>49.0</th>
<th>64.0</th>
<th>66.0 -*</th>
<th>57.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>SGM</td>
<td>19.0</td>
<td>27.5</td>
<td>21.0</td>
<td>36.0</td>
<td>66.0 -*</td>
<td>39.0</td>
<td></td>
</tr>
</tbody>
</table>

* : 66.0 -- shows 66.0 over

Figure 1. Variation of qₜ and Gₘ₀ with the curing periods
probably did not play a role, as it likely did for Tokyo Bay B.

Figure 2 shows the relationship between the flow values of the SGM specimens and $q_u$ on the curing period of 28 day. While it was believed that there is some relationship between flow values and soil strength, the lower $q_u$ value of Tokyo Bay B than that of other specimens suggests that there is some relationship between the strength and the low pH level of Tokyo Bay B. Figure 3 shows the relationship between $G_u$ and $q_u$. It is evident from the graph that there is a strong correlation between $G_u$ and $q_u$ obtained from all the SGM specimens, with an approximately linear relationship between the two variables in each one of them. A previous study showed there is a linear relationship between $q_u$ and Young’s modulus in the small strain range of cement-treated sand, while another study demonstrated that there is a linear relationship, as it was found in this study, between the elastic shear modulus and the stiffness of cement-treated soil where cohesive soil was used as the source soil (Shibuya et al. 2001, Seng and Tanaka 2011). Based on these results, the relationship obtained in this study is believed to be a characteristic common to cement-treated soil. While changes to $G_u$ and $q_u$ with the increase of curing days varied greatly from one soil type to another, it was confirmed that the relationship between the variables fell within a certain range, regardless of the type of source soil used. From this it can be concluded that in air-foam treated lightweight soils, where the amount of cement additive and the required $\rho$ are approximately the same, the relationship between $G_u$ and $q_u$ is approximately the same, regardless of the type of source soil.

Figure 4 shows the relationship between $G_{uh}$ and $G_{vh}$ obtained from the shear wave velocity propagating in horizontal and vertical directions. The slope is virtually uniform, regardless of the number of curing days and the soil type. However, the slope is smaller than that of natural clay deposits compacted to $K_0$ (Kawaguchi et al. 2008). Thus, a relatively uniform distribution of nearly spherical air foam inside SGM (Watabe et al. 2004) and a relatively loose (random) state in which the source soil has stabilized are believed to have made SGM an isotropic material in terms of stiffness.

3.2 Microstructure observation of the SGM specimen

The microscopic structure of SGM was observed using the SEM in order to examine the factors affecting the strength development of the SGM specimens from the changes to the internal structure with the passage of curing time. Photos 1 show the internal structures of Tokyo Bay A, on the curing period of 3 (1a) and 182 days (1b), and Tokyo Bay B, on the curing period of 28 (2a) and 182 days (2b). It is observed that the needle-like ettringite crystals formed by the hydration process of the cement on the curing period of 3 day (see 1a), which was characterized by a higher level of strength. The photos also show how the needle-like crystals spread throughout the entire specimen by the 182 day (see 1b) and filled the void space. In the other hands, the ettringite in Tokyo Bay B were observed on the 28 and 182 day (see 2a, b), more void space was observed in the specimen’s inner structure and the bonding of crystals did not seem very prevalent. The evidence raises the possibility that the formation, growth, and bonding of ettringite crystals play a major role in the development of the strength and stiffness of SGM.
4 CONCLUSIONS

In this study the unconfined compression test, the BE test, and observations of the internal structure using an SEM were conducted on SGM specimens prepared with six different types of source soil to examine how different source soils would impact the strength development of SGM. The findings are summarized as follows:

- It was inferred that while the strength and stiffness of the SGM specimens increased with the elapse of curing days, there is a very large variation in their actual levels due to the mineral components and the constituents of pore water contained in the source soil used. In addition, it became clear that the SGM strength cannot be estimated from the flow values of the specimens.
- In SGMs with an approximately equal amount of cement additive and comparable target wet density, the strength and stiffness have a linear relationship as is the case in other cement-treated soil, and their slopes are approximately the same regardless of the soil type.
- The slope obtained from $G_{0h}$ and $G_{0v}$ is characterized by an approximately 1:1 relationship, showing that the air foam in the specimens makes SGM a very isotropic material in terms of stiffness.
- The observations of the internal structure of SGM using the SEM on the predetermined curing days suggested the possibility that the increase, growth, and bonding of needle-like ettringite crystals, formed by the hydration process of the cement, were a major factor contributing to the development of the strength and stiffness of the SGM specimens.

5 ACKNOWLEDGEMENTS

The authors thank Prof. Satoru Shibuya, Kobe University, Japan and Prof. Satoshi Yamashita, Kitami Institute of Technology, Japan for help to prepare the soil samples.

6 REFERENCES


Application of a Method to Accelerate Granulated Blast Furnace Slag Solidification

Une méthode de solidification accélérée des granulats issus de laitier de haut fourneau

Kikuchi Y.
Department of Civil Engineering, Tokyo University of Science, Noda, Japan

Mizutani T.
Geotechnical Engineering Field, Port & Airport Research Institute, Yokosuka, Japan

ABSTRACT: On-site observations indicate that granulated blast furnace slag (GBFS) solidifies over time, but the entire mass solidifies quite slowly. This means that if GBFS is to be relied upon to be solid, it must be treated in order to accelerate the solidification process. Adding blast furnace slag in micro powder form to GBFS effectively speeds GBFS solidification in seawater. To improve the material’s resistance to separation, we recommend Prior Homogeneous Mixing Treatment (PHMT), which reduces the amount of separation of the material during pouring but does not interfere with the speed of the mixture’s accelerated solidification. PHMT-treated GBFS tends to solidify better when inundated in seawater than in fresh water. It is strong enough to protect against liquefaction if it remains in seawater for about two months.

RÉSUMÉ : les observations in-situ indiquent que les granulats issus de laitier de haut fourneau (GLHF) se solidifient dans le temps, avec néanmoins une vitesse de prise assez lente. L'utilisation de GLHF solidifiés implique donc que ces derniers fassent l'objet d'un traitement afin d'accélérer ce processus de prise. L'ajout sous forme de poudre fine d'éléments de laitier accélère de manière efficace cette solidification sous l'eau. Afin de limiter la ségrégation, il est recommandé d'effectuer au préalable un mélange homogène avec la poudre. Cela réduit la ségrégation entre les matériaux durant le déversement du mélange tout en n'influencant pas l'augmentation de la vitesse de prise. Les GLHF traités avec de la poudre de laitier tendent après un mélange homogène à se solidifier de manière plus efficace lorsqu'ils sont plongés dans l'eau de mer plutôt que dans l'eau douce. Ce mélange a alors assez de résistance pour résister à la liquéfaction si celui-ci reste immergé dans l'eau de mer plus d'une année et demi.

KEYWORDS: granulated blast furnace slag, solidification, backfill, quay wall

1 INTRODUCTION
Granulated blast furnace slag (GBFS) solidifies when it reacts with water. This property is known as latent hydraulicity. However, this characteristic of GBFS was ignored in the Japanese handbook for port construction engineers published in 1989 (CDIT, 1989) due to the lack of adequate information on the solidification of GBFS used in port construction. If used as a self-hardening material, GBFS holds great promise for use in protecting against liquefaction and for earth pressure reduction.

Most GBFS used to backfill quay walls does solidify, but a post-construction follow-up survey showed that GBFS solidification is a lengthy process, it never solidifies uniformly, and some may remain unsolidified (Kikuchi et al. 2005). As a result, some treatment is necessary before GBFS can be used as a self-hardening material.

In this study, we examine a method to accelerate the solidification of GBFS and propose a practical way to apply that method.

2 PREVIOUS RESEARCH
GBFS is vitreous, and the silicate SiO$_2$ it contains is in an unstable condition compared with crystalline material (NSA 1980). GBFS has high chemical reactivity and therefore solidifies in the presence of water and under high pH conditions where the pH exceeds 11.

Nishi et al. (1982) concluded that the latent hydraulicity of GBFS is high in highly alkaline water but low in seawater, which has a pH of about 8.

Figure 1. Acceleration of solidification achieved by mixing cement or PBFS with GBFS and by varying pore water type.

These facts suggest that solidifying GBFS in seawater is difficult because seawater acts as buffer solution with a very high buffering capacity (Christian 1986), and adjusting the pH of seawater is impractical.

A site investigation of GBFS placed as backfill 6-12 years previously showed that most of the GBFS had solidified, but its strength varied greatly (Kikuchi et al. 2006).

Kikuchi et al. (2011) tested various combinations of pore water and additives, attempting to speed GBFS solidification.
Figure 1 shows part of their results. Seawater and fresh water were tested with Portland cement and powdered blast furnace slag (PBFS) as additives. When fresh water was used, cement was more effective than PBFS in solidifying GBFS. But when seawater was used, PBFS was more effective than cement. Using seawater and PBFS was the most effective combination.

3 Issues Regarding Applying the GBFS Solidification Acceleration Method in the Field

There are several issues to consider when determining the most appropriate mixture of GBFS and PBFS for accelerating GBFS solidification: (1) the material can separate during construction, (2) it may separate after construction because of water flow (3) separation of the mixture is likely to affect how the GBFS solidifies, (4) the flow of pore water can affect solidification, and (5) GBFS may solidify differently when the pore water changes from sea water to fresh water (Kikuchi et al. 2010).

In the present study, we examine issues (1) to (5). First, we present experimental results regarding issues (2), (3), and (4). We then explain a way to prevent issue (1), and finally consider issue (5).

3.1 Possibility of material separation after construction

The physical properties of the GBFS used were $\rho_s = 2.845 \text{ g/cm}^3$, $\rho_{\text{min}} = 1.175 \text{ g/cm}^3$, $\rho_{\text{max}} = 1.508 \text{ g/cm}^3$, $D_{15} = 0.28 \text{ mm}$, and $D_{50} = 0.38 \text{ mm}$. The physical properties of the PBFS used were $\rho_s = 2.890 \text{ g/cm}^3$, with 5000 to 7000 cm$^2$/g of specific surface area. Artificial seawater was used as pore water.

The diameter of the PBFS was about 4 µm assuming spherical particles with no small holes. Thus, the GBFS and PBFS may separate when the mixture is poured onto the seabed. The ratio of $D_{15}$ for GBFS to $D_{50}$ for PBFS is more than 50. This ratio is an indicator of the possibility of material separation due to water flow through the material (Ishihara 2001).

We conducted experiments on the separation of the PBFS from the mixture. In this series of experiments, specimens with two layers were prepared. The lower layer of the specimen was a mixture of GBFS with 20% PBFS by weight. The upper layer was only GBFS. The relative density of each layer was 50%. Water flowed from the bottom of the specimen with a hydraulic gradient of from 10 to 40. This test was conducted in a triaxial apparatus at a confining pressure of 50 kN/m$^2$ to prevent boiling. The outlet velocity of the water at a hydraulic gradient of 40 was 120 m/day. The total outlet water volume from the specimen was 6 times the void volume of the specimen.

Figure 2 shows close-up X-ray CT images of the boundary between the layers of the specimen, where the contrast reflects the density of the material. Comparing the images before and after water flow, small differences can be observed. This means that although there may be a little separation when the peak velocity of the water flow is 120 m/day, complete separation of the material does not occur under these conditions.

In practice, the water flow velocity in GBFS used as backfill for gravity quay walls is around several m/day. Thus, GBFS and PBFS will never separate after construction.

3.2 Solidification of GBFS after material separation

The effect of material separation on the solidification characteristics of the material is examined in this section.

The GBFS and PBFS used here were the same as those used in section 3.1. The relative densities of the specimens were 50%. The pore water used was artificial sea water. In each specimen, 7.5% PBFS by weight was added to the GBFS. We tested four experimental mixing regimes: (1) GBFS and PBFS were mixed homogeneously (HMT), (2) PBFS was mixed with GBFS, then artificial sea water was added to achieve a 10% water content ratio and the mixture was cured in air for a week (prior homogeneous mixing treatment or PHMT), (3) One PBFS layer sandwiched between two layers of GBFS, and (4) Two PBFS layers were sandwiched between three GBFS layers.

Each specimen was saturated with artificial sea water and sealed, then cured for a designated period at a constant temperature of 20 degrees centigrade. Each specimen’s unconfined compression strength was measured after the designated curing period.

Figure 3 shows the relationship between the curing duration and unconfined strength. The unconfined compression strengths using HMT and PHMT exceeded 200 kN/m$^2$ after 14 days of curing. These strengths increased as the curing time lengthened. When the materials were separated, such as in cases (3) and (4), the unconfined compression strengths were very low. Figure 4 shows examples of the failure states for each case.

3.3 Solidification of GBFS underground with flowing water

In previous research, movement of pore water has been shown to prevent GBFS solidification (Kitayama 2003). In this section, we examine how pore water flow affects GBFS solidification. In this series of experiments, two water flow conditions and two mixing conditions were tested.

We used the same GBFS and PBFS as in section 3.1, and the HMT (1) and PHMT (2) mixing regimes from section 3.2.
In this series, sand boxes 30 cm wide, 30 cm long, and 50 cm high were fitted with a bulb for supplying water, located 3 cm from the bottom. Model ground 30 cm high was constructed of a mixture of GBFS and PBFS with 50% relative density. This was saturated with artificial sea water at the beginning of the test. Tests were run under static water and flowing water conditions. In the static water case, the pore water was never changed during the experiment. In the flowing water case, a volume of water equal to the volume of the voids in the ground was supplied slowly from the bottom of the ground once every three days. Curing continued for two months at a constant temperature of 20 degrees centigrade. After curing, the bearing strength distribution of the ground was measured using a soil hardness meter, and was converted to unconfined compression strengths.

The results show that the HMT-treated material subjected to static water conditions was the strongest. The material that had undergone HMT was weaker when cured in flowing water. However, for the PHMT-treated material, the opposite was true. With flowing water the PHMT material was stronger than the HMT material, meaning that PHMT has a higher potential to solidify GBFS than HMT under non-static water conditions.

3.4 Improving resistance to separation during construction using PHMT

For this series of experiments, we used GBFS with the following physical properties: \( \rho_s = 2.808 \text{ g/cm}^3 \), \( \rho_{\text{mix}} = 1.199 \text{ g/cm}^3 \), and \( \rho_{\text{max}} = 1.562 \text{ g/cm}^3 \). The median particle diameter \( (D_{50}) \) was 0.74 mm. The physical properties of the PBFS were \( \rho_s = 2.889 \text{ g/cm}^3 \), with 5000 to 7000 cm\(^2\)/g of specific surface area. Artificial seawater was used as pore water.

As the GBFS and PBFS may separate when the mixture is poured onto the seabed, PHMT was used to counter this problem. With PHMT, some of the PBFS attaches to the GBFS granules, making the mixture more resistant to separation and decreasing the turbidity the mixture causes in water.

We mixed 10% seawater and 7.5% by weight of PBFS with GBFS and cured the mixture for a designated period in air. We measured the turbidity it caused after 0, 3, 7, 10, and 14 days of curing. In each test, about 0.460 N of the PHMT mixture was measured the turbidity it caused after 0, 3, 7, 10, and 14 days of curing. Curing continued for two months at a constant temperature of 20 degrees centigrade. As the room was not perfectly temperature-controlled, its temperature was somewhat affected by the outside temperature.

3.5 Effects of changing from sea water to fresh water on the solidification of PHMT-treated GBFS

Here, we address issue (5) described above. The follow-up survey about GBFS used as backfill noted in the introduction revealed that the pore water in the GBFS layer changed completely from seawater to fresh water over a period of 4 months (Kikuchi et al. 2005). This phenomenon occurs because the mean ground water level is higher than the mean sea level and rainfall supplies fresh water. Figure 1 shows that GBFS mixed with PBFS in seawater solidified in a month. With this in mind, we checked the effect of a pore water transition in a series of laboratory experiments.

Figure 6 shows how the experiment was set up. The box holding the sand was 800 mm long, 500 mm high, and 500 mm wide. We used PHMT cured for 7 days, made following the procedure described in section 3.2. The PHMT layer was made when wet and was covered by the sand layer. We used silica sand \( \#4 (\rho_s = 2.644 \text{ g/cm}^3, \rho_{\text{mix}} = 1.342 \text{ g/cm}^3, \text{and } \rho_{\text{max}} = 1.618 \text{ g/cm}^3) \). The relative densities of the PHMT and sand were 50%.

The water used to make the layers was artificial seawater, except for case 4 (Table 2.), in which fresh water was used. The shape of each layer is shown in Fig. 6. After making the model ground, 6 standpipes were installed at the positions marked No. 1, No. 2, and No. 3 to collect pore water. Two pipes were installed at each location to collect water from different depths. The open circles in Fig. 6 show the points where pore water was collected. Water was supplied as shown in the upper right part of the figure at a rate of 6 l per day. Effluent flowed from the bottom of the apparatus as shown in the figure. Since the void space in the model ground layer was about 84 l, the hydraulic retention time of the water in the apparatus was 14 days. Each experiment was conducted at 20 degrees centigrade. As the room was not perfectly temperature-controlled, its temperature was somewhat affected by the outside temperature.

Figure 6 shows how the experiment was set up. The box holding the sand was 800 mm long, 500 mm high, and 500 mm wide. We used PHMT cured for 7 days, made following the procedure described in section 3.2. The PHMT layer was made when wet and was covered by the sand layer. We used silica sand \( \#4 (\rho_s = 2.644 \text{ g/cm}^3, \rho_{\text{mix}} = 1.342 \text{ g/cm}^3, \text{and } \rho_{\text{max}} = 1.618 \text{ g/cm}^3) \). The relative densities of the PHMT and sand were 50%.

The water used to make the layers was artificial seawater, except for case 4 (Table 2.), in which fresh water was used. The shape of each layer is shown in Fig. 6. After making the model ground, 6 standpipes were installed at the positions marked No. 1, No. 2, and No. 3 to collect pore water. Two pipes were installed at each location to collect water from different depths. The open circles in Fig. 6 show the points where pore water was collected. Water was supplied as shown in the upper right part of the figure at a rate of 6 l per day. Effluent flowed from the bottom of the apparatus as shown in the figure. Since the void space in the model ground layer was about 84 l, the hydraulic retention time of the water in the apparatus was 14 days. Each experiment was conducted at 20 degrees centigrade. As the room was not perfectly temperature-controlled, its temperature was somewhat affected by the outside temperature.

3.5 Effects of changing from sea water to fresh water on the solidification of PHMT-treated GBFS

Here, we address issue (5) described above. The follow-up survey about GBFS used as backfill noted in the introduction revealed that the pore water in the GBFS layer changed completely from seawater to fresh water over a period of 4 months (Kikuchi et al. 2005). This phenomenon occurs because the mean ground water level is higher than the mean sea level and rainfall supplies fresh water. Figure 1 shows that GBFS mixed with PBFS in seawater solidified in a month. With this in mind, we checked the effect of a pore water transition in a series of laboratory experiments.

Figure 6 shows how the experiment was set up. The box holding the sand was 800 mm long, 500 mm high, and 500 mm wide. We used PHMT cured for 7 days, made following the procedure described in section 3.2. The PHMT layer was made when wet and was covered by the sand layer. We used silica sand \( \#4 (\rho_s = 2.644 \text{ g/cm}^3, \rho_{\text{mix}} = 1.342 \text{ g/cm}^3, \text{and } \rho_{\text{max}} = 1.618 \text{ g/cm}^3) \). The relative densities of the PHMT and sand were 50%.

The water used to make the layers was artificial seawater, except for case 4 (Table 2.), in which fresh water was used. The shape of each layer is shown in Fig. 6. After making the model ground, 6 standpipes were installed at the positions marked No. 1, No. 2, and No. 3 to collect pore water. Two pipes were installed at each location to collect water from different depths. The open circles in Fig. 6 show the points where pore water was collected. Water was supplied as shown in the upper right part of the figure at a rate of 6 l per day. Effluent flowed from the bottom of the apparatus as shown in the figure. Since the void space in the model ground layer was about 84 l, the hydraulic retention time of the water in the apparatus was 14 days. Each experiment was conducted at 20 degrees centigrade. As the room was not perfectly temperature-controlled, its temperature was somewhat affected by the outside temperature.

During the experiment, pore water was collected from each point at designated times, and pH and salinity were measured. After 8 weeks, the strength of the PHMT-treated GBFS was measured with a Yamanaka soil hardness meter (Kikuchi et al. 2010). About 2000 strength measurements were made in each case. The data collected were converted to unconfined compression strengths using a relationship between strength and hardness determined before the experiment.

Table 1 shows the types of water supplied in each case.

<table>
<thead>
<tr>
<th>Case</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Sea water supplied for 8 weeks</td>
</tr>
<tr>
<td>Case 2</td>
<td>Sea water supplied for 6 weeks, then pure water supplied for 2 weeks</td>
</tr>
</tbody>
</table>
The solidification of GBFS used in port structures can be accelerated by adding PBFS. However, mixing PBFS with GBFS presents some issues when used in actual construction sites. To overcome these problems, we subjected the GBFS to PHMT treatment. This paper demonstrates the superiority of PHMT treatment. This paper demonstrates the superiority of PHMT treatment. PHMT reduces the amount of separation of the GBFS/PBFS mixture and produces sufficient unconfined compression strength after about 2 months of curing in seawater, which occurs automatically when GBFS is used to backfill quay walls. We conclude that PHMT-treated GBFS solidifies at an accelerated rate and can be used to prevent liquefaction.

4 CONCLUSION

The solidification of GBFS used in port structures can be accelerated by adding PBFS. However, mixing PBFS with GBFS presents some issues when used in actual construction sites. To overcome these problems, we subjected the GBFS to PHMT treatment. This paper demonstrates the superiority of PHMT treatment. PHMT reduces the amount of separation of the GBFS/PBFS mixture and produces sufficient unconfined compression strength after about 2 months of curing in seawater, which occurs automatically when GBFS is used to backfill quay walls. We conclude that PHMT-treated GBFS solidifies at an accelerated rate and can be used to prevent liquefaction.

5 REFERENCES


Building on an old landfill: design and construction

Construire sur une ancienne décharge : dimensionnement et exécution des travaux

McIntosh G.W.
Douglas Partners Pty Ltd, Unanderra, NSW, Australia
Barthelmess A.J.
Consulting Engineer, Wollongong, NSW, Australia

ABSTRACT: The Fo Guang Shan Nan Tien Buddhist Order commissioned the first stage of site investigations of the proposed Nan Tien Institute site at Unanderra, NSW Australia in 2000. Wollongong City Council donated the land which includes an old landfill. The site is located directly opposite the Nan Tien Buddhist Temple which opened in 1995 and is the largest Buddhist Temple in the southern hemisphere. The Institute is being developed in accordance with a masterplan which will ultimately cater for 3000 students and 360 staff. Geotechnical and environmental investigations have been undertaken to determine the extent of the remedial works that will be required for site development. The landfill (which closed in 1984), is some 5.7 ha in plan area and occupies nearly 50% of the total institute site. The results of the investigations have enabled geotechnical, environmental and civil design works to be completed for the initial stage of construction (the ground consolidation works) which were completed in March 2011. Stage 1 building works commenced in November 2012. Given within this paper are the investigation results, design overview and monitoring results (noise, odour, vibration, landfill gas and consolidation). Where relevant, comparison is given to predicted values.


KEYWORDS: landfill, dynamic compaction, methane drainage, leachate control, monitoring.

1 INTRODUCTION.

In September 2001, Wollongong City Council donated a 12 ha parcel of land at Unanderra to the Fo Guang Shan Nan Tien Buddhist Order on which is planned the Nan Tien Institute and Art Gallery. The site is opposite the existing Nan Tien Temple at Unanderra, NSW which was opened in 1995 and is the largest Buddhist Temple in the southern hemisphere. About half the Institute site includes a derelict (putricible waste) landfill which was operated by Wollongong City Council up until its closure in 1984.

Geotechnical and environmental investigations have been ongoing since early 2000 with the overall masterplan of the site finalised in 2009. The project architects were commissioned to produce an environmentally sustainable design with development of the site to be undertaken in stages. Stage 1 works include the Cultural Museum, some limited teaching facilities and car parking areas.

Discussed within this paper are details on the geotechnical and environmental investigations, civil design, leachate collection and control, earthworks (including dynamic compaction results achieved during ground consolidation works completed in March 2011), the use of coal washery rejects as a fill source and environmental monitoring undertaken during earthworks (air, noise, dust, odour, landfill gas and vibration). Discussion is also given on foundation systems and gas drainage options that will need to be developed within the design of the future buildings.

2 BACKGROUND.

The existing Nan Tien Temple and the proposed Institute site are located on either side of the F6 freeway at Unanderra, NSW (refer Figure 1). The overall site area is around 15 ha with the derelict landfill occupying about half of the total site area. The main challenge to development of the site is primarily two fold – firstly, the assessment of both short-term and long term consolidation of the waste and then the design of buildings and civil works that can withstand the settlement estimates and secondly, the design of a system that will enable collection, treatment and discharge of landfill gases (of which methane is the biggest concern) and leachate in a safe and environmentally acceptable way over the life of the buildings.

3 THE PROJECT

The proposed Nan Tien Institute is being developed in accordance with a Masterplan which will ultimately cater for 3000 students and 360 staff. It will be a mixed use development comprising formal educational facilities, an art gallery, museum and other cultural facilities. The overall budget for Stage 1 of the project (including remediation of the landfill) is around $30 million AUD. Whilst architectural design is a work in progress, the first stage of the Institute will generally occur over several levels on the site, with basement carparking on the lower levels, then teaching and related facilities to a viewing platform at the higher locations on the site.
4 SITE INVESTIGATION AND RESULTS

4.1 Geotechnical

Investigations to establish a geotechnical model of the site included 103 test pits to depths of up to 5 m, 14 cored boreholes to depths of up to 12 m and the installation of 8 standpipe piezometers. In summary, the natural geological profile of the site comprised topsoil over residual clays with latite and sandstone bedrock (generally of medium to high strength) below depths of 1 – 1.5 m. The profile of the landfill included a coal washery rejects (CWR) and clay capping layer some 0.5 – 3.5 m in thickness (but generally less than 1 m) with the depth of the waste in the order of 4 – 12 m.

The landfill waste was interbedded with CWR and clays, as was expected given the conventional operation of a putrescible waste facility. The density of the landfill was generally loose with some denser sections as reflected by standard penetration test “N” values in the range 2 – 30. Perched water tables were also present. The extent of the landfill is shown in Figure 1.

4.2 Environmental (soil, water, air, noise)

180 test locations were investigated across the site, most of which were in the landfill footprint. Contaminant concentrations were compared to the NSW DECC (2006) Health based Investigation levels. Within the soils, elevated levels of manganese and hydrocarbon (C10 – C36) were recorded. Testing of groundwater indicated elevated levels of iron, manganese, ammonia, nitrate and total phosphorus, typical of levels and contaminants found in landfill leachate. Methane, hydrogen sulphate and carbon dioxide were recorded in the gas monitoring wells with the methane levels within either the “explosive” range or exceeding the “explosive limits” and in a range that may cause asphyxiation.

In the areas outside the landfill footprint, no environmental concerns were recorded apart from random dumping of uncontrolled fill which was managed by conventional construction practices.

5 GROUND CONSOLIDATION WORKS

Site preparation was completed in March 2011 and included construction of a temporary leachate collection system, reshaping and benching of most of the site, dynamically compacting the landfill and undertaking of controlled earthworks to achieve design levels. Monitoring of air quality, noise, vibration levels and leachate was ongoing during the works.

5.1 Civil Design and Leachate Control

During initial site works, the expectation was that a relatively significant quantity of leachate would discharge from the landfill cell which would reduce after dynamic compaction. The reduced quantities were expected to be treated and managed long term by a membrane bio-reactor (prior to discharge off site or re-use on site). As the bio-reactor could not be sized to cater for the high loads during site preparation works, a 2ML leachate pond was constructed downslope of the landfill cell. Leachate was fed into the pond via a 2 m groundwater cut-off trench installed around the toe and flanks of the landfill cell. Once in the pond, leachate was then pumped through a treatment system consisting of pumps, sand filters, activated carbon filters and an automatic sampler prior to discharge into the sewer system via a Trade Waste Agreement with Sydney Water.

The leachate pond was designed to not only suit its purpose during dynamic compaction and site preparation (i.e. as a leachate pond), but to also double as an on-site detention (OSD) pond during the life of the Institute. This OSD pond assists with long-term management of stormwater on the site. The HDPE liner installed in the leachate pond during dynamic compaction was removed and the pond readily transformed for the OSD purpose. This saved having to build two very similar structures twice.

5.2 Dynamic Compaction

In order to improve the density of the landfill (and thus to improve longer term performance by limiting primary compression and secondary consolidation following progressive waste decomposition), dynamic compaction was selected as the appropriate method. The equipment (shown in Figure 2) included a 120 tonne crawler crane dropping a 25 tonne pounder from a height of (nominally) 20 m. Compaction was carried out in two phases. Following placement of a coarse “compaction layer” to provide stability for the crane, the primary phase comprised multiple drops of the concrete pounder (typically 3 – 4) on a 6 m x 6 m grid with the craters backfilled as the compaction proceeded. The final (or ironing) phase was carried out using a pounder of similar mass but a larger footprint (5 – 9 m2) with a drop height adjusted to the pounder size and compression achieved.

Using the methods of Hausmann (1990), an assessment was made of the degree of ground improvement with surface settlements of generally 1 – 2 m expected in the areas underlain by the deeper landfill. The survey results following completion of dynamic compaction and were predominantly within the range 0.5 – 1.5 m, in line with expectations and generally 10 – 12% of overall landfill depth.
Following backfilling of the craters, earthworks were undertaken to reshape the surface of the various benches after which conventional fill placement was carried out to achieve design levels. In areas, this required the placement of up to 2 m of compacted fill which was placed under Level 1 geotechnical control to the requirements of Australian Standard AS3798 – 1996. As a result of the success of the dynamic compaction phase (which provided a solid base), the undertaking of additional earthworks was relatively straightforward with a compaction requirement of 100% of standard maximum dry density achieved in all fill areas. Fill materials included the use of coal washery rejects, a mining by-product from the coal washing process that is obtained at low cost (typical transport only) but has very good civil engineering properties for use as general fill and no negative environmental impacts.

5.3 Site Monitoring

Construction works for the ground consolidation contract were undertaken in accordance with a Construction Environment Management Plan (CEMP). The key objective of the CEMP was to develop a monitory programme for regulatory compliance and early detection of any significant environmental or community impacts.

Given the potential impacts due to dynamic compaction being carried out on the site and the presence of buildings on neighbouring properties, vibration trials were undertaken prior to commencement of compaction. An attenuation graph was prepared as shown in Figure 4 with a boundary buffer distance of 25 m nominated for a proposed vibration limit of 8 mm/sec (sector sum and component peak particle velocity). Texcel Vibration Monitors were installed for continuous data recording (one of which was in a neighbouring building) and adopting the buffer distances established by the trial, only nine exceedances were recorded during the 6 month construction period. No complaints were received from neighbouring properties.

![Dynamic Compaction Vibration Attenuation Graph](image)

Figure 4: Dynamic Compaction Vibration Attenuation Graph

Whilst noise was considered to be the other major environmental impact that could cause community concern during compaction activities, monitoring over the 6 month period recorded a total of only 32 readings above the performance criteria of 75 dBA. Odour was primarily of concern during the initial excavation phase and was managed by minimising waste exposure time. Similarly, dust was managed by the implementation of good construction practices on site. Leachate and groundwater was monitored regularly with all outflow to the pre-determined requirements. Whilst results were typical of those expected from a landfill site, manganese and ammonia were flagged as elements of concern.

The obvious area of concern in all landfill projects is landfill gas (LFG). Methane, carbon dioxide and oxygen levels were monitored both inside and outside the landfill boundary as well as within site buildings. Daily monitory of landfill was undertaken using a GA2000 Gas Meter. Both surface and well measurements were taken as well as barometric pressure and lower explosive limit. Peak methane levels of up to 97% were recorded in wells in the landfill footprint, with levels generally in the range of 14 – 50%. Monitoring in wells adjacent to the landfill boundary was generally below threshold levels or 0% methane. Surface and enclosed space monitoring showed that LFG was not considered to be an issue at any time during the works.

6 FUTURE WORKS AND BUILDING DESIGN

6.1 Civil works, services and stormwater drainage

All civil and building services (eg sewer, water, stormwater, electrical, gas) have been designed such that they will not need to penetrate the capping layer of the landfill. All service trenches and other works that require excavation (eg landscaping) will be within ‘clean’ material and limited to excavation depths of 2m. Earthworks associated with site reshaping will require construction of retaining walls up to 7m high. The walls have been designed as reinforced earth structures able to accommodate ground settlements of 300mm.

6.2 Foundations

The main advantage of dynamically compacting the landfill is that long term settlement of the landfill (post building construction) will be significantly reduced, but not eliminated (Thom 1998). As such, footing design for buildings located within the landfill footprint will be for driven steel piles founding in the underlying latite bedrock. Flexible aprons will be needed between the buildings (which will experience negligible settlement) and adjoining carparks, walkways and recreation areas (which will experience ongoing settlement). Whilst raft slabs may be feasible for some lightweight single story buildings, preliminary analysis has indicated that a 1 m thick reinforced earth raft will be needed to provide uniform bearing and to equalise the longer term settlements so that differential movements will be within acceptable limits.

6.3 Leachate Control and Gas Drainage

Leachate collection drains will be installed across the site and directed to the leachate treatment system. The current options for leachate collection include disposal to sewer, reinjection, spray or drip irrigation, removal by contractor, ammonia stripping, constructed wetlands and membrane bio reactor.

The primary elements of the environmental design are capping profile, methane drainage and leachate control. The requirement of the site capping is twofold; firstly – physical separation by covering contaminated materials and secondly – prevention of infiltration to the substrate, thereby minimising leachate recharge and mobilisation and upward migration of methane. Historically landfill capping systems have included a 0.5 m clay cap however this system alone was not considered intrinsically safe at this site in areas underneath buildings or pavements where piles will breach the cap and gas can accumulate in enclosed spaces.

The preliminary design for the capping consists of HDPE, GCL, geotextile fabric, 300 mm gravel gas drainage layer and a reinforcing geotextile, underlain by the existing waste, refer to Figure 5 below. Undercroft will be constructed where possible to allow for suspension of services and cross-ventilation. In areas outside of the buildings an additional 1 m layer of clean fill material to further protect the cap from stormwater and root infiltration, drying out, cracking and accidental breaches will be installed. The preliminary design requires the landfill cap to
extent 50 m beyond the landfill boundary or the site boundary whichever occurs first. At the landfill boundary within the site, the capping will be keyed in with a sump installed for leachate and landfill gas condensate collection.

Where the landfill extends over the site boundary a bentonite plug will be installed at the boundary to minimise migration of landfill gases through soils, refer to Figure 6.

Landfill gas drainage will consist of a gas drainage layer forming part of the cap. Collection pipes will be placed around the perimeter and across the benches with a maximum spacing of 50 m. An additional gas extraction filter layer and pipe work for the collection and discharge of LFG will be incorporated beneath the slab of all buildings where possible. Three remedial options for the management landfill gas were considered. The use of landfill gas for power generation was not considered feasible due to low gas flows as result of the age and stage of the decomposition of the waste present at the site this option was considered, given its advantage of reducing the landfill gas to a higher percentage of CO₂ and H₂O vapour, however the technical difficulties of operating the flare, the area required for a flare plant and the cost of setting up and maintenance of the plant, among others, far outweighed the advantages. The final remedial option, venting to the atmosphere, was chosen for the zero requirements for a specific treatment plant and operating costs. As such the preliminary design consists of the placement of turbine ventilator stacks around the site. Landfill gas discharge will occur through stacks that will extend 1 m above the proposed maximum building roof level across the site. To monitor the landfill gas and minimise the potential of migration off-site a series of monitoring wells will be installed.

7 LESSONS LEARNT DURING CONSTRUCTION

Of particular interest to those wanting to apply the techniques described in this paper for another site or project, are the lessons learnt during the construction of this project. Primarily, these are as follows:

1. The site was located within 50m of a main highway. A hazard was ‘fly rock’ being mobilized during dynamic compaction and hitting operators or leaving the site and colliding with a vehicle on the highway. To control this hazard, a no-go zone was established around the dynamic compaction rig and the rig was not allowed to operate near the highway.

2. The extent of consolidation during dynamic compaction was significant. At this site, the contaminated water make during dynamic compaction was also significant. Whilst this was predicted and readily catered for onsite (in accordance with the CEMP) it took a considerable amount of time to manage, sample for contamination and then subsequently discharge at an appropriate location. It was also relatively expensive.

3. The use of coal washery rejects, a mining by-product from the coal washing process, was entirely successful. This material was put to use on this site and would otherwise have ended up as landfill. The ability to use what would otherwise have been waste material, as fill material in the overall remediation of another landfill site, is considered best practice and an outstanding outcome environmentally.

4. The selection of an earthworks contractor must include an assessment of their ability to perform the work rapidly. Exposing the waste sections of the site created many hazards, and the contractors ability to perform the work and ‘cap’ the site ready for dynamic compaction in the shortest possible timeframe greatly reduces the exposure to those hazards and any expensive delays caused by (for example) inclement weather.

8 CONCLUSION

The use of dynamic compaction and construction of a landfill gas drainage system together with innovative civil solutions will allow the Fo Guang Shan Nan Tien Buddhist Order to develop their site and create a teaching and cultural facility as part of the existing Nan Tien Temple. Geotechnical and environmental performance was monitored during site preparation works and will be monitored during building construction.

9 ACKNOWLEDGEMENTS

The authors acknowledge the Fo Guang Shan Nan Tien Buddhist Order and affiliates for assistance in the project and their permission to use project specific data for this paper.

10 REFERENCES


Interpretation of mechanical behavior of cement-treated dredged soil based on soil skeleton structure

Interprétation des comportements mécaniques des sols dragués traités au ciment basée sur la structure squelette du sol

Nakano M., Sakai T.
International Member, Nagoya University, JAPAN

ABSTRACT: The objective of this study was to examine the mechanical behaviors of cement-treated dredged soil and evaluate them based on action of the soil skeleton structure through simulation of the behaviors by the SYS Cam-clay model. Besides, the behaviors were simulated by the GEOASIA soil-water coupled finite deformation analysis code. The new findings are summarized as follows; 1) As the soil is treated with high cement with low initial water content under shearing in high confining stress, its effective stress path moves up closer to the tension cut-off before failure. 2) The treated soil approaches the NCL of the remolded cement-treated soil. 3) The treated soil is regarded as a high structure and overconsolidated soil. 4) FEM analysis can describe softening behavior with shear banding through the triaxial compression test.

RÉSUMÉ: L’objectif de cette étude est d’examiner les comportements mécaniques des sols dragués traités au ciment et de les évaluer sur la base de l’action de la structure squelette du sol par la simulation des comportements à l’aide du modèle SYS Cam-clay. Par ailleurs, les comportements sont simulés par GEODESIA, un programme d’analyse de déformations finies sol-eau couplées. Les nouveaux résultats sont résumés comme suit; 1) Comme le sol est traité avec du ciment haut en eau initiale faible avec cisaillement sous contrainte de confinement élevé, son chemin de contrainte efficace se déplace plus près de la tension de coupure avant la rupture. 2) Le sol traité se rapproche de la NCL du sol traité au ciment et remoulé. 3) Le sol traité est considéré comme une structure haute et surconsolidée. 4) L’analyse par éléments finis peut décrire des comportements d’adoucissement avec bande de cisaillement lors d’essai de compression triaxiale.

KEYWORDS: cement stabilization, soil skeleton structure, elasto-plastic mechanics.

1 INTRODUCTION

About 1.3 million m³ of dredged soil is produced annually in Nagoya Bay. However, the temporary storage capacity for the soil at Nagoya Port Island (PI) is limited, so effective use of the soil as a geomaterial has become a pressing issue. The water content of the soil is high, and the unconfined compressive strength is low, so to effectively use the soil as a geomaterial, it is necessary to add a stabilizer such as cement to improve the mechanical properties. Therefore, in this study, the mechanical behavior of cement-stabilized dredged soil (hereafter referred to as treated soil) was determined using laboratory tests and reproduced using an elasto-plastic constitutive model, with the objective of explaining the improvement effect.

Past constitutive equation study into cement-stabilized soil includes, for example, the study by Hirai et al. (1989), Yu et al. (1998), Kasama et al. (2000), Lee et al. (2004), and Wada et al. (2004). This study used the SYS Cam-clay model (Asaoka et al. 2002), an elasto-plastic constitutive model based on the action of the soil skeleton structure. It was assumed that the mechanical behavior for the criteria to define the soil skeleton structure was the mechanical behavior obtained from remolded samples of treated soil (hereafter referred to as remolded treated soil). The one-dimensional compression behavior was reproduced in addition to the shear behavior in order to explain the improvement effect of adding cement based on elasto-plastic mechanics, taking the soil skeleton structure into consideration. In addition, the effect of nonuniform deformation on triaxial test results was investigated by solving as a boundary problem (Asaoka et al. 1995), taking into consideration brittle behavior, which is a characteristic of the treated soil that was observed in the tests, in addition to the constitutive equation response considering the triaxial test to be an element test.

1 THE SYS CAM-CLAY MODEL

This section describes the SYS Cam-clay, the elasto-plastic constitutive model that was used to explain the mechanical behavior of the treated soil. Fig. 1 shows the oedometer test results for natural deposited clay and remolded clay. Natural deposited clay is defined as clay with structure, where the difference in specific volume \( v = 1 + e \) (where \( e \) is the void ratio) from remolded clay at the same vertical stress, in other words, the “bulk” is taken to be the extent of structure. As the vertical stress increases, the compression line of the natural deposited clay approaches that of remolded clay. The interpretation of this behavior in terms of the concept of soil skeleton structure is that there is decay/collapse of the soil structure due to shearing (plastic deformation) of the soil. A triaxial compression test is not shown here, but the critical state of natural deposited clay gradually approaches that of remolded clay as a result of shear. Basically, the structure collapses due to shear. Likewise, overconsolidation becomes normal consolidation as a result of
plastic deformation. The SYS Cam-clay model defines structure, overconsolidation, and anisotropy as the soil skeleton structure, and an evolution rule is introduced that varies them in accordance with the plastic deformation to reproduce the mechanical behavior of natural deposited clay. This study focused on structure and overconsolidation, controlling the ease of change of their states using a degradation index of structure and a degradation index of OCR in accordance with their respective evolution rules (see Table 3), to explain the treated soil as natural deposited clay and the remolded treated soil as remolded soil.

Table 1 Physical properties of dredged soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil particle density $\rho_s$ [g/cm$^3$]</td>
<td>2.67</td>
</tr>
<tr>
<td>Natural water content $w_0$ [%]</td>
<td>50–110</td>
</tr>
<tr>
<td>Liquid limit $w_l$ [%]</td>
<td>52.5</td>
</tr>
<tr>
<td>Plastic limit $w_p$ [%]</td>
<td>25.1</td>
</tr>
<tr>
<td>Plasticity index $I_p$ [%]</td>
<td>27.4</td>
</tr>
<tr>
<td>Clay content [%]</td>
<td>60</td>
</tr>
<tr>
<td>Silt content [%]</td>
<td>36.6</td>
</tr>
<tr>
<td>Sand content [%]</td>
<td>3.4</td>
</tr>
<tr>
<td>Mean grain diameter $D_{50}$ [mm]</td>
<td>0.002</td>
</tr>
</tbody>
</table>

2 REPRODUCTION OF THE MECHANICAL BEHAVIOR OF TREATED SOIL WITH SYS CAM-CLAY MODEL

2.1 Physical properties of dredged soil and treated soil mixture conditions

Table 1 shows the physical properties of PI dredged soil. Almost all of the soil is fine fraction with a high natural water content, and it does not achieve the required strength. Also, Table 2 shows the mixture conditions of the treated soil. In this study, the mixture conditions were assumed to be a water content $w=120\%$, cement contents of $C=30$, 50, and 70 kg/m$^3$, and 28 days’ curing in order to ensure fluidity and strength for the assumed Pneumatic Flow Method (Coastal Development Institute of Technology 2008).

Table 2 Mixture conditions used in the comparison of cement content

<table>
<thead>
<tr>
<th>Dredged soil water content $w_0$ [%]</th>
<th>Cement content $C$ [kg/m$^3$]</th>
<th>Water-cement ratio $W/C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>30</td>
<td>25.2</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>15.0</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>10.6</td>
</tr>
</tbody>
</table>

Compared with the remolded treated soil, there is a certain amount of structure up to the pseudo consolidation yield stress, and when the vertical stress increases further, it gradually approaches the compression line of the remolded treated soil. Fig. 3 shows the CU triaxial test results for the treated soil, and Fig. 4 shows the results for the remolded treated soil. The behavior exhibited in Fig. 3 resembles the behavior of overconsolidated and high structured clay (Asaoka et al. 2002). As shown in Fig. 4, the behavior of remolded treated soil resembles the behavior of dredged soil. However, the treated soil has a specific volume that is distinctly higher than that of dredged soil, so in Figs. 2 and 4, it is considered that remolded treated soil is a material that is different from dredged soil.

2.2 Mechanical behavior of treated soil ($C=50$ kg/m$^3$) and remolded treated soil and material constants

Fig. 2 shows the oedometer test results for treated soil and remolded treated soil with $C=50$ kg/m$^3$, together with the results for dredged soil. Compared with dredged soil, treated soil has a high initial specific volume, and as a result of cement addition, it maintains the high specific volume state up to a certain vertical stress. When the vertical stress exceeds a pseudo consolidation yield stress, a high compressibility is exhibited.
and a high overconsolidation ratio. As the cement content is increased, the maximum value of the stress ratio \( q/p' \) easily exceeds \( M \), and for \( C=70 \) kg/m\(^3\), the effective stress reaches the tension cut-off line (\( q = 3p' \)). The analysis reproduced the behavior of the treated soil for low cement content, but it was difficult to reproduce the behavior above \( q = M_p' \) for treated soil with a high cement content.

Table 3 shows the initial values of the material constants of the SYS Cam-clay model used in the analysis. The addition of cement produced high structure and pseudo overconsolidation. Also, the overconsolidation ratio increased as the cement content increased, but on the other hand, the extent of evolution of structure reduced. This is considered to be due to the fact that the water content of the dredged soil was constant in the mixture conditions used, so as the cement content increased, the water-cement ratio reduced, and this corresponds to the increase of \( N \) of the remolded treated soil as the cement content increased.

Table 3 SYS Cam-clay model material constants and initial values

<table>
<thead>
<tr>
<th>Plasticity parameters</th>
<th>Treated soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement content (kg/m(^3))</td>
<td>( C=30 )</td>
</tr>
<tr>
<td>Water-cement ratio</td>
<td>25.2</td>
</tr>
<tr>
<td>Confining pressure (kPa)</td>
<td>98.1</td>
</tr>
<tr>
<td>Compression index</td>
<td>( \lambda )</td>
</tr>
<tr>
<td>Swelling index</td>
<td>( \kappa )</td>
</tr>
<tr>
<td>Limit state constant</td>
<td>( M )</td>
</tr>
<tr>
<td>NCL intercept</td>
<td>( N )</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>( \nu )</td>
</tr>
</tbody>
</table>

Evolution rule parameters

| | Normal consolidated soil index | \( m \) | 0.01 | 0.60 | 5.00 |
| | Structure degradation index | \( a \) | 0.25 | 0.60 | 1.50 |
| | | \( b \) | 1.00 | 1.00 | 1.00 |
| | | \( c \) | 1.00 | 1.00 | 1.00 |
| | Plastic shear:plastic compression | \( c_s \) | 0.20 | 0.50 | 0.10 |
| | Rotation hardening index | \( b_r \) | 0.00 | 0.00 | 0.00 |
| | Rotation hardening limit index | \( m_b \) | 0.50 | 0.50 | 0.50 |

Initial values

| | Overconsolidation ratio | 1/\( R_o \) | 1.03 | 20.9 | 63.8 |
| | Extent of structure | 1/\( R_o \) | 260 | 10.00 | 5.00 |
| | Vertical stress | \( \sigma_v \) | 19.6 | 19.6 | 19.6 |
| | Specific volume | \( \nu \) | 3.94 | 3.75 | 3.80 |
| | Stress ratio | \( \eta_b \) | 0.00 | 0.00 | 0.00 |
| | Initial anisotropy | \( \zeta_o \) | 0.00 | 0.00 | 0.00 |

3 SOIL-WATER COUPLED FINITE DEFORMATION ANALYSIS FOR TRIAXIAL TESTS

In the above, we attempted to explain the mechanical behavior of treated soil from the point of view of considering the triaxial test as an element test. However, from observation of the failure mode of the test specimens, the characteristic brittle failure had occurred in the treated soil. Therefore, in this section, the effect of nonuniform deformation on the triaxial test results was investigated, taking the triaxial test to be a boundary problem.

3.1 Analysis conditions for the soil-water coupled finite deformation analysis

The soil-water coupled finite deformation analysis code GEOSIA (Noda et al. 2008), which incorporates the SYS Cam-clay model as the constitutive equation for soil structure, was used in the analysis. The analysis was carried out under plane strain conditions, and Fig. 9 shows the finite element mesh and boundary conditions. An undrained boundary was set around the test specimen, and frictional conditions were assumed at the top and bottom end surfaces with a rigid cap and pedestal. A primary asymmetric mode with a cosine curve (half period) having an amplitude of 0.005 cm (Asaoka et al. 1995)
was applied as an initial geometric imperfection to the side surfaces of the test specimen. The shear velocity was $1.4 \times 10^{-6}$ m/s, with a downward displacement velocity applied to the top end surface of the test specimen. The target test was $C=50$ kg/m$^2$ under confining pressure 294 kPa as shown in Fig. 3. The triaxial test was considered to be a boundary problem, and the calculation was carried out using the material constants and initial values for the SYS Cam-clay model (Table 3).

Figure 9. Finite element mesh and boundary conditions

Figure 10. Consolidated undrained triaxial test results for remolded treated soil

Figure 11. Consolidated undrained triaxial analysis results for remolded treated soil

Photograph 1 Failure shape of the test specimen

3.2 Soil-water coupled finite deformation analysis results

Fig. 10 shows the analysis results arranged considering the test specimen to be one element, together with the test results. Fig. 11 shows the shear strain distribution from the analysis. From the axial deviator stress-axial strain relationship in Fig. 10, it can be seen that at around 3% of axial strain and at around more than 6% of axial strain, the deviator stress suddenly drops. In the shear strain distribution in Fig. 11, 'diagonal shear band' occurs at about the same axial strain as in Fig. 10, then a shear band in the opposite direction occurs, and finally X-shaped shear bands are formed. The occurrence of shear bands and the drop in $q$ coincide, so it can be seen that the cause of the drop in $q$ is the occurrence of shear bands. In the test results, a clear load drop occurs at around 7–8% axial strain, but at around 5%, a small load drop can also be seen. Photograph 1 shows a view of the test specimen after shearing. X-shaped shear bands are formed as in the analysis.

4 CONCLUSIONS

We attempted to explain the mechanical behavior and improvement effect of treated soil due to the addition of cement based on test results, the SYS Cam-clay model, which is an elasto-plastic constitutive model that incorporates the concept of soil skeleton structure, and GEOASIA. The following conclusions were obtained.

(1) Mechanical behavior of cement-stabilized treated soil: In the oedometer tests, as the cement content increased, the initial specific volume increased, the consolidation yield stress increased, and the compressibility was smaller up to the consolidation yield stress. In the triaxial tests, as the cement content increased, the maximum value of the stress ratio $q/p'$ increased and approached the tension cut-off line.

(2) Mechanical behavior of remolded treated soil: In the oedometer tests, as the cement content increased, the intercept N and the slope $\lambda$ of the NCL increased. In the triaxial tests, the M did not vary much with cement content.

(3) Reproduction using the SYS Cam-clay model: The addition of cement produces a higher structure and pseudo overconsolidation in the soil. Also, differences in cement content are easily reflected in differences in the overconsolidation ratio, and differences in water-cement ratio are easily reflected in the degree of structure. The analysis reproduced the mechanical behavior of treated soil, but for high cement contents, reproduction by analysis was difficult, which suggests that it is necessary to introduce a new model.

(4) Finite element analysis of the triaxial test: Although there were differences in the axial strain at occurrence of shear banding and the amount of drop in $q$, the analysis was capable of reproducing the trends of both occurrence of shear banding and the sudden drop in $q$. However, material constants and initial values used in the analysis were obtained by considering the triaxial test to be an element test. It is necessary to incorporate the viewpoints of both element tests and boundary problems in order to comprehend the natural behavior of the treated soil.

5 REFERENCES


Noda et al. 2008. Soil-water coupled finite deformation analysis based on a rate-type equation of motion incorporating the SYS Cam-clay model. S&F 48 (6), 771-790.
Utilization of waste copper slag as a substitute for sand in vertical sand drains and sand piles

Utilisation des scories de cuivre en tant que substitut pour le sable dans des drains et des colonnes de sable

Nawagamuwa U.P.
University of Moratuwa, Sri Lanka

Senanayake A.
University of Texas at Austin, USA

Rathnaweera T.
University of Calgary, Canada

ABSTRACT: Vertical sand drains are used as a method of expediting consolidation for ground improvement projects. Unfortunately, the installation of vertical sand drains have become less economically viable due to the high costs and limited availability of good quality sand. Particle size distribution analyses done on samples of waste copper slag obtained from the Colombo dockyard revealed that its gradation was similar to that of sand, which meant that waste copper slag could potentially be used as a substitute for sand, provided that it did not adversely affect the hydraulic conductivity of the resulting mixture. In this study, constant head permeability tests were done on “sand-copper slag” mixes of varying proportions and it was shown that up to 50% copper slag by weight could be added to sand without an appreciable loss in permeability. The performance of sand piles is dependent on strength and settlement characteristics of the sand. Hence, consolidation tests and direct shear tests were also carried out on the “sand-copper slag” mixes to explore how the mechanical properties of sand were affected by the copper slag.

RÉSUMÉ : Sable drains verticaux sont utilisés en tant que méthode d'accélérer la consolidation des projets d'amélioration des sols. Malheureusement, l'installation de drains verticaux sable sont devenues moins rentables en raison des coûts élevés et une disponibilité réduite de sable de bonne qualité. Analyses granulométriques effectuées sur des échantillons de scories de cuivre obtenu à partir des déchets du chantier naval Colombo a révélé que sa gradation était semblable à celle du sable, ce qui signifie que scories de cuivre des déchets pourrait être utilisé comme un substitut pour le sable, à condition que cela ne nuise pas la conductivité hydraulique du mélange résultant. Dans cette étude, des essais de perméabilité constants ont été réalisés sur la tête "sable-laitier de cuivre" mélange des proportions variables et il a été montré que jusqu'à scories de cuivre 50% en poids peuvent être ajoutés au sable sans perte notable de la perméabilité. La performance des piles de sable dépend des caractéristiques de résistance et de règlement du sable. Ainsi, des essais de consolidation et essais de cisaillement direct ont également été menées sur le sable-laitier de cuivre mélange d'explorer la façon dont les propriétés mécaniques de sable ont été affectées par les scories de cuivre.

KEYWORDS: waste copper slag, vertical drains, sand piles,

1 PARTICLE SIZE GRADATION

Laboratory sieve analyses were conducted on the waste copper slag. The material was first sieved through the No.4 (4.75mm) sieve to remove coarse particles which do not fall within the particle size range of sands. The results showed that the copper slag had a very small range of particle sizes. The material could be categorized as “poorly graded” according the USCS (Cu= 2.5 and Cc = 0.9).

The waste copper slag was mixed with a poorly graded sand (Cu =3, Cc =1.1) in proportions of 10%, 20%, and 40% by weight and tested how the geotechnical engineering properties of the sand would be affected.

The results of sieve analysis test conducted on the copper-sand mixes are shown in figure 1. It can be observed that the gradation curve is not greatly affected by the addition of waste copper slag. The only apparent change is a slight shift in the curves towards the right.

2 HYDRAULIC CONDUCTIVITY

A poorly graded material will have a high void ratio as compared to a well graded material. Whilst this may be a undesirable attribute in many engineering applications, it can be an advantage if the material is to be used for sand drains. The purpose of sand drains is to provide a mechanism to expediting the dissipation of excess pore water pressures created in soil
masses by acting as outlets for ground water to flow out. A key attribute of sand drains is to have very high hydraulic conductivities so that there is minimal resistance to water flowing them. High void ratios generally translate into high hydraulic conductivity thus, a material with high void ratio would be ideal for used in such drains.

The particle size gradation curves suggest that the hydraulic conductivity of the mixes should not deviate too much from that of the original sand. A series of constant head test were carried out on each mix in order to check if this was true.

The permeameter was filled with air dried samples of the copper-sand mix by dropping the material through a funnel from a fixed height. Special care was taken during the preparation of the specimens in order to ensure that they were comparable and consistent among all the different copper-sand mixes. The funnel used in this case was one from a “Sand Cone” test used in estimating field compaction values and it was placed on top of the permeameter. The funnel was first filled to the brim and the tap was opened to let all the material fall in to the permeameter in a single step. This method could be estimated to produce samples slightly denser than those specified in ASTM D 4254. The unit weight of the specimen was found by weighing the permeameter and calculating the volumes occupied by the material.

Specific gravity of the waste copper slag was found to be 3.7 and that of the sand was around 2.65. The specific gravity of the copper-sand mixes were calculated as a weighted average based on the proportions of each component. The void ratio of each mix was then calculated. As expected, the void ratio are tightly grouped together, ranging between 0.7 – 0.8. There is a trend showing a peak void ratio at a mix proportion of 20% copper slag. However, these small differences in void ratios are not significant enough to affect the hydraulic conductivity of the materials as can be seen from Figure 2.

3 SHEAR STRENGTH

The samples which were tested in the direct shear apparatus were prepared in ‘loose condition’ with no compacting. As expected, the plots on Figure 3 are typical of ‘loose’ sand as they show a gradual gain in shear strength and then flatten out with no pronounced peak. It can also be seen that the axial strain required mobilizing maximum shear strength increases with the increase in the applied normal stress. Figure 6 shows the summary of Mohr Coulomb failure envelopes for various copper sand mixes and Table 1 provides friction angles those mixes.

The direct shear test results show that the addition of waste copper-slag does not affect the shear strength of the sand. A friction angle of 30 degrees is typical for a ‘loose’ sand and if need be, the friction angle can be increased further by densifying the material.

Table 1: Friction Angles of Copper-Sand Mixes in ‘Loose’ State

<table>
<thead>
<tr>
<th>% of Copper Slag by Weight</th>
<th>Friction Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>31.4</td>
</tr>
<tr>
<td>20</td>
<td>30.4</td>
</tr>
<tr>
<td>40</td>
<td>30.4</td>
</tr>
</tbody>
</table>
4 STIFFNESS

The most significant effect of the addition of waste copper slag to sand was observed in confined compression tests. Dry specimens were tested in a consolidation apparatus to ascertain settlement and stiffness properties of the material. Standard consolidation tests with fully saturated samples were not warranted as the material had a very high hydraulic conductivity and consolidation would have occurred at a rapid rate. Strains were measured for different stress changes by applying loads on to the sample. Figure 5 shows the results in the form of a typical strain vs log stress plot. There is clear distinction between the behavior of the sand and the copper-sand mixes. The addition of just 10% of waste copper slag drastically increases the stiffness of the material. All three curves for copper-sand mixes are tightly grouped which suggest that increasing the proportion of copper slag more than 10% does not have any further effect.

Figure 5: Compression Characteristics of Copper-Sand Mixes

5 GEOCHEMICAL CHARACTERISTICS OF USED THE COPPER SLAG

Chemical properties of the used copper slag as a percentage of total weight are Iron Oxide-Fe₂O₃ 55%, Silica-SiO₂ 35%, Aluminium Oxide-Al₂O₃ 3.01%, Calcium Oxide-CaO 0.20%, Magnesium Oxide-MgO 0.90%, Copper-Cu 0.42%, Titanium Di-oxide 0.60%, and Potassium Oxide 1.02% (Hammarstrom et al.1999). The geochemical characteristic of used copper slag can be analyzed for its element content, pH, acid neutralization capacity (ANC), redox potential (Eh), and electrical conductivity (EC). According to the previous research done by Lim et al. (1997), it is clear that the pH of leachate generated from the used copper slag is around 8.4. This pH value is within the common pH range for soils and groundwater. Figure 6 shows the variation of pH- acid titration curves for used copper slag and natural soils (Moyer et al. 2000).

Figure 6: pH- acid titration curves for used copper slag and natural soils (After Moyer et al. 2000)

Figure 7 clearly illustrates that the used copper slag has a rather low ANC in comparison with the clay. Sand has practically negligible ANC. The low ANC value indicates that the spent copper slag is not resistant to acid attack. The concentration of Pb, Cd, Cr, Ba, As, Ag, Se, Cu, Zn, Ni and Hg in the used copper slag leachate are fairly low. The leachability of Cu and Zn metals are much higher when compare with Cd, Pb, Cr and Ni. The impact of the heavy metals leachability would be nullified by dilution process under larger water: slag ratio. Another important property of the used copper slag is the Eh value. The initial Eh value for the used copper slag is 171 mV at a solid: water ratio of 1:1. The Eh would continue to decrease rapidly days after placing the copper slag. Due to the presence of sulphide minerals, the used copper slag can be oxidized under oxic condition and release H⁺ into the pore water. As a result, there is a marginal drop in pH. The reduction can be expressed by the following reactions such as reactions of pyrite and ferrous. The amount of H⁺ generated from this reaction is very low and do not have enough reaction power to make significant changes in double layer of clay minerals.

Pyrite oxidation:

\[ 4FeS_2 + 14O_2 + 4H_2O \rightarrow 4Fe^{2+} + 8H^+ + 8SO_4^{2-} \]

Fe (11) oxidation:

\[ 4FeSO_4 + O_2 + 10H_2O \rightarrow 4Fe(OH)_3 + 8H^+ + 4SO_4^{2-} \]

The variation in pH due to present of heavy metals can be affected to the groundwater pollution scenario. According to the previous research done on used copper slag, it is clear that the effect of groundwater pollution scenario is very unlikely to occur (Lim et al. 1997).

6 CONCLUSIONS

Results have show that the hydraulic conductivity of the tested sand was hardly affected by the addition of waste copper slag due to the void ratio and the hydraulic conductivity of the waste copper slag itself being very similar to that of the sand. Investigation of the geochemical characteristics of the used copper slag alleviates the concern of possible groundwater pollution by its use. Therefore, it can be concluded that used copper slag can safely and effectively be used as a replacement for sand in vertical drains.

The shear strength of properties of the tested sand-copper slag mix was found to be very insensitive to the amount of
waste copper slag in the mix. In a “loose” state the sand-copper slag mix shows friction angles of 30.4° – 31.4°, which is a deviation of only 1°, even when the percentage by weight of waste copper slag changes from 0 to 40.

The stiffness of the sand was found to be clearly improved by the addition of waster copper slag. The addition of waste copper slag substantially reduced the settlement of the mix when tested in a conventional consolidation apparatus. This shows potential of waste copper slag to be successfully used as a replacement for sand in “sand piles” with the added advantage of improved performance.

It was puzzling to find that the stiffness of the sand-copper slag mix was insensitive to increases in the amount of waste copper slag beyond 10% by weight. However, the authors feel that shear strength and stiffness behavior holds the greatest potential for the use of waste copper slag hence, further testing is already underway.

7 ACKNOWLEDGEMENTS

Officials of Colombo Dockyard for providing waste copper slag for this research.

8 REFERENCES


Tools for Natural Hazard management in a Changing Climate

Outils de gestion de désastres naturels dans un climat changeant

Rogbeck Y., Lofroth H., Rydell B., Andersson-Skold Y.
Swedish Geotechnical Institute, Linkoping, Sweden

ABSTRACT: The paper will give an overview of some existing tools and models that can be used for risk analyses due to natural hazards (landslides, erosion and consequences of flooding) in a changing climate. Tools from several countries have been investigated by a literature survey and a questionnaire. A more comprehensive tool developed by Swedish Geotechnical Institute (SGI) will be presented more in detail. A compilation of tools has been carried out in the project “Baltic Climate”, funded by the EU Baltic Sea Region Programme 2007-2013 and its partners. The investigation shows that there is a general lack of tools for soil movements in the countries in the Baltic Sea Region and that most of the existing ones don’t take climate change into consideration. The paper will present a model that can be used separately or as a complement to more general tools for spatial planning. The tool for soil movements considers the consequences of flooding, landslides and erosion in a changing climate and it can be used on both regional and local levels. The tool is described as a general method with examples from municipal level. The SGI tool has been used in several practical cases both on a regional and local level.

RÉSUMÉ: Cet article donne un aperçu de certains des outils et des modèles existants qui peuvent être utilisés pour l’analyse de risques reliés aux désastres naturels (glissements de terrain, l’érosion et les conséquences des inondations) dans un climat changeant. Des outils provenant de plusieurs pays ont été étudiés par une étude bibliographique et par questionnaire. Un outil plus complet développé par l’Institut Suédois de Géotechnique (SGI) sera présenté plus en détail. Une compilation d’outils a été réalisée dans le projet “Climat Baltique”, financé par le programme de l’UE de la mer Baltique 2007-2013 et de ses partenaires. L’étude montre qu’il y a un manque général d’outils pour les mouvements du sol et que les outils existants ne prennent pas en compte les changements climatiques. Cet article présente un modèle qui peut être utilisé séparément ou comme un complément à des outils plus généraux de l’aménagement du territoire. L’outil des mouvements de sol considère les conséquences d’inondations, de glissements de terrain et de l’érosion dans un climat changeant et il peut être utilisé au niveau régional ou local. L’outil est décrit comme une méthode générale avec des exemples au niveau municipal. L’outil SGI a été utilisé dans plusieurs cas pratiques aussi bien au niveau régional que local.

KEYWORDS: tool, natural hazard, landslide, erosion, climate change

1 INTRODUCTION

In order to establish resilient communities, mitigate damages, adapt the built environment and establish a sustainable society, there is a need for a sound decision basis for buildings, infrastructure, industry and the environment. One cornerstone to reach a sustainable development is to take natural hazards into account both for the situation today and for the consequences of climate change. The predictions of global climate change include sea level rise, in many countries increased precipitation and runoff and more intense and damaging storms which will increase the threats of natural hazards.

2 NATURAL HAZARD MODELS IN CLIMATE CHANGE

In this paper tools are presented that can be used in a climate change adaptation process, with focus on natural hazards such as landslides erosion and flooding. There are also review of more general tools, e.g. “The Baltic Climate Toolkit” which can be used for planning on the regional, local and detailed level. The main purpose of the toolkit is to highlight the importance of climate change mitigation and adaptation aspects in spatial planning [1]. The “The Baltic Climate Toolkit” is developed within the project Baltic Climate (BC) [1]. Adaptation to the future climate conditions, including flooding etc. should be one of the starting points of the planning process proposed in the BC-toolkit.

The comprehensive decision process model described in this paper focuses on natural hazards such as erosion and landslides (soil movements). It constitutes a part of the Baltic Climate project and can be used individually or as complement to the general toolkit for aspects regarding soil movements. It can be used for adaptation aspects especially for spatial planning or in built-up areas to ensure a safe, healthy and sustainable society. The investigation was done by a questionnaire sent to the partners and associated organisations in BC complemented by a brief literature survey.

In addition to the presentation of the decision process model, a practical application of the model in a municipality is presented. The investigation constituted a part of the BC project. According to the results provided by the respondents several countries have started investigations to identify risks of natural hazards such as coastal erosion, landslides and flooding, but the investigations do not always incorporate the effects of climate change. Furthermore, the investigations are normally restricted to currently developed areas [1, 2]. The investigation in the Baltic Sea Region showed that there is a general lack of tools for soil movements and they don’t consider the impacts of climate change. However, in Sweden there is a model, developed by the SGI, which is presented in this paper.

The questionnaire survey of tools/models of soil movements was also expanded to outside the Baltic Sea Region. A questionnaire was completed by respondents in France, Hungary, Italy, Norway, Poland and Slovakia. In all responding countries models for soil movements are in use. The models
presented from Hungary, Italy, Poland and Slovakia do, however, not take climate change into consideration.

The literature survey revealed that a large range of conference papers can be of interest when working with soil movements for example [3] describes the EU project Response: “Applied earth science mapping for evaluation of climate change impacts on coastal hazards and risk across the EU”. The methodology employs commonly available digital data sets in GIS to assess regional-scale levels of coastal risk through production of series of maps. The outputs of the methodology comprise factual data maps and thematic maps and non-technical summary maps as planning guidance.

An on-going EU project is the KULTU-Risk project [4]. It will focus on water-related hazards. In particular, a variety of case studies characterised by diverse socio-economic contexts, different types of water-related hazards (floods, debris flows and landslides, storm surges) and space-time scales will be utilised [4].

In the UK there is a Climate Impact Programme (UKCIP) that contains a range of tools, methods and guidance which can be used for climate adaptation. The programme demonstrates how and where they fit into a risk-based planning process. There is also a National Appraisal of Assets and Risk from Flooding and Coastal Erosion, with adaption options on [5].

In France Baills et al. [6] have developed a method for integrating climate change scenarios into slope stability mapping. The climate factor treated as a variable in the stability calculation is the ground water level. Ground water levels are calculated from a conceptual hydrological model driven by rainfall data, and are described as filling ratio of the maximum ground water level [2].

3 THE SGI DECISION PROCESS MODEL FOR NATURAL HAZARDS

The SGI decision process model describes the potential risk related to a particular natural hazard, and makes it possible to establish a decision basis for spatial planning and climate adaption of built-up areas [7].

The model is partly based on the results of the Interreg Messina project [8] and the EU Life Environment Response project [9]. The model is based on identifying the prerequisites or probability for a natural hazard (P) combined with its associated consequences (C) which will determine the risk (R = PxC). The entire model can be used or only parts of it depending on the situation. The model aims to provide outcomes in the planning process that contributes to sustainable development including risk, environment, economy and social sustainability aspects as shown in Figure 1 [2].

At every stage in the decision process model (Figure 1), more detailed tools/models or suggestions exists that help to handle the questions that arise. For example under potential hazards the output can be a hazard map, and under the stage potential risk areas the output can be a risk map. Other relevant tools for identifying and assessing risk mitigation strategies can be databases or other information on previous experiences of strategies including pros and cons. It could also be a description on functionality and related costs for investment. In the long-term perspective, it could also be more holistic assessments such as life cycle and multi-criteria analyses. If there exists for example a mapping tool/model in another country it can be used instead of the one in this paper, and the other stages in the decision process model can be used together with that method.

For possible measures in spatial planning, or for adaptation of the built environment, socio-economic analyses and environmental assessments could be carried out. National and regional inventories of the natural hazards are necessary for spatial planning, to get an overview of risk areas or making priorities for preventive measures. At the local level the SGI tool can be used as a base for spatial planning, decision making of alternative measures in a municipality or at a specific location. The tool can also be used before investments are made in an area.

3.1 Mapping of potential hazards/Probability

The susceptibility as an indication of the probability of hazards such as erosion, landslides and flooding can be estimated. In Sweden, national overview investigations of landslides, erosion and flooding are carried out and described briefly below.

The Swedish landslide hazard mapping method for fine grained sediments (clay and silt), is used in a nation-wide programme for landslide risk reduction in built-up areas administered by the Swedish Civil Contingencies Agency (MSB). The mapping method is divided in several stages which get more detailed and need more information for each stage. Initially a pre-study is carried out, with the purpose to identify sub-areas considered to be mapped. Thereafter, the mapped areas are divided into areas with and without prerequisites for initial slope failure. The next stage is to identify areas with satisfactory stability based on overview assessment and areas that need more investigations. The results are presented in a susceptibility map with three different zones. Other information of interest for slope stability, such as calculated sections, scars of old landslides, erosion in progress and the presence of quick clay can be shown on the same map [2, 10].

There is also a Swedish landslide hazard mapping for till and coarse soils [2, 11, 12] administrated by MSB, divided in stages in the same way. The susceptibility for landslides and debris-flows in slopes is carried out based on a combination of overview stability calculations (safety factor) and other influencing factors. The susceptibility for debris flows in gullies is based on already occurred debris flows and by mapping and compiling factors that could contribute to triggering of a debris flow. For both cases there is in general a combination of six main factors: topography, hydrology, soil conditions, land use, earlier soil mass movements and existing preventive constructions. It is necessary to calculate the peak discharge,
determine the run-off conditions, the precipitation and the amount of available soil material. A classification is made and the results of the mapping and classification are reported on a map.

A mapping for coastal areas can also be performed. The hazards are identified by evaluation of the present state of the coast and a coastal geomorphologic model can be established. This includes the geomorphology, the topography and bathymetry as well as the driving forces such as water levels, waves, water currents and existing coastal protection. With climate change scenario for the chosen time period the probability for hazards such as erosion, landslides and flooding can be estimated. In Sweden, an overview mapping of the prerequisites of coastal erosion of the Swedish coasts, larger lakes and rivers has been carried out by SGI and maps will be found at [16]. A model for risk analysis has also been developed at SGI, based on the principle of carry out analysis step by step depending on the need for decision basis [14]. In many countries there are on-going works with inundation mapping due to EU directive. In many cases the mapping is done only for today’s climate, but it is important to complement it with climate change scenarios.

3.2 Consequences

Potential consequences of a natural hazard can be described on overview or detailed levels. Within a governmental investigation on slope stability in the Göta River, SGI has developed a detailed method to identify, map and when possible assess consequences of potential landslides throughout the studied area [15, 16].

The method comprises:
- identification of consequences
- inventory/mapping of objects that may be affected
- assessment of the vulnerability, ie the probability of a certain consequence in case of a landslide
- method for monetary assessment

Relevant factors to consider are e.g. population, property, contaminated land, transportation network, industry. Monetary valuation of the consequences and estimation of the vulnerability are performed. The work has been divided in societal consequence sectors: buildings; transport, exposure, vulnerability and life; environmentally hazardous activities and contaminated sites; water and sewage systems; nature; culture; energy and electric supply systems; trade and industry.

The consequence is set to be the product of the inventory of elements at risk, value per unit area, the vulnerability and the exposure. The result is presented in a 2D map with five consequence classes given in MSEK/ha.

3.3 Potential risk areas

The principle is to identify risk areas based on the probability of an event and the consequences of such an event. Depending on the need of information risk analysis can be carried out on overview or detailed levels. In the Göta River investigation five classes of probability and consequence, respectively, are combined in a risk matrix from which three classes of risk are identified (Figure 2);
- low risk level
- medium risk level (investigation required)
- high risk level (preventive measures are required).

The outcome of the risk analysis can be presented in maps covering the investigation area illustrating the extent of the three risk levels. The method has been used in practice in e.g. the Göta river valley [17].

Figure 2. Illustration of risk analysis, where the consequences and probabilities for landslide are grouped into 5 classes and combined by GIS techniques in a risk matrix with three risk classes: low risk level, medium risk level (investigation required), high risk level [17].

3.4 Strategies and alternative measures

At the local level both for spatial planning and the built environment, the need for mitigation and adaptive measures must be identified, and data for the design and construction of such measures must be clarified. Requirements for remedial works can also be predicted using field-monitoring data, which may change the risk management philosophy from a reactive to a more pro-active one.

Mitigation measures for landslides, erosion and flooding risks often require levees, coastal protection and/or other stabilising measures. Such measures require geotechnical information during several stages of the planning and building process. In spatial planning, all factors that may cause risk for health and safety must be identified so that buildings and infrastructure will be located outside present and future risk areas or measures taken to secure these risk areas.

3.5 Socio-economic analyses and environmental impacts

For possible measures in spatial planning or for adaptation of the built environment socio-economic analyses are carried out. When socio-economic analyses are made they have to be based on correct actual data and valid methods to predict future development for different alternatives/scenarios. This is the basis for establishing the risk level that needs to be related to the acceptable risk level, the need of, and which, countermeasures that can be used to alleviate the potential problems. Also the stakeholders must be identified and the activities that are affected by possible changes to the land or coastal area. Analysis can be done for example by a Cost-Benefit Analysis (CBA). The basic way of working with a CBA model is to start by estimating total damage and loss for the “Do Nothing”- alternative. This value is later used as the benefit (or avoided damage) for the investigated options of preventive actions. The next step is to estimate the schedule and cost of implementing the options. Finally, if there still is a risk of damages for the investigated options; the cost of this is also calculated. For a CBA the selection criterion is that if the ratio between benefits and costs is greater than 1 (benefits divided by costs >1) the option is worth doing. The option with the highest benefit cost ratio gives “best value for money” [18].

Most of measures to reduce risks for natural hazards have to be built in environmentally and naturally sensitive areas close to the sea or rivers, in some cases consisting of Natura 200 areas. For that reason, all measures have to be evaluated due to the environmental impacts. For the proposed strategies and alternative measurements environmental consequences have to be considered.

3.6 Basis for spatial planning and adaptation

For spatial planning, following the stages in the model, the decision makers will have a proper and transparent basis for discussion with different stakeholders and the final decision of
the best available way to establish sustainable land and coastal areas.

For the built environment, the decision makers will have a proper and transparent basis for the discussion with different stakeholders and the final decision of the best available way to adapt built environment on land and in coastal areas.

4 EXAMPLE ON THE LOCAL LEVEL

The model or parts of the model has been used in several climate and vulnerability analysis in Sweden, both on regional and local level.

The municipalities have to make comprehensive plans and detailed development plans, where risks for natural hazards must be investigated. In order to consider the consequences of climate change on the planned and existing built environment SGI and Swedish Metrological and Hydrological Institute have on behalf of Nynäshamn municipality carried out an Overview Climate- and Vulnerability Analysis as a basis for the Municipal Comprehensive Plan 2010.

When working with comprehensive plans detailed data normally is not available, so the evaluation was made of the interface between areas with risk for natural hazards and consequences for important society constructions. The aim of the investigation was to clarify the consequences due to increased rain fall and sea level rise for different scenarios. Areas with risks for flooding, landslides or erosion have been investigated and the risk areas are illustrated in maps, see Figure 3. The interface between these areas and important society constructions has also been shown. The constructions can be e.g. special buildings, roads, railroads, dams. Environmental aspects have to be considered for e.g. flooding or landslides in contaminated areas or areas with enterprises with potential hazardous activities or dangerous substances.

An example of such a map is shown in Figure 3. The flooding from sea level rise is shown in the figure for different scenarios and the levels are also shown in Table 1.

![Figure 3. Part of Overview Climate- and Vulnerability Analysis for the Municipality of Nynäshamn in Sweden][2]

Table 1. The sea level for determination of flood along the coast of Nynäshamn (given in meter in the Swedish height system RH00).

<table>
<thead>
<tr>
<th>Case</th>
<th>Level (RH00)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Flood, 100 years return period (today’s climate)</td>
<td>0.68</td>
</tr>
<tr>
<td>2. Flood calculated future (year 2071-2100) high level scenario according to IPCC</td>
<td>1.30</td>
</tr>
<tr>
<td>3. Flood, future (year 2071-2100) water level with 100 years return period, according to the Dutch Delta committee</td>
<td>1.71</td>
</tr>
</tbody>
</table>

As shown in the figure and table there can be rather different results depending on which scenario that are used. It is important to compare different scenarios and act for uncertainties. Recommendations have to be suggested for spatial planning. In built-up areas natural risks measures have to be taken to prevent damages on constructions. Strategies and measures are suggested, for example slope excavations, berms, levees, coastal protection or other stabilising measures.

REFERENCES

4. The KULTU-Risk project, www.kulturisk.eu
5. The UKCIP tool website, www.ukcip.org.uk/tools
13. The SGI website, www.swedgeo.se

3250
Experimental reinforced soil walls built with recycled construction and demolition waste (RCDW).

Murs expérimentaux de sol renforcé construits avec résidus de construction et démolition recyclés

Santos E.C.G.
Polytechnic School of the University of Pernambuco
Palmeira E.M.
University of Brasilia

ABSTRACT: In spite of its well known evolution, the Civil Engineering is yet pointed out as remarkable raw material consumer and one of the leading waste generators in modern society. Nowadays, construction and demolition waste became a complex problem to government authorities due to its economical and environmental impacts. Bearing in mind these aspects, the use of recycled construction and demolition waste (RCDW) in reinforced soil structures appears to be an interesting proposition. In order to investigate this proposal, two instrumented full-scale wrapped face geosynthetic reinforced walls were constructed using recycled construction and demolition waste as backfill material. The instrumentation plan consisted of more than 400 instruments and required the adoption of a careful installation process due to the presence of coarse particles of RCDW. The results have shown that RCDW has excellent mechanical properties - with low variation – which allow its use not just in the suggested proposal but in other geotechnical works. Additionally, based on lessons learned during the construction process, some recommendations are presented with the intention of promoting a better performance of reinforced walls built with this “novel construction material”.

RÉSUMÉ : Malgré sa nette évolution, le génie civil est toujours indiqué comme un grand consommateur de matière première et un des leaders de la génération de résidus dans la société moderne. Actuellement, le résidu de construction et démolition est devenu un problème complexe pour les autorités municipales en raison des impacts économiques et environnementaux. Compte tenu de ces aspects, l'utilisation des résidus de construction et démolition recyclés (RCD-R) dans les structures de sol renforcé, émerge comme une proposition intéressante. Pour investiguer cette proposition, deux murs renforcés avec géosynthétiques de face enveloppé ont été construits avec RCD-R comme matériaux de remplissage. Les murs ont été construits à l'échelle réel et instrumentés. L'instrumentation consistait a plus de 400 instruments et elle a demandé l'adoption d'un procès minutieux d'installation en raison de la présence des cailloux du RCD-R. Les résultats ont montré que le RCD-R possède d'excellentes propriétés mécaniques – avec faibles coefficients de variation – qui permettent leur utilisation non seulement dans la proposition suggérée, mais aussi sur d'autres ouvrages géotechniques. De plus, basée sur les leçons apprises au cours du processus de construction, certaines recommandations ont été déposées dans le but de promouvoir une meilleure performance des murs renforcés construits avec ce "nouveau matériaux".

KEYWORDS: reinforced soil wall, geosynthetics, recycled construction and demolition waste, instrumentation.

1 INTRODUCTION.

The Geotechnical Engineering has provided the development of innovative solutions to complex Civil Engineering problems. This proves its technical capacity to face new challenges. However, besides its well known evolution, the Civil Engineering is yet pointed out as remarkable raw material consumer and one of the leading waste generators in our modern society. Nowadays, construction and demolition waste (CDW) became a complex problem to government authorities due to its economical and environmental impacts.

In this scenario, some aspects related to growth of cities and to the need for adoption of sustainable development concepts may threaten the technical and economical advantages of reinforced soil structures: i) lack of good quality backfill material near to site construction and ii) compliance with environmental laws, which became more strict with respect to exploitation of new raw materials deposits. Bearing in mind these aspects, the use of recycled construction and demolition waste (RCDW) in reinforced soil structures appears to be an interesting proposition.

In order to investigate this proposal, two instrumented full-scale wrapped face geosynthetic reinforced walls were constructed using RCDW as backfill material.

1.1 RCDW geotechnical characterization for use in reinforced walls

The Brazilian Environmental National Council (CONAMA), in its Resolution 307/2002, states that wastes generated in “[...] site preparation and excavation [...]” are classified as construction and demolition waste (CDW). Due to this, huge amounts of soil stockpiles can be found in some Brazilian recycling plants. According to the Construction Waste Collecting Association (2011), in Brasilia (capital city of Brazil) approximately 70% of mass of municipal solid waste consist of CDW. According to Santos (2011), approximately 65% of mass of the recycled construction and demolition waste (RCDW) produced in Brasilia is composed of soil. This fact reveals an interesting perspective to the use of RCDW in geotechnical works.

Santos (2007), in order to evaluate the potential use of RCDW in geosynthetic reinforced walls, carried out a laboratory testing program focused on geotechnical characterization and pH tests. Furthermore, pullout tests with geogrids were performed using clayey sand [typical soil from the southeast part of Brazil] and sand obtained from a local supplier. Clayey sand was chosen in order to compare the behavior of RCDW to others materials. The sand material was compliant with FHWA recommendations for backfill materials.
The RCDW material revealed a low coefficient of variation with respect to geotechnical properties and low alkalinity applicable to be used with geogrid products. The mechanical properties were excellent for the proposed application. The results of pullout tests with RCDW showed that the recycled material yielded a better performance when compared with the standard sand.

Based on the facts listed above and results observed by Santos (2007) as well as on interesting perspective for the use of this waste in geotechnical structures, a research programme aimed at investigating the performance of reinforced soil structures using RCDW as backfill material started in 2009 at the University of Brasilia, Brazil.

2 EXPERIMENTAL REINFORCED RCDW WALLS.

2.1 Recycled Construction and Demolition Waste (RCDW)

The recycled construction and demolition waste (RCDW) used as backfill material consisted of the product of the crushing process of construction and demolition waste (CDW), which is composed mainly of mixed materials including soil, bricks, and small particles of concrete. The RCDW was sampled at the CDW Recycling Plant of Brasilia-DF, located at Jockey Club Landfill (Figure 2). Usually, this material is used by the local government as cover for unpaved roads.

A large-scale equipment was used for the determination of the RCDW shear strength parameters. Because of the presence of coarse grained particles (Figure 2), the dimensions of the shear box used were 800x800x450mm. Table 1 presents the main geotechnical parameters of the RCDW tested.

Table 1. Geotechnical properties of RCDW.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (g/cm³)</td>
<td>2.74</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>35</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>28</td>
</tr>
<tr>
<td>Maximum dry unit weight (kN/m³)</td>
<td>16.9</td>
</tr>
<tr>
<td>Optimum water content (%)</td>
<td>18</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>38</td>
</tr>
<tr>
<td>Cohesion (kN/m²)</td>
<td>14</td>
</tr>
</tbody>
</table>

2.2 Geosynthetics

The geosynthetics used as reinforcement for the walls in this investigation consisted of a polyester geogrid and a polypropylene nonwoven geotextile. Table 2 summarizes the main properties of the reinforcement.

Table 2. Geosynthetics properties.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Geogrid</th>
<th>Geotextile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polymer</td>
<td>PET</td>
<td>PP</td>
</tr>
<tr>
<td>Longitudinal tensile strength (kN/m)</td>
<td>20</td>
<td>19</td>
</tr>
<tr>
<td>Transverse tensile strength (kN/m)</td>
<td>9</td>
<td>21</td>
</tr>
<tr>
<td>Maximum tensile strain (%)</td>
<td>12</td>
<td>70</td>
</tr>
</tbody>
</table>

2.3 UnB Retaining Wall Test Facility

Experimental walls were constructed in the UnB Retaining Walls Test Facility located outdoor at the Foundation, Field Test and Geosynthetics Experimental Field area. The test facility was designed to allow two walls to be constructed up to 3.6 m high by 3.7m wide and extending up to 7.2m from the front edge of the facility edge. The facility can contain up to 214 m³ of backfill material for the construction of two walls simultaneously. Figure 3 shows an overview of the test facility.
3 CONSTRUCTION PROCEDURE AND INSTRUMENTATION.

The walls construction process was conducted using the moving formwork technique, which is a common method for the construction of wrapped-faced walls in the field. In order to reduce the side wall friction, the whole internal walls of the test facility were covered with three polypropylene sheets interspersed with lubrication (liquid silicone).

Three walls sections were named according to their cardinal orientation as West, Central and East. This configuration allows for the instrumented portion of the wall (Central section) to approach a plane-strain condition, free from side wall effects, as far as practical. This procedure has been adopted at Royal Military College of Canada (RMC) in a successful long-standing program on construction of full-scale reinforced walls (Santos et al. 2010).

The construction procedure consisted of placing the backfill material and compacting it in 200mm lifts. In order to provide a light compaction and a satisfactory surface leveling, a manual compaction roll was used. Near to the face, a hand tamping cylinder was used to minimize the effects of the compaction on the facing displacements. The total construction time was 29 days. Figure 4 and Table 3 present RCDW reinforced walls construction history and their main characteristics.

![Figure 4. RCDW reinforced walls construction history (Santos et al. 2010).](image1)

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wall #1</td>
</tr>
<tr>
<td>Geosynthetics</td>
<td>Geogrid</td>
</tr>
<tr>
<td>Height (m)</td>
<td>3.60</td>
</tr>
<tr>
<td>Facing batter (°)</td>
<td>13</td>
</tr>
<tr>
<td>Reinforcement spacing (m)</td>
<td>0.60</td>
</tr>
<tr>
<td>Reinforcement length (m)</td>
<td>2.52</td>
</tr>
</tbody>
</table>

Approximately 400 instruments were installed in the two walls in order to record the following:

- a. strain in reinforcement layers (strain gauges and wireline extensometers installed in wall #1 and wall #2, respectively);
- b. wall face displacements;
- c. vertical earth pressure at the base of the RCDW (earth pressure cells - EPC);
- d. horizontal earth pressure within the RCDW mass (EPC);
- e. settlements along the surface of the RCDW mass (superficial marks);
- f. horizontal displacement of the foundation soil (inclinometer).

Figure 5 shows the instrument distribution profile.

![Figure 5. Instruments distribution profile.](image2)

Additional procedures were necessary to protect the instruments devices against mechanical damages during the walls construction. Because of the presence of coarse grained particles (Figure 2), the installation of the instrumented geogrid layers and earth pressure cells (EPC) were carried out using fine grained particles around the instruments. PVC tubes were used to create a region with selected fine RCDW - smaller than 2mm around EPC and strain gauges. Figure 6 presents a scheme of the region around strain gauges after geogrid layer installation process.

![Figure 6. Region around strain gauges (wall #1).](image3)

The data recording after the walls construction revealed that just one strain gauge was mechanically damaged, which correspond to a survival level of 98% for installed strain gauges. This survival level kept stable until the end of research program even though the rainy seasons in Brasilia. It was observed that all EPC survived to installation process but not to the first rainy season. After 110 days, all EPC failed.

Outward displacements of the walls faces during construction were measured. It was observed for the wall #1...
(geogrid) a maximum end of construction face displacement measured with respect to formwork position of approximately 106 mm. Wall # 2 (geotextile) revealed a maximum end of construction face displacement - measured with respect to formwork position - approximately equal to 254 mm. Figure 7 shows wall #1 post-construction face profile.

5 ACKNOWLEDGEMENTS

Funding for the research programme described here was provided by CNPq, FAP-DF and CAPES. The authors would like to acknowledge the contribution of the Geotechnical Research Group at RMCC, FURNAS, EMBRE, REFORSOLO, ORIGINAL, HUESKER, ÖBER, LINEA G, TORC, LINEA JR, ASCOLE, COOPERCOLETA and CAENGE AMBIENTAL.

6 REFERENCES


It was noticed for both walls that the presence of coarse particles near to face was responsible for the uneven surface and the different magnitude of facing displacements among the walls sectors at the same layer (Figure 8). Although this fact did not affect the mechanical performance of the walls, it is strongly advised to use a selected RCDW near the face in order to provide a better aesthetic aspect.

4 CONCLUSIONS

The results obtained in the research programme have shown that the RCDW used has excellent mechanical properties - with low variation – which allow its use not just in the suggested proposal but in other geotechnical works. Additionally, the adoption of a careful installation process due to the presence of coarse particles of RCDW seemed to be successful once the strain gauges presented a high and stable survival level. Based on lessons learned during the construction process, some recommendations were presented with aiming at promoting a better performance of reinforced walls built with this “novel construction material”.

Figure 7. Post-construction face profile (Santos et al. 2010).

Figure 8. Uneven surface recorded at the wall #1 face.
Comparing the properties of EPS and glass foam mixed with cement and sand

Comparer les propriétés d’EPS et mousse de verre mélangé avec du ciment et du sable

Teymur B., Tuncel E.Y.
Istanbul Technical University
Ahmedov R.
BP, Baku

ABSTRACT: One of the formations in which the waste materials can be used as a component is controlled low strength material (CLSM). CLSM is a self-compacting cementious material that is generally used as back-fill as an alternative to compacted-fill. In this study, the availability of glass foam and Expanded polystyrene (EPS) geofoam as a component for CLSM production was investigated, some geotechnical properties including strength and unit weight characteristics of composite soils were explored using several laboratory tests. When the results are compared, EPS mixtures have lower unit weight and undrained shear strength values compared to glass foam mixtures. Therefore use of glass foam will have the advantage of higher strength compared to EPS mixtures and can be used as a subbase material. As a result, it was found that a mixture containing cement, polystyrene foam or glass foam and sand, can be successfully used in some applications such as improvement of slopes and reduction of embankment weight.

RÉSUMÉ : La formation dans laquelle les déchets peuvent être utilisés comme un composant est contrôlée faible résistance (CMFR). CMFR est un matériau auto compactant ciment est généralement utilisé comme embâllement comme alternative au remblai compacté. Dans cette étude, la disponibilité de verre de mousse et Expanded polystyrene (EPS) mousse géotechnique en tant que composant pour la production de CMFR a été étudiée. Certaines propriétés géotechniques, y compris la force et l'unité de poids caractéristiques des sols composées ont été explorées à l'aide de plusieurs tests de laboratoire. Lorsque les résultats sont comparés, mélanges de l'EPS ont poids unitaire inférieur et les valeurs de résistance au cisaillement non drainé par rapport aux mélanges de mousse de verre. Donc utilisation de mousse de verre auront l'avantage d'une résistance plus élevée par rapport à des mélanges d'EPS et peut être utilisée comme le matériau de remblai. En conséquence, il a été constaté qu'un mélange contenant du ciment, mousse de polystyrene ou mousse de verre et du sable, peut être utilisé avec succès dans certaines applications comme l'amélioration des pistes et réduction du poids de berge.

KEYWORDS: cement, EPS foam, glass foam, sand, soil improvement.

1 INTRODUCTION

With the rapid increase in the need for superstructure and the increase in demand for the multi-storey high-rise buildings, composite materials are used to improve weak soils. The use of environmental and industrial waste materials as raw material in composite soils helps protect environment with the recycling of these materials while providing more economic solutions. One of the formations in which the waste materials can be used as a component is controlled low strength material (CLSM). CLSM is a self-compacting cementious material that is generally used as back-fill as an alternative to compacted-fill. Use of the recyclable materials in civil engineering industry, especially in the geotechnical applications as raw materials, contributes to the economy and the environment.

In Turkey, a considerable sum of solid waste materials is made of glass (DPT, 2001). Glass foam is one of the waste glassware recycling products which are used in certain structural applications. With its porous structure and light weight, glass foam, generally used for thermal and acoustic isolation, is also a potential filling in geotechnical applications where lightweight is crucial. In Turkey, waste glass composes the significant part of solid wastes and one of the recycled glass product is glass foam. In this study, the availability of glass foam and expanded polystyrene (EPS) geofoam as a component for CLSM production was investigated, some geotechnical properties including strength and unit weight characteristics of composite soils were explored using several laboratory tests. A certain mixture design and some engineering properties of this lightweight composite fill were determined by unconfined compression and California Bearing Ratio (CBR) tests.

Lightweight fill materials can be used in geotechnical engineering for the consolidation and bearing capacity problems of very soft soils, for filling applications performed on potentially collapsible slopes generally. Expanded polystyrene (EPS) geofoam which is obtained from the oil is used in low strength and soft soil construction as a lightweight fill material in different places of the world today. Expanded polystyrene foam is supplied as raw materials in the form of small particles. EPS is widely used in various geotechnical applications such as embankments, retaining structures, slope stability, bridge piers and other applications (Aksoy 1998 and Aytekin 1997). EPS has advantages of low cost and durability properties for long years against other types of lightweight materials, as well. Due to these reasons, expanded polystyrene has been popular over time in civil engineering and widely used as light and compressible fill material in many geotechnical application areas.

Geotechnical properties of EPS-cement-sand mixture were determined by unconfined compression tests and CBR tests. Ratio in terms of weight for cement to material of mixture was measured as 12/1. As a result, it was found that a mixture containing cement, polystyrene foam and sand, can be successfully used in some applications such as improvement of slopes and reducing the weight of embankments.

2 EXPERIMENTAL RESULTS

Figure 1 shows the grain size distribution curve for the sand used in the experiments. Sand used is classified as poorly graded sand (SP). The sand has a specific gravity of 2.64,
maximum and minimum void ratio of 0.85 and 0.54 respectively. $D_{50}$ of the sand used is 0.35mm and the internal friction angle was found 41. Considering the weight proportions of cement and glass foam, mixtures with different weight ratios of cement, which is used as the binding material, and glass foam, which is regarded as the main component having the largest volume ratio in the mixture, were prepared. Cement and water were mixed first, sand was added next if denoted in the design, and after these components make up a rather homogenous mixture, glass foam is added to the mixture and stirred till homogeneity again.

When the 7-day experimental results of the mixtures that were produced using different sand ratios was examined, it has been observed that the optimum results were exhibited by the specimens which has equal sand and glass foam quantity and when cement over foamed sand mixture ratio is two. The water/cement ratio used is 0.45. Figure 2 shows the grain size distribution curve for the glass foam used in the experiments. The average value of the saturated unit weight of the glass foam mixture was found as 8.83 kN/m$^3$.

Figure 3 shows the cement, sand and glass foam mixture sample used in the unconfined compression test. The average value of the typical 7-day unconfined compression strength of the mixture was determined as 0.75 MPa while the average of the 28-day unconfined compressive strength was 0.91 MPa. Figure 4 shows the results of these tests, as can be seen from the figure with time the strength of the sample increases. The CBR
value of the 7-day mixtures was observed as 38.4 while this value was 78.9 for 28-day mixtures.

Figure 3. Cement, sand and glass foam mixture sample for the unconfined compression test (Tuncel, 2012).

Figure 4. 7 and 28 days unconfined compression test results for the samples with glass foam.

The aim of the laboratory study of lightweight fill that consist of EPS-cement-sand mixture is investigation for its usability in geotechnical applications successfully. So for the solution of weak soils with low durability that have slope stability problems, it is tried to create more durable and compressible lightweight soil than normal soils. The unit weight of the EPS mixture was 3.80 kN/m³. Figure 5 shows the unconfined compression test sample of EPS, sand and cement mixture. Weight content of materials in mixture was selected, so different proportions such as 100%, 75%, 50% and 25% of EPS content of mixtures were prepared. To determine the ratio of EPS in the cementitious material mixture, where sufficient shear strength is needed, unconfined compression tests were done. Prepared mixture samples were tested, then relevant percentage of expanded polystyrene content of material and cement/material ratio of mixture was determined. The relevant content of EPS in material is determined as 50% and ratio in terms of weight for cement/material of mixture was measured as 12/1. Unconfined compression value of 7 day sample is 0.22 MPa and of 28 day EPS mixed samples is 0.42MPa. According to the results, the mixture is defined as a low permeable lightweight fill and it also has CBR value that can be classified as medium. Figure 6 shows the results of these tests, as can be seen from the figure with time the strength of the sample increases.

Figure 5. Unconfined compression test sample of EPS, sand and cement mixture (Ahmedov, 2012).

Figure 6. 7 and 28 days unconfined compression test results for the samples with EPS foam.

CBR tests showed that the glass foam-sand-cement mixtures have enough bearing capacity to be used as a subbase material. CBR values for 28 days old EPS mixture is 7 making it weak to be used as a subbase material. As a result, by producing lightweight fills with CLSM mixture produced using glass foam-sand-cement can be a solution for consolidation and bearing capacity problems of very soft soils which continually consolidate, constitution of geotechnical fills on potentially sliding slopes and reducing the stress distribution on retaining structures.
3 CONCLUSIONS

When the test results of the glass foam-sand-cement mixture compared with the other lightweight fill materials it was seen that glass foam-sand-cement mixture has a higher unconfined compressive strength and CBR value. The CLSM mixture produced using glass foam-sand-cement can be used as lightweight fills on very soft and continually consolidating soils for solving the consolidation and bearing capacity problems. It can be convenient to use this mixture as fill for potentially sliding slopes and for reducing the lateral stresses that received by soil retaining structures.

When the results are compared, EPS mixtures have lower unit weight and undrained shear strength values compared to glass foam mixtures. Therefore use of glass foam will have the advantage of higher strength compared to EPS mixtures and can be used as a subbase material.

4 REFERENCES


Geotechnical engineering and protection of environment and sustainable development

Engineering géotechnique, protection de l’environnement et développement durable

Vaníček M., Jirásko D., Vaníček I.
Czech Technical University in Prague

ABSTRACT: The paper highlights the positive role of geotechnical engineering for protection of environment and sustainable development, first of all from the view of sustainable construction. The main attention is therefore focused on the problems which are sensitive to society in general as the construction on brownfields, utilization of waste and recycled materials for new construction and rehabilitation of territory affected by open pit mining process which is proposed as new development area. The first point is connected with the protection of greenfields and is defining basic phases of process of rehabilitation as the significance of first phase of geo-environmental investigation, remediation of contaminated subsoil and utilization of old foundation. The second point describes practical experiences with utilization of large volume waste as construction and demolition waste or ash in earth structures – not only from the view of mechanical behaviour but also from the view of potential impact on environment. The last point describes the utilization of the surface of the mining spoil heaps for new construction together with control of long term stability of slopes even for future expected ground water table and heavy rainfalls.

RÉSUMÉ : L’intervention souligne le rôle positif de l’engineering géotechnique dans la protection de l’environnement et le développement durable, et ceci essentiellement du point de vue de la construction durable. Elle se concentre surtout aux problèmes généralement perçus par la société comme sensibles : la construction sur les friches industrielles, l’utilisation de déchets et de matériaux recyclés dans la nouvelle construction, ainsi que le réaménagement du terriê de affecté par l’exploitation minière à ciel ouvert. Le premier point est lié à la protection des greenfields et définit les phases principales du processus du réaménagement, comme la première phase de la prospection géo-environnementale, l’assainissement du sous-sol contaminé et l’utilisation des bases anciennes. Le deuxième point décrit des expériences pratiques avec l’utilisation des déchets volumineux comme p.ex. des déchets de construction et de démolition ou bien des cendres dans les ouvrages en terre, non seulement du point de vue des caractéristiques mécaniques, mais aussi de celui de l’impact potentiel sur l’environnement. Le dernier point décrit l’utilisation de la surface du terril pour les nouvelles constructions, tout en assurant le contrôle de stabilité de long terme des pentes, même pour la nappe d’eau souterraine attendue et les précipitations fortes.

KEYWORDS: brownfield, remediation, contaminant, waste, spoil heap, soil improvement

1 INTRODUCTION

Geotechnical Engineering is falling under the limited group of professions, which to the high extent are able to react not only on classical construction problems but also to new society demands, namely with respect to:
- Protection against natural hazards – first of all against floods, landslides and earthquakes;
- Energy savings – especially with respect of Geothermal energy, as with high potential energy (from large depth) or with low potential energy in the forms of earth aerial heat exchanger, systems utilizing heat pumps or systems utilizing heat reversible pumps either for heating or for cooling with help of energy piles or diaphragm walls;
- Raw materials savings – with high potential for waste and recycled material utilization, especially for large volume waste as e.g. ash, slag, construction and demolition waste etc;
- Protection of greenfields – as GE is playing significant role in the field of “Construction on brownfields”;
- Environmental protection in general – e.g. from the view of safe deposition of waste (landfills, tailing dams, spoil heaps, underground repositories) or with respect to remediation of old ecological burdens – decontamination of subsoil.

However in a matter of fact all above mentioned problems can fall under the umbrella of Environmental geotechnics and are parts of the geotechnical engineering benefit to Sustainable Construction, (Vaníček I. 2012).

The branch of Environmental Geotechnics is now very well established, falling under the important part of Geotechnical Engineering which can be called Geotechnics, Geo-Technology and represents the third column by which Geotechnical Engineering is supported, (Vaníček, I. and Vaníček, M. 2008). Remaining three columns are Theoretical background, Geomechanics and Feeling for ground response, whereas the first column Theoretical background relies on the understanding of natural sciences such as geology, engineering geology and hydrogeology on the one hand, and on the understanding of mechanics, theory of elasticity on the other. The second column relies on the application of existing findings to the behaviour of soils and rocks under different stress - strain states – we are speaking about support from soil and rock mechanics and finally the fourth column relies on a certain feeling of geological environment which Terzaghi (1959) denotes as “capacity for judgment”, and he says that “this capacity can be gained only by years of contact with field conditions”, (see Figure 1), (Vaníček, I. 2010).

However with the respect of the limited range of the paper only three problems will be discussed further.
1 ROLE OF ENVIRONMENTAL GEOTECHNICS IN THE BROWNFIELDS REDEVELOPMENT

Very often the whole process of the brownfields redevelopment can be divided into the following individual steps, (e.g. Vaníček and Valenta 2009):

- site location identification,
- First phase of investigation,
- preliminary economic analysis,
- Second phase of investigation – detailed site analysis
- project of site development and methods of financing – feasibility study
- project and completion of site remediation
- project and completion of construction of new development (including foundation engineering, reuse of old foundations).

From these basic 7 steps, it is obvious that environmental geotechnics is strongly involved in the whole process. But typical for geotechnical engineers are four parts – 1st phase of investigation, 2nd phase of investigation – detailed site analysis, project and completion of site remediation and the problem of foundation engineering, respectively reuse of old foundations. These parts will be discussed further in more detail, (Vaníček 2010).

The first two steps are labelled as the first phase which can be also called the desk study, which is only supplemented by visual inspection. So this first phase mostly uses existing materials, where the study of archive materials and different maps composes the most important part of this phase.

The 2nd phase of the investigation encompasses site investigation, usually starting with borings, field tests, collection of samples and laboratory tests. Classical geotechnical data are useful from the foundation design perspective, geoenvironmental data from the view of site contamination.

The properties of the brownfields ground is usually affected by previous man made activity. These changes have character of physical, chemical or biological change. Owing to biological degradation some problems with gas (mostly with methane) are expected. However in most cases the subsoil remediation is connected with

- Physical improvement of the subsoil quality, with porosity decrease;
- Chemical improvement.

The main principle of physical improvement is to create top layer with much better quality than subsoil to be able to eliminate differential settlement of the subsoil and to guarantee the possibility to create good footing bottom for new foundations, see more in chap. 4.

As the depth of the affected subsoil is usually deeper than the depth for which classical compaction rollers can be used it is necessary to apply other methods. The dynamic consolidation method was for example used for the subsoil improvement of old toxic landfill in Neratovice, (see Figure 2), where on the compacted material a new landfill was constructed, (e.g. Vaníček et al 2003).

In the north part of Bohemia, where there are many inner spoil heaps composed of uncompacted clay clods, a new method called “clay piles” was successfully applied.

A pre-driven profile is backfilled by clay of similar properties as is the surrounding material and subsequently compacted there, (see Figure 3).

- The main aim of chemical improvement is to decrease the degree of subsoil chemical contamination on accepted level. There is a very wide range of different methods which are used for site remediation. It is not the intention of this lecture to present the overview of these methods, because they are covered elsewhere, (e.g. Suthersan 1997), are summarized by US EPA or are a part of activities of ICEG – International Congresses on Environmental Geotechnics. Most of the methods utilize some geotechnical approaches, as drilling, pumping, hydraulic fracturing, and monitoring.

Nevertheless there are 3 methods preferably utilizing classical geotechnical methods as:

- Encapsulation – with the help of the underground sealing wall (Different types of cut-off walls) and the horizontal sealing system (CCL – compacted clay liner, GCL – geosynthetic clay liner, GL – geomembrane liner or composite liner), (Vaníček et al 1997).
- Permeable reactive barrier, e.g. (Jirasko and Vaníček 2009), where the vertical sealing wall regulates the contaminant plume to the permeable window – where contaminated water is cleaned – with the help of sorption, precipitation or degradation, (see Figure 4).
- Stabilization, solidification, - these methods are based on the principle of mixing waste with a bonding agent to create a stiff matrix where the contaminant is bonded. As a bonding agent the different combinations of cement, ash, lime and slag are usually applied.

Question about utilization of old foundations is the last geotechnical problem connected with brownfields redevelopment. This problem is especially sensitive for large cities as the average design life of office buildings is about fifty years.
The degree of compaction can significantly determine the final result – what is for example very important from the environmental point of view, (e.g. Butcher, Powell and Skinner 2006). Nevertheless we can reuse also spread foundations, which were used for old dwellings, e.g. prefab panel buildings; for farm buildings as well as for old industrial structures. Although the price for removal is not as problematic there as for pile foundations, the version of reuse is very attractive. Here the bearing capacity for subsoil composed of clays increased with time as the result of consolidation. Also the foundation settlement induced by new loading can be rather low, as some additional structural strength had chance to develop there with time for particle arrangement given by stresses from the old foundations.

Direction of the new research activity is therefore connected with observation of changes with time not only in subsoil surrounding existing foundations but also at the contact with this foundation. For bearing capacity and for settlement stress and strain paths are more complicated. Schematic drawing what is going on for selected layer below spread foundation is shown in Figure 5 and new laboratory and filed investigation should to prove some expected assumptions.

Human activities produce a huge amount of different waste. Therefore the most important aim is to decrease the volume of such waste. Nevertheless for remaining waste the strategy should be defined and more efficient way is connected with reutilization of this waste. Civil engineering and first of all geotechnical engineering has a great chance to reuse large volume waste as:

- Construction – demolition waste – old bricks, concrete, ceramics, old asphalt pavement, gravel ballast.
- Industrial waste – ash, dross, slag;
- Mining waste – overlying soils, waste rock, quarry waste, residues after washing china clay…

During last period the orientation is also on other relatively large volume materials as tyres, glass, polystyrene… Only one example will be shown, which is combining the utilization of waste for the production of new construction material which can be used for better protection against floods. This new construction material is called brick – fibre – concrete which is composed from old bricks and concrete crushed particles together with classical additives for concrete – cement and water and with new additives – with synthetic fibres, (Vodička et al 2009).

After mixing together the final product looks like on the Figure 6a, where interconnection of individual components is visible. The degree of compaction can significantly determine the final result – what is for example very important from the view of permeability, as this property can be guaranteed in relatively wide range. The impact of the fibers can be seen from the Figure 6b, representing the result of bending test of prepared beam.

After heavy floods there is usually huge amount of the construction and demolition waste and the new product can be applied for the reinforcement of reconstructed part of dikes. Laboratory models up to the scale 1:1 (see Figure 7) proved extremely high resistivity against surface erosion and such reinforced dikes can be applied not only for reconstructed parts but also in selected sections of dikes, where the crest is little bit lower than other part and overflowing can start there as higher resistivity is guaranteed.
3 THE UTILIZATION OF THE SURFACE OF THE MINING SPOIL HEAPS FOR NEW CONSTRUCTION

Roughly 200 mil. m³ of clayey material which overlay brown coal seam are removed and backfilled during open pit mining activity in the Czech Republic each year.

As extremely large part of the country in this area is affected by mining activity the construction of new structures on the surfaces of these spoil heaps is nearly necessity. The first condition is connected with long term slope stability as material properties are changing with time as well ground water level, (Vaníček and Chamra 2008). Second condition is connected with settlement, first of all with differential settlement, as this uncompacted clay fill has sometimes extreme elevation – more than 100 m. Typical example is Ervenice corridor, (Dykast et al 2003), with cross section shown in Figure 8. Only for top layer about 5 m little bit better material was applied and partly compacted. Nevertheless this layer significantly eliminated differential settlement, so that motorway on the top can be used without special limitation (see Figure 9) even when the total settlement exceeded 2 meters.

Figure 8. Schematic cross-section of Ervenice corridor

Figure 9. Photo of the top of Ervenice corridor

For classical objects founded on the spoil heaps surface the following approaches are applied:

- postonning the new construction – however sometimes this condition is unacceptable;
- using some methods of deep foundations like piles – but solution can be limited by height of fill, by economical reasons and a negative skin friction should be taken into account;
- preconsolidation with additional load, which has to be removed after a certain time – very problematic as it is connected with huge volume of additional fill and with time needed for which this additional loading have to be applied.
- Among active measures the different approach is usually chosen for total and for differential settlements. Higher value of total settlement can be accepted if:
  - special technical solution is applied for engineering services as electricity, gas, sewage…
  - rectification can be applied e.g. for railway tracks, pipelines etc.

But most sensitive questions are connected with differential settlements with direct impact on damages to the structural elements and to the manner of the practical use of the structures. There we cannot so easily accept little bit higher values as for total settlement. Therefore if the probability that the expected different settlements will be higher than the accepted ones this situation has to be solved with the help of the following steps:
- to select such construction system which is not so sensitive to the differential settlements; or
- to improve the subsoil beneath foundations as was mentioned at the beginning of this chapter.

4 CONCLUSION

Short overview is stressing a significant role of the geotechnical engineering for environment protection, especially from the view of Sustainable construction, from the view which is very sensitive for all society. Three practical examples supported this general aspect from which new problems which our profession has to deal with are clearly visible.

5 REFERENCES


Applicability of Municipal Solid Waste (MSW) Incineration Ash in Road Pavements Base

Utilisation de cendres d’incinération de déchets solides municipaux (MSW) dans la couche de base de chaussée

Vizcarra G., Szeliga L., Casagrande M.
Pontifical Catholic University of Rio de Janeiro, Rio de Janeiro, Brazil
Motta L.
Federal University of Rio de Janeiro, Rio de Janeiro, Brazil

ABSTRACT: This study presents the characteristics of Municipal Solid Waste (MSW) incineration ash, by-product obtained from electric energy generation power plant in Rio de Janeiro - Brazil, to evaluate its applicability in base road pavements layers through the ash mixture with a non-lateritic regional clay soil with very poor mechanical behavior. Chemical, physical, mechanical tests and the mechanistic-empirical design for a typical pavement structure were carried out on the pure soil and also in the soil mixtures with the addition of different ash content (20% and 40%). MSW fly ash reduced expansion of the material, showing increase in the resilient modulus value with time of cure, load cycle number and reduction of mixture water content. Permanent deformation tests showed mixture soil-MSW fly ash reached a state of plastic accommodation. A typical pavement design was carried out by comparing between pure soil and mixture soil-MSW fly ash; the results showed that it is feasible utilize it in low traffic road pavements, highlighting the positive work of MSW fly ash and its environmental advantages.

RÉSUMÉ: Cette étude présente les caractéristiques des cendres d’incinération des déchets solides municipaux (MSW), un sous-produit obtenu à partir d’une centrale de production d’électricité à Rio de Janeiro - Brésil, afin d’évaluer son applicabilité en tant que couche de base des chaussées en mélangeant la cendre avec un sol argileux non-latéritique régional de comportement mécanique inacceptable à cet effet. Des essais chimiques, physiques, mécaniques, ainsi que la conception mécaniste-empirique d’une structure de chaussée typique ont été effectués sur le sol pur et aussi sur le mélange de sol avec addition de différentes concentrations de cendres volantes de MSW (20% et 40%). L’inclusion des cendres a réduit l’expansion du sol, indiquant une augmentation de la valeur du module résilient avec le temps de durcissement, le nombre de cycles de charge et la réduction de l’humidité du mélange. Des essais de déformation permanente ont montré que le mélange de sol-cendres volantes des déchets solides municipaux a atteint un stade de déformation plastique. La conception d’une chaussée typique a été réalisée pour comparer le sol pur et le mélange de sol-cendres volantes des déchets solides municipaux, les résultats ont montré que son utilisation est possible dans des chaussées de faible volume de trafic, mettant en évidence l’utilisation positive de ces cendres et ses avantages socio-économiques et environnementaux.

KEYWORDS: MSW fly ash, pavement, deformability properties, permanent deformation, resilient modulus.

1 INTRODUCTION.

This study evaluates the application of fly ash obtained from incineration of Municipal Solid Waste (MSW) use in base layers of pavements, by mixing the ashes with a non-lateritic regional clay soil. The Usina Verde is a privately held company located in the Federal University of Rio de Janeiro, and aims to provide environmental solutions for the disposal of municipal solid waste, through incineration with energy co-generation. The Usina Verde receives, daily, 30 tons of MSW Company's Waste Disposal of Rio de Janeiro. In sorting, recyclable materials are segregated manually along with the use of metal detectors; after this process, the composition of MSW is principally organic matter (88%), plastic (10%) and rubber (2%). The MSW is then crushed and separated as fine material and sent to drying. These wastes are sent to the incinerator, which operates at a temperature of 950ºC.

At the end of the incineration process are obtained fly ash and bottom ash, being from 8 to 10% by volume of the two ashes, which represent about 80% of bottom ash and 20% of fly ash (Fontes 2008).

2 OBJECTIVES

The objective of the investigations is to study the effect of MSW fly ash addition on the soil, evaluating deformability and expansibility properties, also thickness layer pavements base of soil with and without MSW fly ash

3 EXPERIMENTAL INVESTIGATION

3.1 Materials and properties

The non-lateritic clay soil in study came from a deposit located in the city of Campo Grande, Rio de Janeiro state. Fly ash comes from the burning of municipal solid waste (MSW) at Usina Verde, which is located on Rio de Janeiro / RJ. The tests performed at Pontifical Catholic University of Rio de Janeiro and Federal University of Rio de Janeiro, aiming to characterize and evaluate the soil and soil-MSW fly ash mixtures. Since there was no research evidence previous to this topic, 20% and 40% as percentages of fly ash were utilized to add to the soil. The symbols used in this study, which describe the materials and mixtures with percent in weight, are presented in Table 1.
MPa and deviator stress ranging from 0.021 to 0.412 MPa.

applied, with principal minor stress ranging from 0.021 to 0.137 constant.

a load and unload, whereas the minor principal stress remains the sample top, always in the compression direction, furthering moisture obtained in the compaction test. 

Janeiro, into molds of 10 x 20 cm compacted at optimum Geotechnical Laboratory of Federal University of Rio de

Tests were performed according to standardized test in the

3.2.2 Resilient modulus test

The tests were performed according to standardized test in the Geotechnical Laboratory of Federal University of Rio de Janeiro, into molds of 10 x 20 cm compacted at optimum moisture obtained in the compaction test. 

In the cyclic load triaxial test, deviator stresses are applied in the sample top, always in the compression direction, furthering a load and unload, whereas the minor principal stress remains constant. 

Each sample was subjected to eighteen stresses states were applied, with principal minor stress ranging from 0.021 to 0.137 MPa and deviator stress ranging from 0.021 to 0.412 MPa. 

The Resilient Modulus (MR) of soil is the relationship between the deviator stress (σd) applied repeatedly in a sample of soil in triaxial test and the corresponding specific recoverable or resilient strain (εr). As shown in Equation 1 (AASHTO TP46-94 1996).

\[ M_R = \frac{\sigma_d}{\varepsilon_r} \]  

Where:

- MR: resilient modulus;
- σd: cyclic deviator stress (σ1 - σ3);
- εr: resilient strain (vertical).

The composite model used in this study relates the resilient modulus of minor principal stress and deviator stress, as shown in Equation 2.

\[ M_R = k_1 \sigma_3 + k_2 \sigma_d + k_3 \]  

Where:

- σ3: minor principal stress;
- σd: cyclic deviator stress (σ1 - σ3);
- k1, k2 and k3: correlation coefficients, derived from results of laboratory tests.

This model was chosen because it presents bigger correlation coefficients to the incorporating the minor principal stress and the deviator stress influence. The nonlinear least squares model estimation was utilized to obtain the correlation coefficients.

In order to evaluate the influence of cure time, optimal water content samples were prepared and next rolled into hermetically closed plastic bags for 7 and 21 days. Soon afterwards, these were proceeded to the resilient modulus tests.

3.2.3 Permanent deformation test

The tests were performed according to Guimarães (2009), using the same molds used in the Resilient Modulus Test. A total of 500,000 load cycles were applied for each specimen.

Three tests were conducted in the Mixture S60/CV40, in the condition of maximum dry density, at stress levels shown in Table 2.

### Table 1. Material’s symbols

<table>
<thead>
<tr>
<th>Material</th>
<th>% Soil</th>
<th>% MSW fly ash</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>100</td>
<td>0</td>
<td>S</td>
</tr>
<tr>
<td>MSW fly ash</td>
<td>0</td>
<td>100</td>
<td>CV</td>
</tr>
<tr>
<td>Mixture 1</td>
<td>60</td>
<td>40</td>
<td>S60/CV40</td>
</tr>
<tr>
<td>Mixture 2</td>
<td>80</td>
<td>20</td>
<td>S80/CV20</td>
</tr>
</tbody>
</table>

### Table 2. Permanent deformation tests

<table>
<thead>
<tr>
<th>Test Number</th>
<th>σ1 (MPa)</th>
<th>σd (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.098</td>
<td>0.294</td>
</tr>
<tr>
<td>2</td>
<td>0.118</td>
<td>0.353</td>
</tr>
<tr>
<td>3</td>
<td>0.098</td>
<td>0.392</td>
</tr>
</tbody>
</table>

### 3.2.4 Pavement design

A pavement structure was assumed (Figure 1) considering Rio de Janeiro’s weather, with the purpose of exploring the effects of adding MSW fly ash in soil on pavement project one. The thickness and mechanical properties of the coated asphalt and subgrade remain constant, so that only the thickness of the base may be modified, according to the parameters of resilience for each material. As for the mechanistic-empirical analysis, the computer program SisPav (Franco, 2007) was used. Bernucci (1995) indicates for Brazilian low traffic roads an N value of 10^6 should be used. Thus, in this study, N value of 10^5 was assumed.

![Figure 1. Pavement structure adopted.](image)

### 4 RESULTS AND DISCUSSIONS

From the test conducted, the characteristics and effects of the addition of MSW Fly Ash into soil were studied.

#### 4.1 Chemical characterization

The main chemical components of soil, which are normally found in residual soils, are SiO2, Al2O3 and Fe2O3, such as showed in the Table 3. Lixiviation and Solubility tests performed according to Brazilian standards NBR 10005 and NBR 10006 for MSW fly ash and soil stabilized with 40% fly ash content. The mixture is classified non - dangerous and non-inert (Vizcarra 2010).

#### 4.2 Physical characterization

MSW fly ash and mixtures can be noted as follows: first, the Atterberg Limits for pure MSW fly ash could not be performed due to the behavior of granular material, which during the test did not show plastic characteristics to their achievement. Second, the inclusion of MSW fly ash decreases the liquid limit and plasticity index, and increases the plastic limit of soil.

According the classification MCT (Nogami & Villibor 1995), the soil is classified as NG' behavior "non-lateritic-clay.” When compacted under the conditions of optimum moisture content and maximum dry unit weight for normal energy compaction, these soils present characteristics of traditional highly plastic and expansive clays.

The use of these soils is related to restrictions resulting from its high expansibility, plasticity, compressibility and contraction
when subjected to drying, its use is not recommended for base pavements, and some of the worst soil for the purpose of paving, from the tropical soils (Nogami & Villibor 1995).

From the curves of soil compaction and mixtures with fly ash obtained from the Modified Proctor tests, it can be stated that by increasing the level of ash in the mixture, the maximum dry density tends to decrease (Figure 2).

Table 3. Soil, MSW fly ash chemical composition

<table>
<thead>
<tr>
<th>Concentration (%)</th>
<th>Compost</th>
<th>Soil</th>
<th>MSW Fly Ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>36 - 43</td>
<td>13 - 21</td>
<td></td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>35 - 38</td>
<td>12 - 15</td>
<td></td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>13 - 21</td>
<td>5 - 7</td>
<td></td>
</tr>
<tr>
<td>SO₃</td>
<td>0 - 1</td>
<td>5 - 10</td>
<td></td>
</tr>
<tr>
<td>CaO</td>
<td>-</td>
<td>32 - 45</td>
<td></td>
</tr>
<tr>
<td>TiO₂</td>
<td>0,9 - 1,7</td>
<td>3 - 4</td>
<td></td>
</tr>
<tr>
<td>K₂O</td>
<td>2 - 4</td>
<td>2 - 4</td>
<td></td>
</tr>
<tr>
<td>Cl</td>
<td>-</td>
<td>4 - 6</td>
<td></td>
</tr>
<tr>
<td>Organic Matter</td>
<td>0,1</td>
<td>0,7</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2. Compaction Curves of Soil and 20% - 40% Soil – Fly Ash Mixtures

4.3 Effect of MSW fly ash addition on resilient modulus

The results of Resilient Modulus tests (Figure 3) show that the Resilient Modulus of soil in study is dependent on the deviator stress and if the MSW fly ash is added, this behavior does not change. It is appreciated that the higher the deviator stress, the lower the value of resilient modulus.

The mixture with 20% MSW fly ash improved the mechanical behavior of pure soil, the mixture with 40% MSW fly ash downgraded the mechanical behavior, but it improved with cure time (Figure 4).

The mixture with 20% MSW fly ash was assessed with several different water contents. The results indicated the resilient modulus increased as the water content decreased.

4.4 Effect of MSW fly ash addition on permanent deformation

As shown in the Figure 5, the permanent deformation tends to stabilize reaching a plateau, it’s observed that Test 3 has a higher permanent deformation, this is due to increased tensions applied to the test.

The resilient modulus is increased with the number of load cycles (Figure 6), this can be explained by the diminution of elastic strain. The occurrence of the plastic accommodation (i.e. Shakedown) was investigated by using the behavior model developed by Dawson and Wellner, cited by Werkmeister (2003). The test results of permanent deformation test for the MSW fly ash – soil mixture were obtained and are displayed by the graph model of Dawson and Wellner cited by Werkmeister (2003) in Figure 7.

By analysis of this Figure it appears that all tests conducted with the MSW fly ash – soil mixture show a typical behavior for level A, i.e., demonstrated plastic accommodation, depending on the model proposed by Werkmeister (2003). The characterization of the level A behavior of both the shape of the curve, roughly parallel to the vertical axis, because when the rate of permanent deformation increase and have reached a magnitude of $10^{-7}$ (x $10^{-3}$ m/load cycle). I.e. at the final load cycles, the specimen’s permanent deformation increased by only 10 mm at each new cycle.

Figure 3. Soil with 40% MSW Fly Ash Resilient Modulus vs. Stresses (21 days of cure)

Figure 4. Resilient Modulus vs. Stress of Soil with 40% MSW Fly Ash – Cure Time Variation.

Figure 5. Accumulated Permanent Deformation Variation.
4.5 Effect of MSW fly ash addition on expansibility

The MSW fly ash decreases the expansion of the soil in study, which had an expansion of 4%, but with the addition of fly ash reduced it to 3.6% for 20% fly ash content and fell to 0.4% to a level of 40% fly ash. However, high content of fly ash when can deteriorate the mechanical behavior, resulting in a thicker layer.

4.6 Effect of MSW fly ash addition in pavement base

The mixture with 20% fly ash improved the mechanical behavior of pure soil, which is revealed by the decrease in thickness of the base compared to pure soil, for the same loading level and same parameters (criteria) for sizing. It is shown in Figure 8 the thickness of layers depending on the project period for each type of mixture, which was obtained by the computer program SisPav (Franco, 2007).

5 CONCLUSIONS

Mixtures with the inclusion of MSW fly ash had a mechanical behavior compatible with the requirements for a low traffic volume. The addition of 20% fly ash to the non-lateritic clay soil improved the mechanical behavior and reduced the expansion of the soil. The soil mixed with a content of 40% of fly ash decrease the mechanical behavior compared to pure soil, with the consequent increase in thickness; however, it improved with cure time and cycle loading number, decreasing significantly the expansion of the soil.

The results were satisfactory, being dependent on the ash content added, cure time and cycle loading number, highlighting the positive work of MSW fly ash for use in base layers of road pavements, eliminating the current problems of waste disposal in dumps and landfills.

6 ACKNOWLEDGEMENTS

The authors thank CNPq (MCT/CNPq 14/2009 # 480748/2009-8 project) for the financial support, as well as Usina Verde S.A. for the Municipal Solid Waste ash supply.

7 REFERENCES


Figure 6. Resilient Modulus Variation.

Figure 7. Shakedown’s occurrence search.

Brazilian Technical Standards Association ABNT. NBR 10005/04: Procedure for obtaining leaching extract of solid wastes.

Brazilian Technical Standards Association ABNT. NBR 10006/04: Procedure for obtaining solubilized extraction of solid wastes.


Research Results of Fine-Grained Soil Stabilization Using Fly Ash from Serbian Electric Power Plants

Les résultats de recherche de la stabilisation des sols de grains fins en utilisant les cendres volantes des centrales électriques serbes

Vukčević M., Maraš-Dragojević S., Jocković S., Marjanović M., Pujević V.
Faculty of Civil Engineering, University of Belgrade

ABSTRACT: This paper presents the results of laboratory research of fly-ash soil stabilization. Tests were conducted on mixtures with two types of fine-grained soils and fly ash sampled in Serbian electric power plant Kolubara. Used types of soils are low plasticity silty clay and very expansive, medium to high plasticity clay. Effects of fly ash on physical and mechanical properties of soil (grain size distribution, Atterberg limits, unconfined compression strength, moisture-density relationship, swell potential, CBR) were evaluated. Test mixtures were prepared at optimum water content from standard Proctor compaction test. Results of the research indicate that fly ash can effectively improve some engineering properties of soil.

RÉSUMÉ : Ce document présente les résultats de recherche en laboratoire de la stabilisation des cendres volantes. Les analyses effectuées concernent les mélanges avec deux types de sols de grains fins et de la cendre volante récupérée de la centrale électrique serbe « Kolubara ». Les types de sols utilisés sont de l’argile sableuse d’une plasticité faible et de l’argile très gonflante d’une plasticité moyenne à forte. Les effets des cendres volantes sur les propriétés physiques et mécaniques du sol (la distribution de la grosseur des grains, les limites d’Atterberg, la résistance à la compression uniaxiale, la relation entre la densité et l’humidité, les possibilités de gonflement, l’indice portant californien – CBR) ont été évalués. Les mélanges d’essai ont été préparés à teneur en eau optimale selon l’essai Proctor normal. Les résultats de la recherche signalent que les cendres volantes peuvent améliorer de manière efficace certaines propriétés techniques du sol.

KEYWORDS: soil stabilization, fly ash, fine-grained soil

1 INTRODUCTION

Fly ash makes the most of the combustion-by-products during the production of electricity in thermal power plants. A very small amount can be recycled, while significant amounts are disposed in landfills. The use of fly ash for soil stabilization can bring multiple benefits – protection of the environment, financial savings and it can also make the poorly-graded types of soils usable.

In Serbia, approximately 7 million tons of fly ash and slag are produced every year, of which only 3% is used in cement industry. The remaining products (about 300 million tons so far), are disposed on landfills, taking up the area of approximately 1600 hectares (Cmiljanic 2008, Cmiljanic 2010).

In Serbia, fly ash soil stabilization research was conducted for the first time during the preliminary design of the Serbian regional waste management center “Kalenic” (Report FCE Belgrade 2011). This research was performed by the authors during 2011. Waste management center “Kalenic” is located at open pit near thermal power plant “Kolubara”. Disposal area of the “Kalenic” landfill is being formed by construction of the outer embankment, instead of soil excavation, which is the usual way. The construction of the embankment needs more than 1.5 million m³ of material and costs are about 33% of total investment. Therefore, the possibility of using existing material from the site was analyzed in Laboratory for Soil Mechanics at Faculty for Physical Chemistry, Belgrade. According to USCS, this soil, known as alevrite, is medium to high plasticity clay (CU/CH), with swell potential.

This paper presents the results of fly ash soil stabilization laboratory research performed during 2011-2012, as the part of the research project funded by Electric Power Industry of Serbia.

2 MATERIALS

Materials used for the experimental research program include: fly ash from thermal power plant “Kolubara” (KFA) and silty clays from project “Kalenic” (Soil A) and from wind park project “Kosava” (Soil B).

2.1 Fly ash

Chemical composition of KFA was determined at the Faculty for Physical Chemistry in Belgrade and the results are shown in Table 1.

<table>
<thead>
<tr>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
</tr>
</thead>
<tbody>
<tr>
<td>50.21</td>
<td>23.83</td>
<td>9.89</td>
<td>4.79</td>
<td>3.12</td>
</tr>
</tbody>
</table>

Because of the high percentage of SiO₂ and Al₂O₃, according to ASTM C 618, KFA belongs to Class F silica mineral ashes, with pozzolanic properties. Class C fly ash is not available in Serbia (Cmiljanic et al. 2010).

2.2 Soils

The soils used for this study are predominantly silty clays.

Soil A: Mineral composition consists of quartz, muscovite and soft minerals of montmorillonite (testing performed at Faculty for Physical Chemistry, Belgrade). According to USCS, this soil, known as aleurite, is medium to high plasticity clay (CU/CH), with swell potential.

Soil B: Material was collected at the site of wind park near Vrsac, Vojvodina. Terrain at the site consists of Quaternary loess sediments. According to USCS, this soil is low plasticity.
clay (CL). Grain size distribution curves for used materials are given in Figure 1.

![Grain size distribution curves](image1)

Figure 1. Grain size distribution curves

3 LABORATORY TESTING

Laboratory testing was conducted in the Laboratory for soil mechanics at Faculty of Civil Engineering in Belgrade. Testing samples were prepared by compaction, with moisture content equal to optimum moisture content from standard Proctor compaction test.

Fly ash soil mixtures were prepared at three fly ash-soil ratios (10, 15, 20% fly ash content by dry weight). After addition of water, mixtures were compacted without delay. According to (Terrel et al. 1979, Ferguson and Levenson 1999) compaction should start immediately after the mixing process and finish within a maximum of 2 hours. Samples were tested immediately after compaction (t=0), as well as after 7, 14, 28 and 60 days. Following engineering properties were determined: unconfined compression strength (UCS), California bearing ratio (CBR), effective shear strength parameters (c’, $\phi’$) and compressibility modulus ($M_v$). All tests were performed according to SRPS Standards.

4 RESULTS AND DISCUSSION

4.1 Soil plasticity

In case of medium to high plasticity soil (soil A), it is observed that increasing of KFA percentage results in decreases in the liquid limit and plasticity index, which is not the case for low plasticity soil (soil B), as shown in Fig. 2.

![Variation in Atterberg limits for mixtures at t=0](image2)

Figure 2. Variation in Atterberg limits for mixtures at t=0

4.2 Compaction

The results (Fig. 3) indicate that maximum dry density decreases and optimum moisture content increases as the fly ash content increases (for both soil types). The decrease in maximum dry density is associated with the fact that used fly ash has much lower weight than soil. Results are in line with Santos et al. 2011 and Sharma 2012, while opposite trend can be found for Class C fly ash stabilization (White et al. 2005 and Ramadas and Kumar 2012).

![Moisture-density relationship of fly ash-soil mixtures](image3)

Figure 3. Moisture-density relationship of fly ash-soil mixtures

4.3 Unconfined Compressive Strength (UCS)

Increased soil strength is the main indicator of the successful soil stabilization. In previous studies (Ferguson and Levenson 1999, Ferguson 1993, Parsons 2002, Edil et al. 2006) strength of soil is usually determined by uniaxial compression test or bearing ratio test. The results of UCS tests shown in Fig. 4 indicate that maximum strength gain for soil A is obtained for mixture with 15% KFA. Soil UCS is increased by 15-25%, dependent of elapsed time.

![Strength gain of soil A for different percentages of KFA](image4)

Figure 4. Strength gain of soil A for different percentages of KFA
UCS for soil B without stabilizer was around 400 kPa and addition of fly ash didn’t result in strength gain.

4.4 Effective shear strength parameters

Effective shear strength parameters have been determined by using direct shear test. Obtained results (Fig. 5) show that long term friction angle doesn’t substantially change with addition of fly ash, for both soil types. On the other side, cohesion significantly increases with time for all tested mixtures. This is associated with pozzolanic properties of used fly ash.

4.5 California Bearing Ratio (CBR)

For both soil types, California bearing ratio tests have been done on mixtures with 15% KFA, which is adopted as optimum. Compared with CBR values for base soils A and B, obtained results showed significant gain. In the case of soil A, CBR value increased nearly 300%, and 260-380% in the case of soil B (dependent on elapsed time). This is especially important for soil A, because it makes it usable for road construction (CBR value was increased from 2.1 to 5.8). CBR values are shown in Fig. 6, and are in line with Mackiewicz and Ferguson 2005, White et al. 2005 and Edil et al. 2006.

4.6 Deformation parameters

Compressibility modulus (Fig. 7) for both soil types increase with addition of fly ash. Influence of time in this case was not significant. Overall modulus increase is around 15-35% for soil A and around 15% for soil B.

4.7 Swell potential

Although strength and deformation parameters of soil A can be considered acceptable, this soil showed significant swell potential, which makes it unusable for most engineering purposes. This property is associated with the presence of expansive mineral montmorionite. Addition of 15% of fly ash, which is determined as optimum, resulted in significant decrease of swell deformation, from $\varepsilon=8.6\%$ to $\varepsilon=1.8-3.1\%$. This results are in accordance with results of other authors.
5 CONCLUSIONS

Although many scientific results show that Class F fly ash cannot be used for soil stabilization without addition of cement or lime, laboratory tests performed in this research have shown that fly ash from thermal power plant ”Kolubara” is effective material for soil stabilization. Main conclusions of this research are as follows:

Addition of KFA decreases the plasticity index of medium and high plasticity soils (type A).

KFA impacts moisture-density relation of tested soils – optimum moisture content increases and maximum dry density decreases.

For soil A, based on UCS gain, amount of 15% KFA is identified as optimum. Strength gain was approximately 20%. There wasn’t UCS gain for low plasticity soil B.

For both soil types, long term friction angle almost doesn’t change with addition of KFA, while effective cohesion significantly increases with time for all tested mixtures.

CBR values increased around 260-380% for mixtures with 15% of KFA, which is adopted as optimum. This is main stabilization effect for soil B and very important effect for soil A.

Compressibility modulus for both soil types increase with addition of fly ash, without influence of time. Overall increase is around 15-35%.

Swell potential of very expansive soil A reduced with addition of 15% KFA. Swell deformation decreased from ε=8.6% to ε=1.8-3.1%.

Despite shown positive effects, the universal principle of soil stabilization using fly ash cannot be easily defined. It is necessary to perform detailed laboratory investigations, with certain types of ash and soil. It is the only possible way to precisely determine the optimal percentage of ash to be added, to determine strength gain and define the technology operations. The presented results of laboratory tests have confirmed the need to develop a research program in this field for Serbia, bearing in mind that the average annual production of fly ash that will be disposed on landfills is around 7 million tons.

6 REFERENCES


Elaborat o geotehničkim istraživanjima terena za potrebe izgradnje regionalnog centra za upravljanje otpadom "Kalenić" u selu Kalenić (Report - Geotechnical investigation of the ground conditions for the construction of regional waste management center “Kalenić” in village Kalenic), 2011. Građevinski fakultet Univerziteta u Beogradu (Faculty of Civil Engineering University in Belgrade)

Ferguson G. 1993. Use of Self-Cementing fly ashes as a soil stabilization agent. ASCE Geotechnical Special Publication No. 36, ASCE, USA

Ferguson G. and Leverson S.M. 1999. Soil and Pavement Base Stabilization with Self-Cementing Coal Fly Ash. American Coal Ash Association, USA


Paterson R. L. 2002. Subgrade improvement through fly ash stabilization. Miscellaneous Report, Kansas University Transportation Center, University of Kansas, USA


Simplified Prediction of Changes in Shear Strength in Geotechnical Use of Drinking Water Sludge

Prédiction simplifiée de changements dans la force du ciseau dans usage Geotechnical de boue de l'eau potable

Watanabe Y.
Central Research Institute of Electric Power Industry

Komine H.
Ibaraki University

ABSTRACT: Drinking water sludge is the aggregation of clay and organic compounds which is formed in flocculation and sedimentation process. This study focused on the decomposition of the bonding by flocculating agent and organic matter, and proposed a simplified method for the prediction of changes in shear strength caused by DWS decompositions. The changes in shear strength of DWS were investigated by triaxial compression tests. The specimens were produced using the DWS which was mainly decomposed by H₂O₂ solutions. As a result, volumetric strain became large in the large axial strain range, and the maximum deviator stress decreased concomitantly with the decrease in ignition loss. After the organic matter was decomposed until 1.38%, the internal friction angle decreased from approximately 38.8° to 37.6°. The changes of shear strength were related to the substantial period in geotechnical works such as road infrastructures. The decompositions of the mechanical bridging and organic matter were described based on diffusion-controlled Al leaching and aerobic biodegradation, respectively.

RÉSUMÉ : La boue de l'eau potable est l'agrégation d'argile et composés organiques qui sont formées dans le flocculation et processus de la sédimentation. Cette étude s'est concentrée sur la décomposition de la liaison par agent du flocculating et matière organique et a proposé une méthode simplifiée pour la prédiction de changements dans la force du ciseau causée par les décompositions DWS. Les changements dans la force du ciseau de DWS ont été enquêtés par les épreuves de la compression du triaxial. Les spécimens ont été produits utiliser le DWS qui était principalement décomposé par les solutions H₂O₂. En conséquence, la tension volumétrique est devenue grande dans la grande gamme de la tension axiale, et le stress du déviateur maximal a diminué de façon concomitante avec la baisse dans la perte de l'ignition. Après que la matière organique ait été décomposée jusqu'à 1.38%, l'angle de la friction interne a diminué d'approximately 38.8° à 37. 6°. Les changements de force du ciseau ont été mis en rapport avec période substantielle dans le geotechnical travaille tel qu'infrastructures de route. Les décompositions de la mécanique qui lie et la matière organique a été décrite basé sur Al diffusion-contrôlé qui lessive et biodégradation aérobique, respectivement.

KEYWORDS: waste, sludge, reuse, organic matter, decomposition, aluminum, leaching, shear strength

1 INTRODUCTION

Drinking water sludge (DWS) which is discharged during water purification is presently classified as industrial waste in Japan. A microphotograph of DWS is represented in Fig. 1. DWS is the aggregation of clay and organic compounds which is formed in flocculation and sedimentation process. Reuse and disposal of DWS are an important viewpoint in the sound material-cycle society.

The geotechnical use of DWS such as a road infrastructure material is greatly anticipated. So far, mechanical and leaching characteristics of DWS have been investigated (Roque and Carvalho, 2006; Watanabe et al., 2009). Specifically, it is presumed that Al leaching results from the Al-type flocculating agent. Watanabe et al. (2011) showed that the organic matter decomposition decreased in shear strength of DWS. To reuse DWS safely, the evaluation of the durability is strongly required. Therefore, this study focused on the decomposition of the bonding by flocculating agent and organic matter, and proposed a simplified method for the prediction of changes in shear strength caused by DWS decompositions in geotechnical works.

2 DECOMPOSITION MECHANISM

The DWS formation during water purification process is based on two phases in Fig. 2. Less than 10⁻⁶ m diameter particles including organic matter were flocculated by chemicals as first binding, and floc settled and consolidated (Montgomery, 1985). Bonding force is generated by the
mechanical bridging of flocculating agent: polyaluminum

chloride is frequently used in Japan. Some hydrophilic parts of flocculating agent remain and bind clods as second binding, then more than $10^{-5}$ m diameter clods presumably form DWS’s porous structure as shown in Fig. 1.

In this study, DWS was sampled in Ibaraki, Japan. Approximate organic matter content of DWS was determined by ignition loss tests. The ignition loss and fundamental properties were listed in Table 1. Ignition loss of DWS was 17.6%–27.3%. The amount of humic and fulvic acids were determined by alkaline and acid isolation (Ohkubo et al., 1998). Specifically, humic acid content was dominant for DWS. It is indicated that organic matter exists in as a solid part and a bonding as well as the mechanical bridging. A main constituent of polyaluminum chloride is Al$_2$O$_3$. Previous study has been already confirmed Al leaching by column leaching tests in Fig. 3 (Watanabe et al., 2009). Organic matter decomposition at 30 degrees has been confirmed in Fig. 4 (Watanabe et al., 2011), which means the loss of DWS particles and binders. Consequently, engineering properties of DWS after the decomposition mentioned above are interest on a discussion of DWS durability.

3 RELATION BETWEEN SHEAR STRENGTH AND DECOMPOSITION

Shear characteristics of DWS after decomposition were investigated to elucidate the necessity of the mechanical bridging and organic matter on DWS’s structure. The DWS which was mainly decomposed by H$_2$O$_2$ solution was used in triaxial compression tests.

3.1 Experimental procedure

Triaxial compression tests were executed using the DWS for which the mechanical bridging and organic matter had been decomposed by the H$_2$O$_2$ solution. The apparatus for the triaxial compression tests is portrayed in Fig. 5. The specimen had 100-mm height and 50-mm diameter. Specimens were produced by dynamic compaction using DWS-A. The dry density in CASES 1–4 was 0.815–0.825 Mg/m$^3$ which corresponds to compaction degree 76%. First, the specimen was isotropically confined by 10 kPa. Then the H$_2$O$_2$ solution (6%, 9%, 15%) was percolated through the specimen by 10 kPa of water pressure. Specimens in CASES 2, 3 and 4 were decomposed by the H$_2$O$_2$ solution. During H$_2$O$_2$ percolation, CO$_2$ was generated by oxidation. The CO$_2$ continuously flow into the sealed desiccator, and the CO$_2$ concentration was measured using a wireless CO$_2$ sensor. The completion of oxidation was confirmed as the CO$_2$ concentration converged, which prevented partial saturation of the specimen during shearing. The decrease in organic matter by H$_2$O$_2$ has been investigated in Table 2. The discharged water was collected, and Al concentration was measured. The distilled water was percolated after H$_2$O$_2$ percolation, and more than 0.95 of the B-value was confirmed for specimen saturation. The isotropic consolidation pressure was 50 kPa or 100 kPa. The triaxial tests were executed in the drainage condition with 0.1%/min of the strain rate.
As mentioned above, Al leaches during \( \text{H}_2\text{O}_2 \) percolation. The deformation of the mechanical bridging by Al leaching causes at the same time as organic matter decomposition. To distinguish the influence of Al leaching on the mechanical deformation from organic matter decomposition, shear characteristics of the DWS for which the mechanical bridging had been decomposed by leaching using distilled water adjust pH 4.0 using HNO\(_3\) which was almost same pH as \( \text{H}_2\text{O}_2 \) had been decomposed by leaching using distilled water adjust characteristics of the DWS for which the mechanical bridging was lost through organic matter decomposition.

The strength of DWS clods decreased because the bonding was lost by the decomposition of organic matter. Consequently, results show that the decomposition of organic matter do not influence DWS cohesion. Assuming that the cohesion is 0 kN/m\(^2\), the relation between the internal friction angle and the decomposition rate of organic matter was shown in Fig. 7. Figure 7. Relation between internal friction angle and decomposition rate of organic matter.

4 PREDICTION OF SHEAR STRENGTH TRANSITION

Al release and organic matter decomposition cause the decrease in shear strength of DWS. To evaluate the durability of DWS as a geo-material, this study related the change of its shear strength to the substantial period in geotechnical works such as road infrastructures.

4.1 Shear strength transition addressing decomposition of mechanical bridging

Decomposition of the mechanical bridging is described as Al leaching behavior. When DWS is used as a subgrade material under groundwater level, Al diffusively leaches. Therefore, the Al leaching behavior is described as a diffusion equation based on the Fick’s law.

\[ \frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \]  

(1)

In that equation, \( D \) is the coefficient of diffusion [m\(^2\)/s], \( C \) is the concentration [mg/L], and \( x \) signifies the distance from particle surface [m]. An initial condition and boundary conditions are shown as follows:

\[ t = 0, x < 0; C = C_0 \]

\[ t > 0, x = 0; C = C_i, t > 0, x = -\infty; C = C_0 \]

where \( C_0 \) represents the internal concentration of material [mg/L], and \( C_i \) is the constant concentration [mg/L]. Assuming \( C_0 \) is sufficiently higher than \( C_i \), the cumulative release \( M \) is derived.
M = 2C_{\text{o}} \sqrt{\frac{D_c}{\pi}}(t_f - t_i) \quad (2)

D_c of Al is $4.77 \times 10^{15}$ m$^2$/s which was obtained by the serial batch leaching test (Watanabe et al., 2010). As $t_f$ is 0, the Eq. 2 transforms to Eq. 3, and it relates the cumulative Al release $M_{\text{exp}}$ obtained in the triaxial tests to the elapsed time $T$.

$$T = \frac{M_{\text{exp}} \cdot \pi}{4C_{\text{o}} \cdot D_c} \quad (3)$$

Therefore, the transition of the internal friction angle caused by decomposing the mechanical bridging is calculated as shown in Table 3. Approximate 1.3% decrease in the internal friction angle supposedly causes during 38 years.

4.2 Shear strength transition addressing decomposition of organic matter

The organic matter decomposition of DWS as a subgrade material is able to be interpreted as following, assuming aerobic and unsaturated condition. Jenny (1941) described the decrease in soil organic matter as Eq. 4.

$$\frac{dX}{dt} = rX \quad (4)$$

where $X$ is the mass of organic matter and $r$ is the rate of decomposition. A solution of Eq. 4 is given by Eq. 5.

$$X = X_0 \cdot e^{-rt} \quad (5)$$

Assuming aerobic biodegradation, discharged CO$_2$ is originated from the carbon loss of decomposed organic matter approximately. From the results of constant temperature storage in aerobic condition of DWS (Watanabe et al., 2011), $r$ is determined by fitting Eq. 5 into the experimental data as shown in Fig. 8. The daily CO$_2$ discharge during organic matter decomposition corresponds to the decomposition mass of organic matter, so the time integral of Eq. 5 approximately represents the total mass of decomposed organic matter $Q_{\text{dec}}$.

$$Q_{\text{dec}} = \frac{X_0}{r} \left(e^{-rt_1} - e^{-rt_f}\right) \times \frac{12}{44} \quad (6)$$

Calculation results for the transition of the internal friction angle caused by decomposing the organic matter are listed in Table 4. Approximate 3.1% decrease in the internal friction angle by organic matter decomposition causes during 22 days in aerobic condition. Assumption of aerobic condition is not suitable for practice, so this study confirmed the organic matter decomposition in site. In the experimental construction that DWS was used as a backfill material of water pipe construction, the DWS layer was taken a position of -0.4 to -0.9 m depth under asphalt surface. The compaction degree was approximately 64–76%. The monitoring term was 19 months. As shown in Table 5, ignition loss slightly decreased at the end of the experiment. It is presumed that organic matter decomposition slowly progressed in contrast to the constant temperature storage because of anaerobic condition and lower temperature. The proposed method with aerobic condition excessively estimates the degradation in contrast of underground conditions.

5 CONCLUSIONS

DWS is the aggregation of clay and organic compounds. Focusing on the chemical bonding by flocculating agent and organic matter, a simplified method for the prediction of changes in shear strength of DWS in geotechnical works was proposed. The decomposition of the mechanical bridging and the organic matter was described based on diffusion-controlled

<table>
<thead>
<tr>
<th>Table 3. Shear strength transition addressing decomposing the mechanical bridging.</th>
<th>Cumulative Al release (mg/kg)</th>
<th>Calculated elapsed time (y)</th>
<th>Internal friction angle (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>37.2</td>
</tr>
<tr>
<td>0.130</td>
<td>17.4</td>
<td>37.6</td>
<td></td>
</tr>
<tr>
<td>0.192</td>
<td>38.0</td>
<td>36.7</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 4. Shear strength transition addressing decomposing the organic matter.</th>
<th>Decomposition rate of organic matter (%)</th>
<th>Calculated elapsed time (d)</th>
<th>Internal friction angle (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>38.8</td>
</tr>
<tr>
<td>0.14</td>
<td>2</td>
<td>39.2</td>
<td></td>
</tr>
<tr>
<td>0.59</td>
<td>12</td>
<td>38.5</td>
<td></td>
</tr>
<tr>
<td>1.38</td>
<td>22</td>
<td>37.6</td>
<td></td>
</tr>
</tbody>
</table>

| Table 5. In-situ monitoring results of ignition loss of DWS | Compaction degree (%) | CBR (%) | Ignition loss (%) before construction | 19 months later |
|---|---|---|---|
| Air-dried DWS | 75.9 | 38.1 | 16.9 | 16.6 |
| Filter-pressed DWS | 64.3 | 55.3 | 24.7 | 24.0 |

Al leaching and aerobic biodegradation, respectively. The methodology proposed in this paper is significant to encourage safe geotechnical utilizations through estimations of the usable term for not only DWS but also available waste or by-products.

REFERENCES

ABSTRACT: Road construction over soft ground presents considerable technical challenges. Such roads often serve remote communities and carry low levels of traffic; construction and maintenance must be achieved within very limited budgets. There are two main approaches to such construction: above ground (floating) and below ground (buried) construction. Floating construction is generally used where a relatively stiff material, such as fibrous peat, overlies a less competent material, such as amorphous peat. Buried construction is generally used in more competent materials, or in soft materials of shallower depth such that removal is viable. In both cases lightweight construction materials are desirable but can be costly. This paper describes tyre bales as a lightweight construction material and specifically addresses issues in relation to their use as a foundation material for roads over soft ground.

RÉSUMÉ : La construction de routes sur sol meuble présente des défis techniques considérables. Ces routes desservent souvent des collectivités éloignées et connaissent de faibles niveaux de trafic ; leur construction et leur entretien doivent respecter des budgets très serrés. Il existe deux méthodes principales pour ce type de construction : au-dessus du sol (construction flottante) et en dessous du sol (construction enterrée). La construction flottante est généralement employée lorsqu'un matériau relativement rigide, comme la tourbe fibreuse, repose sur un matériau moins compétent, tel que la tourbe amorphe. La construction enterrée est généralement privilégiée en présence de matériaux plus compétents, ou de matériaux souples moins profonds dont l’élimination est viable. Dans les deux cas, il est utile d’employer des matériaux de construction légers, qui peuvent cependant s’avérer coûteux. Cet article décrit les balles de pneus comme matériau de construction léger et traite spécifiquement des problématiques liées à leur utilisation comme matériau d’assise des routes sur sol meuble.

KEYWORDS: Sustainability, reuse, recycling, foundations, tyres, bales.

1 INTRODUCTION

The construction of roads over soft ground, such as peat, presents considerable technical challenges. Many such roads serve remote communities, carry only low levels of traffic, and must be constructed and maintained within limited budgets. Where the depth of soft soil is significant, the approach to construction generally involves ‘floating’ the road on the existing subsoil. This may also involve the use of temporary surcharging and/or reinforcement at the base of the construction to help spread the load. If the depth of peat or other soft material is shallow then removal may be an option. The excavated material is then replaced by more competent materials. However, this does leave the issues of disposing of the excavated material and of preventing the adjacent material from flowing into the excavation. The resolution of either or both of these issues can prove costly, and such costs will increase rapidly with the depth of material excavated.

In both cases, the use of lightweight construction materials is desirable. This paper introduces lightweight tyre bales focusing upon their potential use as a road foundation material and draws on the author’s experience in the UK and the USA. Relative to conventional lightweight foundation materials such as expanded polystyrene, the cost of tyre bale construction is relatively low.

2 TYRE BALES

Around 48M tyres (480,000 tonnes) are scrapped in the UK each year. However, the issue of scrap tyres is by no means unique to the UK and Europe. In the USA it has been estimated that over two billion used tyres are stockpiled, and that 285M are added each year (Winter et al. 2006). In the recent past the bulk of waste tyres in the UK was stockpiled, disposed of in landfill or illegally, or sent for energy recovery (Hird et al. 2001) or processed as waste-to-energy. In Europe the Landfill Directive outlawed the disposal of tyres in landfill, with UK exceptions being made for engineered works. In the USA a number of fires in waste dumps comprising whole tyres, and concerns regarding the potential flammability of tyre shreds and chips, led the drive towards alternative solutions.

The majority of R&D activity has addressed tyre shred, chip and crumb for use in construction works. An alternative is the baling of whole tyres to produce rectilinear, lightweight/low density, permeable, porous bales of high bale-to-bale friction.

2.1 Composition, properties and behaviour

Tyre bales comprise 100 to 115 car/light goods vehicle tyres compressed into a lightweight block of mass around 800kg and density circa 0.5Mg/m³. The bales measure approximately 1.3m by 1.55m by 0.8m and are secured by five galvanized steel wires running around the length and depth of the bale (Figure 1). They have considerable potential for use in construction particularly where their low density and ease of handling places them at a premium. A porosity of around 62% and permeability of approximately 0.02m/s through the length and 0.2m/s through the depth (Simm et al. 2005) makes them ideal for drainage applications. The bale-to-bale friction angle is around 35° in dry conditions and stiffness in the vertical direction of Figure 1 is up to around 1GPa (Frielich & Zornberg, 2009; Winter et al. 2006). Furthermore, the process of tyre bale manufacture consumes around 1/16 of the energy required to shred a similar mass of tyres (Winter et al. 2006).
Substances that could potentially leach from tyres are already present in groundwater in developed areas. Studies suggest that leachate levels generally fall well below allowable regulatory limits and have negligible impact on water quality in close proximity to tyres (Hylands & Shulman, 2003) and that rates of release decrease with time (Collins et al. 2002). Similarly there is no evidence of significant deterioration of tyres buried in the ground for decades (Zornberg et al. 2004).

Tyre bale use reflects positively on the sustainable use of materials, or in thinner layers of less competent materials for example applications; and end of service life options.

3 METHODS OF CONSTRUCTION

There are two main approaches to road construction over soft ground: above ground (floating); and below ground (buried). Both use large volumes of granular fill.

Spontaneous fires in whole tyre dumps are not known to the author. In the USA, while combustion due to sparks from agricultural machinery and lightning have been reported, most observers suspect some form of arson in almost all cases. Baling whole uncompressed tyres reduces the volume by a factor of four to five, greatly reducing the available oxygen as well as the exposed rubber surface area as tyre-to-tyre contacts are formed, without exposing any steel reinforcing in the tyres. The exothermic oxidation reaction potential is significantly lower than for whole tyres and the risk of spontaneous combustion from tyre bales is viewed as extremely low. A modelled storage condition for a 17.5m by 6.0m by 3.0m volume of bales needed to reach and maintain a temperature of 188°C for 39 days before spontaneous combustion became possible (Simm et al. 2005). In contrast reports have been made of internal heating of tyre shred and of apparently spontaneously combusted fires in large volumes in the USA (Sonti et al. 2000). Further details of tyre bale properties and behaviours are available (Anon. 2007).

Tyre bale use reflects positively on the sustainable use of materials and energy and other factors. In the last decade the application level has moved from domestic works/river bank erosion projects to slope failure repairs adjacent to a major Interstate Highway in the USA (Winter et al. 2009) and the construction of a lightweight embankment as part of the A421 A1-M1 link road construction which won the British Geotechnical Association’s prestigious Fleming Award.

An adequate supply of tyres, and the resources to turn them into bales must be secured prior to the commencement of a project. As bales are around ¼ to ½ the volume of whole tyres it can be particularly difficult to gauge the volume of bales that will result from a stockpile of tyres. A series of nomograms was developed by Winter et al. (2006) and further refined (Anon. 2007) to rapidly describe the number of bales required to fill a given volume, the number of tyres likely to be used in their manufacture, and the number of eight hour (two man) shifts required to manufacture those tyre bales.

Tyre bale use reflects positively on the sustainable use of materials and energy and other factors. In the last decade the application level has moved from domestic works/river bank erosion projects to slope failure repairs adjacent to a major Interstate Highway in the USA (Winter et al. 2009) and the construction of a lightweight embankment as part of the A421 A1-M1 link road construction which won the British Geotechnical Association’s prestigious Fleming Award.

An adequate supply of tyres, and the resources to turn them into bales must be secured prior to the commencement of a project. As bales are around ¼ to ½ the volume of whole tyres it can be particularly difficult to gauge the volume of bales that will result from a stockpile of tyres. A series of nomograms was developed by Winter et al. (2006) and further refined (Anon. 2007) to rapidly describe the number of bales required to fill a given volume, the number of tyres likely to be used in their manufacture, and the number of eight hour (two man) shifts required to manufacture those tyre bales.

Tyre bale use reflects positively on the sustainable use of materials and energy and other factors. In the last decade the application level has moved from domestic works/river bank erosion projects to slope failure repairs adjacent to a major Interstate Highway in the USA (Winter et al. 2009) and the construction of a lightweight embankment as part of the A421 A1-M1 link road construction which won the British Geotechnical Association’s prestigious Fleming Award.

An adequate supply of tyres, and the resources to turn them into bales must be secured prior to the commencement of a project. As bales are around ¼ to ½ the volume of whole tyres it can be particularly difficult to gauge the volume of bales that will result from a stockpile of tyres. A series of nomograms was developed by Winter et al. (2006) and further refined (Anon. 2007) to rapidly describe the number of bales required to fill a given volume, the number of tyres likely to be used in their manufacture, and the number of eight hour (two man) shifts required to manufacture those tyre bales.

Tyre bale use reflects positively on the sustainable use of materials and energy and other factors. In the last decade the application level has moved from domestic works/river bank erosion projects to slope failure repairs adjacent to a major Interstate Highway in the USA (Winter et al. 2009) and the construction of a lightweight embankment as part of the A421 A1-M1 link road construction which won the British Geotechnical Association’s prestigious Fleming Award.

An adequate supply of tyres, and the resources to turn them into bales must be secured prior to the commencement of a project. As bales are around ¼ to ½ the volume of whole tyres it can be particularly difficult to gauge the volume of bales that will result from a stockpile of tyres. A series of nomograms was developed by Winter et al. (2006) and further refined (Anon. 2007) to rapidly describe the number of bales required to fill a given volume, the number of tyres likely to be used in their manufacture, and the number of eight hour (two man) shifts required to manufacture those tyre bales.

Tyre bale use reflects positively on the sustainable use of materials and energy and other factors. In the last decade the application level has moved from domestic works/river bank erosion projects to slope failure repairs adjacent to a major Interstate Highway in the USA (Winter et al. 2009) and the construction of a lightweight embankment as part of the A421 A1-M1 link road construction which won the British Geotechnical Association’s prestigious Fleming Award.

An adequate supply of tyres, and the resources to turn them into bales must be secured prior to the commencement of a project. As bales are around ¼ to ½ the volume of whole tyres it can be particularly difficult to gauge the volume of bales that will result from a stockpile of tyres. A series of nomograms was developed by Winter et al. (2006) and further refined (Anon. 2007) to rapidly describe the number of bales required to fill a given volume, the number of tyres likely to be used in their manufacture, and the number of eight hour (two man) shifts required to manufacture those tyre bales.
consolidated silts and clays, and soft predominately mineral soils (albeit with exceptions). A geotextile helps to spread the foundation load. Often the repair or reconstruction of an existing road over soft ground is required as a result of differential settlement which leaves an uneven surface with poor ride quality and an increased risk of flooding. The placement of material to raise and regulate the pavement surface increases the formation load causing further differential settlement; replacement of the existing material is thus necessary.

4 CONSTRUCTION APPROACHES

The construction and rehabilitation of low-volume roads over soft ground is an ideal application for tyre bales. While there is currently little information to prove their use with higher traffic levels (in excess of a few hundred vehicles/day AADT) there are no pressing reasons why such uses should not be successful. Low-volume tyre bale roads have been successfully constructed both above and below ground. A geotextile separator is used between the in-situ soil and the tyre bales, usually with a regulating layer of sand. The geotextile helps to prevent differential movement of the bales during and after construction. The decision as to whether the construction should be above or below ground is an important determinant of the approach to the design and construction.

Analytical input for low-volume road design on soft ground is often limited. The strength and stiffness properties of the soil involved are usually at or close to the lower limit of measurement, rendering input parameters subject to large errors. The sampling process may also disrupt the soil structure leading to values lower than the field condition. Accordingly many roads are designed on an empirical, specification-led basis.

The following sections summarise the main construction steps and issues and offer guidance based upon experience of successful projects and established good practice in constructing low-volume roads over soft ground using tyre bales. Further details are given by Winter et al. (2006) and Anon. (2007).

4.1 Excavation and preparation

For buried construction, excavation is the first construction activity. Low ground-pressure, tracked plant is preferred as is working in drier weather when the moisture content of the soil is minimised and strength and stiffness are maximised.

A suitable geotextile should be installed either at ground surface level or in the excavation followed by a regulating layer of sand if required. All geotextile-to-geotextile interfaces should have an overlap of 1m. The use of a geotextile has a number of advantages including aiding working conditions in soft soils, strengthening the structure by tying together the assembly of bales, and providing separation between the bales and the subsoil and thus preventing the ingress of fines. Randomly orientated, bonded, non-woven geotextiles have been found to be effective. Their main function is separation, with strength and resistance to clogging the most important properties. Geotextile design procedures should reflect local standards. The geotextile should be placed in the base of the excavation, or on the cleared ground. Sufficient excess should be allowed at either side to allow the bale assembly to be completely wrapped in the geotextile with a 1m overlap.

Rapid cellular construction minimises excavation size, exposure of the soil to weather and the likelihood of side slope failure. Bale sizes mean that excavations are unlikely to exceed 1m, but an assessment of the possibility of sidewall collapse and the associated risks to workers and others during the execution of such operations is essential.

4.2 Placement and alignment

Tyre bale handling must incur the minimum risk of damage to the steel tie-wires. The most successful means of handling tyre bales has been found to be a ‘loggers’-clam’, which can be attached to a variety of hydraulic equipment and provides an appropriate lift-and-place methodology while allowing the bale to be rotated to the correct alignment. Alternative forms of handling bales include brick-grabs and forklifts (Anon. 2007).

The manufacturing process renders tyre bales inherently heterogeneous. Information on the relative stiffness in each of the three directions is not currently available. Tyre bales exhibit a high stiffness when loads are applied vertically to the 1.3m by 1.55m plane (Figure 1); accordingly they are usually installed as illustrated in Figure 1 for applications that attract high vertical loads such as road foundations. The 1.55m by 0.8m plane is perpendicular to the load applied during manufacture and it is recommended that it is aligned perpendicular to the longitudinal confining stresses (i.e. with the tie-wires in line with the road).

While there are different layout options for the two-dimensional placement of tyre bales (i.e. in a single layer) a straightforward ‘chessboard’ pattern, as viewed in plan, is generally the easiest to construct and is recommended.

A regulating layer of sand is normally required between the top of the tyre bales and the geotextile wrapped over the top of the layer to help eliminate small variations in level.

The foregoing assumes that a single layer of bales is to support the road. If two or more layers are required then the second layer should be placed on top of the first, stepped in at either side to provide around half a bale width of overlap.

4.3 Filling of voids

The sub-rectangular shape of tyre bales means that voids remain at the corners of each bale even when they are buttied up against one other. The design generally requires the stiffness and stability of the structure to be maximized and thus the voids should generally be filled (Figure 3). Coarse sand has been used successfully as have single-sized aggregate pellets. Crushed glass may be less likely to clog or arch than sand when wet, but is expensive. The most effective method of ensuring that the voids are filled has been found to be to bulldoze a 150mm to 300mm layer on top of the bale layer and then to apply a vibrating roller to the layer to vibrate the fill into the voids (Figure 3).

The fill material affects the density of the structure, with the voids taking up an estimated 4% to 8% (Anon. 2007) of the nominal rectangular bale volume, and must be allowed for in design calculations. The effects of regulating layer(s) above or below the tyre bale layer must also be taken into account.
Once the fill operation for a cell has been completed for a section of road, the geotextile should be wrapped around the bale-fill composite with an overlap of around 1m. A crushed rock sub-base should be placed and compacted on top of the completed section. A thickness of 150mm is likely to be sufficient to provide a construction platform for the works to continue without damaging the geotextile. The final thickness of sub-base must be assessed to ensure sufficient capacity during normal use and should be the subject of site-specific design. After these operations are completed the construction may proceed to the next cell, repeating the process described above until the road has been completed.

Figure 3. Bulldozing sand to fill voids, County Road 342 (CR342), 2000 (left); vibrating sand into inter-bale voids, CR647, 1999. (Courtesy Ken Smith, Chautauqua Co Dept of Public Facilities, NY.)

4.4 Pavement construction and drainage

Pavement construction is beyond the scope of this paper but further guidance is given by Winter et al. (2006) as is more detail on drainage considerations. The design should reflect local standards and climatic conditions.

5 SUCCESSFUL APPLICATIONS

Successful applications involving the construction of tyre bale road foundations have been achieved in both the USA (New York State) and the UK (Winter et al. 2005). Chautauqua County Department of Public Facilities completed five projects using tyre bales as a lightweight subgrade replacement for roads over soft ground (Figure 4). The tyres result from the clean-up of a tyre dump and from a tyre amnesty programme. The geology of the County is characterised by sands and gravels in the river valleys with glacially deposited fine silty clays elsewhere, primarily on the hilltops which are often depressed forming high level swamps. These materials are stable if dry but are sensitive to moisture and to the freeze thaw cycle which can turn them into a material like ‘pottery slip’. Conventional unpaved roads constructed on them can turn into impassable quagmires. Tyre bale road construction was targeted on these roads.

Figure 4. Completed CR342 2004 after four years in service (left); B871 in Highland, UK (right, Courtesy G Smith, Highland Council.).

To date with the roads having been in service for up to nine years no major signs of distress have been observed that could be attributed to the presence of tyre bales. In the case of CR342 the traffic levels have been greatly increased (up to around 1,500 to 2,000 vehicles per day AADT) due to a new residential development in the vicinity.

A public road was constructed by Highland Council (UK) in late-2002 (Anon. 2003); performance has been satisfactory despite extreme loadings imposed by a very high proportion of heavy logging trucks using the route (Figure 4).

6 CONCLUSIONS

The use of lightweight tyre bales in the construction of road foundations over soft ground has the potential to satisfy the demand for low cost materials exhibiting such a beneficial property. Such uses also help to address society’s broader problem in respect of the large volumes of waste tyres which, in Europe at least, may no longer be sent to landfill for disposal; clearly such beneficial uses for waste tyres are required. Supply and production issues are addressed and material costs shown to be comparable with conventional materials such as Type 1 sub-base. However, the key strength of tyre bales is their modular nature which leads to potential savings in plant and labour and the associated time savings. In some cases the low cost of tyre bales relative to other lightweight materials, such as expanded poly styrene, may allow the economic construction or rehabilitation of infrastructure in remote areas that would otherwise not be viable. An approach to the construction of low-volume road foundations on soft ground using tyre bales has been developed and is summarized herein.

Tyre bales offer a useful tool for the engineer across a wide range of construction applications that variously exploit their beneficial properties: namely low density, high permeability, high porosity and high bale-to-bale friction.

7 ACKNOWLEDGEMENTS

Copyright TRL Limited 2013. Funding was by the Veolia Environmental Trust Landfill Tax Credits Scheme supported by Inverness & Nairn Enterprise and Transport Scotland. The Royal Academy of Engineering part-funded (International Travel Award No. 04-301) a study visit to the USA.

8 REFERENCES


Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering

CHALLENGES AND INNOVATIONS IN GEOTECHNICS

Actes du 18e Congrès International de Mécanique des sols et de Géotechnique

DÉFIS ET INNOVATIONS EN GÉOTECHNIQUE

The 18th International Conference on Soil Mechanics and Geotechnical Engineering (Paris 2013) was devoted to “Challenges and Innovations in Geotechnics”. The call for abstracts was based on a large series of themes covering most aspects of Geotechnical Engineering and around 800 abstracts were received from the Member Societies. According to the ISSMGE new vision for strengthening the role of the Technical Committees, the papers were distributed to the corresponding Technical Committees, which then selected the General Reporters, the Invited Speakers and the Poster presentations. The involvement of TCs (which could also propose Workshop Sessions on an independent basis) was enthusiastic and successful, which probably explains the success of the Conference with more than 1500 delegates.

The four volumes of the Proceedings contain the Terzaghi Oration, the Honour lectures, the Special lectures followed by the papers presented according to the relevant TC and introduced by the TC General Report. All volumes, together with late papers, will also be made available online free of charge. These volumes will provide a state of the art and serve as an essential reference for practitioners, academics and researchers involved in Soil Mechanics and Geotechnical Engineering.

Le 18e Congrès International de Mécanique des sols et de Géotechnique (Paris 2013) a été dédié aux «Défis et Innovations en Géotechnique». L’appel à résumé était ouvert sur un large éventail de thèmes couvrant la plupart des aspects de la Géotechnique ; les Sociétés Membres ont sélectionné environ 800 résumés. En cohérence avec le souhait de la Société Internationale (ISSMGE) de renforcer le rôle des Comités Techniques (CTs), les contributions ont été transmises aux CTs correspondants, en charge de choisir les Rapporteurs Généraux, les Orateurs Invités et les Présentations sur Posters. L’implication des CTs, qui pouvaient en outre proposer des sessions d’Atelier à leur convenance, a été enthousiaste et fructueuse : ceci explique probablement le succès du Congrès avec plus de 1500 délégués.


Paris 2013