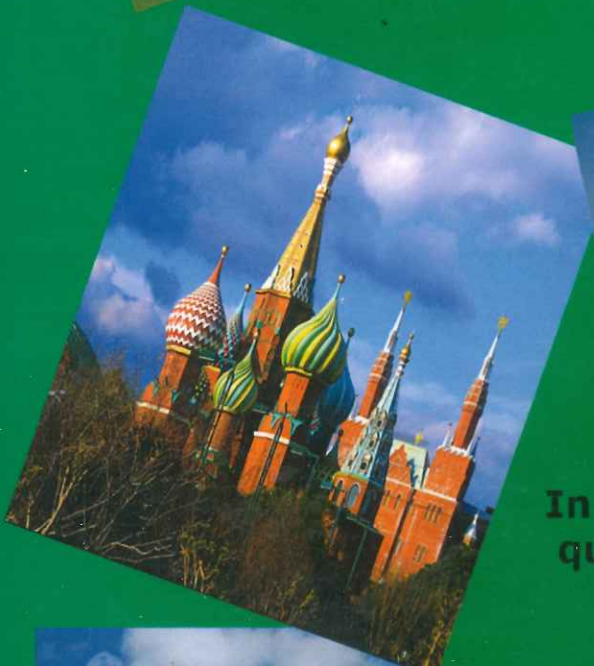
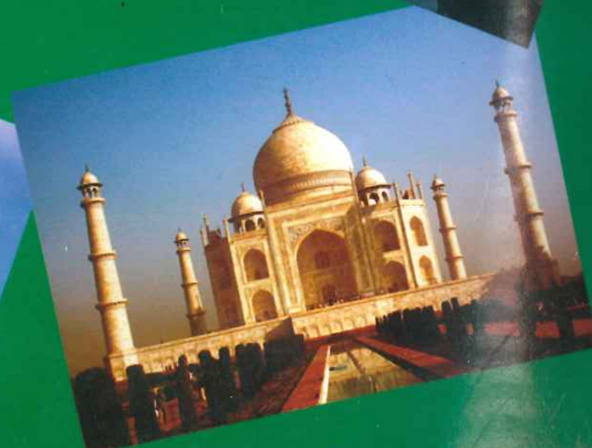
PAPERS FROM INTERNATIONAL CONFERENCES IN GEOTECHNICS

PREPARED
BY
PROF.DR.LULJETA BOZO

Papers From Albanian Authors

PERS FROM INTERNATIONAL CONFERENCES





In this edition you will be acquainted with some scietific works of the Albanian Engineers in the field of Geotechnics. The authors works held in this book are presented

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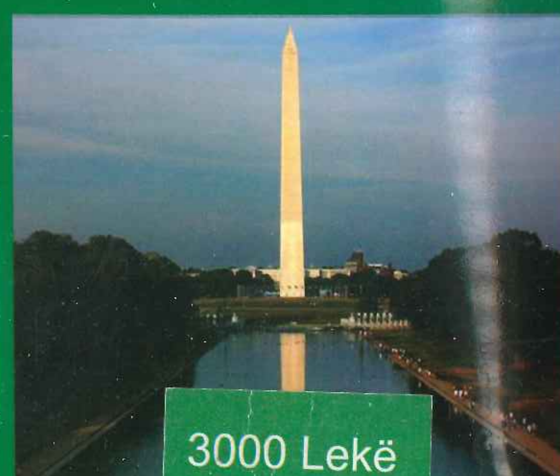

3000 Lekë

PAPERS FROM INTERNATIONAL CONFERENCES IN GEOTECHNICS

2002-2010

BOTIME NE KONFERENCA NDERKOMBETARE NE GJEOTEKNIKE

2002-2010

Msc Ing. Uderzhi

Prepared by Prof.Dr.Luljeta BOZO

Besnikut
me respekt
Luljeta Bozo

TIRANE 2011

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Themeli Andrea
Zeqo Abdulla

Geotechnics- immense field of the researches, explorations inventions to enigma tied by ground.

Scientific works of Albanian Authors.

I NTRODUCTION

Geotechnics is one of the more (nice) beautiful, interesting field, full surprises and engimas, which was provoked the immense-researches of the geotechnical specialists. In these field in Albania, was worked and continuously are working the civil engineers, geologists, geoenvironmental engineers, seismologist etc. The scientific work of Albanian specialists was a modest contribution in development of the geotechnical sciences in our country and in the world. The scientific work of our specialists was realized in the following directions:

- Behaviour of the soils and rocks under different actions
- Interaction soil-structure
- Influence of the climatic, mechanical, human activity ect, on the ground and geoenvironment phenomena's
- Behaviour of the soils and rocks when in them can to constructed different constructions.
- Improvement of the soils by different methods
- Exploration of the new correlations between the soils characteristics and taking off new elements for different laws
- Behaviour of the soils under seismic loads ect.

After creation of Albanian Geotechnical Society (2001) and their inclusion in ISSMGE the scientific works of Albanian geotechnical, geologists, geonevironmental engineers and researches beginning to published and outside of Albania. So their research work was published in the foreign prestige's periodical, in the proceedings of International Conferences and Congresses ect.

In these publication we can to present some of scientific and research work of Albanian authors which was presented in period 2002-2010 in the different International Conferences.

Also the readers can to know our researches by their brief CV

These publication was prepared and edited by Prof.Dr.Luljeta Bozo.

Gjeoteknika fushë e pafund kërkimesh të enigmave që lidhen me truallin.

Punime shkencore te autoreve shqiptare

HYRJE

Gjeoteknika është një nga fushat më të bukura më interesante, plotë të papritura dhe enigma që kanë ngacmuar kërkimet e pafund të specialisteve.
Në këtë fushe, edhe në Shqiperi, është punuar dhe vazhdon të punohet nga specialistë të ndërtimit të gjeologjisë, të mjedisit, të sizmikës etj. Punimet shkëncore të specialistëve shqiptarë kanë qënë një kontribut modest në zhvillimin e kësaj shkence shumë të gjërë si në vend ashtu dhe në botë.

Konkretisht punimet e specialistëve shqiptarë të realizuar në vend jane në drejtimet e mëposhtme:

- Sjellja e reagimi i dherave dhe i shkëmbit ndaj veprimeve të jashtme.
- Bashkëveprimi truall-strukturë
- Ndikimi i dukurive klimatike, endogjene, mekanike e të veprimtarisë humane etj, në truall dhe në mjedis.
- Reagimi i truallit kur në brendësi të tij ndërtohen struktura të ndryshme ndërtimore.
- Vetitë e reja që mund të fitojë dheu pas përpunimeve të ndryshme që i bëhen atij duke u shndëruar në një material krejt tjetër nga ai i origjinës.
- Zhvillimi i mëtejshëm i teorive të ndryshme nëpërmjet zhvillimit të lidhjeve të reja mes parametrave të dheut
- Pasurimi i mëtejshëm i reagimit të truallit në lëkundje sizmike etj.

Pas krijimit të Shoqatës Gjeoteknikët e Shqipërisë (2001) dhe hyrjes së saj si anëtare shumë aktive në Shoqatën Botërore të Mekanikës së Dherave dhe Inxhinierisë Gjeoteknike ISSMGE, punimet kërkimore e shkencore në fushën e gjeoteknikës dhe të gjeomjedisit të autorëve tanë filluan të publikohen edhe jashtë Shqipërisë. Kështu ato janë botuar në revista shkencore prestigjoze, në Proccedings të Konferencave e Kongreseve Ndërkombëtarre.
Në këtë botim po paraqesim një pjesë të punimeve shkencore të autorëve shqiptarë të paraqitura në Konferenca e Kongrese Ndërkombëtare gjatë periudhës 2002-2010.

Gjithashtu autoret e këtyre punimeve lexuesi mund t'i njohë nëpërmjet CV të përmbledhura të tyre.
Botimi është realizuar nën kujdesin e Prof.Dr.Luljeta BOZO

CONTENT

❖ Year 2004

- Fifth International Conference on Case Histories in Geotechnical Engineering. April 11-17 2004. New York USA
  - ✓ Them new resolution of foundations in Durres city. L.Bozo

❖ Year 2005

- International Seminare "Les Compactage des sols". Mars 25-26 Hammamet Tunis.
  - ✓ Compactage des coushes de la route Tirana-Durres en Albanie. L Bozo.
- The second international symposium for construction. 21-22 April Tetovo-Macedonia.
  - ✓ Construction in the weak terrains L.Bozo
- Baltic Geotechnical X 2005. 11-13 May Riga Latvia
  - ✓ Geotechnical Investigations for the rehabilitation and reconstruction of Durres-Port. L.Bozo L.Ahmetaj, S.Allkja
- 2005 International Conference on Landslide Risk Management .May 31-June3 Vancouver Canada
  - ✓ Landslide Risk Assesment on roads in Albania. L.Bozo, Y.Muceku, N.Shkodrani
- International Symposium "50 year of pressure maters" 22-24 August. Paris France.
  - ✓ Report National. Historie de pressiometres et leur usage en etudes et calculation geotechnique in Albanie. A.Zeqo, L.Bozo.
- European Young Geotechnical Conference Zagreb-Croatia.
  - ✓ Coefficient of subgrade reaction Xh. Alliu M Balilaj

❖ Year 2006

- 5th International Congress on Environmental Geotechnics . 24-26 May Cardiff Wales UK
  - ✓ Environmental and Geotechnical Problems of Albania's Mine Site. L.Bozo K.Cela
- XIIIth Danube European Conference on Geotechnical Engineering.29-31 May Lubjana Slovenia.
  - ✓ Influence of the deformation parameters of soils in selection of the static scheme Maliq's Bridge. L.Bozo, L.Ahmetaj, S.Allkja.
- 5-th Hellenic Conference on Geotechnical and Geonevironmental Engineering. 29-May 2 June Xanthi Greece
  - ✓ National report for the activities of Albania Geotechnical Society L Bozo.

❖ Year 2007

- 4th International Conference on Earthquake Geotechnical Engineering 25-28 June Thessaloniki Greece.
  - ✓ Slope stability in active seismic zones in Albania. L.Bozo, A.Frasheri, Y.Muceku.
- XIII Pan-American Conference on Soil Mechanics and Geotechnical Engineering 16-20 Julio Isla de Margarita Republic Bolivarian de Venezuela.
  - ✓ Evolution of Geo-Engineering education in Albania. L.Bozo
- XIVth European Conference on Soil Mechanics and Geotechnical Engineering. 24-27 September. Madrid-Spain
  - ✓ The influence of geological and geotechnical conditions on reinforcement structures for deep excavation nearly existing structures. L.Bozo, S.Allkja, L.Ahmetaj
  - ✓ Influence of deep excavation on nearby buildings. E.Paci
- European Conference of Young Geotechnical Engineers . Ancona Italy
  - ✓ Landslide stabilization by means of deep drainage A.Themeli
  - ✓ Three dimensional finite element analysis of laterally loaded piles. E.Bukaci

❖ Year 2008

- First International Conference on Education and Training in Geo-engineering sciences . June 2-4 Costanzia Romania.
  - ✓ First and second cycle degree programmers and impact of the Bologna process in Albania, L.Bozo
- International Geotechnical Conference "Development of Urban Aeras and Geotechnical Engineering . June 16-19 Sain Peterburg.
  - ✓ The construction aspects of a multistory building above a mediaval wall in Vlora city Albania .E.Paci
- 6th International Conference on Case Histories in Geotechnical Engineering 11-16 August. Arlington USA.
  - ✓ Failure of retaining structures in town Lezha and their consequence in neighboring building. L.Bozo
- 11th Baltic Sea Geotechnical Conference. 15-18 September Gdansk-Poland.
  - ✓ Geotechnical study of industrial zone in reference to limit state. L.Bozo Y.Muceku.

❖ Year 2009

- International Symposium on Geoenvironmental Engineering. 8-10 September Hangzhou China.
  - ✓ Geoenvironmental risk assessment in Albania Gj.Ikonomi, L.Bozo

- 17th International Conference on Soil Mechanics and Geotechnical Engineering 5-9 October Alexandria Egypt.
    - ✓ The mass movement response of tectonics phenomena in urban aeras Albania. Y. Muceku.
    - ✓ Laboratory testing of the cohesive soils and the correlation between the resisting characteristics of soils and their physical parameters. L.Bozo, S.Allkja, L.Harizaj.
    - ✓ Training of Geotechnical Engineers in Albania. L.Bozo
- 2th International Conference Long term behavior of dams 12-13 October Graz Austria.
    - ✓ Damages and risk assessment of Thana dam in Lushnja region Albania. L.Bozo, Y.Muceku.

❖ Year 2010

- Fifth International Conference Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics. 24-29 May San Diego California USA .
    - ✓ An upgrade of the microzonation study of the center of Tirana City. Ll.Duni, N.Kuka, L.Bozo.
    - ✓ Field and laboratory tests in Seman Deposists . L.Bozo S.Allkja L.Harizaj
- First International Conferecne of Soil and Roots Engineering Relationship 24-26 May Ardebil Iran.
    - ✓ Assesment of Soil Erosion in a Mountation Area in the Korca District Albania. O,Marko A, Lako. E.Cobani
- International Conference on Geotechnical Challenges in Megacities . June 7-10 May Moscow Russia.
    - ✓ Preservation of historical buildings in Albania. L Bozo, B Lila
- Third Sympoosium of Macefonia, Association for Geotechnics. June 24-26 Ohrid Macedonia.
    - ✓ Analysis and design of laterally loaded piles. E Bukaci. , L Sharra..
- VIth Hellenics Conference on Geotechnical and Geoenvironmental Engineering. 28-30 September Volos Greece.
    - ✓ Application of the Eurocode -7 in Albania National Rapport. L.Bozo.
- Sixth International Congress in Environmental Geotechnics 8-12 November New Delhi India
    - ✓ Contaminated zone around Fan river and their rehabilitation. L.Bozo, Gj.Ikonomi.
- 20 European Conference of Young Geotechnical Engineers Brno Czech Republic
    - ✓ Problems faced during the design of one bridge abutment. Xh.Alliu
    - ✓ Bearing capacity of shallow foundations . E Goxhaj

CV of the some authors of the scientific work

Prof.Dr.Alfred FRASHERI

- 49 years pedagogical and scientific activity in the Faculty of Geology and Mining of Polytechnic University of Tirana (now part time professor).
- The scientific degree and titles:
  - Msc. 1974
  - Docent 1981
  - Doctor of Science (Ph.D) 1987
  - Professor 1989
  - Invited lecture in Ohrid , Poland, Izmir, Toronto
- Publications.
  - 11 monographies
  - 11 books
  - 48 papers in Albanian Scientific Periodicals
  - 20 papers in International Scientific Periodicals
  - 99 papers presented in International Meeting and Conventions.
  - 53 papers presented in National Meetings
  - 67 scientific –technical rapports.
- Participation in scientific work
  - Geotechnical studies
  - Engineering geophysics
  - Application of mathematical methods in data processing and interpretation of geophysical information.
  - Experimentation and application of new geophysical methods in search for chrome, copper, asbestos ect.
  - Regional geophysical studies.
- Design activity.
  - 22 project in the geophysical field
  - Project leader of GEOTEC sh.p.k.
  - Vice president and director of Canadian-Albanian Joint Ventura "Karma Albania Mining" Ltd.
  - External Senior Geophysical Consultant of QUANTEC IP inc.Toronto.
- Participation of the scientific forums as member of :
  - Commission for scientific training at the Council of Ministers of Republic of Albania.
  - Scientific Council of Polytechnic University of Tirana.
  - Scientific Council of Faculty of Geology and Mining
  - Committee of Science and Technology
  - Scientific Council of University of Tirana.
  - High commission of postgraduate qualification near the council of Ministers of Republic of Albania.
  - Scientific council of Geological Institute of Oil and Gas in Fier.

  - Scientific council Petroleum Branch, Ministry of Industry and Mining.
  - Geophysical Section near the Academy of Sciences of Republic of Albania
- Administrative positions.
  - Director of the Department of Earth Sciences, Faculty of Geology and Mining.
  - Director of Geological-Geophysical Department, Faculty of Geology and Mining.
  - Head of Geophysics sector, Faculty of Geology and Mining.
  - Project leader, Geophysical Team, Tirana Geological Enterprise.
- Actual position.
  - Member of the Albanian Geophysical Society (past chairmen)
  - Member of the executive committee of the Albanian Association of Geoscientists and Engineers (past Chairmen)
  - Honorary member of European Association of Geoscientists and Engineers (EAGE)
  - Honorary member of Balkan Geophysical Society
  - Global member of Society of Exploration Geophysicist USA
  - Member of International Geothermal Society (IGA)
  - Expert of European Databank Sustainable Development (EUDB)
  - Expert of BALWOIS program expert.

Prof.Dr.Luljeta BOZO

- 45 year pedagogical and scientific activity in Civil Engineering Faculty of Polytechnic University of Tirana.
- Scientific degree and title
  - 1981 – Doctor of Science
  - 1984 – Docent
  - 1989 – Doctor of Science in Philosophy
  - 1994 – Professor.
- Publications
  - 3 monographies
  - 25 books for the students and engineers
  - 6 polygraphied lectures (250-300 pages each of them) of the continued education courses for civil engineers.
  - 39 scientific articles in Albania and in the different International Conferences in the world.
- Supervisor (leader) of the :
  - 6 doctorate thesis (dissertations) PhD

- 10 master of science thesis Msc
- Over 100 diploma's student works (when more 50% was research work)

- Participation in scientific work.
  - In three government's themes.
  - Study and design of the engineering measures to stabilized some instability slopes in Albania.
  - Study and exploration of the cases of the enorme deformations of the some important objects in Albania
  - Study for the retaining walls
  - Compilation of the design and construction rules in our construction practice.
- Design activity
  - Some buildings (over 8-10 stories) faculties, schools as constructor.
  - Some infrastructure works (roads, bridges)
  - Some geotechnical retaining structures.
- Participation on the scientific forums
  - Member of the scientific council of Civil Engineering Faculty
  - Member of the scientific council University of Tirana.
  - Member of the scientific council Building Ministry
  - Member of the National High Dams Committee.
  - Member of the organizing scientific committees of some International Conferences.
- Administrative position.
  - Head of Geotechnical Sector
  - Head of Construction Department
  - Vice dean of Civil Engineering Faculty.
  - Member of Senate of Polytechnic University of Tirana.
  - Head of the Professor's Council in Civil Engineering Faculty
  - Head of Didactic Council of Civil Engineering Faculty
  - Director of the Laboratory ALTEA sh.p.k.
- Compiler of teaching plans and programs:
  - Teaching plan of geotechnical profile of civil engineer
  - Teaching plan of structural, transport and manager profiles of Civil Engineers (co complier)
  - Teaching plan of doctor's school of the construction in Civil Engineering Faculty (co complier)
  - Some (over 10) programmers : for the first cycle (two programmer) for the second cycle (8 programmers) and third cycle (6 programs) by Bologna chart.
  - Project for creation of Rock Mechanics laboratory and perfection of Soil Mechanics laboratory.
- President of Albanian Geotechnical Society since 2001 year

As.Prof.Llambro DUNI

- Education.
  - B.S (geophysics) 1978, Tirana University, Albania
  - Ph.D (geophysicis-Engineering Seismology) 1993, Institute of Seismology, Academy of Sciences of Albania.
- Positions held.
  - Professor of Seismology, Tirana University, Dep.of Geophysics (since 2005)
  - Head of the Department of Engineering Seismology, Institute of Seismology of Academy of Sciences of Albania (2001-2004).
  - Head of the Scientific Council, Institute of Seismology of Academy of Sciences of Albania (2001-2004).
  - Responsible for the Albanian Strong Motion Network (since 2002).
  - National Correspondent of the International Association of Seismology and Physics of the Earth's Interior (IASPEI) (since 2002).
  - Individual Member of European Association for Civil Engineering , EAEE (since 2005)
  - Member of the Working Group for the National Plan for Civil Emergencies (2004).
  - Head of the Department of Seismology and Seismic Engineering, Institute of Geosciences of Polytechnic University (since January 2008).
  - Member of the Directive Council of Institute of Geosciences of Polytechic University (January 2008 – May 2009).
  - Member of the Academic Ethical Council of the Polytechnic University (since April 2009).
- Scientific contributions.
  - Analysis of earthquake data
  - Probabilistic seismic hazard assessment.
  - Microzonation studies.
  - Stochastic simulation of strong ground motions.
  - Analysis of strong motion data.
- Participation on national research projects (11 projects)
- Participation on international research projects (10 projects)
- Selected publication in English since 2003 (7 publications)
- Presentations on international scientific conferences since 2003 (22 presentations).
- Works carried out:
  - More than 250 technical reports on site-specific hazard (probalisticc assessment of ground motion parameters evaluation of response spectra, site-response analysis, deaggregation of seismic hazard, scenario earthquakes simulation of spectra compatible motions as well as

liquefaction analysis) for industrial facilities, power plants, various factories, bridges and high rise buildings.

Ph. Dr.Eng.Kristo GOGA

- 45 years activity as designer engineer in the field of the over 400 dams for irrigation intentions, reclamation works and other special water works.
- 25 years design activity in the Institute of Reclamation in Tirana
- Member of the National High Dams Committee.
- Member of the Scientific Council of the Hydrometeorology Institute
- Supervisor for the many projects of the power plants for the monitoring of the dams, for the some important works in the field of the roads infrastructure.
- Part time professor in Civil Engineering Faculty in Tirana and in Prishtina (Kosovo)
- Publications and participation in the scientific Conferences in Albania and in other countries.
  - 6 works in USA, Prague, Macedonia, Kosovo
  - 14 works in Albania
- Member of AGS and Albanian Constructors Society.

Ph.D. Geologist .Ylber MUCEKU

- 23 years activity in the field of Geosciences as specialist , team leader of many projects for urban development, specialist on many geological mapping and geotechnical mapping and many researches.
- Scientific degree:
  - 1996 Msc
  - 2005 PhD
- Publications:
  - 25 papers presented in National and International Conferences
  - 9 oral presentations in National and International Conferences
  - 3 geological mapping – co authors
  - 6 geotechnical mapping – co authors
  - 9 researches studies – author and co-author
- Design activity
  - 25 reports of engineering geology conditions
  - 8 geotechnical investigations

- Participation of the scientific forums
  - Member of Albanian Committee on Large Dams ALBCOLD
- Administrative position:
  - Head of Engineering Geology Section at Tirana Civil Geology, Geological Survey of Albania.
  - Head of the row materials section, Bureli Geological Enterprise Geological Survey of Albania

Ph.D. Eng. Oltion MARKO

- 9 year pedagogical and scientific activity in the Research Instituted of the Forest and in the Civil Engineering Faculty of Polytechnic University of Tirana.
- Scientific degree:
  - 2003 Master of Science
  - 2006 Ph.D
- Publications:
  - 1 monography
  - 2 monography as co author
  - 1 book for students
  - 10 scientific articles in Albania and International Conferences.
  - Supervisor of the 12 diplomas student works
- Participation in scientific works
  - Team for 4 research projects in the environmental field and forest
  - Participant in the 4 national and international projects in the field of forest.
  - Complier of the programes and lessons for first and second cycle by Bologna chart about Erosion and Technology of treatment of residue.

Civil Engineer. MSC Ervin PACI

- 12 years experience in Civil Engineering Faculty as pedagogue and lectures in the field of Tunnels and Infrastructures of Transport
- 14 years experience as designer engineer in more of 200 different objects in CCS-AVE company, IMK company ect.
- Member of Albanian Geotechnical Society and of TC 28 of the ISSMGE
- Many specializations as in France, Denmark, Macedonia, Italy, UK; Turkey, Greece.
- Publication of the some articles in the magazine "Gjeoteknika" and "Ndertuesi"

- Participation in 3 International Conference with papers,

Civil Engineer Kujtim CELA

- 37 years experience as designer and supervisor in many projects as roads, bridges, different objects ect.
- 17 years as designer engineer in Urban Planning District Office Tepelene.
- 7 years as designer own engineer
- 13 year as general manager and legal director of consultation company "Iliriada" PKS Shpk
- Participation in 2 international Conferences (in one with paper0

Publication of 2 articles in magazine BSHT and Gjeoteknika

Geology Engineer Skender ALLKJA

- 31 Years activity in the field of geology
- 15 years experience as geologist in the Geology-geodesy Enterprise, in Railway Construction Enterprise, in the field of the roads and railway.
- 16 years experience as geology engineer in private company ALTEA in all kind of the geological studies (for the roads, slopes, ports, highway, different civil objects, buildings, schools ect), with foreign companies .
- Administrator of the ALTEA laboratory sh.p.k
- Publications and participation in Scientific Conferences in Albania (3) and in other countries (6).
- 650 geo-engineering studies for industrial and residential construction (with local companies).
- Member of AGS.
- Publication in the magazine "Gjeoteknika" 2 articles.

Geology Engineer Luftim AHMETAJ

- 36 years activity in the field of geology
- 20 year experience as geologist in the Geology-Geodesy Enterprise , in Railway Construction Enterprise.

- 16 years experience as geology-engineer in private company Geostudio 2000 in all kind of the geological studies for roads, highway, slopes, ports, different civil objects , factories, water supply, sewerage component etc.
- Chief engineers of Geostudio 2000 Shpk
- 950 geo-engineering studies for industrial and residential construction with local companies.
- Publications and participation in the scientific Conference National and International (8)
- Member of Albanian Geotechnical Society.
- Publication in the magazine "Gjeoteknika" 2 articles.

Civil Engineer Erion BUKACI

- 5 years experience in the following field:
  - Assistant pedagogue in Civil Engineering Faculty in Tirana.
  - Structural engineer on construction office in Tirana (owner) for the 17 different objects.
- Member of Albanian Geotechnical Society and TC 18 "Deep foundations" of ISSMGE.
- Participation in two International Conferences with papers.
- Publication in the magazine Gjeoteknika of one article.

Civil Engineer Andrea THEMELI

- 4 years experience in the following fields:
  - Assistant pedagogue (part time) in Civil Engineering Faculty in Tirana.
  - Employer in ALBEGIS and Co consulting Company as designer and supervisor in 14 different objects.
- Participant in one International Conference with paper.
- Publication of 2 articles in the magazine Gjeoteknika.

Civil Engineer and 2nd level pos University Master Xhevahir ALLIU

- 12 years experience in the following fields:
  - As construction engineer in the section City Planning Laç 2 years
  - As designer engineer in the ILIRIADA PKS sh.p.k 3 year.

- As private designer.
- As assistant supervisor in DIWI Consult International GmH 1 year.
- As designer engineer in Klodiada, Bermuda P.S sh.p.k Companies 2 years.
- As assistant pedagogue in Civil Engineering Faculty in Tirana (part time)

- Publication of one article in magazine Gjeoteknika
- Participation with papers in two International Conferences
- Member of Albanian Geotechnical Society.

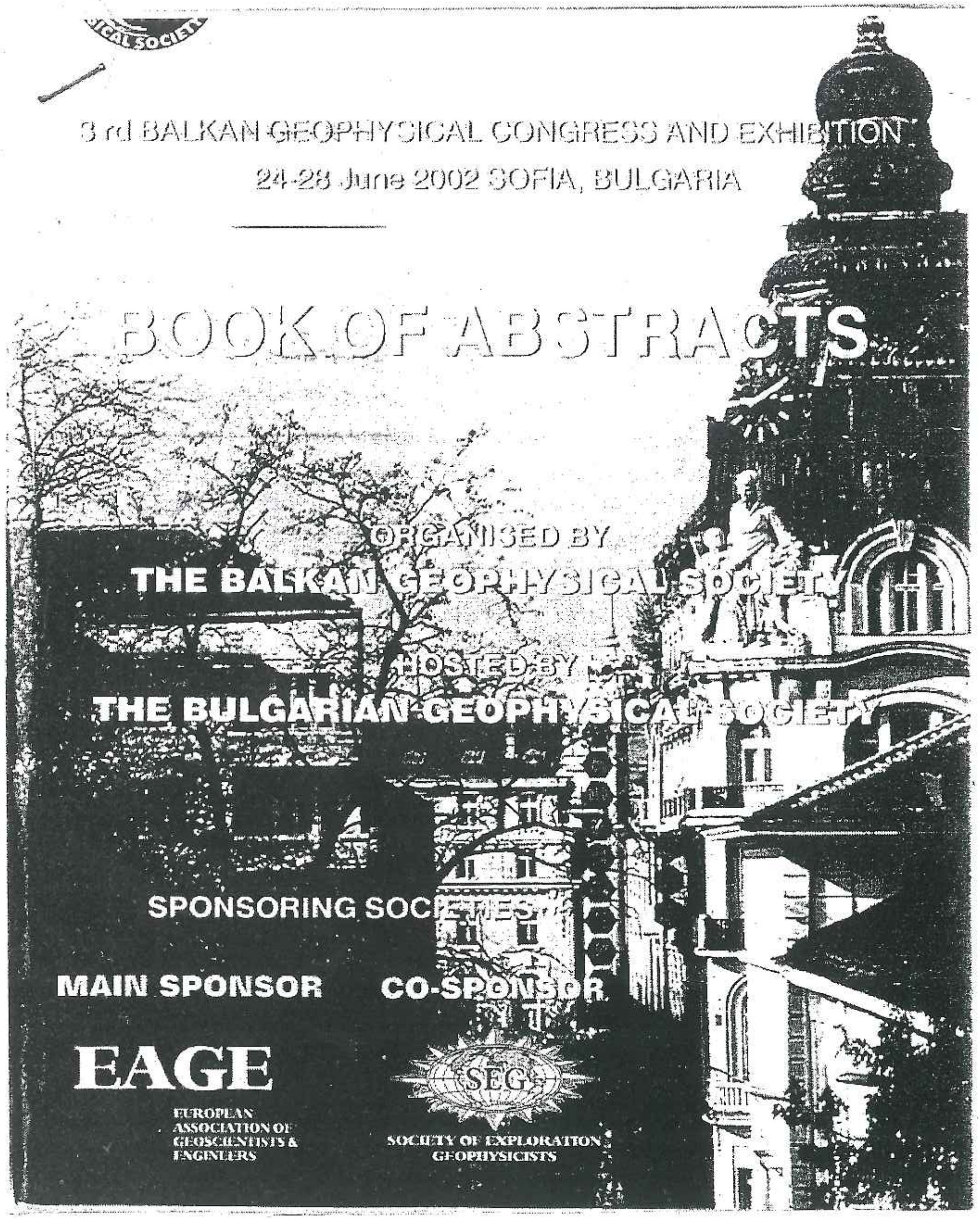
Civil Engineer Lorena HARIZAJ

- 4 years experience in the following fields:
  - Assistant pedagogue (part time) in Civil Engineering Faculty in Tirana.
  - Assistant quality manager and laboratory engineer in the laboratory ALTEA and GEOSTUDIO 2000 sh.p.k.
- Publication in the magazine Gjeoteknika of one article.
- Participation in two International Conferences with papers
- Member of Albanian Geotechnical Society

YEAR 2002

- **3rd Balkan Geophysical Congress and Exhibition**
  June 24-28 Sofia, Bulgaria

- **First International Conference on Scour of Foundations**
  November 17-20, Texas A&M University College Station, USA



Session 4: Engineering Geophysics.

Land Subsidence in Bulqiza Area, Albania

Y.Muceku[1]; L.Bozo[2]; and C.Durmishi[3]

[1]Center of Tirana Civil Geology, Department of Engineering Geology, Albania
[2]Geotechnics Section, Civil Faculty, Tirana Polytechnics University, Albania
[3]Geological Survey of Albania

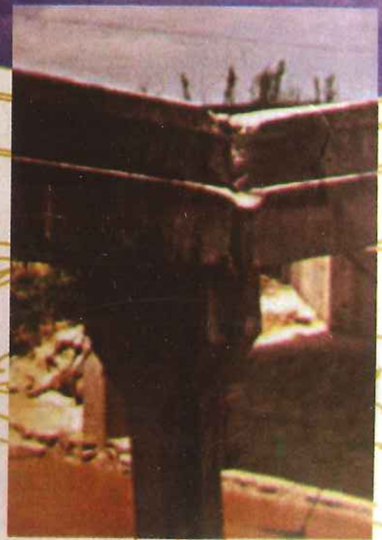
In this article are treated the results of the engineering geological studies carried out last years concerning land subsidence occurred in the urban area of Bulqiza town, Albania. It's located in eastern part of Albania and represents an intermountainous valley, like with amphitheatre shape (3 km long and 1,5km wide). This valley built up by ultrabasics rocks (Middle-Upper Jurassic) on which are situated the terrigenours deposits (Quaternary), that have a thickness up to 180m. They consist of alluvial gravels, sands and clays, as well, as swamp peat and glacial breccias of ultrabasics rocks. As results of opened mine under these deposit, a land subsidence from 2,5 up to 2,8m, due to water drainage is occurred last of 5 years. So, in northern part are created many cracks (0,5km long and 0,3up to 0,7m wide), whereas in central part of the studied area is likely created a small graben. From this phenomenon are demolished many engineering objects as buildings, roads and bridges etc. also, in the article it is given engineering geological condition, conclusion and recommendations as well.
In this area were severely damaged by land subsidence. Some of damages were destruction of many buildings, cracking of about 4 square kilometers and damaging of 2 kilometers of road as buckling and cracking.

First International Conference on

Scour of Foundations

November 17-20, 2002

Texas A&M University,
College Station, Texas, USA



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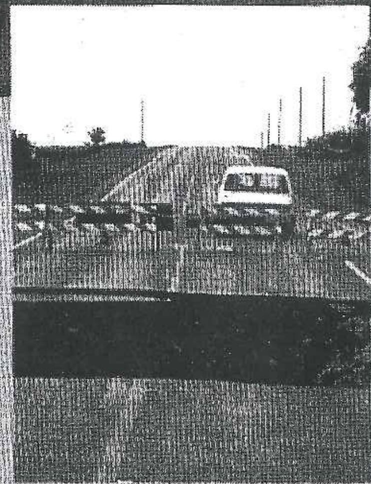

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Damages of the roads-bridges by erosion and remedial measures in Albania

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Abstract

In this paper are shortly described the results of the geotechnics studies carried out last years concerning damages of roads-bridges by river erosion in Albania. The river erosion phenomenon nowadays in our country is become very dangerous, according to:

- The destroying of many engineering objects (roads, bridges) constructed on river banks.
- The loss of many square kilometers of agriculture area.
- The environment damage.

Here we have presented some particular cases of 2 roads and 3 bridges destroyed by above mentioned phenomenon, their remedial measures and causes of this phenomenon, as well.

Introduction

Albania is mountainous-hilly terrain, where 75% of whole area covered by mountains - hills and 25% by fields. There are many rivers and streams, which are very vehement about on 3/4 of their lengths. The erosion phenomenon by rivers water, currently in Albania is present. It's much favored by high amounts o precipitations, which are 2000-3000 mm/year (in some regions they are 100 mm/hour and 300 mm/24 hour) and by man-made activity, especially in plain area, as well as from climatic, seismic and geologic factors. The study of this phenomenon and the hazard evaluation for the protection and rational-complex exploitation of the environment is an important direction of geotecnics-engineering geology. It could also help for solving the problems of design and construction of different objects and their protections. On the basis of the study, some theoretical and practical remarks are obtained.

Result and Discussion

Peshkatari bridge

Site characteristics

The bridge is located along Tirana-Elbasani national highway, over Erzeni River linking together capital Tirana with southern Albania (Elbasan-Librazhd- Korca-Pogradec towns), as well as corridor Nr-8 connecting Albania with Macedonia and Greece.

The site where the bridge was built comprises Quaternary loose deposits and Tortonian molasses. The Quaternary loose deposits makes up the first terrace of Erzeni River, as well as its bed. They are represented by middle to coarse grain gravels, 1/2 re-worked out made up mainly limestone and less sandstone filled by sands, silts and brown clays. They are not completely compressed and are water saturated. The thickness varies from 1.0m to 2.0m in the first terrace up to 6.0 – 7.0m in the river bed. The above rocks overlay Tortonian sandstone's of molasses, which built up also the Erzeni River's banks. The sandstone's rocks are massive, constituted mainly by quartz with scarce siltstone and argillite thin layers intercalation

The sandstone rocks are coarse grains with a thickness up to 1000m.

According to geomorphology the area is almost flat (first terrace) with westwards very gentle slopes bounded to the north with Mulleti's hills with an altitude 280m to 290m above sea level, and southwards of Petrela hills 500-600m above sea level.

The hills are undulated with (watersheds and valleys), which go downward towards the lowlands. The field is crossed by Erzeni River which have created an U-shaped valley 40-100m wide and 7.0 - 10m deep (Fig. 1), as well as, a lot of meanders along its East-West flow.

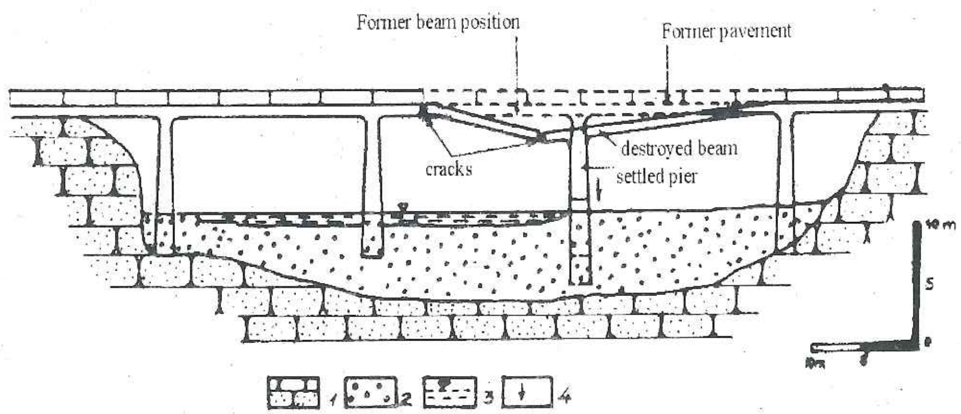


Fig. 1 Cross section of Peshkatari bridge

1. Sandstones, 2. Alluvial deposits, gravels, sands, silts and clays, 3. River water level, 4. Sinking directio

Bridge structure

The scheme of bridge comprises continuous beams with piers and massive shoulders, with shallow foundations supported in gravels (Fig. 2) up to 3,5m below gravel surface. The bridge was built during 1935-1937 (c. 65 years old) and it is a two ways highway.

Rivers activity

The approximate surface of drainage basin, where is located the bridge is 292 km^2. During the most rainfall season (October-May) the average discharge is about 118m^3 /sec and maximal one, up to 170 m^3/sec, whereas the mean discharge/year is 8.6 m^3/sec. The suspended load is 68.4 kg/sec and 2150 · 10^3ton/year, as well as, the bed loads 430 ·10^3 ton/year

Table 1.

Grain size mm	0,01	0,05	0,1	0,2	0,5	1,0	2,0	5,0	10	20	50	100
Suspended load (%)	65	89	97	99	99,5	100	-	-	-	-	-	-
Bed load (%)	17,5	39	60	78,5	82,5	85	88	90	92	96,5	99,5	100

Suspended and bed loads (%) moved by river waters is indicated at the Table Nr. 1. Both the velocity of the river's flowing, which is generally around 0.7m/sec and suspended-bed loads (gravel filled by sands, silts and clays with a relative density from 50% up to 70%) has caused an average erosion of its bed about 1m, it does mean that the river erodes its bed about 1,5 cm/year. The bridge foundation basement consist of gravels with a bearing capacity from 350 KPa up to 400 KPa, that means it stands very good the loads, that comes from construction and river water pressure.

Man-made activity

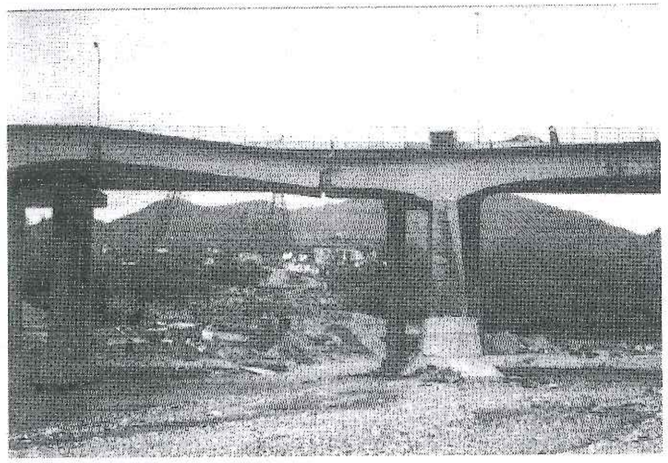
After 1990 year, when the country has passed from a centralized economy (totalitarian regime) to free and open economy, nobody has taken care from the environment protection for a long time. In these conditions, when the due legislation lacked, Albania has suffered so much from ill treatment of environment, giving rise hazard besides the natural ones.

So, in constructions field especially for capital of Tirana, the great needs for the first inert materials as sands and gravels led to intensive exploitation along of Erzeni riverbed. In some parts of river bed, even near by bridge foundation, the bedrock is outcropping lowering 3m up to 5m the base level of the river bed. In this case is artificially increased erosions activity of rivers water from 1,5 cm/year (normal condition) up to 8-10 cm/year.

Damages and causes

From a period of 60 year old of erosions activity, on 1990 year the foundation depth was 2,5m below gravel surface. Whereas, the last ten years the erosion caused by

man-made activity discover about 1.0 m of bridge foundation. So, during a rainfall storm (1999 year) the river water discharges (great flowing) was very much increased and in consequence a intensive erosion on river bed occurred, which caused a sinking from 1,98m one of bridge pier (foundation) (Fig. 1,). Also from above phenomenon, is break down the central part of bridge and it is bent, as well as along of it are created many cracks enabled the destruction of it and put out of use (Fig. 2).



a) ***Peshkatari bridge***

b) It's looking to east, and shows

Fig.2 Peshkatari bridge. It's looking to south, and shows, the new bridge and the destroyed bridge (right side) that is sunken 1.98 m compared with the new bridge.

Fast remedial measures for provisional use
For a quick connection of this very important highway for Albanian economy, were taken as following engineering measures:

- Grouting with pile against movements.other
- Covering with concrete for reinforcement of damage foundation.
- Covering with stores blacks (gabion).
- Stopping of exploitation of raw material on river bed.
- Construction of provisional metallic bridge over existent one.

Remedial works for highways normal function
- A new bridge with static scheme -system beam column and three ways of the highway is constructed last year close by older one (east side). It has deep foundation (6-7m) with molded piles with 1m diameters and situated in sandstone's rocks. Also, for a normal function it built by piers with concrete column.

Mifoli bridge

Site characteristics
The bridge (older one) links capital of Tirana and several towns of northern and central parts with southwestern part of Albania (Vlora, Saranda and Gjirokastra towns). It's constructed over Vjosa River very close its delta. Alluvium Quaternary loose deposits build up the studied area as silts with thin fine sands layer intercalation (40m thick) and gravels, which are covered by first one.

Bridge structure
It has a combined structure, concrete arch with metallic beams. Metallic piles make the foundations in tubular shape (fig. 3).

River activity
The drainage basin surface of Vjosa River, over which is constructed Mifoli bridge is 6680 Km^2 and its mean altitude above sea level is 858m. The suspended and bed load of this river is 190kg/sec and $5.99 \cdot 10^6$/year.

Damages and causes
The interaction of seismic vibrations (sands liquefaction) and rivers erosion activity phenomena enabled discovering of bridge foundation (earthquakes of 1967 and 1969

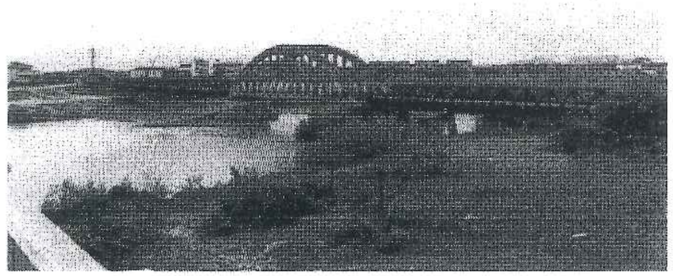


Fig. 3. Photo from Mifoli bridge.

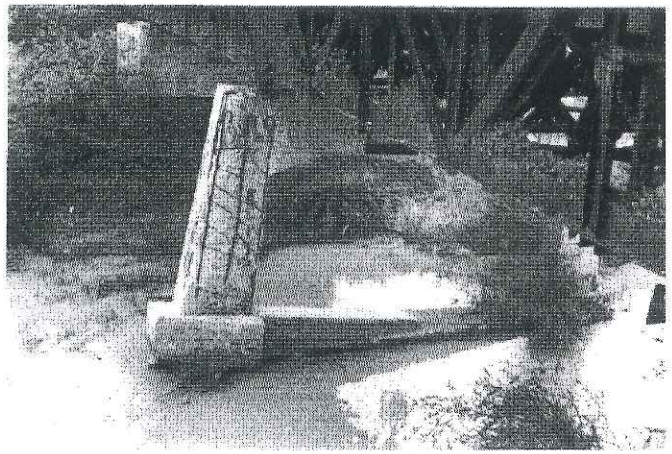


Fig.4 Mifoli bridge.It's looking to north and shows the siking of its pier

years), from which a bridge sinking occurred (Photo NR. 4), and simultaneously a horizontal movement on one of its shoulder is done.
Consequences: It is out of use.

Remedial works for highways normal function
Construction of new bridge for the highway and railway purpose with molded pile foundation (38-42m length and 0,8-2m diameter) supported in gravel deposits.

Kallmeti's bridge

Site characteristics

It's located on Lezha-Shkodra road (old), over one of Drini river branch (Gjon Zefi Stream).

Concerning the geology in the studied area are recognized two major groups of rocks (Fig. 5):

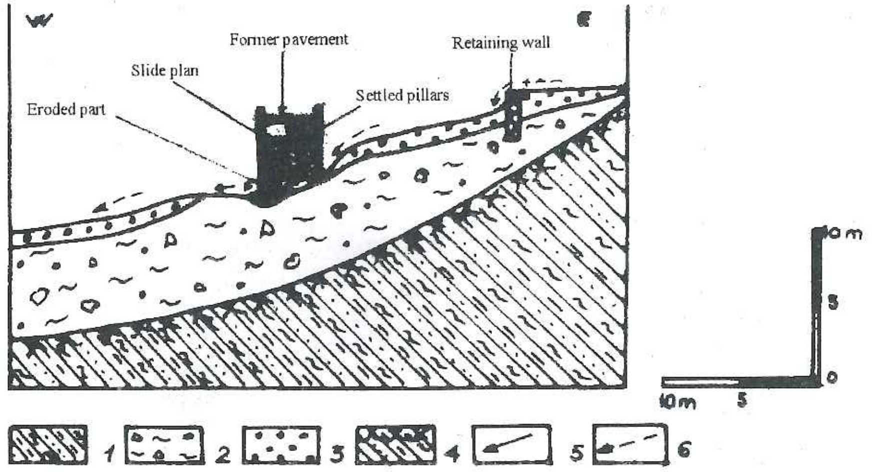


Fig. 5 Cross section of Kallmeti bridge

1. Flysch, 2. Alluvial deposits, clays, sands, gravels, and cobbles, 3. Alluvial deposits, gravels and cobbles, 4. Weathering crust of flysch rocks, 5. eroded soil movement direction, 6. Stream water flow direction

- Quaternary loose deposits comprise proluvium-first terrace and beds deposits of Gjon Zefi stream. The first one consist of silts with 30-35% with the limestone's and sandstones rubble-stones content, that have dimension ranging from 0,05m up to 0,5m. On West Bridge they have e fan shape (10-15m thick), whereas on east they lying along stream valley (1-2 m thick). The beds deposits consist of gravels and less stones filled by sands, silts and clays. They have a thickness varies from 0,5 up to 1,5m.
- Oligocene flysch deposits comprises by thin intercalated layers of claystones, siltstones and sandstones, which have a dip angle 40-45^0 on east direction. They overlain by proluvium deposits.

According to morphology the Gjon Zefi stream valley have a 'V' Shape, with steep banks slope and 4-12m wide and 5-6m deep.

Bridge structure

It's a bridge with cement stone masonry pillars (length L=8m) and concrete armored beam. This bridge is about 70 years old (Fig.6, 7).

Damages and causes

Several factors took part in damaged of bridge, which are as following:

- During 1992,1994,1996 and 1998 year, as results of high quantities of precipitation and without protective measures getting in due time, on stream bed occurred a intensive erosion like a gully, which was extremely rapid. The stream bed erosion from 1992 year up to nowadays is about 1.8m, and below of bridge foundations was created a cavity with 1.4 diameter, as result of it, the bridge foundations has moved downwards
- The dynamic loads actions induced by motorcar traffic, when in the bridge were appeared a lot of cracks.

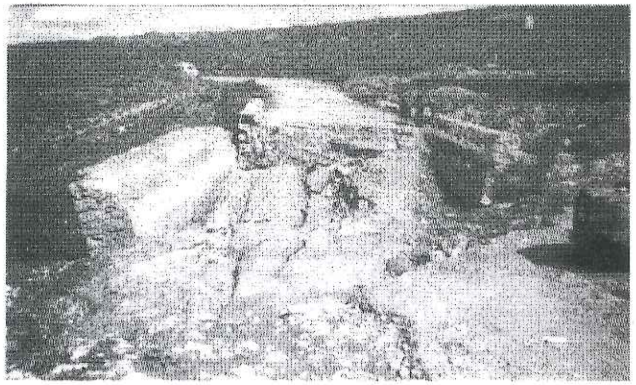


Fig. 6 Kallmeti bridge. It's looking to north, and shows, the sinking of its pillars and pavement

Fig.7. Kallmeti bridge. It shows cracks of pillars from erosion phenomenon

Consequences

The bridge foundation moved 1,4m downward and simultaneously it is associated a horizontal movement, where several failures occurred on bridge pillars walls.

Recommendation for remedial measures

- Construction of weirs and groynes.
- Reinforcement of bridge foundation by its grouting.
- Filling of cavity by durable and less deformation materials.
- Construction a new bridge.

Bogova landslide

It is located on eastern part of Albania along Berati-Corovoda road, on right side of Osumi River valley. On this landslide is constructed above-mentioned road, which is affected by this phenomenon. The river valley slope, where the landslide occurred built up by flysch rocks (much weathered) with dip angle $50\text{-}60^0$ in the same direction with the valley slope inclination, favoring it. Also on bottom of valley slope, flow the Osumi River waters, which continuously contribute to the erosion process, that is one of main factor of landslides creation. The landslide is moving slowly downwards through the valley slope to river bed (fig . 8), together with 200 m of road constructed on its body.

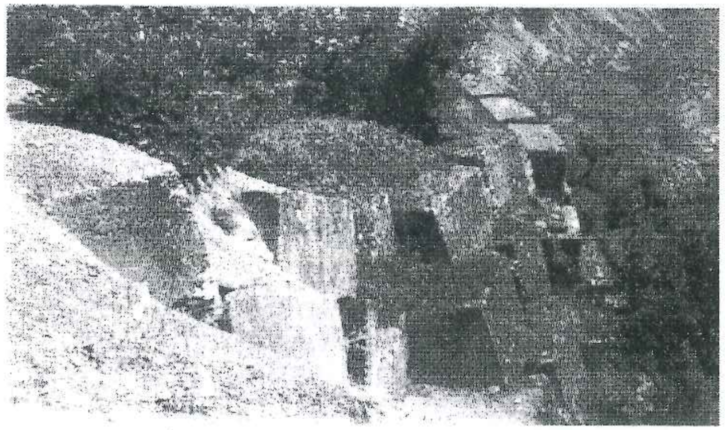


Fig. 8 Bogova landslide. It's looking to east, and shows, a destroyed retaining wall (deformed) by mass movement

A lot of factors influenced to mass movement on the studied area, are as following:

- The velocity of river water flow, which is from 0.4m/sec (summer) up to 1.29 m/sec (winter) and its discharge 49.7m3 /sec together with its suspends and bed loads that are 17.4 kg/sec, operate on the outer bank, causing erosion of it.

- Weathered flysch rocks build up the riverbanks.
- There is a poorly drained system. One drained ditch (constructed 20 year ago) for collected of surface water coming from rainfall or other sources was buried. So, much water running throughout, along and cross of the road, as well as down the bank slope of Osumi River. The other water quantities infiltrate and seep in small cracks of weathered flysch rocks. From high value of hydraulic gradient are created many new cracks. The natural gradient value exceeds the critic gradient value. Therefore, in this case we have the mechanics suffusion development, where small cracks are become wider and deeper, which have helped landslide development.
- The cutting of many trees on this are during last 10 years is an other factor.
- In the studied area, there are many water springs, and high rain falls, that favored degradation of bank slope of Osumi River.

Remedial works

For improving of general stability conditions of Bogova landslide have to make engineering measures:

- Construction of weir and groynes.
- Improving of general stability condition by installing a drainage system (2 ditches) to collect surface runoff, which reduce seepage and infiltration in the soil and this exert a positive influence on the main factor of slope stability.
- The stability of landslide by molded piles.
- Planting of trees.
- Construction of retaining wall with drainage system.
- The slope protection by using of gabions or geotextile against surface erosion.
- Protection interior erosion (systematize of two streams close of landslide.

Shkopeti landslide

Site characteristics

It's located on northeastern part of Albania along national road linking the Miloti and Burreli towns. This road is constructed on banks of Mati River valley and it is affected by landslide phenomenon. The studied area represents a narrow and deep valley (Mati River) formed a "V" shape have a slope inclination from $25\text{-}35^0$ up to $60\text{-}80^0$ and it is built up by loose Quaternary deposits (deluvium and alluvium) and by melange deposits (Fig. 9),. Deluvium consist of rubbles up to stones filled by silts. They have a thickness from 1-2m (upper parts of valley slope) up to 10-12m (down valley slopes). These deposits are situated unconformity over melange deposits. Alluvium deposits are situated along Mati River bed and consist of sands ands gravels, whereas the melange deposits are composed bay rubbles-stones and pebbles cemented by argillaceous material. They on surface are weathered 0,5m up to 2-3m.

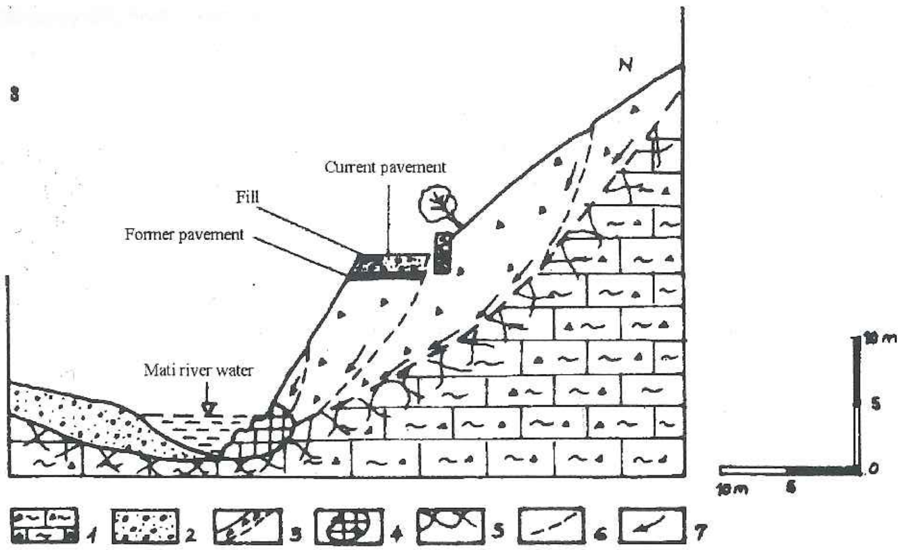


Fig. 9 Cross section of Shkopeti landslide

1. Melange deposits, 2. Alluvial deposits, sands, gravels, 3. Landslide body, 4. Erosional part, 5.Weathering crust of mélange deposits, 6. Slide plain, 7. Slide direction

Fig. 10. Shkopeti landslide. It's looking to north-east, and shows, the movement of Miloti-Burreli road filled by gravels and landslide body

Analysis of phenomenon.

As result of lateral erosion on right side of lower part of Mati River bank slope on which is constructed the road Milot-Burreli, a landslide has occurred. It is 250-300 long, 300-350 wide and 5-10m deep.

One of main factors in the active of landslides is the impact of changing ground water conditions as result of rainfall. By investigation carried out in the area, results that, during high rainfall periods, the river water discharge are much increased, which associated with intensive of bank erosion where the landslide re-initiating going downwards (after 1990 it's moved downward 2.0m) fig.9

Landslide has occurred on the boundary between of the loose Quaternary deposits (deluvions) and the melange deposits. As indicated in Fig.8, the landslide is developed on a steep slope and it is moving on steep slide plain. For that reason favored and from high quantities of rainfall, which infiltrate in slide plain and from bank toe erosion of river water, it begin slowly moving downwards up to reaching of a stationary equilibrium but with possibility to be reactivated from the changing of the environment conditions.

Remedial Measures

The stability of landslide can be done in 2 ways:

a. Construction of concrete wall, which have to support on melange rocks.
b. If the first one is impossible to done, then for stability of landslide needs to construct:

- Two ditches for collect surface runoff, just there where it seeps and infiltrates in slide plain.
- Construction a new retaining wall with drainage system on upper part of road.
- The protection of the road lower part of the bank slope by using of gabion or geotextile against surface erosion.

Conclusion

- The damages of the roads and bridges due to rivers erosion activity are a present and continual phenomenon in Albania.
- Analysis of erosion phenomenon is complex because of in damages of engineering objects from erosion of rivers waters have taken place a lot of factors.
- Engineering measures for stability of situation, created by the above phenomenon are complex, as well. Generally, the solution of this problem can be done from a specialist's team.

REFERENCES

Bozo, L., Technics of foundation (part I) 1987, Faculty of Civil Engineering, Polytechnic University of Tirana, 413 pp., (in Albanian).

Bozo, L., Technics of foundation (part II) 1987, Faculty of Civil Engineering, Polytechnic University of Tirana, 172 pp., (in Albanian).

Bozo, L., Geotechnics, 1995, Faculty of Civil Engineering, Polytechnic University of Tirana (Pp. 261) (in Albanian).

Bozo, L., Behaviour of basement and foundation under seismic actions, 1988 Thesis of Ph. D., Polytechnic University of Tirana, 226 pp., (in Albanian).

Muceku, Y., Slope movement and road risk avaluation in Shkopeti valley, Albania. 8th Congress of Albanian Geosciences 2000, abstracts book, p.213.

Muceku, Y., Engineering geological mapping for urban planning and regional development in Tirana region. Geological Survey of Albania, 2001, 38 pp. (in Albanian).

Saraçi, R., Suspended and beds load of Albania River and water erosion problems, 1990. Water as a national asset. The research and management of water resources of Albania. National Scientific Conference, Tirana 1 – 2 October, p.70-75, (in Albanian).

YEAR 2003

The International Conference of ENHR. May 26-28 Tirana, Albania

XIIIth European Conference on Soil Mechanics and Geotechnical Engineering. August 25-28 Prague. Czech Republic.

MAKING CITIES WORK

TIRANA - ALBANIA, 26-28 MAY 2003

Comparing between "TRANSITIONAL" and "DEVELOPED" Urban and Housing Models

18 Mars 2003

zj. **Luljeta BOZO**
Fakulteti i Inxhinierise se Ndertimit
Tirane

Nr. Prot. 29/03

FTESE PER PJESEMARRJE

E nderuar zj. Bozo,

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MAKING CITIES WORK

TIRANA - ALBANIA, 26-28 MAY 2003

Comparing between "TRANSITIONAL" and "DEVELOPED" Urban and Housing Models

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The International Conference of ENHR
The European Network of Housing Research.
Tirana, Albania 26-28 May 2003

Geotechnical Engineering and Urban Development

Prof.Dr.Luljeta BOZO

Abstract.

Theoretical conception for housing and urban development is very important problem. But its practical realization in fasten expect economical, social problems and with environmental geological, geotechnical problems and earthquakes hazards.
Urban planning is fasten with natural hazards, which have their origin in soil behavior under static and dynamic loading. In this paper we would like to present some problems which binds the urban planning with geotechnical phenomena and soil behavior.

Introduction

In 13 last years in Albania the development of dwelling centers have had haight rhythm, but this process is somewhat not well planed and it is realized without one accurate urban conceptions and perspective.
Housing and urban development have accomplish a partial projects, and now we have serious problems, sometimes we have grave situation for inhabitants, for economic development and for state.
It is time to undertaking hasty and serious steps to make possible to allow this "anarchist" and without criterion process of housing and urban development.

Connection between urban development and geotechnical engineering

Urban planning is fasten with natural hazards, and urban development often depend from soil behavior of land. In this paper we think to present some problems, which to be not considered in last times, and they are with great risk for our economy and for human life.

- Albanian is with high seismicity activity country. Our seismological centre has accomplished a earthquakes studies from all big cities of Albanian. In order

that, earthquake to cause minimum damage. This study must to be land mark for determination of dwelling centers, for height of new buildings in them for dense of houses etc.

These problems are present in Durres, Vlore, Shkodrer cities, where earthquakes movements in many arsis have had disastrous effects. But and other development cities how Fier, Korca, Pogradeci have seismic hazard. In Tirana city we have vigorous development of many storey buildings without one urban plan of development of city. Seismic risk in Tirana city is great because:

- The geological studies are not required scientific level and quality.
- Our experience in design of many storey buildings is little.
- Many storey buildings not yet passing seismic shake.

- In our country we have some great tectonic faults. Near them occurs strong earthquake. The planning of housing and urban development near tectonic faults must to be with great attention and carefully, because these zones are dangerous.
- One part of our sea-side (littoral) I where will have great development of tourist, consist from weak soft and saturated sand deposits .They have ability to liquefied under vibrations (earthquakes) to cause damage or destruction of buildings, drain, pipes, harbor construction ect. Liquefaction induced damage, including slope failures, bridge and building foundation failures, and flotation of buried structures. For this reason the planning of urban development of these zones depends also from liquefaction of their soils. The design process of urban development in liquefied zones must realize in collaboration with geotechnical engineers. For realization of design process is required profound knowledge in geology, geotechnical and seismically study. For application of the regulation in this field absence at the mean date for ground, take the designing work berry difficult. So that it's don't clear the perspective for the investment in this zones.
- One part of the Adriatic lowland consist from swamp deposits which have how bearing capacity and high deformation feature. The planning of urban development of this zone for touristic intentions is tied with soil's behavior. Otherwise the consequences for buildings in these lands will be disastrous.
- Construction of roads and infrastructure of city or dwelling centers is field with such phenomena how enormous deformations and settlements, sliding, slope instability the earth subsides ect. In Albanian all this phenomena they are present. For this reason during the design process of road, water pipes system, sewerage system all this dangerous phenomena must to take into consideration to guarantee they normal functioning.
- The planning of urban development is tied with environmental protection wich have connection with geotechnics in means directions:

- Establishment of big sweepings deposits for different cities is fasten with drainage conditions of lands, soils characteristics, slopes of deposits.
- Establishment of cleaner equipment for potable water and sewage water is fastening with land characteristics and soil features.
- Construction of new housing centers near the tailing dams sometimes is very dangerous problem because the failing material can to be very rich with sulfate (SO_3SO_4). In other hand during the period of 13-15 years these dams are abandoned and dangerous erosion phenomena are developing. So that have grave consequences for environmental protection and this problem is at the same time urban, social and geotechnical.
- Impetuous building of Tirana, Durres, Fier ect, have create problems for constructions refuse so, all excavation material or demonization material are transported in peripheral zones near rivers, sea or laces to damaging environment and to provoking slide of natural slope instability ect.

Conclusions:

- The planning of housing and urban zones must to entry in normal right and legal way.
- Albanian have qualified specialists for realization of study of urban development of all housing and dwelling centers in our country.
- These studies where priority have the architects, must to realize an collaboration with civil engineers, geotechnical engineers, geologist, seismology ect.
- The urban development of dwelling centers is tied with environment, land or soil behavior and all phenomena fasten with geosciences.

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The problems related to the construction and exploitation of the tailings dams in Albania

Les problems que sont relie avec le construction et l`exploitation dela remblais pour le sterile industriele en Albanie

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KEYWORDS: European Conference, instructions , CD-ROM, camera-ready, copies

Abstract

In Albania has existed a rather developed industry for the extraction and processing of minerals. For the storage of the materials extracted from the processing of the minerals and the industrial residues, during the years 1970-1990 have been built many dams. In this material are treated the specific problems that have emerged during their design and construction concerning the territory (location), the type of the material to be stored and the hydrology of the zone. Are treated the negative consequences of some of the dams in relation to the surrounding environment as well as the measures to be taken for the rehabilitation and the protection of the environment from the pollution.

Introducion

Before the years '90, during the period of the development of the mineral and processing industry, for increasing the quality of the minerals has been constructed a rather large of factories quantity for the enrichment of minerals. Almost all the mines in Albania have had also processing factories and these have been accompanied with the dams for retaining and the storage of the tailings. More present this types of construction have been in the proximity of copper, chrome, iron-nickel mines and in a lesser degree in the other mines (fig.1 shows the distribution of the dams in Albania).
The problems of the design, construction and exploitation of the tailings dams in Albania have been solved according to the concrete conditions of the territory, hydrologic conditions, the storage capacity, the works for the conservation, etc.

The dams have been constructed mainly in the mountainous zones, on mountainous narrow gorges providing for the protection against the external waters as well as the measures for the conservation and maintenance, to avoid the pollution of the environment.
Following the year 1991 the mines in Albania have been closed and consequently for the time being do not work neither the enriching factories nor the tailings dams. The abandonment of the dams has created dangerous consequences for the surrounding environment either in respect of the pollution or in respect of it's damage.
In this material will be given an overall panorama of the Albanian experience concerning the design, construction and exploitation of the tailings dams.

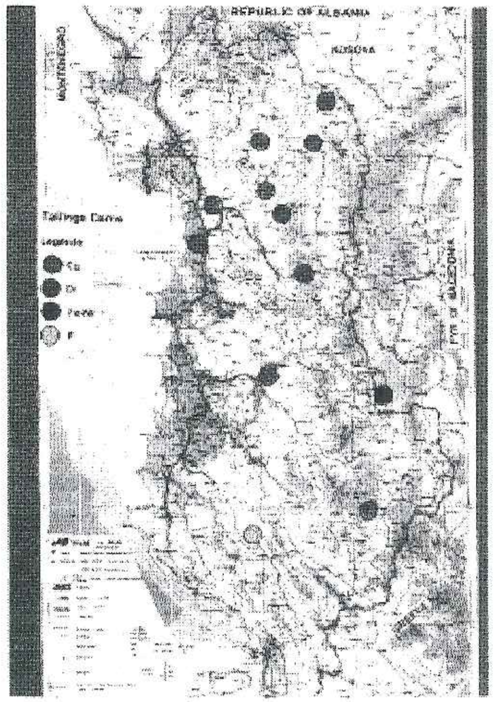

Figure 1. Tailings dams in Albania

The classification of the tailings dams in Albania

Classification according to the stored material (tailing).
Have been constructed mainly the following dams:

- Dams for the tailings produced by the processing of the chrome material with high content of Fe-Cr, Fe-Ni

This types of dams have been constructed in Bulqize, Elbasan, Has and Pogradec.
The characteristics of the stored material are the following:

γ= 400-1200 kg/m^3

d = 0,6-20 mm

and the chemical composition is the following:

Table 1.Chemical composition

Elements	SiO_2	FeO	Al_2O_3	MgO	Cr_2O_3	MgO/SiO_2	MgO/Al_2O_3
%	36.206	1.34	11.63	44.31	4.07	1.22	3.745

The high quality of Mgo limits the fields of utilization in their natural state as binging material. The degree of the activity of the tailing is low, with module

$$\frac{Al_2O_3}{SiO_2} = 0{,}34$$

and it's acid nature has module

$$CaO + \frac{MgO}{SiO_2} + Al_2O_3 = 0{,}9$$

- Dams for the tailings resulting from the processing of copper mineral as well copper containing sulfurous mineral of high content of sulfur.

These have been built in Mirdite, Vithkuq, Kalimash. The stored material has the following characteristics:

γ= 800-1400 kg/m^3
d = 0,1-1,5 mm

and the chemical composition is as follows:

Table 2.Chemical composition

Elements	Cu	SiO_2	FeO	CO	Zn	Al_2O_3	CaO	MgO	S
%	0.25	34.4	30.15	0.035	0.65	6.65	11.22	3.3	0.73

As they have a sufficient plasticity $I_p \cong 7$, can be used for replacing part of the inert used in the mortars for plastering as well as raw material for the construction of the road layers.

- Dams for tailings of other minerals as for phosphoresces, namely in Memaliaj, and the ashes in Elbasan, etc.

Classification according to the height.
In Albania have been constructed tailings dams having a height of 20-50 m, which according to the national classification of the dams, can be grouped in conformity with table 3:

Table 3.Classification of dams in Albania

Class 1	Height pf dams in m	Volum of rezervoir in 10^6 m^3
1	>40	>10
2	30-40	5-10
3	20-30	1-5
4	<20	≤1

In general the volume of the reservoir has been relatively small because of the site where they have been built (extremely mountainous territory) as well as because of productivity.

Design principles

Requirements to be fulfilled by the dam.
During the design of the dams in Albania we have been based on the following requirements to be fulfilled, namely:

- Storage for the tailings of the processing industry
- Sufficient stability for the static and seismic loads
- Source of row materials for the industry of construction materials (tailings) as well as for the metallurgical materials
- Protection of the environment against pollution

Design procedures

For fulfilling the first requirement we have taken into account the territory where the dam should be located and constructed as well as the proximity to the processing factory of the mineral.
Because of the fact that in general the territory where the dams have been constructed is a rugged mountainous territory and near the torrential watercourses sometimes we have been obliged to divert the watercourse aiming at avoiding it's negative impact on the stability of the dam and to avoid the flowing of the water into the settling basin. On the other hand, because the territory is very rugged and our aim has been also to avoid the damage of the agricultural lands whose shortage is significant in our country and consequently we need to save them, we have been obliged to design the dams in sites of high altitudes so creating a relatively small reservoir.

- The principle of decantation

Aiming at the protection of the environment as well as the decrease of the horizontal load on the body of the dam itself, have been designed the necessary measures connected with the principle of decantation. On the basis of the size and chemical distribution of tailing and the requirements concerning the quality of the water coming out from the dam after the process of decantation, has been calculated the necessary length of the settlement by means of the following formula:

$L = \alpha\ H\ Vm\ /\ \omega$ where,

α - Reserve coefficient, taken 1,2-1,5

H - Depth of the water in m

Vm – average velocity of water movement in m/s

ω - Hydraulic quantity of the decantation of the grains, depending on the grain size, temperature, etc.

- Protection against the external waters

This problem has been treated in a special way especially when the dams were built in valleys of mountainous watercourses. Usually we have solved this problem by means of diverting the watercourse in such a way that it's waters could not pass through the settling basin of the pulp of the tailings. These structures, in conformity with the class of the dam have been treated for a probability of occurrence not less than once in 100 years.

- Removal of clean waters

Usually this has been done on the opposite side of the flow of the tailings, which as a rule spilled on the side of the dam. (Fig 2.)

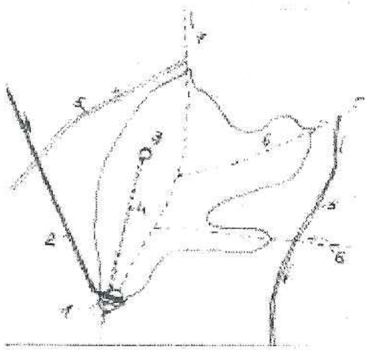

Figure 2. Plan of dam

1. ***Tailings dam***
2. ***The pipe conveying the calf with tailings***
3. ***The spillway of the clean waters.***
4. ***Collector of the removal of the clean waters.***
5. ***Diversion of the external flows.***
6. ***External flows***
7. ***External flows***

- Security of the stability of the dam

The dams have been built with local material. For the construction material of the dam have been determined the geo-technical parameters as:

- specific weight varying from 2,68-2,70 gr/cm^3
- Volume weight varying from 1,9-2,0 KN/m^3
- Grain size distribution (fig.3) (size distribution curves)
- Angle of internal friction ϕ 30^0-38^0
- Cohesion C= 3-5...Kpa

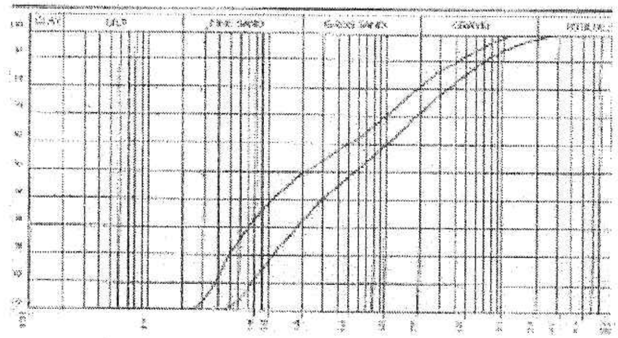

Figure 3.Size distribution curves

For the stored material or tailings have been determined the following geotechnical parameters:

- Specific weight varying from 2,69-2,73
- Volume weight varying from 1-1.5 gr/cm^3
- Porosity varying 55-61%
- Grain size distribution (fig 4)
- Angle of internal friction ϕ16-18^O
- Cohesion 5-10 Kpa

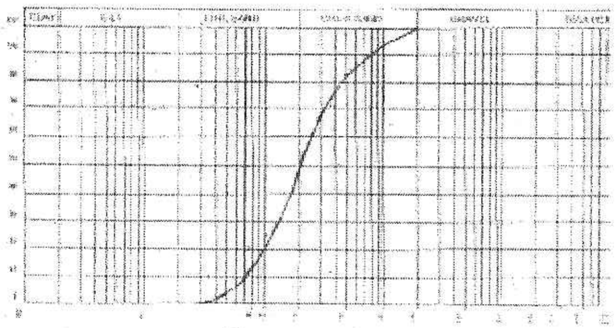

Figure 4. Size distribution curve

The above values have been determined for the material stored on the surface.

Besides have been calculated:

- The stability of the dams according to the classical methods Bishop-Cugajev assuming a circular cylindrical sliding surface
- Stability of the dams according to the method of Wedges (plans) for the construction phases
- General stability when the bottom of the dam has some slope
- Stability against the earthquakes according to the pseudostatical method because the Ant seismic Design Code has been approved and entered into force in the year 1989 when the process of the design of the dams had ended.

Construction technology

The technical literature an engineering practice treats many types of dams. In our practice we have applied mainly two kinds of construction technologies.

- Construction technology in mountainous sites (fig. 5)

It is constructed the initial dam being treated as an authentic (proper dam) taking care especially for not creating troubles connected with seepage of water. In the treatment of filters has been preferred the method recommended.
Construction technology in flat areas

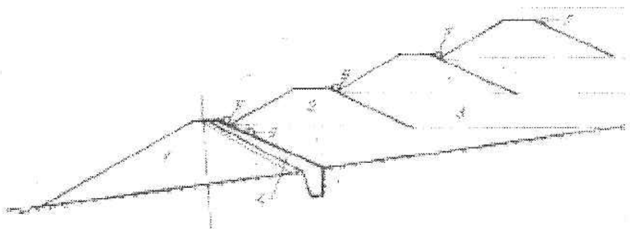

Figure 5. The process of the realization of the dam

1. *Dam*
2. *Phase 1-5*
3. *Settle tailings*
4. *Upstream screen when the initial dam is composed of previous material*
5. *The location of the pipes for the removal of the tailings.*

Exploitation, maintenance, conservation

During the exploitation of the dams have been taken the following measures:

- Continuous control of the chemical composition of the waters seeping through the dam. This is indispensable for keeping under control the degree of pollution of the environment.

- Control at intervals of the degree of the pollution of the soils downstream the initial dam.
- Lining of the completed layer of the tailing with a humus layer aiming at the avoidance of the pollution of environment during the first phase of conservation of tailings.
- Maintenance of the slopes and their protection against erosion (against the climatic conditions, rainfall, snow, etc.).
- The use of the tailings as raw material in the industry and metallurgy. This use can be done later on at the appropriate time. To this aim have been undertaken and are being conducted complex studies by the Faculty of Natural sciences and the Institute for Studies of Construction Technology, etc. Concretely according to these studies, on the basis of the physical-chemical and mineralogical properties of the tailings, these can be used as follows:

- For the preparation of mortars
- For the preparation of special concrete
- As layers of the roads of IV category (rural roads, mine roads, etc.) as well as raw material for the metallurgical industry

Present issues

The abandonment of the tailings dams as a result of the interruption of the activity for the extraction and processing of minerals during the transition period has been accompanied with negative impact.
Physical damage impact
Neglecting totally the maintenance, has started an accentuated erosive activity on the slopes of the dam (fig. 6), (fig.7) which may result in very dangerous consequences as for instance:

- Partial loss of stability
- Activating of the slides in the case of earthquakes
- Spilling of the stored materials outside the reservoir, which may lead to the blocking of the watercourses, destruction of the zone, etc.

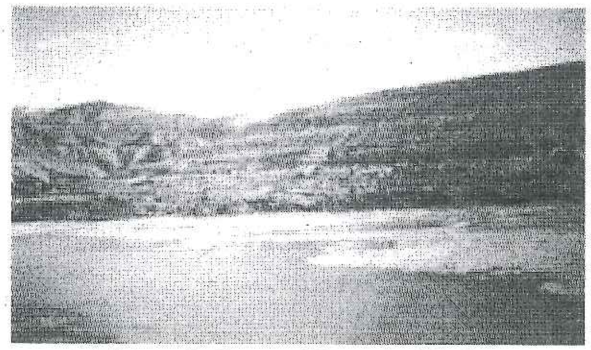

Figure 6. Dam of Reps in Mirdita

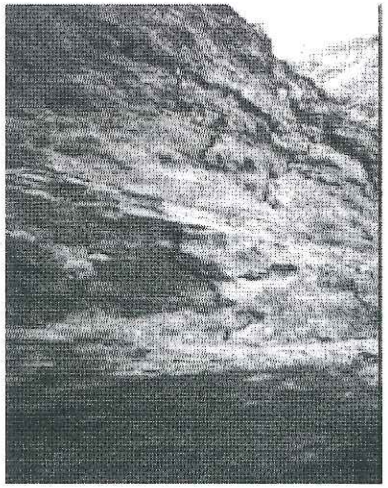

Figure 7. Dam of Reps in Mirdita

Ecological effect (impact)

The blocking of the drainage network, erosive activity, the partial damages, the increase of the seepage through the dam, lack of the protective humus cover leads to very dangerous consequences like the following:

- Pollution of watercourses and rivers.
- Creation of sulphuric and sulphurous acids which are disastrous for the plants in the proximity of the zone of the dam.
- The pollution of the fresh waters which may cause serious damage to the fauna in the surroundings .

In the chemical analyses (table 4) carried out 12 years after the abandonment of the dams, in the dams of Reps and Mirdite it results that for lack of a humus containing layer on the upper part of the dam (fig. 8) is still present the risk of the pollution (see table 2), meanwhile at the bottom of the dam near the watercourse where seep all the waters of the stored material, the percentage of the salts, particularly that of the sulphates, is minimal.

Table 4 Chemical analyses

Sample taken	Totalsaltsgr/100 gr%	cl %	SO4	PH %
In the stored material	2.25	0.020	large quantity	6.0
At the bottom of the dam	0.30	0.016	absent	6.3

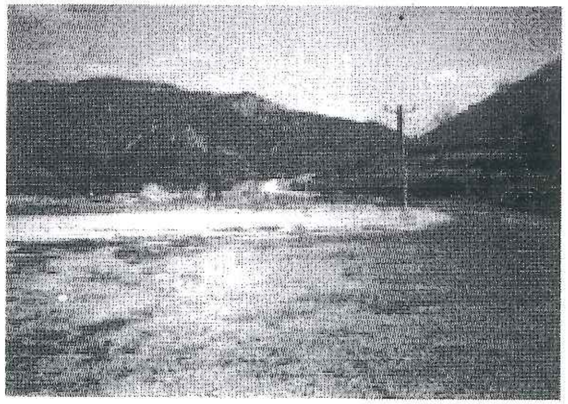

Figure 8. Tailing material in Dam of Reps

Conclusion and concrete proposals

The state should set up an organism or a working group composed of specialists of the respective field to supervise and monitor the development of these negative effects and to propose the protective measures.

REFERENCES.

Beliashevskii,N.N., Slobodian,R.T., Suhocukov,P.A., Kucineski,N.D., Osinskaja,A.A (1964).
Dinamiceskaja ustojcivost osnovanii iz vodonasishcenih otlozheni hvostov I stroitelstvo na nih napornih damb.
Tailings dams – guide to their design, construction and rehabilitation. Draft bulletin. (1994). ICOLD
Tailings dams and seismicity. Review and recommendation. Draft bulletin. (1995). ICOLD Ruli,F.,(1989). Tailings of Fe-Crr and Cu in magical industry and exploitation.

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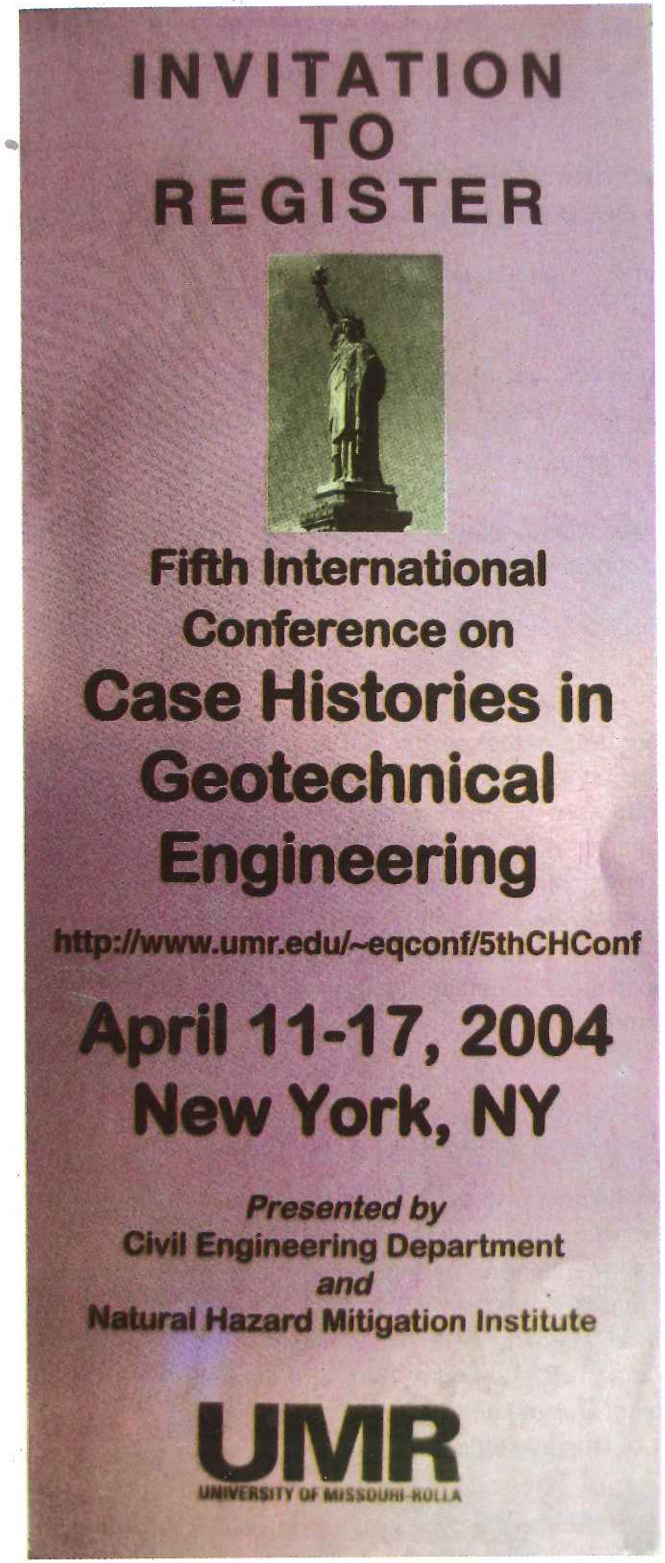

Fifth International Conference on Case Histories in Geotechnical Engineering.
New York NY (USA) April 11-17, 2004.

The New Resolution Of Foundations In Durres City.

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Abstract.

Albania is a country with strong earthquakes. There are observed some disastrous earthquakes in the cities of Durres, Vlora and Shkodra. Recently in the city of Durres are built many high – rise buildings in the low or bad soil-conditions.
In this paper, we would like to present our experience in the calculation and constructions of foundations in this area with high seismicity.

Introduction.

The Durres –City, the greatest port of Albania, is included in the high seismicity zone. The city has been under disastrous earthquakes several times. Before the 1990, the buildings in this city were all masonry structures 4-5 stories with continuous foundations with stones or concrete with stones. After 1990 has begun development of the area with 8-13 stories buildings, which unload in the loose soil basement very big loads, weren't the soil deposit reacts very bad to seismic vibrations. These conditions ask for new solutions for the foundations, are not realized before. In this paper, we want to present some of these solutions. The efficasity of these solutions is to be demonstrating on the first future earthquake.

Seismic activity.

Durres – city was ancient or antiques city Durachium (2500 years old). It is disturbed continually by strong earthquakes with epicenter in Durres-city or its surroundings. In this context we can mention the earthquakes that occur in the years 58, 334, 346, 506 with epicentral intensity 8-9 scale MSK - 64 and earthquakes of the years 1273, 1926 ect. with epicentral intensity of 9 scale MSK – 64. The strong earthquakes like earthquake of Vlora, 1851, Shkodra 1905 and Lushnja 1959 focused far from Durres are felt in this city (of Durres) up to 6 scale MSK – 64.

Seismotectonic of Durres region.

Durres region is established on basement of many active and complicated tectonics failures. This active falls of Quaternar can be classified:

- counter - rise like failure of Kryevidh - Bishti Palles, Thartor - Preze ect.
- over rise or over ascent like failure of Divjake - Kryevidh, Golem - Gjiri i Lalzit and other types of falls.

From the viewpoint of seismotectonics the Durres region participates in the Jonik – Adriatik seismogenealogical zone in wich the expectable seismic potential $M_{max} = 6 - 6.9$
Durres – city takes place in central segment with direction N – NW of the grand failure Shkoder – Peje and Vlore – Tepelene and we expect a seismic potential $M_{max} = 6.5 - 6.9$.
According to regional seismically map of Albania (scale 1:500000), Durres – city is included in such a zone where in the next 100 years are expected earthquake with intensity:

J= 8 scale MSK – 64 for the medium soils conditions.
J = 9 scale MSK – 64 for the weak soils conditions.

Geological conditions.

Durres city participates in Panadriatic lowland. The hills of Durres situated the est branch of Durres anticline and the hills of Shenavlash and that of Shkozet anticline are composed by:

- molasses formations of Mio Pliocen (alevrolitic clays and sands of Merinian Tortonian) N_1^{3mt}
- clay of Helmes suite N_2^h
- sand and conglomerate of Rrogozhine suite N_2^1

Between these hills lie Quaternar's deposits (sands, silty clays, silty sands and marsh clays) over the molasses formation which compose (to constitute) the big hole of Durres. Thicknesses of Quaternary's deposits vary from 30-50 m to 100-130 m.

According to geological investigations the site of the Durres-city is classified in III category (weak or low soils) and some times in IV category (very weak soils)
A geotechnical generalisated model or geological generalisated section is shown in fig. 1

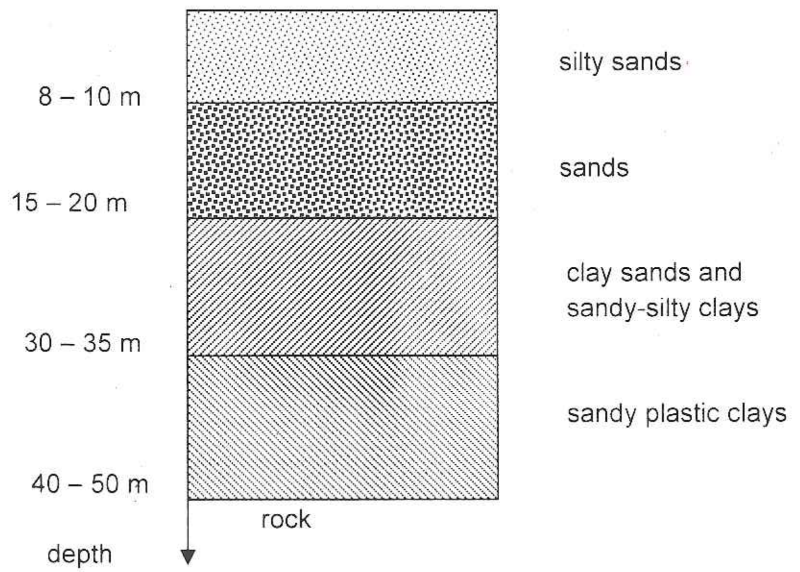

Fig. 1 Geological generalisated model

Geotechnical properties of soils.

For foundation's design of some 10-12 stories buildings in the area of Durres beach are accomplished some bore holes until 20 – 30 m depth for geological and geotechnical investigation.

All disturbed and undisturbed samples are tested in the ALTEA laboratory. The results of testing are:

- Grain-size distributions (fig. 2) of the tested sands show that they are classified in the grain size distribution of liquefied sands.

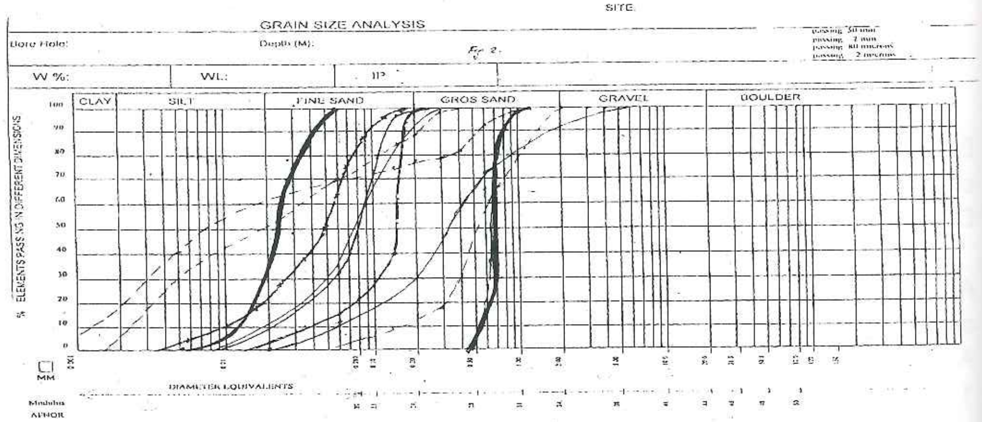

Fig. 2 Grain size analyses

- D_{50} of the tested sands change from 0,058 mm to 0, 52 mm.
- From field exploration in bore-holes [I] after SPT results that:
  N = 5 – 10 until depth 4 m
  N = 10 – 15 from 4 m until depth (8 – 10) m
  N = 15 – 25 from depth 10 m until (21 – 27) m
  N = 25 – 40 in depth greiter 27 m.

In fig. 3 are shown variations of N_{SPT} with depth in some bore – holes.

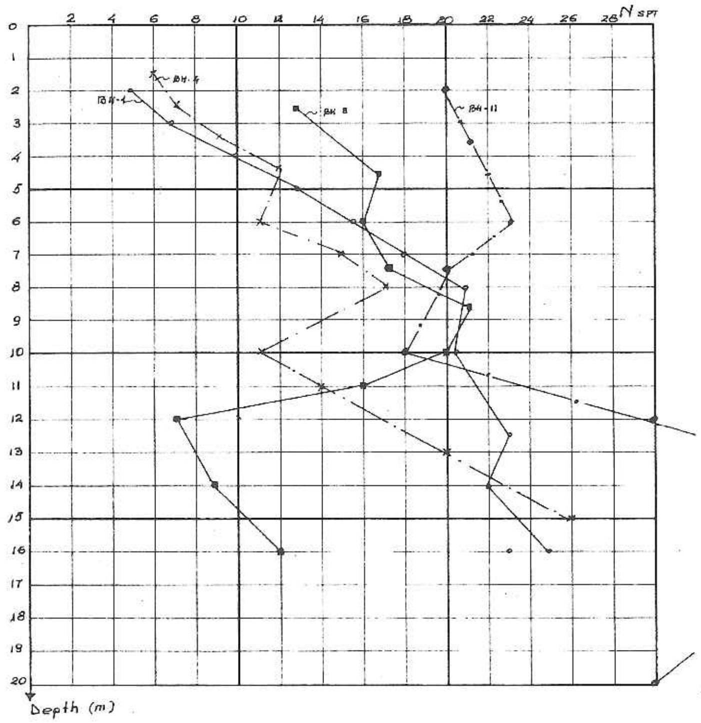

Fig. 3 Variation of N with depth

The values of N are very low and show that the sand have relative density D = (5-35)% so the sands are very porous.

- From geological investigation the level of water table is 0, 5 – 0, 7 m under natural surface.
- Moist unit weight of sands (until 10 m from the surface) is:

$\gamma = 17{,}5 \div 18{,}2 \quad KN/m^3$ and dry unit weight is

$\gamma_d = 13{,}1 \div 14{,}2 \quad KN/m^3$

In consequence, the tested sands have predisposition to liquefied.

- From Vs registrations effected in bore-holes results values which are show in table 1 and the calculated dynamique parameters of soils are shown (table 2) [2]

Table 1

Number of layer	Composition of layer	Thickness	Index plasticity	Vs m/s	γ KN/m³
1	Vegetable soil	0, 5	15	120	17,8
2	Fine sands with organic material	1, 5	-	160	18,9
3	Fine sands and silty sands	2, 5	-	160	19,0
4	Medium sands	10, 5	-	200	18,9
5	Silty clay	1 0, 0	15	245	18,7
6	Plastic clay-sand	15, 0	23	320	19,0
7	Rock (molasic formation)			700	20,8

Table 2

Geotechnical model	Denomination of layer and thickness	Vs m/s	R^n	E_d	G	R_c
II_2^c	Silty sand (4m) and fine sand	160	133	99200	36700	33
	Medium sand (10m)	200	166	163000	60300	65
	Sandy silty clay 10m	245	200	244000	90300	113
	Sandy plastic clay 15m	320	250	400000	148000	221

R^n –bearing capacity of soil KPa.
E_d –dynamic modules of soil KPa.
G –shear modules of soil KPa.
R_c –compressive strength KPa.

- After registrations effected by Seismically Center of Albania [3] results that in Durres – city maximum acceleration in surface of different geotechnical models is a_{max} = (0,18-0,44) g. Table 3.

Some registration of a_{max} for some geotechnical models of Durres

Table 3

Geotechnical model	Denomination of layer	a_{max} (g)	a_{over} (g)	Thickness (m)
I	Silty sand sand Silty clay clay	0,36 – 0,44	0,4	0 -2 3 – 15 15 – 28 28 - 130
II_1	Sand	0,26 – 0,34	0,3	1 – 15
II_{2a}	Sand	0,18 – 0,24	0,21	0 – 15
II_{2b}	Sand Sandy-clay	0,26 – 0,34	0,3	1 – 15 15 – 30
II_{2c}	Sand Sandy-clay clay	0,36 – 0,44	0,4	0 – 15 15 – 25 25 - 40

- The predominant periods of site is T_p = 0,7 – 0,9 s.
  All this parameters after classification of BOZO, L . [2] show that: the Durres - city is classified in the III and IV category of soil. Table 4.

Table 4

Category	T_p s	V_s m/s	N SPT	R KPa	D %
III	0,55 – 0,65	200 – 400	10 – 30	100 - 200	35 - 65
IV	0,7 – 0,9	< 200	0 - 10	< 100	< 35

In analytical way [3] are calculated strong seismic vibration admitted the rock bed a_{max} = 0,2 g and are used like entry functions five accelerogrames:

AL – EW – 15 April 1979 Ulqin
AL – NS – 15 April 1979Ulqin
EL CENT MS – EL Centro USA
PAR – NS – Parkfild USA 1966
YER – NS Loma Prieta USA 1989

In the table 5 are shown the calculate value of maximal acceleration a_{max} and amplification factor DAF for different level in geotechnical model studied of site.

Table 5

Depth m	AL - EW		AL - NS		EL CENT		PAR - NS		YER - NS		a_{max} (g)	DAF
	a_{max} (g)	DAF	a_{max} (g)	DAF	a_{max} (g)	DAF	a_{max} (g)	DAF	a_{max} (g)	DAF		
0	0,333	1,66	0,260	1,30	0,343	1,71	0,331	1,65	0,375	1,87	0,328	1,64
0,5	0,333	1,66	0,259	1,29	0,342	1,71	0,331	1,65	0,375	1,87	0,328	1,64
2,0	0,333	1,65	0,255	1,27	0,337	1,68	0,327	1,63	0,370	1,85	0,324	1,62
4,5	0,316	1,58	0,245	1,22	0,317	1,58	0,306	1,53	0,340	1,70	0,305	1,52
15	0,239	1,19	0,203	1,02	0,211	1,05	0,215	1,07	0,199	0,99	0,213	1,06
25	0,196	0,98	0,199	0,99	0,152	0,76	0,169	0,84	0,144	0,72	0,172	0,86
40	0,200	1,00	0,200	1,00	0,200	1,00	0,200	1,00	0,200	1,00	0,200	1,00

The problems and their solution.

- *Approximate evaluation of potential of liquefaction.*

In the absence of dynamic triaxial and shear cyclic box the evaluation of potential of liquefaction is made after the simple method of Seed – Idris. After calculation results that: the sand in two models can be liquefied up to 3 – 3,5 m and 5 – 6 m depth (fig. 4)

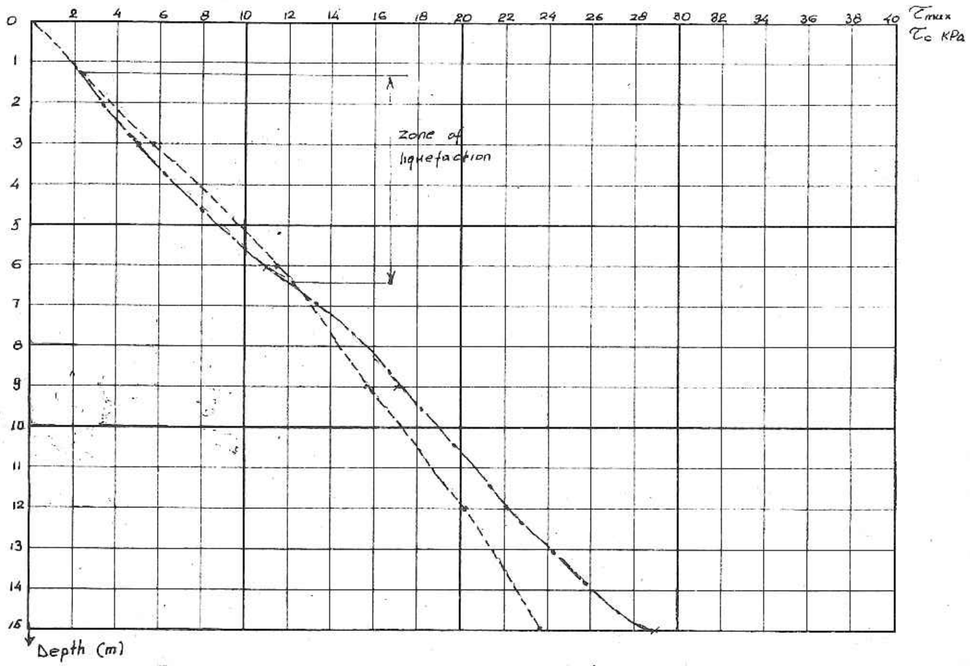

Fig. 4 τ_{max} and τ_c = f(H)

- *Selection of depth of foundation.*

After Albanian Seismically Code the depth of foundation is recommended to be

$$H_f = \left(\frac{1}{10} \div \frac{1}{5}\right) H_b .$$

(H_b –height of building over ground surface).

For the sites where the depth of liquefaction of soil can be H_l = 3 – 3,5 m, we choose H_f = 4 m and we have realized rigid mat foundations of two types: (Fig. 5), a) flat plate and b) slab with basement wall.

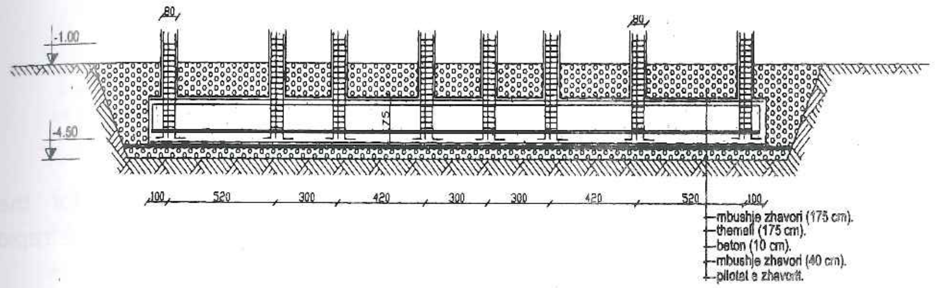

Fig. 5 a) flat plate b) slab with bazement wall

So the foundation is situated under the depth of liquefaction H_l and with its depth provides augmentation of rigidity and diminution of moving of building.
We have calculate the variation of K_v and K_H with H_f. (H_f – depth of foundation) Fig. 6.

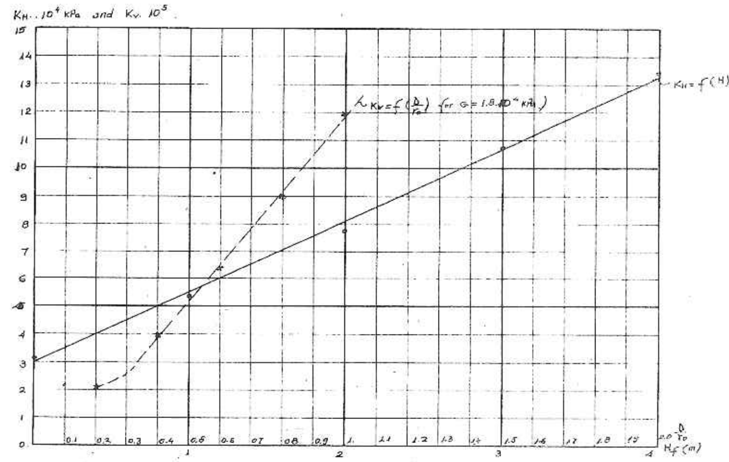

Fig. 6 Variation of K_v K_H(sand with G = 0,3 . 10^4 Kpa, H = 10 m, foundation with R = 1 m)

For the sites where depth of liquefaction of soil can be H_l = 5 – 6 m. we choose H_f = 7 – 9 m, realizing plate foundation over concrete piles or plate over gravel piles fig. 7.

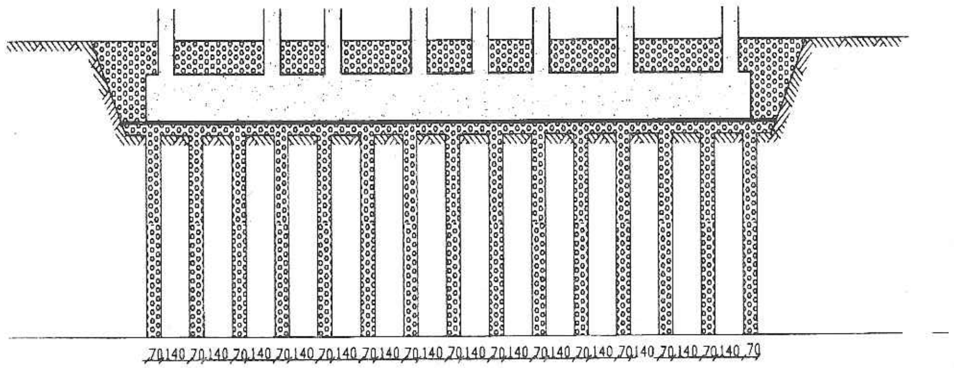

Fig. 7

The concrete piles improve the basement and can realize more rigid springs for the concrete plate, were as the gravel piles make reinforcement of soil and secure rapid dissipation of pore pressure and decrease of liquefaction phenomena.

- *Selection of type of foundation*

  - The following principles have influenced for the selection of the type of foundation (shallow or deep foundations):
    - to provide good fixation with ground
    - to provide structure vibration with period or frequency far from natural frequency or predominant period of soil
    - augmentation of rigidity of structure to provides small movement and deflections of structure
    - to provide stability for the building

  - Type of shallow foundations

    We have realized:
    - flat concrete plate with height 1,7 – 2m
    - slab with basement wall (h = 1,2 – 1,5) m situated 4m depth below natural surface

    Considering the influence of depth of foundation in rapport K/K_z and the influence of the shape of foundation $\alpha = \frac{l}{b}$ in this rapport (fig. 8) we have choose the type of foundation (K_z' and K_z are coefficient of rigidity of foundation respectively in depth H_f and over free surface)

  - In the case of combination of plate with piles.

    The piles are realized in three methods:
    - drive concrete piles with diameter d = 30 – 60 cm and bearing capacity 250 – 350 KN
    - casting or mould on site concrete piles with d = (30 – 80) cm and bearing capacity 200 – 400 KN
    - struck concrete piles with d = (30 – 50) cm and bearing capacity 270 – 380 KN

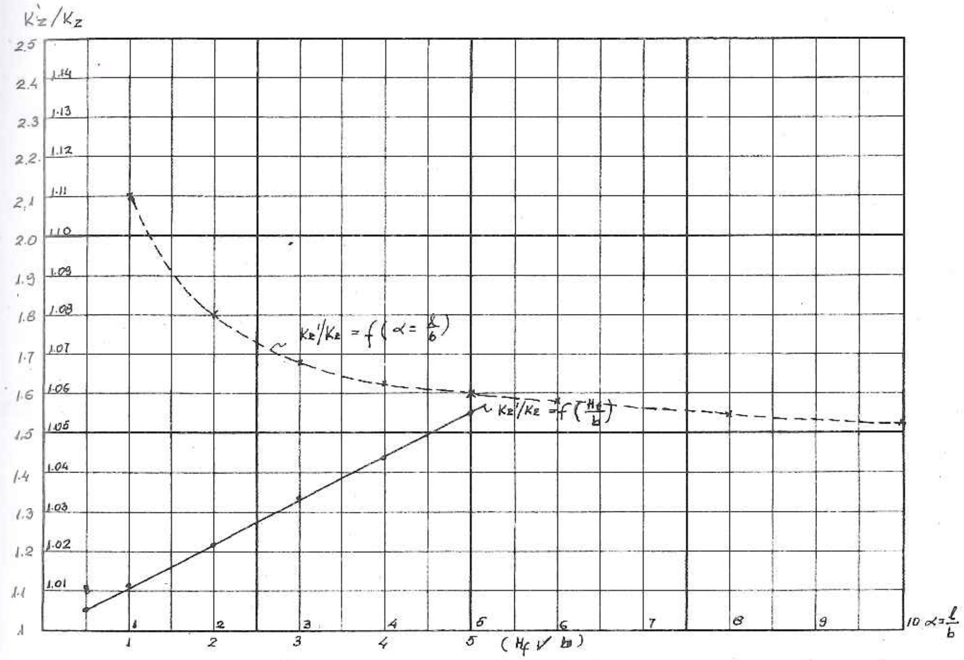

Fig. 8 Variation of report K'_z/K_z from H_f/b and α

  - In the case of the combination of plate with gravel piles with d = 60 cm and length 700 – 900 cm.
    We have achieved these aim:
    - reinforcement of soil (we have arrive D = 70 – 75%)

- realization of vertical drainage which is combinated with horizontal drainage and together provide dissipation of pore pressure
- augmentation of the coefficient of sub grade reaction Ks (coefficient of springs) from 10 - 15 MN/m^3 in natural soils to 130 – 150 MN/m^3, after improvement. Consequently diagrams moment is decreased and the quantity of steel in the mat (plate) foundations

Conclusions

Solution of foundations of 8 – 10 – 12 stories buildings in the Durres – beach provide:

- Good fixation of the building in the ground.
- Augmentation of the rigidity of the structure for all types of vibrations.
- Improvement of ground.
- Reinforcement of soils.
- A decrease of possibility of liquefaction of soils.
- Increase of stability and security of buildings.

REFERENCE

1. Ahmetaj, L., Allkja, S., Bozo, L. — Report of ferry terminal port Durres, 2003.

2. Bozo, L. — Behaviour of the basements and foundations under seismic action. Thesis of Doctor in Philosophy.

3. Duni, Ll. — Engineering and seismological study for the place when are constructed 10-12 stories buildings in beach Durres 2003.

4. Gazetas, J. — Edafodinamiki kai seisimiki mihanik, 1996 Simeon publication.

5. Prakash, Sh. — Soil Dynamics, 1981

YEAR 2005

- Séminaire International "Le compactage des sols" Hammamet- Tunisie
  Mars 25-26 2005
- The second international symposium for construction. Tetovo -Macedonia
  April 21-22 2005
- Xth Baltic Conference Riga Latvia
  May 11-13 2005
- 2005 International Conference on Landslide Risk Management Vancouver Canada
  May 31-June 3 2005
- International symposium "50 years of pressure meters "Paris France
  August 22-24 2005
- European young Geotechnical Engineers Conference Zagreb Croatia.

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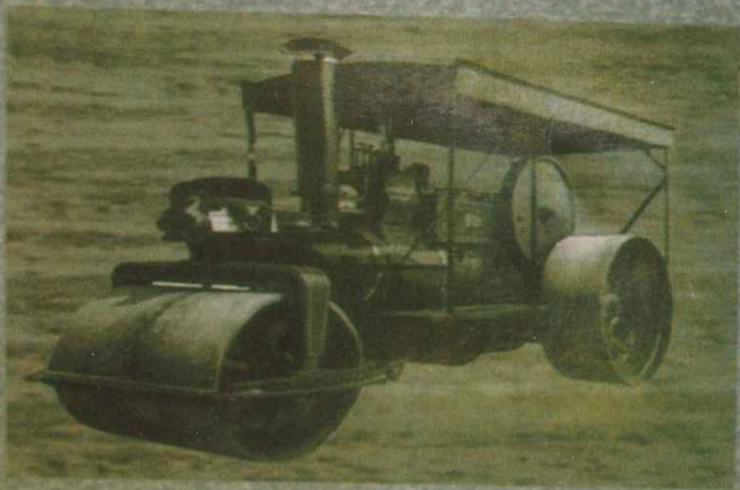
LE COMPACTAGE DES SOLS



Organisé par :

L'Association Tunisienne de Mécanique des sols, ATMS

et

L'Ecole Nationale d'Ingénieurs de Tunis, ENIT

Sous le parrainage de

la Société Internationale de la Mécanique des Sols et de la Géotechnique, SIMSG

25-26-mars 2005

Hammamet - Tunisie

SEMINAIRE INTERNATIONAL

LE COMPACTAGE DES SOLS

25/26 MARS 2005 – Hammamet – Tunisie

Presentation: COMPACTAGE DES COUSHES DE LA ROUTE TIRANA –DURRES EN ALBANIE

Professeur Luljeta BOZO Department de la Geotecnique Faculte de Genie Civil Universite Polytecnique

Tirana, ALBANIA

Resume:

La route Tirana – Durres est la premier route importante (de la categorie I) en Albanie. Pour la sa construction il etait faite des recherches pour les materaux usage, pour la grade de leur compactage , pour le control de qualite ect. Nous voulons presante le notre recherche sur le compactage des sols en zone de Vora et quelque relations entre les parametres physique et mechanique des sols.

En zone de Vora la route passe en terrain faible , pour consequence on construire un remblai avec 7 metres de hauter. Le materiau utilise pour le construction de remblai est sable avec faible cimentation. Les resultats de l'analyse ganulometrique et des limites de plasticite indique , que le materiau n'est pas tre bon qualite. Cette est evidente dans le grade de compactage, la densite maximum est γ_d=18.8 KN/m3 pour la humidite optimale W_{opt}=10,2%.

Nous avons faire une amelioration de materiau utilisante 4-6% ciment ou 6-10% materiau fine. En consequence etre concu le densite maximum γ_d=20.2 KN/m3 pour W_{opt}=9.6%.

Selon les essais CBR nous avons pis pour cette materiau un resultat bon 25-35 %.

D'apres les essais du chargement statique de la plaque avec dyametre 300 mm nous avons pris un resultat satisfeaisant pour le modul de deformation de materiau de remblai M_o=435 Kg/cm2.

Conclusions:

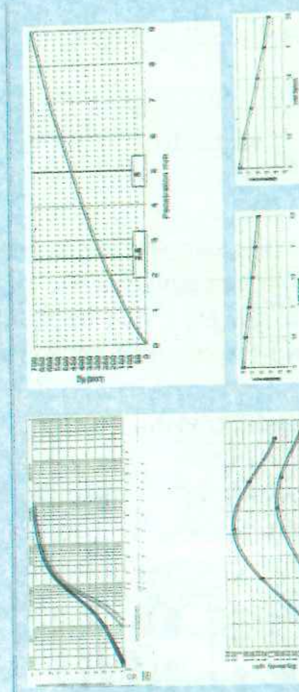
- L'ameloration de materiau sableuse entraine une augmentation de la grade de compactage 10-12 %
- Usage d'une pourcentage de materiau fine augmente le modul de deformation de 222 kg/cm2 jusqu a 435 kg/cm2.
- Entre les parametres physique et mecanique il y a bon corelation:

M_o=(15 – 17) CBR

γ_{max}=18.5+0.05 CBR

(pour CBR=25 – 30 %)

- L' explotation de la route pendant 10 anns on faite sans aucun problem.



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PËRMBLEDHJE E PUNIMEVE

TETOVË, 21-22. PRILL 2005

Construction in the weak terrains.

Prof.Dr.Luljeta BOZO
Geotechnical Sector Civil Engineering Faculty Polytechnic University of Tirana

Abstract

In Albania exist many weak zones as marsh zones in Durres, Torovica, Tirana, Lezha, Divjaka etc. The problematic zones are one part of litorial of Adriatic sea, also active landslide and erosion activity of rivers. In some of them we have serious problems with enormous deformations. In these paper we would like to present how influenced the deformation characteristics of soils in choice of type of structure.

Introduction

Part of Corridor eight pass through the very problematic soils of Maliq marsh. The highway cannot construct in these basement. So we undertake a study to realized one embankment for the construction of high way in these zone. To perform a study we have realized bore holes until 35-37m depth, pits, laboratory tests and in situ tests.

Study of terrain

- From geomorphologics study results that: the zone is a tectonic depression filled by alluviums and surrounded (contured) by soft and round hills no very high.
- The geologic and geodynamic characteristics are the:
  - Erosive activity of rivers Devoll and Dunavec
  - Weathering of limestone and sandstone of hills material.
  - The consolidation of very porous deposits
  - Landslides
- Seismic danger. From seismic investigations results that:
  - The soils of basement are a 3-thd category by Albanian rules KTP-89
  - The seismic intensity is 8 by MSK-64 and with M=6,8-7 by Richter scale.
- Geologic composition (construction).

From geologic map of Albania these zone is concerned a Mirdita zone where the deposits are igneous; carbonate and terrigene origine so in these zone we find:

N_{1a} – neogene compound by conglomerates and sands
N'_{1b} - neogene compound by clay,sand and calcareous.
N^{2}_{it} – neogene of tortonian compound by clay and sands
Q^{dt}_{4} – quaternars delluvions, compound by gravel sands and silty clay.
Q^{kt}_{4} – quaternary march compound by organic clay, silty clay, loam, clay-loam (with high percentage of organic matters).

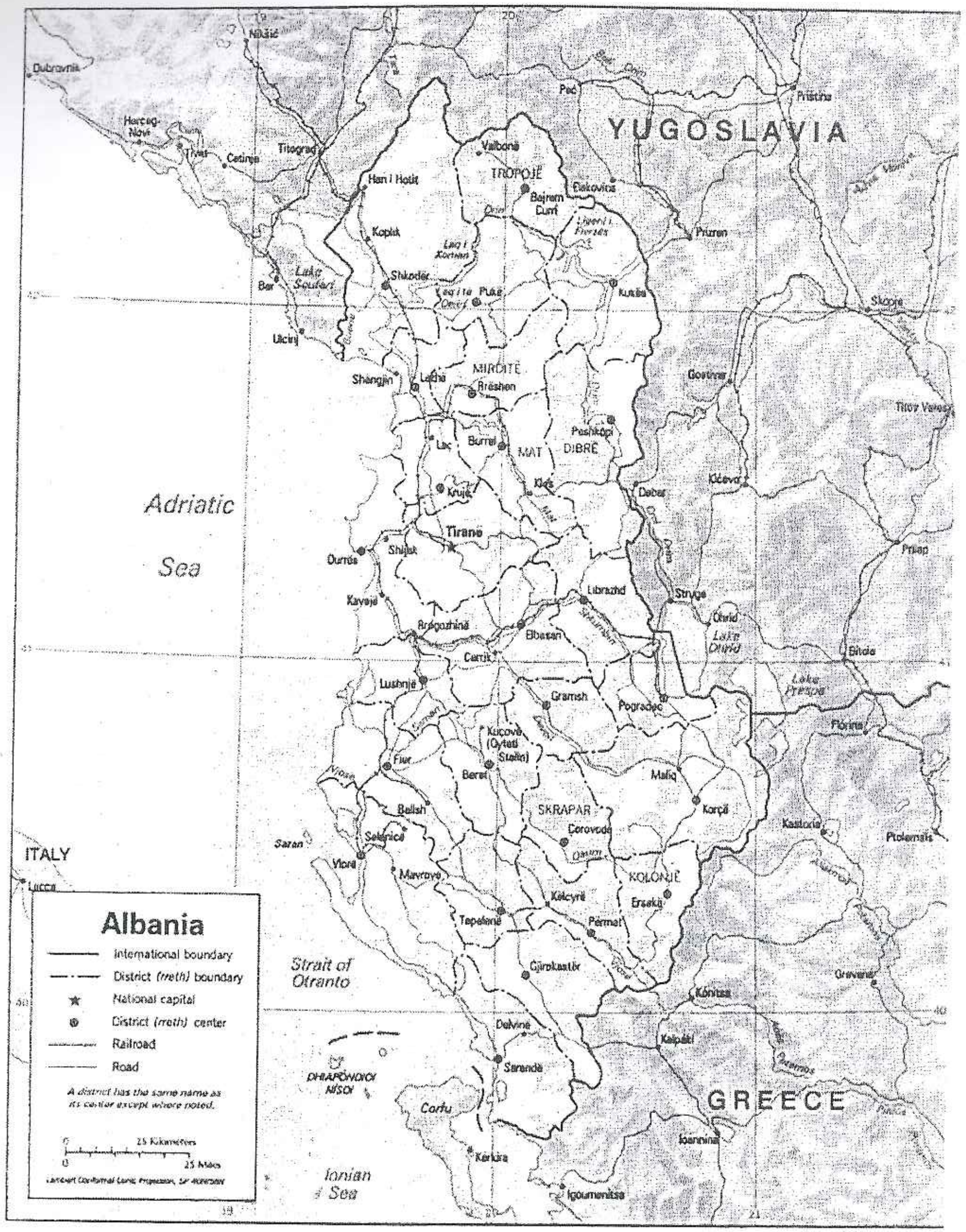


Fig.1 Map of ALbanian

From bore-holes results 9 layers (until 35m depth) where their characteristics give in table 1.

- The geotechnical study.

From the studied zone we can use three geotechnical models (fig.2). Also we can evidenced two general layers which have :

- Percentage of organic matter (1-13,5%)
- Passing in sieve 200 (12-92)% for the thickness H=(0-30)m
- Passing in sieve 200 (4-11)% for the thickness H=(30-37)m

Table 1

Layer	I_P	E KPa 10^4	C cm^2/s 10^{-3}	γ gr/cm^2	φ^0	C KPa	W %	$I_c = \frac{w - w_p}{PI}$	Classification by acs
1	19	0.27	0.70	1.68	23	35	46-55	0.78	ML and OL
2	13	1.1	0.90	1.98	22	34	19-20	0.23	ML
3	20	0.35	0.11	1.89	18	40	37.5	1.1	CL
4	17.5	0.5	0.60	1.72	20	35	42	0.5	MH
5	-	1.1		1.78	26	-	-	-	SM
6	7	0.7	0.8	1.95	24	30	26.5	0.33	CL
7	16	0.2	0.7	1.77	22	32	32	0.26	ML and OL
8	-	2.0	-	1.90	28	-	-	-	GM
9	14	0.38	0.8	1.82	21	20	36	0.52	ML

We have compiled the relation between dry density (γ_{sk}) and void ratio (e) with depth h=(0-30)m Fig.3

Modelet gjeoteknik

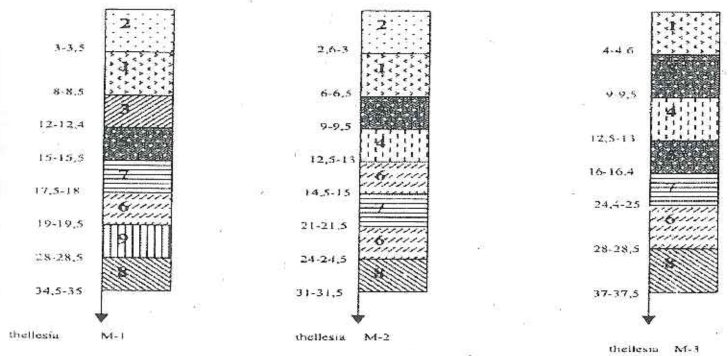


Fig.2 Geotechnical models

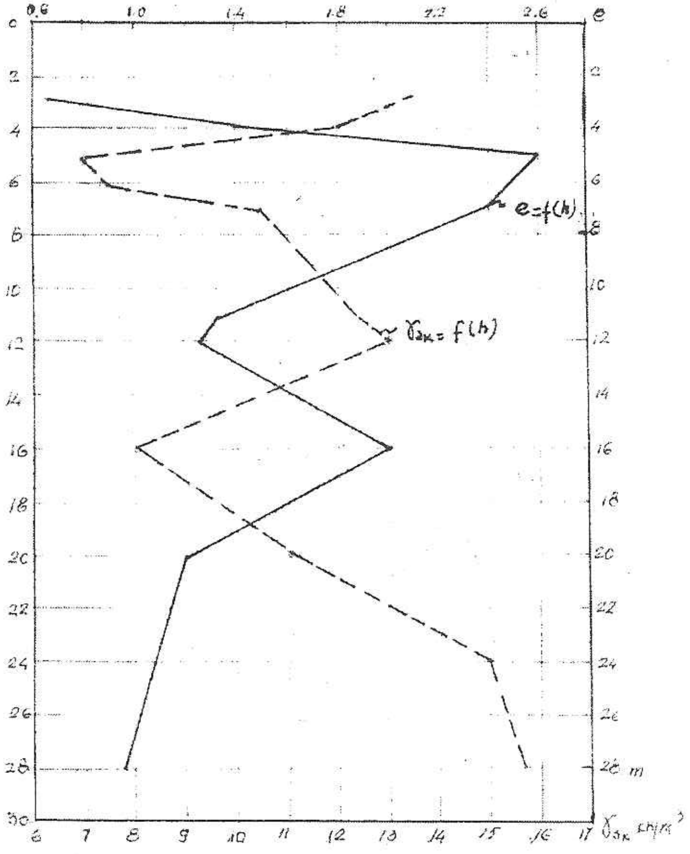


Fig.3 . Relation betwen $\gamma_{sk} - h$ γ_{sk}-e

From eodometric test realized in laboratory we have compile the compression curves (fig.4) and from them results C_c =0,2-0,344, that is to say we have very deformable layers ; C_v =(0,11-0,9)$10^{-8}cm^2$/s.it to say we have long time (duration) for first consolidation; C_α-0,014-0,03 - the secondary coefficient show that the settlements from second consolidation will be considerable.
For the determination of shear stress of these soils, we have realized test in triaxial apparatus and in the direct shear apparatus.
From triaxial tests we have determine
φ_u= 6-10^0
C_u= (16-30)KPa that is to say the basement will be with very low bearing capacity.

From tests realized in direct shear apparatus we have determine the values of angle of friction and cohesion which are very different. These can explend by their different state of moisture content (fig.5 and table 2)

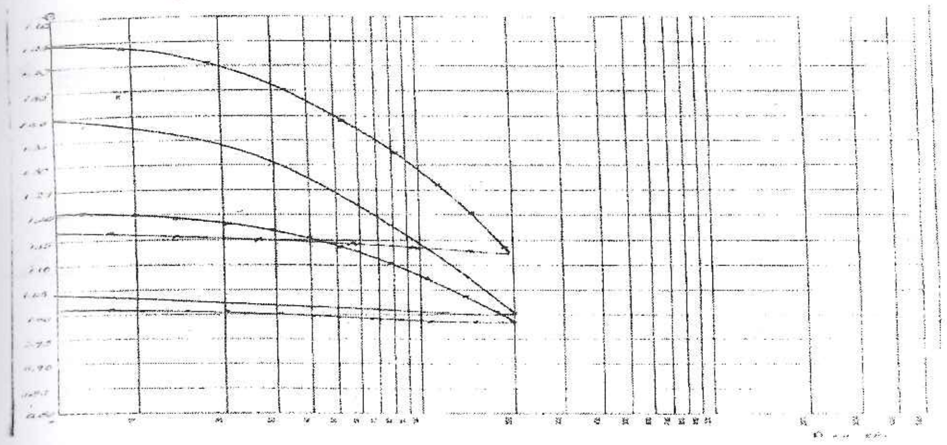


Fig.4 Results from eodometric test

From geotechnical study we arrived in following conclusions:
- The basement is very weak
  e>1 - with very high porosity
  I_k>0,5 – in the soft plastic state of liquid-plastic state. It has very high percentage of organic matter.
- The behavior of basement's soils under the extern loads shall be:
  Very big settlements by first consolidation
  Very long time to finish these settlements
  Low bearing capacity
  Danger to loss of partial stability of the basement.

Structural solution for the road's passage in these zone.

The structural solution can be one of two variants: over pass or embankment.
Analysis of the embankment variant . First of all we have choice the material for embankment.So from Proctor test we have determine dry density and optimum moisture content.
γ_{sk} =22KN/m^3 W_{opt} = 4,7% Also we have determine:
LA= 18,2%   E = 2,15.10^4KPa φ=35^0 C = 23KPa

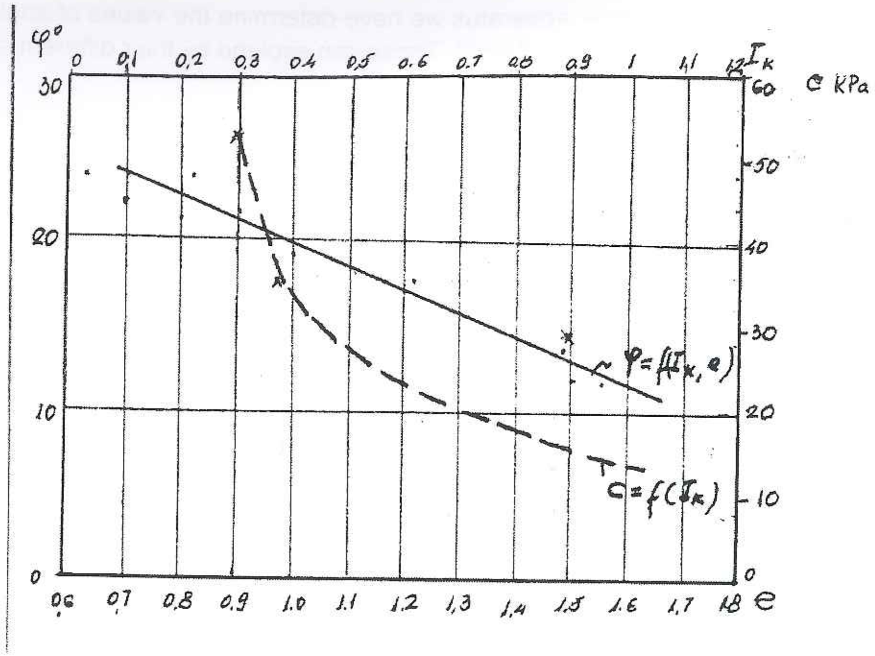


Fig.5 Relation between φ-I_k , φ-e, C-I_k C-e

Table 2

Consistency	Void ratio	Internal angle	% of clay particle	Consistenc	Cohesion
$I_c = \frac{w - w_p}{I_p}$	e	φ^0	%	I_k	C KPa
0.22	0.62	24	44	1.9	20
0.26	0.72	22	40	0.99	32
0.30	0.96	21	38	0.4	35
0.384	0.99	19	38	0.9	18
0.52	1.23	18	36	0.37	36
0.88	1.27	16	34	0.4	30
0.94	1.49	14	30	0.3	55
1.9	2.2	6	20	0.845	18

The geometry of embankment shall be as you can see (Fig.,6) with:

High H=5m

Inclination 1:1,5

Width b_1 = 17m

Width in the base of embankment B=32m

- Embankement rests in geotechnical model M-1

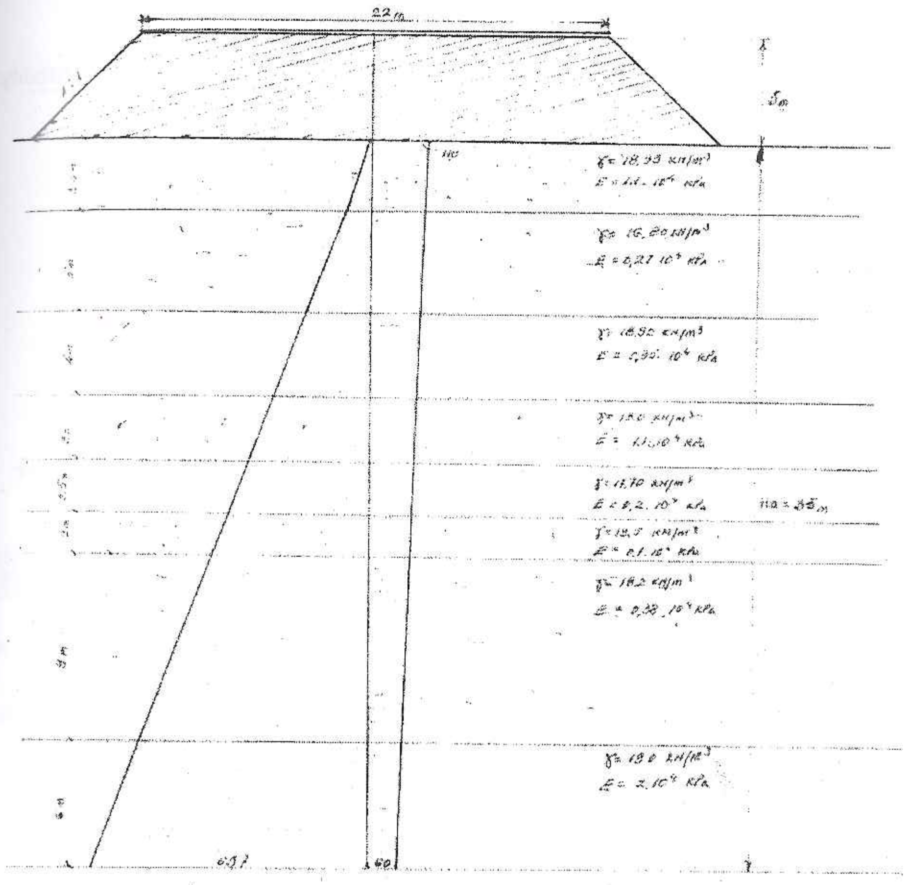


Fig.6 Embankment and their active zones

❖ The embankment shall be rest in the geotechnical model M-1. We have calculated:

- Factor of safety for embankment FS=1,72
- Active zone under of embankment Ha=35m
- Settlements from first consolidation S_1=35cm
- During of these settlements T=8,8 years.
- Settlements from secondary consolidation S_2=18cm
- Total settlements $S=S_1+S_2=53cm > S_{lim}$
- The rapid construction of embankment can to cause the loss of stability of one part of the basement under of embankment (fig,7).

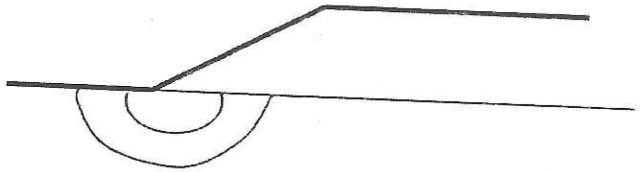


Fig.7. The zones which can loss stability in the basement of embankment

❖ The possible solutions. In these situations we think to improuve the soils of basement by two ways.

- First manner can to be improivement of the basement by gravel piles with length (12-13)m (fig.8). By these improuvement we will profite: Augmentation of moduls of determination which arrive the values $E=1,1.10^4$KPa (three times biggest in rapport with initial modulus).

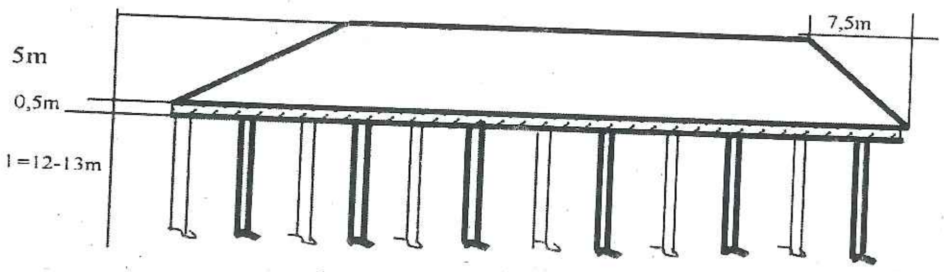


Fig.8 Embankment over gravel piles

Decrease of time of deformation because the gravel piles served as vertical drainage and neutral stresses go out rapidly. These process is favorized by combination of vertical drainage with horizontal drainage.

- Second manner is utilization of geotextil (fig.9). By these manner of improuvement we shall be profit:

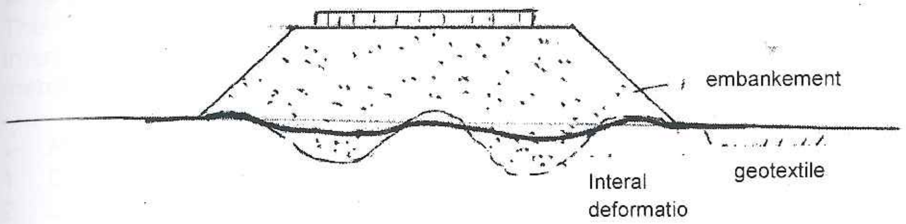


Fig.9 Utilization of geotextile

- Smaller deformation of base of embankment
- Better dissipation of pressures in the base of embankment
- Protection of embankment from fine particles (clay, loam, silt)
- Insurance of normal work for the embankment.
- Insurance of short time for the construction of embankment.

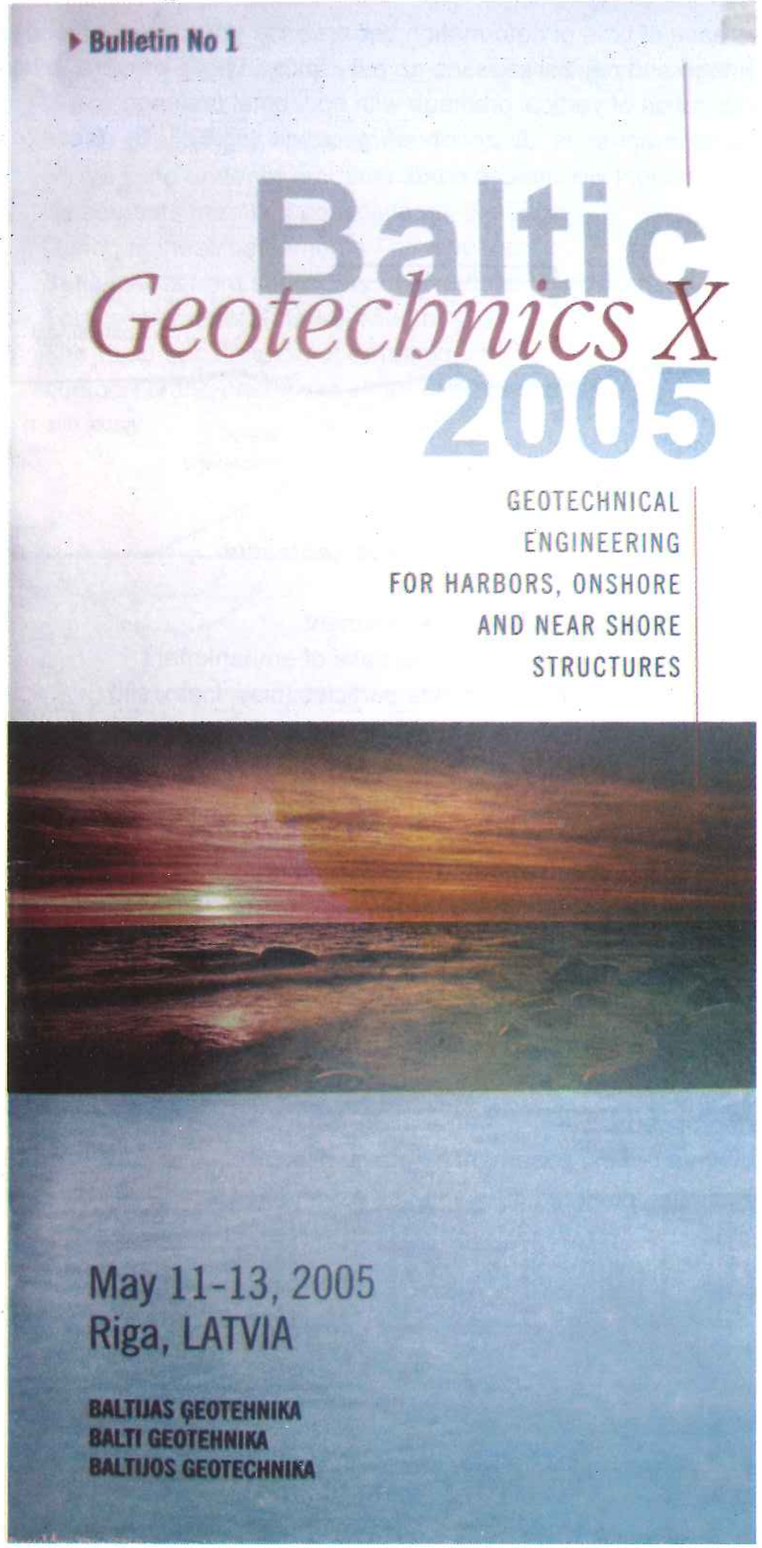
Conclusions

Construction in marsh deposits perform danger and alarming for the objects, which are:

- Very big settlements which surpass the limit value.
- Considerable settlement by second consolidation (about 30% of total settlements).
- Long time to finish the process of deformation of soils

The engineering measures can to eliminate or mitigate the negative phenomena's. Such can be:

- Ulitization of geotextile
- Improuvement of the basement by gravel piles.
- Utilization of combinated drainage vertical and horizontal.



➢ *Objectives*

The main goal of the Conference will be the discussion of all geotechnical aspects concerning design and construction of harbors and structures connecting with them. Geotechnical investigation, research, design, modeling, construction techniques, case histories, environmental protection, quality control and monitoring will be main topics of Conference.

➢ *Participants*

The conference will be mainly held for the members of Baltic Sea Region of the International Society. Members from other countries, who interested, and non-members are also welcome to attend.

➢ *Proposed topics*

1. Geotechnical investigation for harbors, onshore and near shore structures.
2. Design values of ground parameters, geotechnical design and design philosophy of harbors.
3. Environmental aspects of harbors and their infrastructure.
4. Geotechnical construction of harbour structures and dredging
5. Geotechnical monitoring of harbour structures.
6. Case histories.

➢ *Call for papers.*

We are inviting authors to submit papers on topics of interest in geotechnical engineering for harbours, onshore and near shore structures.

Abstracts must be type in English on an A4-size paper with 300-450 words. Abstract must include:

- Title of the paper.
- Name of the author.
- Affiliation, address, fax/phone, e-mail

Geotechnical investigations for the rehabilitation and reconstruction of Durres port.

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Eng.Luftim Ahmetaj ALTEA&GEOSTUDIO 2000 sh.p.k
Eng.Skender Allkja ALTEA&GEOSTUDIO 2000 sh.p.k

Abstract.

The Port of Durres is the greatest in Albania and means ingress of the "8 Corridor" which Connects Europe with Asia. It is in Adriatic Sea in a problematic zone from the geological and seismic point of view. At these time is starting the work for the rehabilitation and reconstruction of Durres port. For that purpose is undertaken a complex geological and geotechnical study. In this paper we would like to present some parts of this research and the problems related to constructive solutions in the port.

Introduction.

The port of Durres is the greatest in Albania and the mean ingress of "8 Corridor" which connects Europe with Asia Increase of the commercial business, mercantile fleet, free movement of people request measures for rehabilitation and reconstruction of the Durres port.
For that purpose in this zone is undertaken a complex geological and geotechnical investigation from ALTEA &GEOSTUDIO 2000 sh.p.k. The results of this study serve for design of portal construction. In this paper we wont to presented some part of this research, the correlation between geotechnical properties and the problems in respect to design of portal construction (pile foundations, concrete plate etc.)

Seismic activity and seism tectonic of Durres region.

Durres city was on ancient or antiques city Durachium (2500 years old). It is disturbed continuously by strong earthquakes with epicenter in Durres-city or its surroundings. In this context we can mention the earthquakes that occur in the years 58, 334, 346, 506 with epicenter intensity 8-9 scale MSK-64 and earthquakes of the years 1273, 1926 etc, with epicenter intensity of 9 scale MSK-64.

The strong earthquakes like earthquake of Vlore 1851, Shkodra 1905, and Lushnja 1959 focused far from Durres are feelt city up to 6 scale MSK-64. Durres region is established on basement of active and complicated tectonics failures. This active falls of Quaternary can be classified:

- Counter rise like failure of Kryevidh-Bishti I Palles, Thartor-Preze ect.
- Over rise or over ascent like failure of Divjake-Kryevidh, Golem-Gjiri I Lalzit, and other types of falls.

From the view point of the seismotectonics, the Durres region participates in the Jonic-Adriatic seismogenealogic zone in which the expectable seismic potential is M_{max}=6-6,9. According to regional seismic map of Albania (scale 1:500000) Durres-city is included in such a zone where in the next 100 years are expected earthquake with intensity:

I = 8 scale MSK – 64 for the medium soil conditions.
I = 9 scale MSK - 64 for the weak soil conditions.

Geological conditions

Durres-city participates in Panadriatic lowland. The hills of Durres situated the est branch of Durres anticlinal and the hill of Shenavlash and that of Shkozet anticlinal are composed by:

- Clay of Helmes suite N^{h}_{2}
- Sand and conglomerate of Rrogozhine suite N^{h}_{2}
- Mollasic formations of Mio Pliocene N^{3mt}_{1} (alevrolitiv clays and sands of Merinian Tortonian).

Between these hills lies Quaternary deposits (sands, silty clays, silty sands and march clays) over the mollasic formations which compose (to constitute) the big hole of Durres. Thickness of Quaternary's deposits varies from 30-50m to 100-130m.

According to geological investigations the site of Durres city is classified in III category (weak or low soils) and some times in IV category (very weak soils).

A geological generalized model, or geological generalized section (from geological investigations) is shown in fig.1a, for model in the sea and 1b, for model in the mainland.

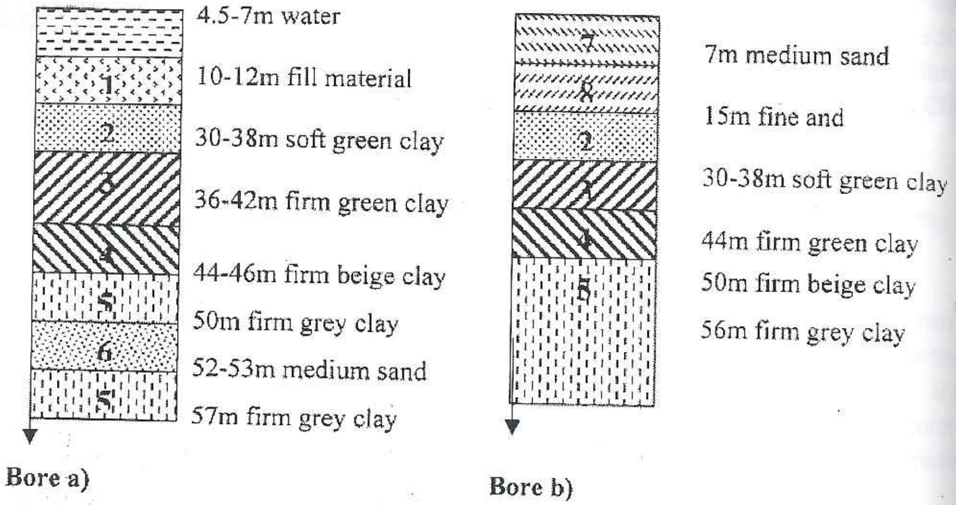


Fig.1 Geological generalized model:
a) In the sea
b) In the mainland

The used method for geotechnical investigations.

For geotechnical investigations the following manner are used:

The test in the bore holes

a) N_{SPT}
b) Van test;
c) Dynamic penetrometer test.

The test in the trial pits.

a) In situ density
b) Plate load test
c) Proctor and CBR test

The laboratory test

a) Classification test
b) Eodometrique test (one dimensional test)
c) Direct shear test
d) Triaxial shear test CD and UU
e) Unconfined compression test

Geotechncial properties of soils.

A. Field investigation.

- From field exploration (the test in situ) realized in bore holes we have taken the variation of N_{SPT}, R_d and S_u (from N_{SPT} test, dynamic penetrometer, van test) with depth, which are shown in fig,2.

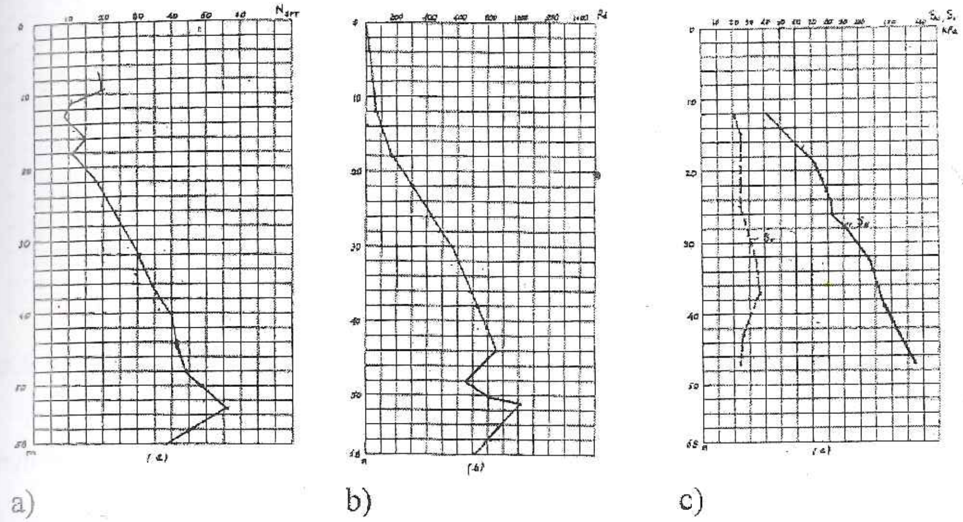


Fig.2 Vartiation of
a)N_{SPT} – h
b)R_d – h
c) S_u -h

- From N_{STP} result that:
  - The first layer with N_{SPT} = 17-30 has D=30-60% and $\varphi=35^0$
  - The second layer with N_{SPT} =12-16 until depth 20-22m has I_c =0,45-0,55 and with N_{SPT} =16-32 from 22m to 38m has higher compressibility.
  - The third layer with N_{SPT} =28-42 has I_c =0,26-0,4 and presence of beds of fine sands.
  - The fourth layer with N_{SPT} =36-42 has I_c =0,2-0,25
  - The fifth layer N_{SPT} =44-48 has I_c = 0,2-0,1
  - The sixth layer with N_{SPT} ==48-56 has $I_c < 0.\left(I_c = \dfrac{W-PL}{PI} \right)$

In fig.3 are shown the variation of N_{SPT} with I_c

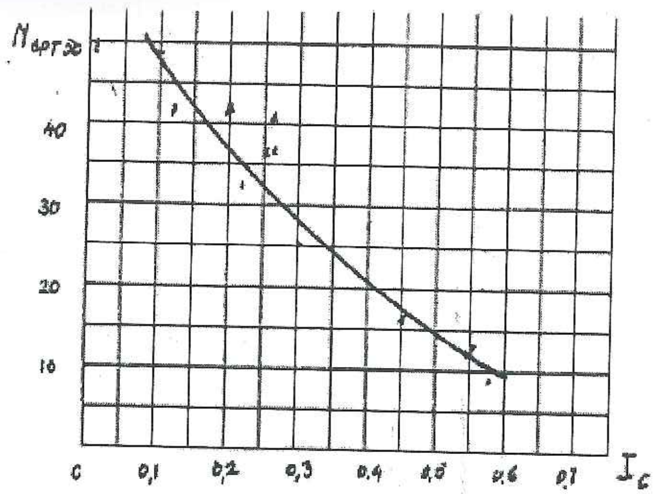


Fig.3 Variation of N_{SPT}vs I_c

- ➢ From dynamic penetrometer test in bore holes result that.
- The clear (evident) correlation between the point resistant R_d of dynamic penetrometer and consistency index I_c (fig.4)
- In depth 38-44m, is presented the layer of the firm to stiff, brown to beige clay silt with parting silty sands, containing shell fragments which was possible very ancient slide from the mainland.
- The good correlation between R_d and point resistance of piles (fig.5)

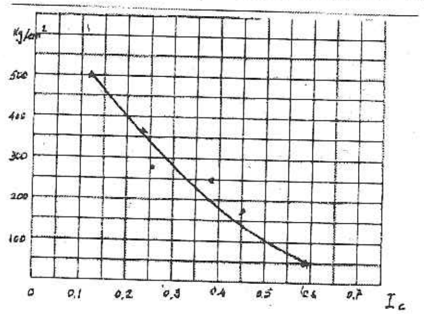


Fig.4 Variation of R_d with I_c

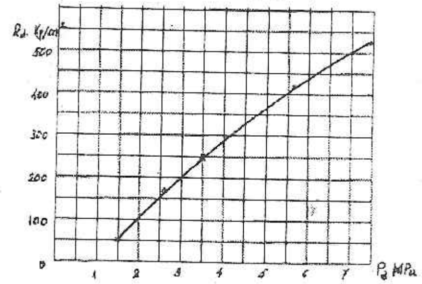


Fig.5 variation of R_d with point resistant of pile P_σ

- ➢ From profiles of the Van test in bare holes result that:
- The maximum cohesion S_u increase with depth (for any layer) (fig.6)
- Ratio S_u/S_r (S_r – residual cohesion) change from 2-3,5 until the depth 20m and S_u/S_r = 3,5-4,6 from 20m to 34m (for second layer). That is to say the clay is

very sensitivity with regard to his structure and that can explain with high percentage of silty materials (content of silt is 35-60%).

- It has a good correlation between R_d and C_u (fig.7) and R_d and C_c (fig.8)
-

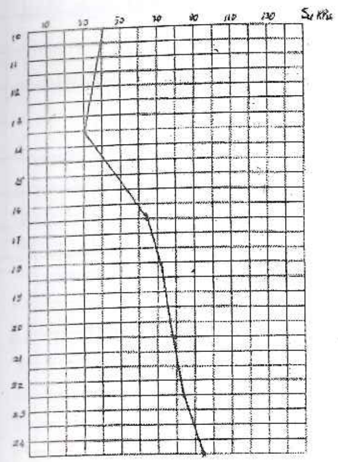


Fig.6 Variation of S_u with depth

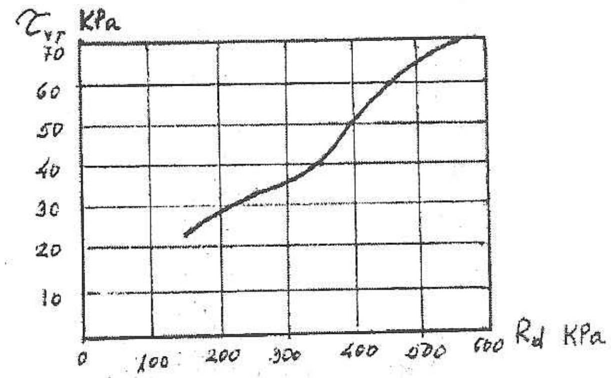


Fig.7 Variation R_d with τ_{vd}

- ➢ From the field exploration realized in the trial pits (for the place of passenger steamers, and merchant ships)until 3,5m depth result that:
- The first layer (0-1,9)m is classified in the A1-a and A1-b (fig.9) group which arrive:

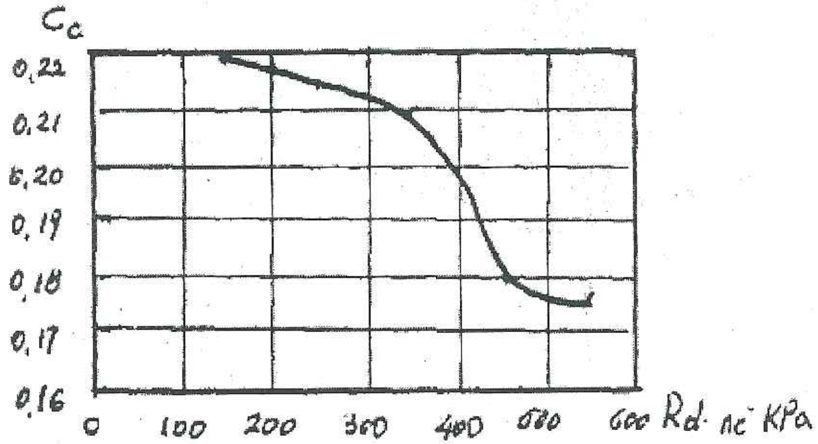


Fig.8 Variation R_d with C_C

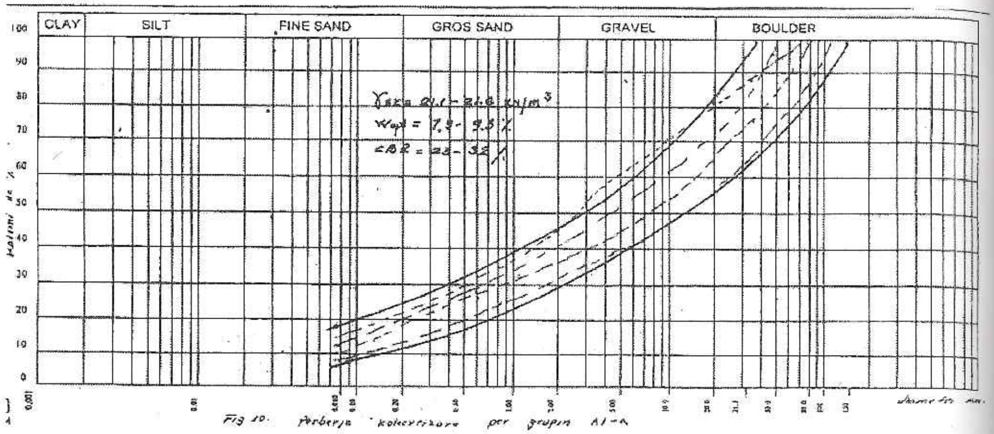


Fig.9 Particle –size distributions of A1-a group

- $\gamma_{d,max}$ = 21-21,6KN/m^3 –dry density
- W_{opt} =7,8-9,1% optimum moisture content
- PI≅0 plasticity index
- CBR = 20-30% Californian bearing ratio
- M_d =(2,5-4,8).10^{2}KPa compression modules from loading plate test.
- M_d/M'_d = 0,2-0,4 M'_d modulus (unloading)
- K_s = 3-10MN/m^3 coefficient of subgrd reaction (coefficient of springs).

- The second layer from 1,5-1,9m to 3,5m is classified in the A4 and A6 group (fig.10). He is arrive:

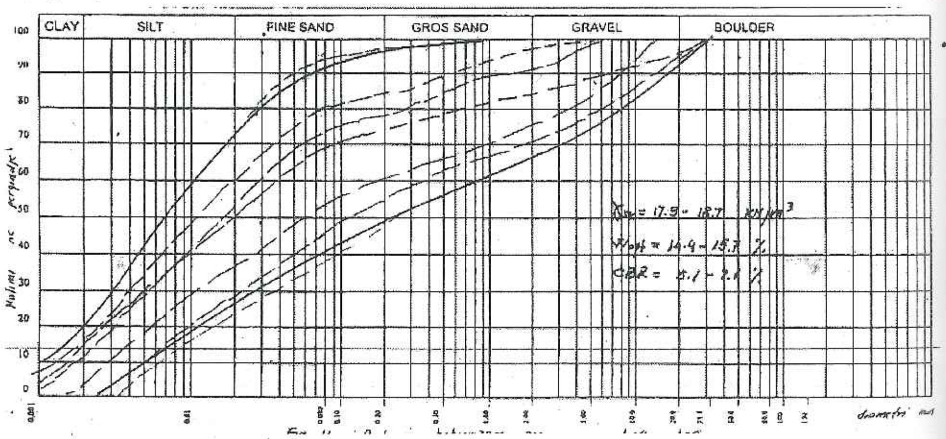


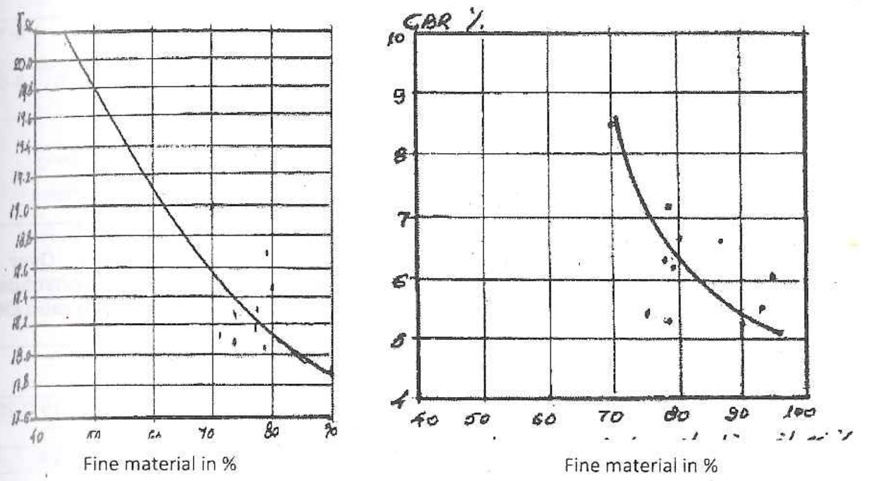
Fig.10 Perticle –size distributions of A4, A6 group

- $\gamma_{d,max}$ = 17,9-19KN/m^3
- W_{opt} =14,3-15,6%
- PI =4-15,8

- W_1 =22-38 liquid limit
- CBR = 5-9%

For the second layer increase of compactage energy from 25 blows to 56 blows) increase (5-6)% the degree of compactage, (40-60)% the value of CBR and decrease the swelling coefficient 2 times.

- Degree of compactage and CBR are depending from the percentage of fine materials (passing No.200) which consist (50-60)% silty particles (fig.11).



a) b)

Fig.11

a) Variation of γ_d with percentage of passing Nr.200 sieve.

b) Variation of CBR with percentage of passing Nr.200 sieve

B. Laboratory investigation.

The laboratory tests are realized for:

- ➢ Classification of soils

From many tests we can to present in the table 1. the generalized properties of different layers.

- ➢ Determination of compressibility of soil.

From one dimensional test realized in undisturbed samples in consolidometer we can to present in the table 2 the generalized compressibility properties of soils.

Table 1 Generalized properties of soils

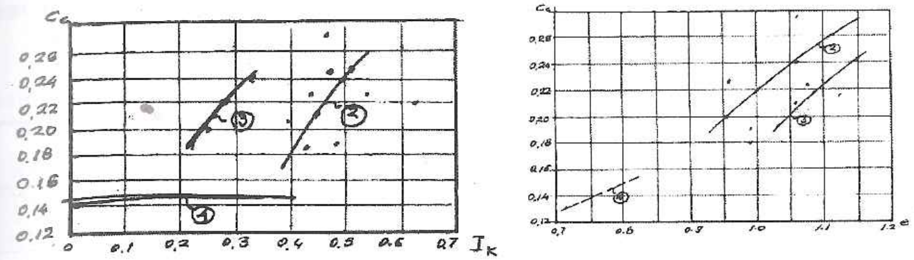
Layer	Class I fiction	Void ratio	Consistent index	Plasticity index	Passing N 200	Liquid limit	Percentage of clays content	thickness
Nr	UCS	e	I_c	PI	%	W_c	%	m
2	CL	1,17-0,81	0,45-0,55	22-30	96-99	48-62	25-40	10-26
3	CL and CH	0,78-0,91	0,26-0,42	21,8-26	95-97	38-50	28-35	5-10
4	CL	0,71-0,88	0,04-0,25	12,3-21,8	83-90	31-44	25-35	5-10
5	CL and ML	0,77-1,14	0,22-0,5	19,3-27,3	95-98	45-59	32-4-	4-6

Table 2 Generalized compressibility properties of soils

Layer	Compression Index C_c	Swell Index C_s	Modulus of deformation E	Coefficient of consolidation C_v	Permeability K	Over consolidation ratio OCR	Rati[illegible] C_s/C[illegible]
Nr			KPa	Cm^2/s	Cm/s		
2	0,17-0,25	0,008-0,0124	$(0,3-0,56)10^4$	$(1,6-2,1)10^3$	$(6,08-8,4)10^3$	0,119-0,28	0,05[illegible]
3	0,19-0,20	0,0135	$(0,3-0,5)10^4$	$1,7.10^3$	$5,6.10^{-3}$	0,033	0,07[illegible]
4	0,117-0,293	0,0085-0,013	$(0,4-0,77)10^4$	$(0,77-1,6)10^{-3}$	3.10^{-3}	0,097	0,06[illegible]
5	0,154-0,211	0,0071	$(0,35-0,74)10^4$	$1,6.10^{-3}$	$4,5.10^{-3}$	0,11	0,04[illegible]

From one dimensional test results that:

- All layers have high compressibility capacity.
- The calculated permeability of soils from one-dimensional test indicates the presence of high percentage of silt in soils.
- The high content of silt is dangerous for the behaviour of soils under seismic loads
- Some of the correlations between the physical properties and compressibility properties of soil are shown in fig.12, 13.



a) b)

Fig.12. a)Variation of C_c with I_c
b) Variation C_c with e

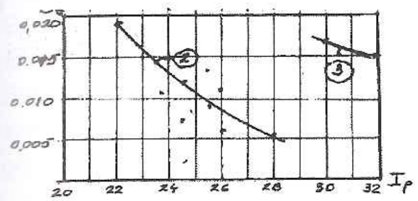


Fig.13 Variation C_α with PI (I_p)

- All the layers are non consolidated OCR<1, that causes important settlement of foundations.
- The small ration C_s/C_r =0,046-0,053 confirms the high percentage of silt in soil and the absence of negatives friction in the piles foundations.

➢ Determination of shear strength of soils.

Shear strength of soils are determined in the: a) direct shear box; b) triaxial test equipment; c) unconfined compression test equipment.

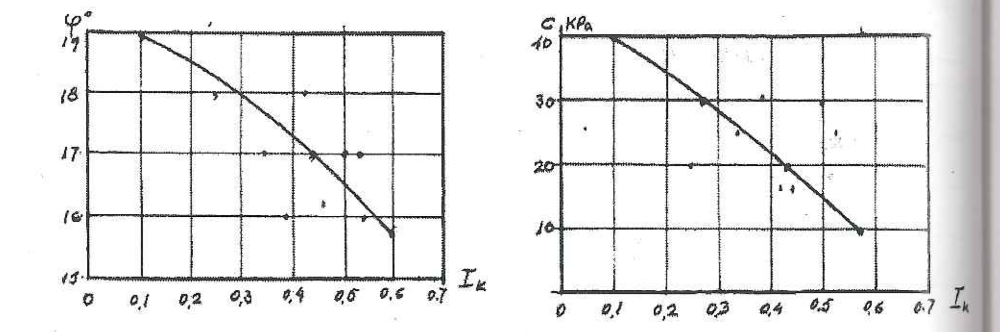
From those tests we can present in the table 3 the generalized resistance parameters of soil such as cohesion C, C_u and angle of friction "φ"

Table 3 Resistant parameters of soils

Layer	Direct shear box		Triaxial equipment			Unconfined compression
			CD		UU	
	φ	C KPa	φ	C KPa	C_u KPa	C_u KPa
2	17	25.35	16.8	23	29	24.5
3	16	28	16	17	38	27.35
4	16	32	15	43	51	31.0
5	16	55	16	53	61	44.8

From this tests result that:

- We can establish tying (exist a correlation) between physics and resistant parameters of soil. They are show in the figures below 14, 15 and 16.



a) b)

Fig.14 a) Variation φ with PI b) Variation C with PI

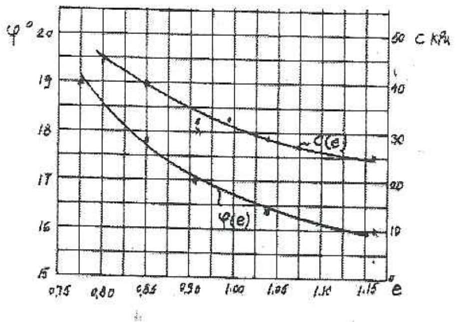


Fig.15 Variation of φ and C with e

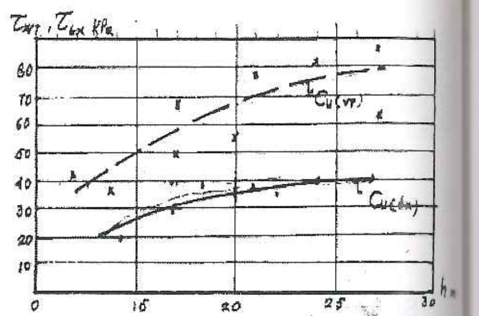


Fig.16 Variation of $C_{u(tx)}$ and $C_{u(VT)}$ with depth

- The ratio $C_{u(tx)}/C_{u(1x)}$ changes from 1,18 to 1,64 ($C_{u(tx)}$ – from test UU in triaxial; $C_{u\,(1x)}$ – from unconfined compression test).
- The comparison value of $C_{u(VT)}$ after Van test in bore holes with $C_{u(1x)}$ – after laboratory test (for second layer) we can shown that $C_{u(VT)}$ increases and after rest constant (fig.16)
- For second layer until 15m depth it can use relation.

$$C_{u(VT)} = (1,8-2)C_{u(1x)}$$

And for depth 15-30m:

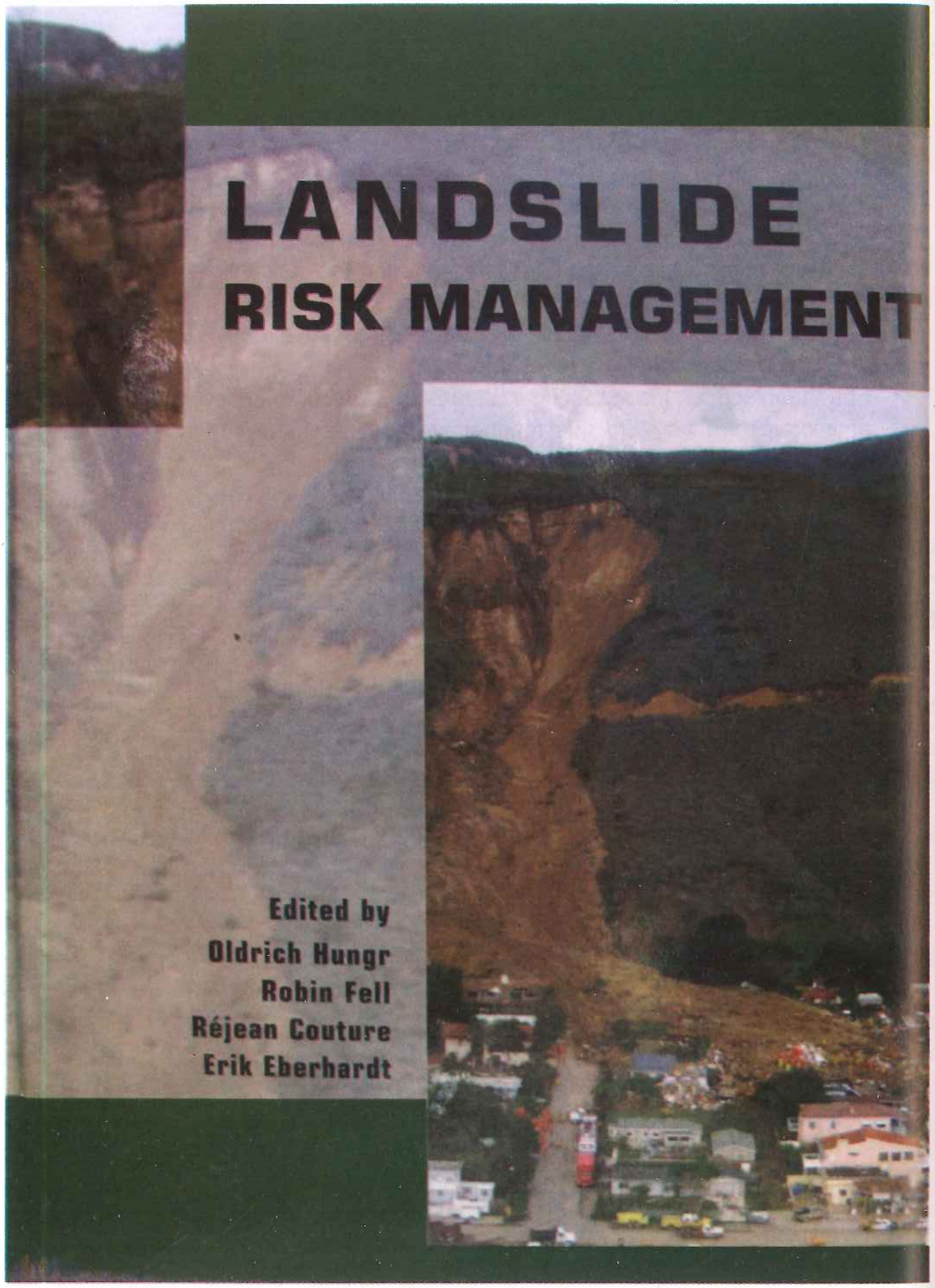
$$C_{u(VT)} = (2,7-3)C_{u(1x)}$$

Conclusions.

- The zone of Durres Port consists from very compressible soils, unconsolidated with weak resistance parameters and problematical in case of earthquake.
- The friction resistance of piles under horizontal loads changes from 19KPa (pile length is 20m) to 25 KPa
- The limit bearing capacity of pile with diameter 50cm, long piles (l=20m) and e/d=0, under horizontal loads changes from 240KN to 300KN.
- The settlement of pile foundations shall be enormous and this is problematic for ultimate limit state and service limit state.
- Process of settlements shall be durable because they are clay soils, unconsolidated with low permeability.
- For the place of the passenger steamers is recommended to improve (to ameliorate) the soils. In this manner we can profit the coefficient of spring greater than 20-25MN/m^3.
- If they use natural soils as elastic basement (no improved soils) for beams or mat foundations is necessary to make their verification for limit service state.

REFERENCE

1. Ahmetaj L Report of ferry terminal port Durres, 2003
   Allkja,S
   Bozo L
2. Bozo L, Sjellja e bazamenteve dhe themeleve ne veprime sizmike. Teze e doktoratures ne filozofi 1989
3. Duni Ll studim inxhiniero-sizmologjik I sheshit te ndertimit te godinave 1-12 kate plazh Durres, 2003



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Landslide risk assesment on road in albania

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Abstract

Albania is neo-tectonic regions and with high seismic intensity. 2/3 of Albania territories are hills and mountainous areas where slope instability is an usual phenomenon. In this material we will discuss about the sliding categorization in Albanian and sliding risk on roads, the primary causes and the technique to avoid these phenomenon.

Introduction

During the last 10-15 year roads and railways in Albania have know a great development. It is constructed a new network of rural and highway roads .During this period the existing road network has undergone enormous damages because of landslide phenomenon and massive deformations of soils. So did happened considerable landslides in the main national roads which connect Albania with neighbour countries as Greece, Macedonia and Kosovo, or in the local roads between different Albanian cities.
In this paper we will evaluate the level of road damages from the sliding events, their causes and also will assess the risk for roads in Albanian territory.

Landslide classification in albanian

Classification of the roads in Albania

Firstly we would like to give classification of the roads in Albania by following criterions (performed by authors)

A. Category of roads
    - National roads are 45% or 3534 km
    - Rural roads are 55% or 4289km

(an average value for all Albanian territory is 0,28km/km^2

B. The height of roads above sea level.

- Mountainous roads (more the 600m height above sea lever) about 32%
- Hilly roads (between 200m and 600m height) about 56%.
- Field roads (under 200m height) about 12%.

C. Seismic intensity of the zone where the roads pass.

- Areas with seismic intensity of 7 degree (MSK-64) that occupies 41% of Albanian territory where pass about 40% of roads.
- Areas with seismic intensity of 8 degree (MSK-64) tha ocupies 41% of Albanian territory where pass about 38% of roads.
- Area with seismic intensity of 9 degree (MSK-64) that occupies 18% of Albanian territory where pass about 22% of roads.

D. Criterion of characteristic phenomenon.

Various damage phenomenons are indentifying in the zones where the national roads pass. These phenomenons depend from the site behaviour of soils and rocks and can be classified as in the table 1.

Table 1.

Zona where passes the roads	Noxious phenomenon	Event frequency in year period.
Field zone (type of soil) - Ex marsh saturated, fine sands, - or silt sands	Non uniform and enormous deformations.	2-3
Over 50% silt	Liquefaction from movements	1 per 25-30 years
Near the river	Destruction of structure from vibrations.	1-2
	Danger of inundate, partial destruction	1-2
Hills zone - Near the river	Erosion, slide destruction of roads.	2-3
- Hills	Small, medium and great slides	10,4,2
Mountatins zone. -calcareous rock	Subsidence of the earth (cars tic phenomena's).	Very rarely
- Mountains	Development of fissure, rolling of rocks mass.	5-10
-main system of faults	Slide after determined surface	2-3

Based on this classification we van achieve the conclusion that (20-25%) of roads in Albania are at risk by sliding phenomenon and great deformations of soil masses in the case of seismic movements.

Classification of landslides

From the different studies (Bozo, Muceku, ect) results that the soil slides in the instability zones (Albanian territory)can be divided based on:

A. Volume of the sliding mass:

- Small slide (until 1000m^3) which represent about (40-50)% of total slides.
- Middle slides (until 10000m^3) which represent about (30-40)% of the total slides.
- Big slides (until 1 million m^3) which represent about (2-3)% the total slides.

B. Depth of the sliding plane:

- About 40-50% of the total slides have a sliding plane 5m deep from the surface and they happen with a height frequency.
- About 30-40% of the total slides have a sliding plane (5-10)m deep and happen with a medium frequency.
- About (10-20) % of the total slides have a sliding plane more then 10m deep from and they happen rarely.

C. Inclination angle of slope.

- About 20-25% of the total slides have an angle of inclination (10^0-15^0).
- About 30-40% of the total slides have an angle 15^0-35^0.
- About (50-60)% of the total slides have an angle >35^0.

D. Geology and tectonics of the zone where the road pass.

Tectonically, Albania comprises inner zones, outer zones as well as depressions.
The inner zones are:

- The Korabi zone, with Ordovician-Devonian such as argyle-silica slates, hyalite, marbled limestone's, quartzite's Permian-Triassic evaporate (gypsum, anhydrite, salts) and Triassic volcano-sedimentary rocks.
- The Gashi zone with Ordovician-Devonian metamorphic deposits, carboniferous-Permian sandstone, conglomerate, Carbon-Permian limestone and marl, few Triassic volcano-sedimentary rocks, Triassic platform-pelagic carbonates as well as Cretaceous granite-diorite.
- The Mirdita zone consists of the Jurassic amphibolites such as intrusive (ultra basic), volcanogenic and volcano-sedimentary rocks, Triassic and Cretaceous limestone's and dolomites. In the region there are also intermountain us depressions of Burreli and Mokra.

The outer zones comprise:

- The Albanian Alps zone consist generally of strongly certified Mesozoic limestones and dolomites.

- The Krast-Cukali zone consists mainly of the Cretaceous-Paleocene flitch and some Cretaceous of Triassic-Cretaceous limestone's structures.
- The Ionian and Kruja zone is characterized by the presence of longitudinal structures with a NW-SE extension. The anticlines consist of Mesozoic and Paleocene carbonates rocks, while the synclines are filled with the Paleocene and Neocene flitch and Quaternary soils deposits.
- Sazani zone, with Cretaceous-Oligocene carbonates and Aquitaine flitch.
- Albanian inner depression, with Burdigalian-quaternary molasses. In this region there are Intermountain with depressions of Burreli and Mokra.
- Tortonian-Miocene Molasses of pre Adriatic depression.

The studied area is extremely affected by the active slides, where many roads are demolished, others are damages by presence of open cracks and differential settlements of its surface have occurred.

The landlides occurrences on traffic road and their spreading point out that, their activity in Albania is closed related to:

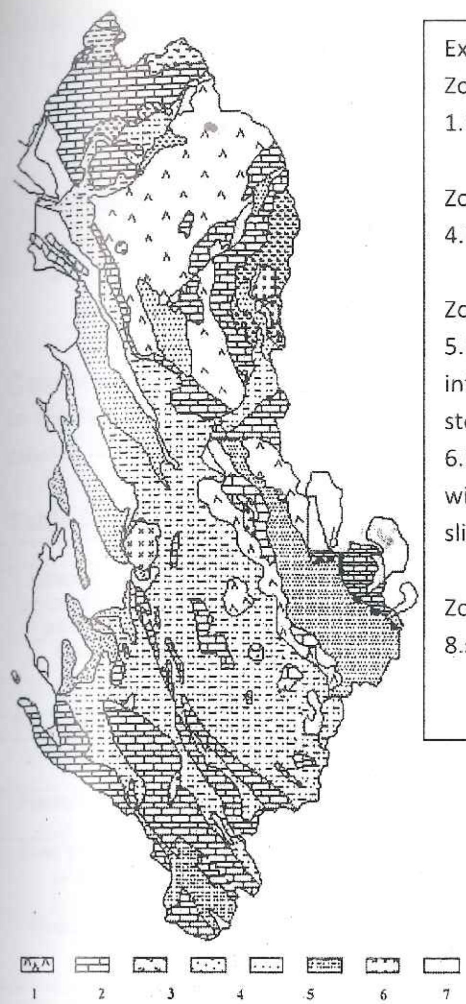
- Geomorphology
- Geological formation
- Geotechnical properties
- Neo tectonics active faults
- Carst phenomenon
- Rainfall events
- Man made works.

E. Geomorphology of the zone where the road pass on Gashi, Alps, Mirdita, Korabi, Cukali and Sazani zones represents a by mountainous terrain with slope inclination varies from 20-30^0 up to 50-75^0, on which a lot of stones and debris are situated on unstable condition. As above-mentioned, mostly of the mountains areas are generally built from hard up to very hard rocks. Therefore the roads, which extend in this zone, are subjected to rocks falls, rocks rrolling and debris slides.

The Krasta, Ionian and Kruja zones, as well as Albanian inner depression-molasses and Tortonian Miocene molasses of pre-Adriatic depression zones are a hilly terrain with gentle to moderate slope inclination.

They are constructed from medium to soft rocks and covered with soils (diluvium, eluvium's)

The roads on these zones are affected by rocks slide, earth slide and earth flow (fig.1)



Explanations:
Zone of hard rocks:
1.Magmatic 2.Limestone 3.slate

Zone of intermediate strength rocks
4.Molasses – conglomerate and sandstone.

Zone of medium – soft strength rocks
5.Molasses-conglomerates and sandstones intercalated with clay stone and siltstone layers, clay stones and siltstones.
6.Flitch –conglomerates and sandstones intercalated with clay stone and siltstone layers, clay stones and slistones 7.Evaporate rocks-gypsum.

Zone have cohesive and cohesion less soils
8.siklts, clays, sands and grevels.

Fig.1 Geotechnical schematic map of Albania (after Konomi 2002)

By the geomorphology point of view, the western part of Albania is a flat area, which is generally built of alluvial soil, and less marshy and marine soils. There are some places on this area consist of peat.

The roads, which are situated on these places, are destroyed each 2-3 year from settlement phenomenon, for instance Velipoje (Shkoder), Gjiri I Lalzit (Durres), Divjake (Lushnje) etc.

Of course, other factors influences the occurrence of landslide are neotectonics active faults and seismicity.

Along active faults are mostly concentrated the earthquake epicenter. After Aliaj, 1998 and Muço , 2001, three main longitudinal seismically active zones, and three main transversal ones, are indentified (fig.2)

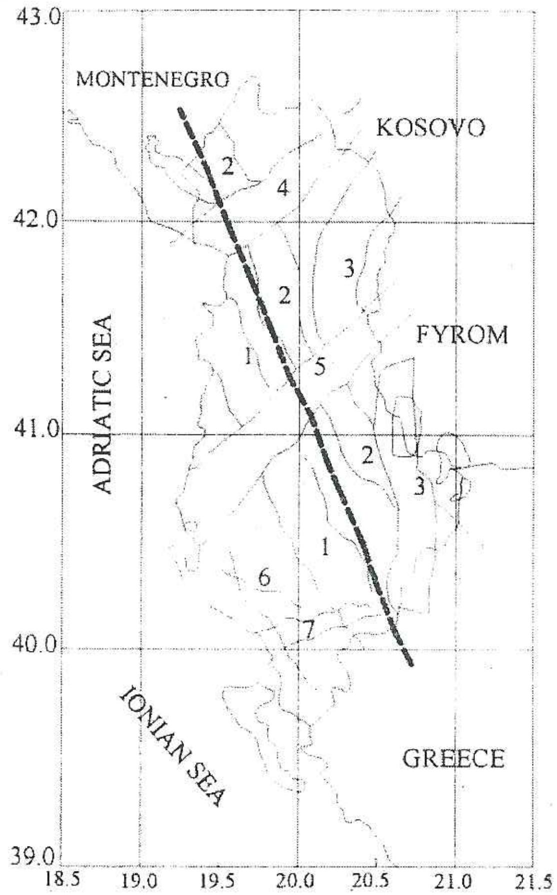


Fig.2 The main seismically active zones in Albania (after Aliaj 1998). The noted numbers are explained in the text.

The Ionian – Adriatic longitudinal seismic tectonic zone, which marks the boundary between the Adriatic micro plate and the Albanian oregano;

The Mat-Moker-Bilisht longitudinal seismic zone;
The Drini-Ohri-Korça seismic tectonic zone.
The Shkoder –Peja transversal seismogenic zone
The Lushnja-Elbasan-Diber, transversal seismogenic zone.
The Vlora-Tepelena transversal seismic faults.

F. Importance of road.

- Very important roads (as highways) where we meet about (2-5)% of the slides. So, in those roads the sliding phenomenon may consider very rarely.
- Important roads (as national roads) where happen (40-50)% of the total slides. So in those roads the sliding phenomenon is usual.
- Slight important roads (as rural roads) where happen 20-30% of the total slides. So in those roads the sliding phenomenon may consider almost usual.
- Unimportant roads (as mine and forest roads) where happen about (10-15%) of the total slides and this phenomenon may consider as rarely.

Causes of landslides and their classification.
In the territory of Albania we meet too much active slides, especially in roads and their causes are different. Authors have classified these causes as below (table 2)

Table 2

Causes	Type of slides	Frequency
Change of climatic conditions (temperature rains, snow etc, and weathering phenomena's)	Superficial	Very frequently, often
Erosion activity of rivers	Middle Great Very great	Frequently(Often) Medium Rarely
Abrasion activity of sea	Detaching of rock masses	rarely
Man made activity	Small Middle	Medium rarely
Presence of weak or feeble plan (surface)	Small Middle	Often medium
Over coal mines or in petroleum or gases contains zones	Surface subsidence Great subsidence Very great subsidence	Medium Medium Rarely
Road passing over diluvium's	Small Middle Great	Frequently (often) Medium Rarely
Disorder of water rate	Small Middle great	Frequently (often) Medium Rarely
Earthquake	Small Middle Great	Medium Rarely Very rarely
	Note: frequently = 10 event in year Medium ≥ 5 event in year Rarely ≤ 1 event in year	

The methods used to determinate slide risk on roads.

Mechanism of failure.
As a conclusion of many and complex (geological , geophysical and geotechnical), studies the slides in Albanian mostly have the following mechanisms of sliding.:

- Slides according to a definite plan.
- Slides according to a circular plan.
- Rolling slides.
- Creeping slides.
- Muddy flow.

Method of analysis.

During our experience we have consider two types of slope stability

- Analysis of slope stability where is not manifested the sliding phenomenon but the possibility to appear exist. In this case the critical surface of sliding and its localization in site is impossible. This is a part of design the slope analysis. In cases where the slopes result unstable are recommended the engineering measurements for their stabilization.
- Analyses of slope stability where the failure has started. In this case the failure surface in site is known and the factor of safety is considered ≤1.

From this type of analysis we get information about causes of instability, about reduction of the soil resistance parameters (soil shear strength), for development of pore-water pressure and about failure mechanism.

The analysis are based on the methods of limit equilibrium where the failure surface is considered circular (tor circle, slope circle or midpoint circle) but in some case this surface is taken according to a definite plan, which is localized in site. Some of them are mention below:

A. Granular soils:

- The c' = 0 method.

B. Cohesive soils (circular failure surface)

- Metodh of slices
- Bishop's simplified methods of slices.
- Spencer's method.
- Morgenstern and Prince method (non circular failure surface)
- Janbu's method
- Culmann's method
- Infinite slope method.

The analyses are realized for static and dynamic conditions, for a 2-dimensional geometry of slope and for long-term and short-term soil resistant parameters. It is nor usual to as designing a 3-dimensional slope.

Our codes ask a value F≥1,5 of factors of safety for stable condition of slope.

Shear strength of soil.

Shear strength of soil masses are determined from the laboratory and in site test.

Some laboratory tests, used mostly in our practice are mentioned below:

- Direct shear box (with consolidated and unconsolidated samples).
- Triaxial test (CD, CU and UU tests).
- Unconfined compression test.

Some of the on-site tests, mostly used in our practice are mentioned below:

- Dynamic penetrometer test.
- Van shear test
- Standard penetrometer test (SPT)
- Velocity V_s
- Pocket penetrometer.

From these tests results that in the existing surface failure the values of φ and c are in these intervals:

$\varphi = 17^0 - 19^0$

C = 5-25KPa

And, for the undisturbed soils in zones which have risk of landslide we have these intervals:

$\varphi = 20^0 - 24^0$

C = 15-40KPa

The difference of φ and c values from the different test applied in different cases it is not more than $2^0 - 3^0$ degrees and 3-7KPa for c.

The value of φ and c are reliable with a probability coefficient 0,85 for small and middle landslides and α=0,95 for big slides.

Influence of the water in rock and soil masses.

From our investigations for this problem results:

- The superficial water is one of important factor for slope instability in hilly and mountainous zones. At the time of rainy weather occur about 50% of slides in Albania.
- The presence of water table and development of high pore-water pressure is a major factor, which contributes to slope instability. In slope this water table is verified using horizontal or inclined bores. In Albania (10-15)% of the total failure of slope is dedicate to the ground water.
- The erosive activity of the rivers and disorder of equilibrium at the base of slope is a very important factor, which causes failures and instability in Albania (35-45)%.

Seismic design parameters.

According to seismic map of Albania and the Albanian "Design Seismic Code KTP-2-'89 edited by Seismological Institute of Tirana and Construction Ministry result that all the project aria is evaluated with intensity 7-9 degree (MSK 64), (fig.3). the influence of local ground conditions on the seismic action shall be accounted for by three subsoil categories.

From seismological studies the epicenter depth of earthquake in Albania are 30-70km (fig.4)

From our studies results that:

- In zones with intensity movement VII degree (MSK 64) the frequency of the earthquake is 1 event for 5 years and in these zones are present small and middle slides.
- In zones with intensity VIII degree (MSK 64) or M≥5,8 the frequency of earthquake it is 1 event in 10-15 years and therein, it can occur medium and great slides.
- In zones with intensity IX degree (MSK 64) or M>6,5 the frequency of earthquake it is 1 event in 30-35 years and in this zones can occur great slides.

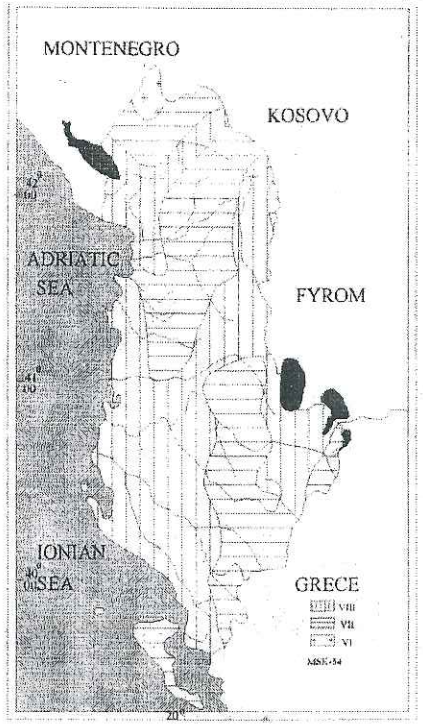


Fig.3 Seismic map in Albania in the scale 1:500000 (after Sulstarova et al.1980

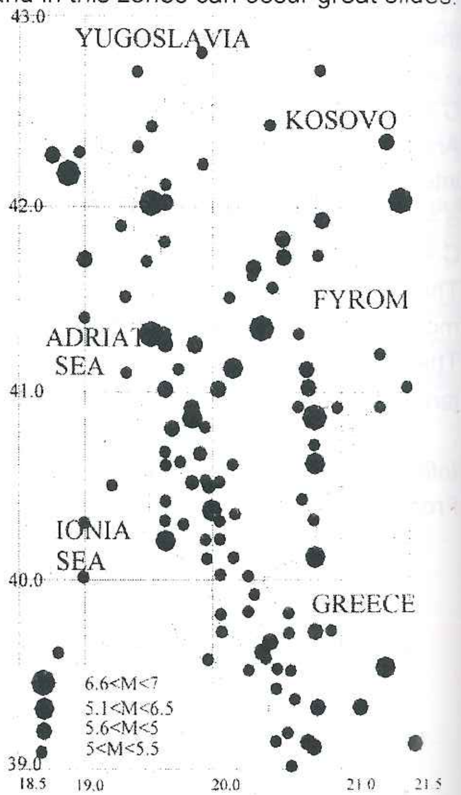


Fig.4 The epicenters of earthquakes in Albania with M>5,0 for the 20th century (After Muco 2001)

Risk. Evaluation and determination of dangerous zones.

It is determined for common slides the following probability.

- Small and medium slides 1-2 event In year.
- Great and very great slides event in 10 years.

After many investigations, observations, studies and statistical elaborations of geological and geotechnical based also at the seismic data the authors have determined in Albania the map of the most dangerous slide zones.

Conclusions

As a conclusion 20-25% of road network in Albania are at risk from slides phenomenon and great deformation of soils during earthquake.

Location of highway on the futures requires knowledge of the regional geology combined with geological and seismic studied. Also, the geotechnical data are required to realize stability analysis and to prevent the engineering remedial measures.

The corrective measures more used in our practice until now and suggested from the authors for the future are:

- Drainage methods and systematization of superficial waters.
- Planting of the slope surface.
- Retaining walls
- Soil anchors
- Pile reinforcement
- Nailed metallic meshes.

REFERENCES

Aliaj Sh. (1998) The geotechnic structure of Albania. Alb.J.Nat.Tech.Sci.4, p.79-98

Bozo L (2000) Rocks mechanic. Civil Engineering Faculty Tirana, Albania

Bozo L (2002) Basements of religious objects in Albania. Civil Engineering Faculty of Tirana, Albania

Bozo L (2004) Soil Dynamics. Civil Engineering Faculty of Tirana, Albania

Konomi N (1997) Geotechnical map of Albania on scale 1:200000. Polytechnic University of Tirana, Albania

Muceku J (2003) Geotechnical map of Tirana-Durresi-Kavaja region on scale 1:25000. Geological Survey of Albania

Muçco B (2001) Seismic zonation in Albania Using deterministic Approach. Inter.J of geotechnics, Geology and Physic of the Interior of earth, Elsevier p.277-288

Sulstarova S (1980) Seismic Zonation map of Albania

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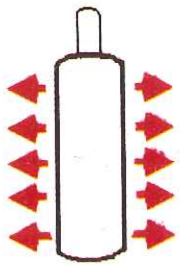

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Rapport national:
Histoire de pressiometres est leur usage en etudes et calculation geotechnique in Albanie

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Résumé

Dans ce matériau nous voulons présenter, en courte terme, le développement des recherché et des études géotechnique in Albanie après le dixième guerre mondiale, jusque à présente, aussi nous voulons parler pour les essais pressiomètrique réalisé in Albanie et utilisation de leur résultats en calcul géotechnique, de même exposer les essais "in situ" développé a dernière période chez nous.

Introduction

Après de la deuxième guerre mondiale, et surtourt après des ans 50, in Albanie il avait une bonne législation en champ de la construction, qui était perfectionné continuellement. At le période 1950-1960 tout les études et les recherches géotechnique on faite dal spécialistes étrangers en collaboration avec le technicien albanaise. Plus tard (1961-1975) cette études on faite dal spécialiste albanaise en collaboration avec les spécialiste chinoise et après le 1975 seulement dal notre ingénieurs.
Durant cette période (1983-1989) ont commencé dans le pratique des études géotechnique les essais pressiomètrique, mais malheureusement cet type d'essais n'a pas longé vite parce que est venu le tempe de changement, du transformation de la système socialiste à capitaliste que il a eté accompagné avec considérable destruction dans tous secteur de la vie. Après le 1997 on commencé la période de réhabilitation et de développement de toutes secteureu donc et de la secteur géotechnique (le recherche, les études, les laboratoires, les essais on place ect).

Bref histoire de développement du secteur géotechnique en Albanie.

Avant du deuxième guerre mondiale les études géotechniques ont réalise seulement par l' étranger entreprises et pour leur intérêt, telle comme italiane, autrichien, français ect pour le zone pétrolifère, pour l' infrastructure routière, pour le ponts ect. Après de la guerre 1944-1957 toute les études géologique e géotechnique (parce que manqué les spécialistes albanais) on faite par les équipes de pays socialiste comme:

L'Union Soviétique pour les ouvrages hidroenergitique et industrielle.
Le Bulgarie pour les ouvrages hidrotechnique en agriculture.
Le Tcheqoslovaki por les ouvrages industrielle e bâtiments.
Le Pologne pour les ouvrages portuelle ect.

In 1957 il était crée le premier Institut pour le recherché géologique-géotechnique e géodésique in Tirana dans le quelle ont concentré tous les ingénieurs spécialisé in Albanie et à l'étranger, toute les outillage et appareillages laboratoriques et les outils de forage ect Cette centre était unique en Albanie et elle était puisse chaque année en faisant possible de fournir les études géologiques et géotechniques pour tout le branche d'économie.

De cette manière ont faite possible la projection et la construction :

- Des grandes ouvrages hidroenergitiques (tous le réseau de hydrocentrales).
- Des ouvrages d'el industrie minérale, du naphte, mécanique, textile, alimentaire.
- Des ouvrages de l'irrigation et bonification, le création de beaucoup réservoirs (plus de 600) pour l' agriculture.
- Des ouverges de l'infrastructure routière et ferroviaire.
- Des ouvrages portuelle
- Les tunnels avec et sans pression ect.(pour autoroute et por le hidrocentrales)

Durant le période 1957-1990 le Laboratoire Central du Mécanique des Sols e de Mécanique des Roches ont enrichir avec le personnel et les appareillages. Dans cet laboratoire ont fait toute les essaies pour le classification des sols e des roches, pour déterminer le leur comportement (les essaies eodometrique, taille direct, triaxial, écrasement un axial, du perméabilité, du suffusion ect).
Parallèlement ont comancé les essais en place comme les essais SPT, BP, PMT.
Cette centre ont fourni avec les plusieurs donné l'géotechnique précis et sûr pour le calcul géotechnique de toute les ouvrages susmentionné.
Après le 1990, le période de changement du system économique et politique à notre pays, était accompagné avec une anarchie dans toute secteures ainsi en secteur géotechnique. A cette manière être endommage grave le Laboratoire Central de Mécaniques des Sols et des Roches, ont faite une dissolution du spécialiste, une part sont parti à l'entranger et l'Institut se désagrège en rester un secteur avec 5-6 personés. A cette période seulement le laboratoire géotechnique de Faculté de Génie Civil et aussi de Géologie e Mine de même le Laboratoire privé ALTEA sh.p.k ont assuré les donné pour le calcul géotechnique.

Après de 1997 le secteur privé et le secteur d'Etat ont commence se développer et maintenant en Albanie il a quelques laboratoires géotechnique mais le meilleur et le plus complété est le laboratoire ALTEA d'entreprise privé ALTE & GEOSTUDIO 2000 sh.p.k. Dans cet laboratoire ont faite tout les essais pour le sols (echantillon intacte et avec le structure reconstitué) les essais en place SPT, DP, FVT, PLT, CBR ect.
Aujourd hui en Albanie s'accomplir les études complexes géologique et géotechnique ou l'investiture sont le Banque Mondiale et Européenne, aussi les investiture puissante albanais.
Déjà sont réalisé les études pour:

- L'infrastructure routière (construction des routes de premier catégorie, routes nationale, routes rurale ect).
- La construction des points.
- La construction des nouveaux ouvrages industrielle.
- Le bâtiment de plus de 10 étages
- La reconstruction des ouvŕage portuelle (Durres, Vlora, Shengjin).
- Le tunnels, ouvrage hydrotechnique ect.

Les essais pressiomètrique en Albanie.

Le pressiomètre Menard est apporté en Albanie pour fournir avec les paramètres géotechnique le calcul géotechnique pour nouveau port de Vlora à 1980-1981.
Il a été impossible déterminer cette paramètres par les autres méthodes parce que le terrain a été constitué par le sable fine, limon sable, argile limon, donc les sols été très molle.
A cette manière pour la recherche préliminaire pour le dimensionnement des éléments de la construction il était utilisé le méthode pressiomètrique. Nous avons utilisé deux types de pressiomètre Menard:

AX avec diamètre 44mm et diamètre du forage (46-52)mm.
BX avec diamètre 55mm et diamètre de forage 60-62mm.
Les essais on faite (ont réalisé) à le part d'Est du port de Durres ou nous avons constaté une colonne géologique comme in fig 1, le même ont réalisé à le port de Vlora en mer et an bord (la colonne géologique comme in fig.2), et à quelques ouvrages in Tirana, Korça Shkodra ect.
A le nouveau port du Vlora ont réalisé 40 essais complète pressiomètrique pour intervalle chaque 1m, on 18 essais ont réalise a la mer et 22 à le bord.
Les enregistrements en mer ont montré que parfois il y a quelques anomalie avec le valeur très différent no seulement en divers couches mais et a le même coushe .Selon notre opinion les anomalies ont venu parce que il y a :

- Une compression du tube fondu (lézardé) durant le forage.
- Une remplissage avec matériau du forage de la zone entre le tube et la membrane.

Les enregistrements en bord ils furent plus normal. Les valeurs de module pressiométrique ont été:

E_p = 90 – 100 bars en sable fine et saturé
E_p = 70 – 90 bars en sable argileuse
E_p = 50 – 52 bars en argile sableuse.

D'après les donné enregistré pour cinque coushes lithologique plus caractéristique dans le zone du port de Vlora nous avons les valeurs que sont présenté an tableaux 1.

Tableau 1

Type de coushe	Poid vol γ gr/cm³	Poid vol seche γ_{sk} gr/cm³	L'angle de frotement φ^0	Le cohesion C kg/cm²	Modul pressiometrique E_p kg/cm²	Le capacite portante Kg/cm²	Le presion limit P_e kg/cm²
Sable limonese en etat suspensive	-	-	-	-	-	-	-
Suble finne sature 0,33<D<0,66	1.79	1.53	26	0.026	10	1.5	2.5
Sable finne et limon, peu dense et saturé	1.96	1.53	18-20	0.15	90	1.2	2.0
Sable finne e sable argileus saturé	1.95	1.58	15-20	0.10	65	1.5	1.5
Argile sableuse legere et plastique	1.94	1.53	8-12	0.20	110	1.0	1.5

Les donné l'après les essais eudiométrique en laboratoire ils sont diverse par rapport des essais presiomètrique:

$$E_{od} \neq E_p$$

Aussi le capacité portante calcule après les essais in laboratoire est différente à le capacité portante calculé après les essai pressiométrique $R'=P_1/3$ et le rapport entre leur et:

$$\frac{R'}{R} = 1{,}2 \div 1{,}25$$

Mais le court temps d'exploitation du pressiométrie en Albanie ne permettre pas de faire les conclusions sûr et tirer les corrélations entre le paramètres physiques et mécanique des sols, tirer les corrélations entre les essais en place et en laboratoire.

Conclusions

- Les essais pressiométrique comme essais en place pour le cas des sols (mou) faible son irremplaçable.
- Parallèlement les autres méthodes en place ils servissent pour déterminer les paramètres géotechnique sûr que s'empoissent pour le calcul géotechnique.
- Les essais en place sont indispensable pour fournir avec les donné l'géotechnique nécessaires pour le calcul géotechnique selon le EC-7

REFERENCES:

1.Bozo.L 2001 Sellja e themeleve me pilota ne ngarkim statik e dinamik. Seminar.Tirane

2.Bozo L. 2002 Njohje dhe zberthimi I EC-7. Seminar.Tirane.Metodat ne terren.

3.Cestari F 1990 Prove Geotechniche in situ (ed 2). Geograph s.M.C Segrati (Mi)

4.Gambin M 1980 Ariwiew of the pressiometres on the last twenty years in Europe. Sel-Soilss Nr.12

5.Zeqo A 1974 Roli e rendesia e studimeve komplekse gjeologo-inxhinierike e gjeologo inxhinierike ne projektimin e ndertimin e veprave hidroenergjitike ne Shqiperi. Buletini Shkencor NGJGJ Nr.4

6.Zeqo A 1997 Studim gjeologo-inxhinierik e gjeoteknik per portin e ri detar te Vlores (faza e projekt idese se pergjithshme). kes.

7.Zeqo A. Kero.J 1997 Perdorimi I metodave in situ ne Shqiperi per percaktimin e vetive mekanik te truallit per projektimin e objekteve te inxhinierise se ndertimit.Referat ne seminar "Arritje, probleme dhe perspektiva ne zhvillimin e gjeoteknikes.

8.Zeqo A. Kero.J 1982 Studimi gjeoteknik I kalates lindore te portit te Durresit. NGJGJ Tirane

9.Zeqo A Kero.J Rambi T 1983 Studimi gjeologo-inxhinierik e gjeoteknik I portit te ri te Vlores (faza e projektit teknik) NGJGJ Tirane.

EUROPEAN YOUNG GEOTECHNICAL ENGINEERS CONFERENCE

ZAGREB - CROATIA

COEFICIENT OF SUB GRADE REACTION.

M.sc.Eng Xhevahir ALIU
M.sc.Eng.Mentor BALILAJ

I. General knowledge of coefficient of sub grade reaction K_s

What's the coefficient of sub grade reaction (general Knowledge)
It's depence
Where use it.

II. Determination of K_s with different method test.

From:
"Plate Load" test
"CBR" test
"SPT" test.

III. Structural design of mat foundations
(general Knowledge)

IV. Results

Relation of K_s from cylindrical length of slab foundation
Relation of K_s from Young's modulus of soil
Relation of K_s from moments in a given direction.

I. General knowledge of coefficient of sub grade reaction K_s

What's the coefficient of sub grade reaction (general Knowledge)

Coefficient of sub grade reaction is one parameter that characterize the soil basis for flexible foundation design. In the approximate flexible method of design, the soil is assumed to be equivalent to an iunfinite number of elastic springs. The determination of this coefficient can be expressed as ratio of:

$$K_s = \frac{\Delta P}{\Delta S} \quad N/mm^3 \qquad (1)$$

ΔP – is change of pressure (load N/mm^2).
ΔS – is change of the settlement by this pressure (mm)

It's depence

Tha value of the coeficient of sub grade reaction is not a constant for a given soil. It depends on several factors, such as:

- From foundation dimensions
- Load
- Structure of the soil (composition)
- The specification and nature of soil,

From foundation dimensions

Exist one rapport between the coefficient of sub grade modulus for one square foundations by coefficient of sub grade modulus for one rectangular foundations.
For rectangular foundations having dimensions of BxL (for similar soil and q-load), can be expressed as:

$$K_{(BxL)} = \frac{K_{(BxL)} \cdot \left(\frac{1+B}{L} \right)}{1,5} \left(kN / m^3 \right) \qquad (2)$$

Where:
B- is foundation width
L- is foundation length
$K_{(BxB)}$ – coefficient of sub grade modulus of a square foundation having dimension of BxB.
$K_{(BxL)}$ – coefficient of sub grade modulus of a square foundation having dimension of BxL.
When:

$$\frac{B}{L} < 0,5 \Rightarrow K_{(BxL)} < K_{(BxB)} \ldots B = C$$

$$\frac{B}{L} < 0,5 \Rightarrow K_{(BxL)} < K_{(BxB)} \ldots B = C$$

If $\frac{B}{L} = 0$ (L>>>>>B) the equation (2) can be expressed.

$$K_{(BxL)} = 0,65.K_{(BxL)}$$

Load

From equation (1) by static plate load of test, from load of coefficient of sub grade reactiv for one constant settlement.

$$K_s = \frac{\Delta P}{\Delta S} \quad (N / mm^3; kN / m^3) \qquad (1)$$

ΔP – is change of pressure (load N/mm^2)
ΔS – is change of the settlement by this pressure (mm)
This signify that: as more is load, so as more is the coefficient of sub grade reaction for one constant settlement.

Structure of the soil (composition)

From laboratory test is survey relation between coefficient of sub grade reaction and structure of soil.
Coefficient of the sub grade is small for soil low granule (Clays soil).
Coefficient of the sub grade is large for soil with high granule. (Sandy and grevel soil).

The specification and nature of soil.

Following are some typical ranges of value for coefficient fo sub grade reaction $K_{(0,3)}$ for sandy and clayey soil.
Where:
$K_{(0,3)}$ – is coefficient of sub grade from Static Load Test by square plate with dimensions 0,3mx0,3m.
Porosity

$K_{(0,3)}$ – 8-25 (MN/m^3)	Loose Sand
$K_{(0,3)}$ – 25-125(MN/m^3).	Medium Sand
$K_{(0,3)}$ – 125-375 (MN/m^3).	Dense sand saturation.
$K_{(0,3)}$ – 10-15 (MN/m^3).	Loose Sand
$K_{(0,3)}$ – 35-40 (MN/m^3).	Medium sand
$K_{(0,3)}$ – 130-150 (MN/m^3).	Dense Sand

Stiffness

$K_{(0,3)}$ – 12-25 (MN/m^3). Stiff Clay
$K_{(0,3)}$ – 25-50 (MN/m^3). Very Stiff Clay
$K_{(0,3)}$ – >50 (MN/m^3). Hard Clay

Where use it.

In the approximate flexible method of design, the soil is assumed to be equivalent to an infinite number of classic springs, as show in figure. This is sometimes referred to as the Winkler foundation.
The elastic constant of these assumed springs to as the coefficient of sub grade reaction k.
The coefficient of sub grade reaction is also a very useful parameter in the design of rigid highway and airfield pavements. The pavement with concrete wearing surface is generally referred as a rigid pavement, and the pavement with asphaltic wearing surface is called a flexible pavement.

Now that we have discussed the coefficient of sub grade reaction, we will proceed with the discussion of approximate flexible method of designing mat foundations. This method, as proposed by American Concrete Institute Committee 436 (1966). The design procedure is preliminary based on the theory of plates. It's use allows the effects (that is, moment, shear, and deflection) of a concentrated column load in the area surrounding it to be evaluated.

I. Determination of K_s with different method test.

From:
"Plate Load" test.
A comprehensive study of the parameters affecting the coefficient of sub grade reaction has given by Terzaghi (1955). According to this study, the value of the coefficient of sub grade reaction decreases with the width of the foundation. In the field,load test can be carried out by means of square plates measuring 0,3mx0,3m, and value can be calculate. The value of "k" can be related to large foundations measuring as follows:
Foundation of Sandy Soil:

$$K_{(BxB)} = K_{(0,3)}\left(\frac{.B+0,3}{2B}\right)^2 \left(kN/m^3\right) \qquad (1.1)$$

Foundation on Clays :

$$K_{(BxB)} = K_{(0,3)}\left(\frac{0,3}{B}\right)^2 \left(kN/m^3\right) \qquad (1.2$$

$$K_{(BxL)} = \frac{K_{(BxB)}\cdot\left(\frac{1+B}{L}\right)}{1,5}\left(kN/m^3\right) \qquad (1.3)$$

Where:
B – is foundation width
L – is foundation length
$K_{0,3}$ – coefficient of sub grade modulus of a square foundation having dimension of 0,3mx0,3m
K_{BxB} – coefficient of sub grade modulus of a square foundation having dimension of BxB.
K_{BxL} – coefficient of sub grade modulus of rectangular foundation having.
Vessic (1961), for long beams, proposed an equation for estimation of sub grade reaction that can be expressed as:

$$K_s = 0,65.\sqrt[12]{\frac{E_s B^4}{E_{th} I_{th}}}\cdot\frac{E_s}{\left(1-\upsilon^2\right)B} \quad (kN/m^3) \qquad (1.4)$$

Where:
E_s – Young's modulus of soil
υ - Poisson's ratio of soil
E_{th} – Young's moduls of foundation material.
B – foundation width
I_{th} – moment of inertia of cross section of the foundation.
For most practical purposes Equation (1.4) can be approximated as :

$$K_s = \frac{E_s}{\left(1-\upsilon^2\right)B} \quad (kN/m^3) \qquad (1.5)$$

"CBR" test.

The value of coefficient of sub grade can be related to CBR as follows:

$$K_s = 4,1 + 51,3\log CBR(MN/m^3) \quad (2.1) \quad CBR = 2\text{-}30\%$$

$$K_s = 314,7 + 266,7\log CBR(MN/m^3) \quad (2.2) \quad CBR = 20\text{-}100\%$$

"SPT" Standart penetration test.

(SPT – standart penetration test)
Scott (me 1981) has proposed that for sandy soils, the value of $k_{0,3}$ can be obtained from standard penetration resistance at any depth as :

$K_{0,3}$ =1,8.N (MN/m^3) (3.1)

Where:
N – corrected standard penetration resistance
$K_{0,3}$ – coefficient of sub grade modulus of a square foundation having dimensions of 0,3mx0,3m

I. Structural design of foundations

The structural design of mat foundations can be carried out by two conventional methods:

- The conventional rigid method.
- The approximate flexible method
- The conventional rigid method.

In the conventional rigid method of design, the mat is assumed to be infinitely rigid. Also, the soil pressure is distributed in a straight line, and the centroid of the soil pressure is coincidental with the line of action of the resultant columns loads.

The approximate flexible method.

In the approcimate flexible method of design, the soil is assumed to be equivalent to an infinite number of elastic springs, this is sometimes referred to as the Winkler foundation. The elastic constant of these assumed springs is referred to as the coefficient of sub grade reaction "k" .

Results.

(Note: Next paragraph is one part of the research in on slab foundations of one 8-floor building).

Relation of K_s from cylindrical length of slab foundation.

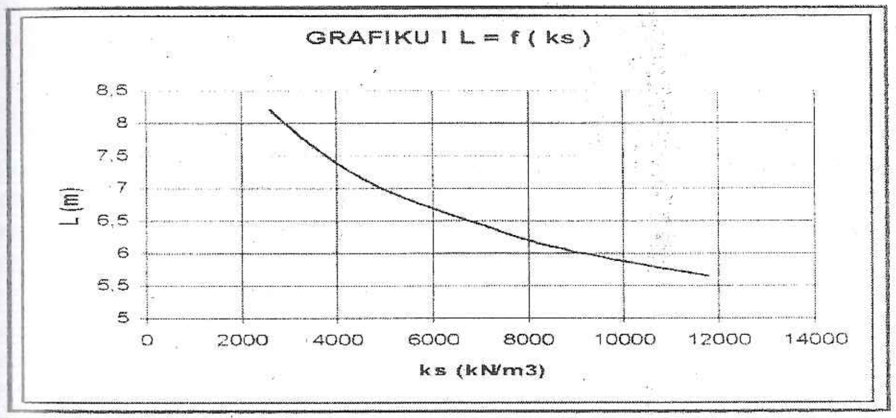
The value of coefficient of sub grade can be related to CBR as follows:

$$L = \sqrt[4]{\frac{D}{K}}(m)$$

Where:
D – 11941026,08 kN.m is the radius of effective stiffness for study slab foundation of 8 –floor builing.
L – cylindrical length of slab foundation
D – radius of effective stiffness of slab foundation.

For

K_s = 2613,016 kN/m^3	L = 8,22m
K_s = 3919,00 kN/m^3	L = 7,43m
K_s = 5226,03 kN/m^3	L = 6,91m
K_s = 7839,05 kN/m^3	L = 6,25m
K_s = 9200,00 kN/m^3	L = 6,00m
K_s = 11758,6 kN/m^3	L = 5,65m



Relation of K_s from moments in a given direction.

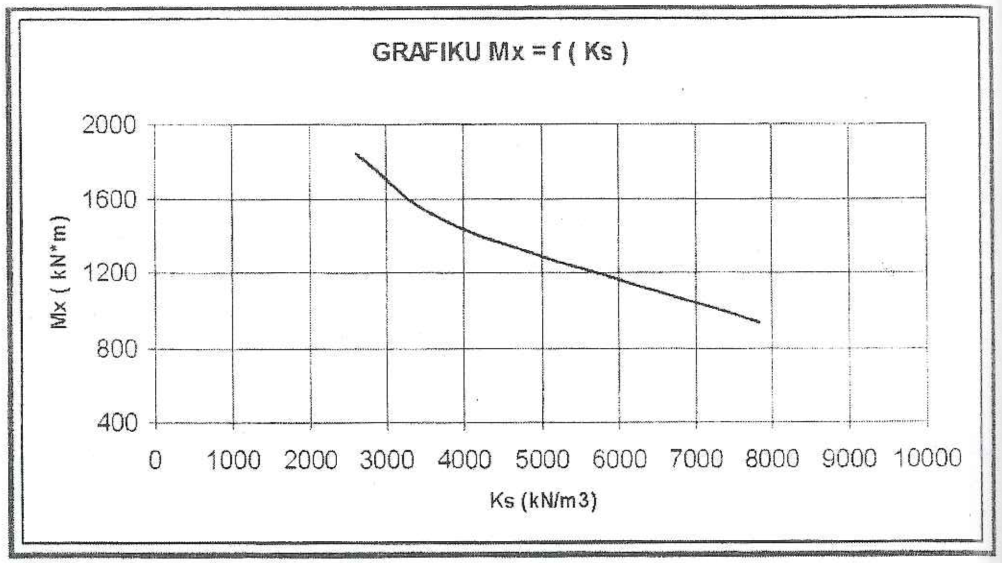
By Bowles Structural Design of Mat Foundation method, moments in two directions to K_s can be related as follow:

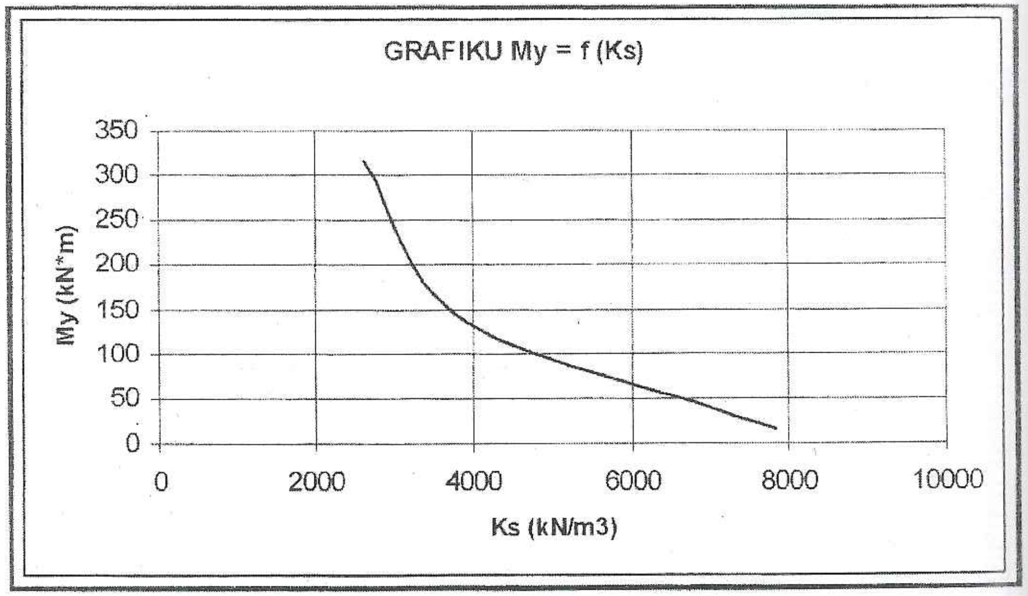
Table 1 : For K_S = **2613,016 kN/ m˘ (L = 8,22m)**

$a_i = r_i / L$	A_1	A_2	N_i (kN)	M_r	M_t	M_x	M_y
0,155794	-0,4	-0,13	3281,147	-242,739	625,321	2861,276	8539,66
0,760511	-0,3	-0,23	3706,76	44,27246	279,8837	1434,303	8225,55
1,123445	-0,12	-0,22	3045,89	-32,9863	139,5315	-944,41	4119,458
1,633073	-0,01	-0,18	2961,56	-60,6563	69,28929	-1793,81	2051,068
1,997506	0,03	-0,05	2584,961	-32,8781	10,27265	-976,716	303,0725
2,362103	0,045	-0,1	3764,51	-75,5796	26,19865	-2247,12	775,571
2,848378	0,05	-0,07	3505,988	-61,7894	10,68964	-1838,8	316,0299
3,505001	0,04	-0,03	2832,838	-33,3838	0,352951	-994,063	9,742813
0,596479	-0,36	-0,23	3089,299	29,66732	294,5237	8448,491	1212,402
0,953883	-0,21	-0,23	3321,791	7,396098	195,9473	2326,093	3733,541
1,262396	-0,08	-0,21	2879,043	-42,276	109,4153	-292,607	2293,357
1,731601	0	-0,17	3506,974	-71,786	71,78598	-1652,67	1652,671
2,078833	0,03	-0,14	2797,74	-60,2676	35,80137	-1570,08	840,9897
2,431265	0,043	-0,11	3205,704	-64,7019	24,51999	-1774,74	577,3181
2,90599	0,05	-0,07	3472,744	-60,8508	10,23551	-1727,82	219,4816
3,551978	0,04	-0,03	3008,052	-35,3777	0,303785	-1025,52	-19,6852
1,11372	-0,12	-0,22	2441,198	-27,3079	112,701	3308,709	-763,994
1,339581	-0,05	-0,2	3258,869	-60,7099	108,2079	1628,741	-213,3
1,574239	0,01	-0,17	2991,112	-74,8246	66,10548	-152,863	-106,966
1,97054	0,03	-0,15	3448,527	-80,5965	50,43913	-1169,31	270,622
2,281695	0,042	-12	2449,967	-2712,24	2682,249	-42981,4	42087,56
2,606843	0,05	-0,09	3347,877	-65,9477	17,15238	-1518,63	64,53255
3,0544	0,045	-0,06	3530,001	-54,1704	7,865655	-1371,42	-8,459
3,674389	0,03	-0,025	3658,025	-32,6245	0,635045	-882,239	-71,0464
1,634703	0	-0,17	2751,458	-59,6594	59,6594	1758,158	-1758,16
1,796207	0,02	-0,16	3542,951	-83,5161	62,86069	1104,091	-1719,62
1,9774	0,027	-0,15	3186,039	-71,8969	46,82114	261,8896	-1009,15
2,305409	0,04	-0,12	3192,358	-66,5694	29,34651	-554,621	-554,621
2,576434	0,05	-0,1	2551,163	-52,535	15,35183	-755,647	-352,412
2,868364	0,05	-0,07	3526,709	-62,0287	10,62691	-1149,12	-382,649
3,280433	0,043	-0,043	3960,4	-53,3982	3,756531	-1170,66	-308,658
3,864326	0,03	-0,02	4065,938	-34,8821	-0,67453	-858,078	-201,509
2,302486	0,04	-0,12	3411,31	-71,1822	31,40628	927,3725	-2112,69
2,419824	0,045	-0,1	3510,316	-69,7371	23,69052	435,4798	-1807,67
2,557205	0,045	-0,095	3307,355	-62,8257	19,44149	109,7395	-1402,59
2,818528	0,05	-0,07	2873,448	-50,7975	8,916986	-329,548	-918,491
3,044208	0,047	-0,057	2779,119	-43,5042	5,428929	-464,565	-670,079
3,294951	0,04	-0,037	3278,562	-40,4617	2,233715	-586,205	-552,991
3,659294	0,027	-0,02	5615,169	-44,3012	0,107048	-797,702	-519,285
4,190728	0,02	-0,01	4383,679	-24,0994	-1,45746	-515,052	-246,542

Table 2 : For K_S = 3919 kN/ m^3 (L = 7,43m)

$a_i = r_i / L$	A_1	A_2	N_i (kN)	M_r	M_t	M_x	M_y
0,172359	-0,4	-0,15	3281,147	-267,259	649,8411	2700,878	8700,057
0,841373	-0,3	-0,23	3706,76	66,73599	257,4202	2081,791	7578,062
1,242896	-0,06	-0,21	3045,89	-61,6129	114,8855	-1796,59	3384,116
1,806711	0,011	-0,17	2961,56	-66,2457	56,74949	-1961,11	1678,117
2,209892	0,041	-0,12	2584,961	-55,7623	24,86815	-1656,01	735,3668
2,613255	0,049	-0,09	3764,51	-73,1471	19,37668	-2175,1	572,7444
3,151234	0,043	-0,047	3505,988	-48,5921	4,646254	-1446,19	136,6062
3,877673	0,033	-0,017	2832,838	-25,9604	-1,29013	-773,052	-39,0128
0,6599	-0,39	-0,22	3089,299	86,46798	264,739	7668,237	2797,731
1,055305	-0,15	-0,22	3321,791	-19,8181	165,0634	1474,127	2854,183
1,396621	-0,04	-0,2	2879,043	-57,1714	90,74101	-760,584	1760,959
1,915715	0,025	-0,05	3506,974	-41,003	15,44588	-1030,59	268,9875
2,299866	0,042	-0,12	2797,74	-59,8126	25,55985	-1581,68	560,9438
2,68977	0,05	-0,08	3205,704	-59,9507	13,22761	-1660,73	268,3849
3,214971	0,042	-0,045	3472,744	-46,5986	4,081786	-1327,66	60,6545
3,929645	0,032	-0,012	3008,052	-25,9796	-2,07947	-754,944	-81,2175
1,232137	-0,078	-0,21	2441,198	-39,1466	94,65211	2773,059	-1118,99
1,482013	-0,03	-0,19	3258,869	-62,6698	91,16859	1263,433	-414,168
1,741621	0,01	-0,17	2991,112	-68,3521	59,63298	-150,755	-109,074
2,180059	0,04	-0,12	3448,527	-74,0632	33,85334	-1192,07	-6,18616
2,524298	0,048	-0,1	2449,967	-49,6357	15,35571	-1023,21	1,672643
2,884018	0,05	-0,07	3347,877	-58,7909	9,995583	-1382,29	-71,8139
3,379162	0,04	-0,04	3530,001	-44,0123	2,85248	-1128,1	-98,4597
4,065071	0,025	-0,01	3658,025	-24,7389	-1,91898	-675,488	-118,916
1,808514	0,015	-0,17	2751,458	-64,2437	52,21293	1536,725	-1895,24
1,98719	0,03	-0,15	3542,951	-82,3322	51,34906	827,755	-1751,05
2,187649	0,04	-0,13	3186,039	-71,3355	34,18625	11,35252	-1118,4
2,550533	0,046	-0,1	3192,358	-62,8089	20,00255	-637,814	-637,814
2,850376	0,048	-0,07	2551,163	-43,6769	7,981002	-685,287	-378,45
3,173345	0,046	-0,048	3526,709	-51,6796	4,38994	-1000,37	-408,865
3,629228	0,038	-0,027	3960,4	-43,767	-0,10235	-982,928	-324,379
4,275203	0,018	-0,002	4065,938	-18,6933	-2,64067	-471,931	-163,822
2,5473	0,05	-0,1	3411,31	-70,5634	20,84358	613,5343	-2095,19
2,677113	0,05	-0,08	3510,316	-65,7503	14,58744	202,1066	-1726,76
2,829102	0,048	-0,07	3307,355	-56,7505	10,474	-71,6216	-1307,42
3,11821	0,042	-0,053	2873,448	-40,3543	5,174679	-299,658	-748,694
3,367885	0,04	-0,04	2779,119	-34,6732	2,268682	-405,251	-560,404
3,645289	0,033	-0,021	3278,562	-30,9861	-0,55198	-481,756	-458,08
4,048371	0,02	-0,01	5615,169	-30,9678	-1,76865	-579,304	-396,241
4,63631	0,007	0,006	4383,679	-6,4886	-2,45629	-157,189	-109,369

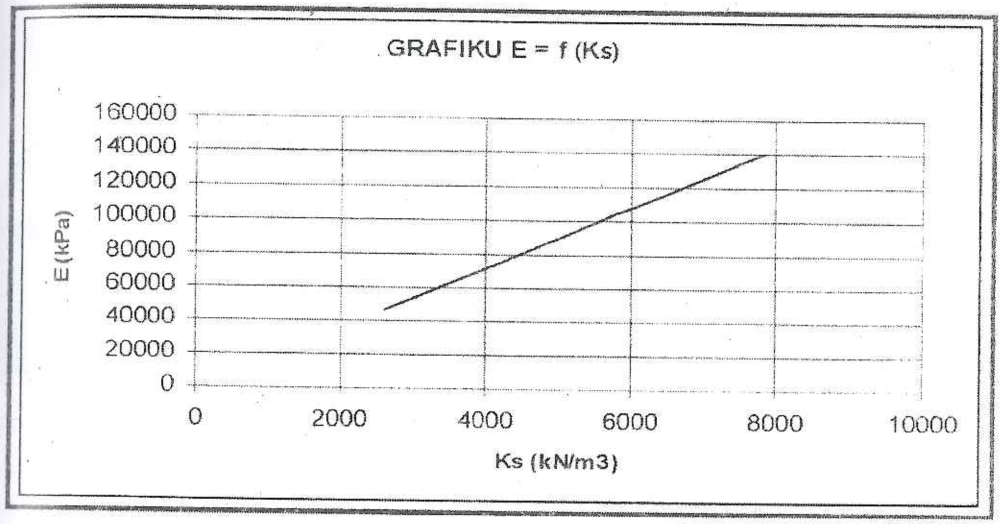




For slab foundation in study:
B – 19,7m width foundation
μ - 0,3 Poisson's ratio of soil
E – young's modulus of soil

Table 3.

K_s (kN/ m^3)	E (kPa)
2613,016	46843,53
3919,0	70255,913
5226,03	93687,039
7839,05	140530,649

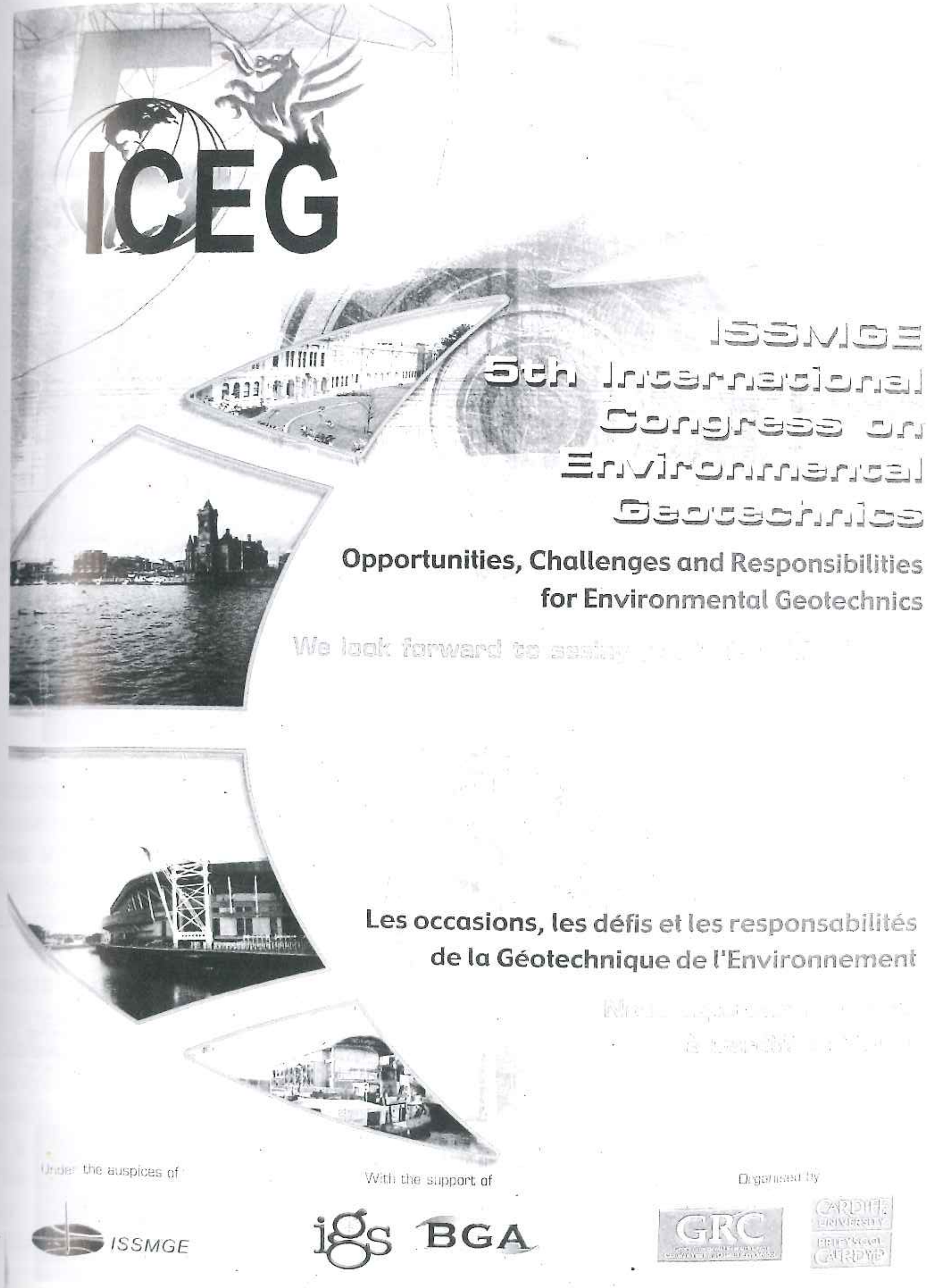


YEAR 2006

- 5-th International Congress on Environmental Geotechnics Cardif UK
  May 24-26 2006

- XIII-th Danube-European Conference on Geotechnical Engineering Lubjana, Slovenia
  May 29-31 2006

- 5TH Hellenic Conference on Geotechnical and Geoenvirnmental Engineering.
  May 29-june 2. Xanthi Greece



GRC
Geoenvironmental Research Centre
Canolfan Ymchwil Ddaearamgylcheddol
Wed, 24 May, 2006

ISSMGE: 5th International Congress
on Environmental Geotechnics
Cardiff, Wales, UK, June 2006

Cardiff: Capital City of Wales

Cardiff, one of Europe's fastest growing capitals, provides a fabulous backdrop for the Congress.

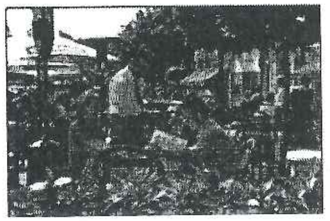
Lively, elegant, confident, cosmopolitan, Cardiff caters for all tastes, offering everything from the excitement of a vibrant city life to the peace and tranquility of the nearby coast and countryside. A city with both heritage and ambition Cardiff has a distinctive character, a good quality of life, and a growing national and international reputation.

As the capital city of Wales it is home to many national institutions including the National Museum of Wales and the much-admired Millennium Stadium. The city centre skyline is testimony to its heritage and ambition, with landmark buildings ranging from the ornate civic centre to the historic Cardiff Castle.

Cardiff is well established as 'Europe's Youngest Capital' being proclaimed a city in 1905 and becoming the capital city of Wales in 1955. But its history dates back more than 2000 years to the Romans. It was once one of the busiest ports in the world, exporting the coal which fuelled the industrial revolution. The famous Tiger Bay docklands have been substantially transformed into Cardiff Bay, a modern development of homes, shops, offices, visitor attractions and the National Assembly for Wales, all surrounding a huge freshwater lake.

A Friendly City

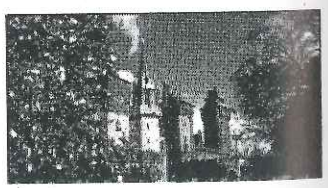
With a population of 327,500, Cardiff does not suffer from overcrowding. The city of Cardiff offers an excellent location in which to live and study. People at Cardiff benefit from the combination of a small, friendly, inexpensive city with the cultural and recreational amenities of an ambitious and progressive capital city.



Two Thousand Years of History

The history of Cardiff stretches back over 2000 years to Roman times when the Romans first established a fortified settlement on the banks of the River Taff. One thousand years later the Normans came to Cardiff. The castle, or keep which can be seen within the modern walls of Cardiff Castle, originates from this time and William the Conquerer came to Cardiff in 1081.

Cardiff remained a small market town for the next few hundred years, but with the dawn of the Industrial Revolution, it would come into its own. The city's greatest period of growth began in the 18th century with the development of the coal and iron industries in the South Wales valleys.



In the early 19th century, first a canal and then the Taff Vale Railway linked Cardiff with Merthyr Tydfil and the Rhondda Valleys, at the time the greatest iron and coal centre in the world. Soon, Cardiff would itself achieve worldwide fame as a great coal-exporting ports. The resultant

Environmental and Geotechnical Problems of Albania's Mine Sites.

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Abstract

Albania is a rich country with diverse minerals. After the Second World War the mines and the minerals industry were more developed. Often the underground work it is situated near earth surface and in the process of they exploitations and after this process did appear many geotechnical phenomena very dangerous for the population and for the economy of the country. In these papers we would like to present some of this phenomena and engineering measures for their stabilization and /or rehabilitation.

Introduction.

Albania is a very rich with minerals such as metalliferoues, coal sand minerals etc. For these reasons the mining and mines industry after Second World War more developed.
The intensive exploitation working of coal often was accompanied with impact on environment. In this paper we would like to present this phenomenon in Albania.
Environment impact can be defined as actions produced by a development of mining works which are capable of affecting the characteristics of the environment. The waste ponds of coal in Albania are those of : basin of Valias (Tirana), basin of Manza (Durresi), basin of Memaliaj (Tepelena), basin of Mborje-Drenove (Korça) etc.
Exploitation of coals nearness (proximity) the earth surface are accompanied with dangerous phenomenon such as damage of buildings, sewerage, roads, telecommunication network, and with grave damage of some villages and their transfer in a secure place.
The environment impact statement for mining has to consider not only the actual effects on environment of exploitation and development frequently the possibility of rehabilitation and the costs of rehabilitation have to be considered.

General Lithology of Coal Deposits.

The coal mineral in Albania is generally found in molasses rocks and less in flysch rocks (fig.1). It's situated between clay stone and siltstone layers. From the structural point of view this mineral is found in folded structures (syncline and anticline). According to geological construction, clay stone and siltstone rocks predominate with to sandstone.
They are about 80-90% of the lithological profile. The main coal deposits in Albania are located in following area (fig.1).

- Coal-bearing flysch formation of Mborja-Drenova and Gore-Moker area
- Coal-bearing molasses formation of Tirana-Shijak-Fushe Kruja area.
- Coal-bearing molasses formation of Tepelena area. Coal-bearing molasses formation of Durresi area.
- Coal-bearing molasses formation of Mati area. In these areas many coals deposits are found and a lot of mines have operated up to 1995 year.

This paper describes the land subsidence of Valiasi, Mezezi and Memaliaj, which are caused from coal mineral exploitation.

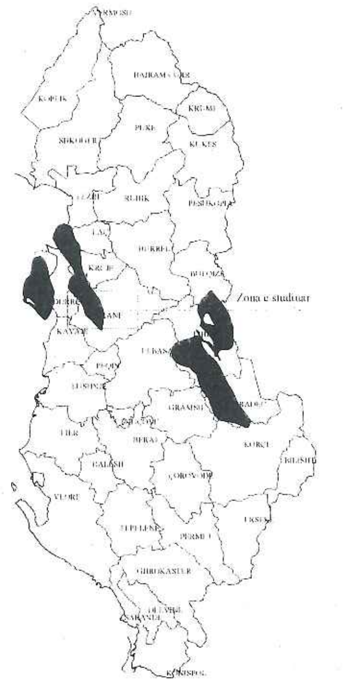

Fig.1 Schematic map of coal mines distributions in Albanian

Valiasi site.

In Valiasi mine the coal mineral is found in eastern limb of the Tirana syncline, which to west with 2-5^0 up to 10-14^0 .Here the claystone rocks are about 44% the lithological profile. They have grayish and grayish-brownish color. Concerning mineralogical composition they are ilite and montmorillonite type. Across the water, they are subject of swelling phenomenon. So the claystone increase their volume up to 50%.
The siltstone rocks have grayish and grayish-brownish color, as 42% of lithological profile. Silstones in the interaction with water will be swelled, increasing their volume up to 30%. Whereas the sandstones rocks consist of quartz, feldspars, micas and carbonate fine up to medium grained, which are cemented by clay and clay-carbonate material. These rocks represent 6-7% of lithological profile. The coal layers have the different dimension. They range from 0,2-0,5 km up to 1,0-3,0 km in the extension direction, from 80-100m up to 200-500 m in dip direction and 0,2-0,4m up to 2-4m thick. The coal deposits constitute about 8,4% of the whole lithological profile. The above rocks are covered by Quaternary deposits, which make up the first sanstone layers. The above roks are covered by Quaternary deposits, which make up the first terrace of Tirana River, as well as its bed. They are represented by middle to coarse grain gravels 35-40m up to 60-70m thick, made up mainly limestone and less sandstone filled by

sands, silts and brown clays, which are situated in lower part of soil profile and clay and silts in upper part of soil profile with thickness 5-15m.
As result of the coal exploitations in this area has occurred the land subsidence furrow shape (fig.2, Photo 1,) in two parts. First one in eastern part of exploitation field has a 0,8x3,0km dimension and other in western part of exploitation field has a 0,5x1,0km dimension.

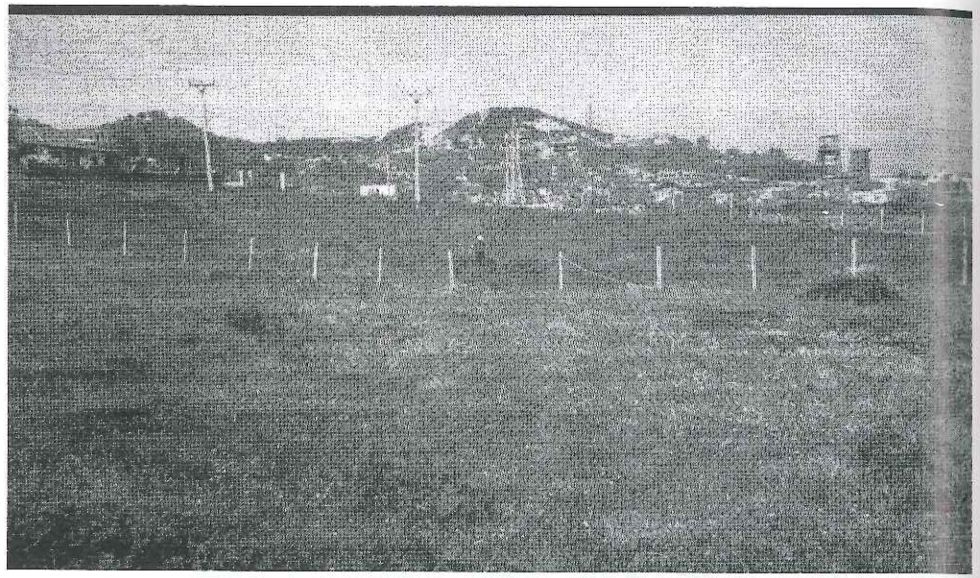

Foto. 1 Subsidence in Valiasi

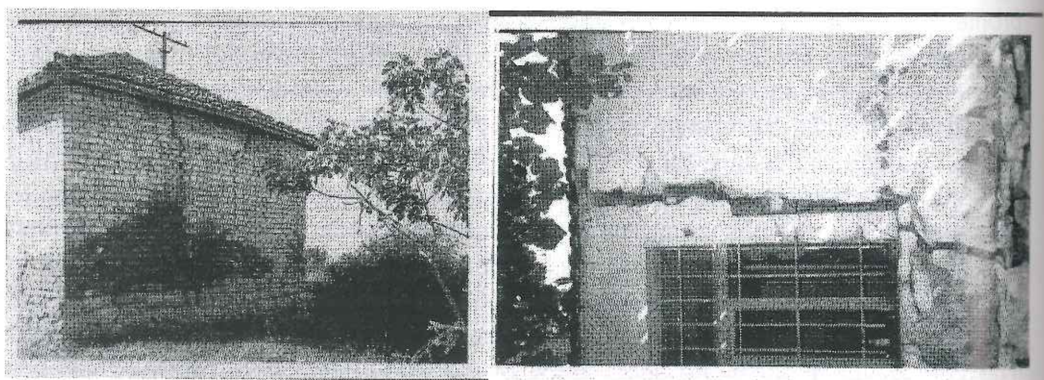

Foto 2 The demages of the buildings

Mezezi site.

Mezezi mine has the same geological construction with the above mentioned mine. But it's differ according to geological structure (fig.3), because of this mineral deposits is situated on western limb of Tirana syncline, which dip to east direction with 70-75^{0}. Mostly of this limb, where the coal mineral is found, overlies by Quaternary deposists 10,0-2,5m thick (silts and clay-upper part of soil profile and gravels-lower part). From the coal exploitation on this area many land subsidence are occurred in tunnel shape, with a 10,0-20,0m diameter and 2-5m thick (fig.3, Photo 2)

Memaliaj site.

Memaliaj Mine is located in coal-bearing molasses formation of Tepėlena area (fig.1). The coal mineral is created between the clay stone and siltstone combinations, which make up a inverted syncline structure. The lithological construction of this deposits is almost the same with above-mentioned sites. But the thickness of the Quaternary deposits is lesser than Mezezi and Valiasi areas. They are 4-10m thick and consist of silt gravels soils. According to physical and mechanical properties the clay stone and siltstone rocks-molasses are soft rocks, whereas.

In these regions of Albania quite strong horizontal and lesser vertical movements have been recorded in a zone up to 50-60m from the margin of open coals cuts, These movements result in damage to some villages in Memaliaj as Cerrik, Mirik, Zhupanik, Licaj etc. where some of theme (Cerrik and Zhupanik) are transferred in other place. Also we did record damage to water supply, sewerage services, drainage network and some rural roads.

Degree of damages.

The houses in the study's zone are 2-5 stores. Some of them undergo very great settlements the others dangerous fissures from horizontal movements. The subsidence due to coal extraction and coal mining areas is very important when the thicknesses of soils over the open coals cuts are 30-50m.
The subsidence zone extent with ray R=(6-10)km from the centre of mining work.
The settlements and the fissures are so enormous that to be considered very important and dangerous for the habitants or search the great costs for their rehabilitation. In other hand the costs of rehabilitation of sewerage, roads etc are considerable,

__Analysis of phenomenon__

Subsidence due to coal extraction.

The earth surface deformation may be explained by covering viscous-elastic to plastic media. Stiffly plastic flow in a rock mass is influenced by underlying workings (fig.4)

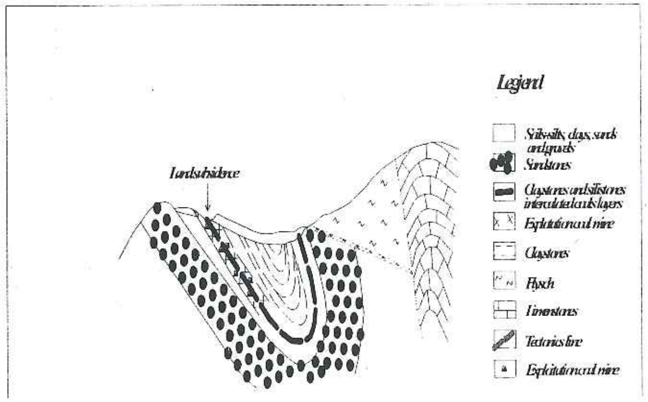

__Fig.2 Schematic cross section of Valiasi mines (Tirana)__

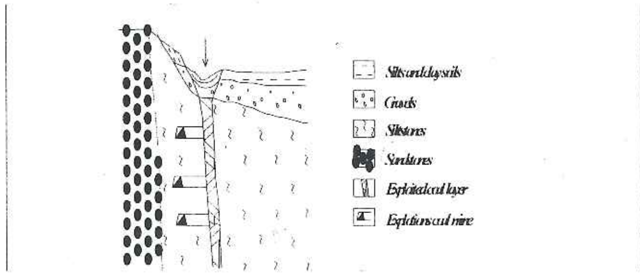

__Fig.3 Schematic cross section of Mezesi mines (Tirana)__

The downward movements spread very rapidly until it research's the upper earth surface. In this process six zones of movement can be distinguished (fig.5) and two different stressed vertical zones (compression and expantion).

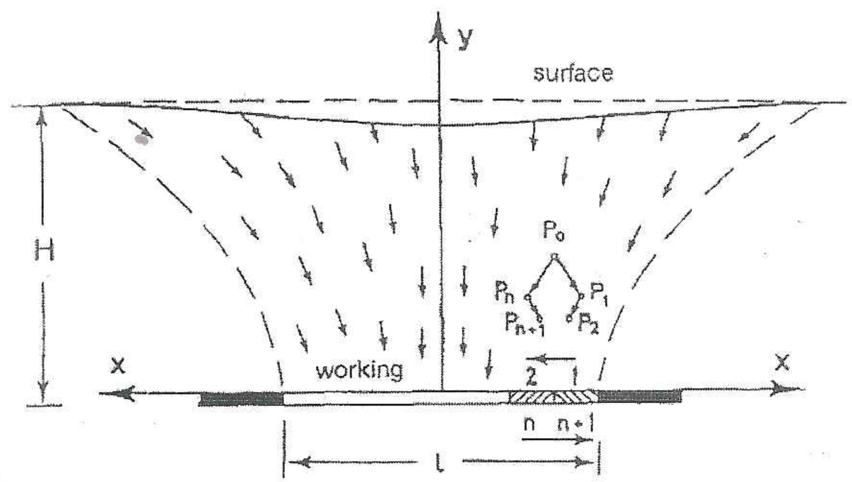

__Fig.4 Stiffy plastic flow in a rock mass influenced by underlying workings__

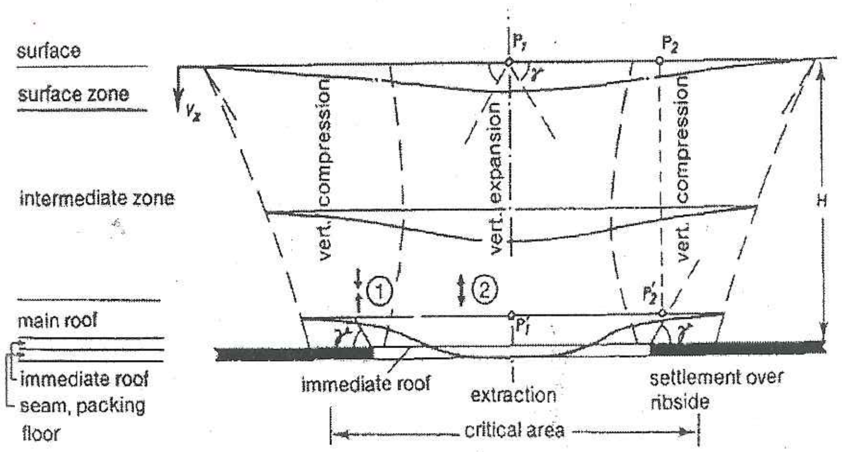

__Fig.5 Vertical deformations and six zones of movement__

__Developed forces.__

The deformations of earth surface are vertical and horizontal. The first due at the buildings the serviceable limit state and the second due the finite limit state for them. Especially for the pipelines water supply and sewerage the curvature of earth surface is very dangerous. So in Memaliaj zone, Mezez zone we have damage of water supply and sewerage.

By our calculation we have determined the vertical deformation which vary from 10cm to150cm (or 1,55-2,45 mm/m relatively). In foundations of buildings seating on native earth, under the influence of mine working, forces are transmitted to the structure, partially directly as earth pressure and friction force through linear horizontal deformations of the ground, and partially as bending forces through curvature of the ground.

As a result of upward (see fig.6) or downward (see fig.7) of the ground built on, the underside of the foundation becomes detached from the ground in places and stands free of it, thereby losing some its support. So we have read placements of load and this effect produces bending stresses in the structure and many cracks in them.

The small degree of ground curvature it is not suffering damage. The visible cracks develop in masonry walls given radius of curvature of 1-1,5km corresponding of a sag f=2cm in a chord 12m length and cracks over 1mm wide appeared in downward from R=10km.

Curvature caused by settlements cause damage to a masonry structure of length L where p_z <500L. The value of base pressure stresses on the underside of the foundation we have calculate by the stiffness index E_s of the ground (E_s =0,5+0,7).10^4 KPa, and from them the bending stress on the foundation which will be smaller the allowing bending stress of material, In many cases this conditions it is not satisfied.

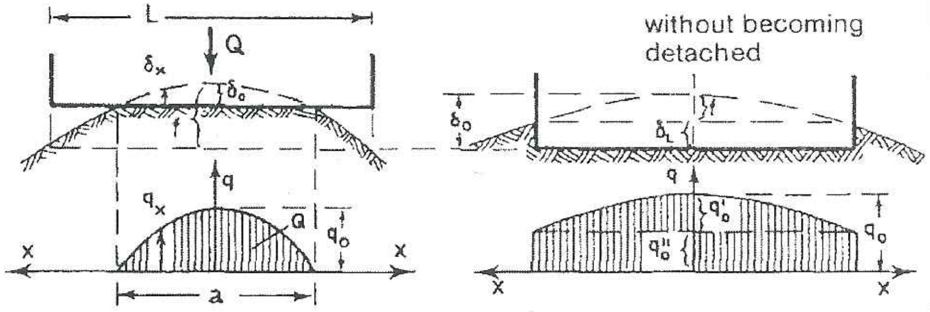

Fig.6 the base pressure "q" where the structure rest on an upwarped saddle

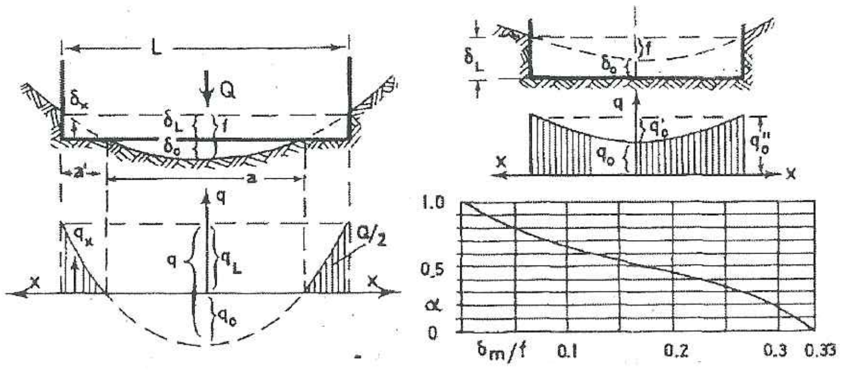

Fig.7 The base pressure "q" where the structure rests on a down warped saddle.

From the horizontal relative displacement of the ground which is (35-45)mm/m a foundation appeared a friction force. (fig.8) They increase up to the middle of the structure and produce in the ground compression or extension state.

Finally the structure warped in such scale the can to be destructive for the houses where the length it is around 9-10m. The biggest damage if houses recorded in these zones arrived where the horizontal relative displacements is 12-15mm/m.

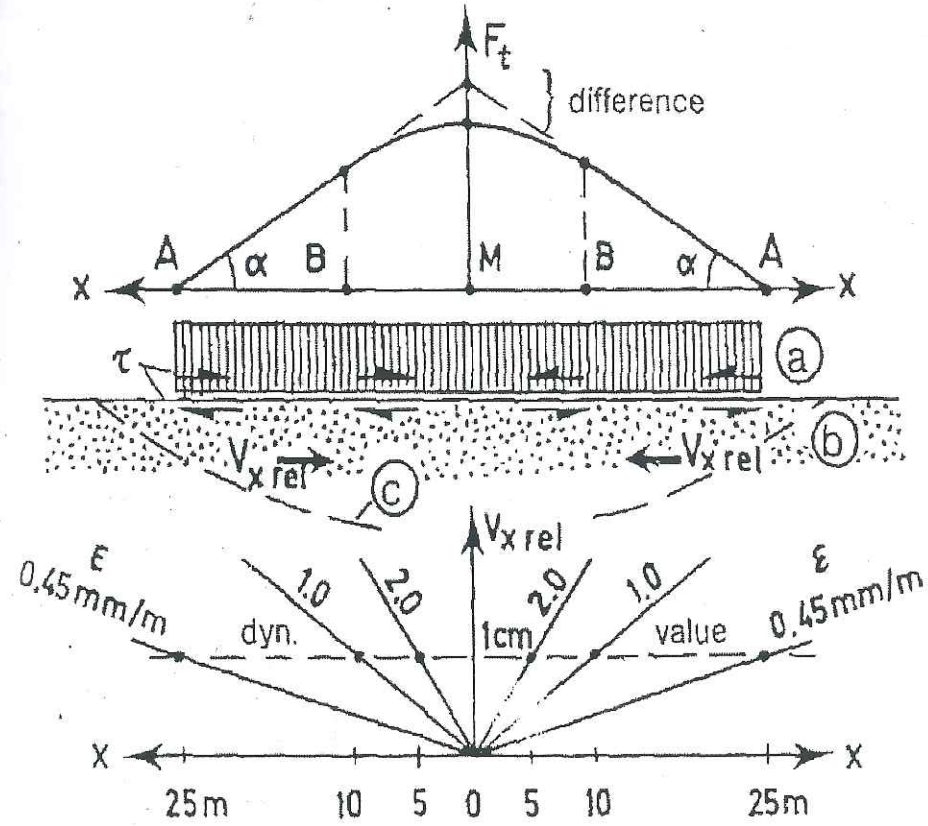

Fig.8 Friction forces tranasmited to the structure

Engineering measures.

The engineering measures consists at increase the rigidity of structure towards the horizontal and bending deformation. For existing buildings reinforcement of foundation will be made, and can putting some concrete belt in different level.

For the new buildings it must predicted of rigid mat foundation and reinforced concrete structure. For pipelines sewerage and water supply in the zones where may be strip mining, which is particularly damaging, it is necessary to predict supplementary connections and utilization of geotextill or other resistant materials towards the deformation of earth surface.

Conclusions

- In existing buildings (2-5) stories which are dominated in mining zones the destructive force is from horizontal displacement at degree 15-35mm/m and for curvature of earth surface R=6km
- The degree of damage of buildings increases with reduction (diminution) of rigidity of structure. They depend from the thickness of rock over coal cuts and from mechanical and physical propriety of soils.
- The maximal settlements are at the centre of mining works and they are from 10cm to 100-150cm.
- The impact on the land surface started 3-6months after exploitation of mines and they terminated 2-4 years after this process.
- The actual development of urban zones should be made in accordance (in conformity) to new regulation law.These laws enforce the investigation and evaluation of the consequences to the environment of all engineering works, such as dams, pipelines motorways', hydroelectric schemes mining projects and urban development
- Geologist and geotechnics are the first in this work and are in a unique position to solve possible issues for the impact study.

REFERENCE

Cela.K (2004)
"Demtimet e ndertesave ne siperfaqe te tokes nga shfrytezimi I minierave qymyrgurore". Revista "Gjeoteknika 3". Albanian Geotechnical Society Edition.

Dimo.Ll (1982)
"Rreth kushteve te formimeve dhe perspektives qymyrmbajtese te depozitimeve mollasike te ultesires se Korces". Buletini I Shkencave Gjeologjike Nr.4 Tirane.

Muceku.Y (2003)
"Engineering mapping of Tirana-Durresi-Kavaja region. Geological Survey of Albania

Muceku Y (2005).
"The study of soil behavior under static and dynamic load for urban development of Tirana-Vlora region". Ph.D.thesis p64-113.

Influence of the deformation parameters of soils in selection of the static scheme maliq's bridge.

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Abstract.

Some part of road's infrastructure in Albania pass in marsh zones which are very difficult conditions for the structure of the bridges. In this paper we would like to present the geotechnical study of Maliq's zone (marsh zone) and the influence of the deformation's characteristic of the soils in selection of static's schema of Maliq's bridge.

Introduction.

Some part of the territory in Albania (about 30% of the field zones) is composed from marsh deposits. Such are the zones of Maliq of Divjaka, of Terbuf, Torrovica etc. One branch of highway (8 corridor) Qafe-Thane-Korçe, passes in march zone of Maliq, where for the road is predicted, a bridge or an embankment.
The very weak soil conditions influence in behavior of bridge's structure. From the interaction soil-structure is selected the static schema of the bridge.

Geological conditions.

Site localition and description.
The field of Korca represent a graben created from the tectonic activity during Neogene-Queternary period. The graben so created is filled with alluvial deposits from rivers and torrents of the zone and at the end of this period a march has been created in its center. There were created the conditions for organic deposits which thickness reaches 20-25m in its center. The total organic matter various from 1% to 13,5%.The water was drained from the field of Korca while deepening the bed of the river Devolli at its outfalls. The surveys of several years, show that a layers parts of the field have create e deformed relief because of the shrinkage of peaty larges. Such peaty deposits present potential problems for the embankment stability and large settlements for them.

Seismic hazard.
Based on the seismic zonation map of Albania and local ground conditions of the zone it is classified in III soil category, with seismic intensity VIII (MSK-64) and seismic activity coefficient K_E =0,26 (after Albanian seismic code).

Preliminary longitudinal profile.
In the zone of the marsh of Maliq the geological investigations identified the following layers:

- Silt clay with organic matter slightly over consolidated with thickness h=1,5-2,5m.
- Clayey peat and peat of deep grey color normally consolidated h=2-4m.
- Very soft to soft silty clay and loose fine sand, unconsolidated with thickness 1-4m
- Medium to dense sand and gravel well consolidated.

The water levels are around 1,5m

Geotechnical study
From the laboratory test of the samples taken in the Maliq's zone we must define the physical and mechanical characteristics of undisturbed and disturbed soils and rocks samples. These samples were taken from boreholes, trial pits and borrow pits. All the tests were performed in the laboratory of "ALTEA&GEOSTUDIO" 2000 in Tirana.
The identification and classification test are performed. Also the triaxial test, odometric test, direct shear test one compression unconfined test ect are performed.
From them we have constructed three characteristic geotechnical models (fig.1). The average value of some their parameters gives in the table 1.

Layer	PI	E KPa 10^4	C_v cm^2/s 10^{-3}	γ gr/cm³	φ^0	C KPa	W %	$I_c = \frac{W-W}{PI}$	Classification
1	19	0.27	0.70	1.68	23	35	46-55	0.78	ML and OL
2	13	1.1	0.90	1.98	22	34	19-20	0.23	ML
3	20	0.35	0.11	1.89	18	40	37.5	1.1	CL
4	17.5	0.5	0.60	1.72	20	35	42	0.5	MH
5	-	1.1		1.78	26				SM
6	7	0.7	0.8	1.95	24	30	26.5	0.33	CL
7	16	0.2	0.7	1.77	22	32	32	0.26	ML and DL
8	-	2.0		1.90	28				GM
9	14	0.38	0.8	1.82	21	20	36	0.52	ML

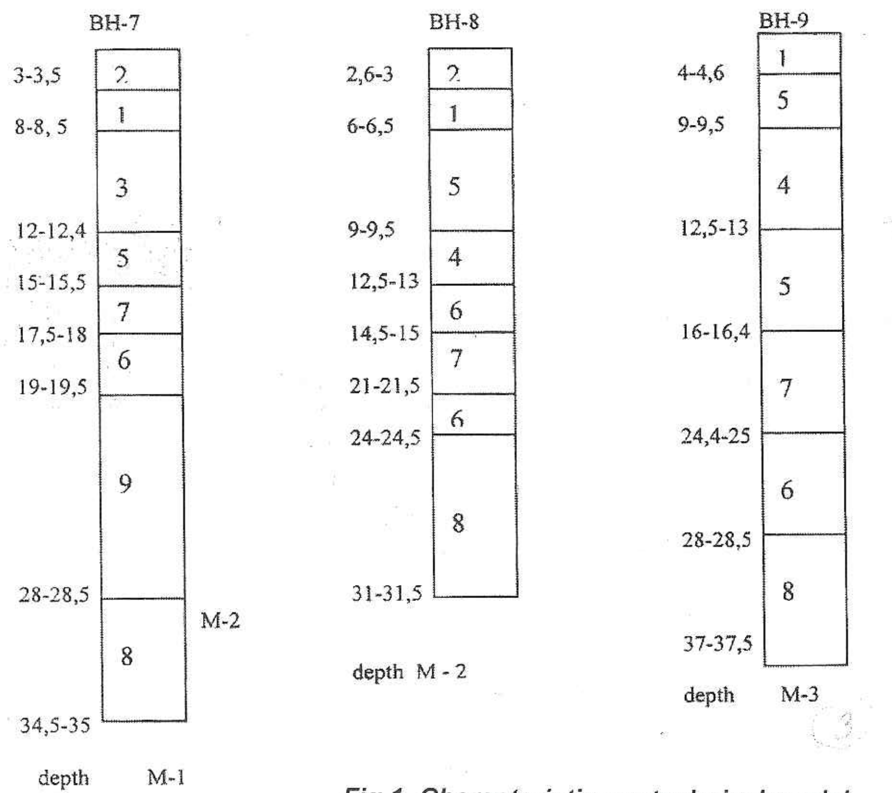


Fig.1. Characteristic geotechnical model

From triaxial test (UU) results very low value of the angle of friction φ=6-10^0 and the value of the cohesion C=(16-30)KPa.
In the same time the value of the void ratio is very high e_0=1,2-2,6. With this property of the soils the expectancy phenomena's are:

- Very large settlements C_c = 0,2-0,344
- The long times for their stabilization (about 8,5 years).
- The large settlements due to secondary consolidation C'_a = 0,014-0,03

Structural solution for the highway in the marsh zone.
Two variants for structural solution are studied.
Variant I – Embankment with 5m high and base B=32m is realized with material which have γ=22KN/m^3 (after compaction)
Variant II – Bridge with space between the foundations 20-25m and the columns were connected monolithically to concert deck (schema statically indeterminate).

For two variants are calculated:

- Settlements due to primary consolidation.
- Times of during of the primary consolidation.
- Settlements due to secondary consolidation
- Comparison of the total settlements with allowed settlements for every structure.

For the embankment we have used first geotechnical model. Active zone is 35m, (where σ_1= (0,1-0,2)P; P=110KPa is pressure on the base of the embankment). Settlements from primary consolidation are 35cm, the time of 90% of consolidation is 8,8 years and settlements due to secondary consolidation are 18cm. So the total settlements are 53cm, in consequence biggest the limit settlement (S_{lim} =50cm). On the other hand the low value of φ and C to causes insufficient bearing capacity of the basement which can be followed by instability or collapse of the embankment.
Second variant. The bearing capacity of the foundation of the bridge for depth 3m area 48m^2= 4x12; b=4m and for the three geotechnical models are about (435-440)KPa. Allowable value is 200 KPa.
The settlement of foundations which rest in three geotechnical models are:
S_1 = 14,32cm for the first geotechnical model
S_2 = 12,855cm for the second geotechnical model.
S_3 = 10,273cm for the third geotechnical model
The difference of settlement between divers foundations is:

ΔS_{max} = 1,465cm to ΔS_{max} = 4,075cm

Allowable value for the schema continually beams is ΔS_{aw} = 0,001. l. ΔS_{aw}=2cm-2,5cm
Two are the solution of the problem.

- Utilization of the deep foundations (pile foundations), length of pile to be 20-25m, supported in gravel...
- Change the static schema of the passing part of the bridge, to used freely stand beams.

Conclusions.

- The soils of Maliq's zone are marsh deposits, very soft and weak soils, very compressible and unstable.
- The values of the secondary compression index for clay layer is considerable and the 30% of the total settlement of the bridge's foundations due to them.
- The process of settlements can last about 10 years.
- This factors dictate the change of static schema of the structure of the bridge.

REFERENCE:

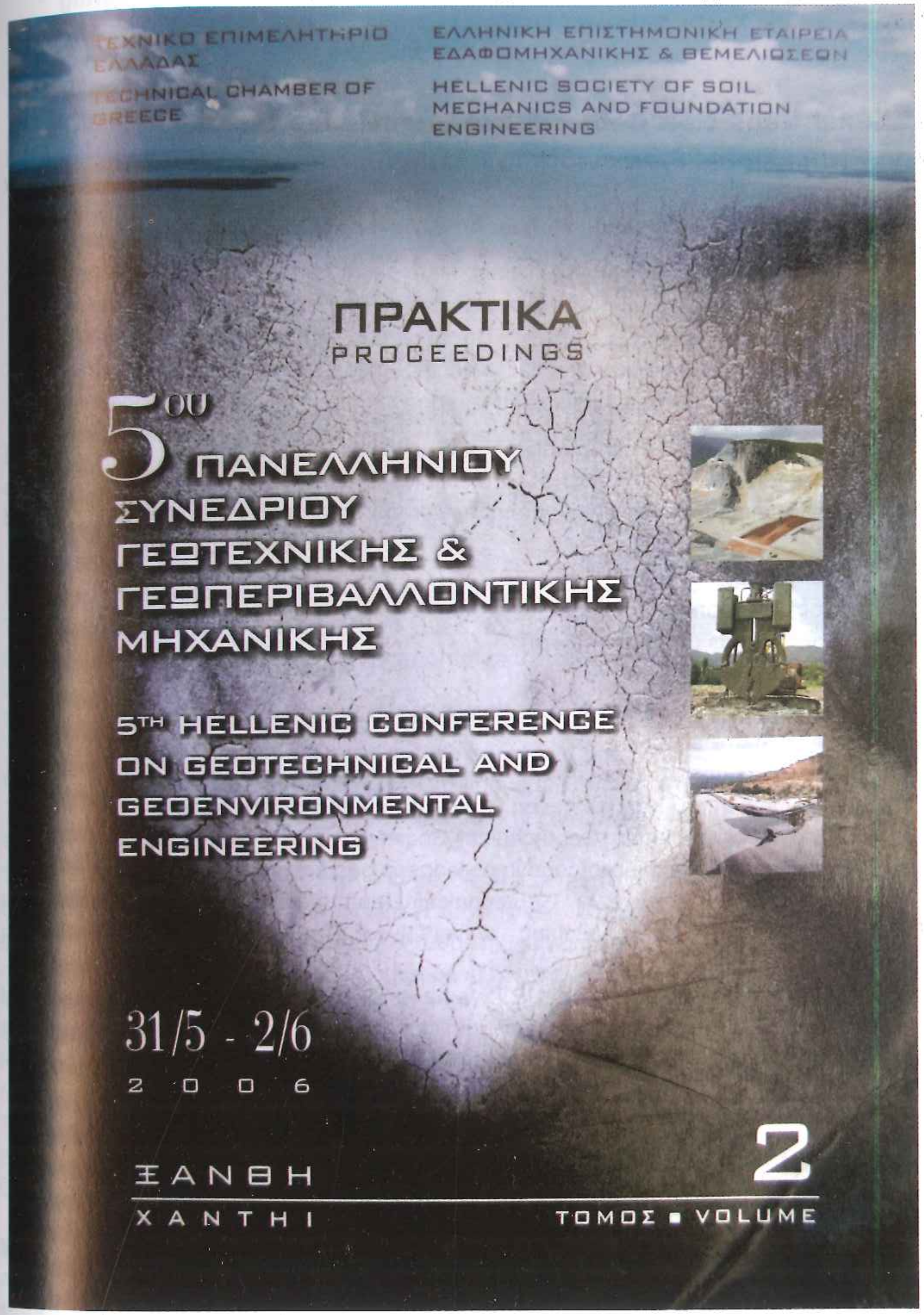
Ahmetaj L, Allkja. S, Hida.S — Final report on the geotechnical and geological investigation for the Plloca-Korca road section. ALTEA &GEOSTUDIO 2000 Tirana 2003.

Bozo L — Rock mechanics. Civil Engineering Faculty Tirana 2000

Bozo L — Soil Dynamic. Civil Engineering Faculty of Tirana 2004

Braja M.Das — Principles of Geotechnical Engineering PWS Publishers Boston 1985.

Tatsuoka F S Shibuya, Kuwano R — Advanced laboratory stress-strain testing of geomaterials A.A.Balkema Publishers 2001 Tokyo.



National Report on Geotechnical Engineering Education Research and Professional Framework in Albania

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Introduction

Albanian Geotechnical Society (AGS) is a new society. It is created at year 2000.
In Albania with geotechnical problems are involved civil engineers who have full geotechnical formation in the field of Geotechnics. Therefore, in the university courses of our Civil Engineering Faculty students are prepared on these below disciplines and programs:

- Soil Mechanics
- Rock mechanics
- Foundations (all type of them)
- Improvement of soils.
- Maintance of foundations from damages.
- Soil dynamics

During the practice work some of the civil engineers are more specialized in geotechnical field but till now we haven't authentic geotechnical engineer.
We founded AGS (Albanian Geotechnical Society) with the intention to create this specialty and to achieve the profound knowledge in geotechnical problems.
The basis for this initiative is Geotechnical Department (or sector) in the Civil Engineering Faculty of Polytechnic University of Tirana (UPT) of geotechnical Department (or sector)
In the Geotechnical Department of UPT we have 50 years of experience in the field of the edition and publication of the geotechnical and scientific researches and proper qualification process that designates a professional civil engineer as a geotechnical specialist.

The main objectives of AGS.

Some of the main objectives of AGS are the information of the civil engineers about:

- The new technology applied in the geotechnical design and practice work.
- The new methods used in the geotechnical investigations of the geotechnical structures etc.

For that purpose AGS in cooperation with Geotechnical Department of UPT during the period 2001-2005 has performed (accomplished) an intensive work in the field of education and supplementary qualification of the civil engineers.
The most suitable form was the use of short intensive courses (2-3 days) for specific problems, about the interesting geotechnical structures and about the problems which engineers need in practice work.

Intention of the courses.

The intention of the courses has these main goals:

- The information for the new calculation methods in the geotechnical structures.
- Acknowledge with new methods of construction works and contemporary technologies used today on geotechnical structures construction.
- Acknowledge of new methods and results in the field of soil compaction.
- Acknowledge of the complex laboratory methods and in-situ geotechnical investigations.
- Acknowledge with the new European technical regulations for construction (EC) and in particular about EC-7.

The organization model.

The courses are accomplished for two levels, for the civil engineers and for students of the last year of faculty (5th year). The program of the course it is announced 3-6 months before its start.
The registration is made with different fees for civil engineers and students.

- The course papers have been theoretical and practical and sometimes it was accompanied with practical in situ demonstration.
- The participants obtained these papers (lectures) at the first day of course or seminar.

The courses have been productive and successful because the civil engineers did profit the new knowledge and with it they have been capable to realize specialized geotechnical works with conformity European, American and others International norms in collaboration with international construction companies and mainly international supervision.
During the courses we have developed live discussion between participants . For these reason the professional profit it is considerable for them. The interest of civil engineers for this kind of courses or seminars is grown up year after year.
The lectures of courses have been prepared by professors of UPT and by foreign experts in collaboration with ENCP or other geotechnical societies as Hellenic Geotechnical Society, American Geotechnical Society, and ISSMGE for Europe etc.

The courses and seminaries organized by ags.

AGS realized the following courses:
1998 – Topic Geotechnics and Infrastructure of Transport.
Duration 3 days

Lectures prepared by: Geotechnical Department of UPT, ENCP, Scetauroute, Irish expert in GRD (General Road Directory).
Participants 30 civil engineers and 25 students.
1999 – Topics: Insurance of quality in design and construction of roads.
Duration 3 days (realized twice)
Lectures prepared by Geotechnical Department of UPT
Participants 50 civil engineers and 40 students.
2000 – Topic: Behavior of pile foundations under static and seismic actions.
Duration 2 days
Lecture prepared by Geotechnical Department of UPT
Participants 30 civil engineers and 20 students
2001 – Topic Maintenance and rehabilitation of roads in Albania.
Duration 2 days.
Lecture prepared by Geotechnical Department of UPT.
Participants 25 civil engineers and 20 students.

2002 – Topic Acquaintance and analysis of EC-7
Duration 2 days
Lecture prepared by Geotechnical Department of UPT
Participants 35 civil engineers and 25 students
2004 – International seminar (workshop)
Topic Geotechnical Earthquake engineering.
Duration 2 days
Guest speakers:
Prof. Emeritus Shemsher Prakash from USA (Oral presentation)
Prof.Seco Pinto from Portugal and Vice President of ISSMGE (paper presentation).
Prof.G.Bouckovalas from NTUA Greece
Prof.D.PE.P.Dakoulas from University of Thessaly, Volos, Greece
Research Associate A.Papadimitriou, from NTUA Greece.
Prof.L.Bozo from UPT Albanian
Prof.A.Frasheri from UPT Albania
Eng.Geologist Y.Muceku from GCT Albania
Eng,civil N.Shkodrani from UPT Albania
Participants 60 civil engineers and constructions from the design firms.

Education of the undergraduate civil engineers.
The members of AGS have taken the initiative to direct the work for diploma (graduation) of some students (10-15 students of year). In this work are resolved different geotechnical problems as:

- Construction in the weak zones.
- Design of pile foundations for the bridges of buildings
- Design of mat foundations

- Study of slope stability and calculation of engineering measures for its stabilization.
- Design of retaining walls and other retaining structures.
- Soil-structure interaction and design of building in seismic zones.
- Geotechnical investigation and study for roads design
- Compaction of soils
- Bridges design
- Design of harbor construction
- Tunnels design etc.

To profit from the each other experience:

- Organization of summer school, seminaries, short courses for the young engineers.
- Common projects in the geotechnical field with other geo-society or university etc.

Impact of AGS activity
Activity of AGS has given the results in those main directions:

- Civil engineers, participant in the courses or in the seminaries to get experience and supplementary qualification serving the execution work of difficult geotechnical structure conditions.
- Geotechnical education is influenced for creation of a group of civil engineers specialized on geotechnical problems and helping them during the geotechnical studies or other related geotechnical work on private or public sector.
- Work with students in the short courses, summer school etc or in the diplomas work as well it leads the establishment of the Geotechnical Engineer in the future, which be realized in the next years in UPT with applying in our university of Bologna's charter.

Problems for the future.
The main activity of AGS shall be on the organization of:

- Common activities with the Balkan's Geotechnical Societies,
- Organization of common activities for young civil engineers and civil Engineering Faculty.

Year 2007

- *4-th International Conference on Earthquake Geotechnical Engineering. June 25-28 2007 Thessaloniki Greece.*

- *XIII Pananerical Conference on Soil Mechanics and Geotechnical Enginnering. Juin 16-20 2007 Isla del Margarita Republica Boliviana de Venezuela*

- *XIV-th European Conference on Soil Mechanics and Geotechnical Enginnering. September 24-27 Madrid Spain.*

- *European Conference of Young Geotechnical Engineers. Ancona Italy Septembre 2007.*

4-th International Conference on Earthquake Geotechnical Enginerring.

June 25-28 2007

Thessaloniki- Greece

4ICEC
Thessaloniki-Greece 2007

4-th International Conference on Earthquake
Geotechnical Engineering
June 25-28, 2007
Paper No-1185

Slope stability in active seismic zones in Albania

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Abstract

Albania is a hilly and mountainous country and it is situated in active seismic region. By our studies exists a considerable risk for instability of slopes under static and dynamic loads. From many cases of studies related to landslide activities in Albania results that they occurred in soils deposits of mountain-hill slope, as well as on weathered molasses and flysch rocks mountain-hill slope, which are affected by tectonics and neo-tectonics phenomena. In this paper we would like to present the integrated geological, geophysical and geotechnical studies, the methodology for in-situ testing to make the evidence of different factors which influence in slope instability and respective engineering measures to reach their stability. From analysis of these phenomena we have specified some correlations between soils conditions, soil's characteristics and safety factor.
Keywords: Albanian, AGS, slope, landslide, stabilization, investigation, monitoring.

Introduction

Albania represents a hilly-mountainous country and Albanides are represented the geological structures with possibilities of instable slope and landslide development (fig.1). The slope stability problems are well know in Albania. They are responsible of considerable economic losses in last decade as increased scarcity of land. The Albania is a major seismic area where slope instability and mass movement are common. A lot of mass movement occurred throughout in Albania. They are related to:

Lithology characteristics of rocks and soils.
Geomorphologic conditions of Albanian area.

Physical and mechanical properties of rocks and soils.
Hydro-geological and hydrological conditions.
Seismo-tectonics and seismicity characteristic.

In Albania ¾ area covered by mountains-hill with slopes angle range from 10-20^0 up to 35-45^0 and some places it's more. According to geology most of Albanian territory is built by metamorphosed rocks-schists, argillaceous shale, serpentines and weathered aphiolitic rocks, as well as molasses and flysch rocks, claystones and siltstones intercalated with conglomerates and sandstones layers, whereas related to geotechnics properties, these rocks included in medium and soft strength rocks group. Also, along active faults and neo-tectonic line are concentrated the earthquake epicenter, favor mass movement accorrence. In Albania, the three main longitudinal seismically active zone and three main transversal ones and many neotectonic lines are indentified. Based on the geological formations and landslide body mass can be present following landslides classification in Albania:

- Instable slopes and intensive landslides developed in weathered bedrocks and overburden bed at lakeshores of hydropower plants.
- Instable slopes and intensive landslides occurred in Oligocene flysch formations.
- Instable slopes and landslides occurred in Neogene's molasses formations.
- Landslides occurred in soils deposits of mountain-hill slopes
- Rock falls occurred in steepes of mountain-hill slopes in the weathered and fractured of hard rocks.

Developing of new landslides or re-activation of the old ones is mainly due to construction works. Special constructions, such as hydro-technical works, civil, industrial, urban and rural constructions, as well as constructions in the infrastructure, particularly during last years have destroyed equilibrium in ecological systems through deforestation etc, contributing and creating landslides events. Landslides are located in the soils deposits (diluvia), and in the altered-bedrocks. The slipping bodies of some landslides have very big volume, exceeding 50 million cubic meters. The biggest ones is observed near of Fierza hydro-technical works,

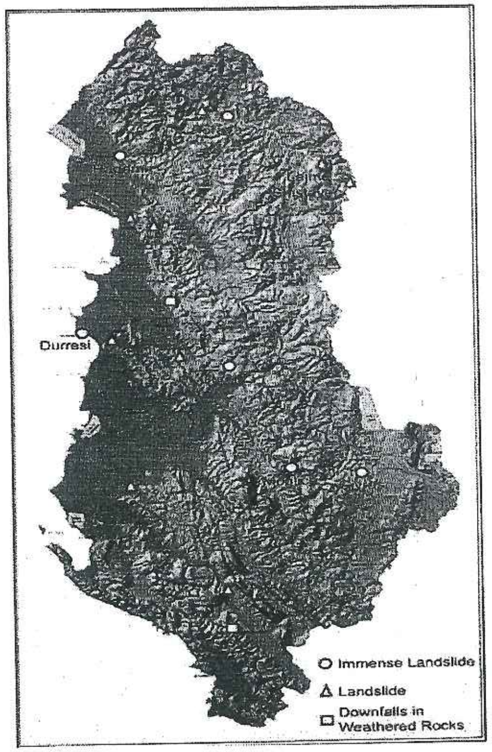

Fig.1 Main location of unstable in Albania

Integrated geological-geophysical in-situ investigation for landslide prognosis, study and monitoring.

In-situ investigations and monitoring for investigation for landslide prognosis, study and monitoring were carried out by integrated engineering geology-geophysics methods.

- Geological Mapping
- Geomorphologic Mapping.
- Hydro geological Mapping.
- Engineering geological Mapping
- Geophysical Mapping, in-situ investigation and monitoring.
  - Gravity micro survey
  - Magnetic micro survey
  - High Frequencies Seismic Tomography and profiling.
  - Geoelectric Tomography, electric soundings and profiling etc.
  - Electrical, radiometric, sonic etc, well logging.
- Laboratory analysis and determinations.

- Geodesic observations.
- The basic method is the seismic tomography and high frequency refraction seismic profiling. The tomography can be combined with refraction seismic profiling of high frequencies at the different sectors of the landslide area.
- The natural seismic-acoustic activity inside and outside of slipping body is necessary to observe. According to the survey's data the velocity of-waves (V_p) and S-waves (V_s) can be calculated as well as, layer thickness. Also, by seismic data, the physical-mechanical properties must be calculated for the soils and rocks as Poisson coefficient, elasticity dynamic modules of Bulk modules, rigidity modules and module of compression volume strength.
- Electrical soundings can be performed by the Schlumberger array.

In situ geophysical investigation and monitoring are programmed to perform in three phases:

- Surface integrated geological-geophysical survey and installation to perform is three phases.
- Drilling of shallow boreholes, cross-hole seismic survey and well logging. Periodical geophysical survey and geodesic observation in boreholes and on ground surface.

Consequently, geophysical-engineering studies have a complex character:

a) To prognoses slope instability and landslide development possibility in the future.
b) To study the landslide body structure and soil of the landslide area.
c) Evaluation of in-situ physical-mechanical properties of soils and rocks and
d) In situ monitoring of landslide phenomena.

Discussion and analyses

Landslides at the lakeshores of the hydropower plants.

Hydro-technical works in Albania are generally constructed in conditions of rugged terrain and in geological formations in which the land sliding phenomena is often present (fig.2) The land sliding phenomena develops in the basement rocks and the overlaid loose sediments. This phenomenon has been more evidently activated after the construction of hydro-technical works. The exploitation period of more than 25 years of such a huge hydro-technical work has influenced to the physical-mechanical properties at various parts of this landslide.

The Porava Landslide.

A study conducted in the Firza hydropower plant, constructed over the Drini River in northern part of Albania, is a clear example of it. This hydropower plant was building in 1974 and has an installed capacity of 500MW.

The lake, created after the construction of the plant, has a water volume of 2,7 billion m^3. The hydropower plant consists of several complex hydro-technical works.

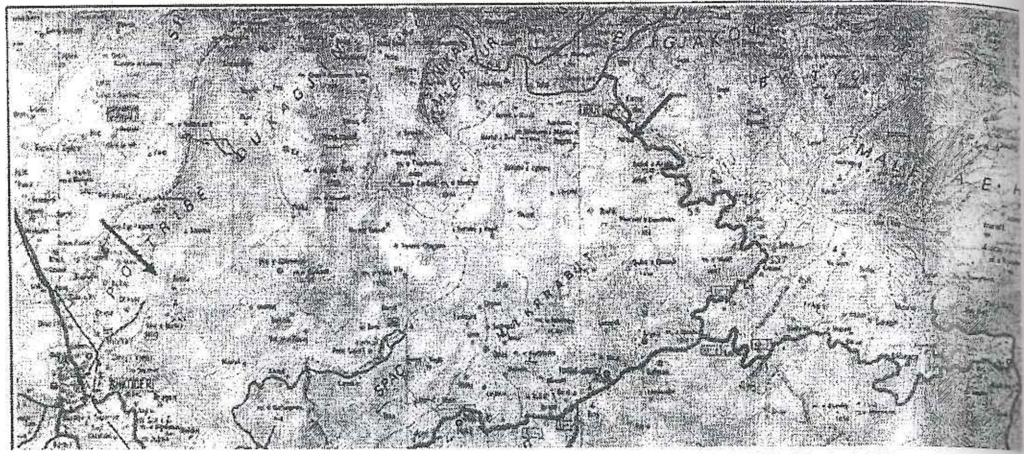

Fig.2. Ragami and Porava landslides location, scale 1:300000 (Drini River Basin)

In fig.5 is presented the detailed geo-electrical-engineering section. This section was compiled based on the date of the vertical electrical soundings. In that can bee noticed the presence of the very heterogeneous electrical medium in strike and depth. There are two categories of geo-electrical borders in the profile. These are primary borders, connected with the separation of the main zones of the slipping body (with that of deepest plains 140-160m deep and with that of the most superficial plane 20m deep).

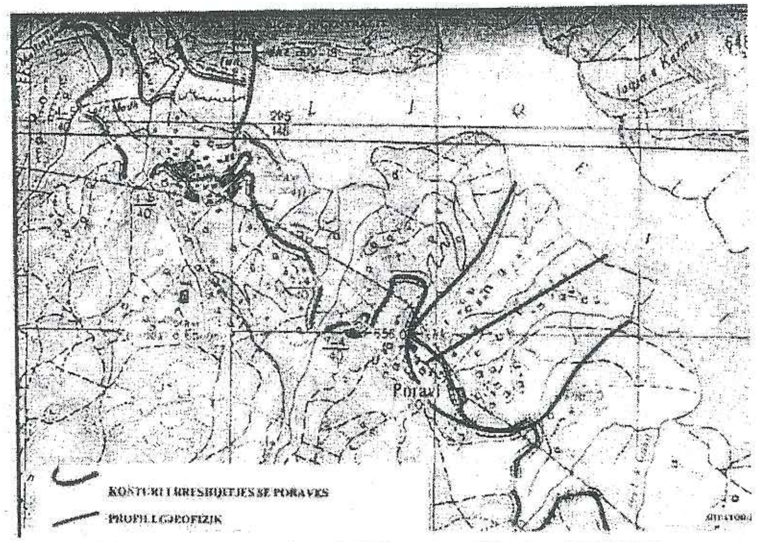

Figura.3 Prova landslide area (Scale 1:25000)

Many buildings have been destroyed. This phenomenon has been more evidently activated when hydro-technical works started to be used. During the exploitation period of more than 25 years, the huge hydro-technical works influenced the physical-mechanical properties in the shore area and caused a series of landslides.

The geology of Poravi area consists of soils, silt-clay mixed pebbles, and rocks, which are composed by schist's and argillaceous shale's formations, The Poravi landslide occurred on left side of the Drini River valley slope (actually artificial lake) that has an inclination angle 16^0 up to 25^0. The slide body consists of silt-clay mixed pebbles in upper part and basalts rocks blocks in lower part. The dimension of it is 1000-1500m long, 500-700m up to 1000 wide and 50-70m up to 120-125m thick with total volume $34000000m^3$. The slides plane is situated on red argillaceous shale. As result of landslide occurrence several engineering object (village's buildings and road) are demolished and damaged. Special attention has been paid, since the projection period of this study, to the big slides in the shore of the Fierza Lake, especially to the Porova one (fig.4). The studies have not only included the geological understanding of the shore's solidity but also the understanding of the landslides, They also include solidity-integrated calculations through the hydraulics patterns. For that, the body fall of the Porava landslide at different speeds (from 5-10m/sec) was simulated. As calculating parameters were used the ones resulted from geological studies of that time. All those studies brought to the conclusion that the dike should be raised 12 more meters over the one initially determined in the project, so that it would be more secure.

Today, based on the data generated from geophysical surveys, the geological knowledge about this zone and the visual study of the actual situation of the Porava landslide, it was realized the respective analysis of these integrated geophysical works.

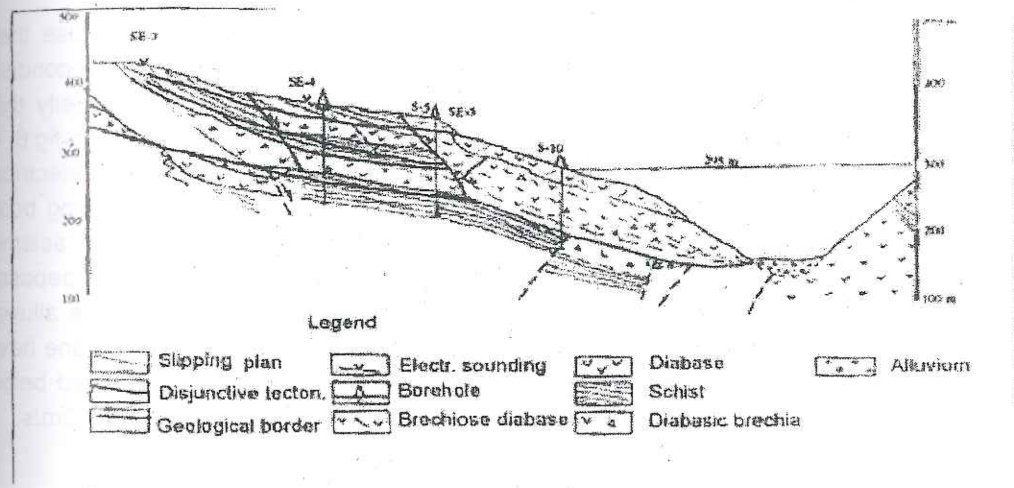

Fig.4 Geological (L.Dhame, 1974) and geo-electrical data comparison, Prova landslide (Frasheri A.etc.1998)

In fig.5 is presented the detailed geo-electrical-engineering section. This section was compiled based on the date of the vertical electrical soundings. In that can bee noticed the presence of the very heterogeneous electrical medium in strike and depth. There are two categories of geo-electrical borders in the profile. These are primary borders, connected with the separation of the main zones of the slipping body (with that of deepest plains 140-160m deep and with that of the most superficial plane 20m deep).

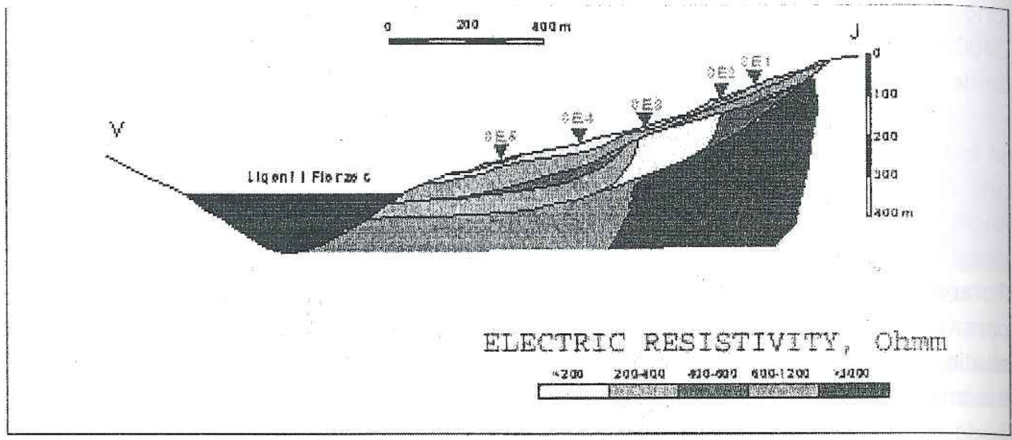

Fig.5 Prova landslide, geo-electrical engineering profile

These slipping plains have very different geoelectrical characteristics, because they have different geological properties. The second category belongs to secondary geoelectrical borders, which clearly express the changes and the heterogeneity that exists in these two slipping planes and in the environment under them. In fig.6 is presented the seismic-engineering section in the same profile with the geo-electrical one. In this figure can be distinguished very well the upper parts of the slipping body (the zone 25m deep). In this section are very well distinguished the two seismic parameters (in the speed of the longitudinal and cross waves). The diluvia deposits have been fixed with V_p =400-1200m/s and V_s = 150-450m/s values, while the alluvial deposits and volcanic rocks of the most upper part, located over the slipped plane have V_p = 800-3880m/s and V_s = 350-800m/s values. The volcanic deposits located below the first slipping plain have been fixed with V_p=1400-3800m/s and V_s = 600-1500m/s.

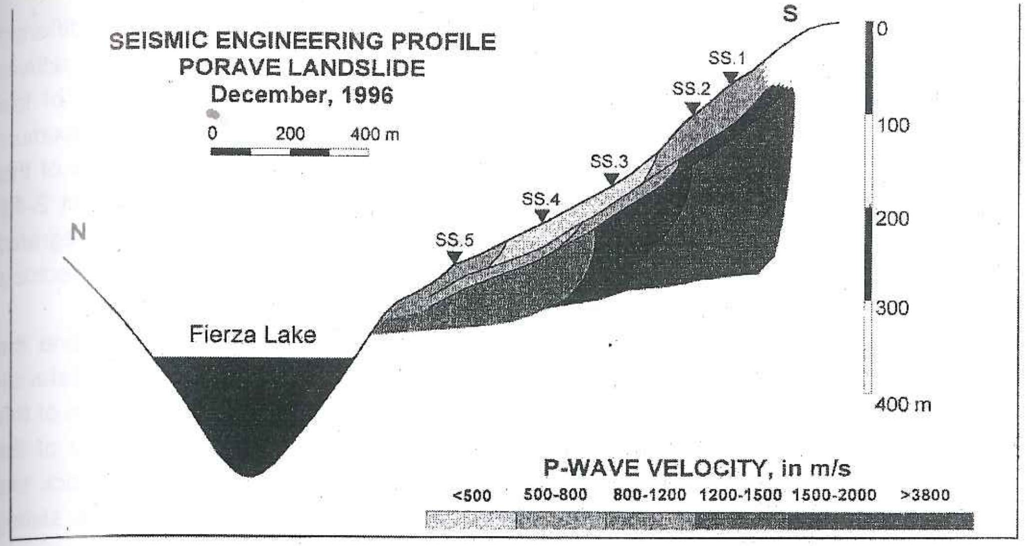

Fig.6a Seismic profile of Porava Landslide

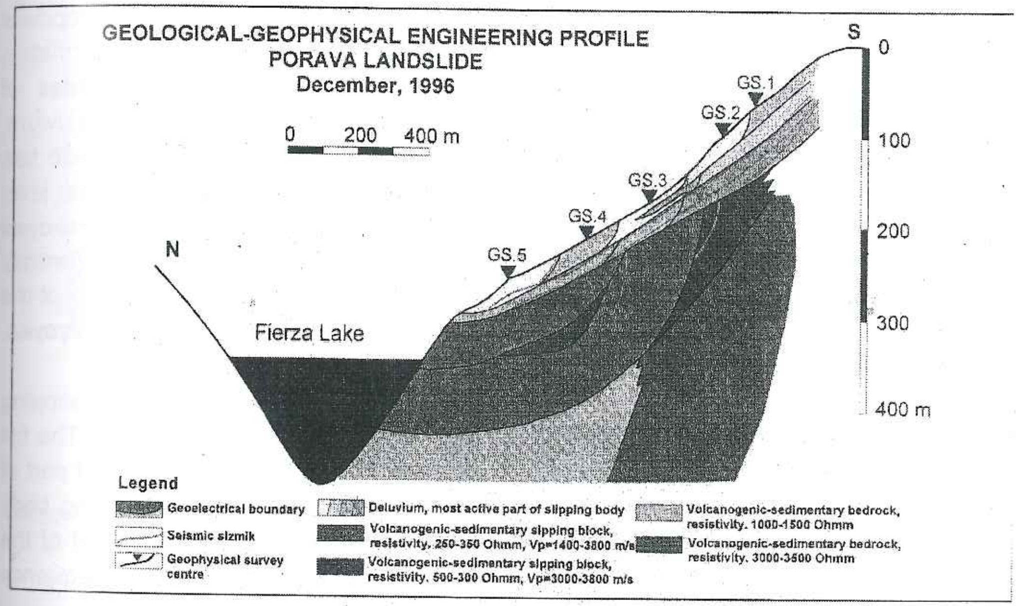

Fig.6b Porava Landslide geological-geophysical engineering profile

Based on the seismic parameters, the evaluation of the physical mechanical characteristics of the rocks of this sliding body was carried out in strike and depth. In this seismic section and in the geotechnical one, can be seen the block kind nature of the upper part of the slipping body and also of the lower part of this body in the

basement volcanic rocks. By studding the natural seismic-acoustic activity, different recordings can be noticed in all the surveying zones. The shows that the sliding activity is different for different parts of the slipping body. The most dynamic zones of this sliding massif are located in places where the micro-movements have maximum intensity values. The Porava village is located in one of these zones. Because of this activity, many houses, and the soil is damaged and slopes have moved about 2-4m within a 2-3 years period of time (1994-1996). In the detailed and integrated geophysical engineering section, can be noticed a concordance between the electrical sounding results and the seismic surveying ones , used for studying this slide.
Also in this section can be determined sliding plains, their nature, situation and the content of the two parts of the slipping body. The most upper part is made of deluvial-eluvial deposits and reaches up to 20m deep, above the first most dynamic plain of this zone. Under this lays the volcanic rock massif, located over the deeper plane of the Porava landslide (100-160m). This plain is determined and separates the block like sliding body from the volcanic rocks, which have not been touched by this sliding activity.

The Ragami Landslide

The typical landslide was developed at lakeshore of the Vau Dejes Lake of Hydropower Plant in Northwestern Albania (fig.7). It is developed in the ophiolitic formation represented by serpentine rocks. The slipping body represents a big mass of serpentines, which is elated, destroyed and covered by a thin layer of diluvium. According to the geological survey in 1992, the landslide did not exist. Landslide has been significantly developed during the last years. The yearly movements of water level at Vau Dejes Lake caused a big landslide at elated, weathered and destroyed serpentine rocks. Slipping body increased in the extent and in the volume substantially during this period. The front part of the slipping body is located along the shores of the lake. This part has the shape of e scrap about 2-3m high and represents a destroyed, schist one serpentines, partly in a from of mylonite.
In fig.8,9 are given the integrated geophysical-engineering sections of the slipping body. Two main sliding plains separate this body. These plains are broken up. The fist plain is at depths of 5-7m. while the second one reaches up to 22m. The lowest part of the second plain touches the lake, under the water level. In this way, the sliding body has a block like nature. The physical mechanical properties of the rock massif of the slipping body are lower than those of the basement rocks, not have a wide frequency band, while outside the body there is not such activity

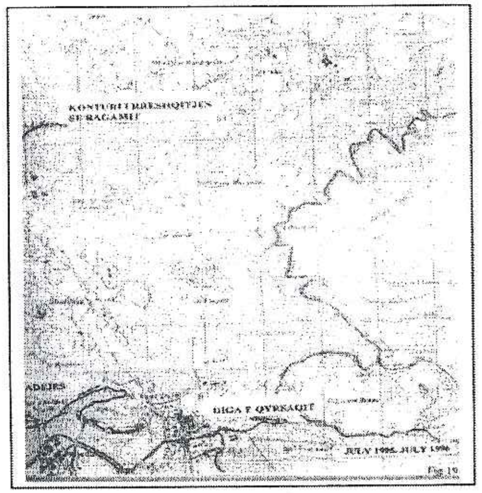

Fig.7 Vau Dejes hydropower plant and Ragami landslide location

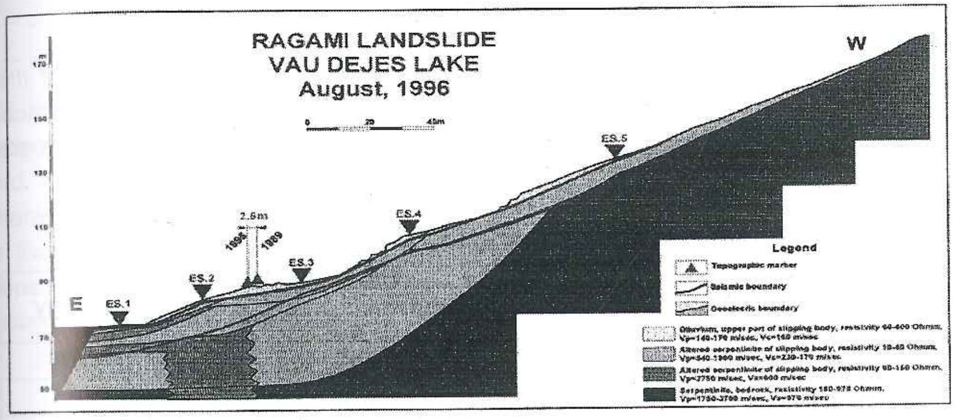

Fig.8 Engineering integrated geophysical transversal section, Ragami landslide

Three failures in different superficial levels can be observed in this landslide

- The first one 35-45m from the shore, with a horizontal dislocations of about 2m.
- The second one about 70-90m from the shore, with a vertical jump of about 2m
- The third one about 115-130m from the shore. This is the newest level and has the lowest amplitude.

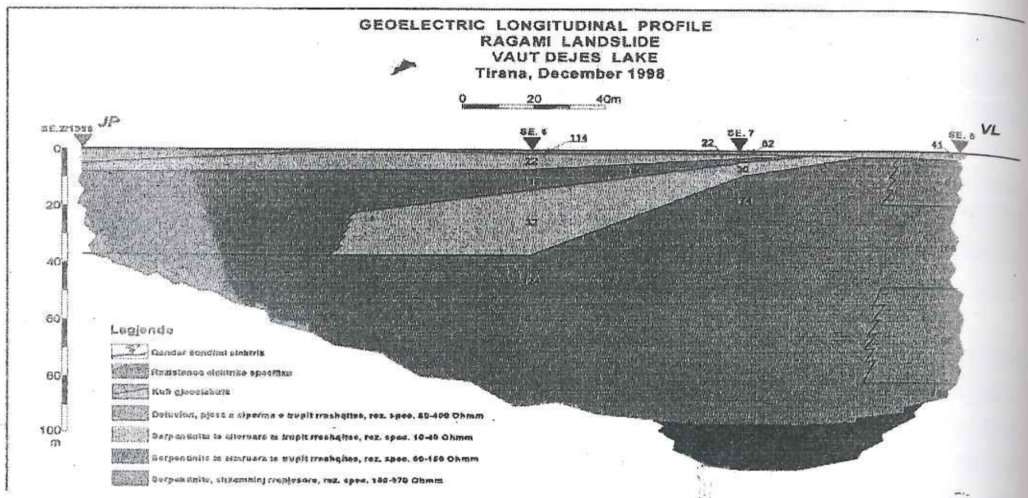

Fig.9 Engineering integrated geophysical longitudinal section Ragam landslide

The physical-mechanical properties of the slipping body are lower than those of the basement rocks, not touched by the sliding phenomena. Physical mechanical properties of rocks in the area of Ragami Landslide according to the seismic data are presented in Table 1 and Table 2.

Table 1 Physical properties in Ragami landslide's area

Layer Number	Thicknees in meters	Resistivity in Ohm	Density in g/cm^3	Wave Velocity in m/sec		Lithology
				V_p	V_s	
			Slipping body			
1	0.7	76.4	1.34	210	160	deluvium
2	4.0	29.5	1.61	540	230	Breaking serpentinite
3	6.5	46.5	2.45	3700	680	Water bearing sepertinite
4	17.4			1500		Breaking serpentinite
			Bed rocks			
		485	2.56	3500	1920	Serpentines

Table 2 Mechanical properties in Ragami landslide's area

Layer Number	Poisson's ratio	Dynamic modulus of elasticity E_d^s in 10^5kg/cm^2	Rigidity modulus G, in 10^5k g/cm^3	Volume compression σin 10^5 kg/cm^2	Rocks state
			Slipping body		
1	0.35	0.00370	0.00140	0.00420	Soft rocks
2	0.39	0.02413	0.00868	0.03630	Destroyed shattered rocks
3	0.48	0.56586	0.19167	3.26503	Cleavages and fissured rocks
4		0.26325	0.09608		Destroyed shattered rocks
			Bed rocks		
	0.29	2.46271	0.96199	1.91408	Compact rocks

As documented in Table 1 and 2, four layers with different physical-mechanical properties create the slipping body. First layer represents the diluvia cover. Layers 2 and 4 are represented by destroyed-serpentines. The third layer in between is characterized by low electrical resistivity and low shear waves velocity. Is corresponds to the water saturated cleavages and fissures in the serpentines, The dynamics of slope movement is also reflected in the natural seismic acoustic activity. The micro-movements in the slipping body are very intensive and have a wide frequency band.

Influence of Soil conditions to factor of safety

Re- activation of the old landslide has been from effect of the water oscillation in Vau Deje and Fierza lakes and the seismic activity in these zones. By these phenomena we have:

a) The softening of soil in the contact planes between sliding body and rock basement. There resistance has decreased 40-50%.
b) The appearance of the water flow in the slope and reduction of the safety factor from 1,5 to 1,0.
c) The presence of water in the failure surface, reduce the effective stress and shear stress is in the minimum value (estimated by combination's methods seismic data and laboratory testes).

$$c_{\min} \cong 0,5c$$
$$\varphi_{\min} \cong 5^0 - 7^0$$

The soils in the sliding body were categorized in second category by seismic properties. The epicenter of the earthquake is 30-50 km and the frequency of the earthquake was M=5,5-6,5is 25-30 years. In these conditions in the sliding masses to act a horizontal inertial force K.W (with K=0,12-0,2, W – Wight) which has favored activation of landslide.

Conclusions

1. Based on the results of this integrated geophysical engineering and geotechnical study result for Porava landslides
   a) There could nor happened an immediate fall at any speed of the Porava slipping body.
   b) Even in cases of powerful earthquakes, the slipping body mass can not fall as a whole, because it is made of broken up block masses. It can fall parts by parts or in fragments. Natural or inductive earthquakes of normal intensity which happen often in this region, till now have not caused massive detachments of the slipping body.
2. After analysis of geophysical investigations in Ragami landslide, have been concluded:
   a) Thick and high volume slipping bodies represent the Ragami active landslide in the shore area of the Vau Dejes Lake.
   b) The extent of the landslide and the position of sliding plains were precisely fixed using the integrated geophysical survey.
   c) The block like character of the sliding bodies brings to conclusion that the block o these bodies can not fall down immediately in any kind of velocity.
   d) In these zones it is necessary to construct buildings which resist large deformations, or the construction must be prohibited.
   e) In the zones which the failure surface is 5-10m depth, it can be used engineering measures as piles, anchored sheet piles, retaining walls, drainages ect, for the stabilization of the situation.
   f) Combination of all methods (geological, geophysical, geotechnical) for monitoring, investigation and study of slope stability have big advantages for the future development of this zones and for the minimization of risk and material damages.

REFERTENCES

Dhame L — 1974. Information about shore stability of Drini River at Porova zone. Ministry of Construction Archive , Tirana 7.1.1976

Bozo L — 1989, Bahaviour of soils and foundations under seismic actions Ph.D.Thesis. Polytechnics University of Tirana, Albania

Bozo L — 2000 Rock Mechanics, University Book Edition, Tirana Albania

Bozo L — 2004 Soil Dynamics University Book Edition, Tirana Albania

Frasheri A — 2005 Engineering and environmental geophysics (in Albania). Published by Academy of Sciences of Albania and Faculty of Geology and Mining. Tirana.

Frasheri A Kapllani L
Dhima F 1998 Geophysical landslide investigation and prediction in the hydrotechnical works. Journal of the Balkan Geophysical Society. V.1 N.3 August 1998

Muceku Y Reci H — Mass movement and risk assessment in Komani hydropower Dams and reservoirs. Societies and Environment of 21 Century. Proceedings of the International symposium and dams in the Societies of the 21 Century ICOLD-SPANCOLD,18-23

Muceku Y ect — 2004 Study of slope stability at Komani and Fierza Hydropower Stations (Ministry of Industry and energy).

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FECHAS IMPORTANTES

Entrega de resúmenes:	28 de Abril de 2006
Aceptación de resúmenes:	2 de Junio de 2006
Artículos para revisión:	29 de Septiembre de 2006
Entrega de Trabajo Final:	5 de Enero de 2007

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IMPORTANT DATES

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Articles for review:	September 29, 2006
Final Paper due:	January 5, 2007

BOLETÍN No. 1
PRIMERA COMUNICACIÓN E INVITACIÓN PARA RESÚMENES
XIII CONGRESO PANAMERICANO DE MECÁNICA DE SUELOS E INGENIERÍA GEOTÉCNICA.

BULLETIN No. 1
FIRST ANNOUNCEMENT AND CALL FOR ABSTRACTS
XIII PANAMERICAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

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XIII Pan-American Conference on Soil Mechanics and Geotechnical Engineering
ISLA DE MARGARITA 16-20 de Julio 2007
Republica Bolivaria de Venezuela
Topic: Evolution of Geo-Engineering education In Albania

Theme: Evolution of Geo-Engineering education In Albania

Prof.Dr.Luljeta BOZO Faculty of Civil Engineering Polytechnic University Tirana Albania

Abstract

In Albania are graduated Civil Engineers from 55 years ago (since 1951). On 1987 we have founded the Geotechnical Sector which has done a voluminous work in the field of education of geotechnical engineers. In this paper I would like to present the fields into which we have worked in the Faculty of Civil Engineering, the main constructed objects from Albanian engineers of geotechnical specialty, the main direction of today's work the close collaboration of Albanian Geotechnical Society with respective department of the Civil Engineering Faculty and the issues which we should resolve in the future.

Introduction

In Albania are prepared geo-engineers since at 1951 when are created in Tirana the Polytecnical Institute. At the start with geosciences are occuped the civil engineers and later also the geologists. In this period we have the beginning of the Geotechnical Sector with the subject "Basement and foundations"
At 1957 are created the "Statement University of Tirana and the geotechnical matter are developed in form of two subjects "Soil Mechanics" and "Foundations".

At 1987 are created Geotechnical sector in the Civil Engineering Faculty of Polytechnic University. Since them the geotechnical matter are developed in form of the subjects:
Soil Mechanics and Rock Mechanics – Geotechnics I
Technique of the foundations - Geotechnics II
Soil dynamics – geotechnics III
Till the 2005 years the geotechnical subjects are developed in some faculty and in some their branches (fig.1).

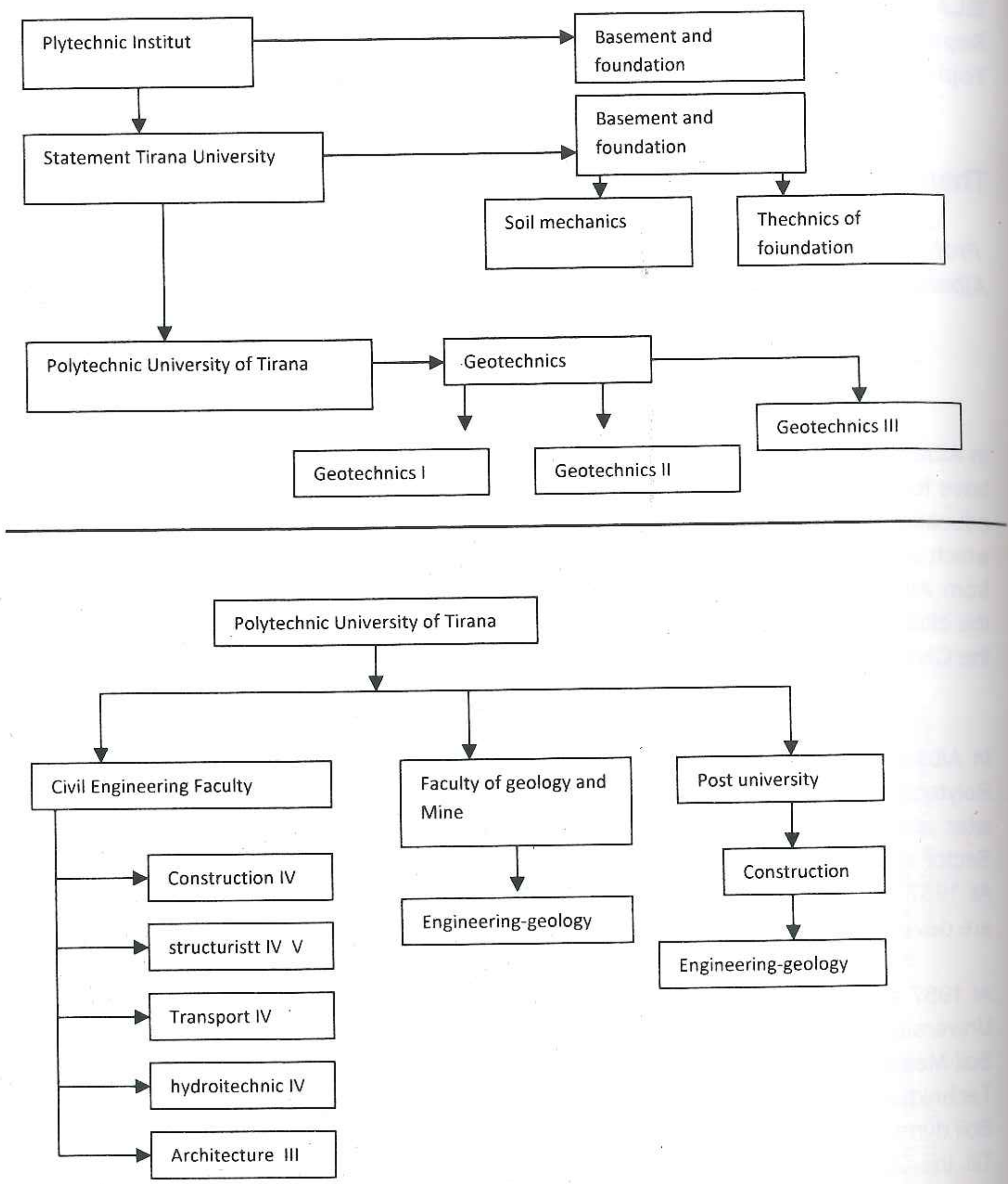


Fig.1 Schema when are developed geotechnical subjects

At 2005 we have implemented in Albania the new system of Bologna. In the first cycle (3 years) are developed the Geotechnics I- Soil Mechanics (with six credits) and

geotechnics II –foundations (with six credits). In the second cycle will be treat six new subjects.

- Rock Mechanics with 2-3 credits
- Soil dynamics with 2 credits
- Deep foundations with 2 credits
- Slope stability with 2 credits
- Road's geotechnique 2 credits
- Improvement of soils and protection of the objects from damages with 2-3 credits. At 2000 years are created the Albanian Geotechnical Society, which closely collaborating with Geotechnical Sector of Polytechnic University and this is a big impulse for augmentation of quality in the field of geotechnical education

The main directions for geotechnical education

The main directions in the field of geotechnical education are:

- Preparation of highly qualified civil engineers. For this reason till now we have prepared and published 18 books in the geotechnical field, it is increased the actual level situation of foundation's design. Also increased the number and the kind of diplomas work of students (now we have 25-30 theses for the geotechnical problems).(fig.2)

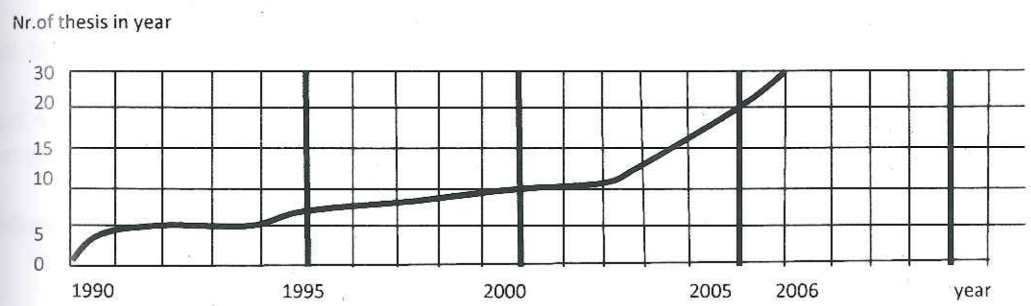


Fig.2 Number of diploma's work during the time

- Augmentation of the qualification of civil engineers in the geotechnical field. This is realized in the three aspects or directions :
  - Qualification of pedagogical staff of Civil Engineering Faculty
  - Post qualification of civil engineers as masters or doctorate thesis

- Rapid qualification of civil engineers for special themes by short courses or seminars.
- Fastening of academics with practitioners which are very good effects and results by:
- Supplying of academics with important and difficult problems which are developed the scientific researches.
- Augmentation of theoretical level of the practitioners.
- Consolidation of the liaison of the school with practice and the continuity of school consolidation.

The main objects realized and construsted by Albanian engineers.

By good professional education in the geotechnical field the Albanian civil engineers are resolved very complicated and important technical problems: Connected with very special geology of our country and great request for all kind of constructions in Albania was developed country.
In Albania can to find all kind of geological formations (from rocks to very weak soils)
The old dictatorial system was obliged the civil engineers and designers to designed and constructed oneself all types of constructions, as civil, hidrotechnical, military, agricultural, industrial objects ect.

So after the second world war (1994) (when Albania is totally destructed) until 1989 year from our engineers are designed and constructed:

- A great deal of tunnels as for railways, automobilistic, hydrotechnic, and for other intensions
- Very much dams (over 50% of them can to consider high dams) for the hydropower plants, (hydroenergitic reasons), for irrigation , tailing dams and for other intensions.
- Hundreds of industrial objects for the mechanical, metallurgies, mineral, textile, nutritive industries ect.
- Hundreds of schools hospitals, and social objects
- Hundreds of reclamation objects and very big irrigation system.
- Hundreds km of automobilistic roads and railways, bridges, many ports and airports.

All the objects designed and constructed by Albanian engineers is results of their good education in the geotechnical field, the closely tie between school and practice, the scientific leadership of university on the solution of the difficult and complicated technical problems of practice.

The main direction of work to day.

During the 16 last years (1990-2006) Albania was entry on the new development line to shake off the dictatorial system and rigorous centralized economy.
The private property, the market economy, the free rivalry are defined the new economical development of Albania and in this occasion (cadre) the development of studies and designes of geotechnical objects and their realizations.
To day we are working in this main directions on the geotechnical field:
- Infrastructure of the transport (roads, highway, railway, motor way) and especially for choice of the suitable or adapt natural to constructed the roads, and investigations to made the best compaction of soils.
- Studies of many instability slopes (which are much moore in Albania) From this phenomena has damaging the cultivated lands, much villages, inhabitant zones, road's infrastructures ect.
- Design and construction or reconstruction of airports, and ports for touristic and commercial reasons.
- Studies for design of underground structures with 2-5 stories in urban zones such as sheet-piles walls, piles, concrete walls teefback structures etc.
- Studies and design of big bridges, subway and underway particularly in the great town as Tirana, Durres, Vlora etc, and their realizations,
- Monitoring, maintenance and management of dams, because in Albania are constructed much of 600 dams.
- Erosion activity of rivers and all dangerous phenomena's tied with them as scour of foundations, slope instability etc.
- Studies and design of watering systems, water pot, pumping station etc.
- The development of touristic village in hills, mountains, zones, or sea side.

The collaboration of AGS (Albanian Geotechnical Society) with Geotecnical Department of Polytechnic University.

This collaboration to lay in two directions.
- To furnish the new information and the new good results in the geotechnical field. For that purpose every year AGS and Geotechnical Department are organized for the civil engineers and geologist the seminaries or the short courses. For the special themes have referred academics and specialized persons from industry. AGS has organized till now six seminaries and short courses with themes:
- Geotechnical investigations and infrastructure of transport.
- Pile foundations in static and dynamic loads.
- Rehabilitation and management of roads.
- EC-7 discusion and application
- Earth geotechnical problems and EC-8
- The assistance of quality of design and construction of roads.

- ❖ Publication of the scientific work of our authors in our periodical review "Gjeoteknika" which are published since 2001 years. In that review are published also the work of young geotechnical engineers on the special rubric.

Our authors are participated in many international conferences and workshop with their papers. Since now Albanian professor and engineers has participated in following international events:

USA	three times
UK	one time
France	three times
Austria	one time
Cesk Republik	one time
Slovenia	one time
Greece	two times
Japan	one time
Canada	one time
Letonia	one time
Spain	one time

The issue which AGS should resolve in the future.

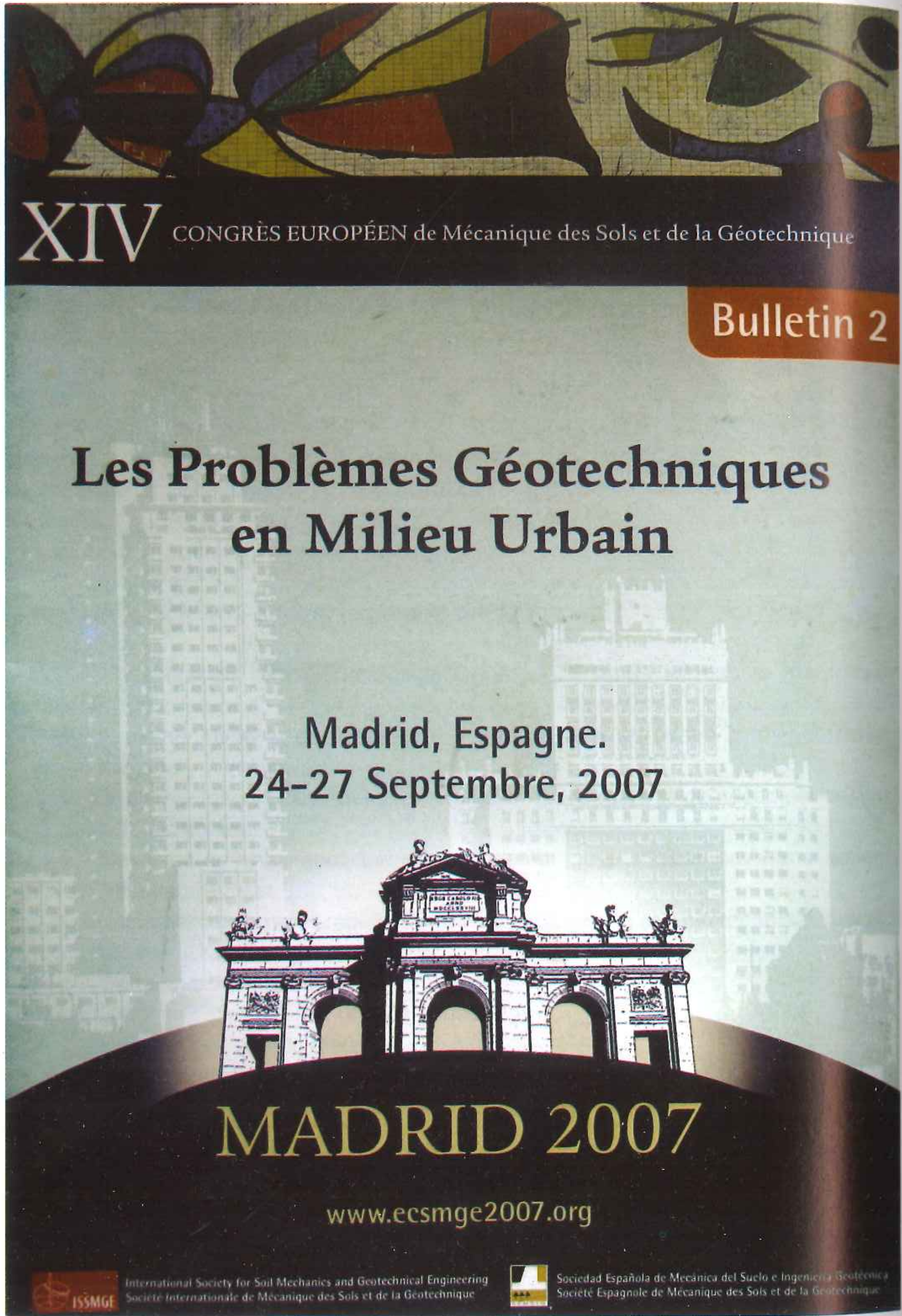
AGS in collaboration with Geotechnical Department things to resolve the following problems in the future:

- ❖ Implementation in our engineering practice of Euro code -7, for geotechnical design.
- ❖ Implementation in our design of rules of EC-8 (part of geotechnical design in seismic zones).
- ❖ Resolved in maximum the environmental geotechnical problems which in Albania are most.
- ❖ Implementation moor rapidly of new technologies in our geotechnical structures as:
- Deep improvement of soils
- Retaining structures in urbane zones.
- Using of geosintetic materials.
- Cleaning and rehabilitation of contaminated zones.
- The new methods for compaction of soils as roads layers.

Conclusions

- ❖ The problematic in the field of study of geotechnical phenomena's is very large and complicate

- ❖ To widen one's knowledge and geotechnical educations is a imperative duty because every geotechnical solution is original and don't reappear himself (themselves)
- ❖ Geotechnical education is very large and with different spectrum. It start with very simple technical problems to very complicated as problems of geotechnical environmental, earth geotechnical etc.
- ❖ Geotechnical education can to realized only in collaboration of AGS with:
- Geotechnical Department of Polytechnic University
- Other engineering faculty and in particularly of Geolognical Faculty.
- Other geotechnical Societies and ISSMGE
- National and International Industry.



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The influence of geological and geotechnical condition on reinforcement structures for deep excavation nearly existing structures

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Abstract

Last year in Albania are constructed many high buildings closed to engineering objects as roads, buildings and schools etc. The new buildings have two or more underground stores. In this case the main problem is accomplishment of detailed geological and geotechnical investigation and the design of sure retaining structures. In this paper we would like to present some of these problems in Durresi town and their solutions in difficult ground conditions, as well

Resume

Les annees dernieres,en Albanie,sont construi les haut bâtiments près du exist edification comme les routes, le bâtiments, lécole ect. Les noveaux bâtiments ont deux on plus etage sous sol. On cette occasion le plus important problème est accomplissement de recherche geologique et geotecnique en detail et projection de la structure de soutenement sûr. Ici nous voulons presenter quelque de cette problemès en ville du Durres et leurs solutions en difficile conditions des sols

Keywords: concrete piles walls, concrete sheet-piles walls, retaining structures, geotechnical properties

Introduction

The Durresi town is one of very important in Albania (after Tirana). Simultaneously the development of roads infrastructures, enlargement of the main port of Albania, we have extraordinary development of high buildings (10-12 stores and more). According to the lithology of the Durresi area is built by sandy-silty soils with high underground water table. It's included in active seismic zone (Magnitude is 6.7-6.9). One part of the construction for the technological and seismic reasons is built 1-2 stores below earth surface. From our studies (1, 2), result that sandy soils of the Durresi area can be liquefied under dynamic loads. The depth liquefaction is 1.5-3.0m. Mean time the new buildings are constructed closed to existing buildings, roads etc. For that reason it is designed and constructed retained structures in conformity with geological and geotechnical conditions and in order to save existing buildings. The retaining structures are secured the normal working conditions in the time of the excavation and good conditions during the construction and exploitation of new object.

The retaining structure types in albania

By geological and hydrogeological conditions, as well as, geotechnical parameters in Albania are realized various retained structures:

Concrete walls in earth (soils) until 5 underground floors in Tirana realized in cohesive soils. Concrete piles realized in many soils types (cohesive and noncohesive), which are applied in Tirana, Durresi, Vlora and Fieri etc.

Concrete sheet-piles walls cantilevered and anchored.

Metallic concrete sheet-piles walls.

Braced tieback walls.

The geological and geotechnical researches

System of buildings to be forecaster with two underground floors and 18-20 stores high. For realized the design of underground structure are accomplished 7 boreholes up to 30m deep for the geological and geotechnical investigations purpose. All disturbed and undisturbed samples are tested in ALTEA laboratory. The new buildings are constructed closed to the former buildings (Fig.1).

Fig. 1. Its show view of a geotechnical investigation closed to former building

From the boreholes logs we have compiled the lithological profiles, in which is defined six layers with different geotechnical properties.

Layer 1, Old embankment with 2-3m thickness. This layer is not suitable to establish foundations.

Layer 2, Fine sands and silty sands, beige colors, with 25-30% kelp (sea weed) and 1.2-2.6m thickness.

Layer 3, Fine sands and silty sands, grey-blue colors, with 1.5-1.9m thickness.

Layer 4, Medium-course sands, grey-blue colors, with 1.2m thickness.

Layer 5, unconsolidated claystones and siltstones rocks, with 7-7.5m thickness.

Layer 6, Soft rocks-claystones and siltstones, with 100-250m thickness.

The underground water table is up to 0.8m deep from earth surface and it's characterized by the aggressive properties. The buildings are constructed along of Adriatic seaside on soils, which are represented by sands mixture with organic matters. These soils have a thickness various 5-10m (Quaternary deposits). Also the buildings are built in eastern part of Adriatic seaside on soft rocks-claystones and siltstones. Finally from geologically investigations we have prepared two geotechnical models (Fig. 2.a and 2.b). The mean value of the first model are given in Table 1a and 1b, and second one in Table 2a and 2b.

Table 1a

Layer s	Grain size analyses			Natural unit weight	Internal Friction angle
	Sands	Silts	Clays		
	%	%	%	γ-KN/m^3	Φ °
1	-	-	-	-	-
2	23.6	57.6	18.8	18.6	22
3	67.5	20.1	11.4	19.4	26
4	75.6	14.8	9.6	19.8	28
5	2.3	57.9	39.8	22.0	26
6	1.8	55.2	43.0	22.2	28

Table1.b

Layers	Cohesion c- kPa	Deform. Modules E-kPa	Bearing capacity E-kPa	Plasticity index	Cu
1	-	-	-	-	-
2	-	$0.65x10^4$	150	-	-
3	-	$0.8\ x10^4$	160	-	-
4	-	$1.2\ x10^4$	180	-	-
5	40		300-350	21.5	300-385
6	120		400-450 $R_{.sh^*(3-3.5)}\ x10^3$	-	400-500

The mean value of the second model are given in Table 2a and 2b

Table2a

Type of layers	Grain size analyses		Natural unit weight	Internal Friction angle	SPT N
	Silts	Clays			
	%	%	γ-KN/m³	Φ °	
A - Non cohesive	33.8	66.2	18.6	24	10-12
B Cohesive	10.9	89.1	20.68	22.4	115

Table 2b

Type of layers	Cohez c- kPa	Deform. Modul E-kPa	Bearing capacity E-kPa	Plast index	Cu kPa	Qu kPa
A - Non cohesive	-	-	-	-	-	-
B Cohesive	48.9		820	19.38	410	820

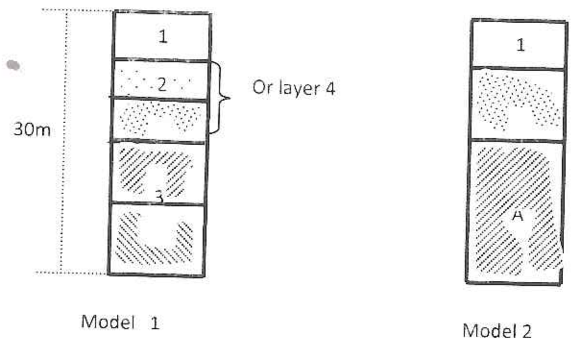

Fig. 2.geotechnical models

For accomplished geotechnical study we have carried out the followed laboratories tests in ALTEA Laboratory:

- Grain size analyses-sieving and hydrometer tests
- Natural water content
- Specific density
- Bulk density
- Dry density
- Atterberg limits (LL, PL, I_p)
- Shearing strength (internal friction angle, cohesion)

- One consolidation test or edometric test
- Standard penetration test-SPT

From above tests carried out on collected samples from studied area results:

- Two characteristics of grain size distributions (Fig.3) the number 1 for the layer 2, 3, 4 in model 1 (layer A in second model) and number 2 for the layer 5, 6 in model 1 (layer B in second model).

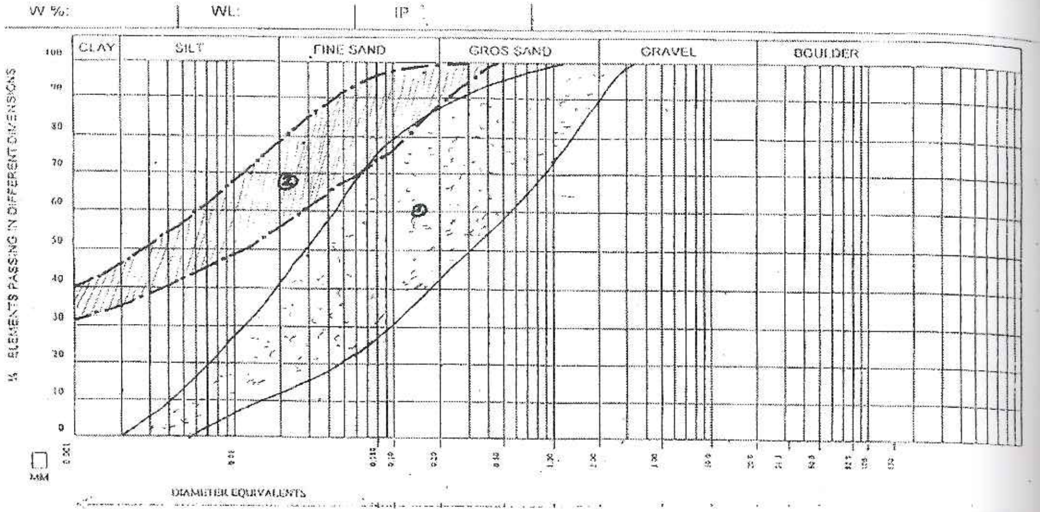

Fig.3. Grain size distributions

- The layer 2, 3, 4, in first model, or layer "A" in second model related to geotechnical properties are very weak soils. It's represent by noncohesive soils with high porosity and high deformation ability under compressed loads (settlements).
- The layer 5, 6, or layer "B" is cohesive soils with good geotechnical parameters. It's represent by good bearing capacity, low permeability and with hard consistency.
- For cohesive soils (which have influenced directly in retaining structures) by the laboratories data we found the following conclusion and relationships.

* The relationship between plasticity index Ip and particles sizes-passes sieve of 2mm and retained sieve 0. 6mm (Fig. 4).

 * The relationship between Liquid limit and particles sizes-passes sieve of Nr. 200.

$$LL = 39.3\left[0.7\left(n-80\right)\right]$$

n-% of fines particles (passes sieve Nr. 200)

The Liquid limit will be increased very rapidly with the increment of fine soils particles (Fig. 5). This relationship is valuable for the soils with n > 80%.

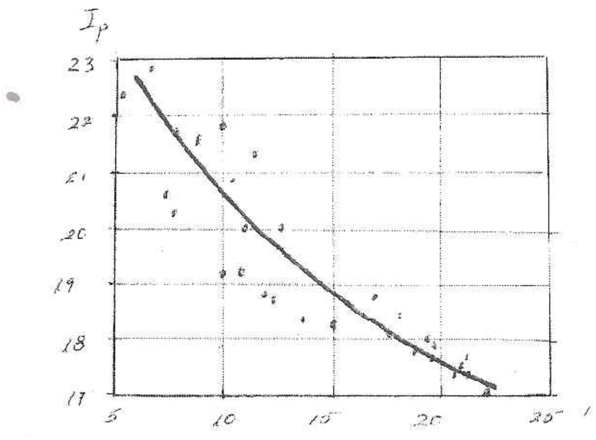

Fig.4. Relationship between plasticity index Ip and particles sizes-passes sieve of 2mm and retained sieve 0. 6mm

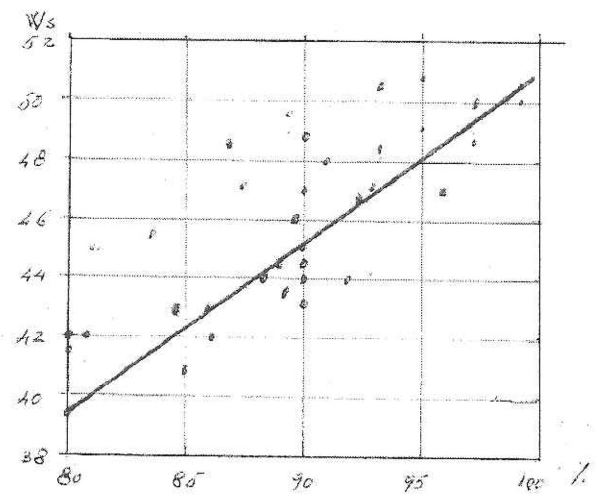

Fig.5. Relationship between plasticity index Ip and fines particles

* The relationship between shear strength "Cu" or unconfined compression strength "qu" and angle of internal friction, cohesion "c", natural water content "Wn" and void ratio "e".

$$Cu = 310 + \left(\varphi - 21^{\circ}\right)20 \text{ (Fig. 6)}$$

$$Cu = 320 + \left(c - 47\right)18.5 \text{ (Fig. 7)}$$

$$Cu = 320 + \left(21 - Wn\right)45 \text{ (Fig. 8)}$$

$$Cu = 320 + (0.68 - e)1285 \text{ (Fig. 9)}$$

Also we have found relationship of:

$$Cu = (3.1 \div 3.5) \cdot N_{SPT}$$

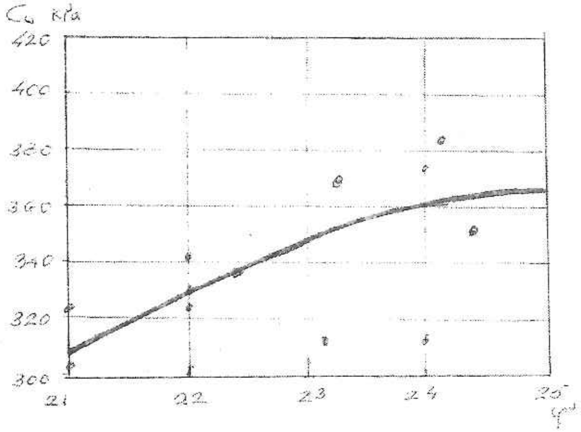

Fig.6. Relationship between Cu and φ (friction angle)

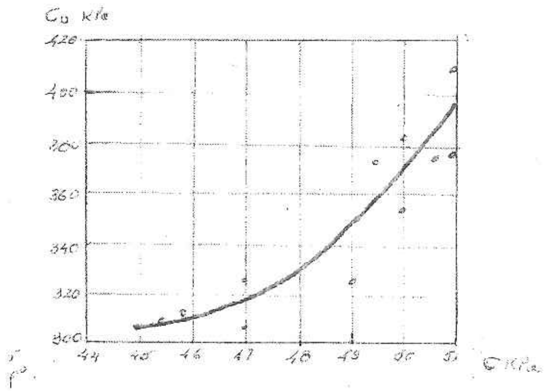

Fig.7 Relationship between Cu and c (cohesion

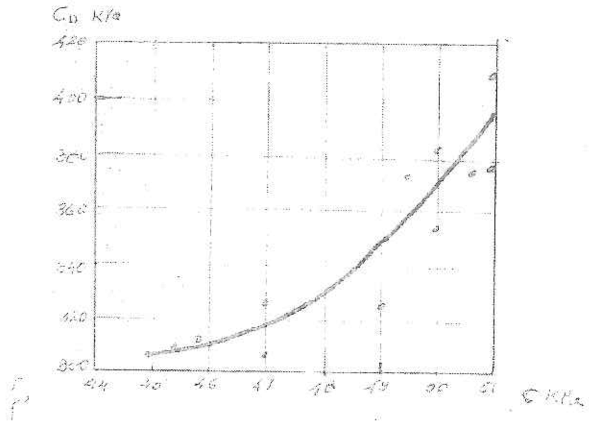

Fig.7 Relationship between Cu and c (cohesion)

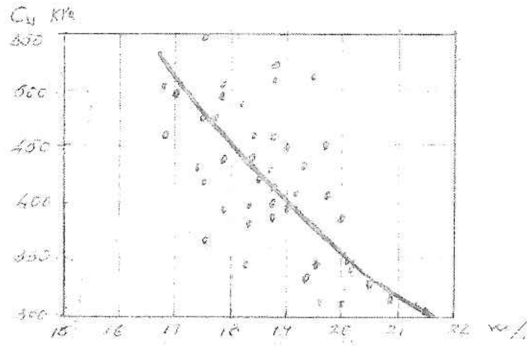

Fig.8 Relationship between Cu and Wn

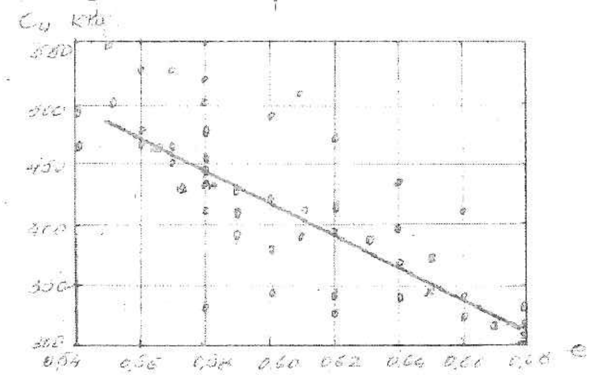

Fig.9 Relationship between Cu and e

The selected retaining structures types

Basing on

- Geological and hydrogeological investigations.
- The reseaerches geotechnical.
- The deep of existing buildings foundation (H=3.5m).
- The pressure in foundations of existing buildings (p=150kPa).
- The deep of new buildings foundation (H=6.0m).

We selected from many retaining structures types only two of them.
Retaining structures with piles walls, where the short piles are lateral loaded by horizontal earth pressure.
Concrete sheet piles walls or metallic sheet piles walls. In the retaining structures of first type is operating an earth pressure "E = 200 KN/m", the diameter of pile is d = 0.7m, the ratio e/d ≈ 2 and L/d = 4.
The limit horizontal capacity of pile is 426 KN and the safety factor results to be FS = 3.0. The deep of piles results L = 2.8m and their length L' = 2.8 + 6.0 = 8.8m. In the retaining structures of second type from our calculations the deep of sheet piles walls results D + Z = 2.5m with FS = 1.4.

Disscussion

The variant pile-wall has the piles length = 8.8m, provides goods stability retaining structures (FS = 3) and secures the normal working conditions during the process of excavations, construction and exploitation of buildings. This variant is very economically because it is constructed it is so rapidly. One pile can be realized in very short time. The variant sheet-pile wall has length 6.5m, provides a safety factor FS = 1.4, provides a normal conditions in the pit (after excavation), but require a very strict juncture of the piles with cement mortar. For this variant need longest time to realized.

Conclussion

The design of such retaining structures is important and very delicate question. For realization of them we need a detailed lithological, hydrogeological and geotechnical investigations. The mean geotechnical parameters for calculations of retaining structures are φ, C, Cu. For the cohesive soils exist the good relationships between:

"Cu" and "φ, C"
"Cu" and "W"
"Cu" and "e"
"Cu" and "LL"
One part of these relationships is in linear (proportional) low. It gives a possibility for the designer engineers to avoid the mistakes during their works and use for similar geological formations the above mentioned relationships.

REFERENCES

Allkja, S., Ahmetaj, L., 2005.
Geological investigations of constructed site of a building with 4-25 stores and 2 stores underground in "Taulantia square ", Durresi town.

Bozo, L. 2000.
Rock mechanics, Polytechnics University of Tirana, p. 292.

Bozo, L. 2002.
Research of church's foundations and their basements in Albania, p. 270. Publications center of Orthodox Church of Albania

Bozo, L. 2007.
Geotechnics and soils mechanics,. Polytechnics University of Tirana, p. 457.

XIV European Conference on Soil Mechanics and Geotechnical Engineeribg
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Influence of deep excavation on nearby buildings
L'influence de l'excavation profonde sur les bâtiments avoisinants.

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Abstract

Recently the urban area has been used very extensively in Tirana Albania. Due to lack of space the new buildings are built near the old ones. This has caused the need of using special techniques for controlling the movements of soil and nearby foundations, which in many cases are not properly designed. These movements together with the structural design and execution errors of existing buildings had brought many problems to the habitants of the buildings. The article will give theoretical aspects of the method we have used in design in same excavations and the problems that have been met during execution. The ground conditions in Tirana are in many cases very complicated with a high water table level and despite the detailed design used, the negligence of the enterprise had led to accidents. Some of them will be presented in this article.

Résumé

Récemment l'espace urbain de Tirana (Albanie) a été considérablement utilisé. À cause du manque d'espace, les nouvelles constructions sont situées près des anciennes. Par conséquent, l'utilisation de techniques spéciales s'est révélée nécessaire afin de contrôler les mouvements du sol et des fondations à proximité, qui dans de nombreux cas ne sont pas correctement conçues. Ces mouvements, par-dessus les erreurs de conception structurelle et d'exécution des travaux, ont causé beaucoup de problèmes aux résidents. L'article présente les aspects théoriques de la méthode telle que nous l'avons utilisée en calcul dans des excavations et les problèmes rencontrés pendant l'exécution. Fréquemment, la géologie (structure, conditions) du sol à Tirana est très complexe et la nappe phréatique est proche de la surface (avec un niveau élevé de table de l'eau). Malgré la conception détaillée (plan détaillé) fournie, les négligences de l'entreprise en charge des travaux ont causées des accidents. Certains d'entre eux seront présentés dans cet article.

Keywords: nearby buildings, pile wall, different settlements, accidents

Introduction

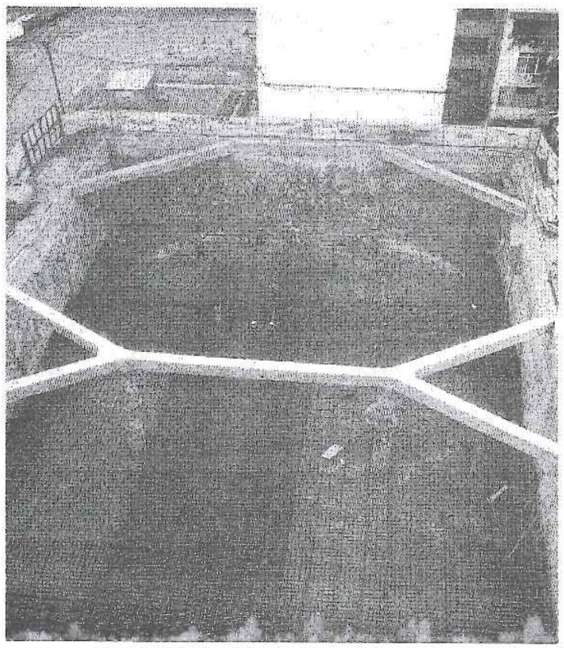
Until now the common idea among the related persons in the field (both professionals and investors) is that the design process of retaining structures near old one is quite easy. Commonly the designers use analytical simple methods for calculation of pile walls or retaining walls. The severe building conditions and the different accidents that have taken place recently have obliged the interested persons and also the government monitoring office for construction to be more careful and to pay much more attention. So now, the designer has to adapt more detailed and sophisticated techniques of calculation.

Many of the near buildings in the site are of shallow foundations (1-2m embedding) so almost every time we must do retaining walls.

Most of the damages to nearby structures were of no great influence in the overall stability of them, but due to lack of careful observation and analysis no one can say undoubtedly if they are secure or not at extreme load conditions.The general intention of the majority of the construction enterprises are to construct a light flexible retaining wall (as consequence a cheaper one) and to dig fast as to assure a short term behavior of soil-wall system

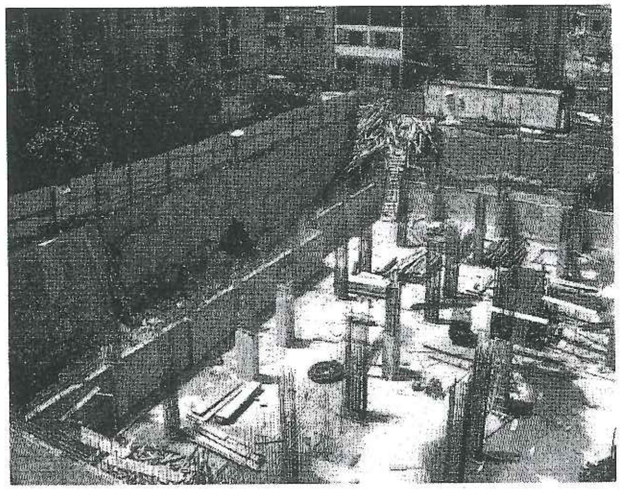
Types of used retaining walls

Due to market price, most of the construction enterprises use pile walls, but other types are used as well.



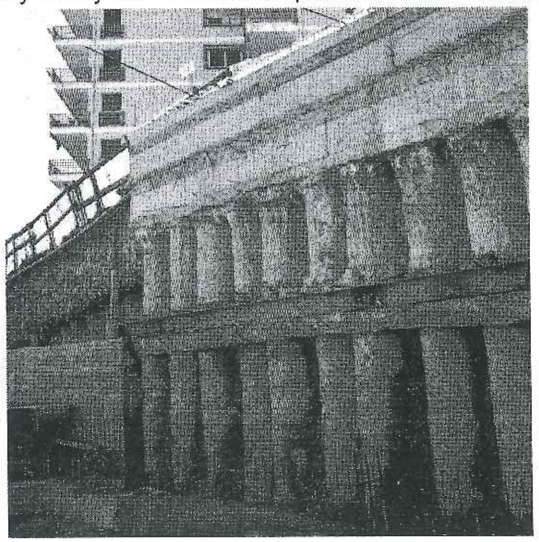
Diaphragm wall with proper capture

Here is a diaphragm retaining wall between two 10 stories buildings. Diaphragm wall has submitted small deformation and nearby buildings have submitted additional settlements of 1-1.5cm. These settlements are within allowable limits for raft foundations. Also, diaphragm wall controls very well the underground water.



Gravity wall (after same accidents his width is quite impressive)

Here is a gravity retaining wall with variable thickness. These types of retaining structures are used very rarely due to lack of space and their construction technology

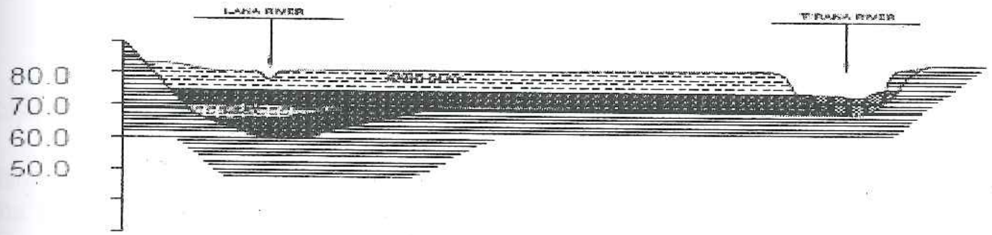


Pile wall with beam for metallic proper capture

Pile wall may be very dangerous in site with high water level. Gaps between piles allow water passing and makes its level lower. This influences a lot the nearby buildings even when pile wall displacements are small.
Other techniques such as building propping, underpinning, compacted grouting, cut-off-piling are not used.

Geology and geotechnic aspect of sites in tirana city.geotechnic aspects are very complicated with abrupt changes and very soft soils.

Deposits of Lana River and Tirana River with thickness from 10 to 25-30m are not consolidated and with organic soils.
Diluvial deposits consist of clays, sands, clay-sand mixtures and red colored strong clays. (Quaternary Q4)
Alluvial deposits consist of gravels of Tirana River. Underneath these deposits, there are clays and sand weathered stones. (Neogene N3).
Here after a schematic geologic section of Tirana is given.



Schematic geologic section of Tirana

Accidents

PARK RINIA

Pile wall failure

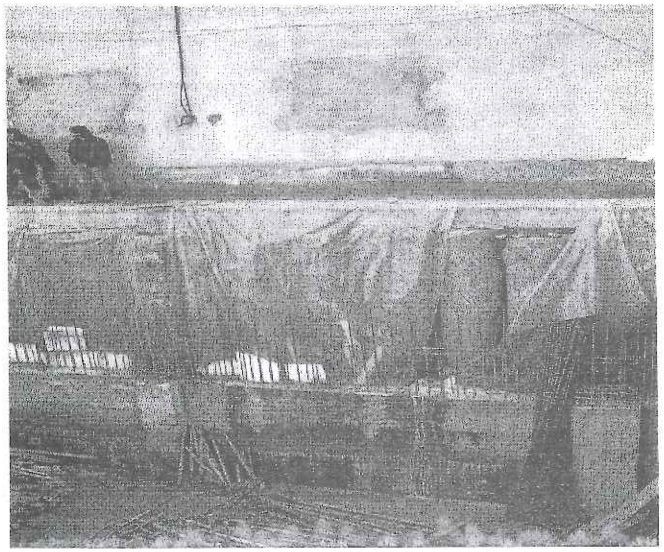
Due to improper realization of piles as presented in the calculation drawings (smaller depth, greater gaps between piles, bad reinforcement of tie-beam etc.) all the south side collapsed

SELITE

Pile wall failure (metallic proper failure)

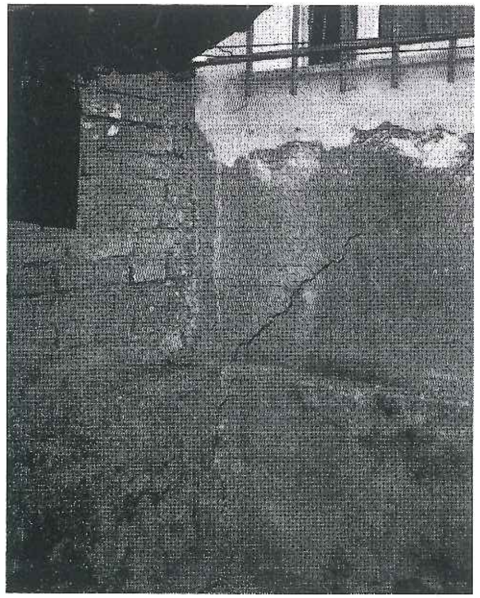
Metallic proper capture collapsed due to bad dimensioning and large deformations of pile wall.

PASHKO



Pile wall (after the raft and perimeter wall are made)

There are excessive settlements of nearby buildings. This accident will be analyzed more carefully.



Cracks in the masonry wall of nearby building

Case study

General

The site is in the east part, about 1km from the center of the city. It has a quadratic shape with dimensions of 22x20m. The building will have 3 stories above ground, 2 stories underground plus thickness of the raft foundation so the excavation is 7.5m deep. Nearby buildings are 3 stories masonry buildings with strip continuous foundations without anti- seismic columns and bad connections of masonry walls.

Piles are placed in three sides. They are 12m long with 70cm diameter without gaps between them. The used concrete is of class C20/25. Piles are connected in the head by a 70x70 reinforced concrete beam.

It is thought to use a concrete pile wall due to lack of space that doesn't allow the enterprise to use any kind of proper capture and consequently slow the speed of excavation. Based to previous studies (Moormann & Moormann) the retained wall and ground movements are independent of the retaining system stiffness. So, once the sufficient stiffness is chosen movements are determined by other relevant factors.

Monitoring was realized with simple methods measuring the fixed-point repers distances during different phases of work.

Soil

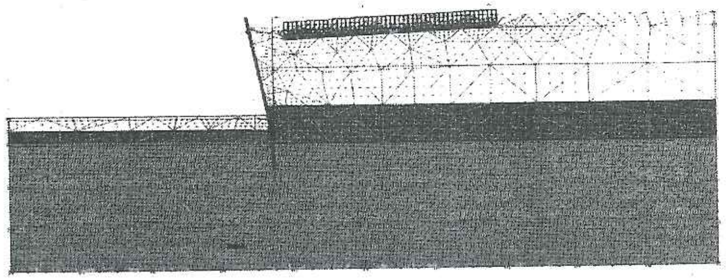
Soils of the site are alluvions, delluvions, weathered clay stones with weathered cracks in a depth of 7m and clay stones considered as bedrocks in a depth of 11m.

Here after some geotechnic characteristics of the soils are presented. (general parameters)

Soil	g	c	f	y	E
Fill	18.2	1	24	0	40000
Clay-sand	18.8	35	17	0	90000
Sand	19	1	28	0	90000
Upper Clay-stone	22	70	28	0	200000

Calculation model

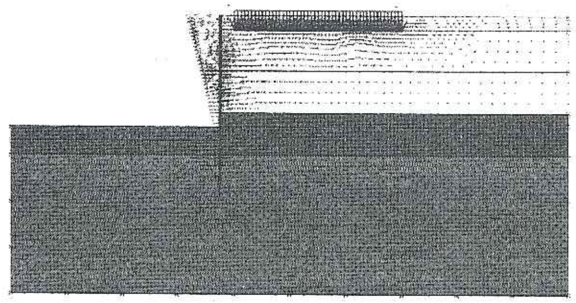
Calculations are made by specialized geotechnic software Plaxis. First a Mohr-Coulomb elasto-plastic law is used. Later, having more detailed laboratory data Hardening soil model is used for the soil and linear elastic beam elements to model the raft foundation of the new building, the piles and the strip foundation of old building.Due to characteristics of the rock where the foundation raft is embedded no other detailed analyses that might give more accurate results were performed.



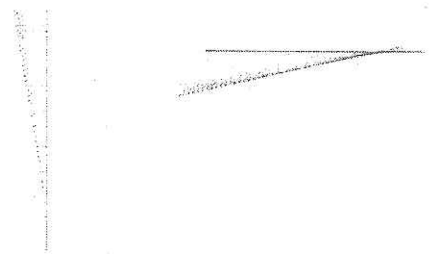
Deformed mesh

Results from the first model show that the displacement in the top of the wall and deformation behind the pile wall are acceptable and do not introduce much disturbance in nearby building. For the settlements of old strip foundations, a simple calculation for different settlements is made in the nearby buildings.

Here are represented the vectors of total deformation and displacements of the pile wall.



Vectors of soil deformation



Displacement of pile wall and additional displacements of strip foundation

The upheaval predicted by the program (only in MC model) is much more smaller in site and doesn't present any problem.

Accident

Due to a buried concrete pipe of old drainage system of the city with diameter 1.5m, after a rainy storm the excavation was filled with water up to the level of the pipe.

The water pump was very small and the excavation remained under water for almost 1 week. After that, same cuttings were shown in the pavement of nearby building the next day.

Second model

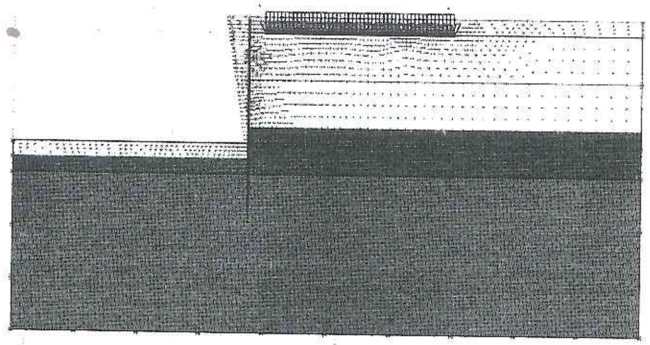
A new calculation was made for the new conditions due to changes of geotechnic parameters of soils. The new parameters of soil were chosen from soil samples (taken with hand drilling equipments) of layers influenced by saturation. Hereafter, the geotechnic parameters of soil used in the second model are again presented.

Soil	g	c	f	y	E
Fill	18.2	1	24	0	40000
Clay-sand	18.8	35	17	0	90000
Sand	19	1	20	0	90000
Modified Clay stone	20.8	50	19	0	100000
Clay-rock	23	110	30	0	200000

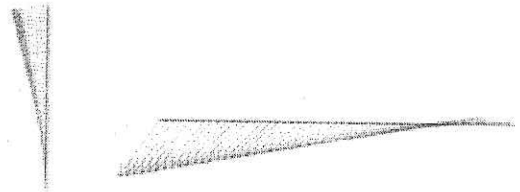
The same calculation model was used for the second case (hardening soil model). After the calculation, it was seen that the deformation of the soil and displacements of pile wall for the second model were increased significantly. The settlements of nearby buildings were at the allowable limits of additional settlements for such type of buildings. The stability of the overall retaining system was again assured because the upper layer of weathered clay stone has not allowed deeper infiltrations of water.

Comparison between calculated displacements and measured ones show differences but these aren't object of this article.

Hereafter, the deformations of the soil and the displacement of the pile wall for the second model are presented.



Vectors of soil deformation

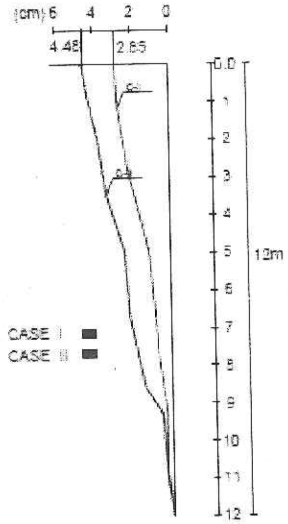


Displacement of pile wall and additional displacements of strip foundation

Comparison of the models

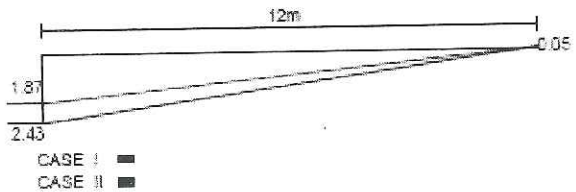
The models of behavior of the soil are the same for both cases and this article is focused only on the influence that may have even the smallest negligence on the nearby buildings in deep excavations. Pile embedments in bedrock divide the influence zones of low displacement and helps in long-term behavior of structures.

Here are the displacements of the two models. As presented, the horizontal displacements of wall in case 2 are increased with almost 2cm.



Wall horizontal displacement for two cases

The vertical displacement of the old foundation are increased with 1.7cm



Old foundation vertical settlements for two cases

The depth of retained wall is almost the same for the same safety factor.

Conclusions

The building response of settlements induced by nearby excavation depends on ground movements as well as the type, regularity, and stiffness of the structure.

In the first case additional settlements are less then allowable limits for such type of structures. Ds/L=0.001-0.002

In the second case the additional Ds/L =0.002, the measured deformation of ground tiles and appearance of evident cracks in nearby building show more significant deformations. So a correction factor is needed to take into consideration the irregularity in plan of masonry existing buildings.

REFERENCES

A.B. Fadeev et al. Deformation of structures during their erection in the dense urban setting of Saint-Petersburg. Soil Mechanics on foundation engineering. Vol 43, No. 1. 2006

Ch. Moormann, H.R. Moormann. A study of wall and ground movements due to deep excavation in soft soil based on worldwide experiences. Geotechnical aspects of underground construction in soft ground. 3rd International Sympposium

Th. Benz, R. Schwab, P. Vermeer. On the practical use of advanced constitutive laws in FE foundation analysis

European conference of Young Geotechnical Engineers.

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Landslide stabilization by means of deep drainage.

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Abstract

Generally, the most important and often determinant causal factor of destabilization of natural slopes is the high level of underground water table. It is significant the fact, that many slopes have failed during the winter,after heavy rainfall.
Drainage is often a crucial remedial measure due to the important role played by pore-water pressure in reducing shear strength. Because of its high stabilization efficiency in relation to cost, drainage of surface water by
superficial ditches, and underground water by networks of trench drains, is one of the most widely used, and generally the most successful stabilization method.
The following study is related to a problematic sliding zone in south of Albania. The landslide in question obtains a specific importance because of its location close to the national roadway Fier - Tepelene that connects the south of Albania with the capital, Tirana. Several times the traffic of the roadway is interrupted due to the periodic activity of the landslide. In the report below is described the optimization of a drainage system, based on the *Hutchinson's Method*, to achieve the stabilization of the zone.

Introduction

Landslides are characteristic phenomena in hilly and mountainous regions. In Albania, due to the dominance of hilly and mountainous relief, this geological and geotechnical phenomenon has a considerable extension. An important landslide can be found near Tepelena city in the south of Albania. This study is extensively focused on the ways for the stabilization of this zone.

Geological and geotechnical study of the sliding zone

This landslide has been active for years. For its stabilization, several times, are carried out engineering proceedures but without any positive evident impact. The chronological development of the sliding is as following: During 2002: we have the presence of a creeping movement of soil masses with a 2m/year velocity.

After an intervention, aiming the reduction of the weight of the moving body, the zone gained a temporary stability.

During 2004: the creeping movement of the sliding body reappears with a 0.5m/year velocity. Again there were applied some measures (as the construction of two retaining walls, and the embankment of the road) but without any final satisfying result. From 2004 and constantly we have some centimeter movement of the sliding body with a velocity 8- 12cm/year.

For understanding the geological conditions of this zone are drilled four boreholes at 15-20 m depth, is performed a geological survey, is installed a plastic pipe system in order to estimate the movement dynamics of soil masses, and to monitor the underground water level.

Disturbed and undisturbed samples are taken for the determination of physical and mechanical properties of the layers encountered in this zone. The geological map and the longitudinal geological profile of the sliding zone are compiled.

a)Site Location, Geomorphologic and Geological Structure.

The sliding zone is located at the km 28+380 –29+000, at the second part of the project Fier – Tepelena Road. It is situated on the left side of the Vjosa River flow. The inclination of the valley has different slopes in different levels. These slopes are conditioned by the geological formation. The parts composed of Sandstone are steeper than the parts composed of Mudstone. However, there are some parts almost horizontal, which are created by Vjosa River. These represent the old terraces of this river.

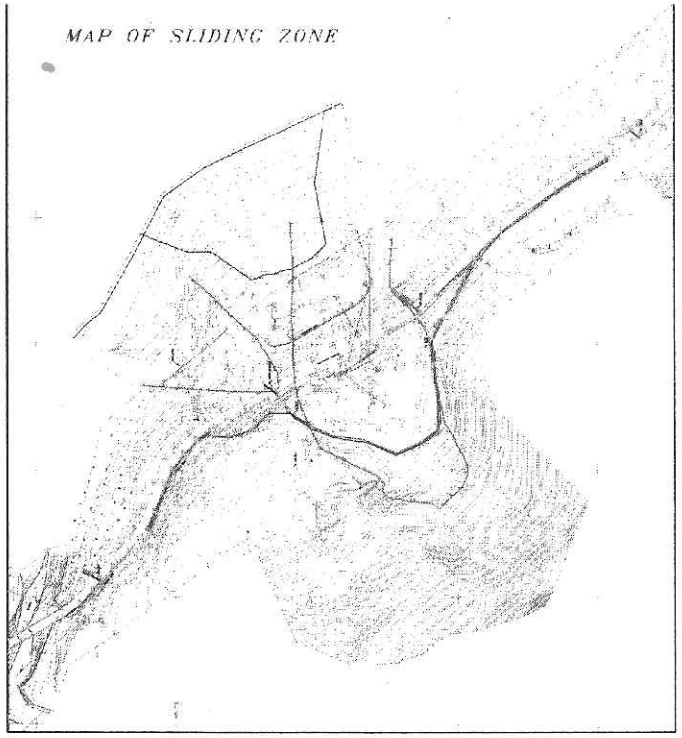
The incline is composed of Mudstone and Sandstone, which are intensively weathered in surface. These deposits are covered by 5-10 m colluvium deposits. The weathered zone depends on the composition of the rocks. Mudstone beds get weathered more easily while Sandstone ones are more resistible. The weathered zone reaches 4-5.00 m below the colluviums deposits. From the ground surface towards the deepness the quality of the rock gets better.

These rocks are interrupted by two discontinuity systems.

One is in the direction of the stratification and the other one perpendicular to the stratification. These discontinuities enable the ground surface water to penetrate deeply and the weathered zone advances deeper. Vjosa River Retaining Walls Existing Perimeter.

b) The Causal Factors Leading to the Loss of Slope Equilibrium.

From the study performed in this sliding zone and in some other slight ones encountered in the zone it comes out that they occurred as a result of the following reasons:



i) The deposits faced in the incline are rocks of low resistance against *weathering* phenomena.

ii) The present colluvium deposits, which are composed of silty clay with gravels, when *fully saturated with water*, lose their mechanical properties and start to move taking with them the most weathered part of the beds of Mudstone and Sandstone.

iii) The phenomenon of *erosion by the surface water* is present. During massive rainfalls water streams are created at the inclined surface, which erode parts of colluviums deposits and the most weathered parts of the rocks.

iv) The equilibrium of the slope is affected also from the *erosion that Vjosa River* creates at the toe of the slope. This is one of the main factors that has induced the sliding.

v) *The presence of the underground water during the most part of the year.* As described above there are beds of sandstone and mudstone in the geological composition of this zone. The beds of sandstone have high water permeability. The water enters at the outcropping parts at the highest points of the relief and it comes out as springs at the lowest altitudes. Beds of mudstones are practically impermeable.

That's why the water springs come out at the contact between mudstone and sandstone. These waters saturate the masses of colluvium deposits and the most weathered part of the rock formations. As a result the weight of the soil masses is on one hand increased and on the other hand the mechanical properties of soils are decreased. Also the shear strength of the soils is considerably reduced by the pore-water pressure. The fact that the sliding zone is more active during the winter signifies that the stability of the slope is strongly influenced by the underground water table level.

From the 'in situ' measurements derives that the fluctuation of the water table level is small. In spring the underground water table level is 1.5m deep, while in September (after the driest period of the year) is 2.5m under the ground level.

c) Geological, Engineering and Geotechnical Conditions of the Sliding Zone.

In this study are determined the active sliding planes. Also are determined the physical and mechanical properties of all the geological layers, the residual parameters in the sliding planes for all types of soils and rocks encountered in this zone. Based on these data, will be carried out the calculation of the stability of the sliding zone.

The sliding plans, identified by the boreholes are marked in the geological profile. The movement of the layer no.2, no.3 and no.4 is more evident and they move with respect to each other; the movement is visible at the ground surface. The movement of layer no.6 and no.7 is slower and it seems as it has reached equilibrium.

Table 1. Physical and Mechanical Properties of the Layers

layer	Visual description of Layers	Clay Fraction	Silt fraction	Sand fraction	Grain fraction	W_L %	W_P %	W %	γ_0 KN/m^3	γ KN/m^3	Void ration ε	φ	C_r KPa
1	Medium dense beige to grey GRAVEL with sand; the gravel is rounded	-	-	-	-	--	--	-	-	-	-	-	-
2	Firm brown to beige silty CLAY with gravel; the gravel is angular	29.70	42.50	16 90	10 90	41.50	23.20	24.60	2.7	1 9	0.78	15	15
3	Weak grey to beige MUDSTONE with fissures. very weathered	31 60	40.80	14.70	12.90	42.70	23.50	23.50	2.72	1.98	0.76	15	15
4	Firm to stiff brown to beige silty CLAY with sand containing a little gravel, the gravel is angular.	24.70	38 50	25.90	10 90	39.80	23 40	24.80	2.70	1.90	0 78	16	15
5	Firm to stiff brown to beige silty CLAY with sand containing a little gravel. the gravel is angular	21 70	41 70	18.50	18.10	38.80	22 30	24.70	2.71	1 92	0 79	17	25
6	Weak beige to grey SANDSTONE with weak matrix Very weathered rock	9.40	32.90	47.90	8.80	28.60	23.40	22.40	2.7	2.18	0.7	23	5
7	Weak beige to grey MUDSTONE and SANDSTONE	35.20	42.70	22.10	–	43.60	22.10	14.60	2 73	2 34	0 48	26	165

d) Mechanism of rupture.

Apparently, according to the geological and geotechnical study we are dealing with a translational slide which is characterized by some well determined failure surfaces in the interior of the slide's body, with elasto-palstic behavior. This is an active consequent (follows the bedding planes) landslide. Different layers move downwards and regarding each other. For analyzing the stability of the slope, the residual parameters of shear strength of the soils, will be used.

e) The measures that must be taken for the stabilization of the sliding zone.

Based on the reasons that we analyzed above, the physical and mechanical properties of the layers that are identified in the landslide and on the mechanism of rupture are recommended the following measures
for stabilization of the zone:

- Lowering the underground water level by means of drainage through sub horizontal ditches, which will reach to the most possible depth.
- Removing the surface water from the sliding body by means of a perimeter concrete ditch. Such a ditch already exists, but it is fissured and instead of removing it gathers the atmospheric water and put it in the sliding body in concentrated way.
- Building some river barrages in order to protect the slope from the erosion of Vjosa River.
- Adjusting the soils created from different excavations at the road, which in some cases have enabled the reactivation of some parts of this sliding.
- It is recommended to reconsider the existing engineering measures at both retaining walls, which have not been constructed according to technical conditions.

Hutchinson's method for the deep seated instability improvement

This method gives very good solution in case of limited inclination slope. The method consists in the design of longitudinal deep trench drains, which will be installed in the incline. By means of these drains is enabled the underground water table lowering, and consequently the reduction of pore water pressure. Longitudinal trench drains are executed by means of special machine. Generally the width of the trenches is 0.5-1m and the depth varies 7-8m or deeper if it is possible and feasible. Ideally, the drain should penetrate the shear surface (such drains are referred to as "counterfort" drains) and in addition to the improvement in stability as a result of reduced pore water

pressure on the slip surface, some additional restraint is achieved by the replacement of the weak slipped material by the stronger material in the drain. The trench is back-filled with suitable freedraining material and in its lower part is placed a perforated plastic pipe, which accelerates the process of evacuation of water. Measures to prevent clogging must be incorporated in the design.

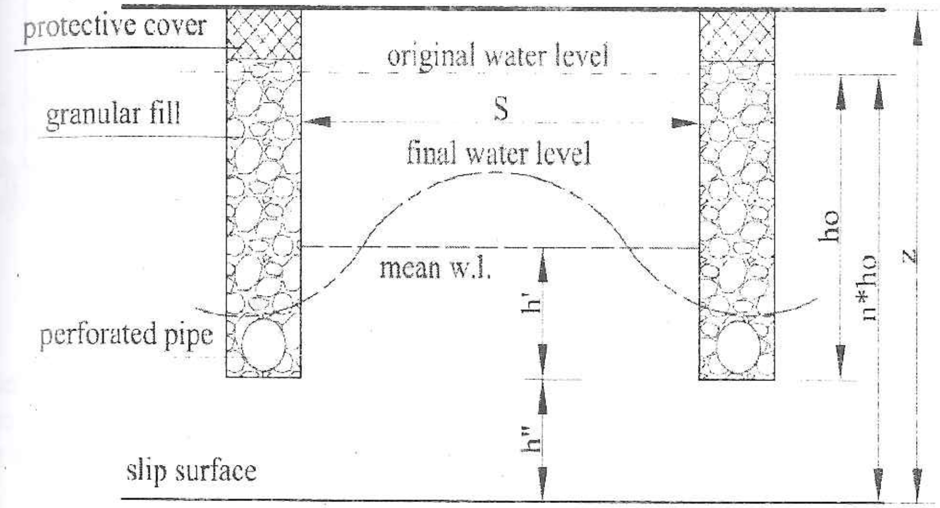


Fig.1 Efect of trenches in the underground water table.

An approximate method for designing trench and counterfort drains has been developed by Hutchinson using finite element analyses and assuming twodimensional steady-state flow. Hutchinson used the results of the analysis to define the average efficiency of the drains (η) and to relate the efficiency to the ratio s/ho where 's' is the spacing of the drains and 'ho' is the depth of the drains beneath the groundwater level (see Fig.2). η is given by:

$$\eta = \frac{h_0 - h'}{h_0} \qquad (3.1)$$

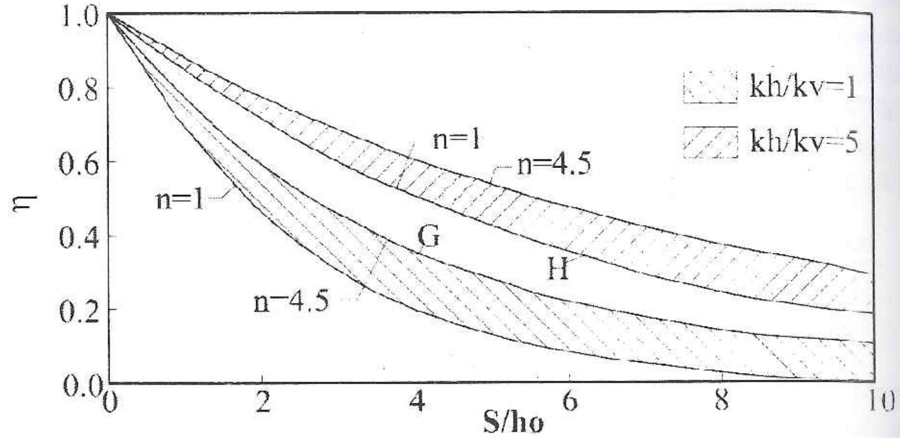


Fig.2 Efficiency of drains.

Is suggested that lines G and H can be taken as reasonable lower and upper bounds for the design of drains. For the purposes of preliminary design, curve G can be regarded as a conservative lower bound. The factor of safety can be assessed by the following formula:

$$F_s = \frac{C' + (\gamma z - \gamma_w h_u)\cos^2 \beta tg\alpha}{\gamma z \sin\beta \cos\alpha} \qquad (3.2)$$

If C'=0, then

$$F_s = \left(\frac{\gamma z - \gamma_w h_u}{\gamma z}\right)\frac{tg\varphi'}{tg\beta} \qquad (3.3)$$

Where:
C' and ϕ' - are shear strength parameters on the shear surface.
γ - is the bulk density of soil.
γ_w- is the specific weight of water.
β - is the inclination of the failure surface.
h_u - is the height of the water column above the failuresurface.
z - is the depth of the failure surface.

These two formulas can be applied for two situations
1.with the original water level $h_u^h = n.h_0$ 2.with the reduced water level (mean w.l.) h_r^u
So we can evaluate the factor of safety before and after the installation of the trench

drains in the slope. To determine the reduced water table level we have to define the efficiency of the drains η . This can be done in relation of the ratio "S/h$_o$" and "$n = \frac{h_u^h}{h_0}$" in the graph at (Fig.2).

$$S_0.\eta = f\left(\frac{S}{h_0}, n = \frac{h_u^h}{h_0} = 1\right)$$

So we will use the line H (n=1) for defining the values of efficiency (η), assuming that the slip surface is located just in the base of the trench drain.
With the defined value of the efficiency η we are able to determine the reduced water level above the failure surface:

$$\begin{aligned} &h_u^g\ h' + h'' \\ &h' = h_0\ (1.\ \eta) \qquad\qquad (3.4) \\ &h'' = nh_0 = h_0\ \ (n\text{-}\ 1) \end{aligned}$$

And instead of h we put h_u^r in the formula (3.1 and/or 3.2) to obtain the factor of safety after the installation of the trench drains in the slope.

The optimization of the deep trench drains system

In the longitudinal geological profile are considered some "columns" (C-1, C-2, C-3, C-4, C-5) in which will be checked the stresses at the sliding planes that the "column" intersects and the factor of safety will be also calculated. In this way in each "column" will be determined the weakest point (the point with the minimum of factor of safety in high water table level conditions).

Based on the weakest point of each "column", will be assessed the needed depth and distance between the trench drains and also the respective underground water table level reduction, to achieve the required factor of safety. According to the "in situ" monitoring, the underground water level, is 1.5m beneath the ground level.
In each "column" are determined the heights of different geological strata (h_i) and the inclinations (β_i) of the sliding plans intersected by the "column".

COLUMN 1 : h_1=2,17m β_1=23°
h_2=3.48m β_2=15°

h_3=3,64m β_3=15°

COLUMN 2 : h_1=5,56m β_1=15°
h_2=2,83m β_2=15°
h_3=2,43m β_3=14°

COLUMN 3 : h_1=7,32m β_1=12°
h_2=3,73m β_2=10°
h_3=3,01m β_3=13°

COLUMN 4 : h_1=4,40m β_1=2°
h_2=4,10m β_2=10°
h_3=0,00m β_3=15°
h_4=0,00m β_4=21°
h_5=4,40m β_5=14°

COLUMN 5 : h_1=9,90m β_1=14°
h_2=3,44m β_2=10°
h_3=1,98m β_3=15°
h_4=1,92m β_4=16°
h_5=2,39m β_5=14°

For calculating the factor of safety for the point „A" will be applied the formula:

$$F_s = \frac{C'(\gamma_{ave} z - \gamma_w h_u)\cos^2 \beta tg\varphi}{\gamma z \sin\beta \cos\beta}$$

$z=h_1+h_2$ is the depth of point "A".
h_u=z-1.5m is the depth of point "A" beneath underground water table level.
$\beta = \beta_2$ is the angle of inclination of the second sliding plan where the point 'A' belongs to.

$$\gamma_{ave} = \frac{\gamma_1 h_1 - \gamma_2 h_2}{h_1 + h_2}$$ is the average unit weight of the soil above point 'A'.

All the calculations are presented in the Table 2. In this table are calculated the factors of safety in two cases; 1. in conditions of high underground water table level (1.5m beneath the ground level) 'F_{S1}' and 2. in conditions of absence of underground water in the slope 'F_{S2}'. (See Table.2) Comparing the two factors of safety comes to light the negative impact of the underground water in the slope. The weakest point of each column is marked with red colour. The trench drain system will be optimized based on these weakest points.

Table 2. Assessment of the weakest point.

Col	Strata	C_r	φ_r	γ	h_i	γ_{ave}	γ_w	h_w	z	β	F_{s1}	F_{s2}
1	2	15	15	19	2.17	19.00	9.81	0,67	2.17	23	1.54	1.64
	3	15	15	19.8	3,48	19.49	9.81	4.15	5.65	15	1,18	1.54
	6	5	23	21.8	2.64	20.40	9.81	7.79	9.29	15	1.05	1.69
Col	Strata	C_r	φ_r	γ	h_i	γ_{ave}	γ_w	h_w	z	β	F_{s1}	F_{s2}
2	2	15	15	19	5.56	19.0	9.81	4.06	5.56	15	1.19	1.57
	3	15	15	19,8	2,83	19.27	9.81	6.89	8.39	15	0.95	1.37
	6	5	23	21.8	2.43	19.84	9.81	9.32	10.82	14	1.08	1.80
Col	Strata	C_r	φ_r	γ	h_i	γ_{ave}	γ_w	h_w	z	β	F_{s1}	F_{s2}
3	2	15	15	19	7.32	19.0	9.81	5.82	7.32	12	1.27	1.79
	3	15	15	19,8	3,73	19.27	9.81	9.55	11.05	10	1.26	1.93
	6	5	23	21.8	3.01	19.81	9.81	12.56	14.06	13	1.11	1.92
Col	Strata	C_r	φ_r	γ	h_i	γ_{ave}	γ_w	h_w	z	β	F_{s1}	F_{s2}
4	2	15	15	19	4.40	19.00	9.81	2.95	4.40	2	10.13	12.76
	3	15	15	19,8	4.10	19.38	9.81	7.06	8.50	10	1.41	2.05
	4	15	16	19	0	19.38	9.81	7.06	8.50	15	0.99	1.43
	5	25	17	19.2	0	19.38	9.81	7.06	8.50	21	0.91	1.25
	6	5	23	21.8	4.4	20.20	9.81	11.46	12.96	14	1.05	1.78
Col	Strata	C_r	φ_r	γ	h_i	γ_{ave}	γ_w	h_w	z	β	F_{s1}	F_{s2}
5	1	0	30	21	9.9	21.00	9.81	0	9.9	14	2.32	2.32
	3	15	15	19.8	3.44	20.69	9.81	3.44	13.34	10	1.65	1.84
	4	15	16	19	1,98	20.47	9.81	5.42	15.32	15	1.08	1.26
	5	25	17	19.2	1.92	20.33	9.81	7.34	17.24	16	1.12	1.34
	6	5	23	21.8	2.39	20.51	9.81	9.73	19.63	14	1.35	1.76

The procedure of the optimization is presented in the table nr.3 and the selected alternatives for the deep trench drain system design, are in coloured background.

Conclusions and recommandations.

In the table nr.2 (of the weakest point assessment) we can see that in the upper part of the incline ("column 2") we have sliding activity between the strata nr.3 and nr.6. In the middle part of the slope ("column" 3), the situation is presented better. Here more active is the plan of contact of layers nr.6 and nr.7. In the "column" 4 we have the most critical point of the entire landslide. Here the sliding activity is evident between layers nr.4 and nr.5 , nr.5 and nr.6. In the table nr.3 (of the trench drain system optimization) can be seen that in the upper part ("columns" 1,2,3) of the landslide are required 8m deep trench drains every 15m, in the "column 4" are required 8.5m deep trench drains every 15m and under the roadway body are required 8m deep trench drains every 15m to achieve the stability of the sliding zone. In "column 5" is assumed that the water level is just in the contact between the road embankment and layer nr.3. At "column 4" we have the presence of the counterfort effect of the trench drain because the body of the drain reaches the sliding surface. The failure was clearly triggered by the high underground water table level. In these cases the dewatering of the ground is an inevitable remedial measure. The method used for improving the stability conditions of the sliding zone is in accordance with its mechanism of rupture. For preventing the clogging of the trench drains, a geocomposite liner will be applied at its slats. Also, for accelerating the process of evacuation of water, a perforated plastic pipe Φ300mm will be placed at the bottom of the trench drain. To gather the atmospheric surface water and to prevent its intrusion in the slide's body, will be constructed a perimeter reinforced concrete ditch. For further improvement of safety of the zone, an intervention at the existing retaining walls is foreseen.

REFERENCES

Atkinson, J. 1997. *Geotecnica, meccanica delle terre e fondazioni.*

Bozo, L. 2001. *Mirëmbajtja dhe rehabilitimi I rrugëve.*

Bozo, L. 2002. *Bazamentet e objekteve të kultit në Shqipëri.*

Bozo, L. 2004. *Dinamika e dherave (Pjesa II-të).*

Hutchinson, J.N. 1977. *Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movement. Bulletin of the Int Assoc. Eng. Geol., 16, 131-155.*

Konomi, N. 2001. *Gjeologjia inxhinerike (Gjeodinamika Inxhinerike).*

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Three dimensional Finite Element Analysis of lateraly loaded piles

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Abstract

Calculation of earth retention, for a three story underground excavation.
Excavation is 9.8 m, 2.8 from which will be done before drilling pile holes.
Soil below the dredge line, is modelled by mean of the modulus of subgrade reaction ks (Foundation Analysis and Design, Bowles 1996). Modulus ks is taken as constant.Finite Element Analysis is done by using ETABS.Some conclusions have been taken.

Introduction

Calculation is made for earth retention of a three story underground excavation, for a 13 story building. From natural soil level, will be done an excavation of 9.8 m.
Earth retention will be guaranteed by mean of drilled piles, with 10 cm space between them. Regarding to Geotechnical Study, underground water is at level - 12.00 m from natural soil level.
The plan of piles is given in fig. 1.
Before drilling pile holes, soil will be excavated till the depth of 2.8m from natural soil level. After that, pile holes will be drilled and piles will be placed as fig. 1.
At the upper level of the piles, will be constructed a large beam, which will create support for the piles. Pile elevation is given in fig. 2.
Besides the piles used for earth retention, will be placed two more piles in the middle, creating so the plane of the connections, like in fig. 3.
Calculation scheme is given in fig. 4 :

Modeling process

Actions on the piles are:

Earth active pressure, p_e. Uniform load on the ground, which could comes from one building near the piles, p_s. Under the dredge line, earth is modelled by means of springs, value of which should be taken from Geotechnical Report, taken from in situ experiments.
Approximately orientative values are given in the book "Foundation Analysis and Design" i Joseph E. Bowles.To control if the depth of the pile under the dredge line is sufficient, we do as below:

1- We look at the values of spring reactions. If this values are greater than the horizontal bearing capacity of the soil, then we have to increase the depth.
2- We look at the displacements of the lower extremity of the pile. This displacements have to be equal or lower than 2 mm (Bowles, Foundation, 1996).
At the top extremity of the piles, we construct the beam, which connects the piles and provides support at their top extremity.The 3D model is given in fig. 5:
From fig. 5, we see that the piles are modelled as wall. To do so the stiffenes of the wall has been modified and made zero in the horizontal direction, so the load is distributed in the vertical direction
The wall thickens is calculated to be equivalent in weight and in stiffenes to the pile.

The soil under the dredge line is modelled by the soil spring, which is given perpendicular to the "wall". In fig. 5 we see the diagonal braces in the top extremity of the piles.From the above model we can take the displacements and internal forces, but to have a more accurate calculation, the pile is modelled separately according to the scheme in fig. 3 and the beam in the top extremity of the pile is modelled as a spring.Modelling the pile under the dredge line, we subdivide the pile in parts of 50 cm high and in any node we put the value of the coefficient of subgrade reaction.

Form the calculation the maximum displacement in the top extremity of the pile is 1.8 cm.The above calculation is for the excavation phase.
After the excavation will start the construction of the building and when the building will reach the second floor from the mat foundation, the two piles in the middle will be destroyed and the diagonal braces will be cut.

So in this phase we have another calculation scheme for the piles.
This scheme is given in fig. 6. As we see now the piles are supported at the floor level.The calculation of this scheme shows that the resistance of the piles calculated in the first phase, is sufficient for this second phase to.

Conclusions
This method is fast and can be performed in large excavations. It takes into account the behaviour of the all underground earth retention structure working together and can be used for different work phases of the structure.

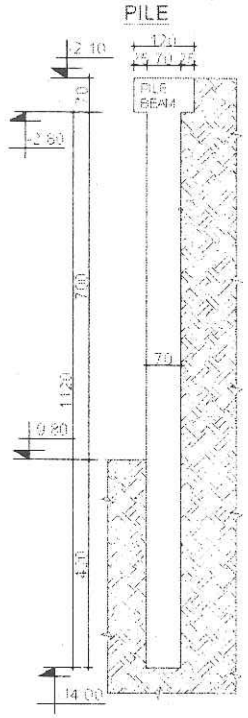

Fig.1. Piles plan

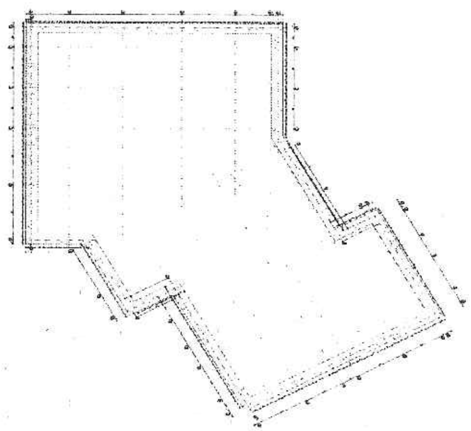

Fig. 2. Pile elevation.

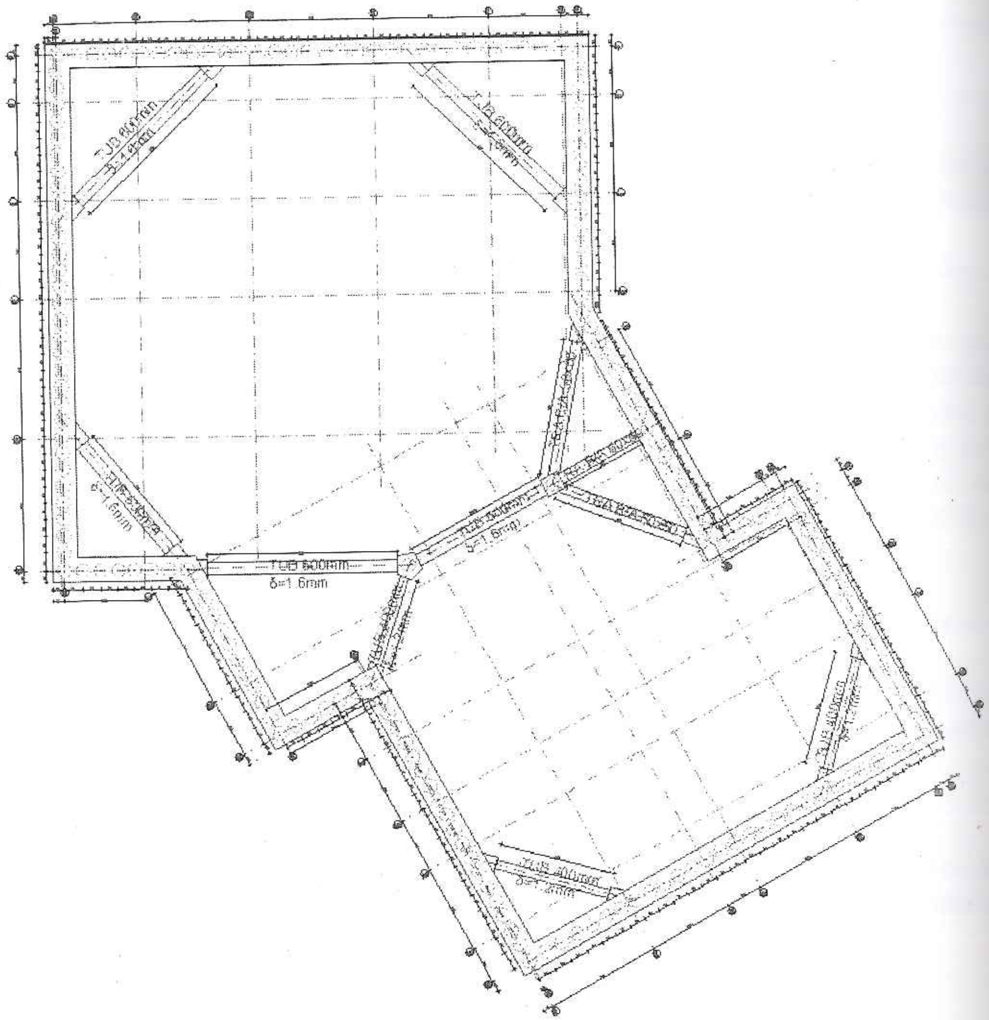

Fig.3 Connections plan

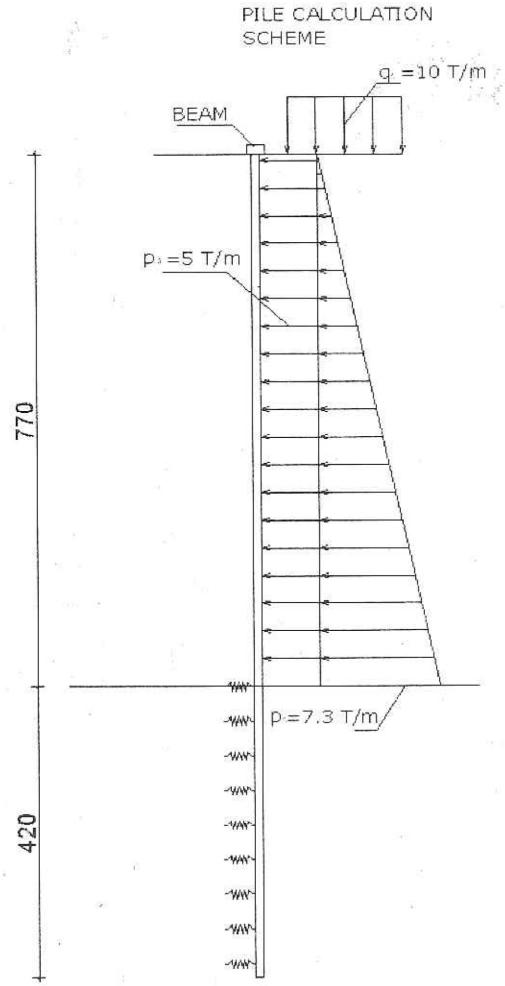

Fig.4 Piles calculation scheme

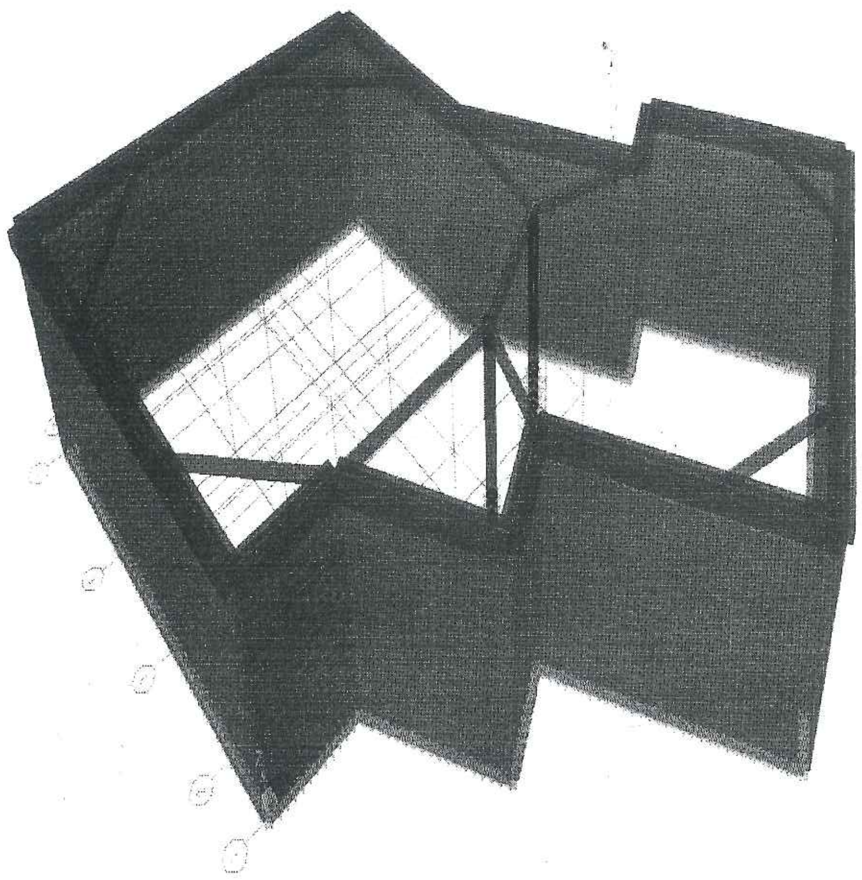

Fig. 5. 3D model

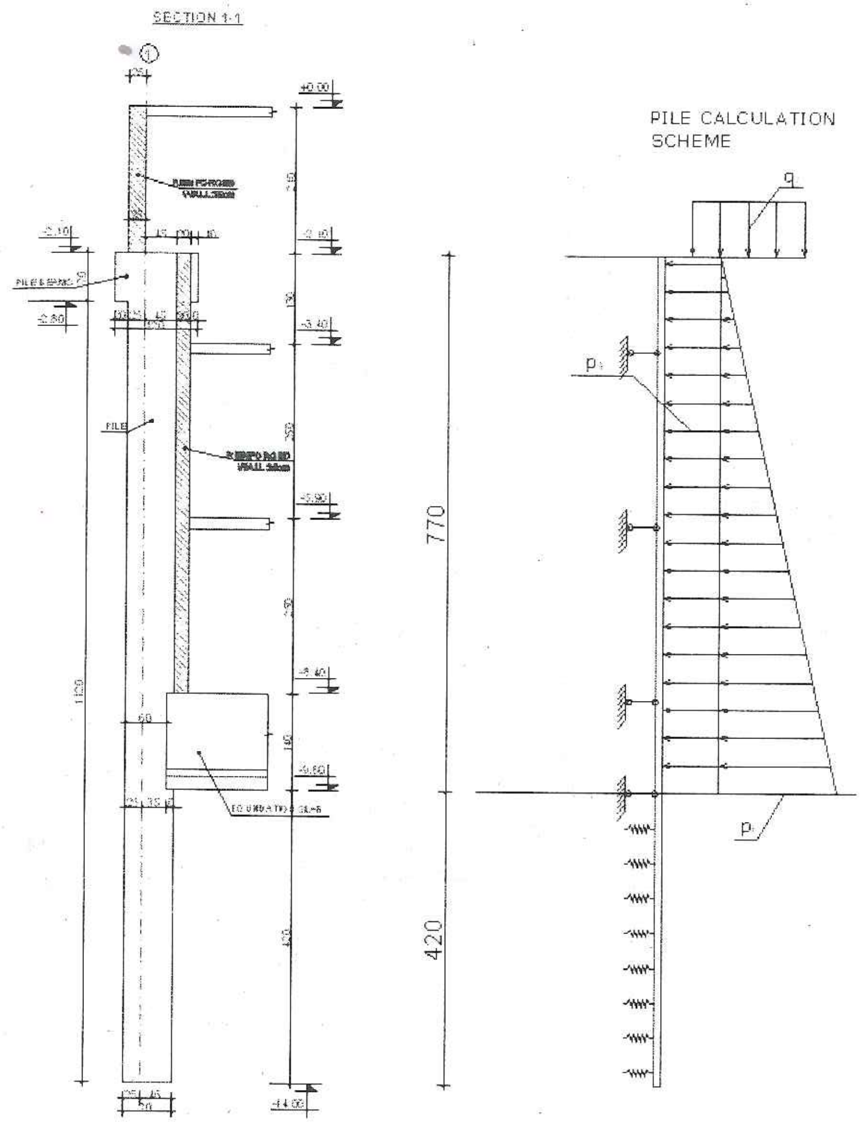

Fig. 6. Elevation after the construction of the three story underground, Calculation scheme in this phase.

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- First International Conference on Education and Training in Geo-Engineering Sciences.
  Costanza. Romania.June 2-4 2008

- International Geotechnical Conference "Development of Urban Areas and Geotechnical Engineering"
  St Petersburg. June 16-19 2008

- 6-th International Conference on Case Histories in Geotechnical Engineering.
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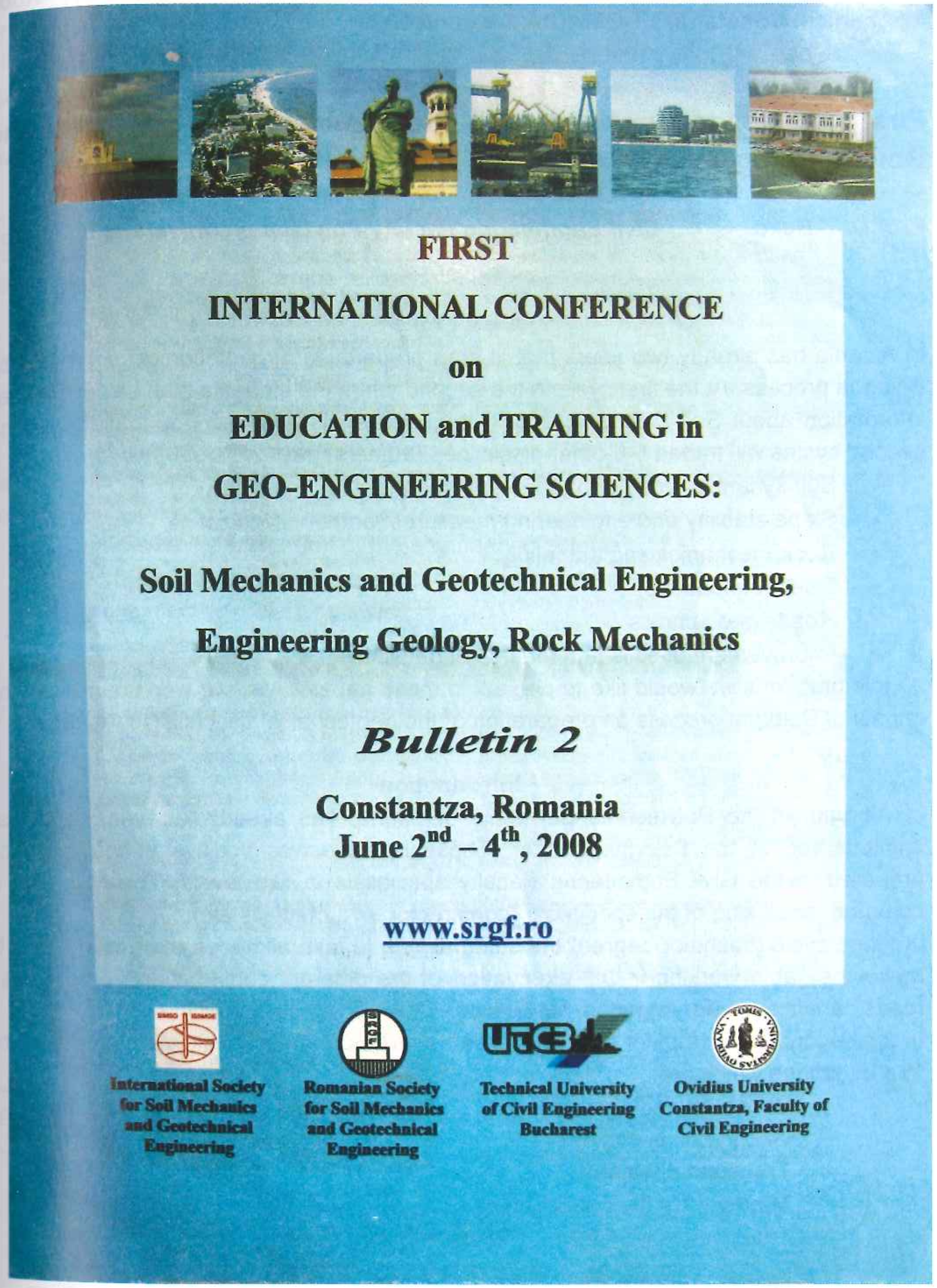

First International Conference on Education and Training in Geotechnical sciences: Soil Mechanics and Geotechnical Engineering Geology, Rock Mechanics, Constannza ROMANIA 2-4 June 2008

First and second cycle degree programmers and impact of the Bolonga process in Albania.

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Abstract

In Albania has already two years that is start preparation of geotechnical engineers by Bologna process. At the first cycle in the second years the students take all necessaries information about Soil Mechanic and in third years will treat the Foundations. At the second cycles will traded 5-6 other disciplines (matters) as:

- Soil dynamics and earthquake engineering
- Slope stability and engineering measures for their stabilization
- Rock mechanics and tunneling
- Deep foundations
- Roads geotechnics
- Improvement of soils and the use of geosinthetics.

All this problems we would like to present in these papers. Also we won't to present the impact of Bologna process on preparation of the geotechnical specialists in Albania.

Introduction

In Albania, in the Polytechnic University of Tirana, has already two years that was implemented of the Bologna charter. These initiative was positive because will be prepared in the Civil Engineering Faculty specialists in two levels. The first level is common for all kind of the specialists (constructor, hydrotecnics etc).

In these cycle (bachelor degree) the students can to take all necessary knowledge's to make they able working in the execuation of the different construction as buildings , roads canalization (for irrigation, sewer) etc.

In the second cycle (master degree) the students can made specialization on one of the four directions which are:

- Structural engineer
- Geotechnical engineer
- Transport engineer
- Manager for the construction.

After second cycle the students are graduate "Civil Engineer" . The first cycle termined with the former or terminal examination while the second cycle termined with diploma work.

In the following we wont to show how we are concepted forming of the geotechnical engineer in the framework of the Bologna process.

Curriculas of the first cycle (3 yerars) in the geotechnical field and geosciences.

During the first cycle the students takes common knowledges in the theoretical sciences and technical theoretical sciences . In the Civil Engineering Faculty in the first cycle are prepared specialist in the following fields :

- Construction 3 years with 180 ETC
- Hydrotechnics 3 years with 180 ETC
- Environment 3 years with 180 ETC
- Geodesy 3 years with 180 ETC
- Architects 4 years with 240 ETC

The students of the construction direction will take necessary knowledges in the geotechnical field for :

- The soils as the basement of the engineering objects
- The soils as the material to build engineering objects.
- The origin of the rocks and soils
- The kind of shallow and deep foundations.
- The kind of geotechnical retaining structures etc.

For these goal we following the schema (fig.1)

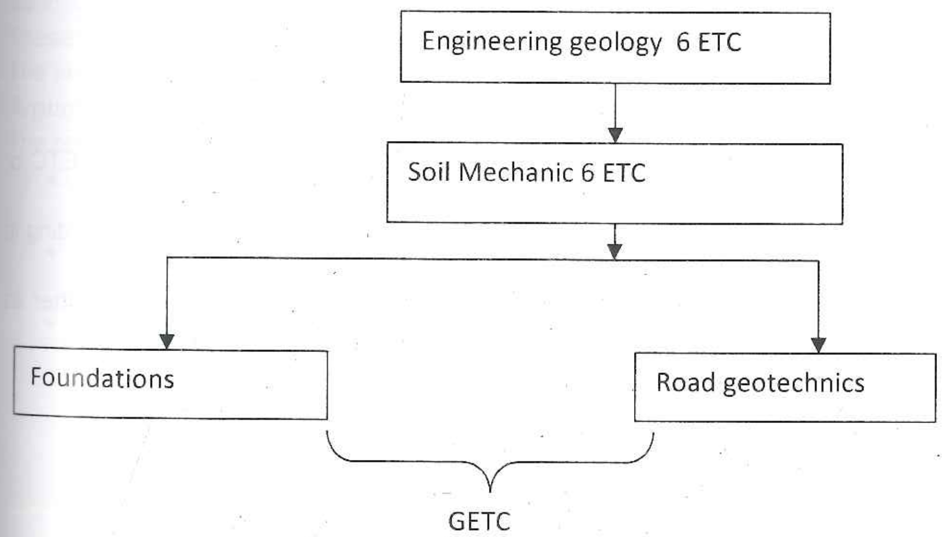


Fig.1 The block schema of the geotechnical education in the first cycle.

In the Engineering Geology treated a subject about:

- The formation manner of the rocks
- The more ordinary geological formations
- The geological phenomena's tied with all kind of the construction
- The geodynamics phenomena's etc.

In the Soil Mechanics treated a subject about :

- Physical nature of the soils and their classification for construction intentions.
- The behaviour of the soils under static loads.
- The effective stress in the soils (under all kind of the loads)
- The nature of the soils deformation (primary and secondary consolidation) the settlement's calculation and duration for the soils deformations.
- The limit state theory for the soils, the critical pression, the soils capacity, the slope stability, the high embankment and calculation of their basement by limit states.
- The theory of te earth pressures the different methods for earth pressure, calculation, the calculation of the retaining walls by limits states.

In the Foundation treated the problems of design of:

- The shallow foundations
- The beams and slabs in elastic basement
- The pile foundations
- The geotechnical retaining structures, as sheet piles, anchored or not the concrete walls in the soils etc.

In the Roads Geotechnics treated.

- The soils as the roads layers
- The dimensionement of the roads layers
- The quality control of the roads layers
- The laboratory and in situ tests
- The methods for the amelioration (improvement) of the soils.

In the total number of ETC in the first cycle the geotechnical maters have 18 ETC or 10% of the total ETC.

In the terminal examination the geotechnical matters have 15% of all ETC including in them.

In all other directions, except the geodesy and environmental, geotechnics matter as following

- Hydrotechnical direction 10 ETC in two semesters
- Architectonic direction 4 ETC in one semester

Curricula's of the second cycle (two years) in the geotechnical direction (master degree)

In the second cycle the geotechnics matters treaded in all Civil Engineering Faculty expect the geodesy direction . For the geotechnical direction we think to prepare a capable graduate engineers which will designed.

- All geotechnical structures under static and dynamic loads.
- In all kind of the soils, normal problematic soils etc.
- In conformity with Europeans rules *Euro codes)

Also geotechnical engineers will be able to executed in correct manner all geotechnical structures by EC-7.

So in the second cycle (which start at October 2008) we think to treated the following disciplines:

- Rock mechanics with 4 ETC
- Slope, dams, tailing dams, embankment stabilization 4 ETC
- Deep foundations and design of the protector works 4ETC
- Roads geotechnics (all test in the roads) 4 ETC
- Environmental geotechnics 4 ETC
- Tunnels 4 ETC
- Geosynthetics 3 ETC
- Soil dynamics 6 ETC
- Sismotectonics 4 ETC
- The security problems in geotechnical structures and codes in geotechnic 3 ETC
- The professional practice 12 ETC
- Diploma work 30 ETC

So in 120 ETC of the second cycle the geotechnical matters have about 60% of them. These is inaf to prepared good quality geotechnical engineer

The professional practice and diploma work are tied by geotechnical problems.

Argumentation for the preparation of the geotechnical engineer

The preparation of the geotechnical engineer in Albania is tied with some reasons.

- The geological and geotectonic construction of our country is such where are present all kind of the rocks mass and soils.
- Our country is 75% mountainous and hills and 25%is field. Almost of the fields are composed by weak and problematic soils.
- During 17 years in our country we have big development of the road infrastructure. In consequence of this we need of deep studies for.
  - Carriers of the materials for roads layers
  - The rocks massives and construction of tunnels in them
  - The stabilization of the unstable slopes
  - The design of the high embankments in very problematic soils.
  - The design of the bridges foundations etc

- In Albania was constructed above 600 dams (erath dams) for the different goals as:
  - Hydropower plants (three high dams 50m 100m and 175m)
  - Irrigation (about 600 dams with high from 5 to 50m)
  - Tailing dams (13 such with high from 20 to 50m)

  All these objects need for monitoring, maintenance, rehabilitation and management, and this is geotechnical work
- During last years in Albania have a big development of the cities as Tirana, Durres, Vlora, Fieri, Shkodra, Lezha, Korca etc. Since now we have problems tied with:
  - Construction of many 10 stories buildings
  - Construction of the new buildings nearly of the existing buildings.

  All these are very delicate problems and can resolved only by geotechnical engineers.
- The big development of the tourism in our country was tied with construction of the touristic villages, roads swimming pools etc which have many geotechnical problems
- Albania is a country with high seismicity. In the last time began to develop the discipline. "Earth geotechnics" very important to made correct design for the geotechnical structures.
- Albania is very rich by water sources, rivers torrents. So in our country are present the erosion phenomena's slope instability, suffosion phenomena's etc. To made these phenomena's no dangerous will undertake the engineering protective measures which are only geotechnical activity.
- During the 15-17 last years in Albania have sensation of the absence of the really geotechnical engineer. Frequently the geotechnical problems we have resolved in empirical and amateur manner and these is accompanied with material damages. For these reason Albanian Geotechnical Society in collaboration with Civil Engineering Faculty was organized many activities as seminars, work-shops, short courses with intention to inform civil engineers for the new methods, new technologies, for the right solution of the different problems etc. With all these is insufficient becauses doing to prepare direct geotechnical engineer which will be able to resolved the geotechnical problems, which are complicate delicate, hard and with big risk for environment economy and people.

Conclusions

The preparation of the geotechnical engineer is absolute necessity

We have the base for their preparation which are:

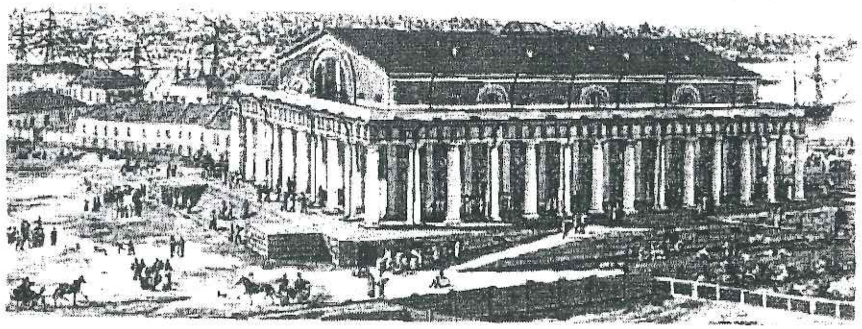
- 50 years experience in the field of preparation of the civil engineers.

- 50 years experience in the field of the geotechnics (soil mechanics and foundations).
- The many publications in these field (25 books)
- The creation of the Albanian Geotechnical Society
- The magazine Geotechnics
- The 7 course made with civil engineers from 1998
- The international activities made by AGS
- The scientific works in these field (about 35 works)
- The doctors thesis in these field prepared by Z.Dibra, N.Lacka. N.Nika. E.Doda. Y.Muceku. N.Shkodrani
- The many publications In the International Conferences in all the world (about 15 papers from 2002 years)
- The many students works with high quality (about 10-15 diploma work every year)
- For the efficacy of the Bologna process and for the quality of the first geotechnical engineers we will to speak after two year, when will termined completely all cycle (5 years).

Welcome to International geotechnical conference

Dear Colleagues!

We have the honour of inviting you to participate in the international geotechnical conference "*Development of Urban Areas and Geotechnical Engineering*", which will take place on 16-19 June 2008 in St. Petersburg. The conference will enjoy participation of the leading Russian and international geotechnical specialists.



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International Geotechnical Conference Development of Urban Areas and Geotechnical Engineering"
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The construction aspects of a multistory building above a medieval wall in Vlora city Albania

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Abstract

The city of Vlora in Albania has very difficult geotechnical conditions. , quaternary deposits (Q4) of very soft soil especially sand, silt, clay sand. On this kind of soil it's going to be built a multistory building and at the same time we had to preserve the medieval city defending wall. For this reason we have made a detailed geotechnical analysis. Some of the aspects of the analysis I have presented in this article. The type of chosen foundation is piled raft. For this type of foundation I have made comparisons with traditional raft methods.

General description

The building site is in front of Vlora stadium, which has been partly swamp zone and now it is used for the construction of high buildings. Geologic data for the city and the construction site are presented in the geologic report. Because of the zone difficulties and building complications we have made also a special seismic analysis.
In the construction site lie down a part of the medieval city wall of the Turkish fortifications for the port. (the plan and sections view are given with the drawings below).

Above them will be build a 12 story high building. We have thought to pass the medieval wall with deep beam of one story height which transfer the loads to new columns sideway the wall. The respective loads are taken from the uperstructure calculation programme (Robot Millenium). Foundations are thought to be piled raft with 1.5m thickness of raft and piles of 10-12 m length. The plate passes under the medieval wall. During raft construction the medieval wall stands over metallic profiles fixed in the pile's head before concreting them. They are joined with horizontal metallic beams on the flange of whitch are pushed on corrugated metallic sheet for supporting the wall. Excavation under the wall is made step by step after the metallic sheet floor is lean. The foundation reinforcement is made based in the Eurocodes , taking in consideration the atmospheric agresivity of the zone. The geotechnical and foundation calculations are made with Plaxis program.

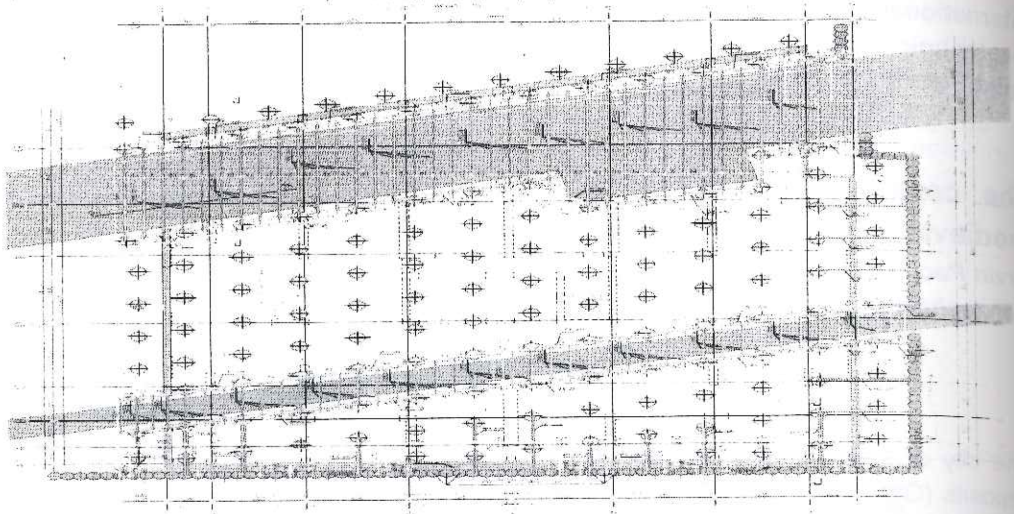


Fig 1 General plan view of the wall, piled raft and temporally excavation supports.

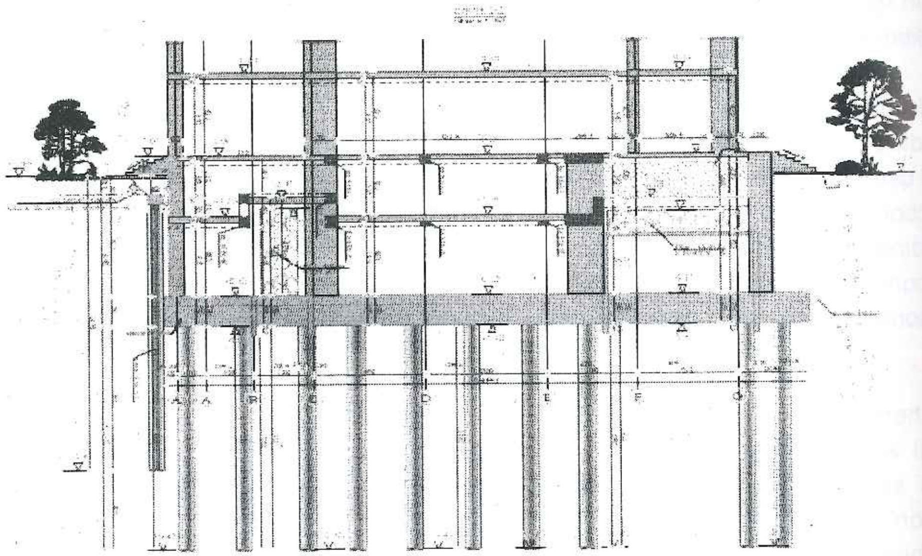


Fig 2. Cross section of the foundation

Geotechnical calculations

Geotechnical study

For this reason are made many borehole and also mesarements in situ (SPT) and laboratory test too.

As i have mention above geotechnic aspect are very complicated with abrupt changes and very soft sandy soils.

Quaternary deposit vary in thicknes from 30m to 50-60m depth not consolidated and with organic soils.

Deluvial deposits consist of sands , clay-sand mixtures of Quaternary (Q4).

Here after are some geotechnic characteristics of soils.(general parameters).

Tab. 1 Layers parameters

Soil	g	c	f	y	E
Fill	18.2	1	24	0	4000
Clay-sand	18.8	20	17	0	6000
Sand1	19.4	1	32	0	9500
Sand2	19	5	28	0	7500

The water level is 1m below the ground surface. The high presence of the water makes more difficult the foundation calculation and construction. Because of the characteristic of the zone, suphosion phenomens has a major influence, that's the reason that for the nearby building is taken special precautacion even if our construction doesn't interfere in the stress strain zone of the old one.

Selected models

To have a large view of the problem are taken in consideration several geotechnical calculation models.

From our experience and from the literature we concluded in two models for the foundation: raft and raft with piles.

Piled raft foundations are an economic solution in places where the raft only, doesn't satisfy calculation requirements. In such situation the additional piles improves the ultimate bearing capacity, settlements and different settlements and it reduce the plate thickness. although the limitations it's been chosen a 2D model for calculation and all the results are considered (under the appropriate safety margins) on comparison with the 3D model taken from the recommended literature. Hereafter is given graphically the model

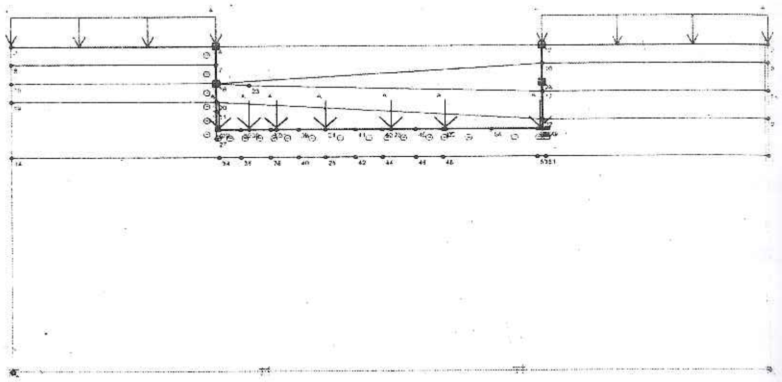


Fig. 3 2D model of raft

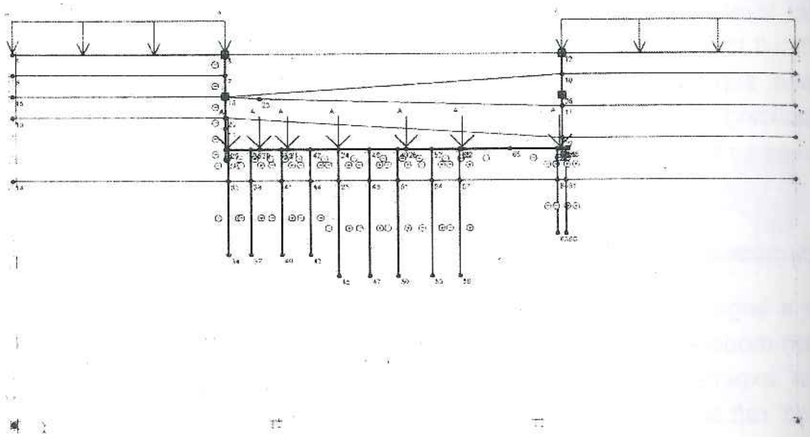


Fig. 4 2D model of piled raft

Initially for taking a general aspect of the problem for the geotechnical calculation we have chosen a Mohr-Coulomb soil behavior. For this model is checkup the soil bearing capacity and as it's seen we must add a consider number of piles to assure the bearing capacity and also to reduce the settlements.

The pile addition was imposed to be done because to pass the medieval wall we had to design a raft with high differences in concentrated loads and raft stiffness couldn't provide unified pressure and settlements.

Finally the models to calculate the foundation – soil system for both fundamental cases (raft and piled raft) are done by modeling soil with Hardening Soil Model.

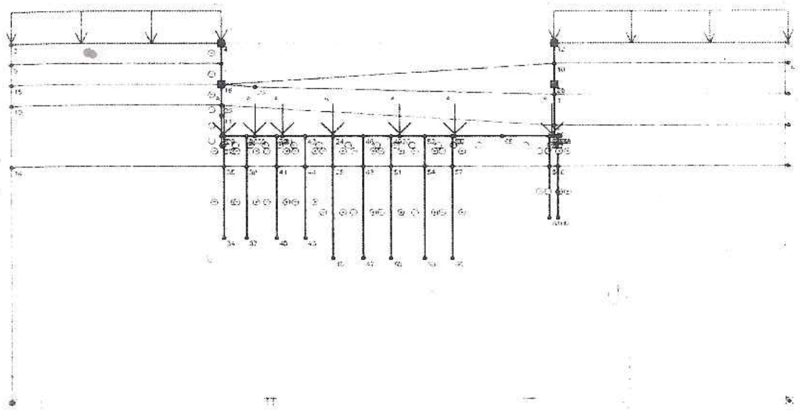


Fig. 5 HS Model

Hardening Soil parameters

Because there were no detailed laboratory tests, HS modeling parameters are taken from the literature using proper correlations. As we mention due to lack of direct tests we choose the minor values offered from the literature.

We underline again that their selection,due to no uniform mixtures inside the litologic formation (not uniform clay percentage and organic remnants) was considered inside the minor recommended limits.

For the type of soil found in our construction site (mostly sand) the selected values were base on SPT values performed in-situ. Those values are represented in the table below.

Tab. 2 SPT values and layers modulus

Soil	SPT	Eoed
Fill	10	4000
Clay-sand	15	8000
Sand1	25	25000
Sand2	15	15000

Comparison of the results

For both models the design parameters have been compared.
The raft foundation overpasses the bearing capacity of the soil, even for vertical loads so it can't be used. So we can use only the second type of foundation, piled raft. The problem consists in finding an optimal configuration of the piles (the number, position, length, the L/D rate) for the given case. For this intention were made some attempts until the solution with 1 concrete driven pile of 70 cm diameter for 9.4m^2 area of raft with different lengths in different parts of raft is chosen. So the displacement of the foundation and the influence over the medieval wall have to be uniform and of a degree of some cm, as to preserve the wall without demage. So the maximum settlements were reduced nearly to 10 cm and the differential one in 1.5-2cm which are inside of the allowed limits.

Bearing capacity of single pile

The primary calculations are made with the traditional methods as a single pile without taking in consideration the influence of the raft and interactions. The bearing capacity is also checkup with a axisimetric model of finite elements done with the program Plaxis. In both the cases we have the same results so the solution are justified. Based on the literature it is applicated in the head of the pile a moment that simulate resultant bending moments transfered from the raft during a earthquake and the interior forces based in vertical loads are modificated. Based on those interior forces are calculated the piles reinforcement. (Fig. 7) Axisimetric model is represented below

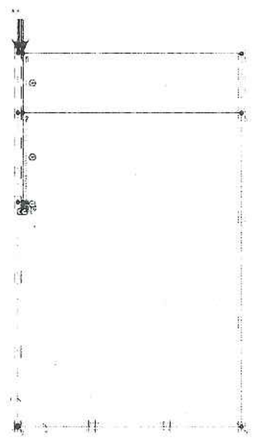


Fig. 6 Axisimetric model for single pile

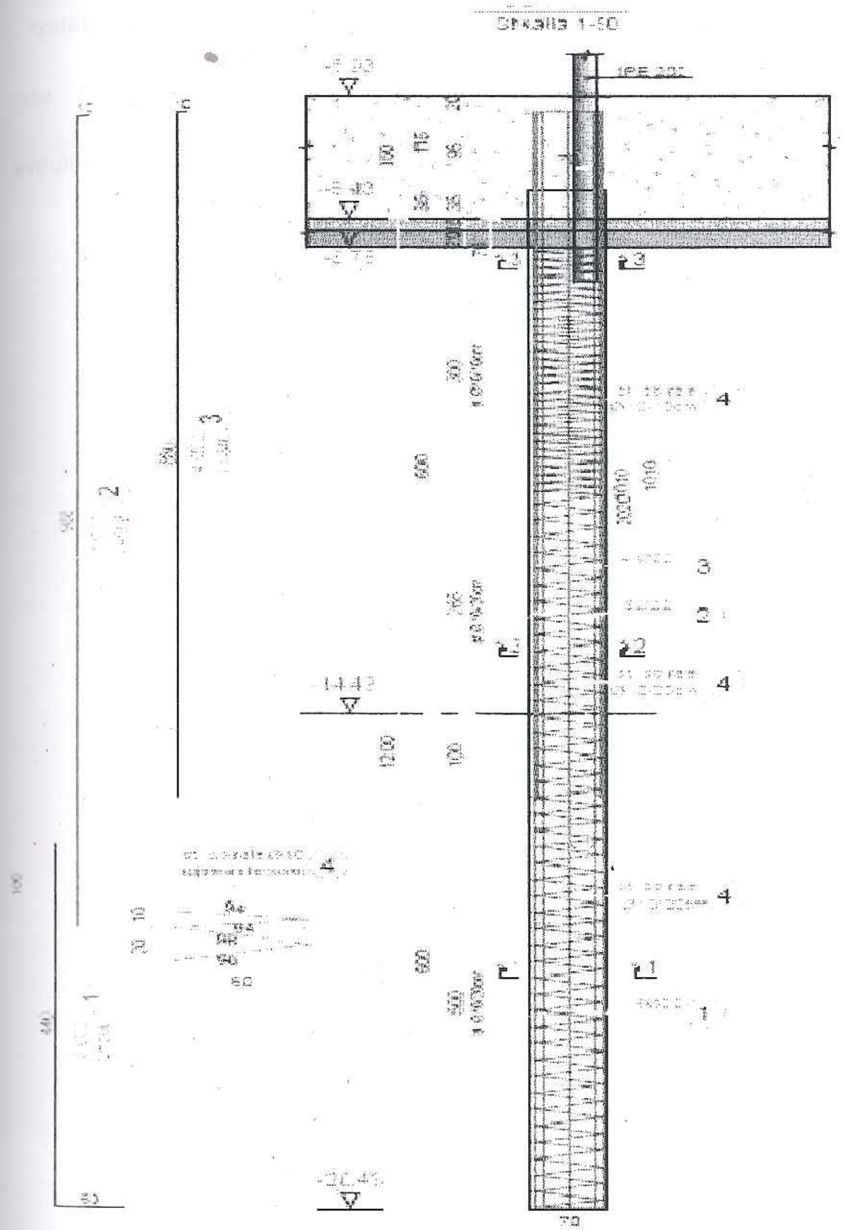


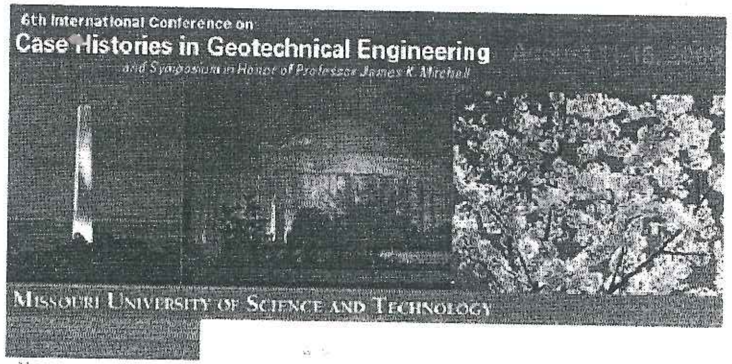
Fig. 7 Reinforcement of the pile

REFERENCES

TC18 Report, 2001. ISSMGE — Methods of analysis of piled raft foundation .

Hans-Georg Kempfert, 2006. Springer — Excavation and foundation in soft soil.

Th. Benz, R.Schwab, P.Vermeer. Plaxis Bulletin, Nr 18, 2005 — On the practical use of advanced constitutive laws in FE foundation analysis.



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Jose Clemente

6-th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA August 11-16, 2008

Failure of reatining structures in town lezha and their cosequence in neighbouring building.

Prof.Dr.Luljeta BOZO Civil Engineering Faculty. Tirana Polytechnics University, Albania.

Abstract

The Lezha town has a very complicate geology, high underground water table and high seismicity. Near of eight building was constructed e new building with two underground stories. The retaining structures at the time of excavation to the undergo damage to cause the serious problems on the existing building, In these paper I would like to present the analyses of phenomena and the engineering measures for rehabilitation of situation. The most dangerous phenomena in the studied area were suffusion problems. From these phenomena the existing eight stories buildings underwent the differential settlement, which arrived near the limit value of settlements. The engineering measures consist on creation of the concrete wall obstructive for the movement underground water and reconstruction of existing concrete wall to arrive rehabilitation of the situation.

Key words: retaining structures, suffusion problems, settlement, and reconstruction of concrete wall.

Introduction.

The Lezha town is located in northwestern part of Albania (fig.1) It's a historical cultural center and touristic place in Albanian, because it is near (10km) of beautiful Shengjini beach.

According of geological and seismical phenomena the town Lezha has e high seismicity and the effect of earthquake was underlined by soft soil deposit, which has a thickness 50-60m.

Soils are represented by the sands and silty sands with organic matters that are very porous. The underground water table is situated 0,5-1,0m from natural surface. During the 15-17 last year in Lezha town has had very fast development related to urban aspect. So many multi stories buildings (8-9 floors) with 1-2 underground floors are constructed on this area. During of this (the buildings construction) sometimes were happened the different damages, essentially by insufficiency of engineering measures.

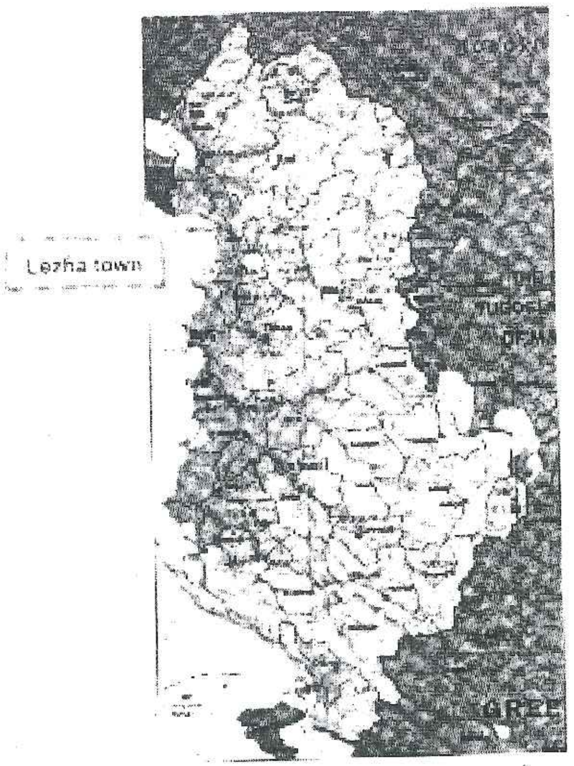


Fig.1 Localition of Lezha town

This is happened because some engineers are not competent in the geotechnical field and consequently they are not treated such specific phenomena, which are present in this kind of soils. Inasmuch, they haven't considered in right manner the soils-structure interaction in this area . In this paper we would like to present a characteristic phenomenon, which was occurred in the soils during deep excavation, the performance of the almost limit state in existing buildings and engineering measures for the rehabilitation of situation.

The characteristic phenomenon

Closed to eight floors building with mat foundation, which are situated on 3,8m deep from natural surface, was realized the two retaining structures (concrete wall) with dimension: length 12m and depth 12,5m. The concrete walls were constructed for new building which has two underground floors (fig.2) During the processe of excavation (when was arrived 3m down a mat foundation of existing building) the concrete wall was revolved and it had damage to caused in the existing mat foundation dangerous settlement. The settlements are not stabilized during the time, therefore they arrived up

to 8cm near to excavation and 1cm at other side of building. During all time of excavation it was pumped water from hole. After 5-7 days by settlements, the inclination of existing mat foundation arrived tag $\theta=0,0035$ very dangerous value because the allowable limit is tag $\theta_{limit}=0,0040$

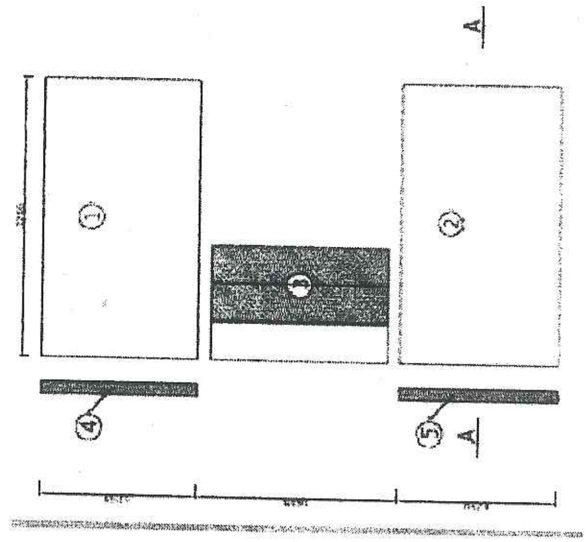


Fig.2ba The deformation of concrete wall 5 and inclination Of the building 2

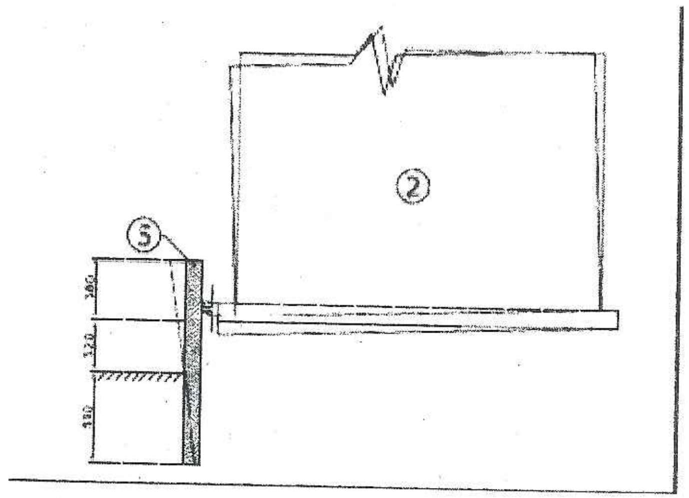


Fig.2a Plan situation of buildings and concrete walls 4 and 5

Analyses of the damages of retaining structures.

Geological Situation.

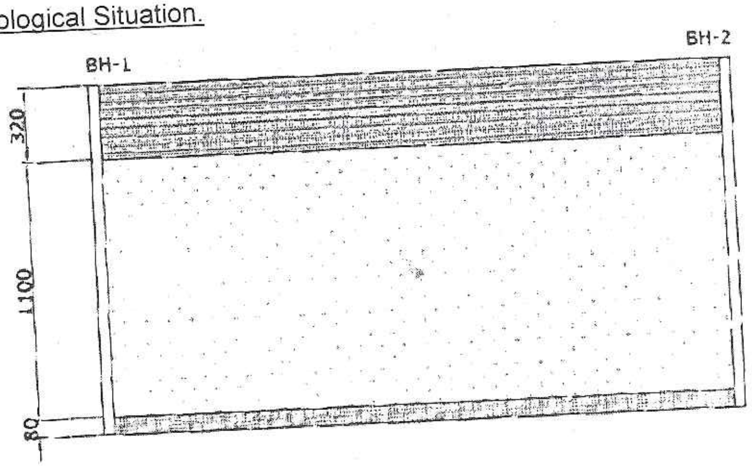
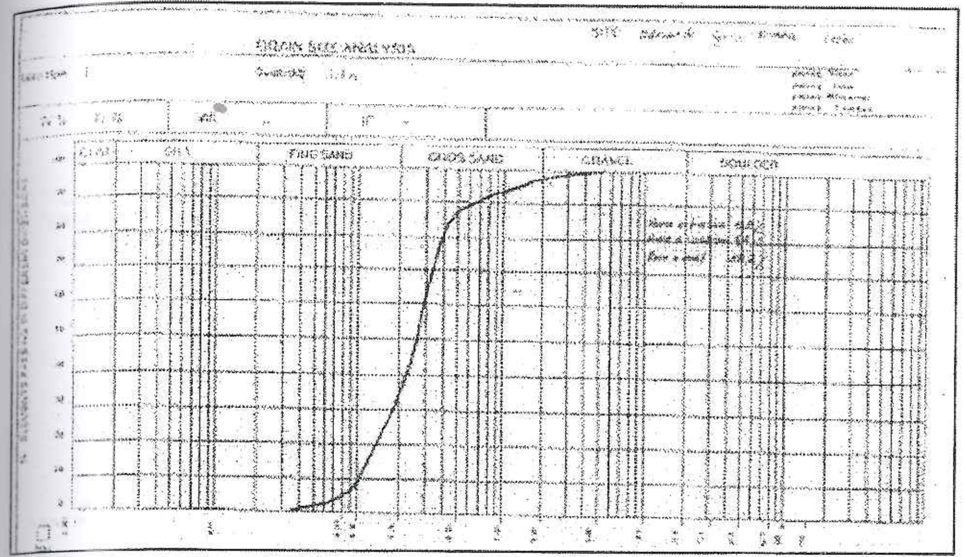


Fig.3 Lithological profile of investigated site

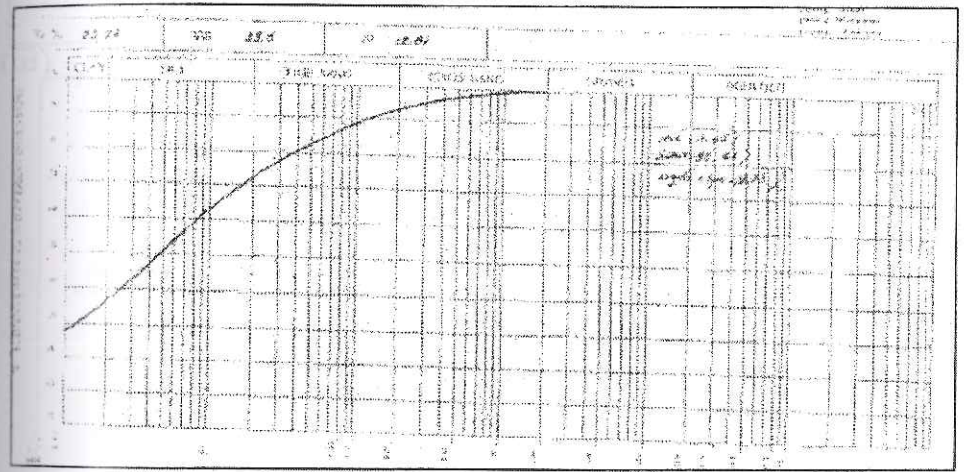
Initially we have realized some new bore holes to clarified the geological situation and to determinate exactly the geological profile and properties of soils. From bore holes result that geological profiles consist from three layers (fig.3) and underground water tables is 1-1,3m deep from natural surface,

Geotechnical Properties of soils.

From laboratories tests in undisturbed samples taken from drillings carried out in the investigated area we have the following results (table 1) and graphics in fig.4,5,6

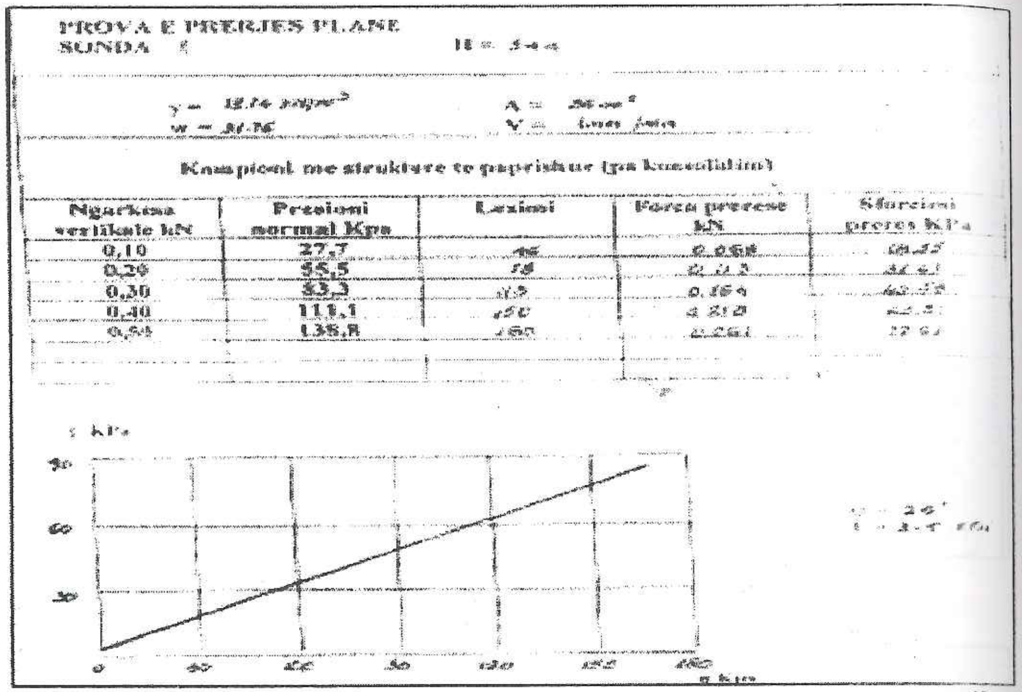


(a.1)



(a.2)

Fig.4 Grain size distribution.

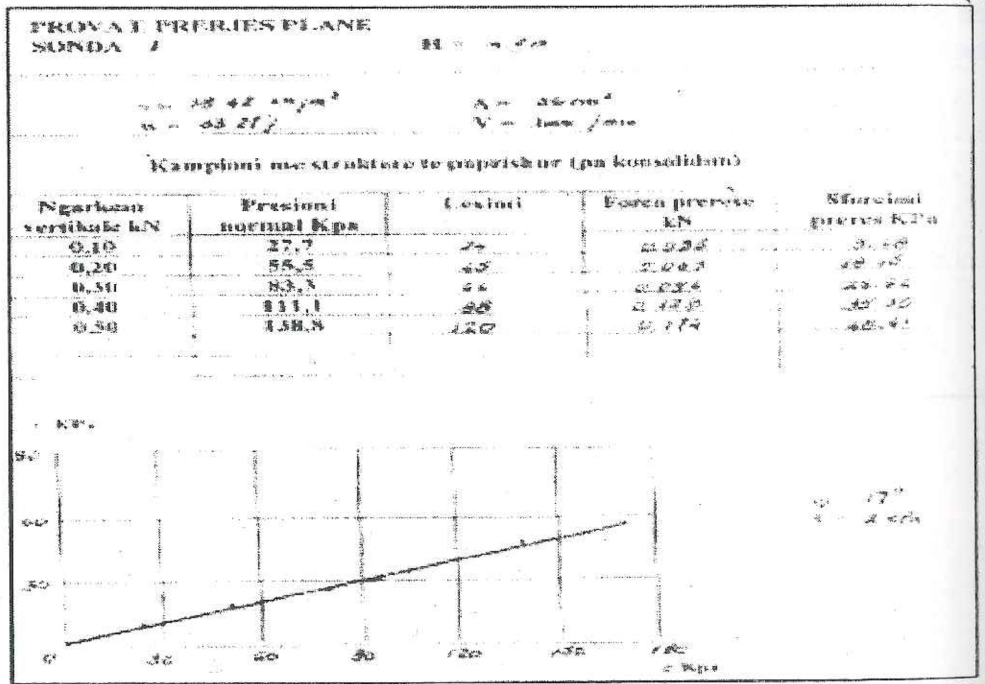


PROVA E PRERJES PLANE
SUNDA [illegible] H = [illegible]

Kampioni me strukture te paprishur (pa konsolidim)

Ngarkesa vertikale kN	Presioni normal Kpa	Lexioni	Forca prerese kN	Sforcimi preres KPa
0,10	27,7	[illegible]	0,068	[illegible]
0,20	55,5	[illegible]	0,113	[illegible]
0,30	83,3	[illegible]	0,164	[illegible]
0,40	111,1	150	0,210	[illegible]
0,50	138,8	160	0,261	[illegible]

(b.1)



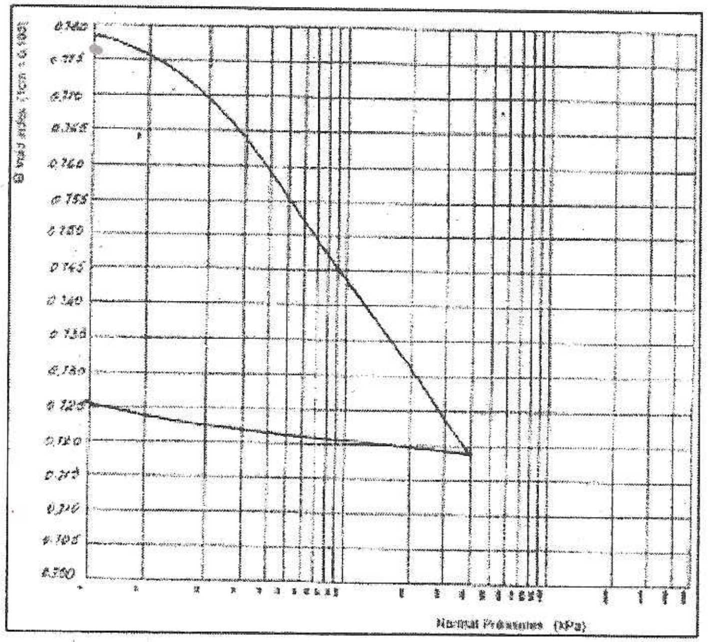
PROVA E PRERJES PLANE
SUNDA [illegible] H = [illegible]

Kampioni me strukture te paprishur (pa konsolidim)

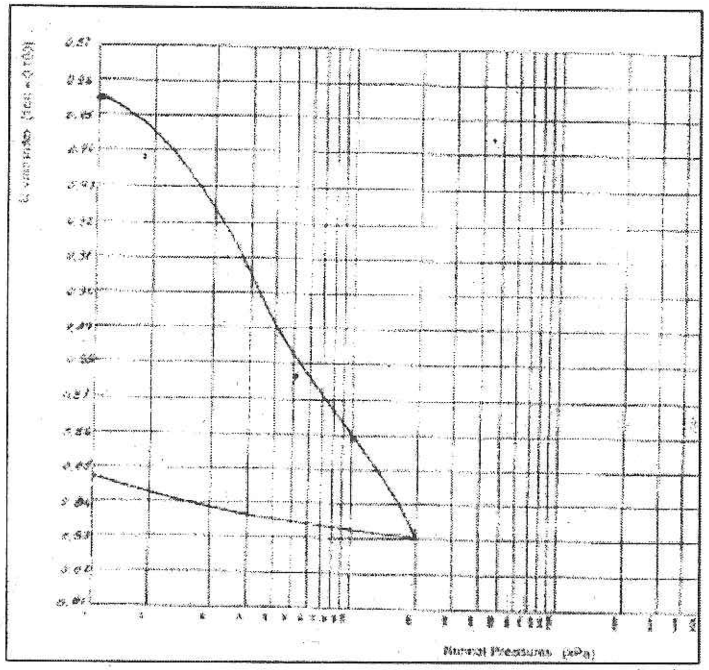
Ngarkesa vertikale kN	Presioni normal Kpa	Lexioni	Forca prerese kN	Sforcimi preres KPa
0,10	27,7	[illegible]	[illegible]	[illegible]
0,20	55,5	[illegible]	[illegible]	[illegible]
0,30	83,3	[illegible]	[illegible]	[illegible]
0,40	111,1	[illegible]	[illegible]	[illegible]
0,50	138,8	120	[illegible]	[illegible]

(b.2)

Fig.5 Result of direct shear tests



(c. 1)



(c. 2.)

Fig.6 Results of consolidation tests

Table 1

Nr	Properties	Unit	Layer 1	Layer 2
	Specific gravity	γ_0 kN/m^3	27,03	26,65
	Bulk density	γ kN/m^3	18,42	19,74
	Moisture content	W -%	33,27	31,76
	Void ratio	E	0,9556	0,7788
	Liquid limit	W_L - %	39,5	-
	Plastic limit	W_p %	26,79	-
	Relative density	Dr. %	-	55
	Frictional angle	φ	17	20
	Cohesion	C –Kpa	2-3	3-5
	Compres.index	C_c	0,16-0,1	0,09-0,053
	Swell index	C_s	0,02-0,01	0,005-0,007
	Consolidate coef.	C_v cm^2/s	0,00179	0,0046-0,0029
	Soils classific.		OL	SP and SM

From the laboratories tests we have calculate:
The bearing capacity of soils for the layer Nr.1 R=120-140 KPa and for layer Nr.2 R=150-170KPa, that indicate for the presence of a weak basement.
The soils are classified in "C" category by EC-7 and "D" category by EC-8.
This situation isn't regarded well by engineers to made good design of retaining structure.

Verification of necessary depth fixation of concrete wall.
After our calculation to consider wall as sheet pile wall (cantilver pile) and to report (refer) a new geological profile and new soils properties results that.
The necessary depth fixation concrete wall shall be 13,8m. In consequence the concrete wall is unstable.(fig.7) The existing fication depth was 5,5m

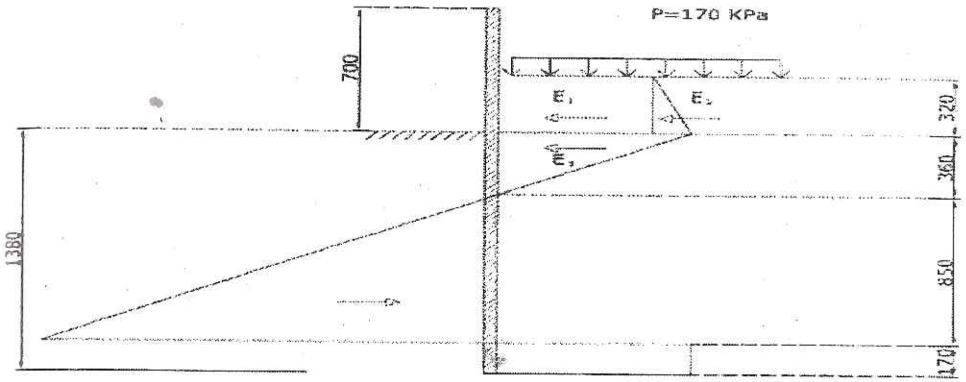


Fig.7Sheet pile walls and earth pressure diagrams E_I=308KN/m E_2 =55KN/m, E_3 = 205KN/m

Verification of hydraulic natural gradient and its comparison with critical hydraulic gradient.
During the excavation of hole it was pumped continually the water from, the bottom of hole. When was arrived the depth 7m from natyral surface we have a hydraulic gradient i=5,7/2=2,85 (L=1,5-2m distance of holes and ΔH=7,0-1,3=5,7m piezometric head).
By physical properties of second layer we have calculate the critical hydraulic gradient.

$$i_{cr} = \left(\frac{26,72}{10} - 1\right) \cdot \left(\frac{1}{1 + 0,7788}\right) = 0,91$$

So, we have very critical situation because of $l > l_{cr}$ and it is sure that suffosion phenomenon will be occurred.
Finally by analysis of damages of existing concrete wall (5) we take the following conclusion:
It has had two errors in design process.
The incomplete acknowledgment of soils situation and their properties, and occure of suffosion phenomenon.

The engineering measures to stabilize the situation.
First of all immediately we take engineering measures to put in the bottom of the excavation hole of 4m. By our calculations results that after application of 100KPa pressure (4m x 25KN/m^3)of concrete (beton) the concrete wall make sure temporally to be fixed in 5,5m depth under the bottom of excavation (fig.8).

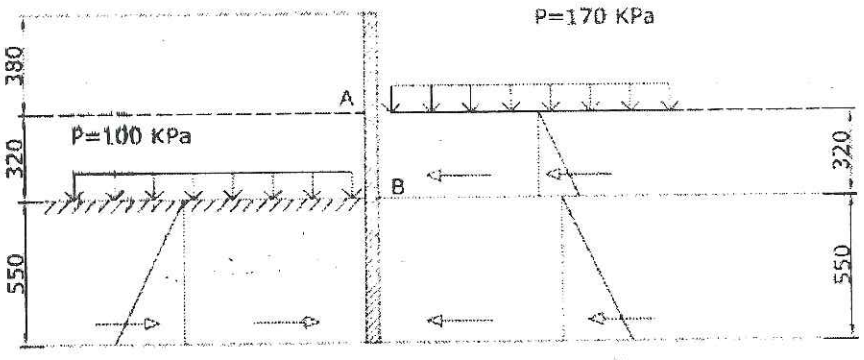


Fig.8 Sheet pile wall after putting of 4m concrete

The existing concrete wall, after excavation was enough damages (injury). For these reasons we have constructed the new sure and resistant retaining structure. We have choose the piles with diameter d=70,0cm fixed under the bottom of hole 8,0m depth and with report L/d = 8,57 (fig.9)

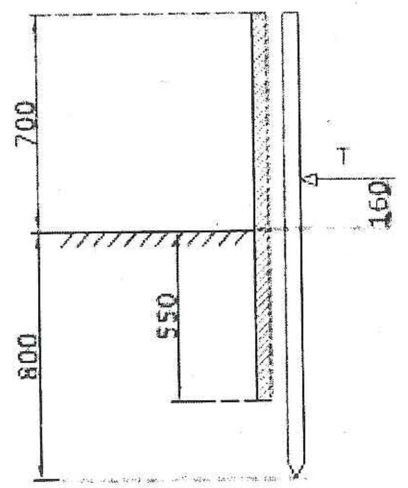


Fig.9 The lateral loaded pile

The bearing capacity of the piles in horizontal load is H=417KN.
The horizontal load from earth pressure is T=418KN/m. We have situated closely piles to have a factor safety F_s = 1,43. At the same time we have realized mortar cement wall against the seepage in the fine sands soils.

For the second concrete wall (4), where it is not coming the excavation we have executed (accomplished) a reconstruction. We have conceived this retaining structure as anchored sheet pile wall or with support. On the first phase we can to excavate until 3,5m depth from natural surface and we can to put the first support. Then we can continue excavation until 7,0m depth. The real fixed depth of concrete wall was 6,5m.From our calculations this depth is insufficient for the stability of retaining structure. The necessary length of concrete wall will be 13,7m (fig.11)

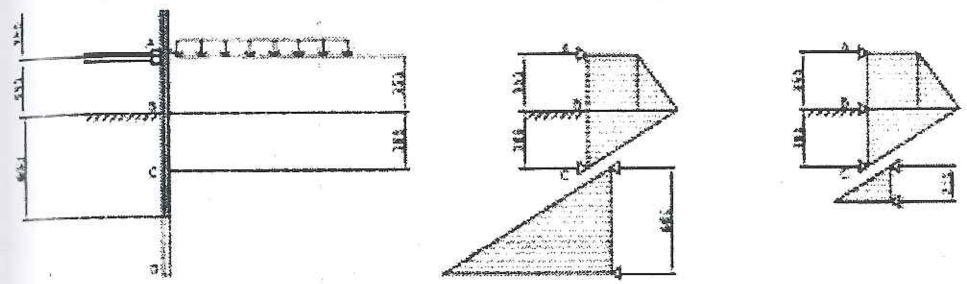


Fig.10 The scheme of second sheet pile wall one and two support

So, it is necessary to put the second support (fig.10b) to insured a resistible retaining structure. The second support shall to put in the bottom of the hole. On second phase, when we can realized the mat foundation of new building, which transmits in the basements 130 KPa pressure. After that we can to take off the support and in these conditions the necessary fixed depth of retaining structure will be 3,5m (fig.11)

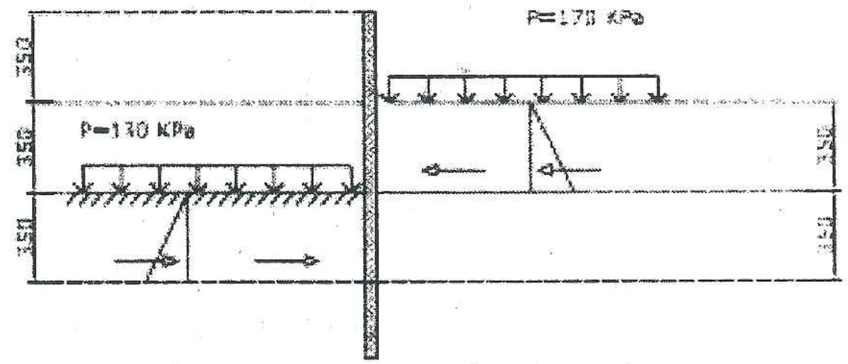


Fig.11 The schema for calculation of second phases

Conclusions

Construction of the underground floors on the weak soils and especially on the saturated fines sands or silty sands presents some serious problems:

The performance of the hydraulic gradient $i > i_{cr}$, shown the suffosion phenomenon has occurred, which is very dangerous for the stability of retaining structures and neighbouring buildings.

The occurrence of the suffosion phenomenon is accompanies with increase of porosity of soils and in consequence they are present the supplement settlement to cause in the existing building the occur of limit state.

The engineering measures in any case will be in conformity with real conditions of soils and they depend from retaining structures attempt rapid stabilization of structure.

REFERENCES

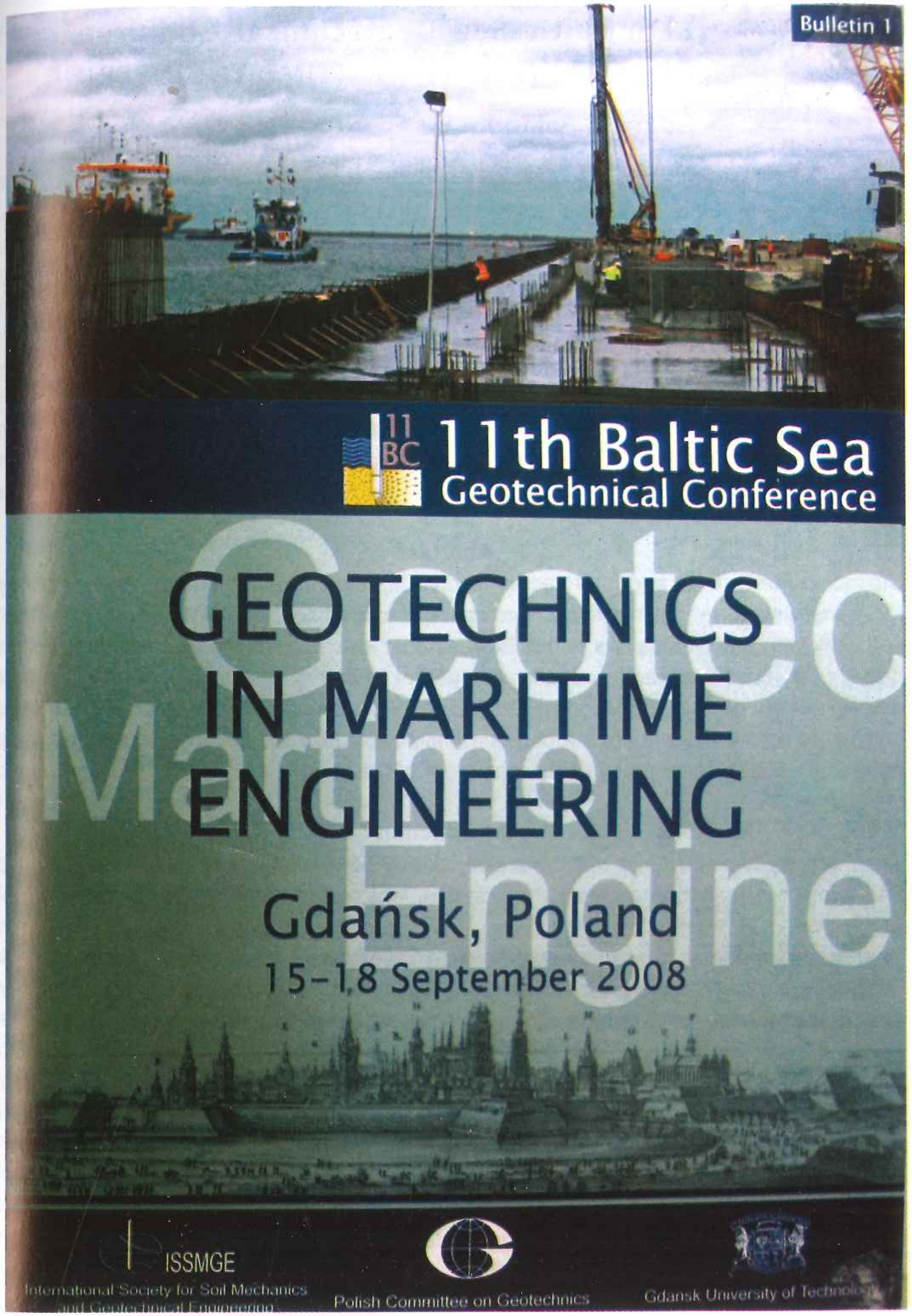
Bozo L (2007) Geotechnics. Civil Engineering Faculty. Tirana Polytechnic University Albania

Braja M (1994) Principles of foundation engineering. PWS-KENT Publishing Company.

Prakash Sh (2004) Pile foundations under seismic loads (with applications). Workshop on Geotechnical Earthquake Engineering. Tirana Albania.

Viggiani Ç (1999) Fondazioni. Helvius Edizione.

Brand H (2007) The collapse of deep excavation pit in urban surroundings. Proceedings of the 14 European Conference on soils Mechanics and geotechnical Engineering Madrid, pp 513



11th Baltic Sea Geotechnical Conference
GEOTECHNICS IN MARITIME ENGINEERING
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15-18 September 2008

Geotechnical study of industrial zone in reference to limit state

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Abstract

Closed to the Durresi port is projected to build a new cement grinding plant with silos, mill construction, magazine, compressors room and office. The site is characterized by soft soils with a thickness that varies from 70m up to 85m. In this paper we would like to present a case research related to geotechnical characterization of site, the determination of the geotechnical model and to forecast the future phenomenon correlated with limit states of structures. Finally we would like to give some correlations between mechanical and physical parameters of the studied soils.

Introduction

The Durresi town and its rounded zone consist by soils deposits with immense potential and such mechanical and physical properties, which classified they in "C" category by EC-7.

That means these soils are soft and from very weak basement, where are included in third geotechnical category. Historically, the Durresi zone was subdued many times by destructive seismic phenomena and their soils are classified in "D" or "E" category by EC-8. In the zone, which we have realized the geotechnical study is planned to constructed a an industrial plant with very strict requirements to limit states. The calculations of different structures need severe geotechnical models and forecast the negative phenomena, which can occur during exploitation period of this plant.

Geological setting

In the investigated area the followed geological formations are found:

1. Quaternary deposits.

These deposits construct the first terrace of Erzeni river. They are represented combination of silts formations and clay formations, which are intercalated by thin layers of fine sands in middle of soils profile and coarse sands in lower part of soils profile. The quaternary deposit thickness in the studied area range from 40, m up to 60 m

Molasses rocks.

According to geological construction clay stones and siltstones rocks predominate to sandstone rocks. From the structural point of view the studied area, takes place in folded structure, which represent by Erzeni syncline. Here the claystones rocks are about 44% of the litho logical profile. They have grey and gray-brown color. Concerning mineralogical composition the clay stones are ilite and montmorilonite type. The siltstones, rocks also have gray and gray-brown color, and build 42% of litho logical profile. Whereas the sandstones rocks consist fine up to quartz, feldspars, micas and carbonate fine up to medium grained, These are cemented by clays material. The sandstone rocks represent 6-7% of litho logical profile. They covered by quaternary deposits.

Hydrogeology

The observed site related to hydrogeology is constructed by two complexes of rocks. They are quaternary deposists and molasses rocks. The lower part of quaternary deposits is build by coarse sands and in the middle and upper part by silts and clays soils are intercalated by thin layers of fine sands. The underground water in the region, part which is the studied area, are related to sands formations which formed a rich aquifer according to water bearing. The main of water recourse these formations is Erzeni River.

The second of complexes represent by the molasses rocks that built from clay stones, siltstones and sandstone rocks. The clay stones and siltstones are very poor related to water bearing, whereas the sandstones formations are a good aquifer to underground water reserves. During drilling works, systematic measurements of underground water levels are made. This procedure is repeated after 24 hours as well as. It is concluded that the underground water table in the Quaternary deposits is 0,9 up to 1,4m deep.

Methodology

We carried out 30 boreholes up to 30m deep and we take of them samples (undisturbed 35 and disturbed 10) to analyses in laboratory for determination of mechanical and physical parameters by ASMT standards. From them we have got the geotechnical parameters, which are give in table 1,2, fig.1 and fig.2. Also according to mechanical and physical parameters we distinguished 10 geotechnical layers of soils.

Tabel 1 Percentage of the particles in each layers.

Layers	Sands	Silts	Clay
2	24-26	70-68	6
3	2-10	73-76	14-24
4	15-25	52-50	24-32
5	28-29	58-81	8-14
6	19	64-65	16-17
7	13-21	58-78	9-21
8	16	53	31
9	48-65	30-49	3-5
10	65	35	10

Table 1,2

Layers	UCS classification	σ KPa	C_c
2	CL-ML	130-170	0.07-0.1 0.16-0.18
3	ML-ML	170-210	0.07-0.08 0.06-0.08
4	ML-OH	180-250	0.05-0.1 0.08-0.12
5	ML-CL	150-180	0.08-0.15 0.12-0.24
6	CL-ML	200-210	0.11-0.16 0.07-0.11
7	CL-ML	130-240	0.1-0.14 0.06-0.014
8	ML	150	0.09-0.016
9	SM	120-150	
10	SM	150	

Table1.3

Layers	C_s	C_v cm^2/s	C_u KPa
2	0.007 0.04-0,026	0.00277 0.00450	15.9 15.9
3	0.01-0.02 0.02-0.04	0.00175 0.00465	52.5 28.6
4	0.02-0.04 0.017-0.05	0.00112 0.00112	45.3 103.4
5	0.01-0.02 0.02-0.03 0.01-0.02	0.00069 0.00502 0.00141	42.9 55.7 31.8
6	0.027-0.018 0.018-0.024	0.00135 0.00277	29.4 69.9
7	0.023-0.028 0.01-0.02 0.01-0.011	0.00270 0.00183 0.00701	22.3 42.9
8	0.03-0.04	0.00502	

Table1.4

Layers	K cm/s	φ	C Kpa
2	$0.5x10^{-7}$ $0.97x10^{-7}$	25 19	7 6
3	$0.22x10^{-7}$ $0.66x10^{-7}$	23 22	5 12
4	$0.65x10^{-7}$ $0.12x10^{-7}$	19 20	12 20
5	$0.18x10^{-7}$ $1.97x10^{-7}$ $0.35x10^{-7}$	22 19 24	7 5 0
6	$0.24x10^{-7}$ $0.5x10^{-7}$	19 22	4 15
7	$0.7x10^{-7}$ $0.5x10^{-7}$ $1.42x10^{-7}$	24 24 17	5 2 5
8	$1.27x10^{-7}$	10	18

Table 1.5

Layers	E.10^4 Kpa	OCR	K_s kg/cm^3
2	0.4-0.5	=1	29
	0.37-0.53	>1	25
3	0.68-0.76	>1	2
	0.5-0.7		35
4	0.5-0.7	>1	30
	0.6-0.85	>1	45
5	0.37	0.66	19
	0.25	0.60	13
	0.28-0.48	0.46	20
6	0.25-0.35	0.37	20
	0.4-0.42	0.69	27
7	0.29-0.39	0.33	20
	0.44	0.36	20
	0.38-0.42	0.15	25
8	0.34-0.39	0.2	20

Tabele 2.1

Layers	I_k	I_p	e
2	0.83	10	0.79
3	0.297-0.354	17.7-19.3	0.84-0.89
4	0.0-0.043	21	0.8
5	0.55-0.78	17.5-18	0.88-0.93
7	0.68	13.9	0.99
8	0.480	22	1.02

Tabele 2.2

Layers	γ_d gr/cm^3	γ_0 gr/cm^3	W %
2	1.33	2.5	44.64
3	1.36-1.38	2.55	46.0
4	1.42	2.56	57.0
5	1.31-1.35	2.54	44.0
7	1.35	2.69	40.0
8	1.33	2.695	52.0

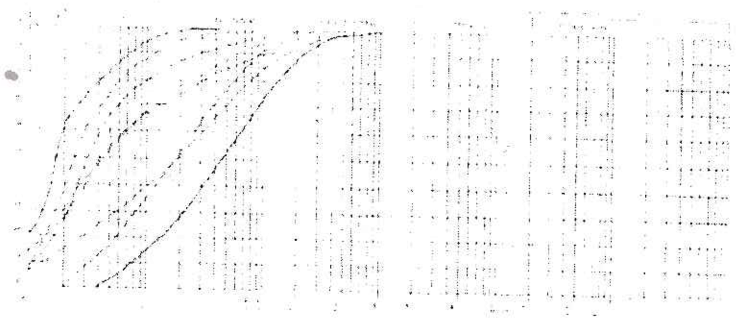

Fig.1 Grain size distributions of studied soils

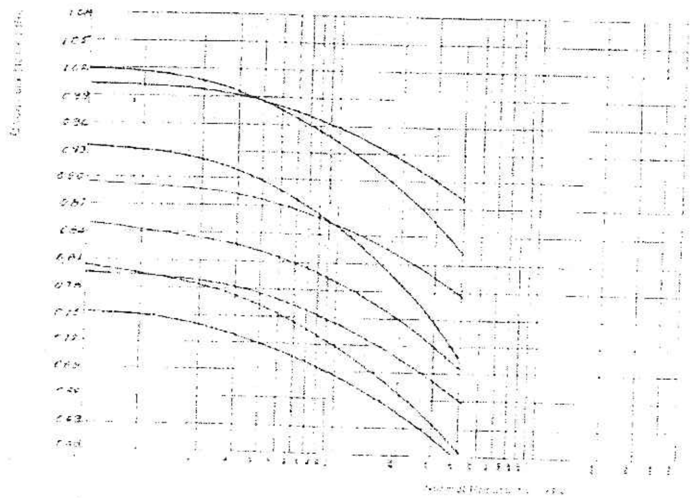

Fig.2 Oedometric curves of studied soils

Correlations between physical and mechanical properties.

From laboratory test and geological study we have take the followings correlations: The bearing capacity [σ] with % silts, which are given in fig.3

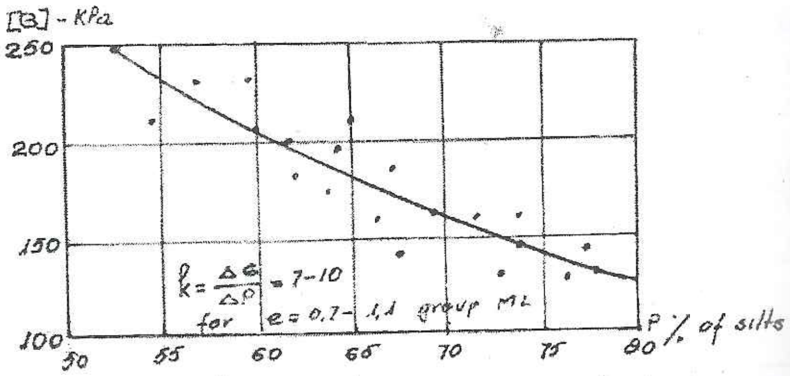

Fig.3 Correlations of bearing capacity [σ} with % of silts

The elastic springs coefficient [K_s] with % of silts, which are given in fig.4

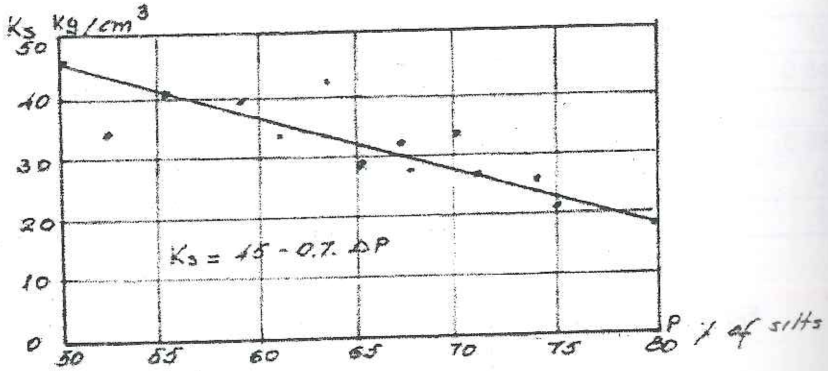

Fig.4 Correlations of elastic springs coefficient [K_s] with % of silts

The swelling coefficient [C_s] with % of silts, which are given in fig.5

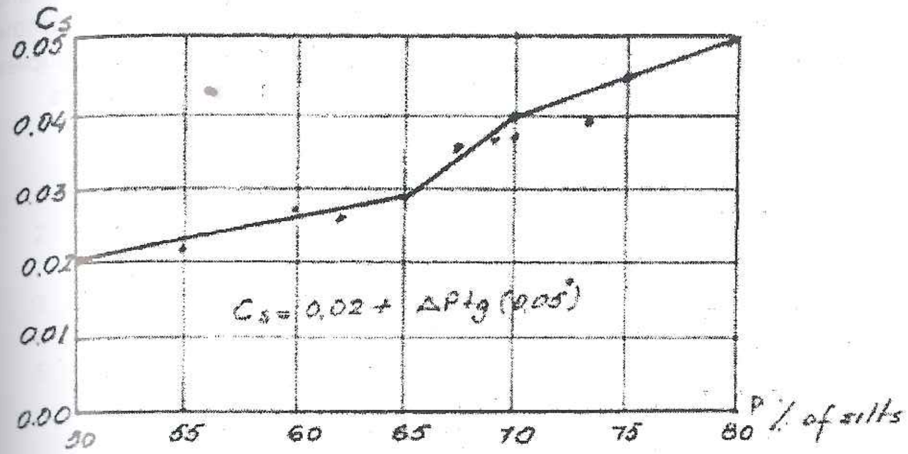

Fig.5 Correlations of swelling coefficnet [C_s]with %of silts

The undrained cohesion [C_u] with % of silts, which are given in fig.6.

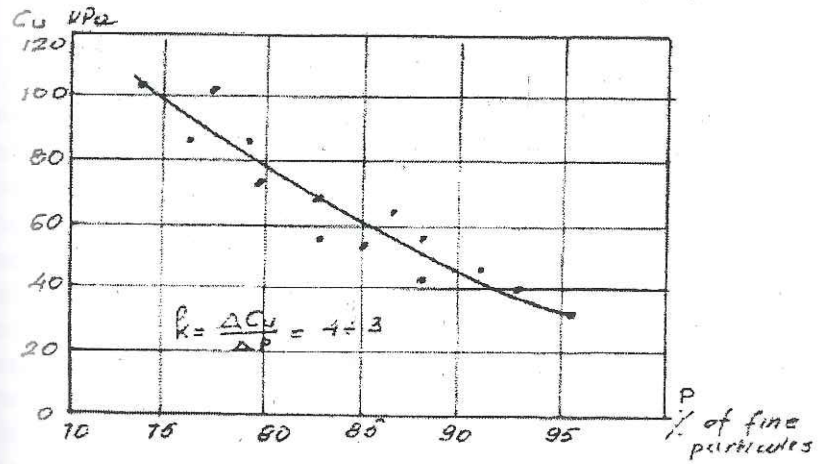

Fig.6 Correlations undrained cohesion [C_u] with % of silts

The geotechnical models

From our study we have determinate two geotechnical models 1 and 2. The first models consist from 8 geotechnical layers and second models by 8 layers , as well, but in other order Table 3,4. In the geotechnical models for the more important object of this plant we have calculated their probable settlements, which can be arrived 38-40cm and these process can extend a minimum 10 years. The calculations are realized for the shallow mat foundations with diameter D=30,0m. If can to used deep foundations (piles foundations) we have determined the specific friction force (KN/m^2) for each layers.

Table 3

Models 1

Legend	Laye.	Thic.	γ	φ	C	I_P
	1	0.5m				
	2	2.0m	1.75	19	6	11
	3	2.5m	1.84	22	8	18
	4	3.5m	1.86	20	15	21
	5	1.7m	1.84	21	5	17.5
	6	1.5m	1.85	20	8	19
	7	11.0 m	1.85	23	4	14
	8	7.3m	1.84	10	18	22

Table 4

Models 2

Legend	Layer Nr.	Thick.	γ	φ	C	I_P
	1	0.5m				
	2	1.1m	1.75	19	6	11
	10	1.5m	1.78	25	0.0	-
	4	2.5m	1.86	20	15	21
	5	3.5m	1.84	21	5	17.5
	6	1.0m	1.85	20	8	19
	7	10.0m	1.85	23	4	14
	8	7.4m	1.84	10	18	22

Conclusions

From laboratory test, we have the following conclusions:

1. The layers 2,3,5,6,7,8 are classified in group "ML" by UCS and they are presented by inorganic silts, clayey silts and loam silts. some times several of them are represented by inorganic clays or silty clays (CL). Layer 4, is included in "OH" group and is represented by organic silts. Layers 9 and 10 represent the fine sands and silty fines sands "SM"
2. The layers of "ML" group have the medium and high consistency. Medium consistency I_k = 0,25-0,50 layers 3,6,8
3. Soft soils-consistency I_k = 0,5-0,75 layers 5,7
4. Very soft soils consistency I_k = >0,75 layers 5,7
5. The soils have very high silts and loam percentage. That verified by:
   - Grain size analysis-silts fractions range from 50,0 up to 81,0%
   - Specific density range from γ_0=2,49 up to 2,56 gr/cm^3.
   - Liquid limit range from 40,0 up to 50,0%
6. The porosity of soils has a high value e=0,75-1,1 and together with their uncosolidations state OCR<1 will be caused high value of deformation. The ranges of:
   - Deformation modules E=(0,25-0,6).10^4 KPa.
   - Compression index C_c=(0,07-0,16) and some cases C_c = 0,24.
   - Spring coefficient K_s =20-27kg/cm^3 and some cases K_s=13-19 kg/cm^3.
   - The high percentage of silts and loams to be caused impermeabilities layers. The permeability coefficient from odometer tests results K=(0,2-0,9).10^{-7} cm/s .In other part we have swelling soils because C_s =0,01-0,03 or C_s (1/5-1/7)C_C
7. The undrained cohesion "C_u", which has value commonly C_u = 16-50KPa and in some cause C_u 60-100KPa is determined by two factors as, the percentage of silt and the high saturations state I_k 0≥0,5
8. The low value of friction angle "ϕ" and cohesion "c" determine the low or medium bearing capacity of soils.
9. Definitively the industrial zone, where is projected to construct a industrial plant consist by very deformability basements, so all construction is necessary to design in conformity to expectance big settlements of soils.
10. The soils have high percentage of silts participles, upper underground water table, as well as, they are in unconsolidated state and in the earthquake phenomena, in these soils can occurred the liquefaction process, that's means it is very necessary to calculated the potential of liquefaction.

We recommend to determinate the right depth of foundations and kinds of the foundation, as well as, the most reinforcement method for improvement of the basements.

REFERENCES

Bozo. L Muceku Y (2007) Geotechnical study of construction site of cement plant. Shqiperia Cement Company, Durresi town, Albania.

Bozo L. (2007) Geotechnics and soils mechanics. Polytechnics University of Tirana P.457

YEAR 2009

- International Symposium on Geoenvinronmental
  September 8-10 2009 Hangzhou China.

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  October 5-9 2009 Alexandria Egypt.

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September 8-10,2009

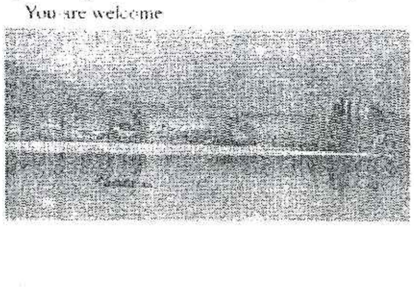
Introduction

Issues associated with Environmental Geotechnics continue to be a major preoccupation for governments, public and private organizations and the general community wordwide. The Chinese Government has been putting great effort on environmental issues, including treatment of wastewater, sanitary disposal of solid waste, prevention of groundwater contamination etc. China also has much to share on the opportunities, challenges and responsibilities for environmental geotechnics with other countries.

Hangzhou is the capital of Zhejiang Province. She is renowned for her natural beauty and charm of the West Lake. In the Yuan Dynasty, the famous Italian traveler Marco Polo described this city as "*the finest and most splendid city in the world*". Hangzhou has also been known as the "*land of silk and tea*". Hangzhou has been making great effort on environment protection, and had won the honor of "*National Model City for Environmental Protection*". In 1991, Tianziling landfill, the first sanitary landfill for municipal solid waste in China, was built in Hangzhou.

Zhejiang University, founded in 1897, is one of the oldest and the top universities in China. MOE Key Laboratory of Soft Soils and Geoenvironmental Engineering was established in the university.

You are welcome



General Theme

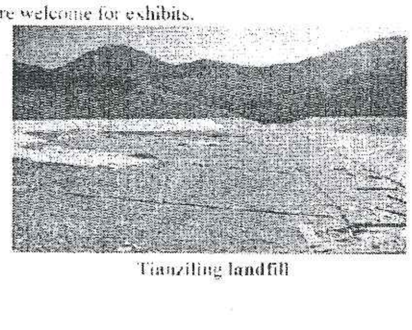
Reclamation of the Past and Foward to Sustainable Geoenvironment

Themes

- Basic and advanced theories for modelling of geoenvironmental phenomena
- Testing and monitoring for geoenvironment
- Municipal solid wastes and landfill engineering
- Sludge and dredging
- Geotechnical recycling and reuse of industrial wastes
- Mine sites, tailing dams and sludge ponds
- Engineering barriers for radioactive waste disposals
- Contaminated land and remediation technology
- Applications geosynthetics in geoenvironment
- Geoenvironmental risk assessment, management and sustainability
- Ecological techniques and case histories

Technology Visit and Exhibition

Tianziling landfill and Xixi wetland will be visited.About 2000m^2 exhibition space can be provided. Lab. field testing, instrumentation, geosynthetics, and other geotechnical manufacturers, suppliers, contractors, installer, and consulting firms are welcome for exhibits.



Tianziling landfill

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and others

Geonvinronmental risk assesment in Albania

Prof.L.Bozo — *Geotechnical Department of Civil Engineering Faculty Tirana, Albania luljeta_bozo@yahoo.com*
Prof.As.Gj.Ikonomi — *Environmental Department of Civil Engineering Faculty Tirana, ALbania*

Abstract

During the last 15-19 years in Albania was evident many phenomena's which are tied with tailing dams, deformation of mine areas, contaminated lands ect. In this paper we would like to present our work about evaluation of geoenvironmental risk, and our proposal for the remediation of the situation in Ohrid lake.
Key words : lake, contamination, pollution, engineering measures, geoenvironmental.

Introduction

Albania is a rich country with minerals. Albania had a developed mineral industry and consequently tailing dams, which served as deposits for residue from the mineral industry. During the last 20 years these tailing dams have been a dangerous factor for the contamination of the geoenvironment. In the other hand, over the coal mines near Tirana and Memaliaj cities were constructed 2-3 floors houses, they are damaged by enormous deformation of the ground. One part of our territory near of the chemical factories which are abandoned during this last 20 years, was contaminated with very dangerous elements like arsenic, sulfur, phosphate ect. We want to reveal the risk that menaces the geoenvironment from the causes mentioned above, to make the evaluation of this danger and to predict the measures for the minimization of danger.

Evaluation of geonvironmental risk in Albania.

Before 1990 in Albania there was a big development of the industries like the:

- Industry of extraction of minerals: Cr, Cu, Fe, Ni,C
- Industry of treatment and enrichment of minerals as metallurgic and siderurgic works.
- Construction of tailing dams.
- Chemical industry particularly of fertilizers in Lac, Fier, Durres cities.

After 1990 this industries stopped their activities completely or partially. Most of them was abandoned without being monitored their effects in the environment. They weren't maintained and worst of all was that they got permissions to construct buildings and to create urban areas in this zone. It is evidenced that the geoenvironment is threatened from this event in three ways.

- Contamination of soils and the underground water which influenced the development of flora and fauna.
- The population takes the dangerous elements from the plants and animals that grow in these zone.
- The danger for serious or incurable maladies which can affect in inhabitants which have constructed their houses in the contaminated zones.

To protect the geonivoronment and the live of the people we think that we must take some defensive measures as:

- To inform and to prepare the population for the danger of these zones have,in their health and their lives.
- The regulation of the existing legislation so that they will not permit any more constructions in this environment.
- The monitoring of the degree of the contamination and the determination of the scale of danger for the health and the lives of the people that live there.
- The taking of some measures for the rehabilitation of the situation and return of these zones in their normal state.

The case of Ohrid lake

Ohrid lake is 2-3 milion years old and is one of the fooldest lakes in the world. It is situated 695m over sea level, is surrounded by mountains 2000m high, has a surface of 537km^2 and is 289m deep. The basin that collected water in the Ohrid lake is 1487 km^2. The lake Ohrid is the biggest biological reserve in Europe with a very rich and uncial flora and fauna. For this reason at 1980 it was classified by UNESCO as a territory with natural and cultural inheritance. The zone around the lake has a population of 100.000 habitants who are divided in three cities Ohrid, Struga, Pogradec and many villages. Near Ohrid lake is developed a huge mineral activity which has a major impact in the surrounding environment disturbing the ecological balance of the lake basin and the surrounding areas. The pollution of the lake is result of : mineral-rich residues that come from the production process which are estimated 30/100% of the mineral production (Cr,Fe-Ni, Cu and C). Another important source of pollution is the technological wastes of the mine activity coming to lake thought the hydrological system of surface waters. The chemical analysis and compounds of the stockpile of some of the mines resulted as following chemical elements.

Table 1

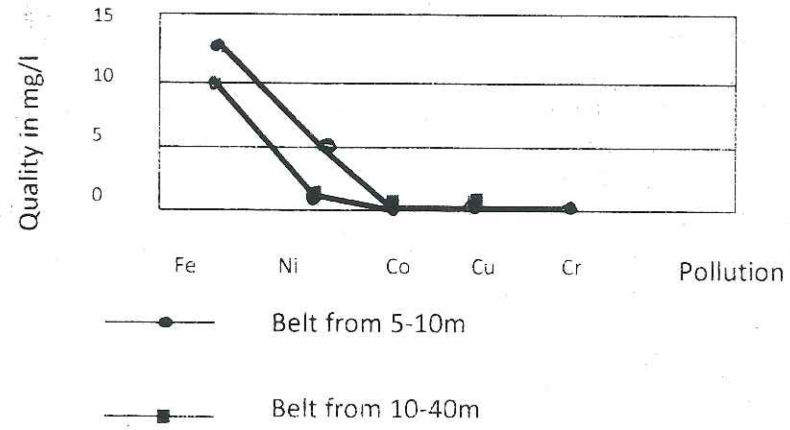
Chemical elements	Fine inert	Rude inert
Fe	14,92%	30,03%
Ni	0,27%	0,95%
Co	0,01%	0,08%
Cr	0,72%	0,24%
SiO_2	18,10%	11,74%

<u>Untreated wastewater</u> from the inhabited areas and the dirty water from the plants. A significant amount of the solid mineral is transported from the streams coming from the slopes of the mountains with mineral activity.

Transportation and exploitation tracks and stockpile of residues are probably significant values for the volume and surface that they have.

The filtration of the contaminated waters as a results of the geological and tectonic construction.

Shed sewage. From the measurements and the sample analysis in 1998 we have compiled the following graph.



The Ohrid lake has special and particular characteristics, it is one of the most preferable touristic place for foreign and Albanian tourists. For this reason the problem of protection of the lake from pollution, the protection of the natural recourses and ecological balance are very important problems.

Geoenvironment of Ohrid lake

From the geological point of view we have the following geological formations:

Triassic and Jurassic deposits consist in carbonate formation, ultrabasic and ofiolitie

Also we have discovered deposits of Oligocene, Tortonian and Pliocene which are presented by conglomerates, sands, alevrolites and clay. Quaternaries deposits with thickness 60-160m and with marsh lake origin are present.

From the hydrological point of view:

We have water bearer complexes the first of Cretan carbonates, the second Triassic carbonates and the third is the ultra basic rocks.

From the geological study results that, in this zones are present three categories of soils, hard medium and weak soils. Also here are present the geodynamic phenomena as : new tectonics, phenomena tied with atmospheric changes, slope instabilities, abrasion phenomena, important settlements etc.

From the seismological study results that, this zone has high seismic activity. From the statistical elaboration of the data in this zone is probable to happen an earthquake with M>5.9 every 50 years.

From the study of the water balance –sheet results that, the total volume of flowing created in the basin of Ohrid lake is 570.10^6 m^3 with module 14L/s.km^2.

From the thermic study, results that the difference of temperature in upper water layer (h=130m)is from 22^0 to 5.7^0C, while in the bottom water layer (h=130-264m) there isn't any more temperature difference and it stays at about 5.5^0C.

The chemical composition of the water of the lake shows that it has e few salts (200-250mg/L). In this salts predominate $HCO_3>Cl>SO_4$, $Ca^{++}>Mg^{++}>Na^+>K^+$

The causes to lose balance of the geoenvirnment

During the 20 last years we have observed the disorder of the ecological and geonvironment balacance in some directions. This is very dangerous for the ecological value of Ohrid lake because it damages the geoenvironment and creating problems for the development of this zone also for health of the habitants. The mines of chrome in Pojska and Memelisht, the mines of Iron-Nickel in Gradisht, Cervenak, Guri I Kuq, and the mines of coal in Alarup, Verdove, and Pretush ect are in a distance 2-2.5 km from Ohrid lake. The ecological miss balances are damages by man activities which are made in three directions.

- Shedding of sewerage directly in the lake, causes the augmentation of the percentage of the organic material
- Flowing of the contaminated industrial waters and water from mines including the toxic substances, heavy ,metals from stockpiles of Chrome minerals ($114000m^3$), Iron-Nickel ($228000m^3$) and Coal ($206000m^3$)
- Shedding of the soil residue of streams from slopes with mineral activity. It results that the solid material which is transported in the lake is from $5000m^3$/year until $40000m^3$/year. In this solid material we can find heavy metals as Fe=10-12mg/l Ni=5,26mg/l; C_0=0,70mg/l; Cu=0.0215mg/l; Cr=0,01mg/l

Shedding of water from plants, from the process of erosion etc.

Measures proposed for the remediation of the situation (conclusions).

The preservation of the ecological balance of Ohrid lake and their around zones needs some engineering measures, the creation of the monitoring system and the management of the data's or information.

The engineering measures are tied with:

Construction of purification plants for the sewerage. Isolation of the flows from the tankers that have dangerous material in them.

Engineering protecting measures for erosion phenomena.

Regulation of the surface water by different drains, canals with different levels ect.

The monitoring system. of Ohrid lake intends to preserved the biodiversity and their ecological functions. We think that for the monitoring system must be created one institute which will have the following responsibilities:

- They should assign a frequency of the monitoring from one month to 10 years.
- To determinate the percentage of the phosphor, nitrogene, organic materials, pesticides, physical and biological parameters ect.

- They doing to determinate the mechanism for the data exchanging.
- They should organize the receiving of the samples. They must analyzed the samples. and determined the local pollution and contamination.
- They must determined the used methods.
- The management of the data's or information from the monitoring system must be done from the local or central authorities for some purposes.
- To inform the public for situation.
- To take the necessary measures for the neutralization of the dangers or the miss balances of the geoenvironment for their degradations and destruction.

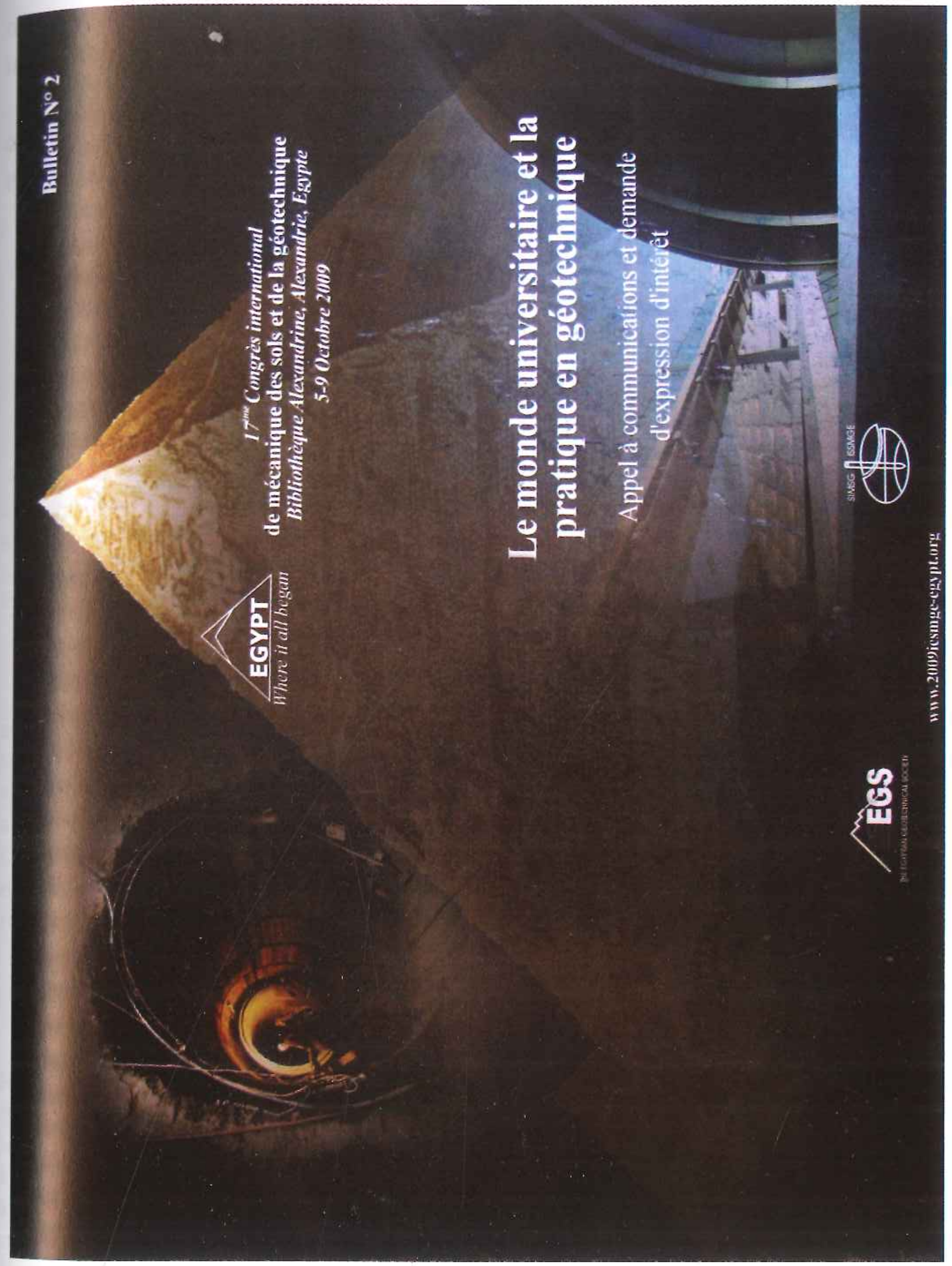
REFERENCES

Bozo L. Gogo K (2004) The problems related to the construction and exploitation of the tailing dams in Albania. Proccedings in XIII International .Int.Conference in Soil Mechanics and Geotechnical engineering Prague.

Bozo L Shkodrani N, Muceku Y, (2005) Landslide risk assessment on roads in Albania Proc.Int. Conference on Landslide Risk Management Vancouver Canada

Fetahu K (2000) Mineral activity in Pogradec region. Int.Seminar Ohrid Macedonia.

Ikonomi Gj. Xhelepi S, (2004) Monitoring of Ohrid lake. Monograph Tirana, Albania.



17-th International conference on Soil Mechanics and Geotechnical Engineering
Alexandria Egypt 5-9 October 2009

Laboratory testing of the cohesive soils and the correlation between the resisting characteristics of soils and their physical parameters.

Des essays en laboratoire des sols cohesive et de la correlation antre les caractéristiques de résistance des sols et de leurs parameters physiques

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Abstract

In Albania are present many serious phenomena's as movement of terrains. landslides slopes instability etc All this problems are tied with the shear strength of soils In this paper we would like to present a part of our study that determination of the resisting characteristic of cohesive soils by different methods in laboratory. Also we would like to establish relations between resisting characteristics of soils determined by different methods and their correlation with physical and mechanical parameters of soil.

Résumé

En Albanie sont présents un grand nombre de graves phénomènes de mouvement de terrain. glissements de terrain, l'instabilité des pentes etc. Tous ces problèmes sont liés à la résistance au cisaillement des sols. Dans ce papier nous voudrions vous présenter une partie de notre étude qui détermine les caractéristiques de résistance à la cohésive sols avec des méthodes différent en laboratoire. Aussi nous tenons à établir des relations entre les caractéristiques de la résistance au cisaillement des sols détermine par différentes méthodes et leur corrélation avec les paramètres physiques et mécaniques des sols.

Keyword: Shear strength. Cohesion. Physical parameters. Correlation.

Introduction

Every year in Albania are present sliding masses of different quantities and slope instability which are provoked from natural conditions. or by human activity. In the other part impetuous construction of highway, particularly in mountain zones. has caused in many cases destruction of natural equilibrium. Except this it happened because there isn't enough knowledge about shear strength of soils and for the factors which influence in their value. For this reason we have realized this study where would like to present the different factors that have influenced in the shear

strength value of cohesive soils and the different correlation between resisting and physical parameters of soils.

The used methods

.In ALTEA laboratory and in the geotechnical laboratory of Civil Engineering Faculty we have made over 50 tests to determine the shear strength of cohesive soils. The undisturbed samples are tested by ASTM normative in : direct shear apparatus, triaxial apparatus, unconfined compression apparatus. All tests are realized with samples of first quality by EC-7. From this study we have drawed some conclusions about the dependence of the shear strength with the physical parameters of soils, their bearing capacity, modulus of deformation etc. Finally we found some correlation between cohesion of soils determined by different methods.

Result of tests

The results of the tests are expressed in graphical mode, in different relations. The relations that we found are between the friction angle ϕ and the physical parameters as(Fig.1,2,3,4),e- void ratio, LL-liquid limit, PI –plasticity index , γ_d – dry density.

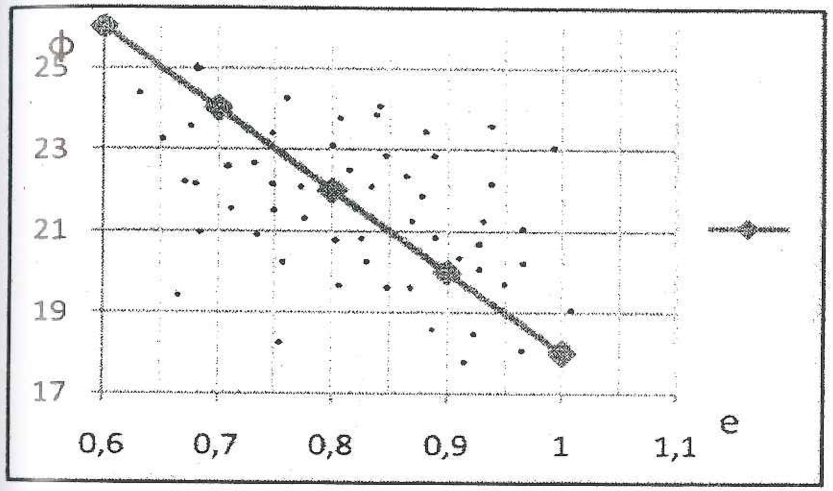

Fig.1 Relation between ϕ-e

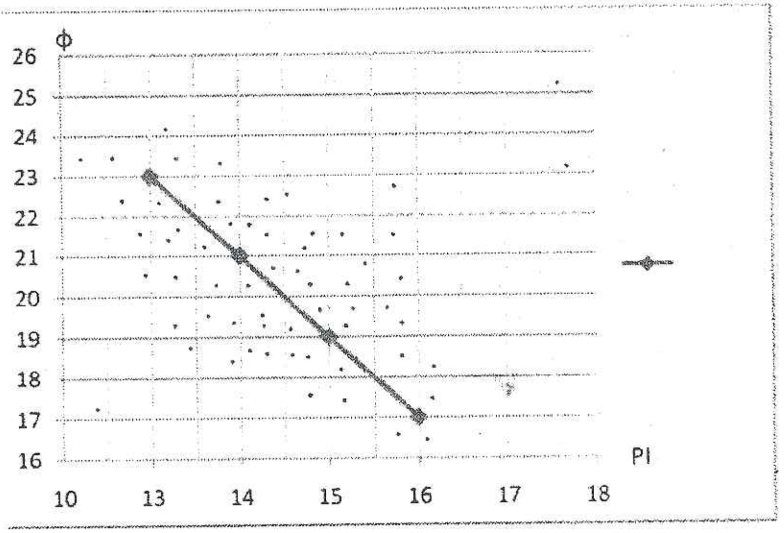

Fig.2 Relation between ϕ -PI

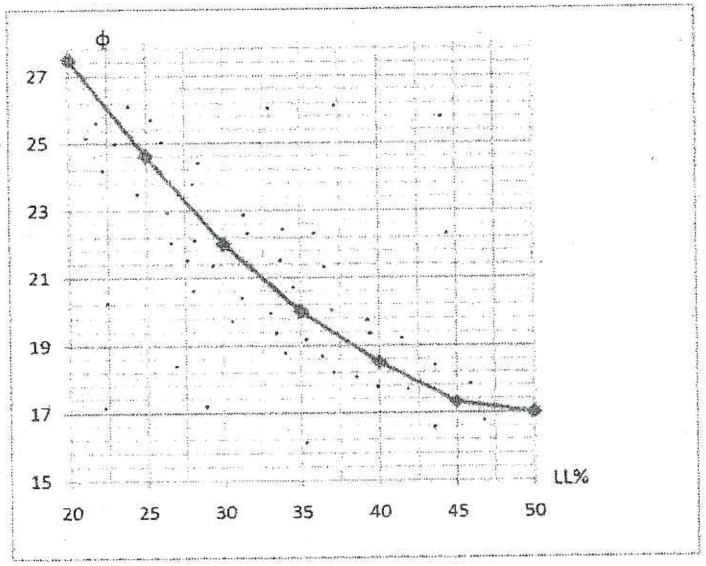

Fig.3 Relation between ϕ-LL (is valuable for PI=1 to 17

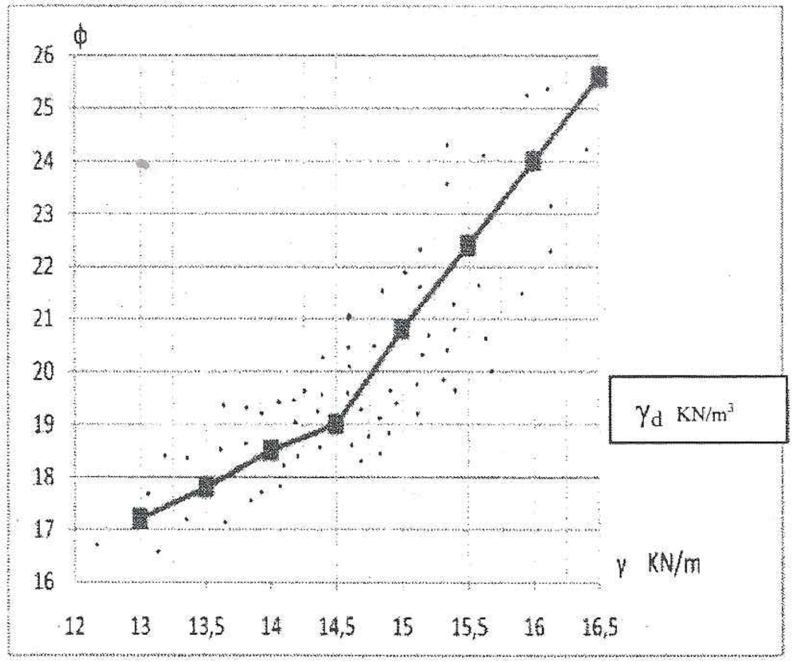

Fig.4 Relation between ϕ- γ_d

We have found the relation between friction angles ϕ and the cohesion C- with modulus of deformations –E.(fig.5.6)

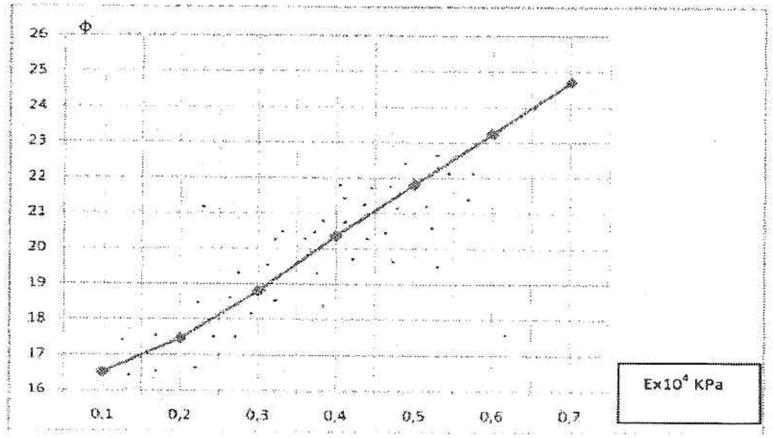

Fig.5 Relation between ϕ-E

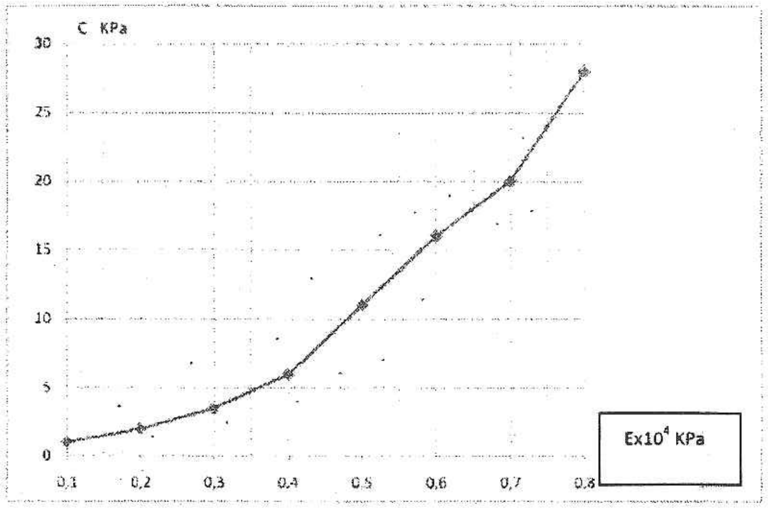

Fig.6 Relation between C-E

Also we have found some relations between C_u determined in unconfined compression apparatus, and physical parameters: as liquid limit –LL, consistency index I_k and bearing capacity of soil R.(fig.7,8.9)

$$I_k = \frac{W - PL}{LL - PL}$$

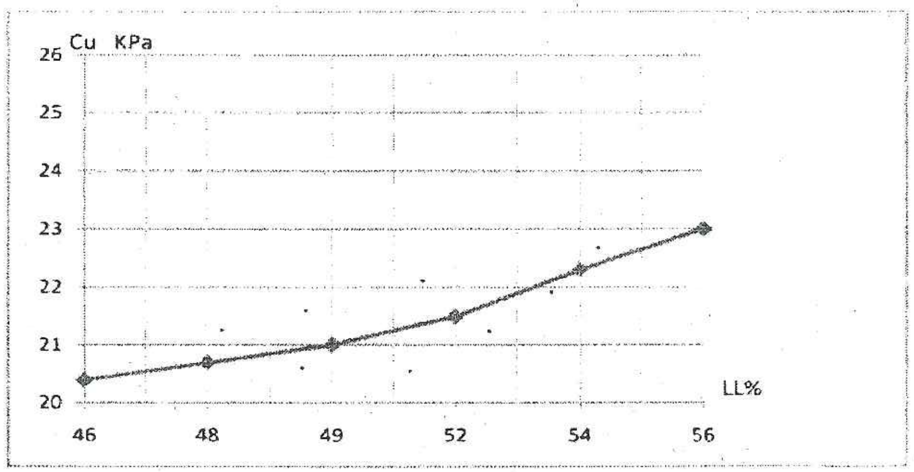

Fig.7 Relation between Cu-LL

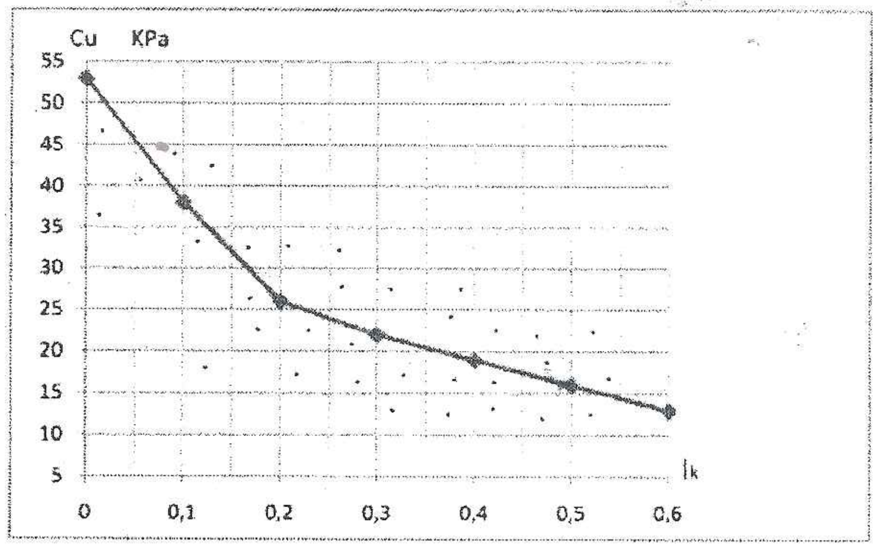

Fig.8 Relation bftween Cu-I_k

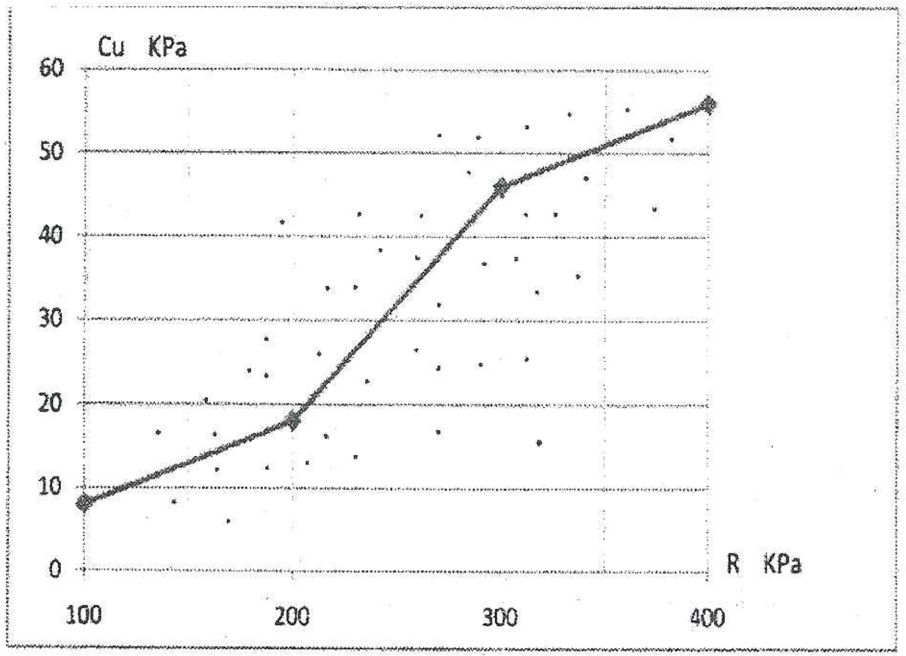

Fig.9 Relation between Cu-R

Finally we had the opportunity to establish some correlation between the value of the cohesion determined by three methods.
C- in direct shear apparatus.
Cuu- in triaxial apparatus by test UU
Cu- in unconfined compression apparatus:

$$C = C_{uu}\left(0,41 \div 0,9\right)$$
$$C = C_{u}\left(0,46 \div 0,94\right)$$
$$\frac{C_{uu}}{C_{u}}\left(0,9 \div 1,2\right)$$

After the statistical elaboration of the results we reached this correlations:

$$C = 0,608C_{uu}$$
$$C = 0,624C_{u}$$
$$C_{uu} = 1,12C_{u}$$

Conclusions

The shear strength of the cohesive soils with PI=7-17 have good correlation with physical parameters, We have determined the following correlations:

$$\phi \approx 26^0 - (e-0,6).20$$
$$\phi \approx 26^0 - (PI-12).2$$
$$\phi \approx 16^0 + (\gamma_d - 12).1,23 \quad until \quad \gamma_d = 14,5KN/m^3$$
$$\phi \approx 16^0 + (\gamma_d - 13,5).3,2 \quad for \quad \gamma_d = (14,5-16,5)KN/m^3$$

$\phi \approx 16^0 + (E-0,1.10^4).14,5$ E is expressed in KPa

$C_u \approx 20,4 + (LL-46).0,18$ for the soils with PI > 17

$C_u \approx 50 - I_k.140$ for the soils with I_k = 0-0,2

$C_u \approx 25 - (I_k.0,2).30$ for soils with I_k =0,2-0,6

$C_u \approx (0,084-0,14)R$

In case of the preliminary geotechnical studies can be used this correlations to create approximately the geotechnical model and to do the preliminary calculation.

REFERENCE

Atkinson J (1997) An introduction to the mechanics of soils and foundations. London Mc.Graw-Hill
In text (Atkinson 1997)
Bozo L 2007 Geotechnic I, Soil Mechanics. Tirana Printing Natyra.

In text: Bozo 2007.
Sobolevsky D.1995 Strength of dilating soil and load holding capacity of deep foundation, Rotterdam, A.A. Balkema
In text (Sobolevsky 1995).

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Training of geotechnical engineers in Albania
Formation des ingénieurs en géotechnique en Albanie

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Abstract

The last 15 years, in Albania were evidenced many dangerous phenomena's on construction practice, which are tied with geotechnical problems. The absence of the specialized institutes and the total privatization of construction sector, have done the necessary to train and specialize the civil engineers in geotechnical field. In this paper we want to present our experience for the training of the civil engineers, their profits and advantages. Also we would like to present the work that is done in the Civil Engineering Faculty for preparation of geotechnical engineers, capable to doing design of geotechnical structures, to resolve difficult geotechnical problems and to realize all kind of geotechnical works.

Resumes

Les derniers 10-15 ans, en Albanie en témoignent de nombreux phénomènes dangereux sur les pratiques de constroction, qui sont lies à des géotechniques. L'absence d'instituts spécialisez et de la privatisation totale du secteur de la construction, ont fait le nécessaire pour former et de spécialiser les ingénieurs civils dans le domaine géotechnique, Dans ce papier, nous voulons présenter notre expérience de la formation des ingénieurs civils, de leurs bénéfices et avantages. Aussi nous tenons à présenter le travail qui est fait dans la Faculté de Génie Civil pour la préparation de la ingénieurs géotechnique, capable de faire la conception des ouvrages géotechniques, afin de résoudre des problèmes géotechniques et de réaliser toutes sortes de travaux géotechniques.

Introduction

During the transitory period in Albania, was made the total privatization of construction sector. Very big development was seen in some sector as: constroction of multi stories buildings with 1-5 underground floors, constructions of roads infrastructures as rural roads and highways, construction of bridges, subways and tunnels, construction of airports, reconstruction and enlargement of ports, construction of watering system and maintenance of over 600 dams constructed before, for hydropower plants and irrigation.

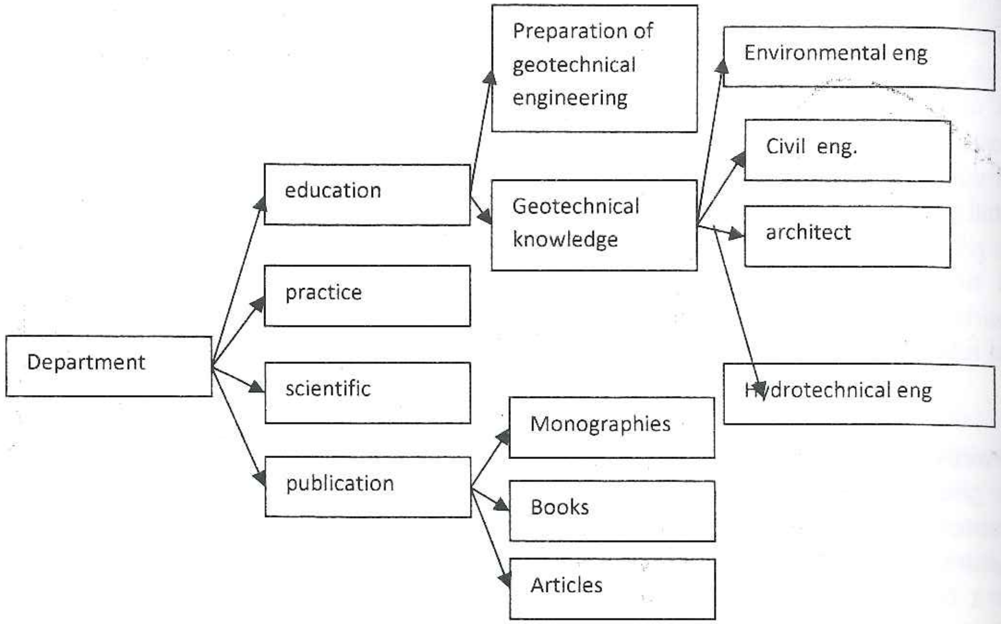
Albania is mountain country (75% of territory are hills and mountains), 50% of field zone is marsh with high seismicity. Albania has many water sources and torrential rivers, The big development of constructions is accompanied with dangerous phenomena's as: all kind of slope instability; very big deformation of soils; damage of existing buildings because the deep excavation near them, vigorous erosion activity of rivers; the destruction of the environmental equilibrium from the constructions without criterion in the hill zones etc.

This problems have evidenced the necessity to acquire knowledge in the geotechnical field and deepening in the geotechnical studies. Without doubt the Geotechnical Department, AGS and the cooperation between academics and practitioners has the main rule in the professional education

The activity of Geotechnical Department

The geotechnical Department of the Polytechnic University was developed good education, scientific and practical activity. It has a close collaboration with AGS. During 40 years of its life it begun with two disciplines" Soil Mechanics" and "Foundation", and now it has eight more disciplines which are made in the second cycle of Bologna process. They are: "Rock Mechanics", "Experimental Geotechnics", "Road geotechnics" "Slope stability and design of dams, embankment, tailling dams", "Soil dynamics and foundation under vibration" , "The geotechnical codes", "Deep foundations", "The security problems in the geotechnical structures.

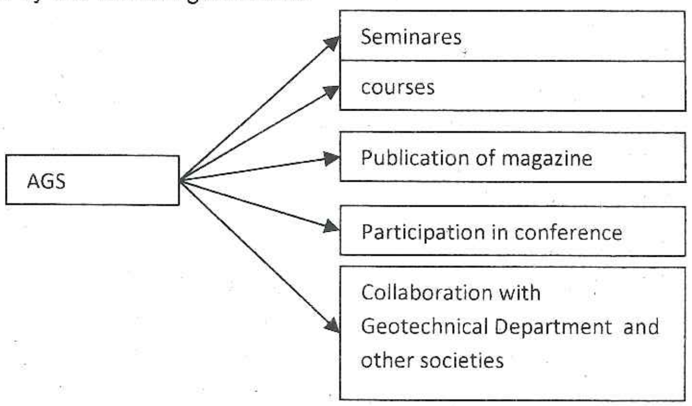
The geotechnical department has made over 30 publications and books which serve for students and civil engineers. Every year the department's member lead 20-22 students in diploma work with the design of geotechnical structures, scientific themes and to do them in practice. Already it is create a good stock with the works of young geotechnical engineers. In geotechnical department was working continuously for the qualification of their members. So, till now, was finished 6 PhD thesis, one thesis for Doctor in Philosophy, many master's thesis and the evaluation for other 10 PhD thesis etc. Now we have the third cycle of education or Doctorature Scholl which will prepare every year 2-3 PhD thesis. All scientific problems which are planed and are realized by geotechnical department are from engineering practice. The department has very close relation with practice, making the consulter the survey, the difficult geotechnical design, the common project etc. In the same time the department keeps connections with their colleges from Europe, America, Asia and Africa. All activity of Geotechnical Department can be expressed by following schema:



The biggest success of Geotechnical Department is the opening for the first time in Albania of the geotechnical specialty in Civil Engineering Faculty. The first specialist in this field graduate in 2010.

The activity of the Albanian Geotechnical Society AGS.

AGS is a new society. It was created in 2000 and during 9 years it has good activity in the field of geotechnical education of the civil engineers and geologist. Their activity is presented by the following schema.

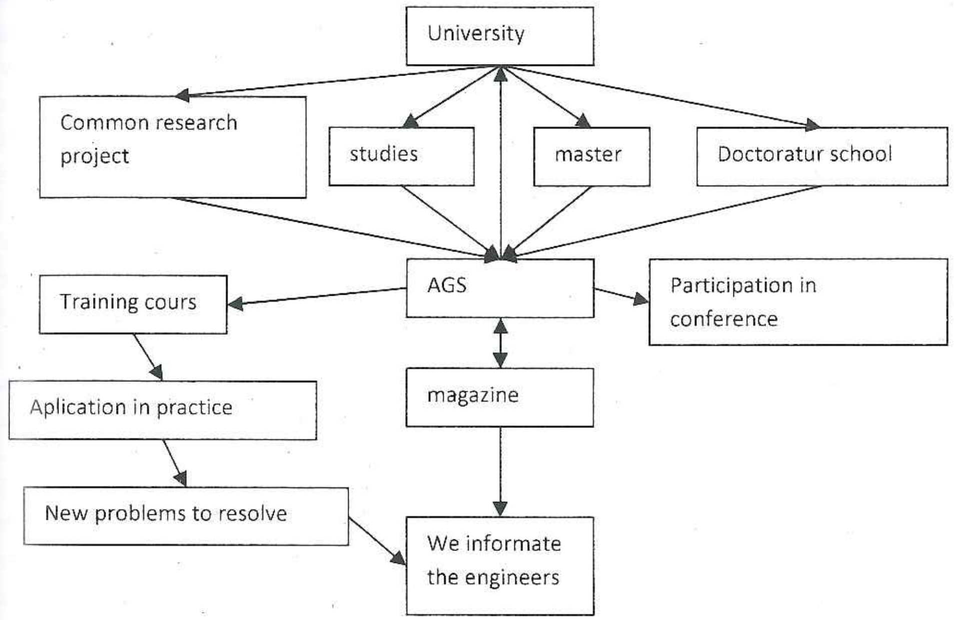


So AGS has made three international activities with ENCP, workshop with professors from USA and Greece, Touring lectures in collaboration with ISSMGE.

Also it made 5 training courses and 2 seminaries with civil engineers. AGS has published annual magazines for 9 years. Each magazine has: research rubric, result of practical problems, research of young geotechnical engineers and different information etc. Since 2001 every years members from AGS have participated with their papers in international conferences, symposiums and workshop. This participation has made possible to exchange the experiences. The new theories, technologies, are published by AGS magazines. AGS organizes common activity with Civil Engineering faculty and with different private companies.

Collaboration between academics and practitioners.

Every good result in geotechnical studies by: experimental studies; master's thesis: PhD thesis, common research project between universities, are made effective by cooperation academics and practitioners. This collaboration functions by the following schema.



So the cooperation between academics and practitioners is realized by AGS, which takes from the academics all the new information and transmits this to the practicioners and takes from practitioners dhe problems that need solution and transmits them to the academis. In this manner are developed all studies in the geotechnical field.

The profile of the geotechnical engineer.

Alredy in Albania is prepared the geotechnical engineer (by Bologna system). In the second cycle of this system the student in the geotechnical profile is prepared and specialized in theoretical and practical field. They learn all problems that are tied with EC-7 and EC-8. Also the students learn all kind of foundations, geotechnical structures, dams, embankment, slope instability and engineering measures to stabilize them, different improvement methods etc. The start of the third cycle or Doctor's school will have a huge importance in the development of scientific and research work in geotechnical field.

Conclusions.

The AGS has a big role in:

- Collaboration between academics and practitioners.
- Training of civil engineers
- Promotes the studies of the young engineers and other studies.
- Evidence the most important geotechnical problems in Albania to sensibility the public opinion.
- Propagation of a new technology and methods in geotechnical works.

17-th International conference on Soil Mechanics and Geotechnical Engineering Alexandria Egypt 5-9 October 2009

The mass movement response of tectonics phenomena in urban areas, Albania.Le glissement de terrain comme réponse aux phénomènes tectoniques dans les zones urbaines, Albanie

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Albania is a neo-tectonic region with high seismic intensity. Two of thirds of Albania area is built by hills and mountains, which are much affected from tectonics and neotectonics events. These events have played a main role in the slopes stability, from which are formed many landslides, where are damaged and destructed a lot of engineering objects as population buildings; heritage and historic center, roads and geoenvironment of urban areas. In this paper we present the mass movements-earth slides, earth flow, rockslides, which are caused from tectonics phenomena in the main town of Albania.The present paper is based to the engineering geological investigations carried out in the Albanian area during the year 2000-2007. It takes under consideration some more serious stability slopes, where besides engineering objects (buildings, heritage and historic center, roads etc.) are threaten population life from the development of this phenomenon. That's why are carried out a lot of studies in urban areas, as Lezha, Kruja, Tirana, Durresi, Gjirokastra, Vlora etc. towns, where are occurred landslides on hills and mountains slopes, which are much favored by tectonics and neotectonics, processes.

Résumé

L'Albanie est une région néotectonique à séismicité intense. Deux tiers de l'Albanie sont constitués par des collines et des montagnes qui ont été beaucoup affectés par des événements tectoniques et néotectoniques. Ces événements ont fortement influencé la stabilité des pentes, ce qui ont produit des glissements de terrain et qui ont détruit et endommagé des édifices du génie civile ; des habitations, le centre historique, des routes et le géoenvironnement de la zone urbaine Ce papier est basé sur les investigations géologiques effectuées en Albanie durant les années 2000-2007, et il prend en considération la stabilité des pantes qui posent des sérieux problèmes au niveau des édifices du génie civile (bâtiments, centre historique, routes, etc.,), ainsi que pour la vie de la population. C'est pour cette raison que des nombreux études ont été effectuées dans les zones urbaines, telles que les villes de; Lezha, Kruja, Tirana, Durresi, Gjirokastra, Vlora, etc., où des nombreux glissements de terrain sur les pantes des collines et des montagnes sont plutôt favorises par des evenements tectoniques et neotectoniques.

Keywords: landslide, tectonic, lithology, slope morphology, manmade activities, rainfall, urban area

Introduction

Mass movement is a serious geologic hazard in Albania. In whole of Albanian territory the mass movement-landslides cause damages and demolish many engineering objects, as well as, lost many hectares land. This work is a summary of the engineering geological mapping and geotechnical investigations carried out in the main towns of Albania during the year 2000-2008. In this paper we present the engineering geological mapping and geotechnical results taken in the urban area of Lezha and Kruja towns. The Lezha town is located in northwestern part of Albania (Fig.1). It's a historical, cultural center and touristic place in Albania, because of is near (10 km) of beautiful Shengjini beach. The Kruja town is located in central part of Albania on Skanderbeg Mountain's slope. It represents one of more famous heritage and historic centers in Albania, as well. For as much as, on some parts of these towns have occurred landslides from which many engineering objects as buildings and roads are damages and demolished and are threatened others we undertaken a detailed engineering geological mapping (scale 1:5000) and geotechnical investigations, where are done many drilling and laboratory test analysis of soils and rocks.

Discussion and analyses

2.1 *Geological and tectonics-neotectonics setting*

According to geological and tectonics-neotectonics phenomena the eastern part of Lezha and Kruja town take part in Kruja tectonic zone, which include in external area of Alpine folding (Fig.1). It is strongly affected by pre-Pliocene compression phases. The Kruja tectonic zone has been deformed by folds, reverse faults-thrusts and occasional back thrust, as well as by strike slips from the main Alpine compressive phases which folded the above mentioned tectonic zone. Generally, the structures Kruja tectonic zone extend from NW to SE (Fig.1). It must be emphasized the hilly urban area of Lezha town is situated on intensive tectonic zone (thrust faults), which is formed by movement of Krasta tectonic zone over Kruja tectonic zone (Fig.1, 2,). The eastern part of urban area of Kruja town is built on tectonic zone caused from uplift of the normal thrust of limestone's formations over flysch formations (Fig.1, 3, 5).

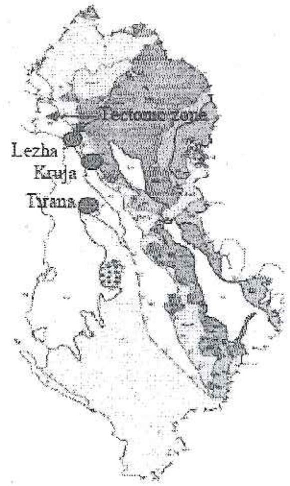

Figure 1. Schematic geological map of Albania

Geologically the urban area of Lezha town are represented from Lower Oligocene formations-flysch rocks are composed of are thin rhythmic clay-siltstone-sandstone-flysch that are part of Kruja tectonic zone and Maastrichtian-Eocene formations-flysch are sequences composed mainly of siltstones intercalated with sandstones and marl layers, which take part on Krasta Cukali tectonic zone. The second one uplift with intensive tectonic zone on Lower Oligocene formations (Fig. 1, 2).

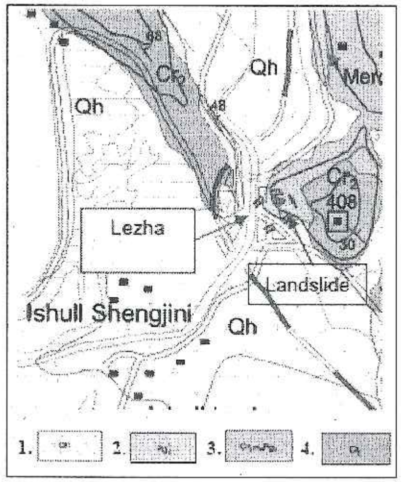

Figure 2. Geological map of Lezha region

1. Quaternary deposits,-silts and sands, 2. Flysch rocks- siltstones and claystones, 3. Flysch rocks-siltstones intercalated with sandstones and marl layers, 4. Limestone,

From the lithological point of view, the studied area of Kruja town is built from the Lower Oligocene formations-flysch rocks Mostly of the bedrocks covered by the Quaternary deposits are silts with gravels and sands mixtures (Fig. 1, 3), which form an overturned syncline with dip angle of the limbs to east. Over this structure with intensive tectonic with normal thrust uplift the Upper Cretaceous limestone's rocks, which extent on eastern part of Kruja town. The flysch rocks represent from combination of the siltstones and claystone's layers. Generally flysch rocks are covered by the Quaternary deposits are silts and silty clays with mixtures of gravels and sands.

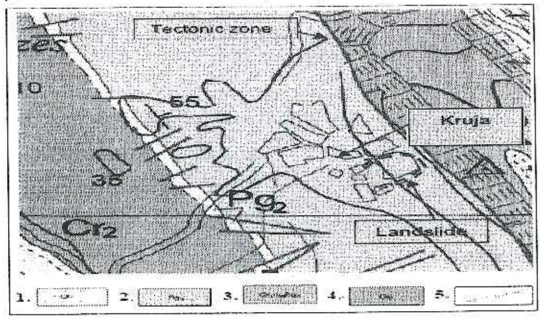

Figure 1. Geological map of Kruja region

1. Quaternary deposits,-silts and sands, 2. Flysch rocks- siltstones and claystones, 3. Flysch rocks-siltstones intercalated with sandstones and marl layers, 4. Limestone, 5. Tectonic,

2.2 *Geomorphology of the site*

Based on field investigations, the studied area of Lezha and Kruja towns represent a hilly zone built by flysch rocks with slope inclination range from 30° up to 45°. The slopes are characterized by the features such the concavo-convex profile, which are formed as result of the geodynamics phenomenon occurrences-earths lips and erosions. During rains, a lot of hill slope mass in the form of debris and rock fragments flows down through these drains and gets accumulated in the middle and lower horizons hilly zones. Where-ever the slope angle exceeds the angle of repose of the slope material, the hill mass shows signs of distress and fails. The failed slopes are generally devoid of any vegetation whereas the surrounding area is moderately vegetated with pine trees. It has been found that a number of cracks are present in the upper horizons, particularly in the south and eastern parts. These cracks mostly trend north – west and dip towards the hill slope.

2.3 *Mass movements*

Slope instability activity is related to various influencing factors, which caused landslide in the studied area. The main factor, which has initiated the favored this phenomenon. Also, for that have influence and lithology (soils and rocks),morphology (slope inclination and slope shapes), rainfalls and manmade works. In this paper we treated two case of story selected from geodynamic phenomena occurred in Albania.

2.3.1 *Lezha landslide*

The landslide occurred on hills slopes extend en eastern of urban area of Lezha town (Fig.2, 4). This part is one the most active tectonically area in Albania constituted by thrust fault, where limestones rocks uplift on flysch rocks. During the

uplifting of the Krasta tectonic zone over Kruja tectonic zone (Fig.1, 2, 4) the flysch rocks of urban area have changed their geotechnical properties, transforming from rocks to soils. So, decomposition of the flysch rocks from this phenomenon and weathered processes produces conducive conditions to mass movement in this area. Beside of them, the landslide was triggered by the rainfalls-storms, with the most notable being the disturbed zone by tectonic phenomenon. From this phenomenon are damaged demolished 8 homes (1-2 stores) and main town road etc. The homes damages are cracks of the walls 4.0-10.0cm, cracks and subsidence of the floor and demolition of the home. There also was widespread debris flow and damage to homes, buildings, and roads in areas along the main boulevard and hospital road of Lezha town. The landslide according to classification of Varner 1978 include in rotational types that are generally shallow-deep. The geologic structure, lithology and morphology of the urban area are profoundly influence by the active processes of regional tectonic. Thus, as above mentioned it, the natural events such as tectonics phenomenon, rainfall, slope morphology, manmade construction activities and underground water have caused of the mass movement. The landslide is vary greatly in their volume of soils, the length, width, depth of the area affected, frequency of occurrence, and movement. Therefore, inherent factors related to rock discontinuities provide a basis for understanding landslides and also in the formulation of long and short-term pro-active responses to natural hazards from slope instability. The size of a landslide is 450-500m wide, 220-250m up to 300m long and 7.0-10.2m deep, whereas the its volume is 1 009 375 m^3. Slides move in contact with the underlying surface, which are the much disturbed or decomposed of flysch rocks.

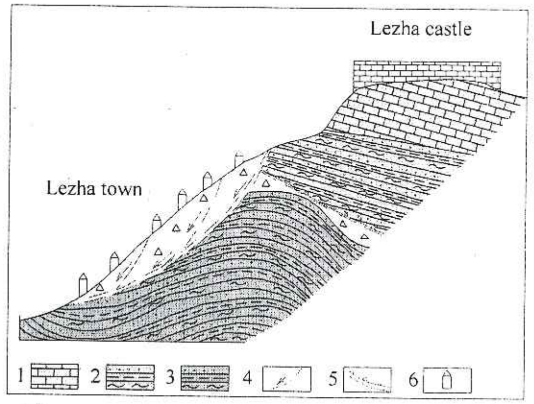

Figure 4. Cross section Lezha landslide

1. Limestone, 2. Flysch rocks- siltstones and claystones, 3. Flysch rocks-siltstones intercalated with sandstones and marl layers, 4. Landslide, 5. Tectonic, 6. Building

2.3.2 *Kruja landslide*

It's located in south-east part of the Kruja town. On this earth slide is constructed a square of the Kruja town, where 12 buildings are demolished by this phenomenon. The slide body is lying over siltstones and claystones of flysch formations (Fig.5).

The dimension of the earth slide is 500.0-550.0m long, 400.0-500.0m wide, 8.0-12.5m deep and volume of 2 362 500 000 m^3 (Fig. 3, 5). The slide mass consist of clays, silts, sands, gravels and limestones blocks. The slide plane occurred in these flysch formations, and it has a typically a polished surface at the base, where on it is a thin soft layer (0.2m up to1.5 -2.0 m). The Kruja town is situated on hills slope, which have e dip angel ranging from 30° up to 45°. It's moving slowly downwards on hills slopes. Taking into account the landslides according Varner 1978 can be classified as rotational and earth flow types that are active, deep and shallow. Some result in private property damage, while other landslide affects transportation corridors, fuel and energy conduits, and communication facilities. They also pose a serious threat to human life. The landslide generally is slow moving and less rapidly moving (debris flows). First at all it cause significant property damages, but are less likely to result in serious human injuries, where rapidly mass movement present the greatest risk to human life, and people living in mass movement prone areas are at increased risk of serious injury. The upper part of the landslide body is situated directly on tectonic zone, which the main factor that has favored this phenomenon. That's because of flysch rocks formations are intensively disturbed becoming more susceptible to landslides than others, the tectonics zone serve as a collector for the karstic water, which drained from limestones rocks of Scanderbeg mountain and infiltrate in the contact of the soils and flysch rocks doing lowering of geotechnical properties of the soils and rocks. It is certified by the several waters spring, which are found along of landslide body. It often triggered by periods of heavy rainfall (1800-20000 mm/year) and excavations, as well. Also, the morphology of the steep slope of this area can also increase susceptibility to landslide event. Beside of them in this area the people can be exacerbated by human activities as the grading for road construction and development where is increased the slope steepness, as well as, the grading and construction of several buildings, which decrease the stability of a hill slope by adding weight to the top of the slope, removing support at the base of the slope, and increasing water content. Other human activities effecting landslides include excavation, drainage and groundwater alterations, and changes in vegetation. From landslide occurrence are damaged 14 homes-2 stores, town roads etc.

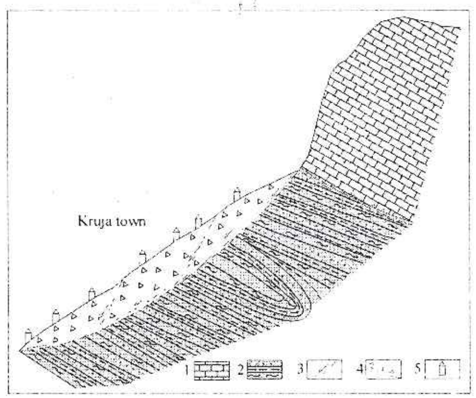

Figure 5. Cross section Kruja landslide

1. Limestone, 2. Flysch rocks- siltstones and claystones, 3. Flysch rocks-siltstones intercalated with sandstones and marl layers, 4. Landslide, 5. Tectonic, 6. Building

2.4 *Geotechnical investigations*

The studied area are represented by the two of the main town in Albania are Lezha and Kruja towns, which are historical, cultural, heritage centers and touristic place of Albania. So, they are very important places for Albanian culture. Therefore it was very necessary to carried out the engineering geological and geotechnical investigation related to slopes stability of at this site. Also, evaluations of the physical-mechanical properties of the slope materials in conditions are completed. The field investigations include the engineering geological mapping on scale 1: 5000 carried out in whole area of urban area of both towns and drillings works were done in the landslides body, from which determine the slide plane. In the Lezha town in related to slope stability are carried out 12 drillings 8.0m to 12.0m deep and taken 22 undisturbed and 6 disturbed samples. Whereas, in the Kruja town has been done 8 drillings and taken 18 undisturbed and 7 disturbed samples. The rock and soil samples (both disturbed and undisturbed) have been collected out and in landslide body for laboratory testing. The soils geotechnical parameters of landslide materials as bulk density, specific density, moisture content, grain size distribution, Atterberg's limits, shear strength and uniaxial compressive strength of rocks have been determined from these samples in the laboratory. The results are briefly discussed here and given in tables1, 2, 3, 4, 5 and 6.

2.4.1 *Lezha landslide*

From field works and laboratories test analyses are determine as following geotechnical unit:

Geotechnical unit nr. 1.

Represents landslide body soils. These soils consist of the sands and clays mixtures with pebbles and rubbles content which are in medium geotechnical conditions and have a thickness range from 7.0-10.0 up to 14.0-16.0m (Fig. 4). According to unified soil classification system these soils are "SC" type.

Geotechnical unit nr. 2. These soils built the slide's plane. They are made of inorganic silts and very fine sands. This layer is situated below geotechnical unit nr. 1, and have a thickness varies from 1.4 up to 1.7 m (Fig. 4). According to unified soil classification system these soils are "ML" type.Geotechnical unit nr. 3. This geotechnical unit consist of sandy clays soils. They are situated under the slide's plane and over the flysch fresh rocks. This geotechnical unit represents the weathered flysch rocks. According to unified soil classification system these soils are "CL" type.

Table 1.. Physical properties of soils and rocks, Lezha landslide

Geotechcal unit	Fines(clay&silt) (%)	Sands (%)	Gravels (%)	W_L (%)
1	48.0	27.4	24.6	40.2
2	60.6	33.12	6.28	42.3
3	53.4	40.6	5.0	31.4

Table 2.Physical properties of soils and rocks, Lezha landslide

Geotechcal unit	W_p (%)	W_n (gr/cm^3)	γ (gr/cm^3)	γ_o (gr/cm^3)
1	23.3	32.1	1.85	2.66
2	25.9	38.4	1.72	2.69
3	20.8	20.4	1.99	2.68

Table 3Physical properties of soils and rocks, Lezha landslide

Geotechcal unit	c (kg/cm^2)	φ (o)
1	0.1	30
2	0.05	8
3	0.57	26

It must be emphases the laboratories results indicate that the soils nature of the geotechnical unit nr. 3, which are the weathered flysch rocks included in "CL" type-sandy clays soils with low-medium geotechnical properties.

2.4.2 *Kruja landslide*

In the studied area of Kruja fro field works and laboratories test are determine:

Geotechnical unit nr. 1. The soils of this geotechnical unit built the landslide body soils. They consist of inorganic silty clays with sands content and have a thickness range from 7.0-10.0 up to 14.0-16.0m (Fig. 5). According to unified soil classification system these soils are "CL" type.Geotechnical unit nr. 2. It's built the slide's plane. These soils are made of silty clays with sands content with sands, and situated below geotechnical unit nr. 1, having a thickness varies from 1.1 up to 1.9 m (Fig. 5). According to unified soil classification system these soils are "CL" type.

Geotechnical unit nr. 3. Represents the weathered flysch rocks. It's consist of inorganic sandy clays soils. They are situated under the slide's plane and over the flysch fresh rocks. According to unified soil classification system these soils are "ML" type.

Geotechnical unit nr. 4. It's represents by soft rocks – combination of claystones with siltstones layers, grey color and is found below the layer geotechnical unit nr. 4 (Fig. 5).

Table 1. Physical properties of soils and rocks, Kruja landslide

Geotechcal unit	Fines(clay&silt) (%)	Sands (%)	Gravels (%)	W_L (%)
1	60.2	39.0	0.8	41.4
2	65.6	34.4	-	40.3
3	58.9	41.1	-	38.6
4	-	-	-	-

Table 2.Physical properties of soils and rocks, Kruja landslide

Geotechcal unit	W_p (%)	W_n (gr/cm^3)	γ (gr/cm^3)	γ_o (gr/cm^3)
1	23.6	34.5	1.84	2.69
2	22.6	37.7	1.70	2.70
3	24.1	22.1	1.98	2.69
4	-	-	2.2-2.5	2.53

Table 3. Physical properties of soils and rocks, Kruja landslide

Geotechcal unit	c (kg/cm^2)	φ (o)	σ_{sec} (kg/cm^2)	σ_{an} (kg/cm^2)
1	0.1	16	-	
2	0.05	7	-	
3	0.60	24	-	
4	-	-	4.6x10^3	13.4-36.1

γ - bulk density, γ_o - specific density, W_n - moisture content, W_L - liquid limit, W_p - plastic limit, c – cohesion, φ – inner friction angle, σ_{sec} - secant module Young uniaxial, σ_{an} - compressive strength of rocks

It needs to mention the laboratories analyses shown that the soils condition of the geotechnical unit nr. 3 of the Kruja landslide are the weathered flysch rocks and related to unified soil classification system they are "CL" type- sandy clays soils with low-medium geotechnical properties. It means that these soils contribute on the new mass movement development. To protect from these phenomena in the studied area of Lezha town are used several engineering measures as retaining structures as concrete walls and piles, whereas in Kruja town are applied retaining concrete walls.

Conclusions

Weathering and decomposition of the flysch rocks from tectonics phenomenon produces conditions conducive to mass movement.

The cooperation of tectonics phenomena with several geo-factors such as lithology, slope morphology, rainfall, manmade construction activities and underground water have caused of the mass movement, which have demolished respectively 8 homes and 14 homes (1-2 stores) in Lezha and Kruja town, as well as injured of the main roads and gardens.

REFERENCES

Muceku Y., (2004-08): Engineering geological mapping for urban development of Lezha and Kruja towns, scale 1:5000. IGJOE Tirana Polytechnic University, Albania, pp 105.

Varnes DJ., (1978): Slope movements: type and process. In: Eckel EB (ed) Landslides analysis and control. Transport analysis and control. Transport Research Board, Spec Rep 176. National Academy of Sciences, Washington, DC, pp 11-33.

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Damages and risk assessment of Thana dam in Lushnja region, Albania.

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Abstract

This paper is a summary of the geotechnics investigations of Thana earth dam, which collapsed by several landslides occurred. It's located in central part of Albania at Lushnja region. The Thana reservoir is the biggest one constructed for irrigation purpose in Albania. The dam is 3.5 km long and 12.5 m high. As result of processes development as waters reservoir fluctuation, rainfalls and erosion during 1978 through 1990 years on its body occurred 5 earth slides. For this reason on 1999-2001 years a geotechnical investigation is carried out, results of it are presented in this paper. Several drillings are carried out and many soil samples are taken on studied site and examined in laboratory. Also, it's very important to note that Thana reservoir is operated with great risk is hard to give full scope to designed benefit, and seriously imperil the safety of life and property of the people residing in the downstream. So, with the development of the economy and the densely populated downstream area, the risk assessment with Thana reservoir dam shall be paid key attention by government, sciences institutes and public that become key problem. Finally, we'll give recommendations related to engineering measures need to take for remediation works.

Introduction

In Albania after 1955 up to 1988 years are 627 dams built for irrigation purpose, which are earth fill type. The dams conditions currently 389 dams are in good condition, 207 dams with technical operations and geotechnical problems, as well as 31 dams are in collapse conditions. So, in these dams occurred landslides and erosion phenomena, means they are threaten the human life and their business extend on flood flat area. In this paper we treated one of them. It is called the Thana dam. This dam is located on center part of Albania at Lushnja region. As result of damage of this dam by landslides occurrences during 1980-1995 year are carried out many engineering geology and geotechnical works as engineering geology mapping, drillings and are taken a lot of soils samples for analyses in the laboratory [6]. These results we presented here. It should be noted that these landslides are

active. They are moving day by day down and landslide hazard poses a severe threat to life, property and infrastructure, and becomes a major constraint on the development of the many villages are located lower water flow of the Thana reservoir. Strategies should be made to understand landslide process related to Thana dam, analyze threatening landslide hazard and taking engineering measures to remediate it.

Methodology

As result of occurrence of landslides phenomena on Thana dam a detailed engineering geology and geotechnical investigation are carried out in the Thana reservoir zone. Firstly, an engineering geology mapping on scale 1: 2000 is done and 20 drillings with 12.0m up to 13.0 are completed on dam crest [6]. From these works are taken respectively 45 and 10 undisturbed and disturbed samples to analyses in laboratory for determination of physical and mechanical parameters. So, the soils samples taken mainly in the failure site from of the dam body are analyzed in the laboratory to determine of physical and mechanical parameters as bulk density, grain size distribution, Atterberg's limits, moisture content, specific density, dry density, porosity, porosity coefficient, shear strength and oedometer module, which are given in Tables 1, 2, 3 and 4.

Geological setting

The studied area on which is constructed the Thana reservoir is built by the geological formations as (Figure 1):

Quaternary deposits

These deposits extend on whole area and are situated on rocks. They are represented by mixture of silts soils with sands-gravels. They are thick 2.5-4.0m up to 6.0-10.0m.

Molasses rocks

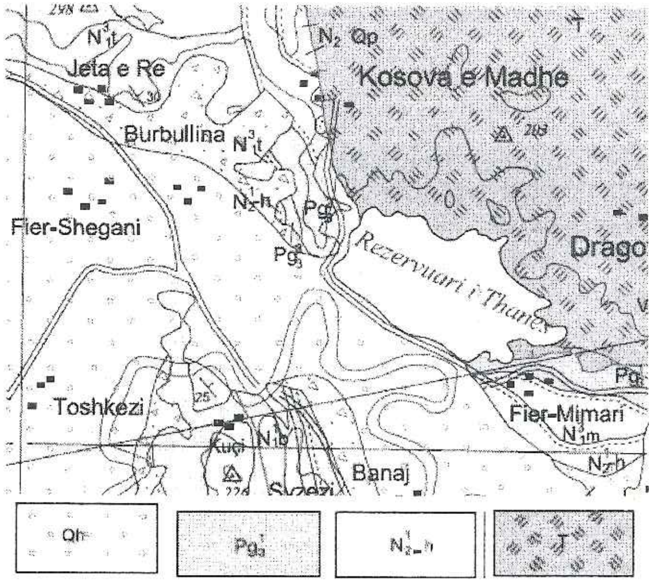
They extend on western part of reservoir and represented by Helmesi of geological unit and Messinian rocks. The Helmesi of geological unit composed by claystones siltstones formations intercalated sandstones layers. These formations lie on Messinian rocks and are covered by Quaternary deposits. Whereas, the Messinian rocks consist of thick sandstones layers intercalated with thin clays layers.

Flysch rocks

They are formations of Lower Oligocene and consist by clay stones and sandstones layers intercalated with limestone's layers. These deposit uplift with thrust tectonics on evaporates rocks, which are the Dumre diaper.

Evaporates rocks

These rocks are the most important occurrence of the evaporates rocks in Albania. They form the Dumre diaper, which is extended close to crossings of tectonics transverse zone of Lushnja – Elbasani. In general the above mentioned diaper is in contact with flysch rocks. Evaporites rocks extend in center and eastern part of the reservoir. They comprise gypsum, salt, anhydrite and more rarely tuffs, fragments of dolomitize etc.



1.Quaternary deposits, 2. Molasses rocks, 3. Flysch rocks, 4. Evaporites rocks

Figure 1: Geological map of Thana reservoir zone

Reservoir and dam characteristics

The Thana dam was projected on 1958 year and built on 1961 year. It was constructed in the Murrizi marsh, which extend on the right side of the Semani River. Its aim was to irrigate 26 900 ha of Myzeqe flat, which is largest flat in Albania. The volume capacity of the reservoir is 65 million m^3 of water, from which are 12.5 million m^3 dead volumes and 53 million m^3 of water are exploited. The reservoir water surface in normal condition is 850 ha. The dam is a homogeneous earth fill type built from impermeable materials taken from the valley slope and field soils, which are clays and silty clays. It is 3500 m long, 12.5 m high and 4.5m wide in crest. The upstream escarpment of the dam is 1:3.75, whereas downstream escarpment of the dam is 1:2.75. The upstream escarpment is cover up by riprap. The dam was most probably shaped and formed by a bulldozer. At the base of the dam a stone gravel filter was made. The freeboard of the crest over the normal head water level was 1.68 m for flooding periods. The Thana reservoir has no outlet for extraordinary period.

Landslide and risk assessment

Thana dam is especially subject to slope movements one year after its construction. So, on its body have occurred 5 landslides (Figure 2, 3). The landslides are the

approximate 125m up to 160m long and mass movement going down progressively day by day and month by month.

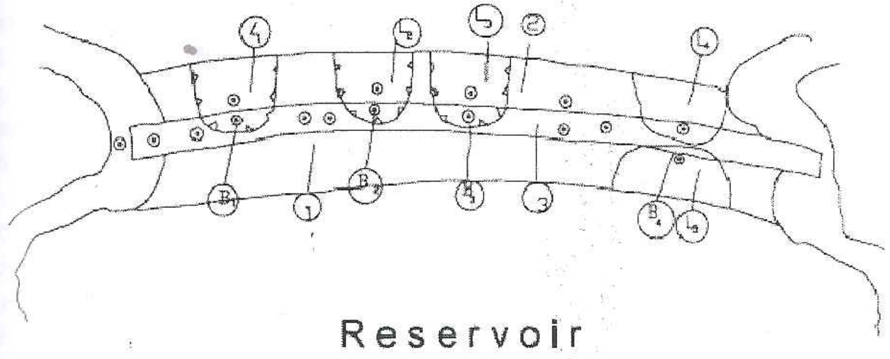


Figure 2: The dam of Thana reservoir

1. upstream escarpment, 2. downstream escarpment, 3. crest, 4. landslide, 6. boreholes

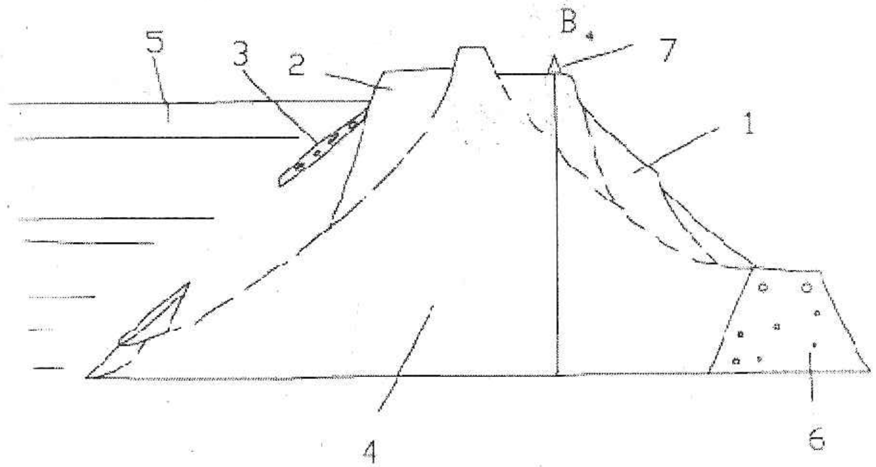


Figure 3: The dam landslides

1. downstream escarpment landslide, 2. landslide of upstream escarpment, 3. Facing (limestone's boulders), 4. dam body, 5. reservoirs water, 6. Stone gravel filter, 7. borehole

Type of movement and type of soil as well as the location of the sliding surface were determined during the field investigation, and the geotechnical parameters of the soil from the sliding zone were obtained by laboratory examination [6]. These landslides can be classified as the earth flow type [7]. The earth flows are caused by many factors. Firstly, during the construction phase the dam soils were not compacted by according to the project, which is certified by the laboratories results. The presence of moisture-laden winds facilitates heavy and often intense rainfall are other main factor. Inherent slope stability factors of the dam include fluctuation of the reservoir water. These variables combine to produce a sensitive terrain where hydrologic and seismic factors (earthquake ground shaking due to site tectonic location in the seismically active plate boundary zone) are particularly effective in producing of landslides. For that reasons the slopes dam are unable to sustain their material under these conditions. The destructive potential of landslides, damage caused, and their role in the degradation of the dam has been summarized above. Sediment supply as a consequence of landslides has reduced the storage potential of water reservoirs. During the rainy periods the water intake for the Thana reservoir is always choked with fine landslide debris. Landslides frequently damage dam body. Dams slopes made bare by landslides are the sites of accelerated soil erosion and appear to be a major cause of dams degradation. Thus, the Thana dam as result of landslide phenomena step by step is degraded. In this condition it is in collapse. From destruction of this phenomenon, it threaten the rural population living on a relatively flat area, a lot of infrastructures and agricultural lands, fruit trees and crops. It follows from the above discussions that in damage of Thana dam from landslides phenomena are a force to reckon with and should be considered as a serious and recurrent natural hazard in many of village of Fieri and Lushnja region. The flat area is extensively used by the local people for farming and further movement will likely result in a significant economic loss. To design a set of remedial measures, a qualitative hazard assessment, including the analysis of the landslide's characteristics, was performed.

Geotechnical conditions

A geotechnical analysis of the dam embankment, based on field and laboratory investigations is written here [6]. Also, from these works result, that the change in groundwater level from fluctuation of reservoir water and heavy rainfall have much influence on the slope stability of the dam. Hazards related to landslides are a major societal and environmental concern to the flat area, which are located on western part. Recurrent landslide damage in Thana dam caused specially by frequent rain storms should be a growing concern to the general public, ministries of the central and local government. From the boreholes logs carried out on dam body, we have determined the lithological profiles, in which are defined 3 layers with different geotechnical properties. It is very important to be emphases in various section of the dam length we found the different geotechnical properties. Therefore, we treat geotechnical condition of the dam in the landslide nr.1, landslide nr.2 and landslide nr.3.

Geotechnical condition of landslide nr.1

From the upper part to lower part of dam lithological profile (Figure 4) there are:

Layer 1. It is represented by inorganic silts and very fine sands-ML with brown color. It's situated on upper part of dam's body. These soils have a thickness various 1.7-3.0m (Figure 3). The physical-mechanical properties of these soils are given in Tables 1.1, 1.2 and 1.3, where is indicated that they have low values.

Layer 2, This layer are organic silts and organic silty clays of low plasticity - OL with grey color. It's situated on middle part of dam's body. These soils have a thickness various 5.0-6.0m (Figure 3). The physical-mechanical properties of these soils are given in Tables 1.1, 1.2 and 1.3, where is indicated that they have very low values.

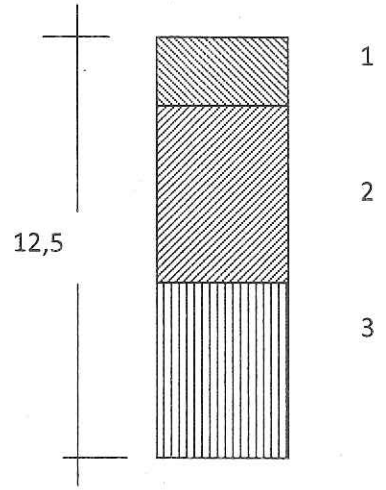


Figure 4: The lithological column of Thana dam

Layer 3, They are Inorganic clays of high plasticity, fat clays- CH with grey color. It's situated on lower part of dam's body. These soils have a thickness various 2.8.0- 4.0m (Figure 3). The physical-mechanical properties of these soils are given in Tables 1.1, 1.2 and 1.3, where is indicated that they have low values.

Table 1.1 Physical properties of dams soils

Layers	Clays %	Silts %	Sands %	W_L %	W_p %
1	-	-	-	45,1	26,3
2	40,4	54,1	5,5	51,95	32,1
3	13,5	78,0	13,5	47,5	80,4

Table 1.2 Physical properties of dams soils

Layers	I_p	W_n %	γ kN/m^3	γ_d kN/m^3	γ_o kN/m^3
1	18,8	28,5	19,3	15,0	27,0
2	18.85	36,1	17,4	12,5	270
3	16,8	42,5	17,4	12,2	26,9

Table 1.3 Mechanical properties of dam's soils

Layers	E	φ	C	USCS
	Kpa.10^4	o	KPa	
1	-	13	55	ML
2	0,326	10	25	OH
3	-	11	7.5	ML

Geotechnical condition of landslide nr.2

The dam lithological profile (Figure 4) from the upper part to lower part are as folloing: Layer 1. It is represented by Inorganic clays of low to medium Plasticity-ML with brown color. It's situated on upper part of dam's body. These soils have a thickness various 2.0- 4.5m (Figure 3). The physical-mechanical properties of these soils are given in Tables 2.1, 2.2 and 2.3, where is indicated that they have low values. Layer 2, This layer are organic silts and organic silty clays of low plasticity - OL with grey color. It's situated on middle part of dam's body. These soils have a thickness various 5.0-6.0m (Figure 3). The physical-mechanical properties of these soils are given in Tables 2.1, 2.2 and 2.3, where is indicated that they have very low values. Layer 3. They are inorganic silts and very fine sands- ML with beige color. It's situated on lower part of dam's body. These soils have a thickness various 2.8.0-4.0m (Figure 3). The physical-mechanical properties of these soils are given in Tables 2.1, 2.2 and 2.3, where is indicated that they have low values.

Table 2.1 Physical properties of dam's soils

Layers	Clays %	Silts %	Sands %	W_L %	W_p %
1	7.7	87.6	4.8	49.2	29.0
2	39.6	53.6	6.8	48.1	29.1
3	57.3	39.8	2.9	43.7	26.6

Table 2.2 Physical properties of dam's soils

Layers	I_p	W_n %	γ kN/m^3	γ_d kN/m^3	γ_0 kN/m^3
1	20.2	35.7	18.3	13.5	27.0
2	19.0	30.0	18.7	14.4	27.0
3	17.1	30.3	19.2	15.9	27.0

Table 2.3 Mechanical properties of dams soils

Layers	E	φ	C	USCS
	Kpa.10^4	o	KPa	
1	0.5769	8	35	CL
2	0.611	9	25	OL
3	0.4382	14	55	ML

Geotechnical condition of landslide nr.3

The lithological profile (Figure 4) of dam is: Layer 1. It is represented by inorganic clays of low to medium Plasticity-CL with brown color. It's situated on upper part of dam's body. These soils have a thickness various 2.2-3.5m (Figure 3). The physical-mechanical properties of these soils are given in Tables 3.1, 3.2 and 3.3, where is indicated that they have low values.Layer 2, This layer are organic silts and organic silty clays of low plasticity - OL with grey color. It's situated on middle part of dam's body. These soils have a thickness various 5.5-6.5m (Figure 3). The physical-mechanical properties of these soils are given in Tables 3.1, 3.2 and 3.3, where is indicated that they have very low values.Layer 3. They are inorganic silts and very fine sands- ML with brown color. It's situated on lower part of dam's body. These soils have a thickness various 2.8.0-4.0m (Figure 3). The physical-mechanical properties of these soils are given in Tables 2.1, 2.2 and 2.3, where is indicated that they have low values.

Table 3.1: Physical properties of dam's soils

Layers	Clays %	Silts %	Sands %	W_L %	W_p %
1	35.0	44.2	20.8	39.1	29.9
2	10.5	79.3	10.5	47.1	24.3
3	7.9	87.0	5.1	51.9	30.2

Table 3.2 Physical properties of dam's soils

Layers	I_p	W_n %	γ kN/m^3	γ_d kN/m^3	γ_0 kN/m^3
1	15.2	-	-	-	-
2	22.8	32.4	18.9	14.3	27.0
3	21.7	24.0	19.5	15.7	27.0

Table 3.3 Mechanical properties of dams soils

Layers	E	φ	C	USCS
	Kpa.10^4	o	KPa	
1	-	15	8.5	CL
2	-	15	10	CL
3	-	15	23	CH

It's very important to note that physical-mechanical properties taken from Proctor test of these (raw materials) soils before of construction dam or better to says in projected phase are shown in Table 4. From these results we see that physical-mechanical properties of the dam fill soils are lower than calculated one, where can mention the mean respectively optimum water content and dry density according to project are 25.2% and 15.2 kN/m3, whereas fill soils in the dam actually are 12,2-12,5 KN/m^3 up to 14.3- 14.5 kN/m^3 (dry density) and optimum water content 28.5-30.3% up to 35.7-42.5%. That means the dam fill soils are unconsolidated conditions and we will have recurrence of the landslide phenomena on the dam body.

Table 4: Results of Proctor's test of dam's soils

Simple number	W_{opt} %	γ_d KN/m^3
10	25.9	15.4
11	24.9	15.3
12	26.8	15.2
13	25.2	15.2
Mean value	25.2	15.2

Notes

Wn-Natural water content, Wopt- Optimum water content, (WL, Wp) Atterberg limits, Ip-plasticity Index, γ-Bulk density, γd-Dry density, γo-Specific density, Shearing strength (internal friction angle φ, cohesion c), modulus of the deformation E from Consolidation test or eodometric test and unified soils classification system-USCS

Conclusions

Thana dam constructed since 1961 year is subject to slope movements. In Thana dam body 5 landslides have occurred, which have the dimension range 125m up to 160m.The mass movements in Thana dam are going on progressively day by day and month by month. The landslides are classified as the earth flow type. The earth flows are caused from many factors, which are not enough compacted soil during the construction phase according to the project, absence of the maintance ets

The presence of moisture-laden winds facilitates heavy and often intense rainfall is other main factor. Inherent slope stability factors of the dam include fluctuation of the reservoir water, which combine to produce a sensitive terrain where hydrologic and seismic factors (earthquake ground shaking due to site tectonic location in the seismically active plate boundary zone) are particularly effective in producing of landslides. From destruction of it is threatened the rural population living on a relatively flat area, a lot of infrastructures and agricultural lands, fruit trees and crops. It has necessary needs to use remedial measures on the dam against any possible catastrophic event.

REFERENCES

[1] Bozo L., Frasheri A., Muceku Y. (2007). *Slope stability in active seismic zones in Albania.* 4th International conference on earthquake geotechnical engineering, June 25-28, paper no. 1185.

[2] Bozo L., Muceku Y. (2005). *Landslides risk assessment on roads in Albania.* International Conference on Landslides risk

management. 31 May-3 June, Vancouver, Canada, Volume nr.2, p.19-24.

[3] Muceku Y. (2006). *Relation between tectonics and landslides in Albania.* International Project on "Seismo-hydrogeological vulnerability of the environment and society in the Balkan region", second meeting 22-26 November, Sofia, Bulgaria, p. 13-22.

[4] Muceku Y., Reci H. (2006). *Mass movement and risk assessment in Komani hydropower area, Albania.* Dams and Reservoirs, Societies and Environment of the 21st Century. Proceedings of the International Symposium on dams in the Societies of the 21st Century, ICOLDSPANCOLD 18-23 June 2006, Barcelona, SPAIN, Vol. 1, p. 723 -729.

[5] Muceku Y. (2005). *Influence of chemical-mineralogical composition on physical of cohesive soils of Tirana-Vora region.* AJNTSAcademy of Sciences of Albania Vol. Nr 2, p. 45 - 54.

[6] Muceku Y. (2002-2007). *The geotechnics study of the geodynamics phenomena in Albania.* (Geological Survey of Albania).

[7] Varnes DJ. (1978). *Slope movements, type and process.* In: Eckel EB (ed) Landslides analysis and control. Transport analysis and control. Transport Research Board, Spec Rep 176. National Academy of Sciences, Washington, DC,Pp11-3

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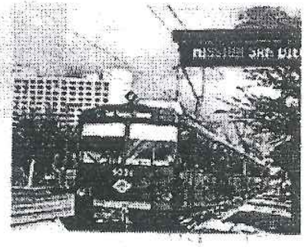

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An upgrade of the microzonation study of the centre of Tirana city

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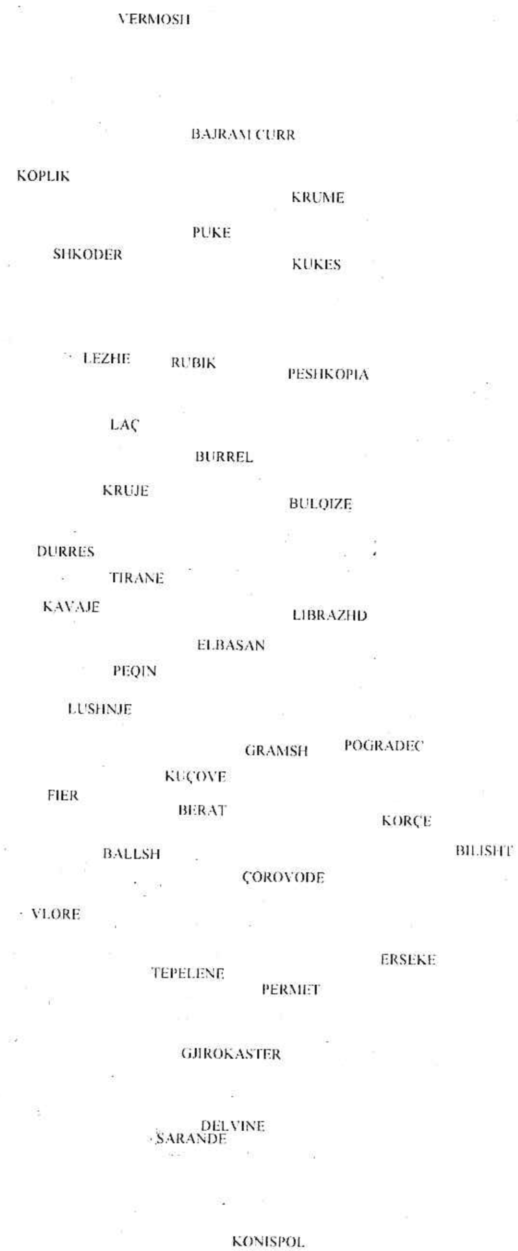
Abstract

An attempt was made in this paper to present an upgrade of the seismic microzonation of a part of the Tirana City. Some results of the previous, complete study performed in the late 80-s were used in this paper. The new aspect of this study is related to the probabilistic assessment of the ground motion parameters. Using gridded seismicity methodology, the PGA and the uniform response spectrum at the base rock corresponding to the 475-years return period were estimated. Based on them, five acceleration time histories are selected, which are later used to assess the response of the geotechnical models that compose this part of Tirana City.

Key words: seismic microzonation, PSHA, gridded seismicity, soil response, Eurocode 8.



Introduction

Until late, the macroseismic intensity was the basic parameter for the assessment of seismic hazard in Albania (Sulstarova *et al.*, 1980). The map of seismic zonation of Albania published in 1980, as well as the seismic hazard maps compiled during the seismic microzonation studies performed during the period 1984-1991 for the seven largest urban areas (Vlora, Durrës, Shkodra, Tirana, Korça, Fier and Pogradeci towns) have been used as reference for such assessments.
The seismic microzonation study of Tirana City was completed on 1988 (Konomi *et al.*, 1988, Kociu *et al.*, 1988) and since that period of time we are facing more rapid changes regarding the methodology of seismic hazard assessment. It is already a necessity in the engineering practice to represent the results of microzonation, and engineering seismology studies in general, in terms of strong motion parameters, such as acceleration, velocity and displacement of ground motion shaking during strong earthquakes.
The Centre of Tirana has been reconsidered recently and a new regulatory plan is foreseen. Under these circumstances it is of great interest to review the seismic hazard assessment aspects of this part of the City. For this purpose, we used all the available information on seismic microzonation of Tirana city completed in 1988, regarding the geotechnical and engineering geological aspects of soil, including the physical-mechanical and seismic wave velocity properties given in that study (Konomi *et al.*, 1988, Kociu *et al.*, 1988).
The evaluation of seismic hazard that can threat Tirana Centre (Fig. 1) is carried out using probabilistic methodology. Seismic hazard has been represented in terms of strong motion parameters, such as peak ground acceleration PGA and spectral accelerations SA, for five periods of soil vibration. We calculated the seismic hazard curves of this area and the response spectra expressed in terms of absolute acceleration. The seismic hazard deaggregation in terms of PGA was done and based on the results of this procedure, the time histories for rock site conditions were developed. Then, these time histories are used as input motion and are propagated into the geotechnical models given in the study of microzonation of Tirana for this part of the City. Finally, the PGA for different levels of soil depth is calculated. The comparison of the mean resulting response spectra with the relevant spectral shapes of the Eurocode 8 (Eurocode 8, 2003) was performed.

Geologic framework of Tirana city

Tirana City takes place in the Periadriatic Depression, right on the most southern plain part of Tirana molasse syncline. Tirana syncline, about 80 km long and 10-12 km wide, represents an asymmetric syncline with the strongly dipping up to overturned western flank and gently dipping eastern flank. It is built by molasse deposits of Middle-Upper Miocene age and partly by Pliocene molasse in the most northern part of it (Aliaj, 2000).

Miocene molasse is placed transgressively and with unconformity on the carbonate-flysch structures of Ionian and Kruja Zones (Fig. 2). Only on the eastern flank of Tirana syncline is observed the transgressive and discordant placing of Miocene molasse over the Oligocene flysch of Kruja Zone.

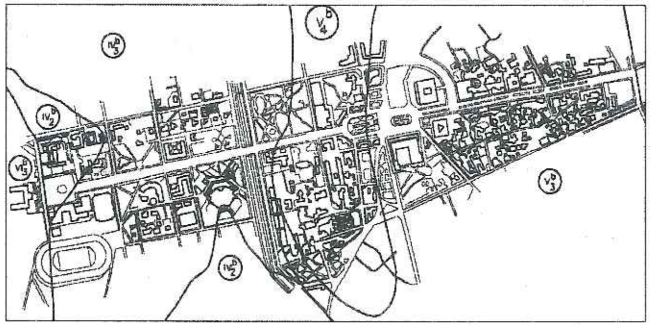


Fig. 1. Tirana Centre area and position of geotechnical models of this part of the City

Serravalian sediments, about 600 m thick, are represented by lithotamnium and organogene limestones in the lower part of section, passing upward into clays and sandstones. Tortonian sediments are characterized by clays passing upwards into clayey-sandstone intercalations, 100-2000 m thick.

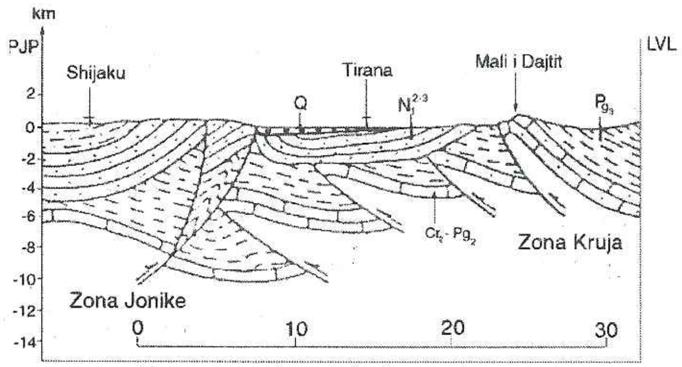


Fig. 2. Shijak-Dajti Mountain geologic cross-section (Aliaj, 2000)

Quaternary sediments are represented by gravels intercalated with clayey and sandy layers. They are about 15-20 m thick in Tirana City, and towards the north up to around 200 m thick near the Mati River (Aliaj, 1996).
Tirana City takes place in the most south-eastern part of plain area, 100-140 m over the sea level. From the east, south and west Tirana plain borders by low hills, built by Miocene molasse sediments. This plain, overplaced on Tirana syncline, represents a

graben-like structure, bordered from the west by Preza backthrust and from the east by Dajti thrust (Aliaj, 2001).

Seismic activity of Tirana area

Tirana is a relatively new inhabited area. It became capital of the country in 1920. As a consequence, the data for the seismic activity for this City and surrounding area are limited (Sulstarova and Koçiu, 1975; Sulstarova *et al.*, 1980).

The strongest earthquake that has hit the town of Tirana is that of 09.01.1988, with M_s=5.4 and intensity I_o=7-8 degree (MSK-64), (Koçiu and Pitarka, 1990). The effect of this earthquake on the surface showed once more the importance of the ground conditions in the intensity of moderate earthquakes.

From the seismic faults that surround Tirana many earthquakes have been generated, the strongest of them are: 1617 year, with I_o=8 degree (MSK-64) in Kruja, 26.08.1852 with I_o=8 degree (MSK-64) in Rodoni Cape, 16.05.1860 with I_o=8 degree (MSK-64) in Beshiri bridge, 04.02.1934 with M_s=5.6 in Ndroq, 19.08.1970 with M_s=5.5 and I_o=7 degree (MSK-64) in Vrapi area, 16.09.1975 with M_s=5.3 in Rodoni Cape, 22.11.1985 with M_s=5.5 in Drini Gulf and 09.01.1988 with M_s=5.4 in Tirana. So, Tirana area is stricken from historical earthquakes with I_o=8 degree (MSK-64) and during the XX century from earthquakes of magnitude M=5.3-5.6.

Engineering geological conditions of Tirana city centre

Tirana City Centre from engineering geological point of view is characterized by four zones, from which the zones III and V belong to the second terrace of Tirana river, the zone IV belongs to the first terrace of Lana river, while the zone VI belongs to the rocks of Upper Miocene molasse, with or without the elluvial-delluvial cover (Konomi *et al.*, 1988; Koçiu *et al.*, 1988).

In zone III is included a part of first terrace of Tirana river, extending from ex Dinamo plant up to the train station and following in the shape of an narrow belt 400-500 m wide up to ex School of Party. According to the basement on which the Quaternary sediments are overplaced, two subzones are divided:

a. Subzone III^a with sandstone basement
b. Subzone III^b with clayey-silty basement.

In the most northern part of the City Centre, at train station the model III_3^b is developed.

In engineering geological zone IV is included the first terrace of Lana River, which is extended along the Lana River flowing. This zone is divided into two subzones:

a. Subzone IV^a with sandstone basement, and
b. Subzone IV^b with clayey-silty basement.

In Tirana City Centre two geotechnical models IV_2^b and IV_3^b are developed (Fig. 1).

In engineering geological zone V included is the second terrace of Tirana River, which follows along the Tirana City Centre from the train station up to the Lana River. This zone is divided also into two subzones:

a. Subzone V^a with the sandstone basement, and
b. Subzone V^b with clayey-silty basement.

In Tirana City two geotechnical models V_3^b and V_4^b are developed (Fig. 1).

In engineering geological zone VI included is the hilly unit, which is also divided into two subzones:

a. Subzone with sandstone basement, and
b. Subzone with clayey-silty basement.

In most southern part of Tirana City Centre is developed only the geotechnical model VI_3^b (Fig. 1). In the following paragraph are treated in details all geotechnical models building the Tirana City Centre.

Geotechnical models of Tirana Centre area

The effects of earthquakes on the surface depends not only from the regional characteristics of the geological settings, but also from the local soil layers that forms the soil profile from the surface up to bedrock.

Depending on their geometrical, physical-mechanical and dynamic characteristics, surface layers modify the amplitude-frequency content of seismic waves, whereby they directly affect the intensity of their destructive effect upon structures.

For the determination of the above mentioned characteristics of the surface layers and their effect upon earthquake motion at the basement of structures, it is necessary the geological and geophysical investigations with seismic methods of geotechnical models to be performed, in order to determine their velocity properties, apart physical-mechanical ones. Based on these values, the determination of the representative geotechnical models of specified areas that are necessary for the study of the effect these models exercise on earthquake vibration is possible. Geotechnical models of Tirana centre represented in Fig. 1 are those compiled in the framework of microzonation study of Tirana city in 1988 (Konomi *et al.*, 1988; Koçiu *et al.*, 1988). These are the following: model IV_2^b, model IV_3^b, model V_3^b, model V_4^b and model VI_3^b. The amplification medium of seismic waves is the surface Quaternary layers with general thickness starting from 6.5 to 21m.

Due to the good stiffness and physical characteristics, the wide extension and the large thickness supposed, the medium of Neogene sediments was adopted as seismic bedrock. Models structure with the necessary geometrical, physical (H, V_S, γ) and lithological parameters are presented in the tables 1- 5. In these tables, presented are the plasticity indexes and appropriate data base numbers of 11 relations for the normalized shear modulus and damping ratios in regard to the shear strain level as they are required in the computer code WESHAKE5 for the earthquake equivalent linear analyses used in this study for the assessment of geotechnical models response (Yule *et al.*, 1995).

Table1. Geotechnical model IV_2^b						
	Layer's Index		Thickness (m)	Plasticity Index	V_S (m/s)	Bulk Density (T/ m3)
	Sh.M	D.R				
1	7	6	1.5	(15)	100	1.50
2	7	6	2.5	(15)	210	1.62
3	7	6	2.0	(15)	210	1.90
4	7	6	3.0	(15)	350	1.87
5	1	1			600	1.96
1-Filling; 2-Inorganic silts and fine sands; 3-Inorganic silts and fine sands Inorganic silts with 10-30% gravel content; 5-Bedrock						

Table 2. Geotechnical model IV_3^b						
	Layer's Index		Thickness (m)	Plasticity Index	V_S (m/s)	Bulk Density (T/ m3)
	Sh.M	D.R				
1	7	6	1.0	(15)	150	1.45
2	7	6	1.5	(15)	250	1.60
3	7	6	1.5	(15)	350	1.74
4	7	6	3.5	(15)	350	1.74
5	2	2	5.0		450	2.00
6	1	1			700	2.10
Layers: 1-Filling; 2-Inorganic silts and fine sands; 3- Mixture of silts with gravel and sands; 4- Mixture of silts with gravel and sands; 5-Gravel; 6-Bedrock						

Table 3. Geotechnical model V_3^b						
	Layer's Index		Thickness (m)	Plasticity Index	V_S (m/s)	Bulk Density (T/ m3)
	Sh.M	D.R				
1	7	6	1.0	(15)	190	1.47
2	7	6	3.0	(15)	300	1.48
3	7	6	6.0	(15)	490	1.90
4	2	2	2.0		500	2.03
5	1	1			700	2.10
Layers: 1-Filling; 2-Inorganic silts and fine sands; 3-Inorganic silts with 20-30% gravel content; 4-Gravel; 5-Bedrock						

Table 4. Geotechnical model V_4^b						
	Layer's Index		Thickness (m)	Plasticity Index	V_S (m/s)	Bulk Density (T/ m3)
	Sh.M	D.R				
1	7	6	2.0	(15)	190	1.47
2	7	6	2.0	(15)	290	1.67
3	7	6	4.0	(15)	430	1.83
4	2	2	13.0		500	1.93
5	1	1			800	2.15
Layers: 1-Filling; 2-Inorganic silts and fine sands; 3- Mixture of silts with gravel and sands; 4-Gravel; 5-Bedrock						

Table 5. Geotechnical model VI_3^b						
	Layer's Index		Thickness (m)	Plasticity Index	VS (m/s)	Bulk Density (T/ m3)
	Sh.M	D.R				
1	7	6	2.5	(15)	130	1.40
2	7	6	1.5	(15)	220	1.57
3	7	6	2.5	(15)	340	1.74
4	1	1			550	2.05
Layers: 1- Eluvial-deluvial formations; 2-Inorganic silts; 3-weathered formation; 4-Bedrock						

Probabilistic seismic hazard assessment for Tirana centre

Of various probabilistic methods in use, we choose the spatially smoothed seismicity approach, developed by Frankel (1995) and further refined by Lapajne *et al.* (2003), and widely used today (Frankel *et al.*, 2000; 2002; Petersen *et al.*, 2008). The method still follows the basic approach established by Cornell in 1968, but no delineation of seismic sources is needed. An earthquake data file comprising the Albanian territory and extending about 100 km from its geographical borders, that covers the time period 373 BC up to 31/12/2005, and the area between 18.0-22.5°E and 38-43.5°N comprising a total of about 2770 events with $M_W \geq 4.5$ was used in the calculations (Kuka and Duni, 2007). The observed area is divided into grid cells, and in each cell the seismic activity rate (the number of earthquakes above the threshold magnitude) is calculated and then spatially smoothed with a Gaussian function. The annual rate of exceedance of the specified level for a given ground motion parameter, and finally the relevant value corresponding to a given return period is calculated. The adopted approach considers different alternatives about fundamental hypothesis on input parameters to account for and to propagate uncertainties in the model within a logic-tree framework.

The hazard computations have been carried out by the use of an upgraded version of the "OHAZ" software (Zabukovec, Kuka *et. al.*, 2007).

Site specific PSHA

PSHA has been performed for the Tirana Centre area for return periods of 95, 475, 975, and 2475 years, corresponding to probabilities of exceedance of 10% in 10 years, and 10%, 5%, and 2%, respectively, in 50 years. PGA and spectral accelerations SA 10, 5, 3.3, 2, 1, and 0.5 Hz has been target of our study.

The reference site condition is firm rock, defined as having an average shear-wave velocity of 800 m/sec, corresponding site class A of Eurocode 8 provisions (Eurocode 8, 2003). The doubly-truncated exponential GR recurrence relation is used, with b-value equal 1.2, lower bound magnitude M_W=4.5, and upper bound magnitude M_{max}=7.2. The maximum distance applied in the computation is 100 km. As predictive ground-motion model we used that of Boore *et al.*, (1997).

Table 5. Maximum horizontal bedrock PGA and SA at the Tirana Centre area

Period Sec	Spectral Acceleration, g			
	RP=95y	RP=475y	RP=975y	RP=2475y
PGA	0.184	0.274	0.319	0.385
0.10	0.262	0.434	0.526	0.671
0.20	0.349	0.562	0.682	0.853
0.30	0.306	0.499	0.610	0.771
0.50	0.196	0.331	0.408	0.529
1.00	0.008	0.141	0.177	0.235
2.00	0.043	0.075	0.095	0.125

Hazard curves

The relationship between the ground motion level and its annual probability of occurrence is described by a hazard curve. In the Fig. 3 presented are the hazard curves we developed for PGA and response spectral accelerations for a suite of periods with engineering interest, for Tirana Centre area. Then, the annual frequency of exceedances are plotted (dashed horizontal lines), which correspond to probabilities typically used for the design, as 10% in 10 years (RP=95years), and respectively 10% (RP=475), 5% (RP=975), and 2% (RP=2475) in 50 years. In Eurocode 8 provisions, two hazard levels, which are 10% probability of exceedance in 10 years (damage limitation requirements), and 10% probability of exceedance in 50 years (no-collapse requirements), are considered.

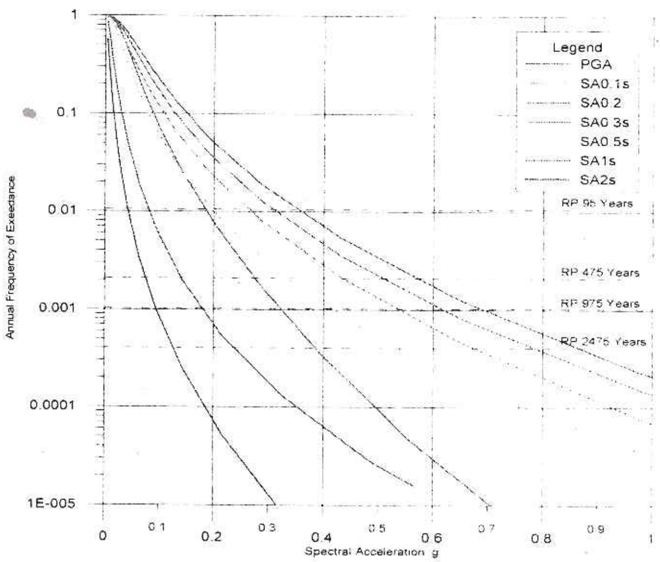


Fig. 3. ***Seismic hazard curves (rock conditions) for PGA and spectral accelerations SA 10, 5, 3.3, 2,1- and 0.5 Hz, for Tirana Centre area***

Uniform Hazard Response Spectrum (UHRS)

The decision to use response spectral values is based on earthquake data obtained during the past 20 plus years showing that site-specific spectral values are more appropriate for design input than the coefficients based on peak ground acceleration used with standard spectral shapes. The differences are particularly pronounced for the short-period portion in the response spectra (Leyendecker *et al.*, 2002). In this study we considered four hazard levels: 10% of exceedance probability in 10 years, and 10%, 5%, and 2% of exceedance probability in 50 years, corresponding to the 95-, 475, 975- and 2475-year return periods, respectively. The maximum horizontal bedrock PGA and spectral accelerations (SA) for each RP are obtained from PSHA and are listed in Table 5. The uniform hazard spectra (UHRS) for each return period are plotted in the Fig. 4.

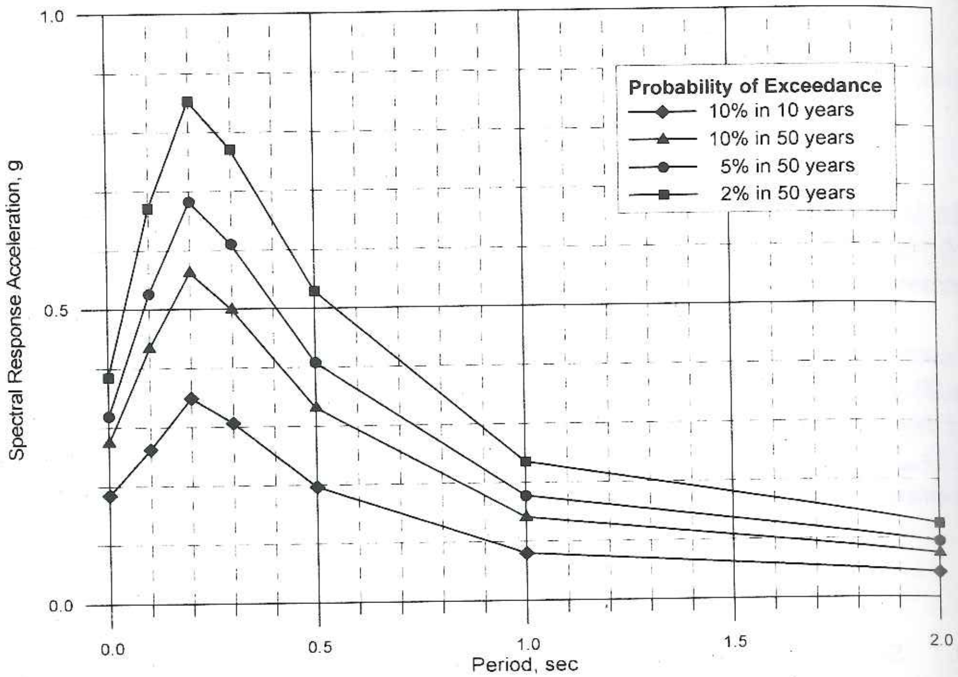


Fig. 4. Uniform hazard spectra for 2%, 5%, 10% probability of exceedance in 50 years, and 10% probability of exceedance in 10 years

Deaggregation of seismic hazard for Tirana Centre area

PSHA uses aggregate contribution of various potential earthquake sources to calculate the annual rate of exceedance of a given site. The resulting hazard does not represent a single earthquake scenario associated with a specific magnitude and distance. It is, however, possible to calculate the relative contributions of the individual sources to the hazard. An important element of probabilistic seismic hazard analysis is the incorporation of ground-motion uncertainty from the earthquakes sources. The results are commonly displayed in terms of relative contributions for a range of magnitude, distance, and epsilon, ε, which represents the number of standard deviation from the median ground motions estimated from the Predictive Ground Motion Model (PGMM). It allows estimation of earthquake scenarios that have high likelihood of occurrence. This process is referred to as deaggregation.

For the modal-event R,M,ε_0, denoted $(\hat{R},\hat{M},\hat{\varepsilon}_0)$, it is reasonable to suppose that (Harmsen *et al.*, 2003):

$$SA_0 \approx \exp(\mu_A + \hat{\varepsilon}_0\sigma_A) \qquad (1)$$

where μ_A and σ_A are mean and standard deviation of the normal distribution of $\ln(SA)$ given distance $\hat{R}$ and magnitude $\hat{M}$. SA_0 is the ground motion associated with a

specific probability of exceedance (PE), for example, 10% in 50 year. The right-hand side of (1) should be computed using a PGM model that is used in the PSHA.

The deaggregation for Tirana Centre area was done for PGA for return period 475 years and is presented in Fig. 5. The deaggregation of seismic hazard was performed using the computer codes of USGS National Seismic Hazard Maps Program (2002-2003), provided to us thanks the courtesy of S. C. Harmsen of USGS. Because the original codes were written for UNIX platforms, we adjusted them for use in PC environment.

So, considering the deaggregarion results for PGA in terms of R, M, ε_0, for the mean event we have D=7.6 km, M=5.4 and epsilon =1.52.

In fact, if we observe the map of active faults presented in Fig. 6, we can assume that the above mentioned scenario is related to the nearest faults circumscribing the City. Based on the map of active faults of Albania (Aliaj, 1997; 2000), as most hazardous for this site are the two active tectonic faults that circumscribe the Tirana syncline: in the west by the Preza back-thrust and to the east by the Dajti thrust, so here is formed a graben-like structure (Aliaj *et al.*, 2001). These faults are shown in greater details in Fig. 6. The shortest site distance of Centre area of Tirana from the two above mentioned faults is in the order of 6.4 – 7 km.

Another source of threat for Tirana City is the system of active faults of back-thrust type, active during Quaternary age that are evidenced very well in the Durrësi area, at the distance of 30-35 km from Tirana. The maximum magnitude of these faults is estimated at the order of M_{max}=6.9 (Aliaj, 1988; 2000). So, the second distance-magnitude pair for Tirana Centre that can be considered as source of threat is D=35 km, M=6.9.

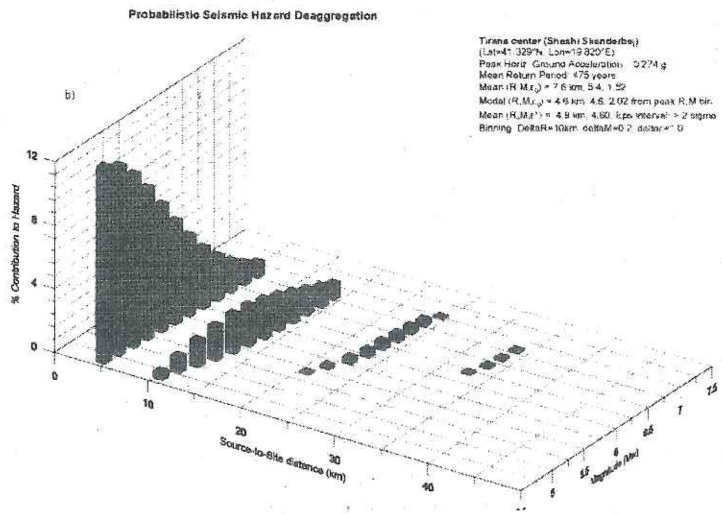


Fig. 5. Deaggregation of seismic hazard at the Tirana Centre area by magnitude and distance for PGA, 475-year return period

Psha and development of ground motion time histories

One of the objectives of this study is to develop time histories for use in linear or/and non-linear time history analysis of the various structures to be built in the area of Tirana Centre. For this reason we generated synthetic seismograms using the well-tested program SMSIM_TD, by David Boore (Boore, 2000). This program uses the stochastic method and assumes a point source.

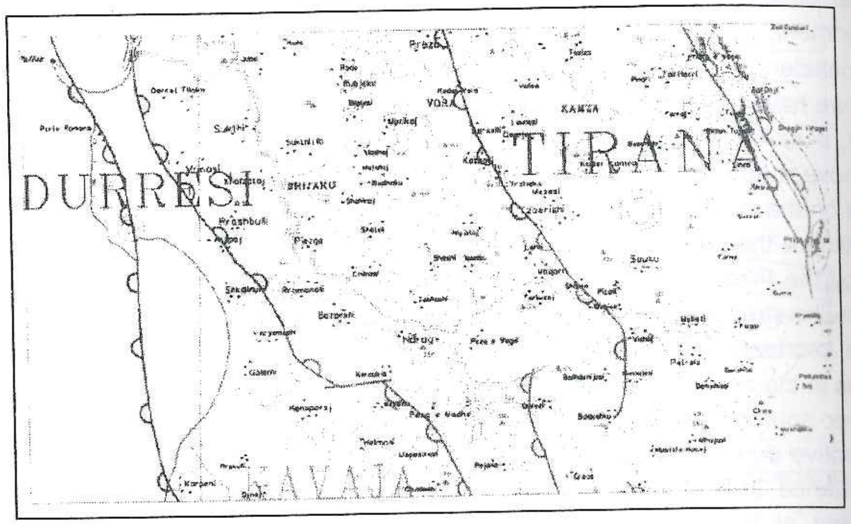


Fig. 6. Map of active faults surrounding Tirana City

Stochastic simulation of strong motion time series

The most widely used method to generate the synthetic ground motions is the stochastic point-source method (Boore, 1983). The stochastic models utilize the characteristics that observed ground motions can be characterized as finite-duration band limited Gaussian noise with an amplitude spectrum specified by a simple source model and path effects. Based on the analysis of the latest records of the Albanian Strong Motion Network (ASMN) that has been recently put in operation, a seismological model that supports the simulation of high frequency ground motions was established making use of the theoretical models, the most popular of which is the stochastic model of Hanks and McGuire, (1981), (Duni, 2004, Duni and Kuka, 2005; 2008). The adequacy of the single corner-frequency ω-square source model of Brune (1970, 1971) in estimating stress-drop parameter comparing visually simulated and observed PSRV response spectra derived from a small number of recordings taken at five stations of ASMN, was shown. Two distinct variation intervals of Brune stress-drop are observed analyzing the records of our data set. The first interval is related to values between 15 bars and 60 bars with main shocks having the largest values $\Delta\sigma$ = 50–60 bars, typical for areas of normal focal mechanism. The second interval is characterized by values in the range $\Delta\sigma$ = 200–300 bars, but the estimates are based on only three records generated by two earthquakes (Duni and Kuka, 2008). These values of stress-drop release are typical for thrust focal mechanism. Estimates of high-frequency diminution parameter κ_o by our data set have differenced the ground type A according EC8 (EC8, 2003) by values in the range $0.02 \le \kappa_o \le 0.04$, while for ground type B the estimates for two sites is quite the same, $\kappa_o = 0.07$. The conclusion was reached that these results can be used for predictions of ground motions in Albania taken into account not only the different $\Delta\sigma$ values found for the two main tectonic zones characterized by compressional and extensional movements, but also the κ_o and amplification factors used in the analysis (Duni and Kuka, 2005; 2008). For the generation of synthetic time histories for Tirana Centre site the value of stress-drop $\Delta\sigma$ = 200 bars and high-frequency diminution parameter κ_o = 0.0338, typical for rock conditions at Tirana, were used.

The two time histories TIR1_S and TIR2_S simulated using the SIM_TD code of the SMSIM programme (Boore, 2000) for D1=7.6 km; M1=5.4 and D2=35 km; M2=6.9, respectively are shown in the Fig. 7.

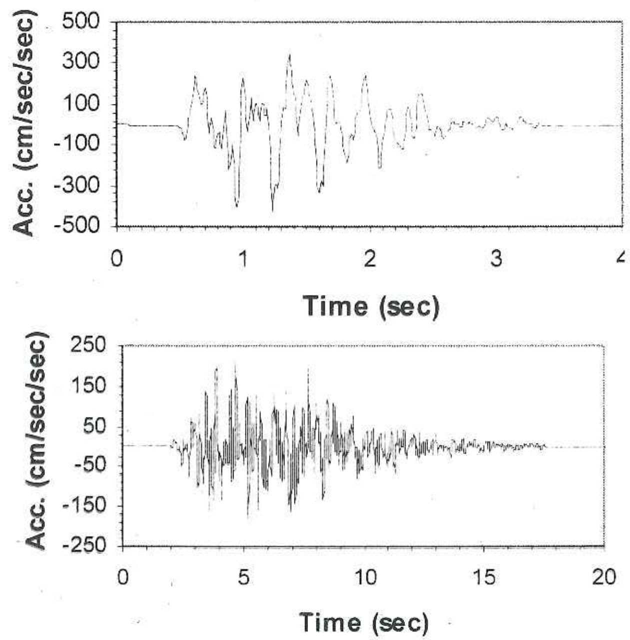


Fig. 7. ***Simulated acceleration time histories for Tirana Centre: TIR1_S (D1=7.6 km; M1=5.4, upper, and TIR2_S (D2=35 km; M2=6.9) lower.***

Representative recorded acceleration time histories for the analysis

Except the two above mentioned simulated time histories, we can choose three records taken by the ASMN and other networks of the region. One of the strongest earthquake that has hit Tirana City is the event of January 9, 1988, M_S = 5.4 (ISC), I_o = 7-8 degree of MSK-64 scale and epicentral distance D=8 km. This earthquake was recorded by the strong motion instrument of SMA-1 type of the Tirana Seismological Station situated on

Tortonian sandstones, near the causative fault. The maximum acceleration recorded on E-W component, showing a strong directivity effect, was PGA=0.4 g, PGA=0.1 g on N-S component and PGA=0.07 g on vertical one. The time duration was no more than 6 seconds (Koçiu and Pitarka, 1990). We have nominated these two records as: TIR1_R (TIR-EW) and TIR2_R (TIR-NS). The other record is that of the Montenegro, Aprill 15, 1979 earthquake (Mw=6.9), AL-NS, recorded on rock condition (sandstones) in Albatros Hotel in Ulqini (Montenegro). The similarity of the seismotectonic environment between coastal areas of that country and Albania is evident that gives us confidence to use this record for the assessment of seismic hazard of Tirana Centre. On Fig. 8 presented are these three recorded time histories used for the analysis of the soil response of the geotechnical models that compose the area of our study.

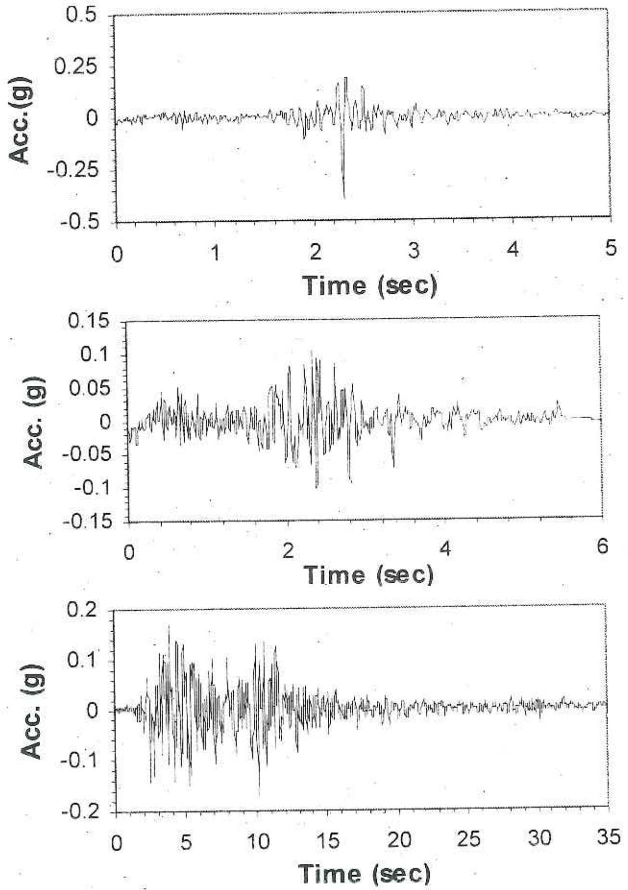


Fig. 8 Recorded acceleration time histories TIR1_R (upper), TIR2_R (middle) and AL-NS (bottom)

Dynamic response of the geotechnical models

For the study of geotechnical models behavior during earthquakes, represented in Tables 1-5, WESHAKE 5 computer code was used (Yule *et al.*, 1995). As input motion functions five acceleration time histories shown in the Fig. 7 and 8 were used. All these time histories have been scaled to the PGA value 0.27 g, according to the seismic hazard estimated for the area of Tirana Centre in bedrock level. In Fig. 9 and Table 6 presented are the PGA values at the top of each soil layer according to the parameters of every geotechnical model.

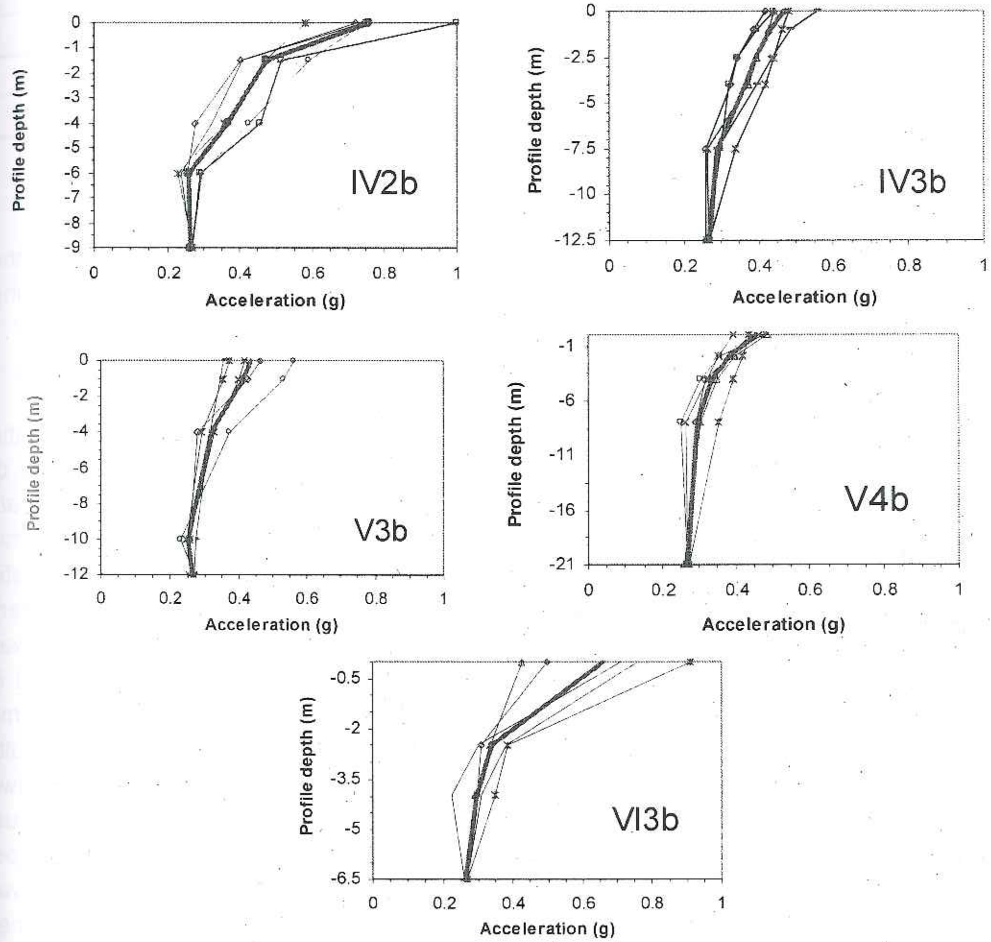


Fig. 9. Variation of PGA in the 5 geotechnical models of Tirana Centre

Table 6 Distribution of the mean PGA values into the soil profile							
IV_2^b	Depth (m)	0.0	1.5	4.0	6.0	9.0	
	PGA (g)	0.76	0.48	0.37	0.26	0.27	
IV_3^b	Depth (m)	0.0	1.0	2.5	4.0	7.5	12.5
	PGA (g)	0.47	0.43	0.39	0.37	0.29	0.27
V_3^b	Depth (m)	0.0	1.0	4.0	10.0	12.0	
	PGA (g)	0.44	0.41	0.32	0.25	0.27	
V_4^b	Depth (m)	0.0	2.0	4.0	8.0	21.0	
	PGA (g)	0.45	0.38	0.34	0.29	0.27	
VI_3^b	Depth (m)	0.0	2.5	4.0	6.5		
	PGA (g)	0.66	0.34	0.30	0.27		

In Fig. 10 we present the elastic acceleration response spectra with 5% damping at the surface of the geotechnical models together with the EC8 spectral shapes according the soil categories of this design code.

Discussion and conclusions

Presented in this paper are the results of an investigation for the microzonation of the Centre of Tirana City. The data regarding the soil profiles included into the study of Tirana microzonation performed in the late 80-s (Konomi *et al.*, 1988; Koçiu *et al.*, 1988) were used. The new aspect of the actual study regards the level of the seismic input that serves for the evaluation of the soil profiles response. The probabilistic methodology is used for the assessment of seismic hazard and the PGA for 475 years return period as well as the uniform hazard spectrum for this part of the City was determined. Deaggregation of seismic hazard for PGA for 475 years return period is accomplished and the mean D and M pair is used for the simulation of acceleration time histories, to be used as input motion. From the comparison with the system of faults that circumscribe the City, it is evident that these deaggregation results reflect the two active tectonic faults at two sides of Tirana syncline: in the west the Preza back-thrust and to the east, the Dajti thrust. Further, the system of active faults of back-thrust type, active during Quaternary age that are evidenced very well in the Durrësi area was taken as another source of threat for Tirana City. The total number of time histories chosen as representative for the assessment of soil response is five. The time histories were propagated into the five geotechnical models and the PGA values at the top of every layer were evaluated, together with the response spectra of 5% damping were shown.

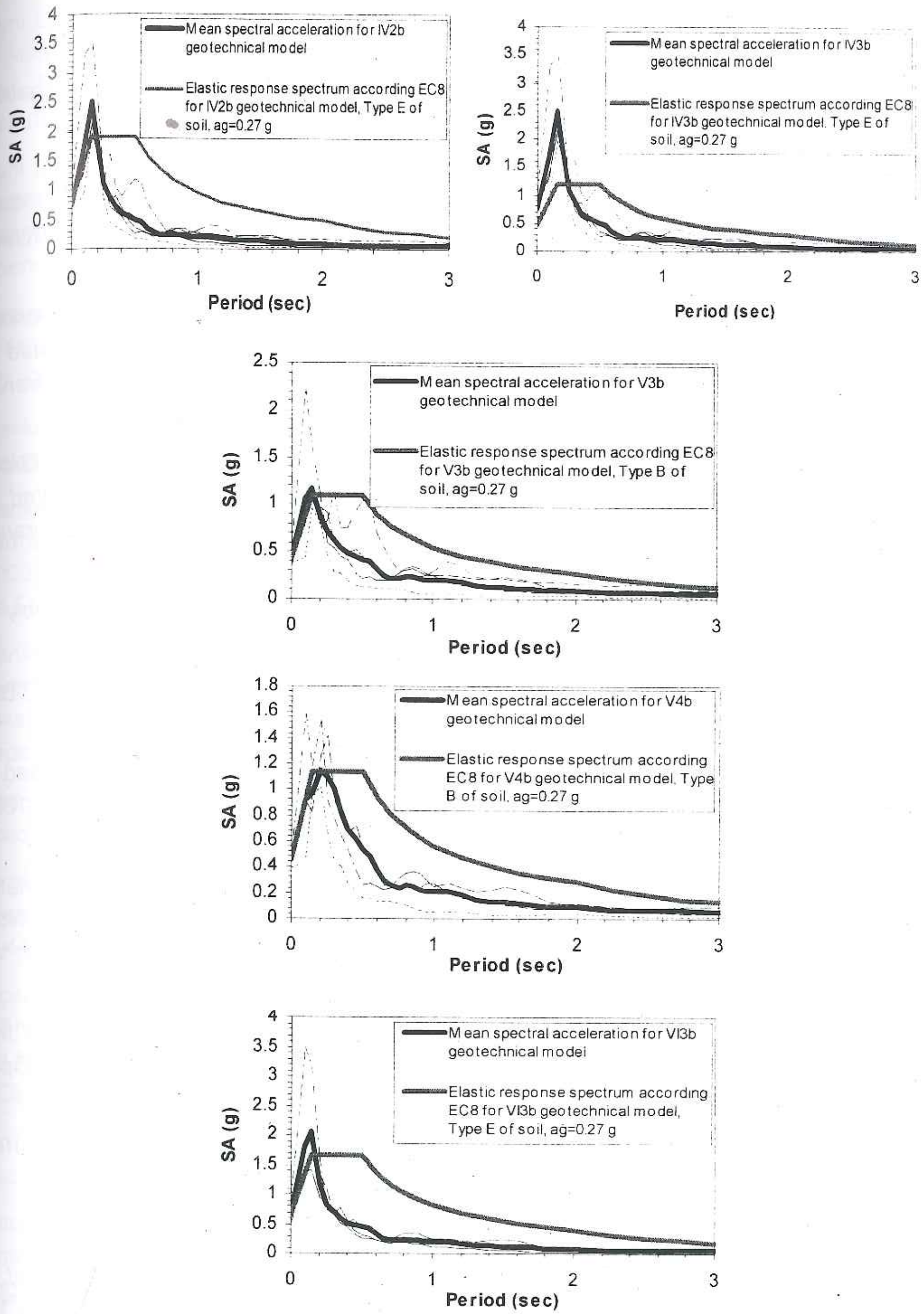


Fig. 10. ***Elastic acceleration response spectra with 5% damping at the top of soil profiles as well as the spectral shapes of EC8.***

Acknowledgement

The help of Dr.S.C. Harmsen of USGS who provided the seismic hazard deaggregation computer code is gratefully acknowledged.

REFERENCES

Aliaj, Sh. [1988]. "Neotectonic and seismotectonic of Albania". Doctoral thesis, Archive of Institute of Geosciences, Tiranë, (in Albanian).

Aliaj, Sh. [1996]. "Neotectonics of Tirana Region (Albania)". *Proc. Of the First Working Group Meeting Int. Project on "Expert Assessment of Land Subsidence Related to Hydrogeol. and Eng. Geol. conditions in the Regions of Sofia, Skopje and Tirana"*, Sofia, pp. 72-81.

Aliaj, Sh. [1997]. "Active faults in Tirana Region". *Proc. of the Second Working Group Meeting, Inter. Project on "Expert Assessment of Land Subsidence Related to Hydrogeol. and Eng. Geol. conditions in the Regions of Sofia, Skopje and Tirana"*, Skopje, pp.37-45..

Aliaj, Sh. [2000]. "Active Fault Zones in Albania". *Abstrakt, General Assembly of Europian Seismological Commission*, Lisbon, Portugal, September, 2000.

Aliaj, Sh. [2001]. "Quaternary subsidence zones in Albania: some case studies", Bull. Eng. Geol. Env., 59, pp. 313-318.

Boore, D. M. [1983]. "Stochastic simulation of high-frequency ground motions based on seismological models of the radiated spectra", Bull. Seism. Soc. Am., 73: pp. 1865-1894.

Boore, D. M., W. B. Joyner and T. E. Fumal. [1997]. "Equations for Estimating Horizontal Response Spectra and Peak Ground Acceleration from Western North American Earthquakes: A Summary of Recent Work", Seism. Research Letters, Vol. 68, No.1, pp. 128-153.

Boore, D. M. [2000]. "SMSIM-Fortran programs for simulating ground motions from earthquakes: Version 2.00 - A revision of OFR 96-80-A", US Geological Survey Open-File Report OF 00-509. 55 p.

Brune, J. N. [1970]. "Tectonic stress and the spectra of seismic shear waves from earthquakes", J. Geophys.Res., 75, pp. 4997-5009.

Brune, J. N. [1971]. "Correction", J. Geophys.Res. 76, 5002.

Duni, Ll. [2004]. "Strong motion monitoring in Albania", *Proc. of the Intern. Conf. on "Seismic Risk Management Issues in Montenegro and the Region"*, Podgorica, pp.183-198.

Duni Ll. and N. Kuka [2005]. "Stress-drop estimates from recent strong motion records in Albania", *Proc. of the 2-nd intern. Conf. of Central Europian Initiative (CEI), "Science and Technology for Safe Development of Lifeline Systems"*, Bratislava, Slovak Republic, 6 p.

Duni Ll. and N. Kuka [2008]. "*Estimates of stress-drop and high frequency diminution parameter from strong motion data recorded in Albania*", "Earthquake Monitoring and Seismic hazard Mitigation in Balkan Countries", E. S. Husebye editor, Spinger Science+Business Media, pp. 57-79.

Eurocode 8 [2003]. "*Design of structures for earthquake resistance; Part 1: General rules, seismic actions and rules for buildings", Draft No. 6; Version for translation (Stage 49), Doc CEN/TC250/SC8/N335*", European Committee for Standardization, pp. 1-30.

Frankel, A. D. [1995]. "Mapping seismic hazard in the Central and Eastern United States", Seism. Research Letters, V. 66, No. 4, p. 8-21.

Frankel, A.D., C. S. Mueller, T. P. Barnhard, E. V. Leyendecker, R. L. Wesson, S. C. Harmsen, F. W. Klein, D. M. Perkins, N. C. Dickman, S. L. Hanson and M. G. Hopper [2000]. "USGS National Seismic Hazard Maps, Earthquake Spectra", Prof. Journal of the Earthq. Eng. Research Institute, Vol. 16, No. 1.

Frankel, A.D., M.D. Petersen, C.S. Mueller, K.M. Haller, R.L. Wheeler, E.V. Leyendecker, R.L. Wesson, S.C. Harmsen, C.H. Cramer, D.M. Perkins and K.S. Rukstales [2002]. "Documentation for the 2002 update of the national seismic hazard maps", USGS Open-File Report 02-420.

Hanks, T. C. and R. McGuire [1981]. "The character of high-frequency strong ground motion", Bull. Seism. Soc. Am., 71: pp. 2071-2095

Harmsen, S. C., A. D. Frankel and M. D. Petersen [2003]. "Deaggregation of U.S. Seismic Hazard Sources: The 2002 Update", U.S. Geological Survey Open-File Report 03-440.

Koçiu, S., Sh. Aliaj, A. Pitarka, V. Peçi, N. Konomi, H. Dakoli, K. Prifti, A. Koçiu, J. Kero, V. Shehu, K. Goga, N. Goro, L. Kume, L. Kapllani, P. Papadhopulli, R. Eftimi, M. Kondo and N. Puka [1988]. "*Microzonation of Tirana City*", Technical report, Archive of Institute of Geosciences, (in Albanian).

Koçiu, S. and A. Pitarka [1990]. "A Check-up on seismic hazard assessment: Tirana case study", Natural Hazard, Vol. 3, No. 3, pp. 293-305.

Konomi, N., H. Dakoli, K. Prifti, A. Koçiu, J. Kero, V. Shehu, K. Goga, N. Goro, L. Kume, L. Kapllani, P. Papadhopulli, R. Eftimi, M. Kondo and N. Puka [1988]. "*Engineering geological zonation of Tirana City*", Technical report, Archive of Faculty of Geology and Mine, (in Albanian).

Kuka, N. and Ll. Duni [200]. "*Probabilistic Seismic Hazard Assessment of Albania*", Preliminary Report, Archive of Institute of Geosciences, 58 p.

Lapajne, J. K., B. Šket Motnikar, P. Zupančič [2003]. "Probabilistic Seismic Hazard Assessment Methodology for Distributed Seismicity, Bull. Seism. Soc. Am. 93, No. 6, pp. 2502-2515.

Leyendecker, E.V., A. D. Frankel, and S. C. Harmsen [2002]. "Earthquake Ground Motion for Use in Design in the United States", USGS, 2002, 19 p.

Petersen, M., A. D. Frankel and S. C. Harmsen S.C. [2008]. "Documentation for the 2008 Update of United States National Seismic Hazard Maps", Open-File Report 2008-1128, USGS, 120 p.

Sulstarova, E., S. Koçiaj [1975]. "Catalogue of Albanian earthquakes". Published by Academy of Sciences of Albania, pp. 170, (in Albanian).

Sulstarova, E., S. Koçiaj and Sh. Aliaj [1980]. "*Seismic zonation of Albania*", "Mihal Duri" Publishing House, Tiranë, pp. 297.

Zabukovec B., N. Kuka, M. B. Sostaric, B. Sket Motnikar, T. M. Suler [2007]. "OHAZ 6.0: A Computer Program for Seismic Hazard Assessment", *Joint contribution of Environmental Agency of the Republic of Slovenia and the Institute of Geosciences of Albania*, 2007.

Yule, D. E., R. E. Wahl and D. C. Wallace [1995]. "Weshake 5 Manual", Geotechnical Laboratory, U.S. Army Corps of Engineers Waterways Experiment Station, Mississippi USA.

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and Symposium in Honor of Professor I.M. Idriss
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Field and laboratory tests in S0eman deposits

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Eng. Lorena Harizaj
ALTEA&GEOSTUDIO2000" LTD

Abstract

The construction of the big energetic park near the spilling of the river Seman needed a geotechnical study to be performed by thoroughly investigations of the sand's deposits. We would like to present in these papers all the field investigations, laboratory tests and geotechnical study carried out by "ALTEA&GEOSTUDIO2000" laboratory in order to identify the behavior of soils under static and seismic loads. It is possible to create some geotechnical models based on this study and to determine their behavior under static and seismic conditions.

Introduction

The great economic development of our country, demands first of all for the energy sources development. This is why, the construction of different energetic parks is planed in Albania: the energetic park of Durres, Vlora, Seman etc near the Adriatic sea. In the spilling of the Seman's river, spreads the area planed for the construction of a big energetic park. Big gas deposits, roads infrastructure and necessary installations are part of it. "ALTEA&GEOSTUDIO2000" l.t.d laboratory carried out a full geological study, many in –situ soil tests and laboratory tests in order to determine the physical and mechanical properties of the soils in the area where the energetic park shall be built. As a result, a thorough geotechnical study was prepared which predicted some of the dangerous phenomenon that could occur in that area, characteristic phenomenon, and giving the respective recommendations.

The geological study

The geological study was carried out by drilling 12 boreholes (BH) of 30 m depth (10 BH) and 80m depth (2BH) Fig.1 (The plan of geological works)

Geomorphology

Geomorphology presents a flat zone with alluvial, maritime and marshy deposits. They have a thickness of more than 100 m in the peripheric parts and 250 m in the center of the area of study. The most characteristic geodynamic phenomenon in this area is the consolidation of the marsh deposits during a long time and the stabilization process of soils under seismic loads (these areas have M=6-6.2).

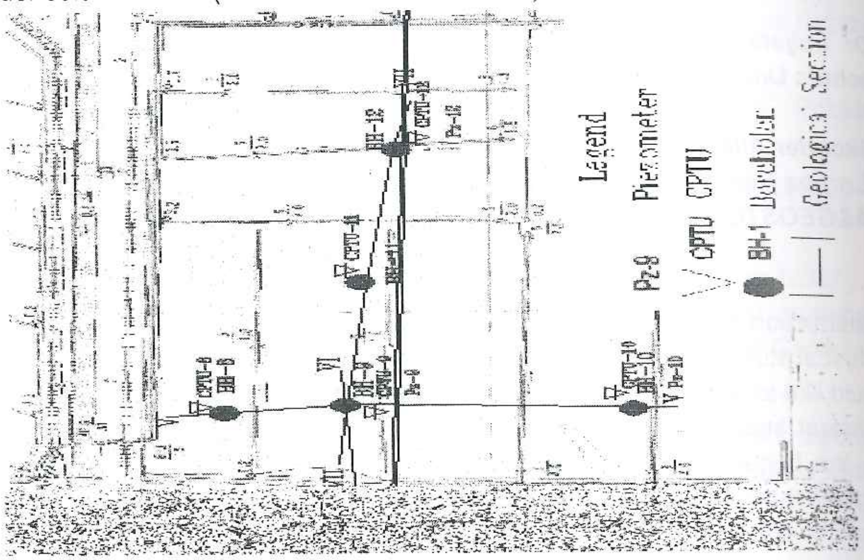


Fig. 1 Locations of the testing bore holes (BH)

Geological and hydro-geological structure

The marsh of Hoxhara, where the energetic park will be constructed, is part of the west depression of Albania where Neogen's and Quaternary's deposits are present. This area represents a deep hole of tectonic origin which during the Quaternary has been filled by swampy deposits. Marshy deposits are combined in here with maritime deposits. The Quaternary deposits are small sized gravels, sands, silty sands, silty clays, clays, peat and organic matters. Neogene's deposits consisting of Mudstone and Sandstone, weathered on top of them are met below the Quaternary deposits. According to the studies made in the marsh area of Hoxhara, it results that the level of the underground water is almost equal either in winter or in summer: (0.5÷1.5m) from the ground surface. The chemical analysis show that these are salted waters and aggressive against iron and concrete.

The encountered layers.

We have encountered about 15 layers by drilling the 12 boreholes which by further elaboration of the samples are grouped into 7 characteristic layers. During the field works, SPT and CPTU tests have been performed in the Boreholes. You can see the borehole columns in Fig. 2

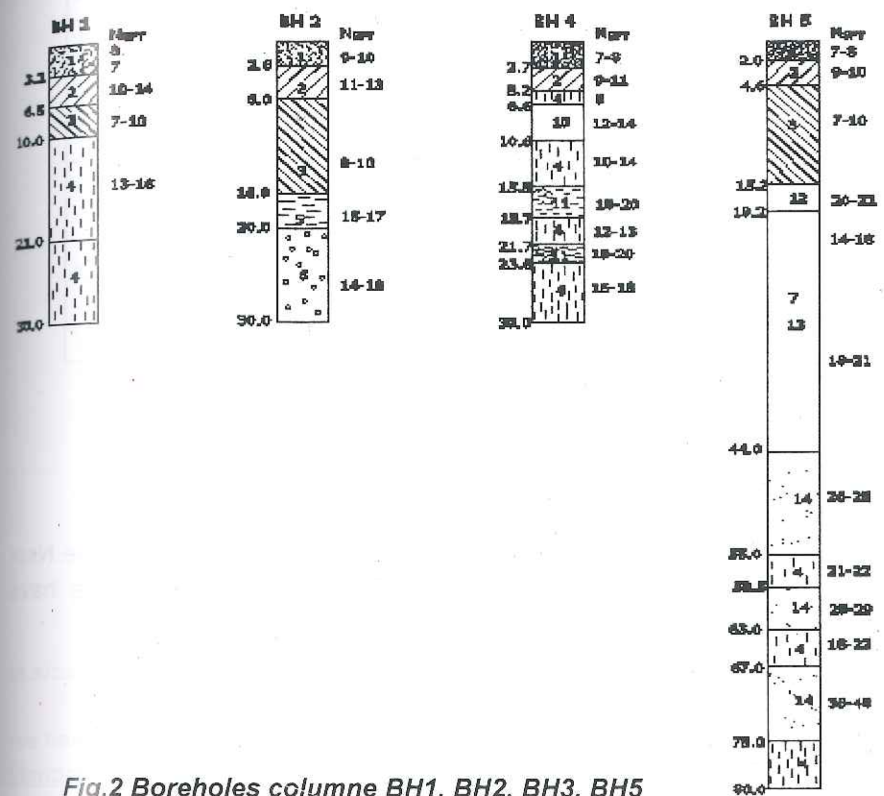


Fig.2 Boreholes columne BH1, BH2, BH3, BH5

In the same time, we have determined the geological profiles from I-I to VII-VII (see Fig. 1). Based on the geological profiles presented in Fig.3 (one of them) , we deduce that we have to deal with a very heterogeneous and complicated geology.

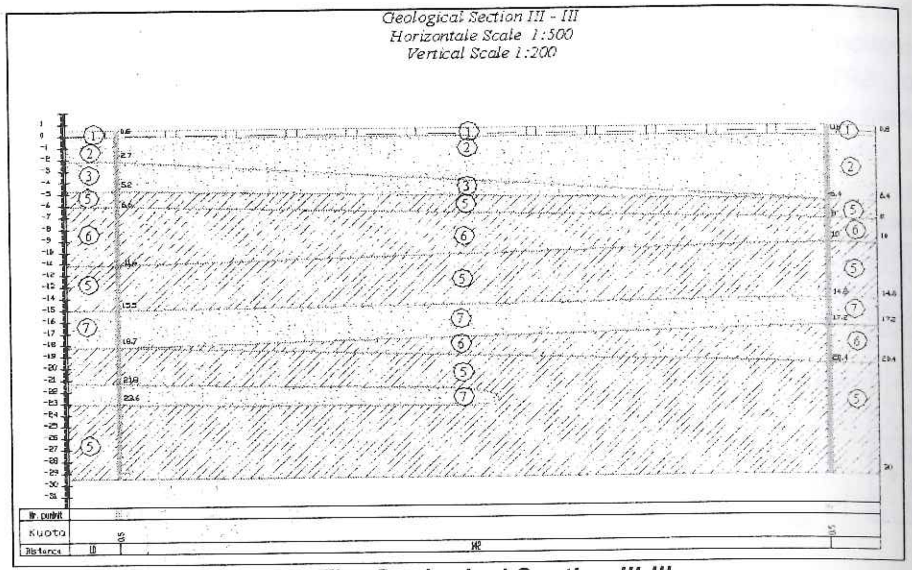


Fig. 3 The Geological Section III-III

The generalized Geotechnical Models

According to the descriptions of layers in the 12 boreholes and based also on the Nspt values, we can create four generalized geotechnical models (Fig.4) and we have evidences of the presence of the 7 following layers:

1- Loose fine beige sand containing organic matters.

2- Loose to medium dense green to gray fine to medium sand.

3- Soft green to gray silty sand + silty clay containing organic matters.

4- Soft to firm green-gray clayey silts or silty clays containing organic matters.

5- Medium dense green to gray sand + stratum of silty clay

6- Loose to medium dense silty sands to sandy silts.

7- Soft to firm green to gray silty clay +sands.

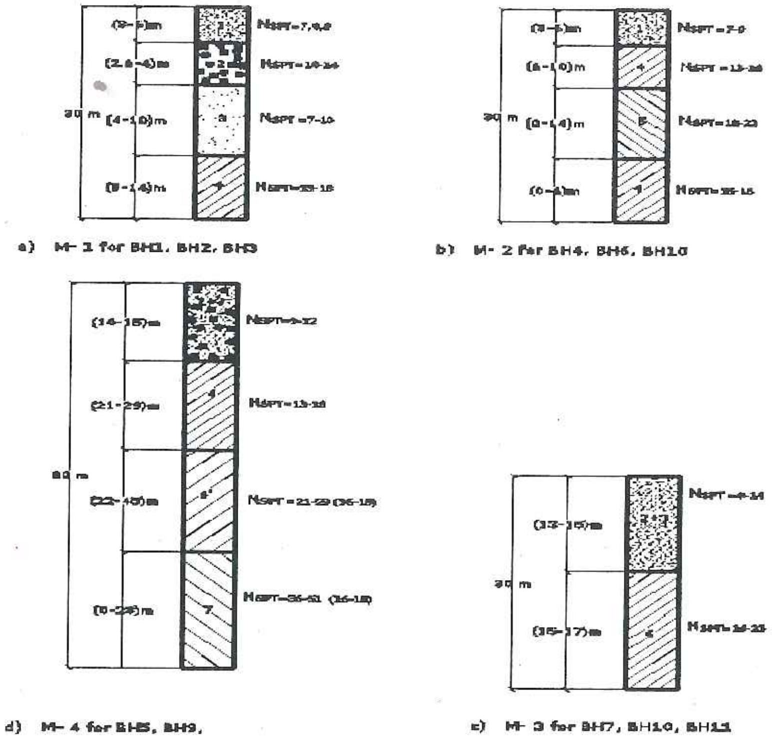


Fig. 4 Generalized Geotechnical Models

In situ tests

We have carried out three different types of tests:
Standard penetration tests SPT in all the boreholes
Based on these tests we have evidenced four generalized geotechnical models. The relation Nspt-Depth for these models is presented in Fig.5.

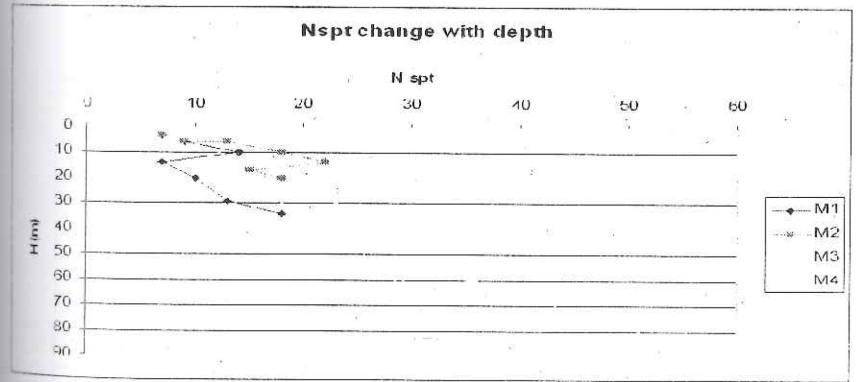


Fig. 5 The change of N_{SPT} in relation to the depth

According to the Nspt tests results, that:

- Up to 10 m of depth, Nspt=7÷14. The deposit is very loose.
- (10-20)m of depth, for the models M1 and M2, the soil properties worsen : Nspt=7÷10 while for the other models the soil properties are ameliorated : Nspt=18÷20.
- Below 20 m of depth, the soil properties for all the models are ameliorated.
- In the interval (60-75)m, Nspt=25÷43. These Nspt values testify that the soil properties are good.

<u>Measurement of Soil Resistivity on Site for 1m,3m, up to (6÷10)m of depth.</u>

Resistivity is defined as the electrical resistance of a unit volume of a material. Earth resistivity is measured by the Wenner four electrodes method, using a Megger Earth Tester (according to ASTM G 57-58 standard test method).The selected locations for performing the Soil Resistivity Tests are shown in Fig.6

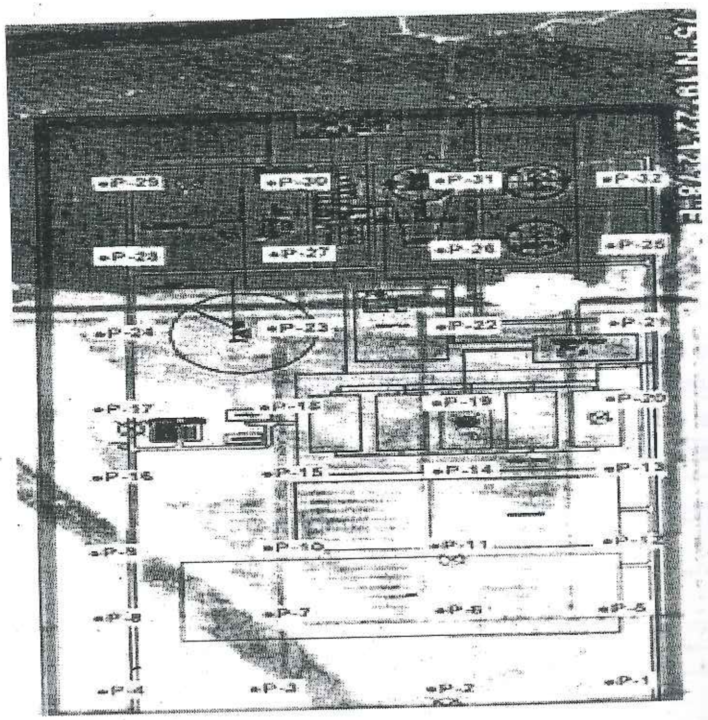


Fig 6 The 32 locations of the soil resistivity measurement

We can see the scheme of these measurements in Fig.7.

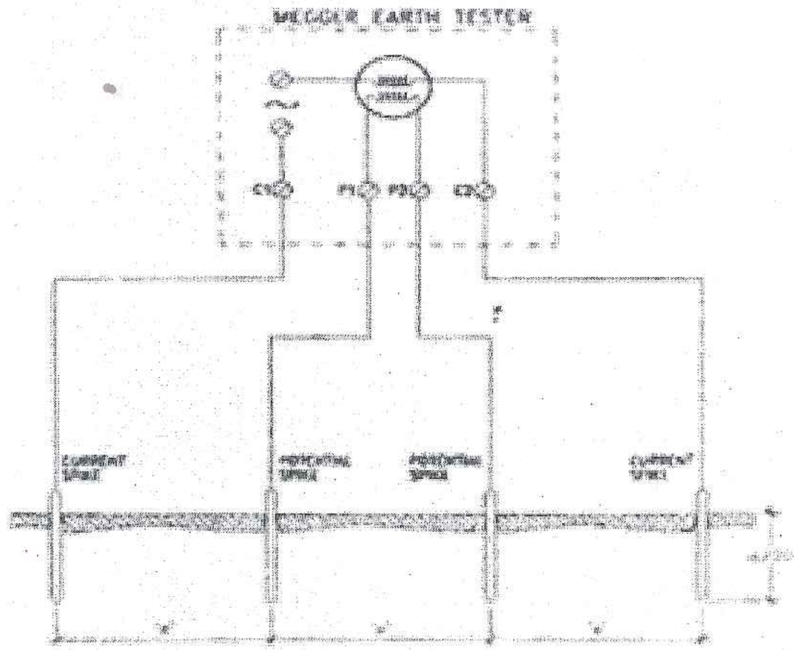


Fig. 7 Soil Sensitivity Measurement Scheme

Based on the measurements , we can calculate the electric resistivity using the formula:

$$\rho = 2\pi * a * R \text{ (ohm*m) where:}$$

a – Space between spikes
R – Resistance reading
ρ - Soil Resistivity

The test results are summarized in the following table:

Table A

Points	Electric Resistivty ρ (ohm*m)
1, 2, 3, 5, 6, 7, 11, 12	2 ÷ 3.5
4, 8, 9, 10	9 ÷ 12
13, 14, 20	1.2 ÷ 1.9
15, 16, 17, 18, 19	4 ÷ 17
21, 22, 23, 24, 25	2.5 ÷ 10
26, 27, 28, 29, 30	20 ÷ 128
31, 32	5 ÷ 33

According to these test results, the study area can be separated in three zones:

- The first zone lies between the points 26,27,28,29,30 where the soil resistivity has the biggest values. This fact is in accordance to the layers met in the boreholes BH1, BH2, BH3, BH4, BH5 and BH6 (Nspt = 7 ÷ 11).

- The second zone lies between the points 21, 22, 29, 25 where the values of soil resistivity are ρ = 2.5 ÷ 10 in accordance to the layers met in the boreholes BH3, BH9, BH10 (Nspt = 8 ÷ 14).

- The third zone lies between the points 1,2,3,4,5,6,7,11,12 where the soil resistivity has the lowest values ρ = 2 ÷ 3.5 in accordance to the layers met in the boreholes BH11, BH12 (Nspt = 13 ÷ 18).

We have also noticed a correspondence between the zones mentioned above and the four geotechnical models as following:

For the first zone are acceptable the geotechnical models M-1 and M-2; for the second zone is acceptable the geotechnical model M-3 and for the third zone is acceptable the geotechnical model M-4.

<u>Cone Penetration test CPTU</u>

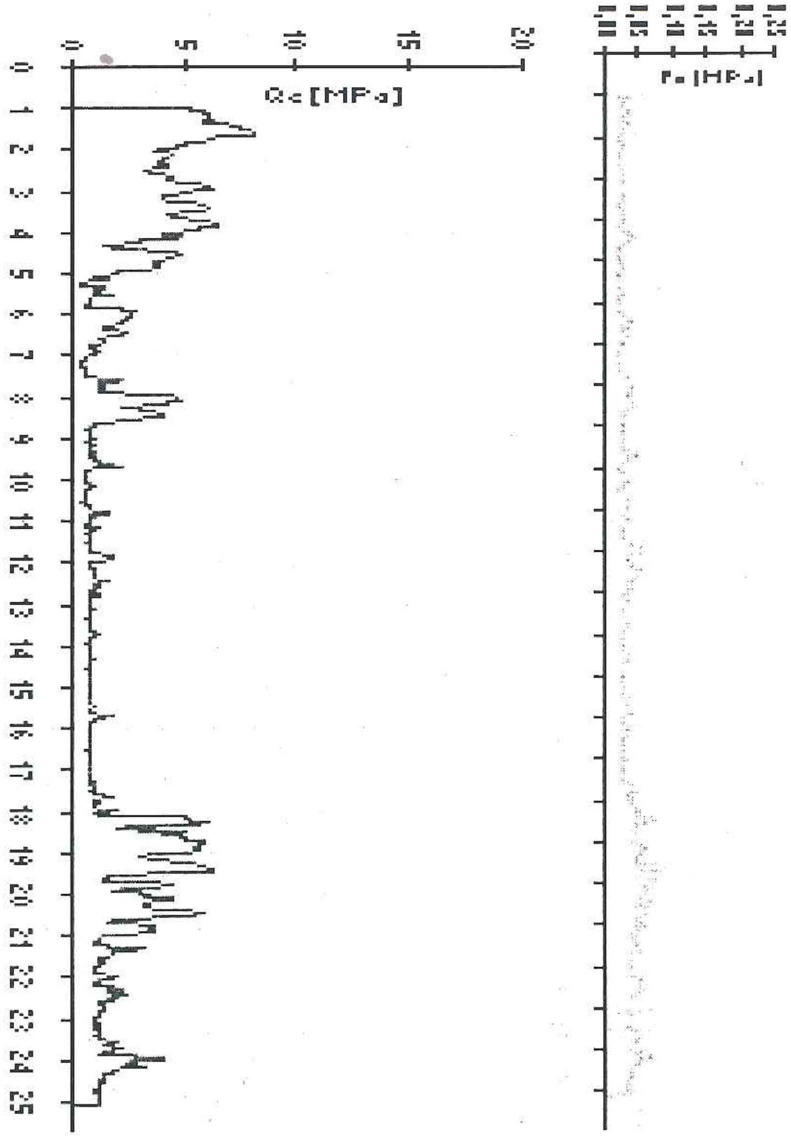
The CPTU tests are performed up to 25 m of depth in all the drilled boreholes.

q_c - Resistance in the cone apex (MPa)

f_s- Friction resistance (MPa)

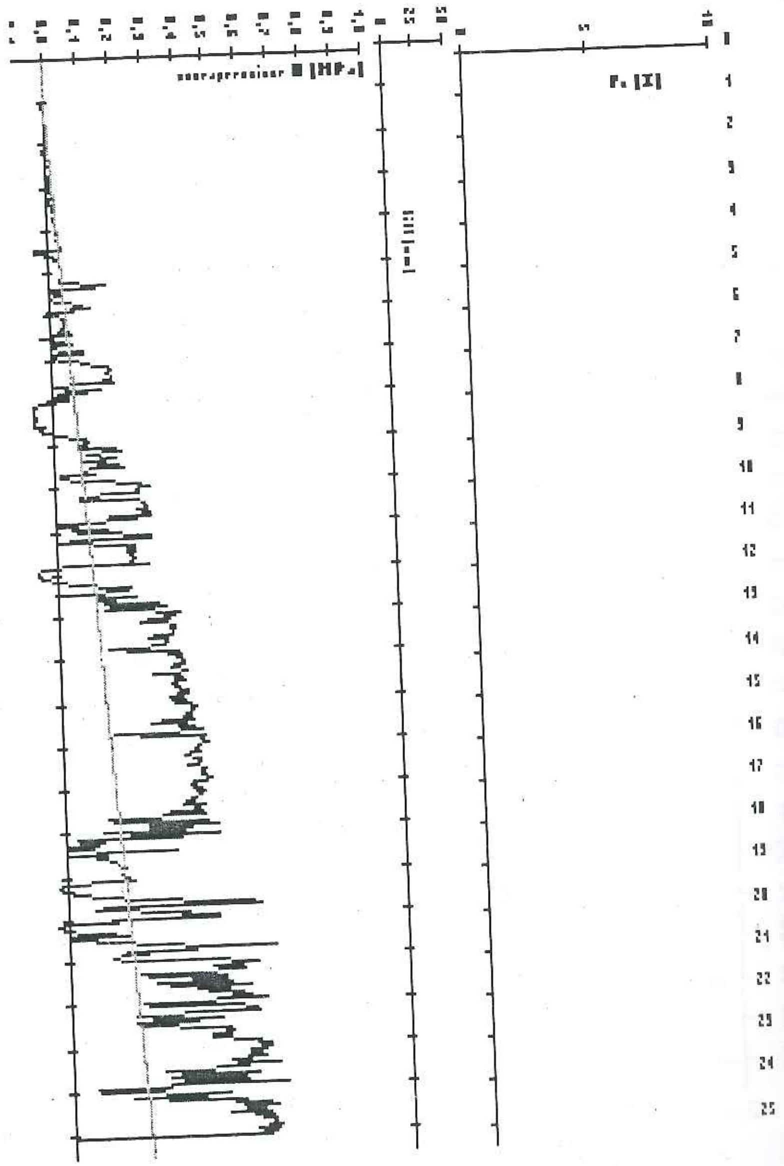
U - Pore pressure (MPa)

The test results for some bore-holes you can see in fig. 8aI,8aII.8aIII



a) Q_c-h *(b)*

Fig.8a-I CPTU test Records for the BH-1



(a) P_{ef} - H (b) P_w -H

Fig. 8a-IICPTU test Records for the BH1

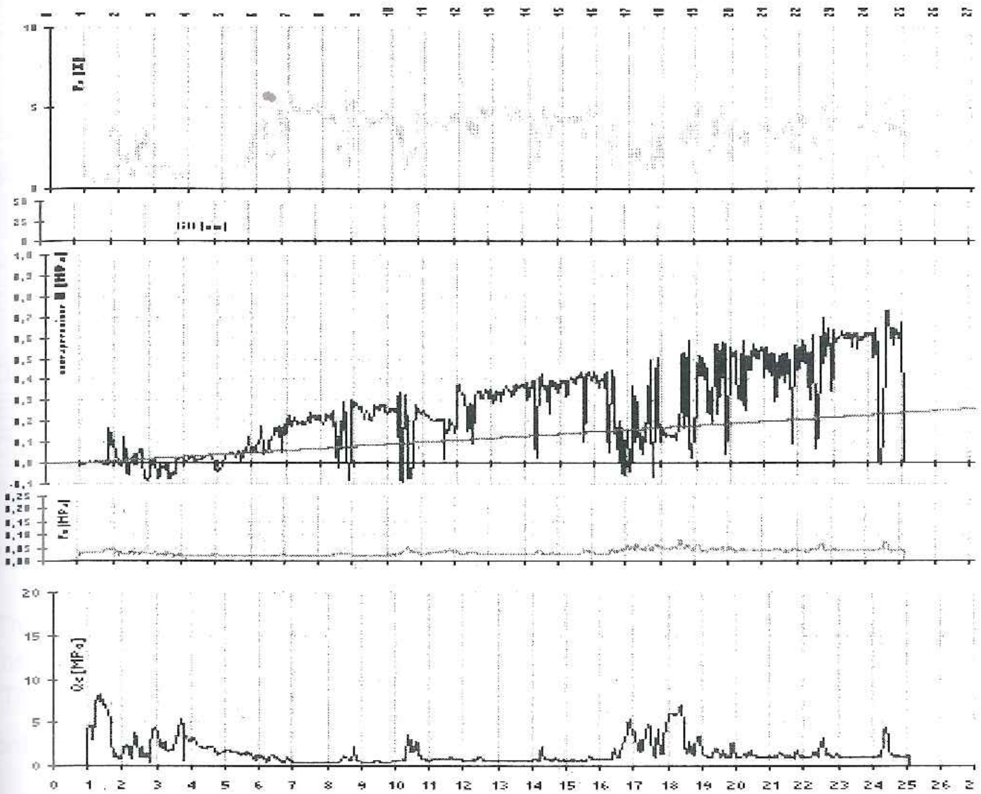


Fig.8b CPTU test records for BH-2

After the measurements we can calculate the Friction Ratio: $R_f = \frac{f_s}{q_c} * 100$

and the Indices of friction: $I_f = \frac{1}{R_f}$. According to the measurements, we can distinguish some characteristic zones with their respective depths for the geotechnical model M1 (table 1).

Table 1

Depth (m)	q_c (MPa)	f_s (MPa)	U (MPa)
3 ÷ 5	6 ÷ 8	0.02 ÷ 0.03	0.05
5 ÷ 8	1.5 ÷ 5	0.02 ÷ 0.03	0.1 ÷ 0.2
8 ÷ 17	0.08 ÷ 2	0.03 ÷ 0.04	0.3 ÷ 0.4
17 ÷ 25	6 ÷ 9	0.04 ÷ 0.08	0.7 ÷ 0.9

Based on the records taken in all the 12 boreholes we can conclude:

- The friction resistance is nearly constant in almost all the characteristic zones $f_s = (0.02 \div 0.05)$ MPa except for the interval $(15 \div 17)$ m depth in borehole BH 5 where its value is $f_s = 0.15$ MPa.
- Based on the test records and test results for the resistance q_c , pore pressure U and fratio R_f we can divide the study area in four characteristic zones (Table 2):

Table 2

Depth (m)	q_c (MPa)	U (MPa)	R_f (%)
3 ÷ 6	3 ÷ 8	-0.01 ÷ 0.01	0.6 ÷ 1.2
6 ÷ 15	1 ÷ 3	0.01 ÷ 0.04	3 ÷ 4
15 ÷ 17	(7 ÷ 9) or (7 ÷ 10)	-0.08	1 ÷ 3
17 ÷ 25	1 ÷ 3	0.06 ÷ 0.07	4 ÷ 6

Conclusions from in-situ tests

According to the Nspt, CPTU and Soil Resistivity tests' results we can conclude that:

- There is a good accordance between Nspt, CPTU tests and Electric Resistivity of the soils.
- The division of the study area in four generalized geotechnical models is relatively exact.
- There are weak, unconsolidated soils (U>0) up to (25-30)m of depth.

- The friction force for the pile foundations will be small.
- There is a good accordance between the "q_c" values and soil classification, Dr, φ, Em, Cu determined by laboratory tests.
- After the elaboration of data of in-situ tests for the geotechnical models M-1 and M-2 we have determined the mechanical properties of different layers as shown in table 3.

There is a good accordance between the three zones determined by the electric resistivity tests and the geotechnical models M-1, M-2, M-3 and M-4.

Table 3

Layer	1	2	3	4
Thickness (m)	51	4	7	14
Nspt	7-9	10-14	7-10	13-18
q_c (MPa)	4-7	1.5-5	0.08-2	3-6
f_c (MPa)	0.02-0.03	0.01-0.02	0.03-0.04	0.03-0.05
U (MPa)	0.05	0.1-0.2	0.3-0.4	0.2-0.6
R_f %	0.8-1	2-4	3-5	1-5
Clkasification	Fine sand	Fine medium sand+silty	Silty-sand	Silty clay
Dr	Loose	Loose	Very loose	-
Em (MPa)	10-20	10-20	10-20	-
Cu (KPa)	-	-	-	37-40
φ (^{0})	26	28	27	-

Laboratory tests

There was a great number of soil samples (disturbed and undisturbed) taken from all the layers encountered in 12 drilled boreholes.

The laboratory tests performed on these samples are as following:

- Grain size distribution,

- Plasticity Limits LL, PL and PI, liquidity index $I_L = \frac{w - PL}{PI}$

- Moisture content w, unit weight γ, specific gravity Gs.

- coefficient of permeability k, modulus of compression E, coefficient of consolidation Cv and shear strength of soil φ, c.

We have plotted a graph by the grain size distribution tests data (table 4, Fig.9) . Based on this graph we conclude that:

The dominant fractions for the layers of type 3,4,5,6,7 in the study area are *silt* and *fine sand* (60 ÷72)%.

Then come *fine to medium sands* (64 ÷66)% for the layers of type 1, 2.

Clay particles in the end with a percentage of <30 %.

Table 4

Layer	Particles Percentages			
	< 0.005	< 0.002	< 0.075	< 0.63
1	11.40	4.60	30.50	64.90
2	10.60	7.70	24.60	67.70
3	36.50	22.80	71.20	6.00
4	47.00	29.40	63.80	6.80
5	20.90	12.80	67.40	19.80
6	15.3	10.70	59.70	29.60
7	-	24.60	71.60	3.90

This vast presence of unconsolidated and very loose fine sands and silts may be a major cause for the liquefaction phenomenon in cases of earthquakes'.
The maximum magnitude of an earthquake expected in the study area is M = 6 ÷ 6.2.

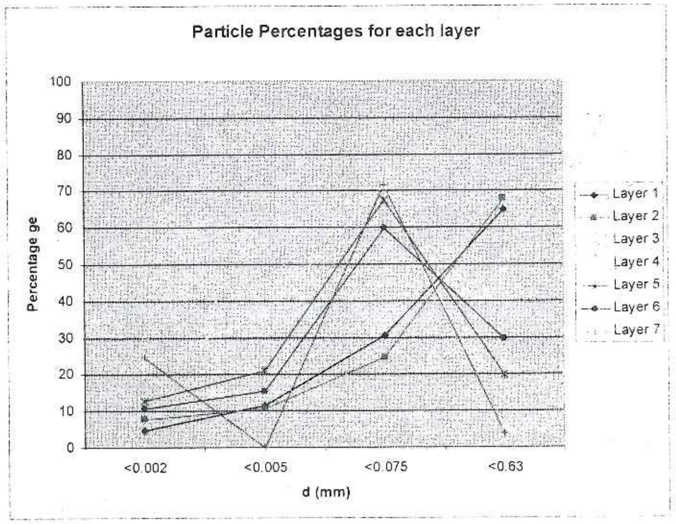


Fig. 9 The percentage of the particles for the seven layers

The above mentioned layers are classified after UCS as shown in table 5:

Table 5

Layer	LL (%)	PI	Passing No. 200 Sieve	Classification
1	-	-	30.50	SM
2	-	-	24.60	SM
3	43.60	22.60	71.20	CL
4	33.46	15.56	63.80	ML
5	28.00	14.45	67.40	ML
6	23.71	8.94	59.70	CL-ML
7	39.71	20.01	71.60	CL

<u>The physical conditions</u>
(porosity "e", density "Dr" and degree of saturation S, Liquidity Index IL)

The physical properties of the layers determined from laboratory tests are summarized in table 6.

Table 6

Type of Layer							
	1	2	3	4	5	6	7
Nspt	7 ÷9	10÷ 14	7 ÷10	14 ÷15	8 ÷12	16 ÷18	16 ÷18
W (%)	20.63	22.11	38.98	35.52	25.18	21.38	32.77
LL (%)	-	-	43.63	33.46	27.77	23.71	39.17
PI	-	-	22.59	15.56	14.45	8.94	20.01
IL	-	-	0.79	1.13	0.82	0.74	0.68
e	0.650	0.695	0.880	0.950	0.910	0.643	1.094
S	0.85	0.82	Plastic liquid	Plastic liquid	Plastic liquid	Plastic soft	Plastic soft
Dr (%)	30	40	-	-	-	-	-

Based on the tests results we conclude that we have to deal with weak soils with low bearing capacity which calculated after the classical method resulted [σ] = (100 ÷180) KPa.
The graphical presentation of the relation between the bearing capacity of soils and depth for the geotechnical model M-1 is shown in Fig. 10

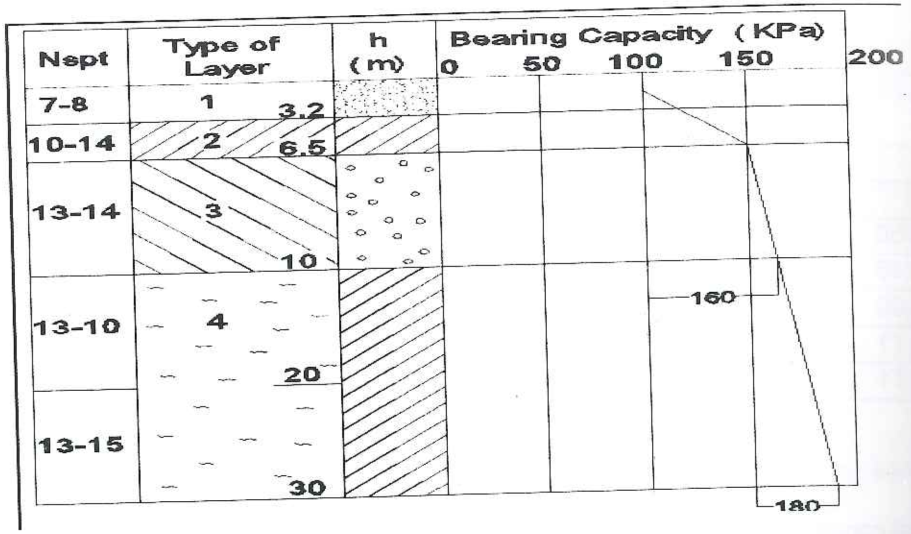


Fig. 10 Change of the soil bearing capacity

The results from the oedometric and permeability tests are given in table 7.

Table 7

Layer	e_0	$Cv^* 10^{-2}$ (m^2 / sec)	Eo (KPa)	K (cm/sec)
1	0.650	-	$1.05*10^4$	9.92
2	0.695	-	$1.15*10^4$	4.01
3	0.880	9.6	$0.5*10^4$	$6.89*10^{-2}$
4	0.950	38.6	$0.6*10^4$	$4.63*10^{-2}$
5	0.910	-	$0.7*10^4$	$3.15*10^{-3}$
6	0.643	-	$0.8*10^4$	$2.95*10^{-3}$
7	1.094	16.9	$0.89*10^4$	$9.8*10^{-3}$

Based on that we conclude that the layers 3, 5, 6 are very compressible which will cause important settlements to the objects that will be constructed there.
So, if we analyze the geotechnical model M-1 we can calculate the settlements of the foundation of a gas deposit (diameter =30m, depth =2 m, transmitted pressure P= 100 KPa) see Fig. 11.

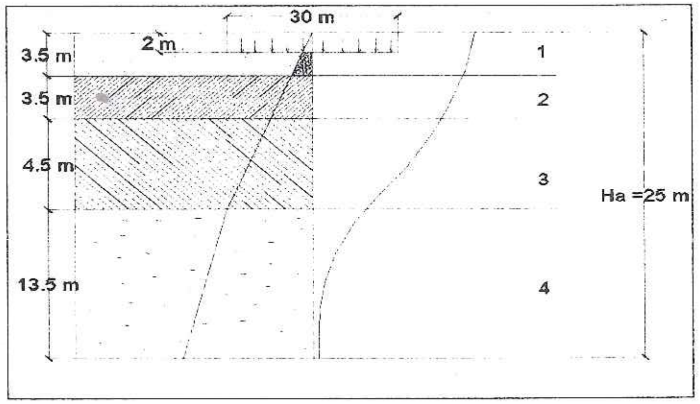


Fig. 11 Active zone under gases deposits foundation located in the model M-1

Results: Active zone Ha= 25 m; The settlements are S= (12-15) cmThe settlement values may exceed the limit values.
The large presence of sand and silt particles in the layers determines a high value of the coefficient of permeability: Kmed= 0.059cm/sec, and as a result a short time of the primary consolidation(a few days)

<u>The results of direct shear tests</u>

Internal friction angle φ and cohesion c are summarized in table 8. By using this data we can calculate the bearing capacity "R" after the "Limit state" theory.
According to the "Limit state theory," and by considering safety factor Fs= 1.5 ÷2, the above layers appear to have low bearing capacity. (R_1 and R2 are the allowable bearing capacities respectively for Fs =1.5 and Fs=2; $R_1 = R/1.5$, $R_2 = R/2$)

Table 8

Type of Layer	φ (°)	c (KPa)	R (KPa)	R_1 (KPa)	R_2 (KPa)
1	26	-	192	128	96
2	28	-	219	146	110
3	23	7	203	135	101
4	20	11	194	129	97
5	24	5	202	135	101
6	22	4	174	116	87
7	19	32	299	200	150

The behavior of the basement under static and dynamic loads

The data collected from geological, geotechnical and seismological studies permit us to predict the future behavior of soils under static and seismic loads.

Static loads

In this case we have to say attention to:

- Low bearing capacity of shallow foundations
- Low bearing capacity of deep foundations (pile foundations) because of the small friction force of the different layers (the layer of fine sand, silty sand or silty clay in plastic liquid conditions).
- Enormous settlements of the basement because the layers are loose, unconsolidated and very compressible.

These settlements can cause ultimate limit state or service limit state in the construction.

- The time of primary consolidation will be longer than normally (several months) because of the high pore pressure resulted from CPTU tests.

Dynamic loads

Dynamic loads from possible earthquakes can cause dangerous situations such as:

-Possibility of liquefaction because:
- the thicknesses of loose deposits are enormous,
- the deposits are mostly fine sands and soft silts or silty sands.
- the density of deposits is Dr<50%
- the deposits are under the groundwater table.

In these conditions we must determine: the maximum acceleration "a_{max}" =n*g on the ground surface, the possibilities of liquefaction and the potential of a possible liquefaction.

- If the liquefaction phenomenon has big chances to happen, then before the construction begins the area must be improved by using gravel piles which serve as vertical drainage or by using a combined foundation slab with piles.
- In order to construct safe foundations we must evaluate the reduction of the bearing capacity of the soil because of the PGA.

Conclusions and recommendations

The main deposits in the construction area of the energetic park are maritime, marshy and alluvion.Q_{4kt} , Q_{4dt} , Q_4, represented by silty sands, sands and slimy clays, peaty clays and gravel from slightly consolidated to normally consolidated. Below them , there are Neogene's deposits composed by mudstone, sandstone and conglomerates.

The thickness of the deposits varies from 80m to 100 m. The groundwater table is (0.5÷1.5)m from the ground surface. The water is aggressive to concrete and iron.The bearing capacity of the layers is low or very low.

The geological sections show that the geological composition is very complicated and heterogeneous, nevertheless we can use for calculations four generalized geotechnical models. We distinguished seven layers in these models.

The in-situ and laboratory tests results confirmed our four geotechnical models.

The constructed buildings in this area will have considerable settlements under static loads and as a consequence the may appear the ultimate limit state ore the service limit state.

Under seismic loads, the deposits of this area can be liquefied, can loose stability and undergo to supplement enormous settlements. This is why is necessary a seismic study on

The behavior of the four geotechnical models and hereby evaluating the soil-structure interaction. The soils of this area belong to E classification category after EC- 8.

In order to assure the safety of the buildings that will be built in this area we have to make three provisions:

- the improvement of the building area by injecting silica gels in the ground, by explosion in order to obtain soils more compacted with Dr>50.
- By using vertical and horizontal drainages in order to eliminate premises for the liquefaction phenomenon.
- By using combined foundation: mat or slab with piles.

REFERENCES:

ASTM standards (2003)

Geological and geotechnical study for Fier - Seman Road (2000) - "ALTEA&GEOSTUDIO2000"

Geological and geotechnical study for PLR Petrolium Fier Terminal- Seman (2001) - "ALTEA&GEOSTUDIO2000"

Geological and geotechnical study for Fier-Vlore Road (2004)- "ALTEA&GEOSTUDIO2000"

Beniawski Z.T.(1989) : Engineering Rock Mass Classification

Bozo L.(2007) Geotechnics I (soil Mechanics) 455 p Tirana shtypshkronja Natyra.

Bozo L. (2008) Geotechnics II (Foundations) 590p Tirana shtypshkronja Natyra.

British Standards (1990,BS 1377).

British Standards (1999): Code of Practice for site investigations (BS 5930-1999)

Powrie W. (1986) Soil Mechanics, concepts and applications. University of Southampton, Hinfield Southampton 50171BJE & SPON London.

International Company of Sabalan Eco-engineering Research (ICSER)

World Association of Soil and Water Conservation (WASWC)

International Union of Forest Research Organizations (IUFRO)

European Geosciences Union

European Society for Soil Conservation

Iranian National Retrofitting Center, North-West Branch

IAU-Science and Research Ahvaz

First International Conference of Soil and Roots Engineering Relationship (LANDCON1005)

24-26 May 2010 Ardebil, Iran

Polytechnic University of Tirana, Faculty of Civil Engineering, Department of Environmental Engineering, Tirana - Albania

Ardebil, 10 March, 2010

Subject : Acceptance of the paper and Invitation for Oltion Marko to attend LANDCON 1005, May 24-26, 2010, Ardebil, Iran

On behalf of the Organizing Committee of the First International Conference of Soil and Roots Engineering Relationship (LANDCON 1005) that will be held at the Ardebil, Iran from 24-26 May 2010, we are pleased to extend and invitation to participate in the above conference being organized by the International Company of Sabalan Eco-engineering Research (ICSER).

Dear author has submitted the paper entitled "ASSESSMENT OF SOIL EROSION IN A MOUNTAIN AREA IN "THE KORCA DISTRICT"-ALBANIA" which has been accepted for publication in the proceedings of LANDCON 1005 and is invited to present the paper in a technical session scheduled during the Coference.

Thank you for your assistance in the support to our conference.

Sincerely,

Dr. Ghassem Habibi Bibalani
Chairman and head of scientific editors of conference LANDCON1005

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Assesment of Soil Erosion in a Mountain Area in the Korca District, Albania.

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Introduction

Republic of Albania is part of the Southeast Europe, western part of the Balkan Peninsula. Albania is in bor-der with Greece in south and southeast, with FYR Macedonia in east, with Kosovo in north and northeast and with Montenegro in north and northwest. Adriatic Sea and Ionian Sea surround the west part of Albania.
Albania based on different studies result as one the countries with high quantitative level of soil erosion with-in the Europe and elsewhere. The amount of soil eroded from the surface water erosion is from 20 – 40 t/ha/year and in extremely cases result up to100 t/ha/year from the deepest erosion.
A zone more affected by erosion phenomena, in our country is Korça district. Seeing the actual situation and outspoken problems, we think to take into consideration on our study, the Vithkuq Commune, which is under administration of the Korça Forest Service Directory.
This study has had as his main aim to determination the erosion degree on the forest area, according to the scale of forest vegetation, sloping and the rainfall indicators. Those will serve as special information on the land use by the farmers, commune, and state, depending in the property on the forest and pasture area.

Method and material used

The methodology developed by splits the factors affecting erosion into two main groups: (1) external and (2) internal ones.
External factors include all those variables that contribute to create the surrounding environment, extending their influence to a more or less large area and creating a situation that remains rather un-changed at the time scale of the forest management decision-making process.

Internal factors, on the contrary, are related to forest and pasture stands and subject to major changes due to stand development, species succession and man-gement practices. They can be modified by sylvi-cultural treatments in order to create conditions that favour the performance of the forest with respect to erosion.

When erosion phenomena occur, they can be local-ised and mapped as "point" information: a map of points can be created showing all the places affected by different signs of erosion.
Thus, the introduction of the "area unit" concept is needed; a common ground for all considered factors on which the results of the overall evaluation will take place.
Data sources, expert opinions, computational power needs and literature browsing are all methods and information sources considered to select a minimum "area unit" which is the forest or pasture parcel. The degree of severity of the climatic factor is evaluated according to the following equation:

$$I_R = \frac{R_v . R_k}{10000}$$

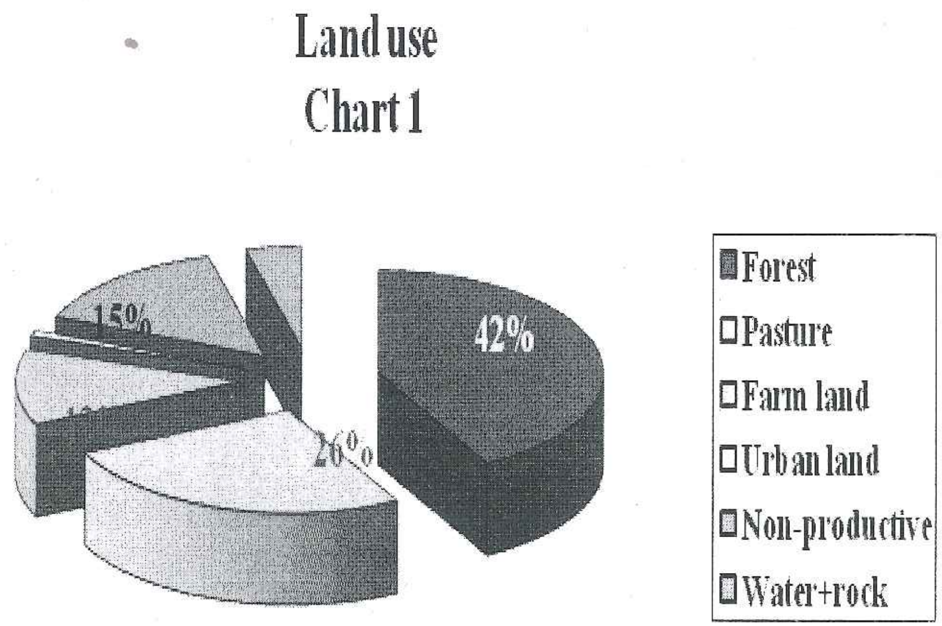
where, Ir – rainfall indicator, Rv – amount of the average annual rainfall in years, Rk – amount or rainfall in critical periods, 10.000 – reduction coefficent

Results and discussions

The total land surface is 22392 ha, devided into the following eight water-collecting basins :

1. Panarit water-collecting basin 3670 ha;
2. Shera water-collecting basin 2756 ha;
3. Rungaja water-collecting basin 2766 ha;
4. Katundi water-collecting basin 3426 ha;
5. Vithkuqi water-collecting basin 2390 ha;
6. Lubonja water-collecting basin 3168 ha;
7. Gjanci water-collecting basin 3794 ha;
8. Roshanj water-collecting basin 422 ha.

Land use is represented in details in chart 1.



He vegetation in this study area is part of the *Fagetum, Picetum and Alpinetum* phyto climatic area, upper border. The biologic diversity represents about 142 species, which are distributions in 45 family.

The forest vegetation mainly consists of the following types: *Fagus sylvatica, Pinus nigra, Abies alba is* artificial forestation,, *Carpinus orientalis,etc.* Chart 2 show that the larges surface of the water-collecting basin has a small coverage degree, which favours the development of the erosion dynamics.

Taking into consideration the decay of vegetation in this area, the effect of mountain side slope will be greater. The mountain side slope varies from average to high in this area. This is shown in figure 1.

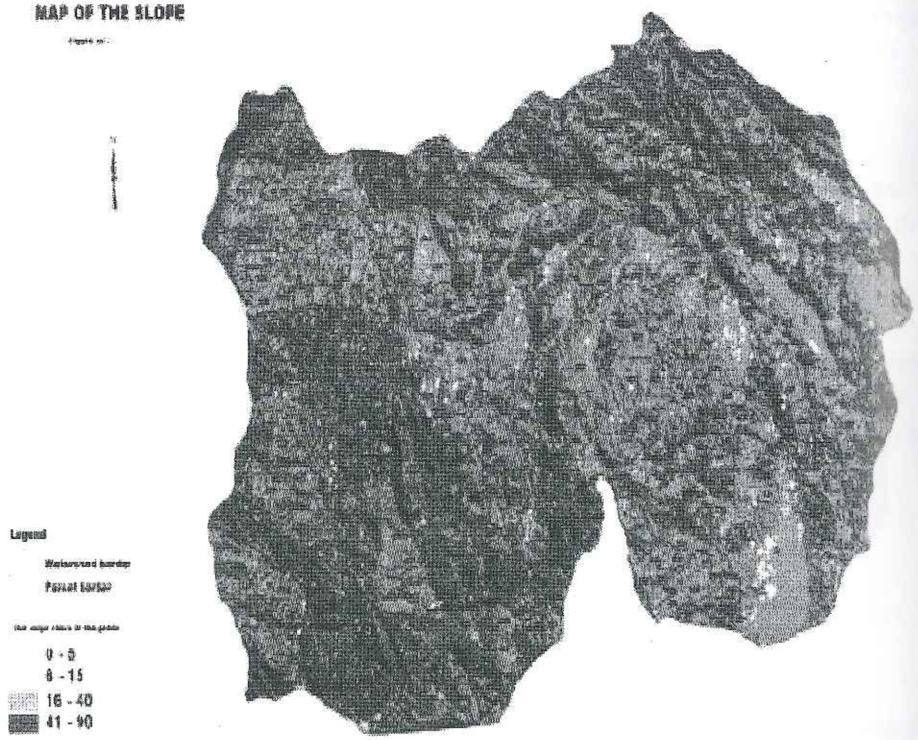
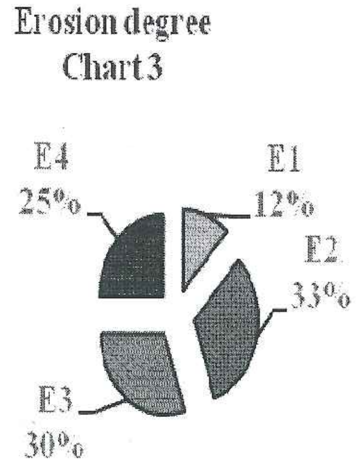


Figure 1. The slope class

The degree of erosion was devided in 4 levels. This is shown in chart 3 and figure 2.



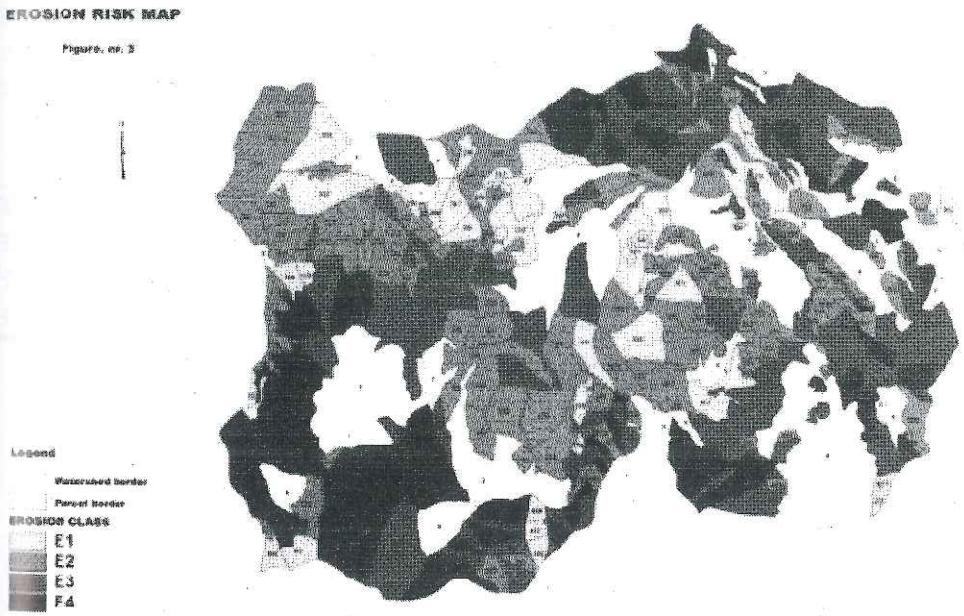
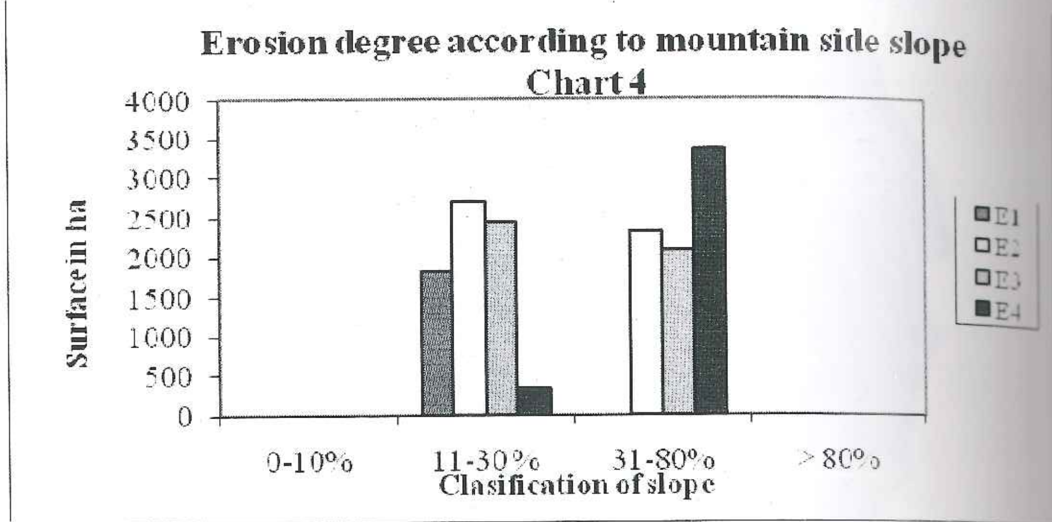
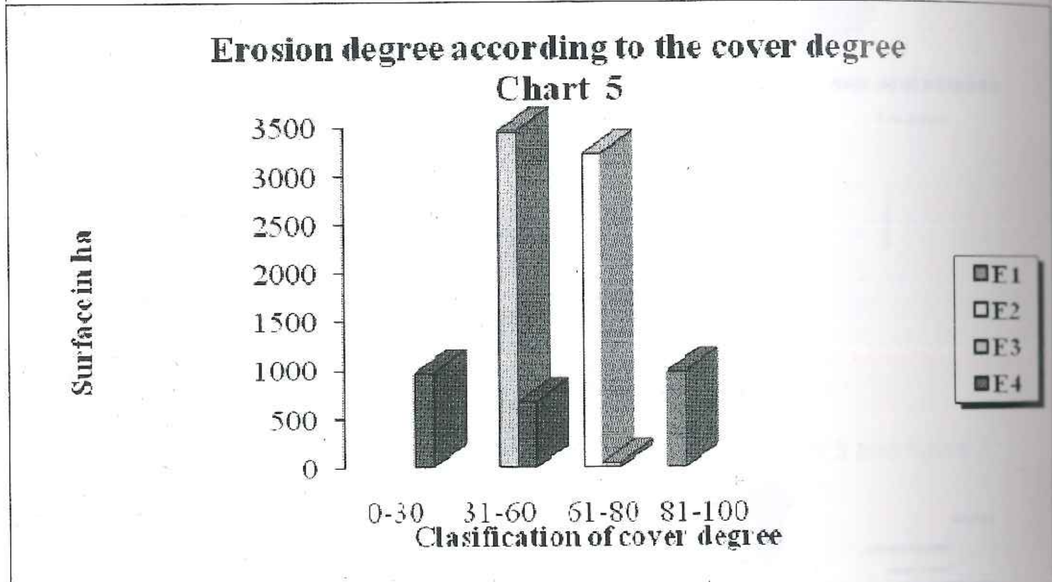


Figure 2. Class of erosion

The erosion problem in this area is quite high. The vastest surface is part of the third and forth level of erosion.

Chart 4 and 5 show the erosion according to the degree of mountain side slope and vegetation cover land.





According to the rainfall indicator the erosion risk varies, from the altitude 800- 1000 m above sea lev-el, 2nd class of erosion, from the altitude 1000-1600 m above sea level, 3rd class of erosion, >1600 m above sea level 4th class of erosion.

Findings

There is a surface of 9352 ha of forest, 5807 ha of pasture, 2661 ha agriculture land, 145 ha urban land, 3528 ha non-productive land and 899 ha water and rock land.

The dynamics and the present state of erosion are critical:

- 1835 ha or 12% of the forest and pasture land are part of the 1st class of erosion,
- 5036 ha or 33%, are part of the 2nd class of erosion,
- 4540 ha or 30 % are part of the 3rd class of erosion,
- 3748 ha or 25 %; are part of the 4th class of erosion,

Erosion risk is estimated according to the calculations of the rainfall indicators and falls into three categories, the secon, third and forth.

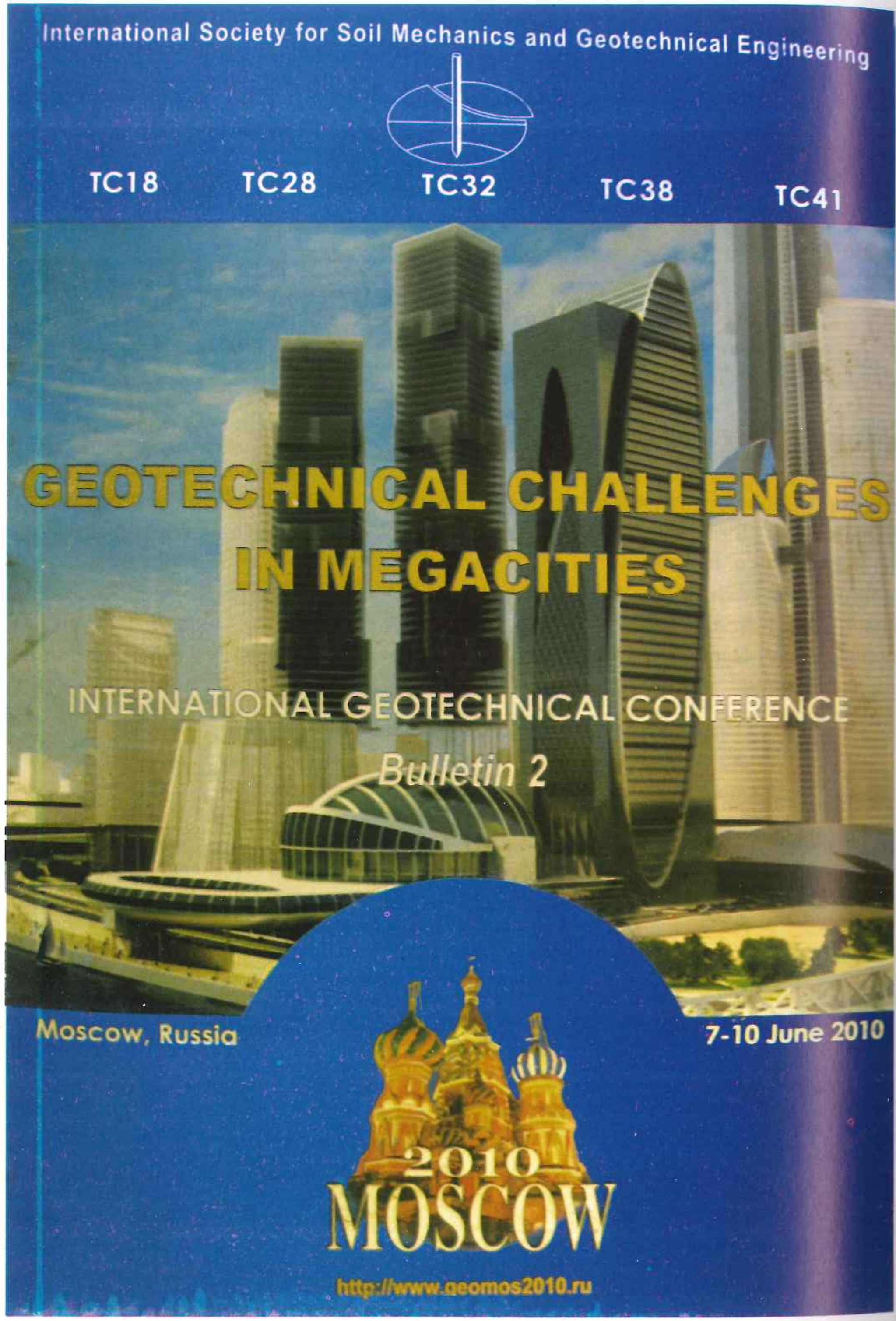
REFERENCES

ANFI 2003, Albania Natonal Forest Inventory.

Gatzojannis S, Stefanidis, 2001. An inventory and evaluation methodology for non-timber functions of forests.

Marko O, 2006, Ph.D. Assessment of erosion risk, for forests and pastures areas in Vithkuq area, Korca –District.

Papanastasi V. Dynamic of land use changes and their conflicts in the Mediterranean type ecosystems.



Preservation of historical buildings in Albania.

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Abstract

In Albania exist a lot of old churches constructed in the XV-XVI century. They are very interesting by their construction and marvelous paintings. In many of them have appeared damages which sometimes have created serious situation. In many cases the damages have geotechnical origins which are connected with deformable and resistant characteristic of soils. In this paper we would like to present the damage analysis of the Ardenica's monastery near Lushnja city and Saint Maria church near Permet city. Also we wont te present the geotechnical investigation that we made in these areas, to discover the main cause of the damage and the engineering measures taken for the rehabilitation of the objects.

Introduction

Albania is a small country with 28000km^2 area and about 4 million habitants. Albania is situated in south west part of Balkan and in littoral of the Jonian and Adriatic seas. Albania has a lot of cultural wealth and it is one of the most ancient population in Balkan and in Europe. A part of this cultural wealth is expressed in the cult objects. We have about 500 churches which are generally small but astonishing from their art which are stone-cutting, wood engraving and chiseling. Also the used construction techniques were particular and the mural paintings are miraculous. The longevity of many churches is tied with the place where they are constructed as in the dense forests, hills, mountains which were protected from different occupations. Also it is tied with the quality of used materials and best building techniques. In the following part we would like to present the work realized for the reconstruction of two cultural monuments in Albania. This work would be impossible without the research activity to discover the causes of the damages and considering soil-structure interaction.

Monastery of Ardenica

Ardenica's Monastery is one of the most beautiful and known in Albania. It has picturesque view and nice relaxing landscapes. It has a dominant position in Myzeqe.

Location of Ardenica monastery.

It is located at the top of hills zone in Myzeqe (central part of Albania) and in the middle of the pine forest. The monastery is confined in the west side by Libofsha field and in the east side by Kolonja field.(fig.1)

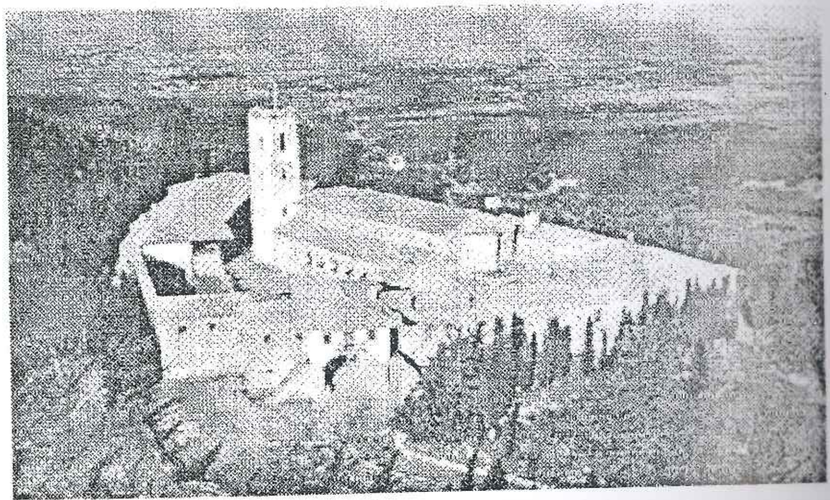


Fig.1 View of Ardenica's monastery from above.

Construction time.

Ardenica's monastery was constructed in 3 phases: the first phase was around year 1450, the second phase was from year 1740 to 1750, when was constructed the church of Saint Maria with the famous painting of Zografi brothers (fig.2), the third phase was the construction of the bell tower in 1925

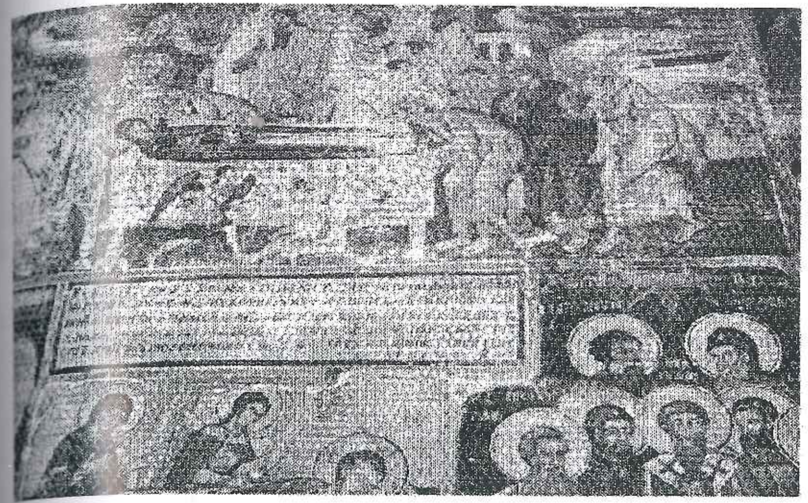


Fig.2 Zografi brothers' paintings

Fig.3 Bell tower

Construction type.

The church, monastery, library and other annexes are the combination of stone construction (the foundations, walls, columns) with wood construction (inner part, some columns, ceiling, iconastas ect).The monastery is a two floor building and it is surrounded by high retaining walls.

The plan and the damages that have appeared.

The monastery is deployed in a triangular form at the hill top. It went through many restorations but during 1990-1997 many serious damages appeared as split walls in the first floor, serious deformation of the stone walls as result of their bending and the destruction of some transversal support elements of the retaining walls. (fig.4)
We think that all damages are tied with geotechnical phenomena's as partial slide of soils, equilibrium dissolution in some zones by earthuqkes, different settlements ect.

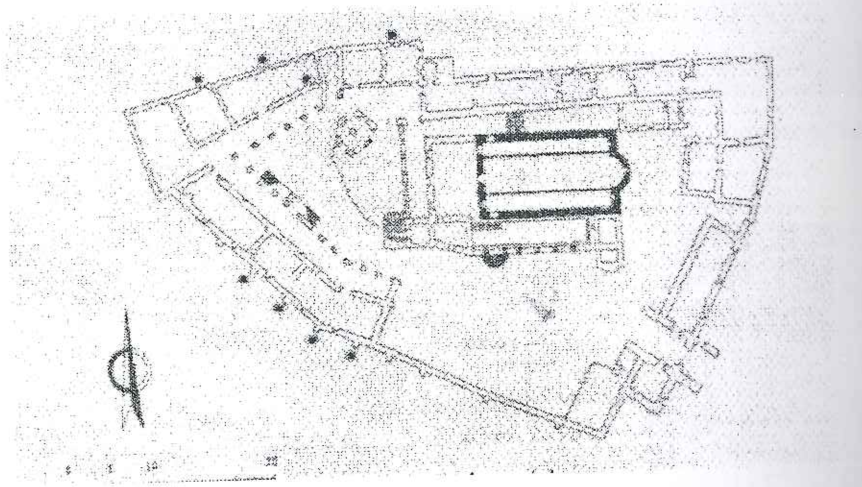


Fig.4 The plan and places of the damages.

The completed study.
To find the damage causes we made a complex study: geological, geomorphologic, seismic, historical and geotechnical.
The geological situation is that, we have Neogene rocks N_{2r} thick from 100-1000m. The geological situation is that, we have Neogene rocks composed by alevrolite, conglomerates and sandstones. Over them are situated eluvions and deluvions. The monastery with 2500m^2 are is leaned on the layers with different deformable and resistant characteristics(fig.5).

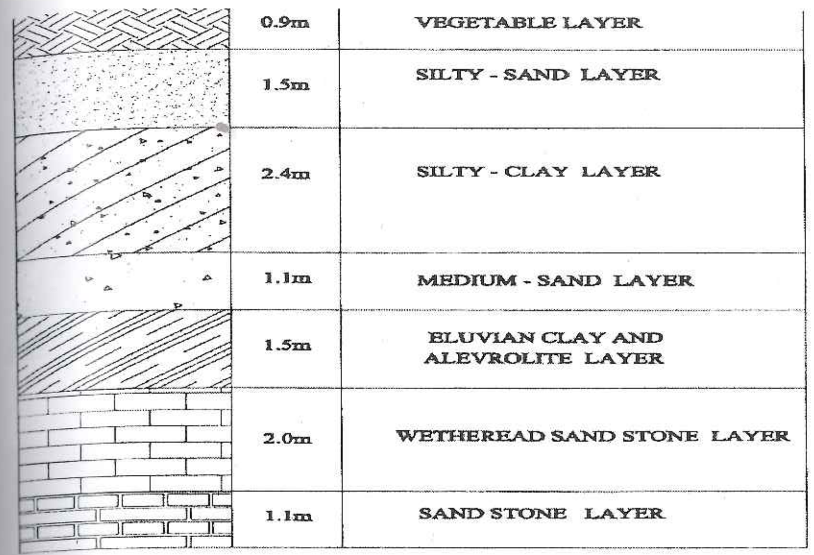


Fig.5 Section from bore-holes 9.

From the geomorphologic point of view the object was build in hills zone and the geological formation which is much altered can show volume changes of soils, change of the mechanical characteristics φ, C, E. We can find this phenomena's until 9m depth. From the seismically point of view this zone has a high seismic activity (M=6-6,4). During the last 50 years in (1959, 1962, 1982) happened three strong earthquakes with time length 20-40 seconds. The cyclic loading-unloading has influenced in the diminution of the resistant characteristic of the soils φ,C and in the development of the new active forces in the slide planes, which have the same orientation as the slope. From the geotechnical point of view the behaviour of the soils is different because there are two layers with different characteristics determined by bore-holes and laboratory tests. In the central part we find weathered rock with e=0,39, E=2.9.10^{4}KPa, R=400KPa, while in the peripheral part we find the eluvial and deluvial deposits with e=0.85, E=(0.7 to 0.8).10^{4}KPa, R=200KPa. In some zones where appered fissures and deformations in the stone walls, we find weak soils with E$\leq$ 0,4.10^{4}KPa, R$\leq$200KPa. In the zone where the destruction of the transversal support elements of the retaining walls happened the soils have low resistant value φ=18^0 , C=3 to 7KPa, while in the central parts this values are changed to φ=28^0 , C=100KPa. From historical point of view we notice that during the 550years of longevity of this object, except all factors mentioned above and climatic, biologic factors there is even the material ageing that influences in the fissures that appeared in the construction. Finally from the study results that from the geotechnical phenomena's as non uniform settlements, slide, swelling, softening, the new horizontal and vertical displacements by earthquakes had been the causes for further damages in the construction of this object.

2.6 Recommendation of the engineering measures for stabilizing the situation.
Noticing that, all damages appeared in the peripheral zone, near the slope or at the top of the hill we recommended.:
To stop the superficial sliding must be done a forest planting of the zone. In the deluvial zone which is in a limit equilibrium state must be prohibited every building construction. The construction of the new transversal support elements of the retaining walls which will rest in the second layer with the following characteristics $\varphi=20^0$, C= 23KPa , $E=1.12.10^4$, R=250KPa, e=0,8. The damaged walls must be reinforced with the combination of concrete and metallic elements. In the deformable zone must be reinforced the basement by jet ground cement.

Church of saint maria in Leus Permet.

The st.Maria church rises in the middle of a dense but beautiful forest. It has a dominant position over Vjosa valley. It's a place where poets were inspired by the marvelous nature that surrounds it.

Location of Saint Maria church.

The church is located in the south-east part of Permet city (south-west part of Albania). It is constructed in a small hill rounded by dense forest and with a rich water source. (fig.6). It is famous not only for the construction but also for the mural paintings and iconostasis which is the most beautiful in the Balkan region.

Fig.6a View of St.Maria church

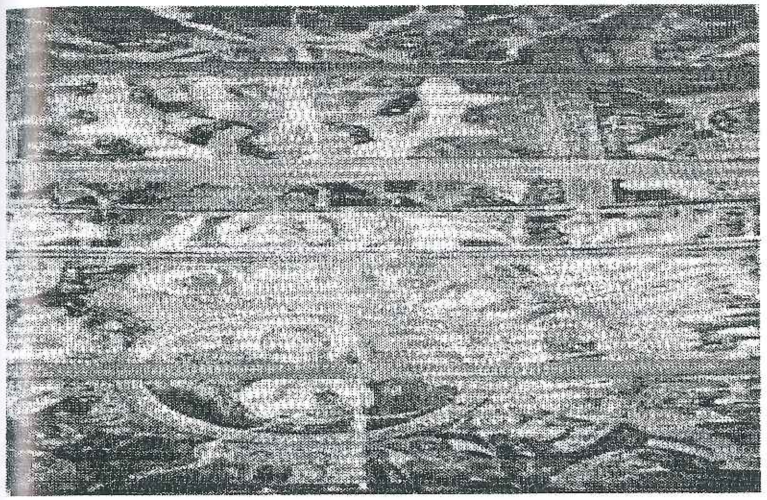


Fig.6b View of the churches mural paintings

Construction time and type

The St.Maria church was build in 1810-1812 and its construction is a basilica with cupola, structure type T_3V_1 (fig.7)

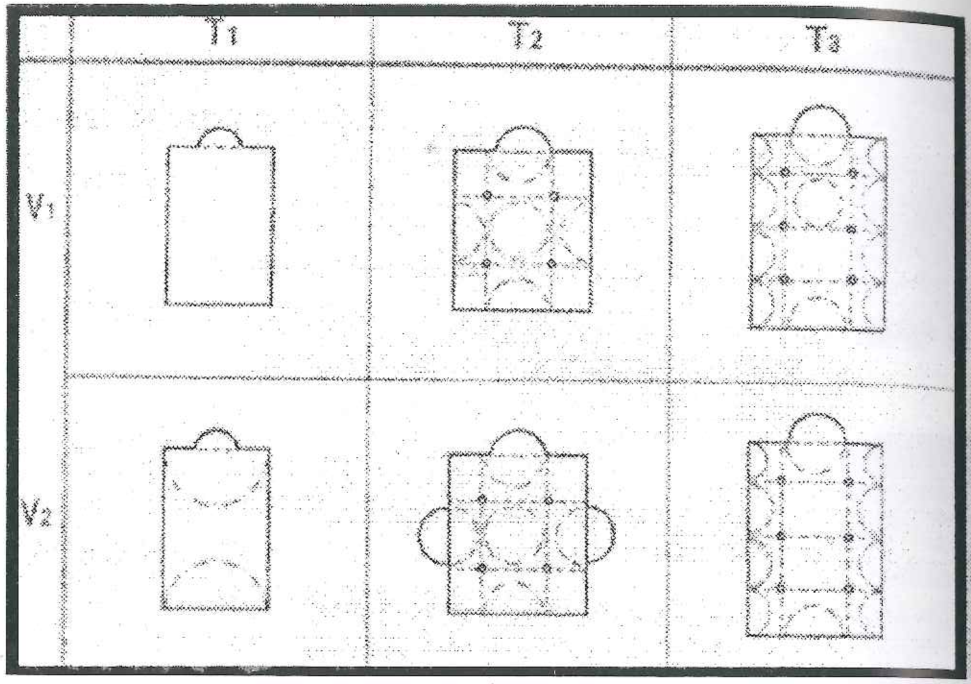


Fig.7 Church typology

The inner part is composed by two lines of stone columns. They are 10.9m high in the center and 8.8m in the lateral parts. Over the columns rest the arches in two directions creating perfect space structure. In the central part we can see the cylindrical arches which end with cupola. (fig.8).

The plan and the damages that have appeared.

The church is constructed in a trapezium form platform, with 0,2Ha surface. The church and the graveyard are surrounded by stone walls 3-6m high. The ground under the wall is very inclined, A small and rush torrent flows in the north part of the platform. The water sources are situated 30m far from Leus church. During 1981-1990 some serious damages occurred in the church as: one of the central columns row suffered 10cm settlements; the splitting of the floor in longitudinal direction; the splitting of the arches in different points; the bending and splitting of the longitudinal wall in the south part of the church; the splitting of the stone stair in the churches entry; displacement of the retaining wall near the longitudinal stone wall of the church in its south part. (look at figure number 8).

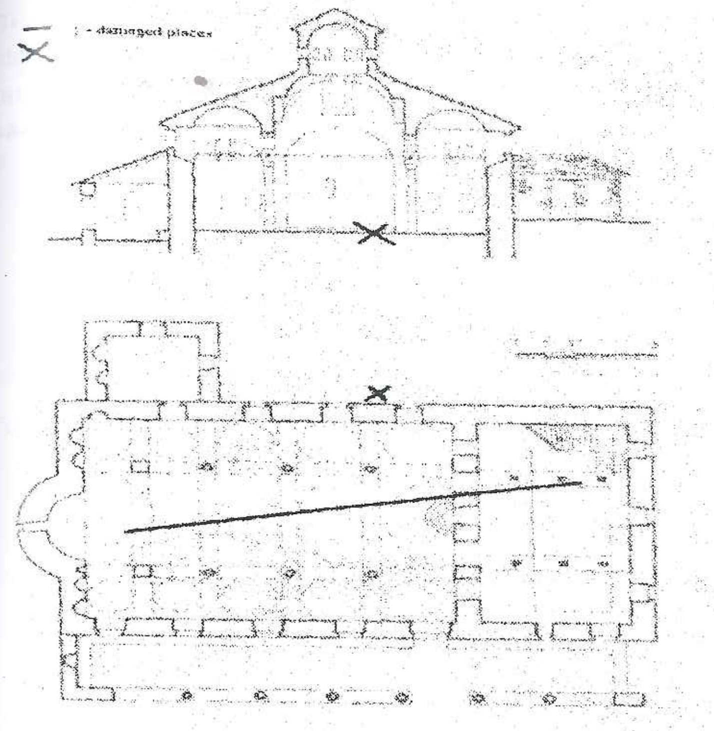


Fig.8 Section and plan of St.Maria church

The analysis of the causes of the damages.

To do this analysis we undertake a geological and geotechnical research. From this research results: in the geological aspect we have rocks of the Eocene (Pg_2) and Oligocene (Pg'_3), composed by calcareous (massive and slab) and mergel package. Also we find quaternary's deposits Q_{1-3} and Q_{4dl-el} composed by breccie and flysh. The geomorphology shows that the place of the church is situated in hills zone near a small torrent which has big erosive activity. The inclination of the slope layers is the same with the hill inclination. In the north part the slope inclination is nearly 90^0 (fig.9). From climatic condition show that we have a zone with plentiful rains (1000-2000mm rain/year), medium temperature $18,5^0$C and many water sources. The zone is relatively quiet from seismic point of view (M 5-5,4) and the last earthquake with M=5,4 happened in 1845. From the hydrological condition we can see that many water sources appear 30-40m above church. The natural drainage (the small torrent) and the geological composition of the land (flysh 40-45% silt) makes possible to have massive water flows in the underground environment and big hydrological gradients. The geotechnical conditions show that we have two layers with the following characteristics: elivium from colluviums with $\varphi=33^0$, C=14KPa, R>200KPa, e=0,72 and 63-65% gravel particles; eluvium from flysh with $\varphi=26^0$, C $\approx$0KPa, R<150KPa, e=0,8-0,82 and 40-44% silt particles.

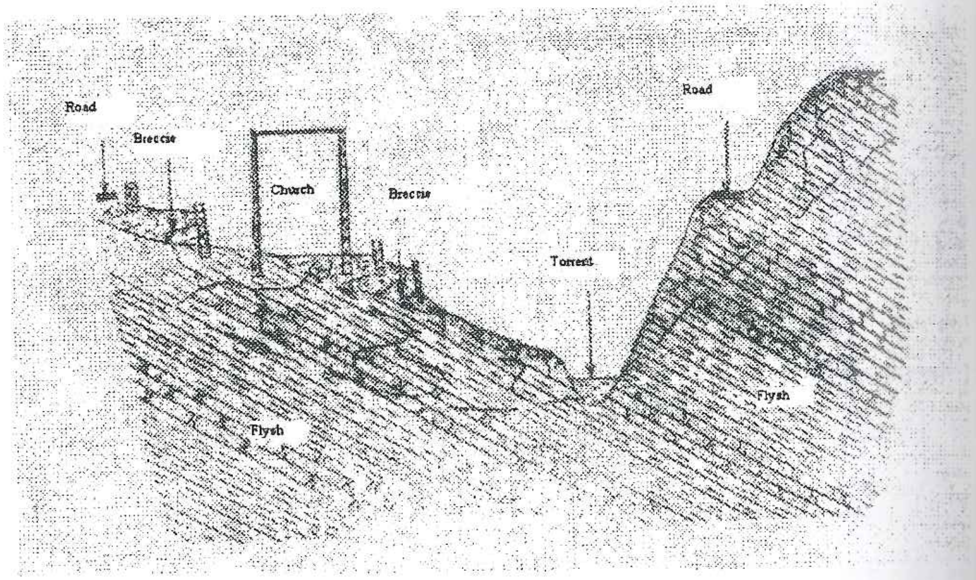


Fig.9 The schematic geologic section

The damage of the church construction is the result of the geotechnical phenomena's happened in the last 30-40 years (1950-1990). These phenomena's are:

- The presence of the superficial waters, their undisciplined movement and the terrain inclination which have caused very fast water flows, dredge of the cavities under foundations, augmentation of the bulk density, the rise of an unpredicted hydrostatical pressure in the retaining walls of the south part.
- Big supplement settlements of one part of the church foundations (over 10cm) caused by suffusion phenomena's.
- The displacement of the north retaining walls caused by hydrostatical pressure and augmentation of bulk density.
- Small slides in the west part where the inclination of the layers is the same with the slope.

- The different settlements of the church foundation because they rest in two layers with different deformable characteristics. Beside the geotechnical phenomena's in the object damages have influenced the human activity.
- The build of the transversally wall of the church (fig.10). When the retaining wall moved to stop it, a protective measure was taken to build a transversally wall which was leaned in the longitudinal wall of the church. The new horizontal load caused bending and fissure of the stone wall. In the other hand the build of a pit near the foundation for the collecting of superficial waters caused the development of the suffusion phenomena's in the soils located under the foundation. This was accompanied with supplement settlements.

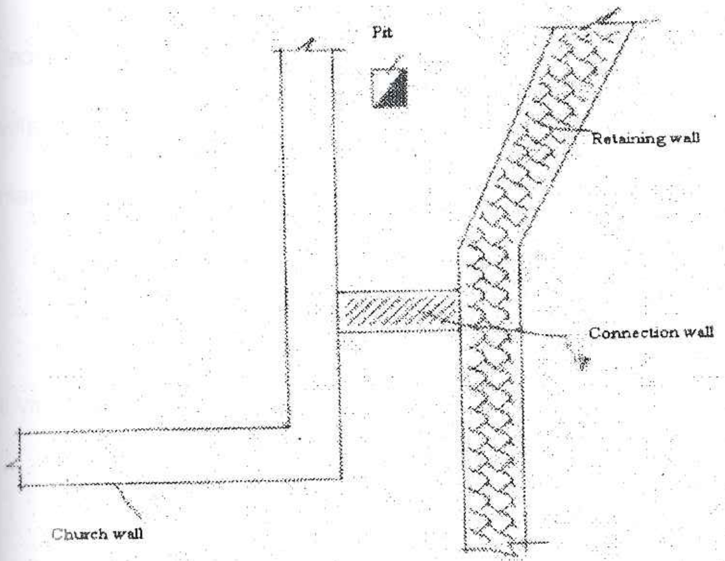


Fig.10 The connecting wall and the pit.

The engineering measures for the church repair

After the situation analysis and our geological and geotechnical study, we proposed two groups of engineering measures. The first group is:

1. The reconstruction of the foundation realizing the cross concrete beams which will strengthen the church construction and assures the uniform settlement.
2. The reinforcement of all constructive elements as columns, arches, stairs, walls and cupola.
3. The removal of the connecting wall.

4. The second group is for the improvement of the basements conditions, the channeling of the surface waters, and the reinforcement of the retaining structures. We made the design of:
   - The all canal system to collect the superficial waters.
   - The reconstruction of retaining walls by adding new transversal support elements.
   - The construction of the new retaining walls near the torrent to protect the slope behind the church from erosion.

Conclusions

The main damage causes of the old historical, cultural objects in Albania are these three:

1. Factors tied with geo environment as geology, seismic, geotechnical, weathering, erosion, water activity, suffusion, climatic conditions etc.
2. Factors tied with ageing of construction material and the lost of their quality from the climatic conditions.
3. Factors tied with uncontrolled human activity. To evidence the damage main causes of the different objects should be done:
   - A complex geological, seismic and geotechnical studie.
   - A study to see how the material behavior changed.
   - A study for the soil-structure behavior.

The problem solution to repair and reconstruct the cult objects, can be resolved only if we know well the basement and structure behavior and soil structure interaction.

REFERENCE

Adhami S 1989 Voskopoja in her flourishing century
Tirane, Science Academy (in Albania)

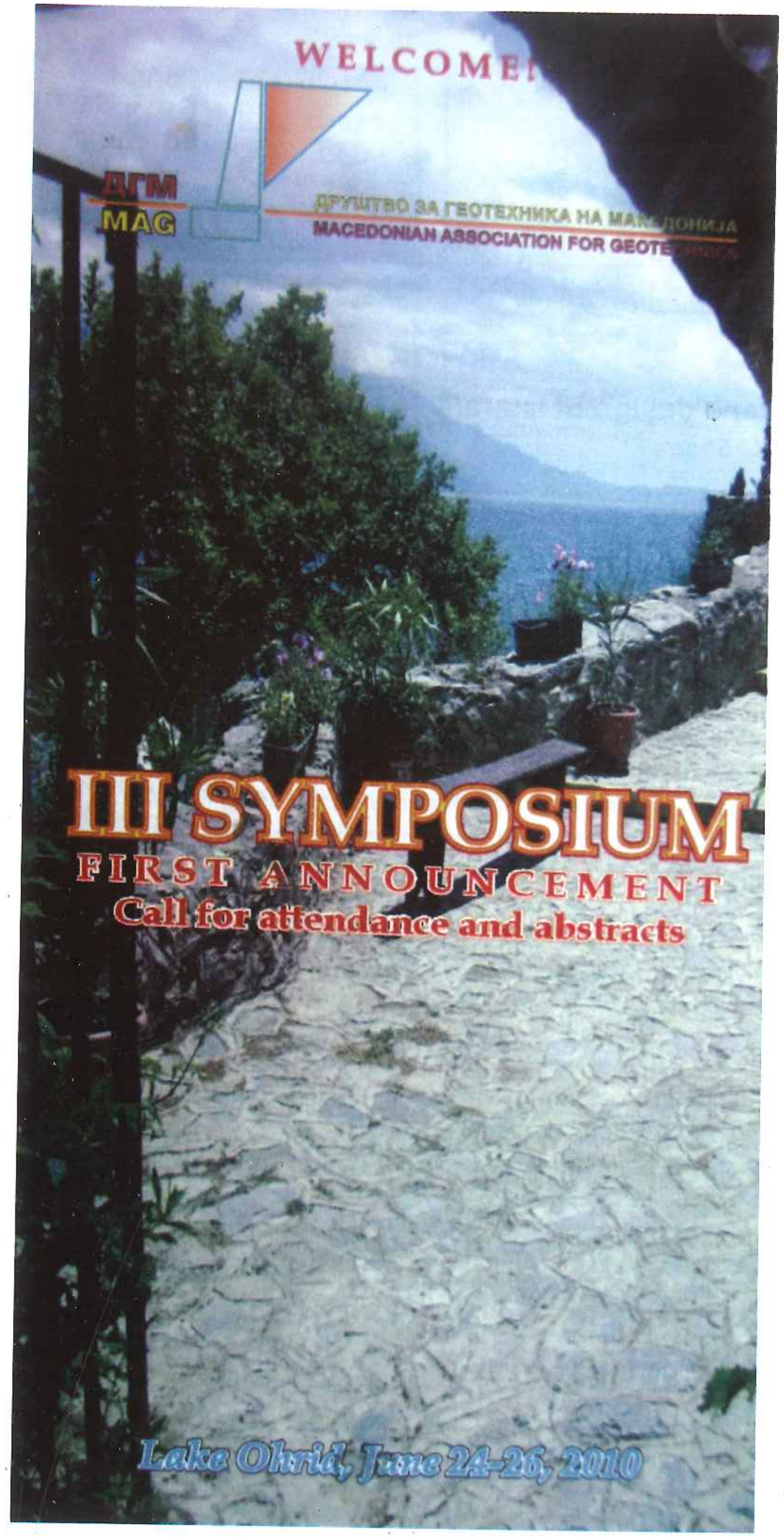
Bozo.L 2000 Rock Mechanics. Tirane, Polytechnic University

Bozo L 2001 Geotechnical phenomena's and objects damages.
Tirane Geotechnical magazine Nr.1 pp10-15

Kociaj S 1985 Earthquake and high seismic resistance projects.
Tirane. Newsletter Nr.6 Science Academy

Sulstarova E 1985 The seismic map of Albania and its use in design.
Tirane . Newsletter Nr.6 Science Academy

Thomo P 1998 After Byzantine churches in south Albania
Tirane. KOASH press.



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ДРУШТВО ЗА ГЕОТЕХНИКА НА МАКЕДОНИЈА
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Analysis and design of laterally loaded piles

E. Bukaçi, L. Sharra

Summary

Calculation of earth retention, for a one story underground excavation.
Excavation is 4.5 m.
There will be used some models to calculate this structure.
One of them is using Winkler solution in which the soil below the dredge line, is modelled by mean of the modulus of subgrade reaction ks. Modulus ks is taken as constant. Active earth pressure is given by Rankine equation. Finite Element Analysis is done by using ETABS.
The other solution will be the elastic-plasticity theory using PLAXIS.
Using the two models above, will be calculated internal forces and displacements.
The two solusions are compared and conclusions have been given.

Key words

Earth retention, finite element nalysis, etabs, plaxis, laterally loaded piles.

Introduction

Calculation is made for earth retention of a one story underground excavation, for a 9 story building. From natural soil level, will be done an excavation of 4.5 m.
Earth retention will be guaranteed by mean of drilled piles, with 10 cm space between them.
Regarding to Geotechnical Study, underground water is at level - 1.00 m from natural soil level.
At the upper level of the piles, will be constructed a beam, which will create support for the piles.
Soil parameters and layers are as below:

SOIL 1:
$\gamma_1 = 18.6$ kN/m^3
$\varphi_1 = 20°$
c = 0
E = 10Mpa
SOIL 2:
$\gamma_2 = 23$ kN/m^3
$\varphi_2 = 32°$
c = 0
E = 35MPa
SOIL 3:
$\gamma_1 = 21$ kN/m^3
$\varphi_3 = 0°$
c = 30kPa
E = 14MPa
LOAD:
q=30 kPa

$L_1 = 1$ m
$L_2 = 3.5$ m

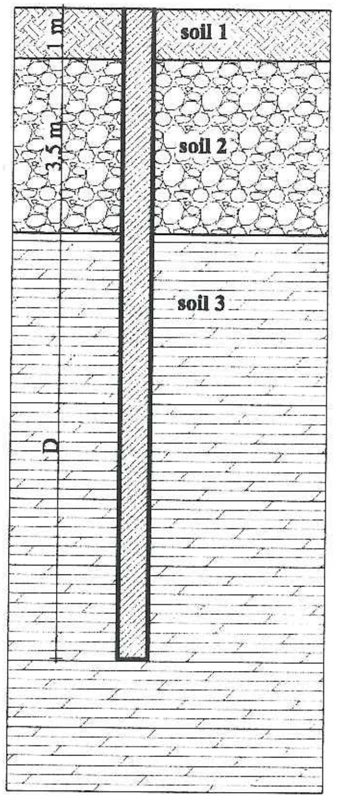

Figure 1 Soil layers

Pile calculation using finite element method (method 1, soil modeled with springs)

Actions on the piles are: Earth active pressure, p_e.

Uniform load on the ground, which could comes from one building near the piles, p_s.
Under the dredge line, earth is modelled by means of springs, value of which is calculated by Vesic formula (1).

$$k_s = 0.65 \sqrt[12]{\frac{E_s \cdot D^4}{E_p I_p}} \cdot \frac{E_s}{D*(1-\mu_s^2)} \qquad (1)$$

Ku :

E_s=14 Mpa

E_P=30000Mpa

μs=0.25

D=0.6m

ks=11340 kN/m3

$$K_{S2} = K_s \cdot d \cdot l / 2 = 11340 \cdot 0.6 \cdot 0.5 / 2 = 1701 \text{ kN/m}$$

$$K_{S1} = K_s \cdot d \cdot l = 11340 \cdot 0.6 \cdot 0.5 = 3402 \text{ kN/m}$$

Length of pile on the ground, is taken so that the displacement on lower bottom of pile is smaller than 2mm (Bowles, Foundation analysis and design, 5th edition)

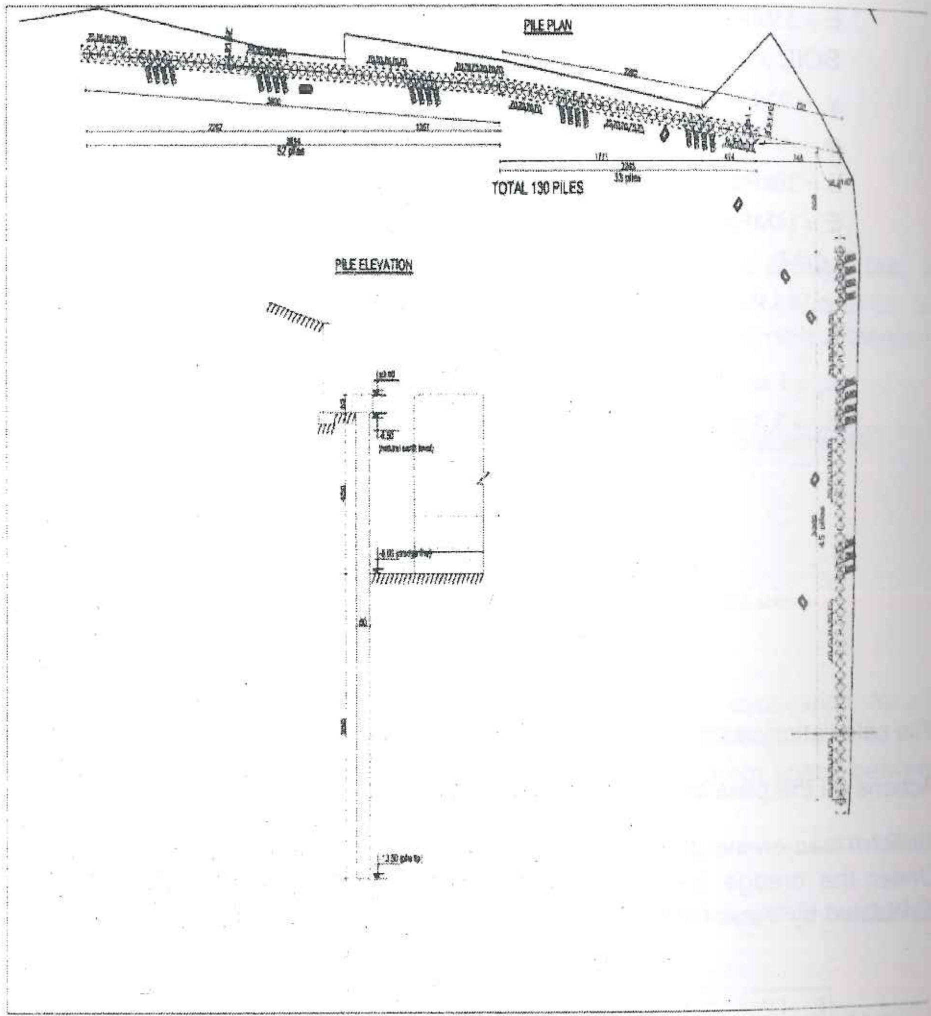

Figure 2 Pile plan

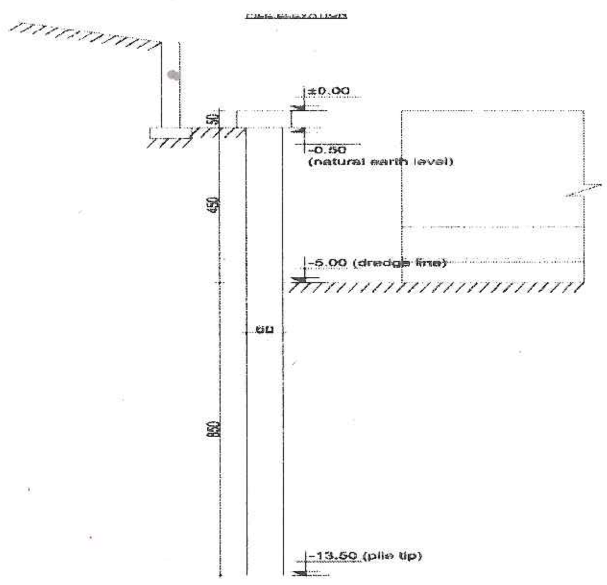

Figure 3 Pile elevation

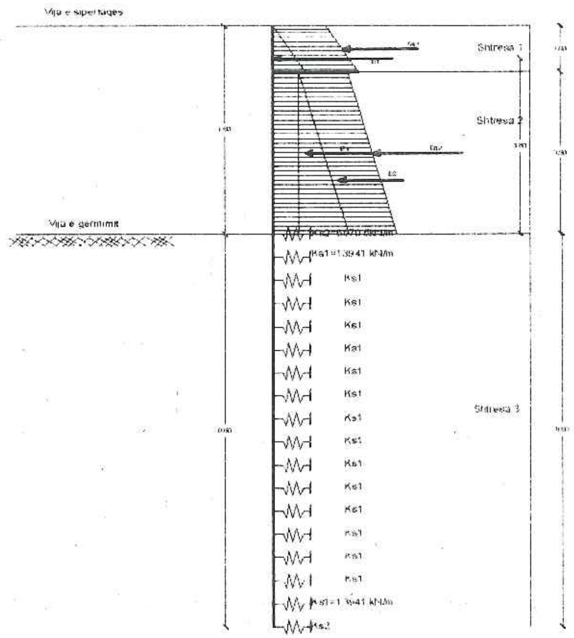

Figure 4 Model for calculation with the soil modeled as springs

Below are shown deformed shape, bending moment and shear force for the pile (fig.5,6,7):

Fig.5 Deformed shape

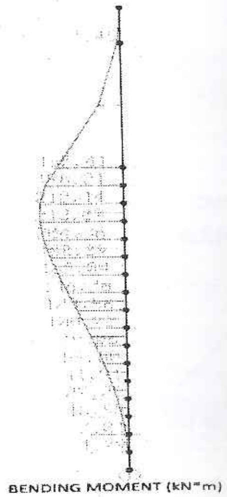

Fig.6 Bendig **Moment (Mmax=213.07 KN*m)**

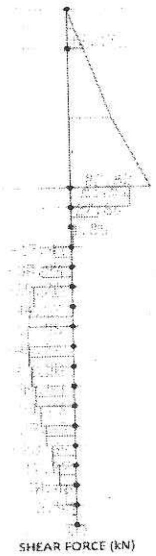

*Figure 7 Shear force (Qmax=82.82 KN*m)*

Pile calculation using finite element method (method 2, soil modeled as elasto plastic material using Plaxis)

Using Plaxis soil is modelled using Mohr-Coulomb material model, pile is modelled as plate element, giving EI, EA, weight according to the pile dimensions.

Calculation is done using plastic theory.

Result are given below (fig.8,9,10,11) :

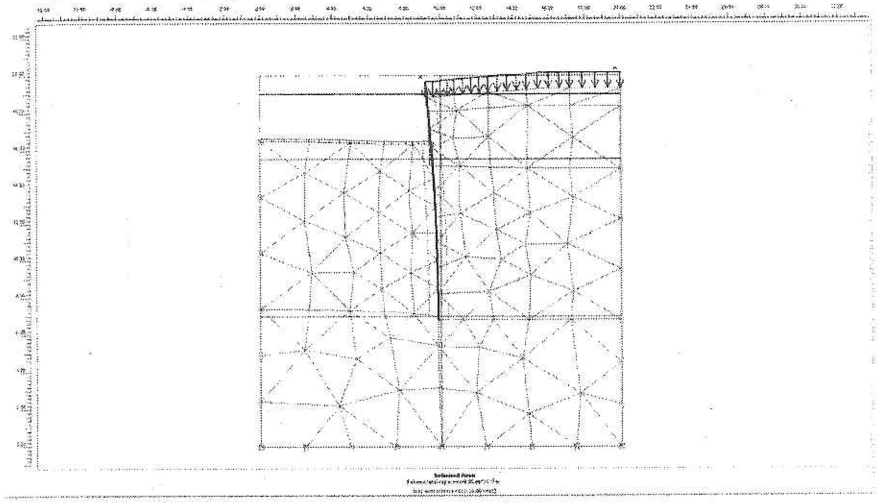

Figure 8 Deformed shape

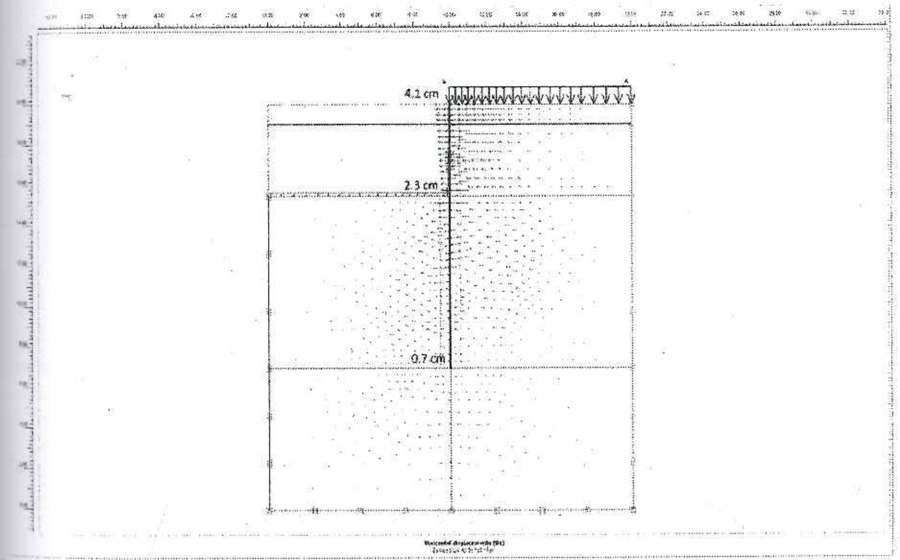

Figure 9 Horizontal displacements

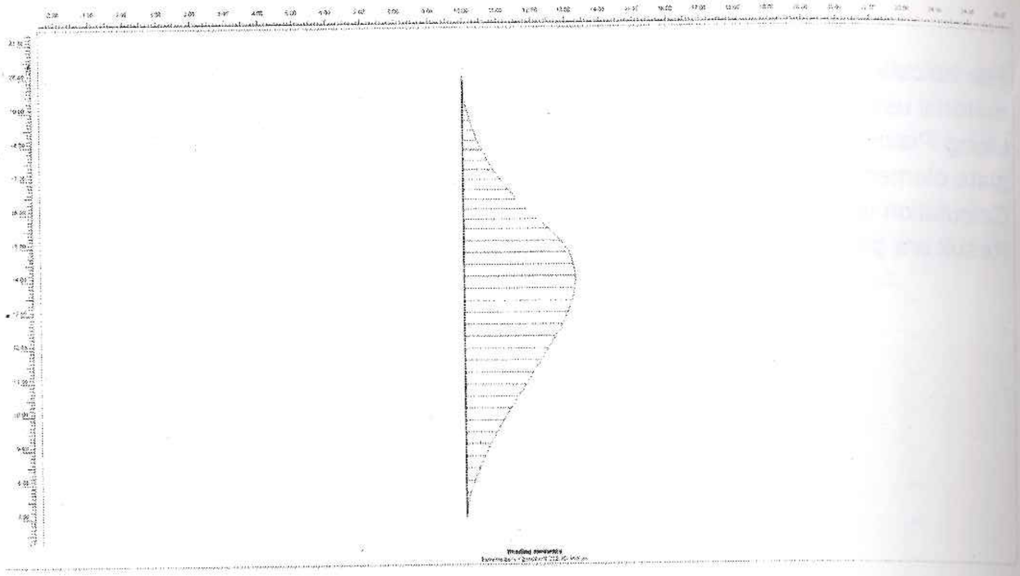

*Figure 10 Bending moment (Mmax=212.7 KN*m)*

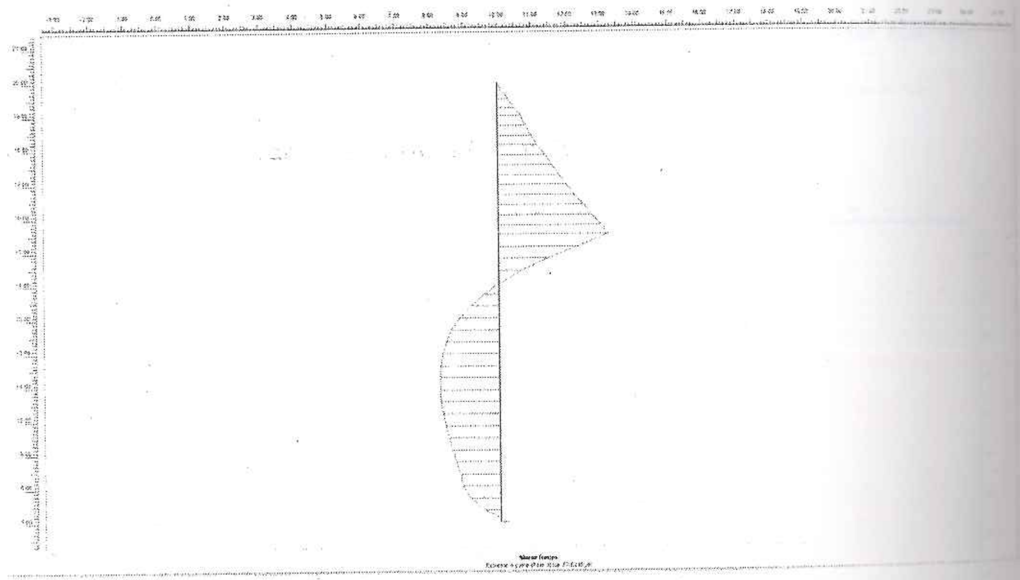

*Figure 11 Shear force (Qmax=77.62 KN*m)*

Conclusions.

In the table below are the result from both methods:

	Soil modelled with springs	Plaxis
Mmax (KN*m)	213.07	212.7
Qmax (KN)	82.82	77.62
Δ (cm)	4.1 1.2 0.2	4.1 2.3 0.7

We see that internat forces are almost the same from the two models, different result we take for the displacement at the dredge line and at the low bottom of pile.

The model in which the soil is modelled with springs uses a lower number of parameters and is simpler to perform, and we see that from both models we take almost the same result for internal forces.

Plasix gives more accurate results, so we conclude that displacements from plaxis are more exact.

REFERENCES

1. Braja M. Das: „Principles of Foundation Engineering" 2007, 750p
2. Braja M. Das: „Principles of Geotechnical Engineering" 2002, 589p
3. Joseph E. Bowles: „Foundation Analysis and Design" 1996, 1175p

Application of the Eurocode-7 in Albania National Report

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Abstract

The report presents information compiled by author which shows for the efforts made in Albania for the implementation of the European rules in our engineering practice. Especially in these report we would like to present our achievements on application of EC-7 in Albania, the problems to appear during these process and the future duties for these problem.

Introduction

In Albania we have about 10 years that we made endeavor for the implementation of the EC-7 in our geotechnical design practice.
The work think came from necessary of the great changes as:

- The passage from the dictatorial system to democratic system.
- The passage of the construction practice practically as private activity.
- Entrance of the new technologies in the buildings in the construction of the roads, bridges, tunnels etc.
- The big and rapid development of the cities as Tirana, Durres, Fier, Vlora, Shkodra etc, and consequently the construction of the many stories buildings in them (more 8-10 floors)
- The very big development of the roads infrastructure as rural roads, national roads, motor ways, high way etc.
- The construction of the many big bridges with 3-4 passages.
- The construction of tunnels
- The maintenance and rehabilitation of the over 400 dams etc.

In other part the great development of tourism was accompanied with building in the coastal zones (sea side) and in the mountains zones where are present the problems of the big deformations of the soils and slope instability.
All these phenomena's and problems disclosed necessity of the implementation of the European rules in the field of the construction, in consequence of the EC-7 In the field of the geotechnical design.

Main directions for the application of the EC-7 in Albania.

Albanian Geotechnical Society in collaboration with Civil Engineering Faculty for the implementation of the EC-7 in our practice was worked in the following direction:

First direction was acknowledgment of the EC-7, their, analyses and opinion's sensitivity for the necessity of their application in our practice.

For that purpose some seminaries and continued courses with civil engineers was made. So AGS and Geotechnical Sector of the Civil Engineering Faculty were made the following seminaries and short continued education courses:

- Insurance of the quality in the design and construction process of the roads (1998, 1999 2000)
- Eurocode-7 acknowledgment, analysis and example for calculation by EC-7 (2002).
- Pile foundations under static and dynamic loads (2001)
- Earthquake geotechnics- application in the retaining walls, pile foundations and in the embankment and dams (2004).
- The stability of the dams and high embankment (2007).

Second direction was preparation by Bologna chart, of the civil engineers with geotechnical direction (or profile).

In these year (2010), for the first time in Albania, to bring out the first specialists in these field, so the first geotechnical engineers.

For their geotechnical education in the second cycle (master of first level) they have made 9 different disciplines as:

- Experimental geotechnics.
- Roads geotechnics.
- Rock mechanics.
- Tunnels.
- Deep foundations.
- Soil dynamics and foundations under vibrations.
- Natural slopes and dams, tailing dams, embankment.
- The code in geotechnical design and insurance of the geotechnical works.
- The numerical methods in geotechnics ect.

For the forming of the new geotechnical specialist very important was organization of the professional practice for students. The professional practice was made in the design studio, in the geotechnical laboratories, or in the construction enterprise, always tied with their diploma's work. The diploma's themes of the first generation of the geotechnical engineers have treated the following problems:

- The first group was tied with laboratory and in situ testing, conception of the geotechnical model and their behavior in interaction process with structure
- The second group was tied with design of the shallow and deep foundations and retaining geotechnical structures.

- The third group was tied with slope stability and the engineer's measures for the stabilization of the unstable slopes.
- The fourth group was tied with construction on the problematic soils as the liquefied soils, unstable soils, and the subsidence or falling down soils.

All diplomas' works was realized in very good quality and their recapitulations will be published in the special magazine periodical of AGS.

Third direction was made the design process more scientific and based in the limit states (ultimate and serviceable)

To this one intention has served specialization of the civil engineers in the geotechnical field, as:

- In other countries (France, Germany, Italy, Greece, Netherlands, UK. USA)
- In the specialized enterprises in Albania
- Knowledge and application of the new calculated programs for the geotechnical structures.
- Confrontation of the results of the applications by the new methods, or new programs with results profited by monitoring of the geotechnical structures.

The exact and reliable design of the geotecnichal structures search credible data and information about soil characteristics (physical and mechanical).

This has lead in the increase of the quality of the investigations in the field, in the accomplishment of the deepening laboratory studies and in the combination of the geophysics and in situ tests for the soils and rocks.

So today we use many penetration tests in our practice as N_{SPT}, CPT, CPTU, PD.

Also we use the perforation by georadar, seismic reflection the ultrasonic test or acoustic mission and resistivity (electrical) tests.

To determinate the dynamic characteristic of the soil we use only the test based in propagation of the shear waves and other type of the waves and SPT, CPT, CPTU tests.

Now in Albania we have all possibilities to make serious, credible and exact designs of the geotechnical structures based in laboratory tests and in the field tests. In some of geotechnical laboratories can to make all kind of tests as:

- Triaxial test
- Unconfined compression test.
- Eodometric test
- Direct shear test
- Permeability of the soils.
- Chemical analysis.
- Grain size analysis.
- Test for the classification of the soils.
- Petrography and mineralogy analysis.
- All test for the rocks.

Also in our laboratories we have possibility to make all analysis for the disturbed samples, when the soils serve to constructed road embankment, airport, and bitumen conglomerates etc.

Fourth direction was the verification of the new geotechnical structures in the practice. For this goal the results by calculation programs was verified by group specialist, which based in the monitoring system. To take the monitoring data we have possibility to see if the real behavior of the geotechnical structure is the same with forecasted results and also if the interaction soil-structure was well predicted.

In this manner was operated in the:

- Rehabilitation of the Durres port.
- Building more 20 stories in Tirana.
- Deep excavation over 5 floors in Tirana centre.
- Underground construction till 20-25m depth etc.

Problems to resolve

In spite of achievements we have yet some problems to resolve:

- Main to them is that the EC-7 it is not yet applicable as low obligatory to put into practice from all civil engineers. Resolution of this problem depends not only from AGS but from government.
- Also we have problems with the continued adjournment of the civil engineers by short forming courses. This problem can to resolve by common activities of AGS and Civil Engineering Faculty.
- One other problem is lack (absence) of contacts with our colleges in other countries. We need for more contacts with foreign specialist for resolution of different projects in the geotechnical field as:

- Reconstruction of ports airport.
- Rehabilitation of dams and tailing dams.
- Constructions of the industrial plants, or energetic parks in very difficult soils or conditions.
- Usage of the geotextil, geomembran, or geogrid etc for diverse intentions etc

We hope resolved quickly these problems when the Albanian specialist can to moving loosely in other countries

Conclusions

Finally we can say that we doing worked more in following direction:

- For entire (complete) implementation of the EC-7 in our practice
- Specification of all phases to arrive at the final objective which is approval of laws based in EC-7 for geotechnical calculations
- Acknowledgment of the civil engineers with phases of geotechnical design by ultimate states and serviceable states.

- Training and continual education of the civil engineers in the geotechnical field process which can to realized:

- In the Civil Engineering Faculty.
- In the master level.
- In the doctor's school.
- In the continued forming courses.
- In the common projects with our colleges from other countries.
- In the research and scientific regional works etc.

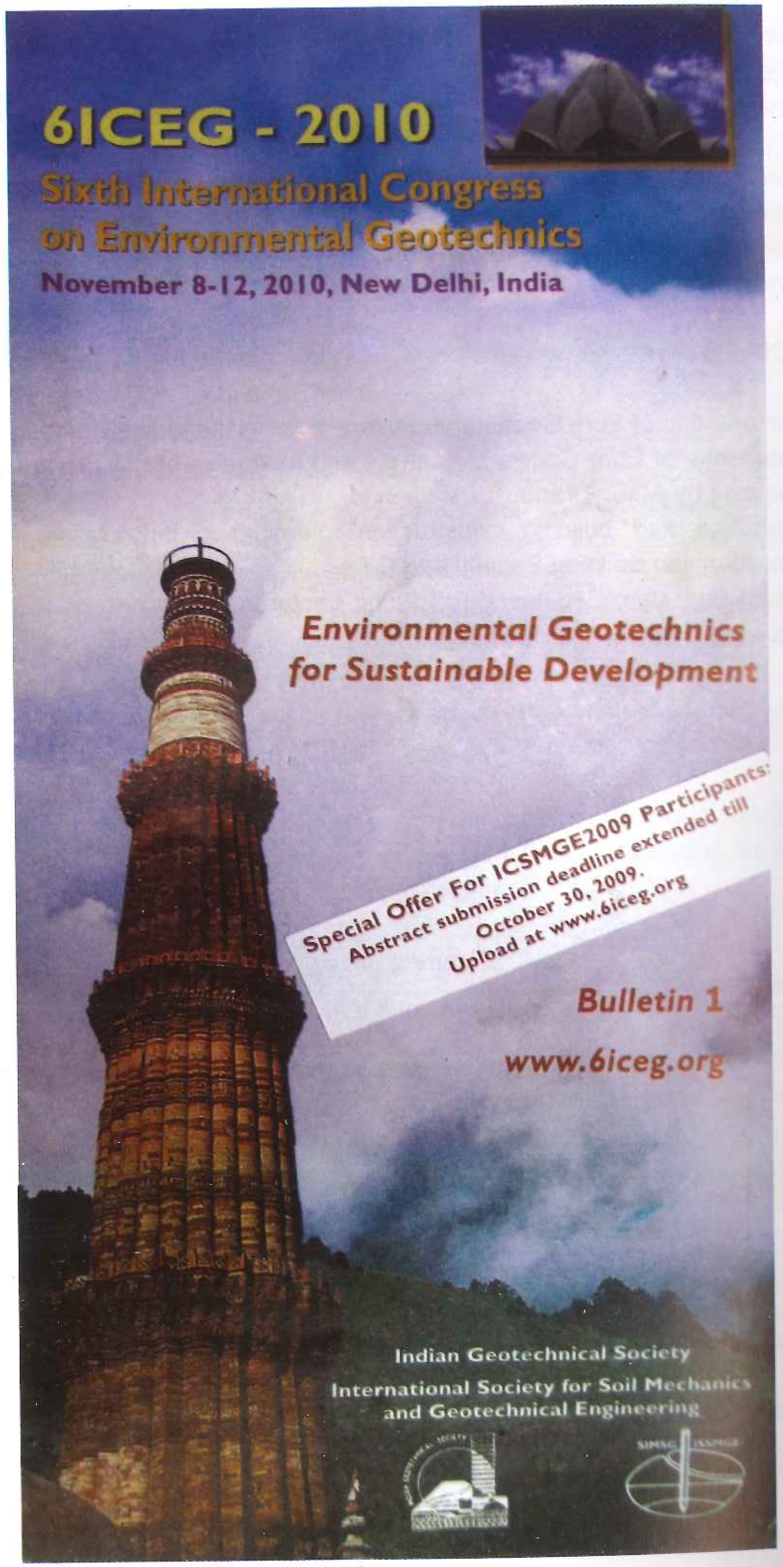
REFERENCE

2002 "Implementation of Euro Codes in Albanian" National seminar, Tirana.

2002 "Acknowledge of Euro Codes 7, analysis and examples of calculation" Formation course organizing by AGS. Tirana.

2004 "Earthquake and building construction" Seminar organizing by Ministry of Construction, Albanian Building Society and GTZ.

2007 "Embankment dam Engineering" Touring Lectures, organizing by ISSMGE, and AGS. Tirana.



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Contaminated zone around Fan river and their rehabilitation

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Abstract

In the north part of Albania there are big mine industry for the extraction and treatment of minerals as iron, copper etc. These mines are situated around river Fan. The zones around this river are contaminated in different quantities by dangerous chemical elements and heavy metals. In this paper we would like to present our study about the degree of the contamination of the zones around Fan river and our measures for their rehabilitation.

Introduction

Albania is traversed by a dense river network (Figure1).

Fig.1 The hydrological map of Albania.

In the north part of the country flows in an accented mountain terrain the Fan river which is the main branch of the Mat river. This zone is very rich with minerals. During 1960-1990 this zone had a very big development in extraction and elaboration in mineral industry, mainly copper, zinc, iron, chrome etc. Also for the refute deposits from mineral enrichment the tailing dams were constructed with height 30-50m.

Mineral extraction, their enrichment and the siderurgic industry have grown the pollution of the zone around Fan river to make a serious danger for the fauna and flora that is developed there. We have undertaken many studies for the determination of the pollution degree and the danger of the environment of this zone. In the following paper we are going to present one of these studies.

The natural pollution of Fan river

The Fan river is the main branch of Mat river which is one of the most important river in Albania (Figure2).

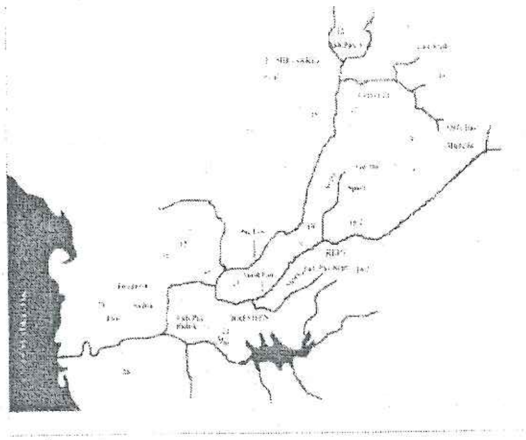

Fig.2 River Fan and its branches.

The water collect surface of Fan river is 1076km^2 and after it is joined with Mat river the water collect surface grows up to 2441km^2. Fan river is formed by the connection of two branches: big Fan, 76.9km long which stems from Qafa e Malit (1397m height) and small Fan, 54.5km long which stems from Runes (1856m height). After 17.5km of their connection it flow's in to Mat river. The medium precipitation for year is 1750mm in the big Fan and 1660mm in small Fan. The feeds between October-June are from 14.4m^3/s to 59.7m^3/s. The erosion activity of Fan river is big and it transports 208000 T/year solid material. Also the total quantity of the solid feeds in suspended and underwater condition is 119000 T/year. From the chemical point of

view the waters are carbonates and their mineralization (mg/l) is shown in table 1.

TABLE 1 Jones content in river water.

Jones	Ca	Mg	Na	K	HCO_3	Cl	SO_4	Σj
Big Fan	25,73	11,1	2,31	0,5	86,08	8,55	10,57	144,8
Small Fan	39,19	9,68	4,19	0,49	126,8	7,66	29,48	217,5

The factors that influenced in the contamination of the zone around Fan

As we said before the zone where the Fan river flows is rich with minerals, but the Cu-mineral creates the biggest toxic risk for the water, land and air when it enters in contact with them. The factors which have contaminated this zone are the following:

The mines.

On the surface and underground in mine's zone there is flowing water which permits to extract from them the minerals. Water from mine's can be extracted directly or by putting pumps in the pits. This water contents heavy metals and consequently presents high pollution degree for flora and fauna.

The liberation of the lixiviates.

After the abandonment of the mines (20 years ago) began the creation of the percolated acids or lixiviates from the solid remains outside of the mines. The lixiviates have a big percentage of sulfuric acid and heavy metals as Ag, Cd, Co, Cu, Mg, Hg, Ni, Pb, Zn, As, Se. The mineral remains have extension along the entire Fan river contaminating all the zone. Except the "Fe" all the other metals are toxic.

The tailing dams.

The acid is created rapidly in the tailing dams. This depends from sulfite content in minerals, from climatic conditions and necessary oxygen for the oxidation process. All this phenomena's made possible the contamination of the surface and underground waters. Also the oxidation of sulfites can lead to the liberation of the heavy metals as Ag, Cd, Co, Ni, Pb, Zn, As etc. The zones with great acid generates is positioned 15km far from tailing dams. This acids cause not only destruction of the flora but even the ravage of the fishes and other micro organisms in the river.

The enrichment industry in Fushe-Arrez, Reps, Rreshen and the foundry in Rubik.

All this industries are other factors for the air and water pollution which pollutes them by dust which comes out from the grinding process of the minerals. The elaboration capacity of the minerals reaches 300.000 T/year and the content of "Cu" goes from 12-24%.

Determination of the contamination degree of the zone around Fan river

For the determination of the pollution degree we took many water samples from the rivers (Big Fan, Small Fan etc). The samples were exanimate in chemical laboratory of Tirana's University. For the monitoring and determination of the contaminated degree we built about 30 stations to take the samples from water collect basin of the Mat river and their branches. The result of the analysis taken from the monitoring stations during period 1998-1999 (10 years after mines were closed) we can see in the tables 2,3,4,5.

Table 2 The metal content mg/l for Big Fan (1999)

Station (when take the samples)	As	Cd	Co	Cr	Cu
	Sept	Sept	Sept	Sept	Sept
9	10	1	1	2	2
10	10	1	1	2	34
11	10	1	3	2	37
12	10	1	51	2	150
14	10	1	1	2	10
15	10	1	1	2	110
24	10	1	13	50	43

Table 2 Continue

Station (when take the samples)	MO	Ni	Pb	V	Zn
	Sept	Sept	Sept	Sept	Sept
9	5	2	5	2	2
10	5	5	5	2	7
11	5	6	5	2	23
12	5	15	5	4	210
14	5	2	5	3	2
15	5	3	5	4	34
24	5	70	5	66	25

Table 3 The metal content mg/l for Small Fan (april 1998 –January 1999).

Station (when take the samples)	Ca	Fe	K	Mg	Na	S
	April	April	April	April	Jan	April
9	7	0.14	0.49	7.5	0.98	2.8
10	6.5	0.31	0.37	8	1.3	3.3
11	8.3	0.28	0.39	11	3.7	5.9
12	29	1.2	0.31	7.9	1.4	30
15	10	1.2	0.30	7.1	6.85	6.3
24	13	1.2	0.48	4	1.7	5.2
24M	14	0.52	0.43	12		4.9

Table 3 Continue

Station (when take the samples)	Ag	As	Cd	Co	Cr	Cu
	April	Jan	April	April	April	Jan
9	0.46	10	0.04	0.5	1.4	7
10	1	10	0.1	10	10	20
11	1	10	0.1	10	10	310
12	1	10	0.1	10	10	20
15	1	-	0.1	10	10	-
24	0.1	10	0.12	2.5	4.8	60
24M	0.05	-	0.06	1.1	3.5	-

Table 4 PH and concentration of Cu and Zn in the samples took from Lumi Zi (LZ) and Big Fan river (FM)

Station (when take the samples)	PH		Cu mg/l		Zn mg/l	
	April '98	Sep '99	April '98	April '99	April '98	Sep '99
1(LZ)	7.8	6.2	85	11	86	2
2(LZ)	7.6	7.9	40	16	20	34
4(LZ)	6.4	5.8	730	4200	320	480
5(LZ)	7.6	5.5	47	7	16	2
6(LZ)	7.4	6.2	18	12	7.2	2
8(LZ)	7.0	6.2	130	14	45	6
13(LZ)	7.3	7.2	220	120	110	220
9(FM)	7.6	7.7	16	4	7.7	2
10(FM)	7.6	7.6	30	5	10	7
11(FM)	7.1	7.5	20	9	10	23
14(FM)	-	8.4	-	5	-	2
15(FM)	7.5	6.9	90	50	50	34
24(FM)	7.9	7.9	46	14	19	25

Table 5 PH and concentration of Cu and Zn in the samples took from Stefa (S) Small Fan (FV) And Fan (F) rivers.

Station (when take the samples)	PH		Cu mg/l		Zn mg/l	
	April '98	Sep '99	April '98	April '99	April '98	Sep '99
16(S)	4.5	3.4	680	2200	610	4200
17(FV)	8.5	8.5	14	5	5.6	2
18(FV)	7.5	7.6	30	120	40	210
19(FV)	7.3	5.3	120	87	120	160
20(FV)	7.8	7.6	52	20	10	6
21(FV)	7.7	7.9	60	140	60	200
23(FV)	7.9	7.9	59	150	43	120
25(F)	8.0	8.1	48	18	26	3
28(F)	7.6	7.9	16	32	11	31
29(F)	7.8	7.7	24	140	23	170
32(F)	7.9	7.9	30	52	20	50

From the graphics compiled by us, we show the changes of the concentration of some heavy metals (Cu, Zn) in the water from the source of Big and Small Fan until they join together from this variations (fig 3,4), we can see these:

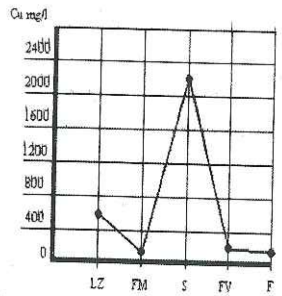

Fig.3 Variation of Cu concentration in different zones

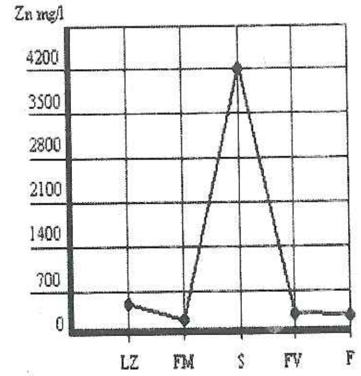

Fig.4 Variation of Zn concentration in different zones.

The most contaminated zone is around Stefa river.
Nevertheless all zones around Fan river and its branches have a high contamination level by heavy metals Cu, Zn, Cr, Cd, As. Usually with pronounced acid characteristics is the water of Small Fan. Also with high Fe content are Small Fan, Lumi Zi, Big Fan and Fan.
Highly contaminated by P_b is Small Fan.
High content of Pb have Small Fan and Fan.

The meassures for the rehabilitation of the situation

We think that it is necessary to create a permanent monitoring system which must observe how the scale of contamination changes year after year in Fan river and its branches. It is necessary to neutralize the remains of dangerous minerals especially in the zones around Lumi Zi and Sefta river. Another measure is the maintenance of the 7 existing tailing dams with intension to impede the slope erosion. Also the tailing dams must be covered by geomembranes or vegetable layer with goal to protect the air from dangerous elements. Finally if in the future the mining activity restarts they must use only the adapt technology which can protect the environment from pollution. Also in the zones were the soils are not alkaline there must be an intervention with intention to create the necessary capacity to neutralize the acids and to precipitate the deleterious metals in them.

Conclusions

The Fan river with its branches has a high degree of contamination in acid water and high content of dangerous elements as Cr, Cd, As, Zn, Cu etc. Rehabilitation of the zone around Fan river and its branches needs to undertake urgent measures as: reduction of acidity, elimination of the heavy metals in the underground and surface waters, the maintenance of the tailing dams etc.

The damage of the flora and fauna in this zone is with great consequences for the life and health of the people for that reason the engineering measures and chemical interventions, must minimize the toxic matter in the river waters.

REFERENCES

Bozo L. (2009) Geotechnic for the environmental engineering. Press Natyra, Tirane, Albania.

Nobre R.C.M and Nobre M.M.M and Bonaparte L.B (2006) The evaluation of soil and ground water remediation at an industrial site using risk analysis. Proceedings of 5th ICEG, Cardiff.

Pastarus J.R (2006) Ground surface subsidence risk assessment by underground mining in Estonia. Proceedings of 5th ICEG, Cardiff.

Stratoberdha P. (2007) Hydraulic modeling theory of the rivers. Science Academy of Albania.

Science Academy of Albania (2008) Albanian Hydrology

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Soil Erosion Assessment through the Evaluation of the Main Elements of Erosion in Albania

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Abstract

Albania based on different studies result as a country with high quantitative level of soil erosion in Europe and elsewhere. The amount of soil eroded from the surface water erosion is from 20 – 40 t/ha/year and in extreme cases result up to100 t/ha/year from the deepest erosion. This preliminary study will identify the more specific places in order to do more details analyses in the future to take protection measures in highly sensitive places with high erosion risk. This will be a quantitative assessment of places with different level of erosion risks. Further on these results will be generalized at the level of main water basins of Albania. The following main elements of soil erosion Passessment are evaluated in this study include:

- land cover;
- slope;
- density of the hydrologic net.

Background

Soil erosion represents one of the most important destructive phenomenons of the soil, through surface and depth erosion. The activity of water erosion in Albania is favoured from some factors like relief, geological structure, slope, soil, etc. Erosion depth growth is closely related with the vegetal cover ravage high rate and in the first instance that of woodland flora in sloping ground.

The degradation of the flora or total destruction of it is defined from many factors, but in notably from social-economical system of every country. It can be seen also in some special areas in our country, were as results of the human negative impacts in natural environment, the erosion phenomenon is more problematic, especially during intensive raining time.

This phenomenon also is evidently influenced in massive sliding which now is very patency in other zones of our country, causing damages either in environmental or in economical terms

The materials and method

Albania is one of the typical zones with regard to the erosion phenomenon development in our region. To do a correct assessment of soil erosion in Albania are needed other data like rock, soil, rainfall, etc (Gatzojannis S. et al 2001), but in the absence of this information, we have done this study, based in the available data that we have. The factors which are taken in analyse for assessment of erosion risk are:

- land cover;
- slope;
- density of hydrologic net.

The data for the land cover are collected from the Albanian National Forest Inventory (ANFI 2003). To estimate the risk erosion, about 36 class of the land cover are classified in four classes for modality, in base of the land cover percent in a specified area.

The slope is another element important in the process of erosion risk assessment (Marko O, 2010). Assessment of the erosion risk according to the slope is classified as below:

- **Low erosion risk** includes 0–100 interval. This rate is considerate with minimal erosion.
- **Moderate erosion risk** includes 11 – 150 interval. This rate is considerate with medium erosion.
- **High erosion risk** includes 16 – 300 interval. This rate is considerate with high erosion.
- **Very high erosion risk** includes 31 – 900 interval. This rate includes the land with degradation bank, where the erosion is very high.

Soil erosion assessment is done with combination of two elements, which are described together with elements of watershed net (Academy of Sciences of Albania, 2005

Results and discussions

Land cover is a very important element for assessment of erosion risk. The data which are taken from Albanian National Forest Inventory, to classify the cover category are given according to FAO classification (ANFI 2003). To estimate the erosion risk, about 36 classes of land cover are classified in four classes for modality that influence the erosion, in base of the cover percent in a specified area as shown in table nr.1.

TABLE 1. Category of Land Use

Land classification	Estimate (points)	Erosion risk rating
Natural vegetation with high density (forest), flat agriculture areas, natural and artificial aquatic areas and urban land.	100-150	low
Natural vegetation no high density (forests and shrubs), hilly agriculture areas and fruit yards	151-250	moderate
Natural vegetation low density (areas with forest and shrub degradation).	251-350	high
Areas without vegetation	351-400	Very high

In base of this classification on natural vegetation and referring table nr.2, results that about 15% of the area or about 4'330 km2 are areas without vegetation, which presents very high erosion risk.

TABLE 2. The Erosion Risk According to Natural Vegetation

The erosion risk rating	Distribution of areas according to erosion risk	
	Surface (km2)	Distribution %
Low	11'129.50	39
Moderate	12'916.50	44
High	411.87	2
Very high	4'325.04	15
TOTAL	28'782.91	100

A very important element in estimate of soil erosion is also topography; where a very important element in this estimate is the slope. The slope is classified in four classes, to estimate the erosion risk given in grades.

In 0-100 interval are included the lands which are managed as agriculture and urban lands. In the flat area, there are some forests which are forested with Mediterranean pine and with fruit yards. In 11-150 interval are included the lands where collection of solid material comes from mountain flow. They are part of stability formations and are characterized from depth erosion.

TABLE 3. Slope Classification

Slope classification in grade	Estimate (point)	The erosion risk rating
0-100	100-150	Low
11-150	151-250	Moderate
16-300	251-350	High
31-800	351-400	Very high

In 16-300 intervals the speed of water flow is highest, extending thereby the erosion rate. These areas are valued as high erosion risk category. In 31-800 interval are included the very steep lands, where the speed of water flow is very high. In this area is very difficult the development of vegetation.

From elaboration of these dates in GIS system for four slope class is prepared the map, which is given in fig 2. To estimate the erosion rate is done the combination of two elements in addition with those of hydrographic net for nine watersheds. In table nr.4, are given the data to erosion risk rating.

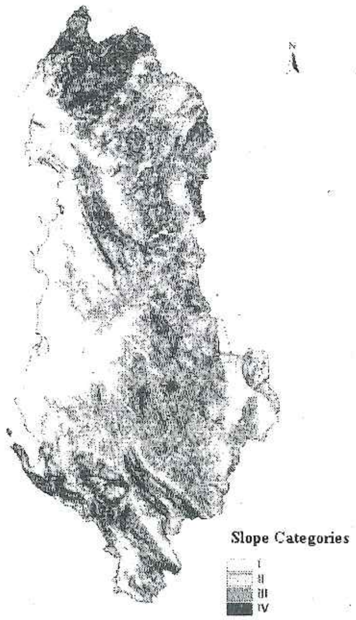

Fig. 1 The Albanian slope map

TABLE 4. The Erosion Rating

The erosion risk rating	Distribution of areas according to erosion risk	
	Surface (km2)	Distribution %
Low	10954.44	38
Moderate	12530.76	43
High	2611.64	9
Very high	2686.64	10
TOTAL	28783.48	100

From data elaboration results that in Albania about 19% of area includes areas with high risk erosion, which 9% of all or 2611 km2 are classified as land with medium erosion rate (3d class of erosion risk) and 10% or 2686 km2 are classified as land with high erosion rate (4th class of erosion risk).

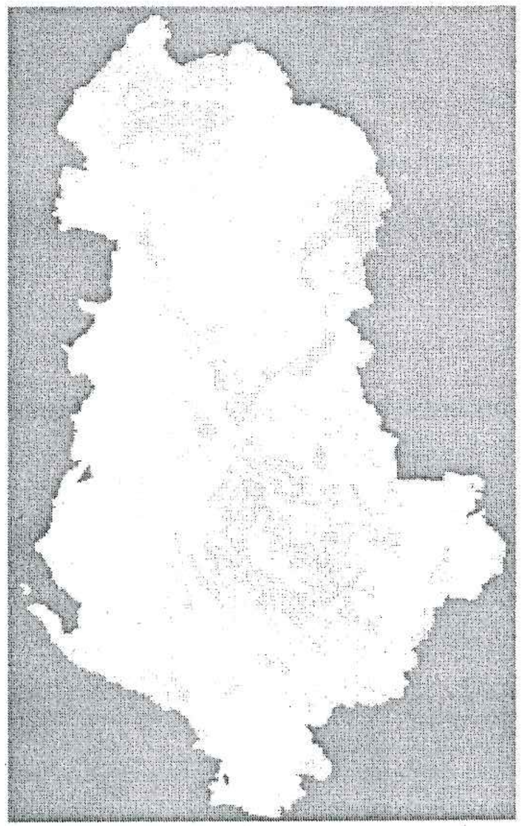

Fig. 2 Map of the erosion risk of Albania

These data are used to estimate the erosion risk in nine watersheds in Albania.

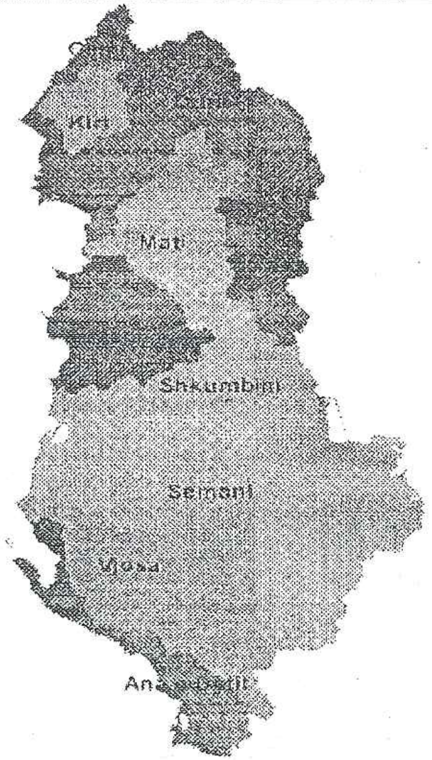

Fig. 3 Map of watersheds in Albania

TABLE 5. Distribution of Erosion Risk for Albanian Watersheds

Watershed name	Erosion risk areas (km2)			
	Low	*Moder.*	*High*	*Very high)*
Ana e detit	585	667	191	248
Cemi	66	193	60	113
Drini	1956.	2926	526	640
Erzeni	1392	724	97	95
Kiri	339	343	78	82
Mati	871	1240	282	223
Semani	3078	3115	621	429
Shkumbini	888	1193	205	222
Vjosa	1351	2026	530	627
TOTAL	10526	12427	2590	2679

From these data, results that highest degree of risk erosion is in the Cemi watershed with 26%, Ana Detit watershed with 15%, Vjosa watershed with 14%, Kiri Watershed with 10%, etc.

Conclusions

From the study and analysis of the results based on computerized processing and GIS programme used, we add up to some precursory definitions of the study:

Current rating of the erosion phenomenon results to be very critical. This is clasified in four classes from where in the first class (low risk) have 10954.44 km2 or 38 % of the total surface, in second class (moderate risk) 12530.76 km2 or 43 % of the total surface, in third class (high risk) 2611.64 km2 or 9 % and in fourth class (very high risk) 2686.64 km2 or 10 % of the total surface.

In base of this classification results that in land without vegetation, there is a very high erosion risk.

In base of this classification, results that in area with high slope, there is a high to very high erosion risk.

REFERENCES

- ANFI (2003) Albanian National Forestry Inventory
- Gatzojannis S. & Stefanidis P. & Kalabokidis K. (2001) An Inventory and Evaluation Methodology for Non-timber Functions of Forests.
- Marko O. (2010) The Assessment of Soil Erosion Risk in Mountain Watersheds (Monography)
- Academy of Sciences of Albania (2005) Albanian Hydrology.

European Conference of Young Geotechnical Engineers

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Problems faced during the design of one bridge abutment.

Civil Engineer Xhevahir ALLIU

Abstract

To have the following data:

- The category of bridge (first category)
- The construction zone (parameters of geology engineered studies)
- The Seismic Zone (7 ball according to MSC64, $a_g = 0.20g$)
- The purpose of the Study

It will be studied the problems of the preliminary horizontal displacement, the inter forces and tensions in the soils of the foundation during the construction (exploitation) of one bridge abutment.

Introduction

- The development of infrastructure during last years in Albania.

In the last years, in Albania the road network is greatly developed due to the main factors:

- The development of economy and trade in the southeastern region during last year's.
- The passage of the transition period from a dictatorial regime with a closed economy in that of a country candidate for EU. To be emphasized that the road network before 1990 has been limited.
- The augmentation of vehicles in the last years. (Below is given the table of the incensement of the vehicles for 1000 inhabitants of Albania, Greece and EU).

	Year's								
	1970-80	**80-90**	**90-95**	**1996**	**1997**	**1998**	**1999**	**2000**	**2001**
Greece	13%	7%	4%	6%	7%	7%	8%	10%	10%
EU	5%	3%	2%	2%	2%	2%	2%	1.80%	17%
Albania				13%	15%	**19%**	**26%**	**25%**	**17%**

SOURCE : The internet European Commission.

According to the facts above in Albania, are realized and are still in the process many investments for the road infrastructure

To be mentioned the "Corridor from Durres to Kosovo" 130 km road, with 4 passing lines with several bridges, with two tunnels, each 6.0 km (the longest in the region). An investment around 1 milliard Euro.
Also, many road segments are in the execution process. The Albanian territory is mainly mountainous, leading to the designing of the bridges.

Factors taking part in the horizontal displacement of the bridge abutment.

The principal factors are:

- Kinds of ground represented through parameters of Physics-Mechanics characteristics of geological layers at the road axe

Main parameters are:

  - The inter angle of ground - φ (°)
  - The volume weight of stuffing γ (kN/m^3)
  - The module of deformation E (kN/m^2). etc..

- The geometrical form and dimensions of the bridge abutment.
- The spring coefficient with which the ground is module, in numerical modulation. This coefficient is taken in the function of Parameters f (R,E), where :
  - R-is the holding capacity of basement
  - E -is the deformation module.

Needed to underline also, that in case of definition of the spring coefficient, to exactly define the spring coefficient in diverse direction (C_x, C_y, C_z).

Factors which affect at the preliminary dimensioning of the bridge abutment

The principal factors which taking part, in preliminary dimensioning are:

- The substructure loads (N,M,H), for example permanent loads (Dead Loads), Live Loads, Moving Loads (vehicles) etc..Their effect, can taking out from the loads combinations
- Seismic action, according the ground acceleration
- The holding capacity of basement - R (kN/m^2)

Some aspects during designing of the bridge abutment

Content
Model notes
Input Data
Construction
Loads

Results
Modal Analysis
Seismic Design
Concrete Design

Model Notes

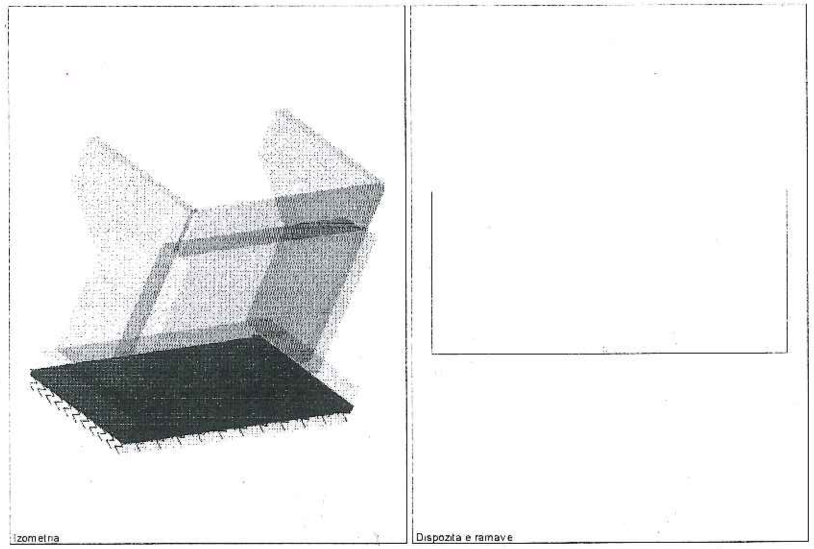
Calculation Mode: 3D model

[X] I-st Range Theory
[X] Tones Analyses
[ ] Stability
[ ] II-nd Range Theory
[X] Seismic Design
[ ] Beam Offset
[ ] Stages of construction

3D Model

Number of Nodes:	678
Number of plate elements:	630
Number of beam elements:	0
Number of boundary elements:	2208
Number of basic cases of loads:	5

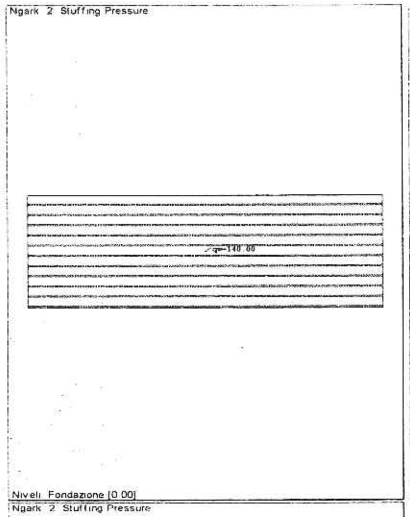
Inputs data – Construction

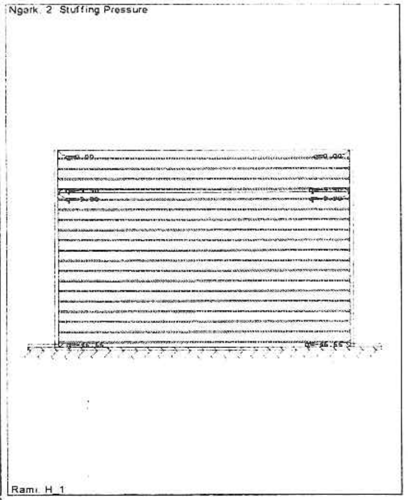


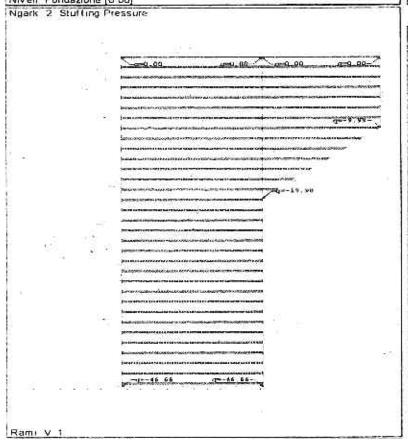
Input data Loa

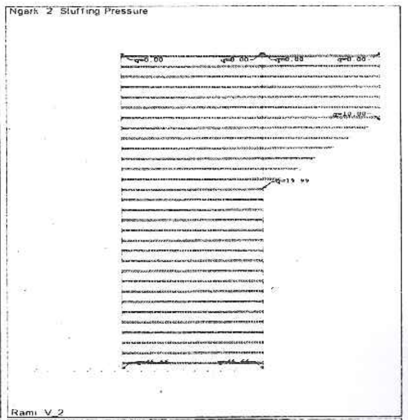
List of Cases of Loads

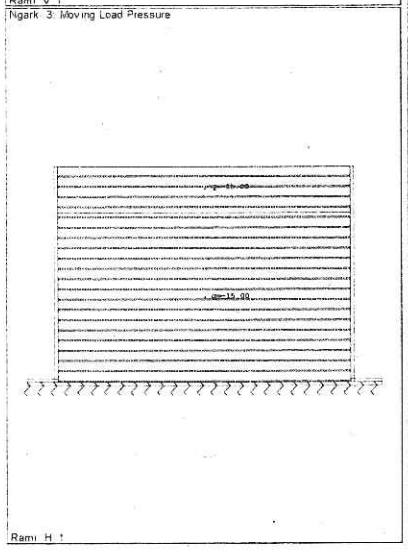
No	
1	Dead Load (g)
2	Stuffing Pressure
3	Moving Load Pressure
4	Sisma Sx (seismic action)
5	Sisma Sy (seismic action)

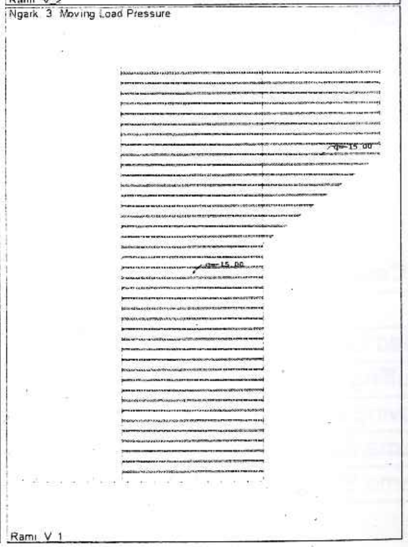


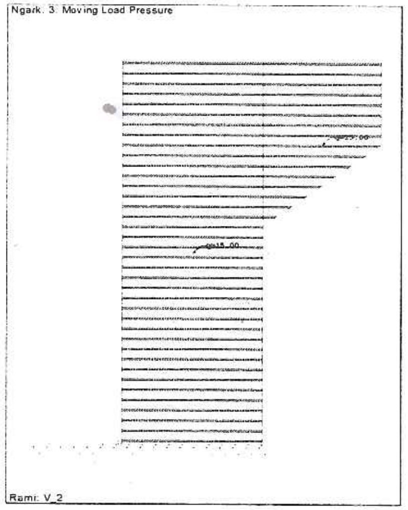












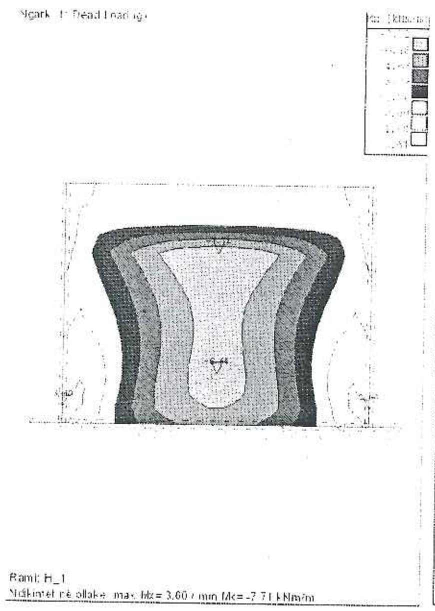
Modal analysis

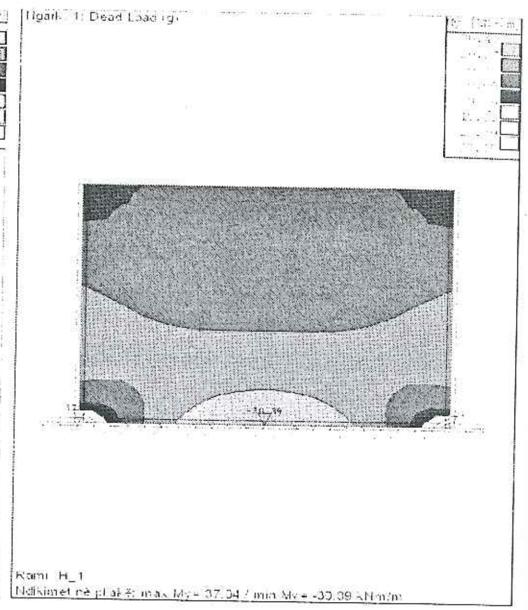
Loads Factor for mass calculations

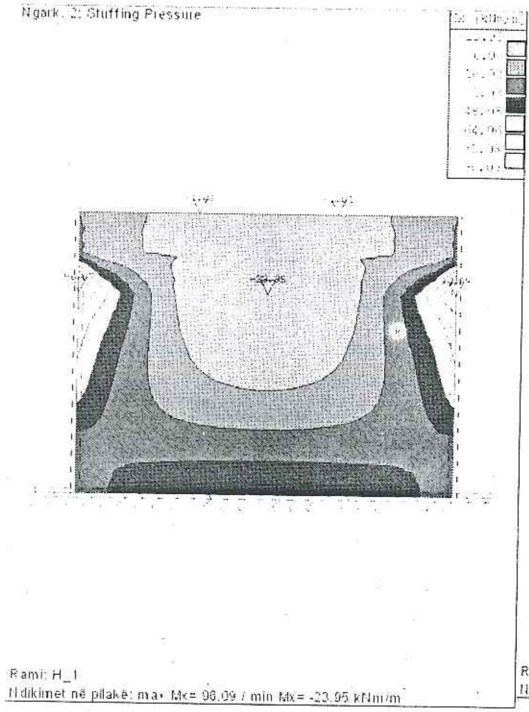
	Coefficients
Dead Load (g)	1.00
Stuffing Pressure	0.00
Moving Load Pressure	0.00

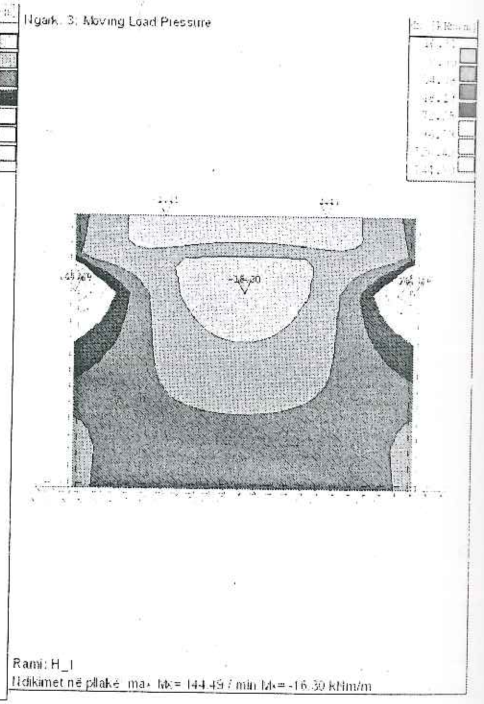
No	T [s]	f [Hz]
1	0.4932	2.0275
2	0.2744	3.6446
3	0.2156	4.6380

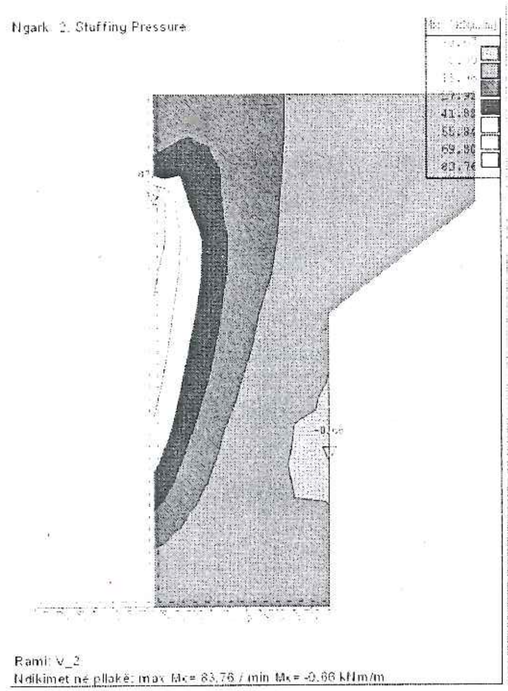
(Static Design), forces, displacements

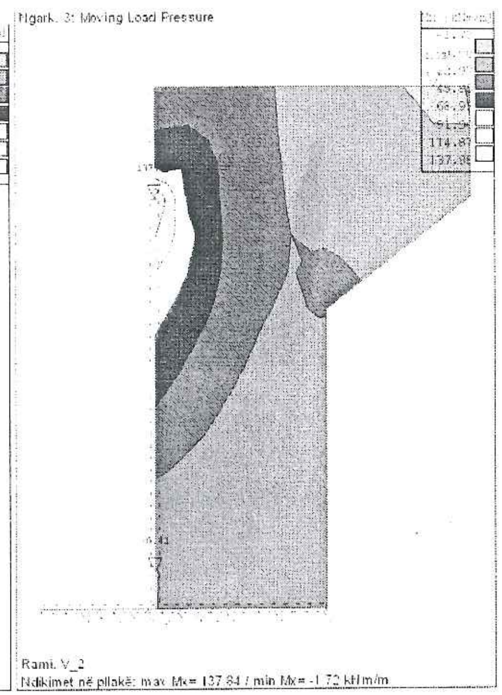


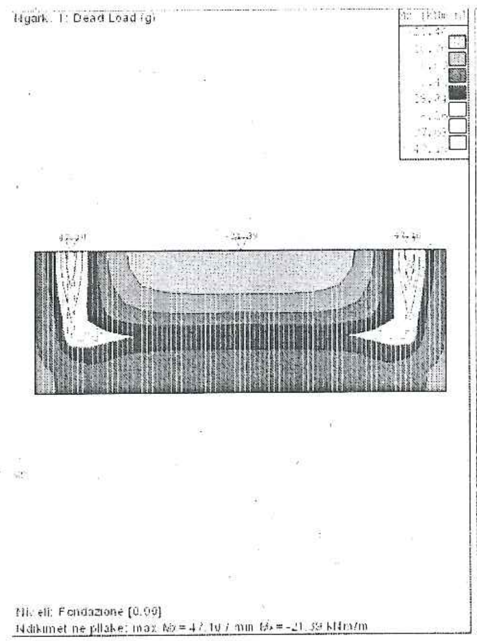


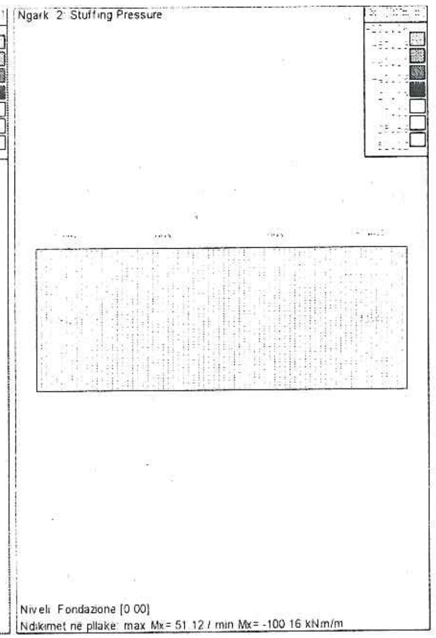




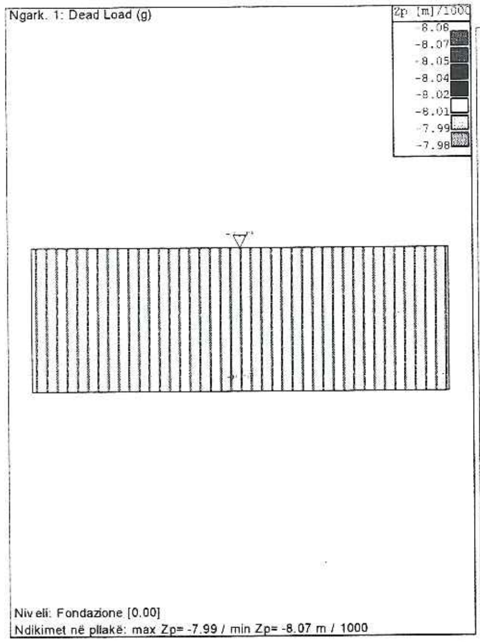


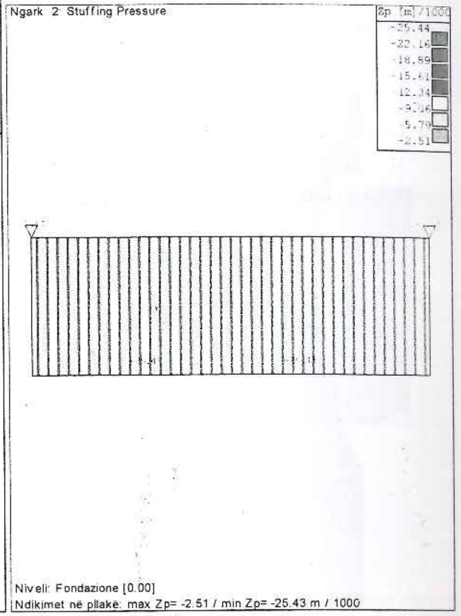


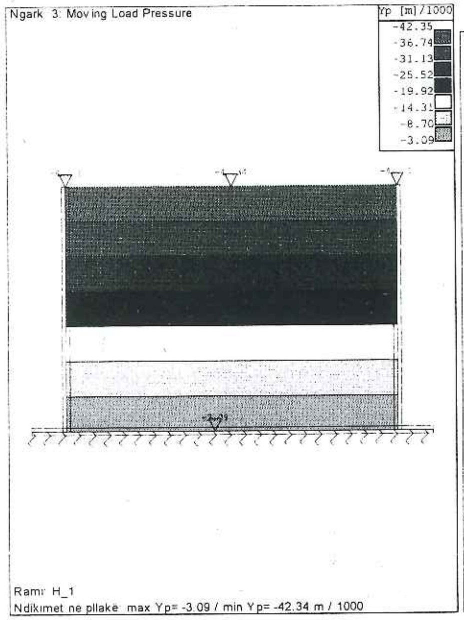


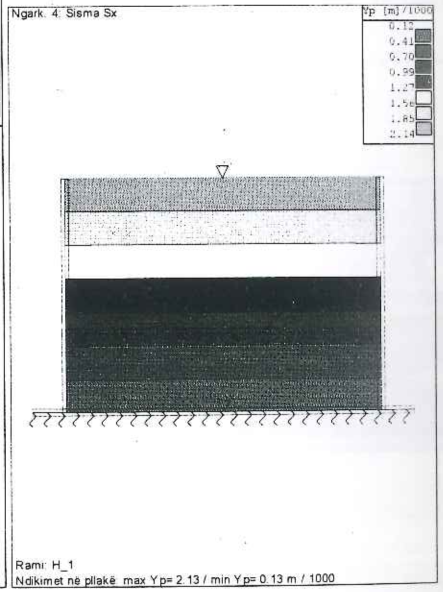


Displacements









Combinations loads – by EUROCODE

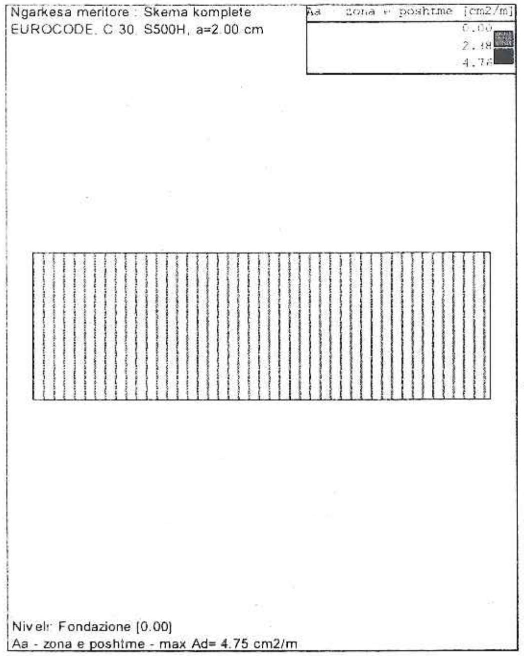
Loads cases

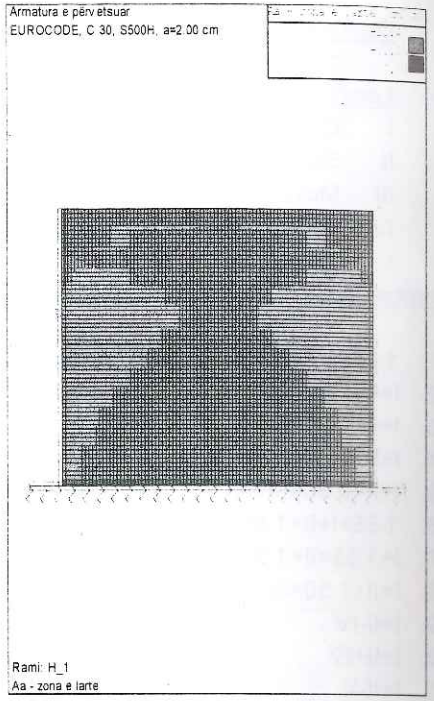
I Dead Load (g) - <Dead Load>
II Stuffing Pressure - <Permanent loads>
III Moving Load Pressure - <Shfrytëzuese - A>
IV Sisma Sx - <Seismic action>
V Sisma Sy - < Seismic action >

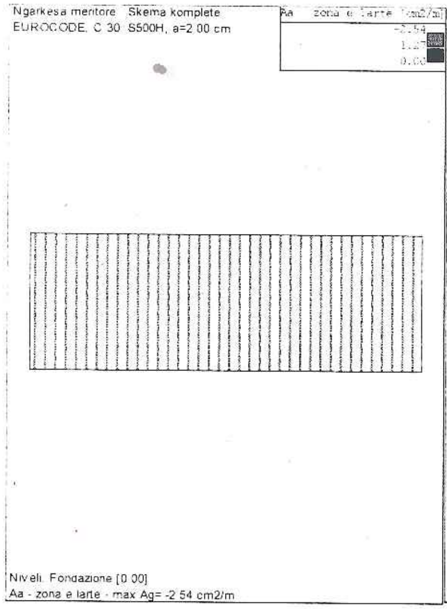
Combinations

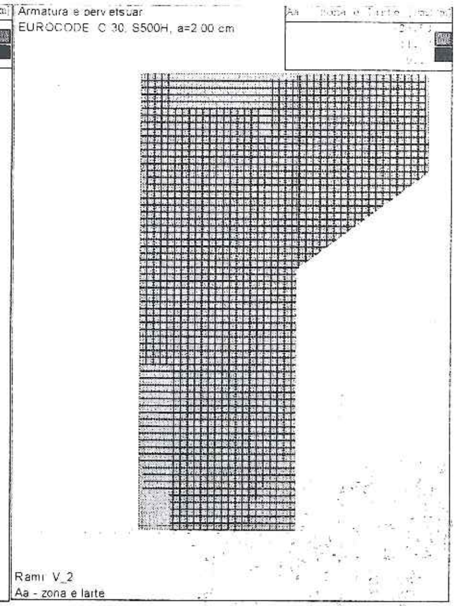
01. 1.35×I+1.35×II+1.50×III
02. I+II+0.30×III+V
03. I+II+0.30×III-V
04. I+II+0.30×III-IV
05. I+II+0.30×III+IV
06. 1.35×I+II+1.50×III
07. I+1.35×II+1.50×III
08. I+II+1.50×III
09. I+II-IV
10. I+II+IV
11. I+II-V
12. I+II+V
13. 1.35×I+1.35×II
14. I+1.35×II
15. 1.35×I+II

16. I+II









REFERENCE:

1: Bozo L. 2007, Soil mechanics, University Books Edition, Tirana, Albania.
2. Carlo VIGGIANI, Fondazioni.
3. Priyantha JAYAWICKRAMA, University Books, Texas Tech University.

European Conference of Young Geotechnical Engineers
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Bearing capacity of shallow foundations

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Abstract
Many constructions built in the ancient times exist on our days, but even more have been destroyed and one of the reasons is the insufficiency of foundations. This is a very common problem; one reason is the wrong calculation of the foundations and another is the unfamiliarity accurate responses to land. In the realization of different constructions and the works of art is very important the design of the foundations that in this study will be based on the calculation of the bearing capacity of the shallow foundations with the equations of the different authors (stability problems).
"The problems of soil mechanics can be divided into two principal groups – stability problems and elasticity problems" -Karl Terzaghi, 1943 Father of modern soil mechanics

Key words
Bearing capacity; shear failure; E; γ; φ; allowable bearing capacity; stability problems; factor of safety; groundwater table effect.

Introduction
In Albania a common problem is that a lot of construction firms haven't drilled to take soil samples needed for the design of the foundation, so we have a false design or better said an incorrect one. The consequences from these are the different modes of failure as:
1. General shear failure
2. Local shear failure
3. Punching shear failure

In this study the bearing capacity will be calculated with the bearing capacity formulas of four different authors. They are: Terzaghi; Vesic; Hansen; Meyerhof. We will see the differences between the authors; which author is the most economic; which is the influence of the foundation dimensions; the depth influence on the bearing capacity and the influence of the ground conditions.
So at the end, which influences the most?
How do we estimate the maximum bearing pressure that the soil can withstand before failure occurs?
Bearing capacity of shallow foundations depends on and can be determined by these factors:

1. Soil Strength Parameters
2. Dimensions of foundation
3. Groundwater Effects

Important too:

4. Foundation shape (cost and labor)
5. Moment loads and eccentricity
6. Weight of the foundations

The case of study
The case of study is the overpass in Durres-Morine highway, we will analyze the ground conditions where will be placed its foundation. This overpass is located in the 1+600 km of the new highway.
In order to define the geological and geotechnical conditions at this site three borings at 20.0m depth (BH-331, BH-332, BH-333,) were performed.

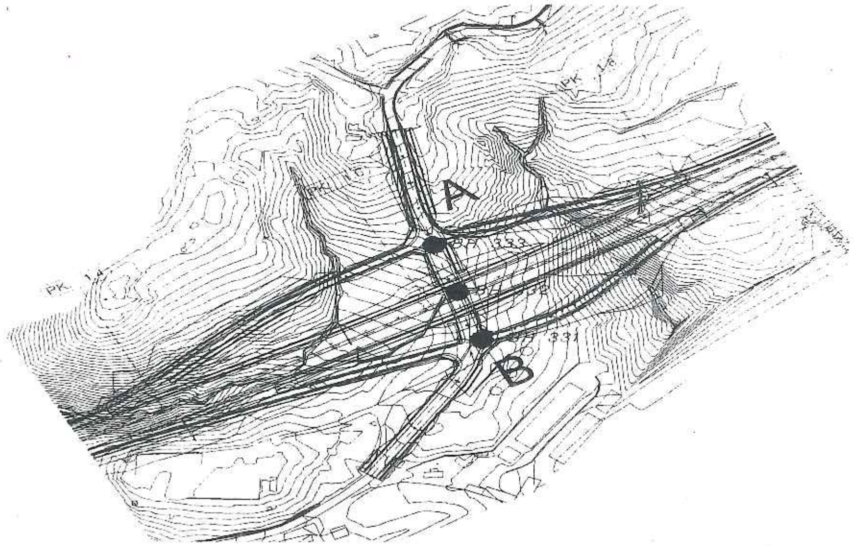

Fig.1 The overpass place

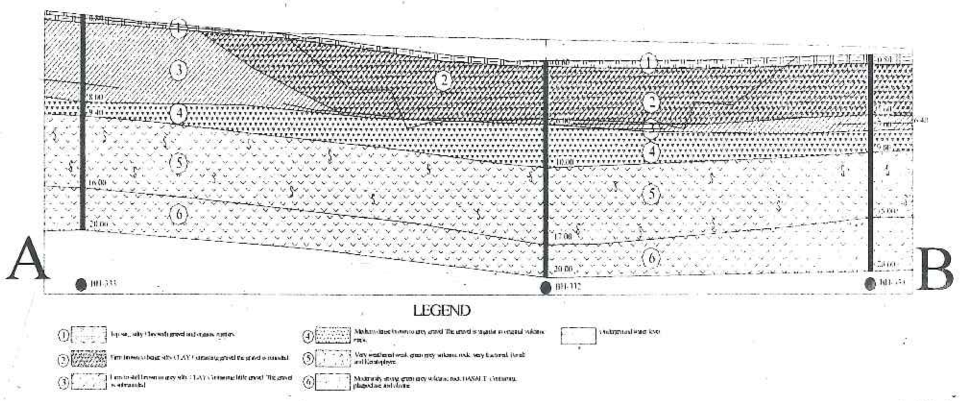

Fig.2 The geological section

3.Comparison of the four authors: Terzaghi;Vesic;Hansen;Meyerhof.

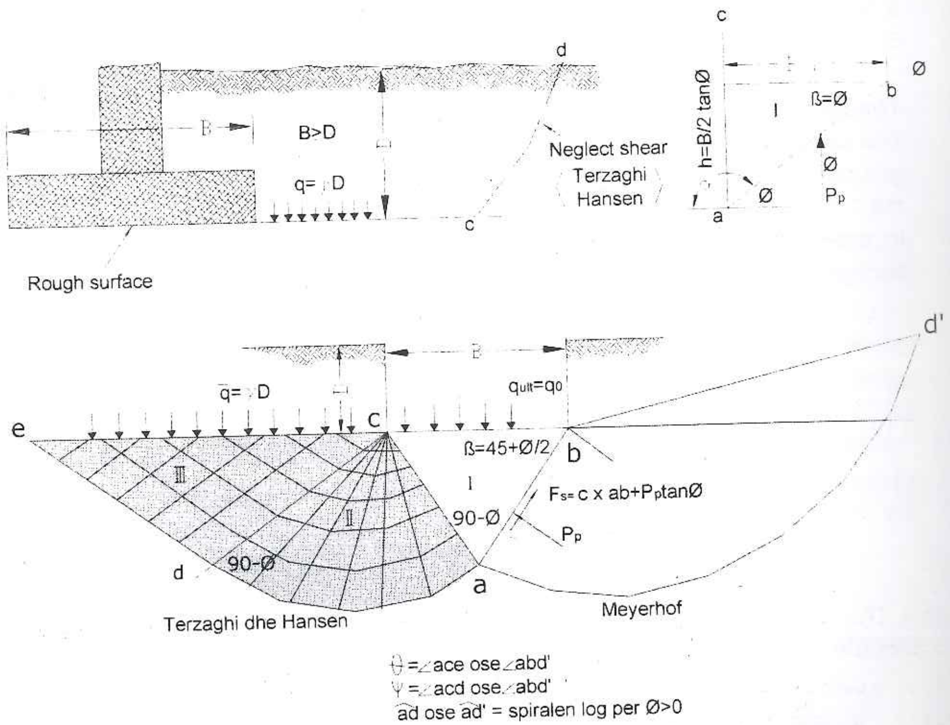

Fig.3 Comparison scheme

Based in the different models of the four authors and in their formulas we have calculated the bearing capacity of shallow foundations with B=2÷20 m and D=2 m.

Allowable bearing capacity

$$q_a = \frac{q_{ult}}{F} \quad (1)$$

qa ..... allowable bearing capacity
F ..... factor of safety

In table 1 shows the values of factor of safety.
In table 2 shows the values of allowable bearing capacity for layer No.2.

Tab.1 Minimum factor of safety

			Design Factor of safety F	
Categ.	Typical Structures	Characteristics of the category	Thorough and complete soil exploration	Limited soil exploration
A	Railway bridges,warehouses, blast furnaces,hydraulic, retaining walls,silos.	Maximum design load likely to occur often,consequences of failure disastrous	3.0	4.0
B	Highway bridges,light industrial and public buildings.	Maximum design loads may occur occasionally,consequences of failure serious	2.5	3.5
C	Apartment and office buildings	Maximum design load unlikely to occur.	2.0	3.0

Tab.2 The bearing capacity values calculated with the equations of the four authors above

Bearing capacity of the layer no.2 , depth D=2 calculated with the equations of the 4 authors above.

B (m)	4	8	12	16	20
qa=kN/m2	qa=kN/m2	qa=kN/m2	qa=kN/m2	qa=kN/m2	qa=kN/m2
Meyerhof	300	317	340	366	392
Hansen	305	299	306	317	329
Terzaghi	281	312	340	367	395
Vesic	329	335	354	376	399

From the results of table 2 we have compared the authors; in the comparison chart we can see the dependence of the bearing capacity from the width "B" for a constant value of depth D=2 m

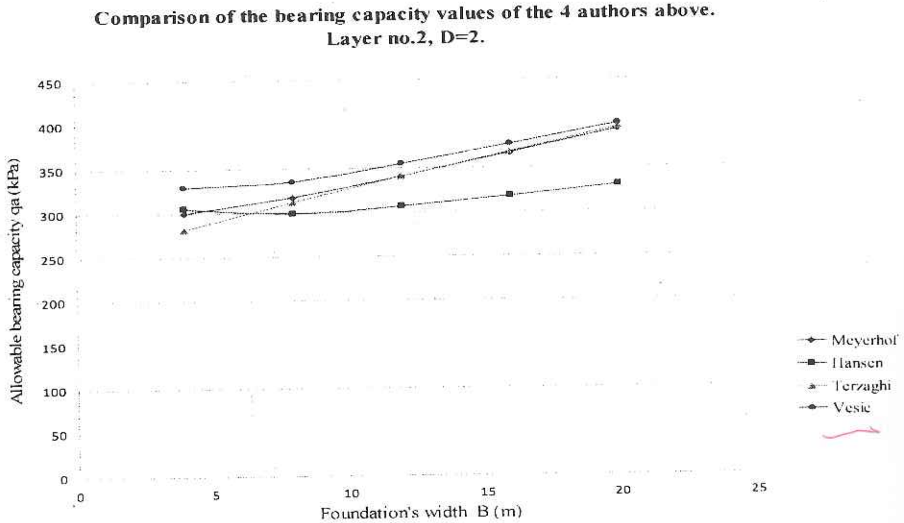

Fig.4 Comparison chart of the four authors.

4. Comparison of the different layers (terzaghi's equations)

In fig.5 ÷ fig.7 we have given the bearing capacity values for layers No.2; No.3 and No.4 and their dependence by width of foundations "B" and depth "D

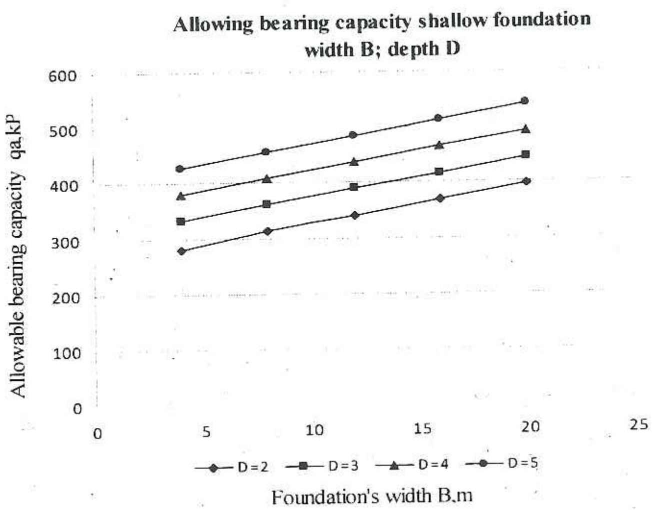

Fig.5 Bearing capacity for layer No.2 for B=4 ÷ 20m; D=2 ÷ 5m

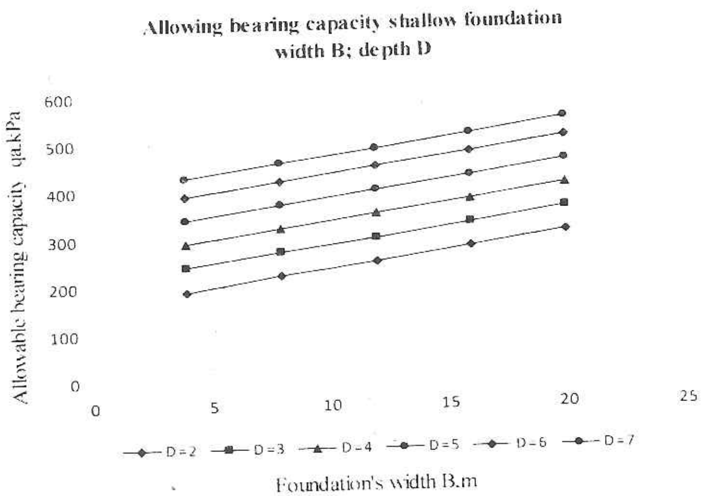

Fig.6 Bearing capacity for layer No.3 for B=4 ÷ 20m; D=2 ÷ 7m

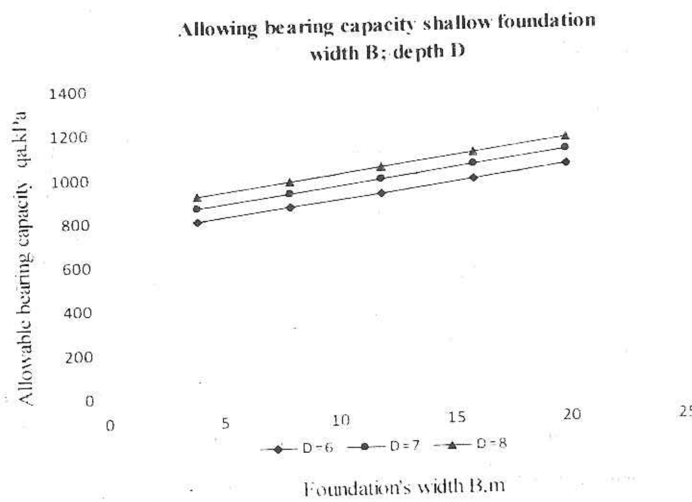

Fig.7 Bearing capacity for layer No.4 for B=4 ÷ 20m; D=6 ÷ 8m

Bearing capacities of shallow foundations are calculated for three soil layers; No.2. No.3, No.4.

The considered foundation depth D varies from 2.0 ÷ 5.0m, for foundations found on layer No.2, from 2.0 ÷ 6.0m for foundations found on layer No.3 and from 6.0 ÷ 8.0m for foundations found on layer No.4. Bearing capacity is related to foundation width B, which varies from 4 ÷ 20m

1. The geological and geotechnical conditions for layer No.2 are: C=16 kPa; φ=20°; γ=21.7 kN/m^3 Dw =6.4 m.
2. The geological and geotechnical conditions for layer No.3 are: C=3 kPa; φ=20°; γ=22.5 kN/m Dw =6.4 m.
3. The geological and geotechnical conditions for layer No.4 are: C=19 kPa; φ=24°; γ=23.8 kN/m^3 Dw =6.4 m.

Conclusions

1. The most economic author? Certainly Hansen is the less economic. He is more conservative.
2. Vesic; Meyerhof and Terzaghi give very large values. For the three of them, there is a point when the values merge. In this study we find that Vesic's formulas give the most economic foundation.

3. Bearing capacity depends from soil characteristics "C" and "φ". Also large values of "C" and "φ" gives large values of bearing capacity for the same values of width "B" and depth "D". We see that the two layers, layer No.2 and layer No.3 have an equal value of friction angle"φ"; and for the same value of B=4 and the same value of D=2 the allowable bearing capacity for the layer No.2 is: q_a=281 kPa; for layer No.3 the bearing capacity value is q_a=187 kPa. The difference of the bearing capacities is 94 kPa. Analyzing the different values of these two layers for the same values of B; D we can say:
   In this case for $\varphi_2 = \varphi_3$ and $C_2 \sim 5C_3$ the values of the bearing capacity vary approximately with the difference between them from (85 ÷ 95) kPa; $q_{a2} \sim 1.5\ q_{a3}$

4. Studying the values of q_a we find that the dimensions of the foundation also affect more on them. We find that depth D has a bigger influence. Increasing the depth from D to D+1m..........q_a increases 10 ÷ 15 % for layers No.2;No.3 and it increases ~5% for layer No.4. Increasing the width from B to B+1m...........q_a increases 2.3÷3.5 % for layer No.2; 3.4÷5.3 % for layer No.3; 2.8÷3.8 % for layer No.4 .

5. Allowable bearing capacity depends also from factor of safety "F"
   Factor of Safety depends on:

- Type of soil
- Level of Uncertainty in Soil Strength
- Importance of structure and consequences of failure

- Likelihood of design load occurrence

REFERENCE:

[1] Reference No.1: Bozo L. 2007, Soil mechanics, University Books Edition, Tirana, Albania.
[2] Reference No.2: Braja M. DAS 2006, Principles of Foundation Engineering.
[3] Reference No.3: Joseph BOWELS 1995, Foundations Analysis and Desing.
[4] Reference No.4: Carlo VIGGIANI, Fondazioni.
[5] Reference No.5: Priyantha JAYAWICKRAMA, University Books, Texas Tech University.

List of authors

Authors	Year of participation							
1.Ahmetaj L	2005	2006	2007					
2.Allkja S	2005	2006	2007	2009				
3.Alliu Xh	2005	2010						
4.Balilaj M	2005							
5.Bozo L	2002	2003	2004	2005	2006	2007	2008	2009	2010
6.Bukaçi E	2007	2010						
7.Cela K	2006							
8.Cobani E	2010							
9.Duni LL	2010							
10.Durmishi C	2002							
11.Frasheri A	2007							
12.Goga K	2003							
13.Goxhaj E	2010							
14.Harizaj L	2009	2010						
15.Ikonomi GJ	2009	2010						
16.Kuka N	2010							
17.Lako A	2010							
18.Lila B	2010							
19.Muceku Y	2002	2005	2007	2008	2009			
20.Marko O	2010							
21.Paci E	2007	2008						
22.Starja K	2010							
23.Sharra L	2010							
24.Shkodrani N	2005							
25.Themeli A	2007							
26.Zeqo A	2005							