NUMERICS IN GEOTECHNICS
and STRUCTURES

Thomas Zimmermann
Andrzej Truty
Krzysztof Podleś

1985-2010, 25 years Z_SOIL®.PC
NUMERICS IN GEOTECHNICS
and STRUCTURES
2010

Edited by

Thomas Zimmermann
Andrzej Truty
Krzysztof Podleś

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Preface

This book reunites a selection of contributions presented at the yearly meeting on “NUMERICS IN GEOTECHNICS & STRUCTURES”, which took place on September 1, 2010, at the Swiss Federal Institute of Technology, in Lausanne.

2010 marks the 25th anniversary of ZSOIL.PC, which came out in early 1985 after three years of intensive development at Zace Services Ltd, in Switzerland. The initial software development started in 1982, under supervision of Thomas Zimmermann, following inspiring discussions with F. Vuilleumier (BG ingénieurs conseils SA) and B. Saugy (EPFL). A number of people joined the development team in the following years, and collaborations with the Swiss Federal Institute of Technology and several major engineering companies led to the software available today, which includes fully coupled two-phase media, thermal and humidity diffusion, full 3D capabilities, including structures, and dynamic analysis.

The present Version 10, developed under supervision of A. Truty and Th. Zimmermann is now available, it introduces general dynamic capabilities, novel constitutive models, and full support of 64bits OS systems with a significant increase of speed and problem size.

With hundreds of installations active worldwide in daily practice, network installations for students in many universities, dozens of new student versions authorized every month, and thousands of real-world cases studied, some of them illustrated in this book, ZSOIL.PC has become one of the most powerful tools for the modern civil engineer.

This 25th anniversary celebration gives a new opportunity to distinguish outstanding contributions with ZSOIL awards, this year’s laureates are:

Dr Stéphane Commend, for his contributions to the development of stabilized formulations for geomechanics,

Dr Rafał Obrzud, for his contributions to the development of constitutive approaches and parameter identification for geomechanics,

Dr Matthias Preisig, for his contributions to the development of meshfree formulations for geomechanics.
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Scientific support of the following laboratories of the Swiss Federal Institute of Technology is acknowledged: the Laboratory of Structural and Continuum Mechanics (Profs. F. Frey & Th. Zimmermann), the Laboratory of Rock Mechanics (Prof. F. Descoëndres) and the Laboratory of Soil Mechanics (Profs. L. Vulliet & L. Laloui).

Financial support of the Swiss Commission for Technology and Innovation (CTI) under grants 2672.1, 2387.1 and 2995.1, for version 4, and grant 4182.1, for version 6, is acknowledged.

Advice on finite element methodologies provided by T. J. R. Hughes (ICES, Austin TX) since the early days has often been crucial, his advice is gratefully acknowledged. The many inspiring discussions with J. P. Wolf (EPFL) throughout the years, with J.-H. Prevost (Princeton Univ.) at the early stage of the project, and the fruitful collaborations with W. Ammann (GRF Davos), P. Roelfstra, E. Spacone (University "G. D'Annunzio" of Chieti-Pescara), D. Eyheramendy (Ecole Centrale de Marseille), and S. Yufin (Moscow State University of Civil Engineering) are also gratefully acknowledged.

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Andrzej Truty, Cracow
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Preface

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Risks, Disasters, Crisis and Global Change – From Threats to Sustainable Opportunities

Walter J. Ammann, Global Risk Forum GRF Davos, Promenade 35, CH- 7270 Davos, Switzerland
walter.ammann@grforum.org, www.grforum.org

Keywords: risks, disasters, crisis and global change

Abstract
Problems related to the dynamics of population pressure and mobility, the various facets of globalization, economic and financial crises, climate change, environmental degradation and erosion of ecosystem services, health and food issues, urban risks and the issue of very large-scale disasters and megacatastrophes are dominating the development agenda. The portfolio of risks, disasters and security is getting more and more complex and interwoven. Risk prevention, resilience, disaster preparedness, risk management, and risk governance are getting crucial elements for a sustainable development.

The globally growing number of crises and disasters, and the more and more intricate, complex and multifaceted nature of risks require an innovative, integrated and problem-oriented approach to risk and disaster knowledge and management. If humanity is to steer our planet, its biosphere and human civilization into a more sustainable future, the issues of risk, security, development and sustainability ought to be integrated — in science, policy, management, and business practice. It is only through a holistic perspective that the complexity of the risk theme and its entire phenomenology can be addressed, understood, conceptualized, and tackled.

Engineering sciences, known for problem solving, can substantially contribute to reduce threats and turn them into sustainable opportunities.

1. Introduction
The increasing world population, coupled with globalization and urbanization, has greatly increased the risks and the impact of disasters. Climate change and man-made land degradation aggravates the situation in terms of intensity, occurrence and complexity. Recent disasters, such as the Asian Tsunami, Hurricane Katrina, the earthquakes in China, Haiti, or Chile, the floods in Pakistan and many more events confirm this fact. Emerging
trends from risk management confirm the instincts of the general public: the world we live in today is more complex, more vulnerable and more interdependent than at any time before in history. When settlements or infrastructures overlap with hazard prone areas, natural events can cause significant damage, thus natural hazards limit the availability of living space causing social costs. Studies from the World Bank (World Bank 2005, Global Facility 2007) have shown that more than 3.5 billion people and about 80% of the world’s gross domestic product are located in areas exposed to at least one natural hazard with a significant probability of occurrence.

Numerous catastrophes of the last few years have highlighted clear limits to how far life, limb and property can be protected. The protection of life is certainly the primary concern, but also economic damage has to be reduced as to protect vital economic growth, especially in developing countries. Sustainable development and poverty reduction go hand in hand with disaster risk reduction strategies to achieve the UN Millennium Development Goals (UN MDGs). Disasters and risks should no longer be seen as an isolated humanitarian affair but as an integrated part of both, sustainable development and climate change adaptation strategies. Poverty, famine, diseases, lack of education, forced (environmental) migration and under-development pose clear risks to the global society and undermine its resilience.

The World Conference for Disaster Risk Reduction held in Kobe, Japan, in January 2005 (WCDR 2005) brought consensus that to achieve risk-resilient, sustainable societies, the management of unexpected events - such as natural hazards, climate change, pandemics and diseases, man-made hazards or terrorism - has to be approached in an integrated way. As a result, the “Hyogo Framework for Action 2005-2015: Building the Resilience of Nations and Communities to Disasters” HFA (UN-ISDR 2005) was approved by the 168 government representatives gathered in Kobe. Under the Hyogo Framework for Action, the governments committed to make disaster risk reduction a priority by raising awareness, reducing risks, and being prepared and ready to act in case of an emergency.

2. **Disaster risk reduction DRR**

Disaster risk reduction (DRR) refers to a wide range of work on risk reduction and disaster management. Risk reduction includes prevention, preparedness, and part of the recovery process and focus on reducing vulnerabilities. Disaster management includes early warning, emergency management, response and part of recovery and focus on increasing resilience. DRR aims at limiting risks, if not avoiding risks at all, and concentrates on minimizing the adverse impacts on disasters – both within the broad context of sustainable development. Disaster risk reduction thus has two main directions:
• **Risk reduction** to be understood as how to reduce or limit the risk due to a hazardous situation. This can be achieved by good prevention

• **Disaster management** to be understood as how to reduce or limit the resulting damages of a disaster. This can be achieved by good preparedness, an efficient disaster crisis management and an effective recovery process (disaster management).

DRR is thus a process of both, risk reduction and disaster management and is also often called integral or integrative risk management (IRM – Ammann 2006). Next to natural hazards risk, currently aggravated by climate change, IRM includes numerous other risks to be considered simultaneously such as risks of a technical, biological and chemical nature, pandemics, terrorism, financial risks, etc.

DRR is not only a multi-risk approach but also a multi-stakeholder approach. Although the HFA recognizes that governments have the primary responsibility for guiding the implementation of measures for the disaster risk reduction, to create the necessary political will in national terms, a wide group of risk management experts, practitioners, scientists and key players from civil society and other sectors with a strong emphasis on implementation at “the last mile” has to be involved and has to interact with key players from line ministries and disaster management authorities.

Practice, science, policy and decision making have to be closely linked in the search for sustainable solutions for the complex risks society is facing today. Only an interdisciplinary approach can secure the proper bridging between problems and their main causes on the one hand, and governance and technology perspectives of problem solving on the other hand. Demand-driven, practical application has to overcome purely supply-driven scientific knowledge. The task of protecting people as well as of private and public goods has to be taken on in this knowledge, and achieved in a sustainable manner.

### 3. Climate change and DRR

As climate change is aggravating the meteorological hazards in terms of frequency, intensity and interdependency, the measures, and in particular measures for climate change adaptation (CCA) have to be closely linked to DRR measures. The harmonization of DRR and CCA measures will become crucial for the future. The review of the achievements of the Millennium Development Goals (MDGs) by the UN General Assembly in September 2010 also has revealed that climate change, its root causes, impacts and response options needs to be part of the equation for achieving the MDGs. Climate change and the MDGs are intrinsically linked and deeply interdependent. The very cause of anthropogenic climate change runs counter to global sustainable development and therefore to the MDGs.
Climate change is not only about the environment. It results in growing risks and vulnerabilities, particularly in developing countries. Desertification and lack of natural resources will cause massive (environmental) migration, asking for innovative approaches to more North-South balance in global climate negotiations. This means large costs for wealth, development and human security. Climate change thus severely undermines the achievement of the MDGs. We risk derailment of the MDGs if we fail to mitigate and adapt to climate change effectively. Effective mitigation is largely linked with green growth. Social and economic opportunities deriving from such new trajectories can help developing nations.

At the same time, effective and sustainable adaptation policies must be identified to assist poor and underdeveloped countries, particularly in the most vulnerable regions (Africa, Asia, and Small Island States). As most of the devastating disasters – besides earthquakes - are of meteorological origin – thus climate change driven, adaptation is also disaster prevention. Both strategies have to be linked and harmonized. Such policies can help attain the MDGs in a variety of ways.

4. Integrative risk management, risk culture and risk governance

The key questions are: How do we move to a safer world and how can our current know-how support this change process? The approach must be that of integrative risk management across subject areas, professions, and sectors, encompassing natural and social sciences, engineering, and scientific understanding with business, policy responses, and citizen participation. Stronger ties with adequate public-private-partnership models have to be built among risk management communities, and approaches should be devised to move towards a more truly integrated way of thinking about risk: a holistic approach to risk reduction with safety, security and sustainability at the center. This is an approach that will help policy makers and business people, risk managers and civil society to address the complex risks around them more effectively.

Dealing with natural hazards is not just complex, but also contradictory when technical, social, economic, and ecological aspects have to be balanced. It is no longer adequate for risk management professionals to focus solely on the risk within a particular realm. Rather, in a world with interdependent systems of rapidly growing complexity (critical infrastructures, processes and services as examples), risk management must have a new vision that overcomes boundaries between subject areas, one that reaches across specialism and departments. Safety and security have to be seen as a whole, enabling better planning, response, and reduction of the most pressing risks.

To be able to take effective and efficient decisions for disaster risk
reduction and climate change adaptation measures, which lead to transparent and comparable results in different risk situations, a consistent and systematic risk management approach has to be followed. Hereafter, this approach will be called “integrative risk management”, a process that contains a systematic framework for the risk analysis and risk assessment procedures, finally leading to consistent decisions and to an optimized, integral planning of risk reduction measures. A consistent risk concept provides a substantial base and allows the comparison of various risk scenarios at different locations and originating from different natural disasters. Therefore, a risk based management, instead of a purely hazard related approach, is the key for the future. A significant driving force for this paradigm shift is the demand for accountability and improved effectiveness of risk reduction measures.

The public perception of natural hazards differs from the perception of ecological, technical or social risks leading to conflicting security philosophies, which hinders consensus on integrated measures. Different ways in which people perceive risks have an important effect on how they will accept any imposed measures. A strategy for the protection from natural disasters has to find a way to put the various risks onto a common scale to allow for comparability and to serve as a platform from which measures can be agreed upon. Any risk to humans and the environment has to be considered within the context of social, financial and economic consequences and increased interdependencies between the various risks. The way a society handles questions of safety and security may be summarized with the term “risk culture”. Risk culture emphasizes that insecurity can only be controlled by risk-oriented thinking.

Risk governance looks at how risk-related decision-making unfolds when a multitude of stakeholders or actors is involved, requiring co-ordination and possibly reconciliation between a profusion of roles, perspectives, goals and activities. Good risk governance stands for transparency in decision-making, effectiveness and efficiency of the measures, accountability, strategic focus, sustainability, equity and fairness, respect for the law and the need for the solution to be politically and legally realizable as well as ethically and publicly acceptable. Integrative risk management and good risk governance are complicated by the fact that many risks of today’s society are not isolated, single events with limited extent, but are often trans-boundary risks affecting countries with different political systems and coping strategies.

5. Framework for DRR and climate change adaptation CCA

The concept for DRR, i.e. the concept of the integrative risk management is shown in Fig. 1. The integral risk management starts with the risk identification and risk analysis process answering the question “What can happen?”, followed by the risk assessment, answering the question “What is
accepted to happen?” and leading to the planning of measures. The ultimate objective is to plan and implement protective measures. The main criterion for choosing the correct protective measures is cost-effectiveness. DRR (and CCA) have to overcome a number of problems and facts:

- The risk-oriented approach and the methodology of dealing with uncertainties. This applies both to the analysis and the assessment of risk.
- The limits to safety efforts versus the expectations of the civil society.
- The various points of view, attitudes, and values of all stakeholders involved and affected by the risk.
- Disaster risk prevention and mitigation measures have to take the whole set of pre- and post-disaster measures into consideration, as well as measures during the event itself or risk transfer by insurance (Fig. 4).
- All solutions have to fulfil the criteria of sustainability, i.e. a sustainable way in disaster risk management has to be a socially, economically, and environmentally equilibrated approach.
- Integral risk management also needs a strategic and systematic process of controlling, including the periodic evaluation of the risk situation and a comprehensive risk dialogue between all stakeholders.
- The need for dialogue and communication to ensure the participation of all stakeholders, when setting limits for the protection and defining the processes of decision-making. Risk communication can have a major impact on how well society is prepared to cope with risks and how people react to crises and disasters.
- Find a balance between accepted residual risk and economic costs of risk reducing measures

6. Risk Concept

To be able to compare different types of natural hazards and their related risks and to design adequate risk reduction measures a consistent and systematic approach has to be established (Fig. 1). The risk concept represents the methodological base for an integral risk management, for the decision-making process of risk reduction and mitigation and for disaster management, and serves as a transparent base for the risk dialogue between all stakeholders (Ammann, 2006). The basic principles of the risk concept are represented in Fig. 2 and can be summarized by the following key questions:

- How safe is safe enough?
- What can happen?
- What is acceptable (to happen)?
- What needs to be done?
The question “*What can happen?*” has to be answered by a risk analysis procedure, the question “*What is acceptable?*” by the risk assessment. The necessary steps are summarized in Fig. 3. The goal of a risk analysis is the most objective identification of the risk factors for a specific damage event, object or area. The question „*What can happen?*“ has to be answered considering a variety of influencing factors.
Risk assessment aims for an explicitly subjective answer to the question “What is acceptable?”; thus inquiring how big a residual risk is acceptable. Risk assessment is by nature very complex and has to deal with the fact, that risk is a mental construct but not fully rational. One important aspect is the risk aversion towards disastrous events: people’s wish to prevent large events rises over-proportionally to the event’s real consequences. The acceptance of a risk also depends on whether it is taken by active choice or not. Risk categories are defined to the extent of self-reliance being deployed.

Risk assessment is closely linked with the pursued protection goals. A protection goal is a set of criteria for the implementation of the primary goals of all efforts to improve the safety and represents the limits of the acceptable risk level, i.e. define how far the measures should go. A protection goal has different meanings as it has to cover individual and collective perspectives. A person’s protection goal is often defined in terms of death toll probabilities. The marginal costs (Ammann 2006) for the safety measures have proven to be the most useful protection goal definition for the collective, societal perspective. Those marginal costs represent certain expenses per avoided fatality or per saved human life. The safety measures can be increased until the aimed risk level is reached.

Determining the marginal costs of an avoided fatality can lead to the misunderstanding that a price gets allocated to a human life. The criterion of marginal costs should be seen as the optimization of safety measures in terms of saved lives within the limitations of available means and resources.

The planning of measures serves the identification of measures that are necessary and appropriate to reach the protection goals. The main function of the planning of integrated measures is to achieve the intended level of safety in the most cost-effective way. Organizational, technical and biological protective measures must be planned, checked for effectiveness, and undertaken in concert, while keeping in mind that prevention, intervention,
and reconstruction are all equally valid risk management measures (Fig. 4). Whereas preventive measures serve primarily to reduce the vulnerability, preparedness and intervention measures primarily serve to strengthen resilience. Further criteria such as sustainability, acceptability, feasibility, and reliability of solutions have also to be kept in mind.

![Risk Circle Diagram](image)

Figure 4. The risk circle and possible measures for risk reduction and mitigation measures

Safety measures always come along with „side effects“. The most obvious among them is the financial aspect. But aspects of ecology, of landscape protection or of land use planning can be of equal importance. The optimal coordination of all measures has to consider all relevant aspects and all activities in the field of disaster risk reduction have to obey the principles of sustainability. Measures have to be environmentally sound, consider societal preferences and be cost effective. Disaster risk reduction has also to be part of the sustainable use of natural resources and of sustainable development, and therefore, is considered a cross-cutting issue.

The socio-political aspects of sustainability are a question of development and welfare priorities and have to be seen in context with other targets such as education or health care. Especially in developing countries, a reallocation of resources is often needed after major catastrophes for recovery purposes – resources which have been allocated originally to be used for e.g. investments in education, health care, welfare. A political balance between long-term investments for prevention and short-term measures for intervention and recovery is therefore needed.

7. Risk dialogue and Strategic controlling

The integral risk management not only dictates that the measures are planned, assessed and applied in accordance with the risk concept, but also
that all those who are involved and affected are included in a comprehensive risk dialogue, in the process of the planning of protection measures. Risk communication and risk dialogue with all stakeholders have to start very timely and at an early stage it will be dominated by questions more than answers and by processes rather than solutions. A continuous, comprehensive risk dialogue is therefore of vital importance, making a contribution towards a transformation of risk management to become transparent, understandable and an affair of public trust.

Active information and communication plays a dominant role in crises situations. A well informed public will sustain a catastrophic situation much better and the risk to panic and for long term damage can be limited.

A strategic controlling periodically checks the risk situation and the costs and benefits of measures. It also has to monitor residual risks. Integral risk management shows, through the base of the risk concept, how the overlying aims can be reached, with corresponding technically, economically, societal and environmentally justifiable protection measures.

Numerous factors can increase risk in the future creating additional uncertainty. Among the most important factors that have to be considered, monitored and periodically checked are globalization, mobility, vulnerability, the spreading of populated areas and the increase in their value, sensitivity (through increasing economic interdependencies), international leisure activities, socio-political changes, and changing climate and weather patterns. Developments in the hazard and risk process flow must be followed carefully and the potential for optimization exploited. For the future, the challenge will be to understand and cope with constant change; new risk scenarios, new hazards, climate change, new social-political conditions, etc.. This means that strategies for dealing with risks due to natural hazards will have to be adapted periodically.

8. The role of the millennium development goals (MDGs)

Disaster risk reduction, climate change and the Millennium Development Goals are closely linked. The international community has to embark on an increased effort to implement the MDGs and meet the targets, and integrate these existing elements with the broader agendas of global environmental change, climate change and human climate justice in particular, safety and security, and effective risk reduction and disaster management (risk governance).

As the process of advocating, catalyzing and implementing the MDGs is moving into its third phase – ten years from their inception in the year 2000 – new and emerging challenges as well as aggravated factors will have to be taken into account and shouldered so as to keep the goals on track and ensure utmost success. Among these challenges which pose enormous risks for
international development and security, and which are often inter-linked, are certainly:

- The persistent fragility of our global financial and economic system,
- a further declined performance of ecosystem services for human well-being (loss in biodiversity, land degradation, desertification, deforestation, water scarcity)
- eroded human and food security
- the manifold challenges of global climate change (water scarcity, land degradation, forced environmental migration) with an increased number and scale of natural hazards, disasters and extreme events (weather extremes like the floods, hurricanes, heat waves, but also sea-level rise, etc.).
- progressive urbanisation and increasing vulnerability of societies, critical infrastructures and services

An agenda for enhanced MDG implementation will have to include the following objectives:

- Global inequality and its reduction should be given emphasis: an explicit call for the reduction of inequality should be paired with the objective of economic growth.
- It is a prerogative to stress the importance of cultural diversity and social values, and the principles of accountability and legitimacy for effective implementation.
- Fragile statehood, political instability and larger societal uncertainties oftentimes pose risks to MDG success and require the attention of policy-makers.
- The MDG process needs to entail effective tools and standards for risk and safety assessment and integrated risk management in a variety of areas, including natural hazards and extreme events, technology, environment, finance and trade.
- The science and technology capacity and access to education in developing countries is critically important for MDG implementation and should be stressed.

The ultimate challenge will be that policy and practice development at all levels have to seek the resilience of the social as well as the ecological spheres. Through an integrated risk governance approach, the very notion of sustainable development, i.e. the balance between economy, ecology and equity can be brought about and thus re-invigorate the MDGs while at the same time effectively address the multitude of risks imposed upon us.
9. Conclusions

Disaster risk reduction with the two-fold meaning of risk reduction and disaster management addresses the whole risk cycle of prevention, intervention and recovery. Reactive disaster management in most countries is still the focus in coping with natural hazards whereas proactive risk reduction in terms of preventive measures is politically more difficult to address and to justify – or to cite Kofi Annan, former UN SG „The benefits of prevention are not tangible; they are the disasters that did not happen“. To strengthen prevention is only possible with a risk related approach and needs a paradigm shift from hazard oriented reaction to risk related pro-action. The benefits of prevention can only made clear with a strict risk controlling process, and political majorities for prevention and climate change adaptation only be gained with continuous public awareness raising initiatives.

The international community has to embark on an increased effort to integrate the existing set of MDGs with the broader agendas of global environmental change, climate change and human climate justice in particular, safety and security, and effective risk reduction and disaster management (i.e. risk governance). Global inequality and its reduction should be given more emphasis: an explicit call for the reduction of inequality should be paired with the objective of sustainable, economic growth. Fragile statehood, political instability and larger societal uncertainties oftentimes pose risks to MDG success and global security and require the attention of policy-makers. The MDG process needs to entail effective tools and standards for risk and safety assessment and integrative risk management in a variety of areas, including those of natural hazards and extreme events, technology, environmental change, finance or trade. The Summit did not touch upon these critical questions.

Ultimately, policy and practice development at all levels has to seek the resilience of the economic as well as the social and the ecological spheres. Through an integrative risk governance approach, the very notion of sustainable development, i.e. the balance between economy, ecology and equity, can be brought about and thus re-invigorate the MDGs while at the same time effectively address the multitude of risks imposed upon us.

10. BIBLIOGRAPHY


On the use of the Hardening Soil Small Strain model in geotechnical practice

Rafał F. Obrzud
GeoMod Consulting Eng.

Keywords: Deep excavation, retaining wall, tunnel excavation, advanced soil model

Abstract
This article highlights the importance of using advanced constitutive soil models in numerical modelling as daily geotechnical practice. In this context, the Hardening Soil (HS) model is presented as the most advanced soil model implemented in the ZSoil finite element code. The article recalls the basic features of the HS model and it explains their role in numerical simulations. The role of pre-failure non-linear behaviour of soils is also discussed. An application of the HS model is illustrated on typical geotechnical problems such as retaining wall excavation, tunnel excavation and undrained loading followed by consolidation.

1. Introduction

The use of the finite element (FE) analysis has become widespread and popular in geotechnical practice as means of controlling and optimizing engineering tasks. However, the quality of any stress-strain prediction depends on the adequate model being adopted in the study. In general, a more realistic prediction of ground movements requires using the models which account for pre-failure behaviour of soil, i.e. a non-linear stress-strain relationship before reaching the ultimate state (cf. [1]). Such behaviour, mathematically modelled with non-linear elasticity, is characterized by a strong variation of soil stiffness which depends on the magnitude of strain levels occurring during construction stages. Pre-failure stiffness plays a crucial role in modelling typical geotechnical problems such as deep excavations supported by retaining walls or tunnel excavations in densely built-up urban areas.

It is commonly known that soil behaviour is not as simple as its prediction with simply-formulated linear constitutive models, which are commonly and carelessly used in numerical analyses. Complex soil behaviour which stems from the nature of the multi-phase material exhibits both elastic and plastic non-linearities. Deformations include irreversible plastic strains. Depending
on the history of loading, soil may compact or dilate, its stiffness may depend on the magnitude of stress levels, soil deformations are time-dependent, etc. In fact, soil behaviour is considered to be truly elastic in the range of small strains as schematically presented in Figure 1. In this range of strain, soil may exhibit a nonlinear stress-strain relationship. However, its stiffness is almost fully recoverable in unloading conditions. Following of pre-failure nonlinearities of soil behaviour, one may observe a strong variation of stiffness starting from very small shear strains, which cannot be reproduced by models such as the linear-elastic Mohr-Coulomb model.

Engineers who are looking for **reliable and realistic predictions** of the engineering system response should be aware that by applying linear-elastic, perfectly plastic models in the FE analysis, soil ground movements may be underestimated, which may influence the magnitude of forces which are computed for supporting structural elements. The models which account for high stiffness at very small strains concentrate the development of high amplitudes of strain around the close neighbourhood of the source of deformations, similarly to what is observed in reality. This can be the case of **retaining walls** or **tunnel excavations** where soil stiffness degrades increasing soil deformations in the close vicinity of unloaded boundaries, and appropriately reducing them away from the unloaded zone (cf. [2]).

![Figure 1 Typical representation of stiffness variation in as a function of the shear strain amplitudes; comparison with the ranges for typical geotechnical problems and different tests (based on [1] and updated by the author); SCPT - seismic cone penetration test; CPTU - piezocone penetration test; DMT - Marchetti’s dilatometer test; PMT - Pressuremeter test.](image)

Figure 1 Typical representation of stiffness variation in as a function of the shear strain amplitudes; comparison with the ranges for typical geotechnical problems and different tests (based on [1] and updated by the author); SCPT - seismic cone penetration test; CPTU - piezocone penetration test; DMT - Marchetti’s dilatometer test; PMT - Pressuremeter test.
The Hardening Soil (HS) model in its two variants HS-Standard and HS-SmallStrain can be a solution for modelling of the problems which have been listed above, as they account for most of soil behaviour features (see Section 3). Despite the mathematical complexity of the HS model, its parameters have explicit physical meaning and can be determined with conventional soil tests [3]. The present article presents a few examples of the application of this constitutive model for typical geotechnical problems.

2. Choice of the constitutive model

The finite element code ZSoil® includes a variety of soil models from simple linear elastic, perfectly plastic (e.g. Mohr Coulomb), elasto-plastic cap models (e.g. Cap, Modified Cam Clay) to advanced nonlinear-elasto-plastic cap model HS-SmallStrain. The choice of a constitutive model depends on many factors but, in general, it is related to the type of analysis that the user intends to perform, expected precision of predictions and available knowledge of soil. As regards the type of analysis, geoengineering computations can be divided into two groups (see Figure 2): (a) those whose goal is to assess bearing capacity and slope or wall stability which are related to the ultimate limit state analysis (ULS), and (b) those which are related to the limit state analysis (SLS), such as deep excavations or tunnel excavations in urban areas.

In general, as long as assessment of ULS for bearing capacity or slope stability is foreseen, the analysis may be limited to basic linear models such as the Mohr-Coulomb model (but this is not a rule). On the other hand, a precise deformation analysis requires the application of advanced constitutive models which approximate the stress-strain relation more accurately than simple linear-elastic, perfectly plastic model, and in effect, the form of displacement fields can be modelled more realistically.

Figure 2 General types of geoengineering computations.
The Hardening-Soil model realistically reproduces soil deformations, as the $\sigma$-$\varepsilon$ relation is approximated with a non-linear curve (the hyperbolic function by Duncan-Chang, for details see [7],[3]). Moreover, as the formulation of the HS model incorporates two hardening mechanisms, it is suitable for modelling both domination of shear plastic strains which can be observed in granular soils and in overconsolidated cohesive soils, as well as domination of compressive plastic strains which is typical for soft soils, see Figure 3.

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<th>CLAYS</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Dilatant, Low compressible</td>
<td>Non-dilatant, Compressible</td>
<td>Degree of Overconsolidation</td>
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<td>SLS</td>
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<td></td>
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<td></td>
<td>ULS</td>
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<td></td>
<td>Low Normal clay</td>
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<td>CAP</td>
<td>SLS</td>
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<td>Normal Soft clay</td>
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<td></td>
<td>ULS</td>
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<tr>
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<td>SLS</td>
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<td>ULS</td>
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<td>HS-Small Strain</td>
<td>ULS</td>
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<td>HS-Std</td>
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Figure 3 Recommendations for the model choice for soil type and types of analysis. Dashed line: may be used but not recommended in terms of quality of results; Solid line: can be applied; Green fill: recommended.

3. A short overview of the Hardening Soil model

3.1 Features of the model

The Hardening Soil model (HS-Standard) was designed by [2], [5] in order to reproduce basic macroscopic phenomena exhibited by soils such as:

- **densification**, i.e. a decrease of voids volume in soil due to plastic deformations,
- **stress dependent stiffness**, i.e. commonly observed phenomena of increasing stiffness modules with increasing confining stress (also related to increasing depth);
- **soil stress history**, i.e. accounting for preconsolidation effects;
- **plastic yielding**, i.e. development of irreversible strains with reaching a yield criterion;
- **dilatancy**, i.e. an occurrence of negative volumetric strains during shearing.
Contrary to other models such as the Cap model or the Modified Cam Clay (let alone the Mohr-Coulomb model), the magnitude of soil deformations can be modelled more accurately by incorporating three different input stiffness parameters which correspond to the triaxial loading stiffness \(E_{50}\), the triaxial unloading-reloading stiffness \(E_{ur}\), and the oedometer loading modulus \(E_{oed}\).

\[
q = \frac{q_f}{\sqrt{3}} \quad \text{Shear strain} \quad \varepsilon_1
\]

Figure 4 Common definitions of different moduli on a typical strain-stress curve for soil.

An enhanced version of the HS-Standard, the Hardening Soil Small model (HSSmallStrain) was formulated by Benz [6] in order to handle the commonly observed phenomena of:

- **strong stiffness variation** with increasing shear strain amplitudes in the domain of small strains (S-shape curve presented in Figure 1);
- **hysteretic, nonlinear elastic stress-strain relationship** which is applicable in the range of small strains.

These features mean that the HS-SmallStrain is able to produce more accurate and reliable approximation of displacements which can be useful for dynamic applications or in modelling unloading-conditioned problems, e.g. excavations with retaining walls or tunnel excavations.

A detailed description of the Hardening Soil model can be found in ZSoil reports [3][7].

### 3.2 Hardening mechanisms and their role in FE simulations

In the Hardening Soil model, accounting for the history of stress paths is possible thanks to **two hardening mechanisms**, i.e. isotropic and deviatoric. The first one, the volumetric plastic mechanism in the form of cap surface is introduced to account for a threshold point (preconsolidation pressure) beyond which important plastic straining occurs, characterizing a normally-
consolidated state of soil. Since the shear mechanism generates no volumetric plastic strain in the contractant domain, the model without volumetric mechanism could significantly overestimate soil stiffness in virgin compression conditions, particularly for normally- and lightly overconsolidated cohesive soils. Such a problem can be observed when using, for instance, the linear elastic-perfectly plastic Mohr-Coulomb model. The isotropic mechanism is thus very important when modelling consolidation problems related to footing or ground water lowering (see Figure 5a). In the case of a footing problem, the HS model may generate non-linear relation $\varepsilon / q$ before reaching the ultimate state even for a purely deviatoric stress path (e.g. loading in undrained conditions). Note that the ultimate state in the HS model is defined by the Mohr-Coulomb criterion. In the case of normally- and lightly overconsolidated soils two hardening mechanisms may be activated by footing-related stress paths, resulting in important plastic straining.

Another feature of the cap mechanism is that it enables a degradation of soil stiffness (i.e. secant modulus $E$) with the increasing level of strain (see Figure 6b). This feature can be useful for modelling unloading modes of soil, due to excavation for instance. It is often observed in numerical analyses that not differentiating between loading and unloading stiffness modules in the Mohr-Coulomb model may result in an unrealistic lifting of the retaining wall associated with unloading of the bottom of an excavation. A combination of input parameters for the Cap model allows the user to distinguish between loading and unloading-reloading modules for which a typical ratio is around $E_{ur}/E \approx 3 – 10$ as the ratio for compression indices $C_c / C_s$ is typically measured in oedometric tests between 0.1 and 0.4. In the Cap model, the input modulus $E$ can be considered as the unloading-reloading modulus $E_{ur}$ and the slope of the normal consolidation line $\lambda$ which controls the magnitude of current soil stiffness for a given vertical stress defines the initial loading modulus $E$, assuming that a material is in normally-consolidated state (note that for OCR = 1, the initial stress point is located on the normal consolidation line):

$$E = \frac{(1 + \nu)(1-2\nu)}{(1-\nu)} \frac{E_{oed}}{oed}$$

where

$$E_{oed} = \frac{2.303(1 + e_{ref}^{oed})}{C_c} \sigma_{oed}^{ref} \approx \frac{(1 + e_{ref}^{oed})}{\lambda} \sigma_{oed}^{ref}$$

with $E_{oed}$ denoting a tangent oedometric modulus which corresponds to a given reference oedometric (vertical) stress, and $\nu$ is the Poisson’s ratio.

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Figure 5 Typical stress paths related to footing and ground water lowering (on the left) and activation of both hardening mechanisms in the Hardening-Soil model for a footing type stress path (on the right).

Figure 6 Typical stress-strain curves obtained from triaxial compression test with different constitutive models: (left) Mohr-Coulomb model, (center) Cap model linear elasticity (at the beginning of $\varepsilon_2-q$ curve) for lightly-overconsolidated material, (right) Hardening Soil-SmallStrain model with nonlinear elasticity for lightly-overconsolidated material.

With such a simplified approach, the user may reduce the problem of exaggerated swelling of the excavation bottom, but there is still a problem with soil elements which are located behind the retaining wall. Using the Cap model which is characterized by linear elasticity, soil stiffness behind the wall may be overestimated because the stress paths before yielding remain in the linear elastic domain (refer to the direction of the stress paths for a soil
element behind the retaining wall in Figure 7a). Non-linear $\varepsilon - \sigma$ relationship before yielding can be modelled using the Hardening Soil-Standard or SmallStrain models thanks to the deviatoric hardening mechanism.

The shear (deviatoric) mechanism is introduced in order to handle the soil hardening which is induced by the plastic shear strains. The domination of plastic shear strains can be typically observed for granular materials such as sands and heavily consolidated cohesive soils. One may also expect domination of plastic shear strains for soil elements behind a retaining wall (see stress paths in Figure 7a) as it can be concluded from observations of soil settlements which may occur behind a retaining structure. Note that when using constitutive models such as Mohr-Coulomb or the Cap model, soil response behind the retaining wall is linearly elastic during excavation, and thus horizontal displacements and wall deflection may be underestimated (see Figure 8a).

![Figure 7](image_url)  
Figure 7 Typical stress paths related to infiltration and excavation problems (on the left) and activation of the deviatoric hardening mechanisms and unloading modes for excavation-related stress paths (on the right).

![Figure 8](image_url)  
Figure 8 Example of a deep excavation in Berlin Sand (for details see [2]). Comparison of model predictions and in situ data: wall deflections (left), surface settlements (right).
3.3 Model limitations

Although the HS model can be considered an advanced soil model which is able to faithfully approximate complex soil behaviour, it includes some limitations related to specific behaviour observed for certain soils. The models are not able to reproduce softening effects associated with soil dilatancy and soil destructuration (debonding of cemented particles) which can be observed, for instance, in sensitive soils.

As opposed to the HS-SmallStrain model, the HS-Standard does not account for large amplitudes of soil stiffness related to transition from very small strain to engineering strain levels ($\varepsilon = 10^{-3} - 10^{-2}$). Therefore, the user should adapt the stiffness characteristics to the strain levels which are expected to take place in conditions of the analyzed problem. Moreover, the HS-Standard model is not capable to reproduce hysteretic soil behaviour observed during cycling loading.

As an enhanced version of the HS-Standard model, HS-SmallStrain accounts for small strain stiffness and therefore, it can be used to some extent to model hysteretic soil behaviour under cyclic loading conditions. A recent extension of the HS model which has been implemented in ZSoil (the densification model by Zienkiewicz [10]) includes liquefaction effects.

4. Practical applications of the HS model

4.1 Retaining wall excavations

The differences in predictions between the HS model and the basic Mohr-Coulomb model can be illustrated on a FE model of a deep excavation in Berlin Sand - a benchmark problem distributed by [8]. Details of FE modelling for this example can be found in the ZSoil technical report [7]. Figure 9a demonstrates the results of computing with the Mohr-Coulomb model and the effect of wall lifting which has been discussed in Section 3.2. On the other hand, realistic settlements behind the retaining wall, as well as, expected directions of wall displacements are obtained with the aid of the Hardening Soil model.

Considering mathematical complexity of the Hardening-Soil model, one may expect an increased computational effort. Indeed, the user may observe an increased number of iterations for each computational time increment comparing to modelling using simple models, but in times of fast personal computers this factor does not play an important role.
The example of a complex 3D modelling of 15-20-meter deep excavation is presented in Figure 10. Despite a large number of continuum elements and structural elements like shells and trusses, computing of a phase-by-phase excavation took about 5 hours on a 32-bit system with the aid of 4-core processor and 4Gb of RAM memory.
Figure 10 3D modelling of retaining wall excavation in Montreux (GeoMod SA archives).

Figure 11 shows a 2D section for the FE model presented in Figure 10. It can be again observed that the use of the HS model allows reliable modelling of the settlements behind the excavation wall, as well as the domination of horizontal displacements for the diaphragm wall without the effect of wall lifting.

Figure 11 2D section with fields of vertical displacements and displacement vectors for the FE model presented in Figure 10.
4.2 Tunnel excavations

Tunnel excavations in densely built-up urban areas require a careful and precise deformation analysis taking into account the presence of existing buildings which are located next to the excavated zone. The underestimation of settlements for surrounding subsoil may thus have an important influence on the state of existing structures. Hence, reliable modelling of tunnel excavation requires advanced analysis which accounts for pre-failure non-linear soil behaviour.

This example demonstrates the importance of modelling tunnel construction problems with the use of advanced constitutive models such as Hardening Soil model. The example highlights the differences in predictions of subsurface displacements during tunnel excavations in the stiff, heavily overconsolidated London Clay modelled with (a) linear-elastic, perfectly plastic Mohr-Coulomb model, and (b) non-linear elastic, perfectly plastic models: HS-Std and HS-SmallStrain.

This study reanalyzes the excavation model of the twin Jubilee Line Extension Project tunnels beneath St James’s Park (London, UK) which has been reported in the original paper by [2].

The problem statement, i.e. subsurface stratigraphy and the orientation of tunnels is presented in Figure 12 and Figure 13. Numerical modelling involved coupled hydro-mechanical analysis (consolidation). Stiffness parameters for the HS model were calibrated using laboratory $\varepsilon_1$-$q$ data points for the isotropically consolidated undrained extension triaxial test (CIEU), Figure 14 and Figure 15. It can be noticed that the model well reproduces strong stiffness variation which were observed in laboratory tests.

![Figure 12 Soil stratigraphy and diagonally oriented tunnels at St James’s Park, London, UK.](image-url)
PRESSURE BC 
imposing hydrostatic
ground water pressure

SEEPAGE elements around
the tunnels simulating free
drainage during excavation

BEAM elements simulating
the tunnel lining

Figure 13 FE mesh for tunnel excavation in London Clay.

Figure 14 Stress-strain curves: comparison between non-linear models (HS-Std, HSSmallStrain, and J-4 model), linear Mohr-Coulomb model and laboratory test data points obtained in the isotropically consolidated undrained extension test.
Figure 15 Variation of the undrained secant stiffness-strain curve $\varepsilon = E_{ud}$; comparison between numerical models and laboratory data points obtained in the isotropically (CIUE) and anisotropically (CAEU) consolidated undrained extension tests.

Figure 16 Surface settlement profiles after excavation of 1st tunnel: comparison for different models.

Figure 16 presents numerical predictions for the surface settlement profiles after excavation of the 1st (westbound) tunnel. It can be noticed that predictions generated by the M-C model are strongly underestimated compared to the field data. Non-linear pre-failure analyses predict deeper and narrower profiles. HS-SmallStrain model gives narrower shape of surface settlements than HS-Std. The initial higher stiffness of the HS-SmallStrain concentrates the strain levels at the unloading boundary giving slightly deeper profile than HS-Std, and therefore the displacements from 10m-offset from the tunnel axis are reduced further away to the mesh sides.
Figure 17 shows that in the case of the Mohr-Coulomb model, soil displacements around the excavated tunnel can be significantly smaller than those predicted by the HS-SmallStrain model. Moreover, decomposition of absolute displacements along horizontal and vertical directions which is presented in Figure 18, shows considerable differences in soil movements in both directions.

Figure 17 Comparison of the absolute displacement fields around the excavated westbound tunnel.

Figure 18 Excavation of the westbound tunnel: (left) vertical settlement in the tunnel axis; (right) horizontal displacements along tunnel axis level of the westbound tunnel.
4.3 Undrained loading and consolidation

The application of the HS-model for a consolidation problem is presented on a trial embankment problem near Almere, Netherlands. The analysis compares results derived from two simulations with the Mohr-Coulomb model and the Hardening-Soil SmallStrain model.

The topology of the embankment problem is illustrated on Figure 19. The upper layer consists of a lightly-overconsolidated organic clay layer (OCR=2) which is deposited on a stiff sand layer. The embankment is constructed up to a height of 2 m. The material involved is taken from the upper soil layer. After the construction, the embankment is backfilled and the clay layer follows consolidation.

![Figure 19 Topology of an embankment problem at Almere, the Netherlands.](image)

The stiffness characteristics for were taken as follows:
- for the M-C model: constant Young modulus $E_{MC} = 13.75$ MPa.
- for the H-S model the stiffness moduli were specified for the reference minor stress $\sigma_{ur} = 30$ kPa: the unloading-reloading modulus twice as Young modulus for M-C ($E_{ur} = 2E_{MC}$), the secant modulus $E_{50} = E_{MC}/2$, the initial stiffness $E_0 = 2E_{ur}$, and tangent oedometric modulus $E_{oed} = E_{50}$.

Figure 20 presents numerical predictions of displacements for the crest and base of embankment. The initial branches of curves are related to the backfilling process which corresponds to the undrained loading. It can be noticed that for both measuring points, HS model generates superior displacements since the undrained loading is primarily deviatoric and the shear hardening mechanism of the model is activated. Activation of this mechanism mobilizes degradation of soil stiffness and the non-linear relation $\varepsilon_1$-$q$ can be reproduced. Clearly, this relation for the M-C model is linear.
The second part of the curves is related to consolidation, i.e. increasing effective mean stress $p'$ and deviatoric stress $q$. For this part, both models show essentially the same response in terms of vertical and horizontal displacements increments since the material is overconsolidated (OCR = 2 for HS model) and consolidation does not involve volumetric plastic straining. On the other, by changing the degree of preconsolidation to almost normally-consolidated state (OCR=1.2), consolidation activates volumetric hardening plasticity and a considerable increase of settlements can be observed.

Figure 20 Comparison of numerical predictions for displacements at the top and the base of an embankment using Mohr-Coulomb model and HS model with OCR=1.2 and OCR=2.0.

5. Summary

This article highlights an importance of using the advanced Hardening Soil model in numerical modelling of typical geotechnical problems. It has been illustrated that the model:

- correctly reproduces the strong reduction of soil stiffness with increasing shear strain amplitudes;
- can be recommended for Serviceability Limit State analyses as predicted soil behavior is more closely matched to field measurements than basic linear-elasticity models;
- is applicable to most soils as it accounts for pre-failure nonlinearities for both sand and clay type materials regardless of overconsolidation state.

Despite the mathematical complexity of the HS model, its parameters have
explicit physical meaning and can be determined with conventional soil tests or they can be estimated following geotechnical evidence (cf. [3]).

6. References


Modelling aging shotcrete: parameter identification and application

Matthias Hofmann
ALPINE BeMo Tunneling GmbH, Innsbruck, Austria

Keywords: aging concrete, creep, parameter identification

Abstract
This contribution deals with the use of the aging concrete constitutive model of Z_Soil for modelling aging shotcrete. The parameters of the models are back calculated from creep tests by means of an optimization method. The use of Z_Soil within the framework of an optimization software will be presented.

1. Introduction
In tunnelling shotcrete is widely used as a construction material for temporary linings. Due to the early loading of young shotcrete in tunnelling a suitable material model for shotcrete should take into account aging, creep and stress dependence. The material properties of shotcrete in early ages are time dependent as a consequence of the hydration process. This contribution deals with the use of the Z_Soil aging concrete constitutive model for modelling shotcrete in early age. For the definition of this material model many material parameters are required. Not all of them can be defined from experiments directly. That is why parameter identification is of special importance.

2. Aging concrete model of Z_Soil
The aging concrete model of Z_Soil is a visco-elastic constitutive model. It is intended to model the time dependent behaviour of concrete in early age, mainly the aging and creep behaviour. It is a rheological model and consists of a series of parallel Maxwell (springdamper) units (see Figure 1). For a detailed description of the model see [1].

The aging concrete model requires a large number of material parameters to be defined, e.g. for three Maxwell units and 7 maturity points 24 parameters.
They cannot be identified directly from experiments. In the next section an approach to identify the material parameters from creep tests by an optimization method is presented.

One drawback of the model is that the time dependence of strength cannot be simulated since that would require a visco-plastic constitutive model for example. Nevertheless it is a useful model because the aging and creep behaviour of shotcrete can be modelled well with this simple model (see section 3).

![Maxwell chain model for aging concrete (taken from [1])](image)

3. Parameter identification approach

Generally, deterministic models relate a set of responses \( \mathbf{x} = \mathbf{x}(\mathbf{p}) \) with the model parameters \( \mathbf{p} \). In the parameter identification procedure the measurements are given from experiments. The parameters \( \mathbf{x}^* = [x_1^* \ldots x_m^*] \) are determined by minimizing the difference between measurements and predicted model responses which results in the following objective function

\[
J = (\mathbf{x}^* - \mathbf{x})^T \mathbf{W}(\mathbf{x}^* - \mathbf{x})
\]

In the present study the DAKOTA-toolkit [2] was used for the solution of the optimization problem. DAKOTA offers several optimization algorithms that can be used for the parameter identification, e.g. global optimization and nonlinear least square methods. In the following examples a nonlinear least square method was used.

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A similar parameter identification approach was used by [3] to identify the material parameters of the Barcelona Basic Modell [4] for unsaturated soils.

### 3.1 Communication between DAKOTA – Z_Soil

For the use of Z_Soil within the DAKOTA framework information has to be exchanged between DAKOTA and Z_Soil. To this end DAKOTA writes a set of material parameters to a file and calls a driver program. A python [5] script is used in the present study as the driver program. Within this python script the current parameter set is read and inserted in a template Z_SOIL input file. Then Z_SOIL_calc is called and the results are written in a file for the response metrics. This file has a format that is understood by DAKOTA. Finally DAKOTA updates the parameters according to the used optimization method and another iteration starts. A sketch of this procedure is shown in Figure 3.

---

**Figure 2: Sketch of the parameter identification approach**

**Figure 3: Sketch of the communication between DAKOTA and Z_Soil**
3.2 Back calculation of creep tests

The presented parameter identification approach was used for the back calculation of creep tests. The creep test data taken from [6]. Two creep tests with constant loads of 5 and 7 MPa are used for back calculation. It has been shown that two Maxwell units for the aging concrete model were sufficient for these simple tests. Nevertheless 9 parameters have to be identified. It has to be noted that probably more tests and load changes are required to identify the parameters reliably. But the aim of this study is to show an approach for the parameter identification and therefore this two tests are sufficient.

In Figure 4 the template Z_Soil model for the parameter identification of the creep tests is given. It is a simple single element model. As explained above the material parameters of the aging concrete model are adapted within the optimization process by a python script.

![Figure 4: Z_Soil template model for the parameter identification of the creep tests](image)

Figure 4: Z_Soil template model for the parameter identification of the creep tests

Figure 5 shows a comparison of the experimental data with the prediction of the aging concrete material model. For the diagram on the left side only the creep test with 5 MPa was used for the parameter identification whereas for the diagram on the right side both experiments were used. There is a good agreement between the experimental and numerical data and therefore the aging concrete model is able to represent the behaviour of the two experiments realistically.
It should be noted that the proposed parameter identification approach can be used for other constitutive models as well (see e.g. [3]). Likewise it can also be used for the identification of other parameters (e.g. loads, geometrical parameters, ...).

4. Conclusions

The present study shows that the creep behaviour of shotcrete in early age can be modeled with the aging concrete model of Z_Soil realistically. The material parameters are identified from creep tests for shotcrete by means of the optimization framework DAKOTA. To this end the input and output files of Z_Soil are utilized by python scripts.

5. References

Single and two-phase dynamic soil-structure interaction in Z_Soil 2010

Andrzej Truty
Cracow University of Technology
Department of Environmental Engineering
Institute of Geotechnics
ZACE Services Ltd

Keywords: Dynamics, finite element analysis.

Abstract The transient dynamic module developed within the Z_Soil.PC 2010 code is designed to carry out soil-structure time history analyses for single or two-phase fully or partially saturated media. The main focus in this paper will be put on Domain Reduction Method (DRM), following work by Bielak [2],[10], Preisig [8], Kantoe [5], and Kantoe et al. [6], modeling viscous boundaries and on stabilization techniques that prevent strong pressure oscillations near the undrained limit [9]. In the current development the \( u - p \) formulation is adopted. It allows to analyze practical problems concerning earthquake engineering. In this approach the effect of relative acceleration of the fluid with respect to the solid skeleton is neglected.

1. Governing equations for two-phase dynamic consolidation

Based on static formulation by Aubry and Ozanam ([1]) the two-phase dynamic consolidation of fully or partially saturated media, designed in the \( u - p \) format, can be written as a set of the following differential equations and corresponding boundary and initial conditions

- the overall equilibrium equation for the solid and fluid phases written in terms of the total stress

\[
\sigma_{ij,j}^{\text{tot}} + \rho \, g \, b_i = \rho \, \ddot{u}_i \quad (1)
\]

\[
\rho = \rho_{\text{dry}} + n \, S \, \gamma^F \quad (2)
\]
where total stress is denoted by $\sigma_{ij}^{\text{tot}}$, earth acceleration by $g$, solid skeleton bulk density by $\rho_{\text{dry}}$, water specific weight by $\gamma^F$, porosity by $n$ and current saturation ratio by $S$

- the extended effective stress principle after Bishop

$$\sigma_{ij}^{\text{tot}} = \sigma_{ij} + \delta_{ij} S \rho_p$$  \hspace{1cm} (3)

where Kronecker’s symbol is denoted by $\delta_{ij}$, effective stresses by $\sigma_{ij}$ and pore pressure by $p$

- the fluid flow continuity equation including the effect of compressibility of the fluid and partial saturation

$$S \dot{\varepsilon}_{kk} + v^F_{k,k} = \left(n \frac{S}{K^F} + n \frac{\partial S}{\partial p} \right) \dot{p}$$ \hspace{1cm} (4)

where $k$–th component of Darcy velocity vector is denoted by $v^F_k$, fluid bulk modulus by $K^F$

- the linearized strain-displacement relations

$$\varepsilon_{ij} = \frac{1}{2} \left( u_{i,j} + u_{j,i} \right)$$ \hspace{1cm} (5)

where engineering strain is denoted by $\varepsilon_{ij}$ and $i$–th component of displacement vector is denoted by $u_i$

- a nonlinear elasto-plastic constitutive relation

$$\dot{\sigma}_{ij} = D_{ijkl}^{e} (\dot{\varepsilon}_{kl} - \dot{\varepsilon}_{kl}^p)$$ \hspace{1cm} (6)

where $D_{ijkl}$ denotes $ijkl$ component of fourth-order current elasticity tensor and $\dot{\varepsilon}_{kl}, \dot{\varepsilon}_{kl}^p$ denote rates of total and plastic strains respectively.

- extended Darcy’s law (including optionally the inertial term)

$$v^F_i = k_{ij}k_r(S) \left( \frac{1}{\gamma^F} p_{j} + b j - \frac{1}{g} \ddot{u}_j \right)$$ \hspace{1cm} (7)

- simplified constitutive equations for saturation ratio $S$, after van Genuchten [3], and relative permeability coefficient $k_r(S)$ after Irnay [4]

$$S = S(p) = \begin{cases} 
S_r + \frac{1 - S_r}{\left[1 + \left(\frac{p}{\gamma^F} \right)^{2}\right]^{1/2}} & \text{if } p > 0 \\
1 & \text{if } p \leq 0 
\end{cases}$$ \hspace{1cm} (8)
\[ k_r (S) = \left( \frac{S - S_r}{1 - S_r} \right)^3 \]  
\[ \text{(9)} \]

where the residual saturation ratio is denoted by \( S_r \), and \( \alpha \) is a material parameter responsible for a decrease of a saturation ratio with an increase of a pressure suction

- near field boundary conditions to be satisfied at any time \( t \in [0, T] \)
  \[ \sigma_{ij}^{\text{tot}} n_j = \mathbf{n}_i \text{ on } \Gamma_i \]
  \[ v_i^F n_i = \mathbf{q} \text{ on } \Gamma_q \]
  \[ u_i = \mathbf{u}_i \text{ on } \Gamma_u \]
  \[ p = \mathbf{p} \text{ on } \Gamma_p \]

\( \Gamma_i, \Gamma_q, \Gamma_u, \Gamma_p \) are parts of the boundary where the total stresses, fluid fluxes, displacements and pore pressures are prescribed.

- far field boundary conditions
  Farfield boundary conditions will be discussed in section 3.

- initial conditions
  \[ u_i(t = t_0) = u_{i0} \]
  \[ p(t = t_0) = p_o \]

(11)

2. Semi-discrete form of balance equations

The time history analysis requires a certain time integration scheme to be applied. In the following derivations standard Newmark and HHT\(-\alpha\) implicit schemes will be used to fulfill unconditional stability condition. The Newmark scheme is the most frequently used in the finite element codes, however, its numerical damping properties are very limited. Any setup for integration coefficients in the Newmark scheme that deviates from \( \beta = 0.5 \) and \( \gamma = 0.25 \) will generate numerical damping for both low and high frequency modes. To remedy this problem we will formulate governing balance equations using HHT\(-\alpha\) scheme that can nicely damp high frequencies without disturbing low ones. Notice that the Newmark method is easily recovered from HHT\(-\alpha\) scheme assuming \( \alpha=0 \).

Following the book by Zienkiewicz et al.[11] the following expressions are used to integrate solid displacements, velocities and pore pressures in time

\[ u_{n+1} = u_n + \dot{u}_n \Delta t + \frac{\Delta t^2}{2} [(1 - 2\beta) \ddot{u}_n + 2\beta \ddot{u}_{n+1}] \]  
\[ \text{(12)} \]

\[ \dot{u}_{n+1} = \dot{u}_n + \Delta t [(1 - \gamma) \ddot{u}_n + \gamma \ddot{u}_{n+1}] \]  
\[ \text{(13)} \]

\[ p_{n+1} = p_n + (1 - \theta) \dot{p}_n \Delta t + \theta \Delta t \dot{p}_{n+1} \]  
\[ \text{(14)} \]
where

\[-\frac{1}{3} \leq \alpha < 0\]  
\[\gamma = \frac{(1 - 2\alpha)}{2}\]  
\[\beta = \frac{(1-\alpha)^2}{4}\]  
\[\theta \geq \frac{1}{2}\]  

(15) \hspace{1cm} (16) \hspace{1cm} (17) \hspace{1cm} (18)

In the HHT scheme we will need certain quantities, their first and second order derivatives, to be evaluated at \(t_{n+\alpha}\), using the following rule

\[\ldots_{n+\alpha} = (1+\alpha)\ldots_{n+1} - \alpha\ldots_n\]  

(19)

Therefore variation of the above quantity can be expressed as follows

\[\delta(\ldots)_{n+\alpha} = (1+\alpha)\delta(\ldots)\]  

(20)

The semi-discrete matrix form of the overall equilibrium in the HHT scheme can be written as follows

\[M\ddot{u}_{n+1} + C\dot{u}_{n+\alpha} + F_{\text{int}}'(u_{n+\alpha}) + C^Fp_{n+\alpha} = F_{\text{extn+\alpha}}\]  

(21)

One may notice that the inertia term is computed at the end of the time step while the damping and internal force terms at \(t_{n+\alpha}\). To preserve symmetry of the resulting matrix form (neglecting the inertial term in the Darcy law) the semi-discrete matrix form of balance of the mass for the fluid phase is written at \(t_{n+\alpha}\)

\[(C^F)^T \dot{u}_{n+\alpha} - \frac{1}{\gamma^F}H^F p_{n+\alpha} + R^F \ddot{u}_{n+\alpha} - h^F - M^F \dot{p}_{n+\alpha} + Q^F = 0\]  

(22)

where

\[F_{\text{int}}'(u_{n+\alpha}) = \int_{\Omega} B^T \sigma(u_{n+\alpha}) d\Omega\]  
\[C^F = \int_{\Omega} N^T S_{n+\alpha} I^T d\Omega\]  
\[H^F = \int_{\Omega} \nabla N^T k \nabla N d\Omega\]  
\[M^F = \int_{\Omega} N^T c_{n+\alpha} N d\Omega\]  
\[R^F = \int_{\Omega} \nabla N^T k_1 \frac{1}{g} N d\Omega\]  
\[h^F = \int_{\Omega} \nabla N^T k bd\Omega\]  
\[Q^F = \int_{\Gamma_\gamma} N^T q d\Omega\]  

(23) \hspace{1cm} (24) \hspace{1cm} (25) \hspace{1cm} (26) \hspace{1cm} (27) \hspace{1cm} (28) \hspace{1cm} (29)
In the above expressions matrices of the standard shape functions are denoted by $N$, the storage coefficient $c_{n+\alpha} = c(n+\alpha) = n \left( \frac{S_{n+\alpha}}{K} + \frac{dS_{n+\alpha}}{dp} \right)$, the $k_v$ is a penalty factor for seepage elements, $g$ is an earth acceleration ($g=9.81 \text{ m/s}^2$) and imposed fluid flux at boundary $\Gamma_q$ is denoted by $\overline{q}$.

The final linearized semi-discrete form, with solid accelerations and pore pressure rates as primary unknowns, can easily be obtained from eq.(21) and eq.(22) assuming that the current saturation ratio $S$ and storage coefficient $c$ are kept constant within the time step and equal to their values at step $t_n$.

$$\begin{bmatrix} K_{uu} & K_{up} \\ K_{pu} & K_{pp} \end{bmatrix} \begin{bmatrix} \delta \ddot{u} \\ \delta \dot{p} \end{bmatrix} = \begin{bmatrix} \delta F \\ \theta \delta Q \end{bmatrix}$$ (30)

where

$$K_{uu} = M + K(1 + \alpha)\beta \Delta t^2 + C(1 + \alpha)\gamma \Delta t$$ (31)

$$K_{up} = C^F(1 + \alpha)\theta \Delta t$$ (32)

$$K_{pu} = (C^F)^T (1 + \alpha)\theta \Delta t + \frac{\theta}{\gamma}(1 + \alpha)R^F$$ (33)

$$K_{pp} = \frac{\theta}{\gamma}(1 + \alpha) \left( -\frac{1}{\gamma}H^F \theta \Delta t - M^F - P^F \theta \Delta t \right)$$ (34)

$$\delta F = F_{extn+\alpha} - M\ddot{u}_{n+\alpha} - C\dot{u}_{n+\alpha} - F_{int} - C^F p_{n+\alpha}$$ (35)

$$\delta Q = -Q^F + h^F - (C^F)^T u_{n+\alpha} + \frac{1}{\gamma}H^F p_{n+\alpha} - R^F \ddot{u}_{n+\alpha} + M^F \dot{p}_{n+\alpha}$$ (36)

To retain symmetry of the generalized stiffness matrix one may neglect the term $\frac{\theta}{\gamma}(1 + \alpha)R^F$ in $K_{pu}$, however, even elastic problems will require few iterations to achieve the equilibrium state. It has to be mentioned that within Z_Soil code we use exclusively low order elements (BBAR or EAS) hence equal interpolation order for displacement and pressure degrees of freedom is the only choice. This setting may yield spurious spatial pressure oscillations in the incompressibility limit, and for that reason a special stabilization procedure, based on pressure Laplacian, is used to circumvent this deficiency (same as for the standard static consolidation problems) [9].

3. Absorbing boundaries for dynamic consolidation of two-phase media

In the FE approach to soil-structure interaction problems one has to truncate the subsoil subdomain adjacent to the structure. This results in spurious wave reflections when front
of the wave hits subsoil boundaries. To circumvent this deficiency absorbing bound-
aries, using zeroth-order paraxial formulation as proposed by Modaressi and Bonzenati
[7], are adopted here. This formulation is coherent with the \( u - p \) type formulation. The
total viscous stress and fluid flux vectors, taking into account effect of partial saturation,
are defined as follows

\[
\sigma^s = - \left\{ \rho \frac{c_p^2}{V_{p1}} (\lambda_s + 2\mu_s) \mathbf{n}\mathbf{n}^T + \rho c_s \left( \mathbf{t}_1\mathbf{t}_1^T - \mathbf{t}_2\mathbf{t}_2^T \right) \right\} \mathbf{v}^s + \mathbf{n} (S p - S_0 p_0) \quad (37)
\]

\[
\Phi = k \left[ \rho \left( 1 - \frac{c_p^2}{V_{p1}^2} - \rho \right) \right] \mathbf{n}^T \mathbf{a}^s \quad (38)
\]

The resulting viscous force and flux vectors that are added to the right hand side of
the momentum and fluid mass balance semi-discrete equations are defined as follows

\[
F_{u}^v = - \int_{\Gamma} \mathbf{N}^T \sigma^s d\Gamma \quad (39)
\]

\[
F_{p}^v = - \int_{\Gamma} \mathbf{N}^T \Phi d\Gamma \quad (40)
\]

In the above equations \( \mathbf{N} \) is a matrix of standard shape functions.

The corresponding shear and dilatational wave velocities, for solid phase, are defined
through the expressions

\[
c_s = \sqrt{\frac{\lambda}{\rho}} \quad c_p = \sqrt{\frac{\lambda + 2 \Gamma}{\rho}} \quad (41)
\]

while the approximate first dilatational wave velocity for medium filled by a compress-
able fluid is defined as

\[
V_{p1}^2 = c_p^2 \left( 1 + \frac{Q}{\lambda + 2 \Gamma} \right) \quad (42)
\]

and

\[
\frac{1}{Q} = \frac{n S}{K_F} + n \frac{dS}{dp} \quad (43)
\]

Solid velocity and acceleration vectors are denoted by \( \mathbf{v}^s \) and \( \mathbf{a}^s \).

The normalized normal and tangential vectors are denoted by \( \mathbf{n} \) and \( \mathbf{t}_1, \mathbf{t}_2 \) respectively. In the expression for the total viscous stress the \( S_0 p_0 \) term is added to cancel the
initial saturation/pressure prior running dynamic time history analysis. The \( \frac{dS}{dp} \) term is
computed using van Genuchten’s law while \( k \) is the permeability value along normal
direction (\( \mathbf{n} \)). It is computed using the the formula

\[
k = \mathbf{n}^T \mathbf{k} \mathbf{n} \quad (44)
\]

It has to be mentioned that this formulation is only approximate and hence spurious
reflections are not fully eliminated.
4. Domain Reduction Method (DRM) for single and two-phase partially saturated media

The Domain Reduction Method (DRM), applied to single-phase media, was proposed by Bielak at al.[2], [10]. Recently, Kantoe, Zdravkovic and Potts published a paper [6] on application of DRM method to carry out time history analysis of fully saturated two-phase media. In this paper we will present derivation of the DRM method for fully or partially saturated two-phase media and contrary to the last referenced paper the method will be formulated at semidiscrete level rather than at the discrete one.

The main goal of this method is to analyze the computational model that concerns the structure and only a small adjacent part of subsoil. This way size of the discretized model to be solved is significantly reduced. To understand the method let us consider a domain that includes a fault, being a source of the excitation forces $P_e(t)$, and a structure (see Fig.1).

At this point we will analyze the two computational models. The first one, called background model (see Fig.(2)), includes the subsoil and source of the load $P_e(t)$, while the second one, called reduced model (see Fig.(3)), includes the structure and small part of the subsoil.

The aim of the background model is to find the free field motion induced by $P_e(t)$. Resulting free field displacements, velocities, accelerations and pore pressures are denoted by $u^0(t)$, $\dot{u}^0(t)$, $\ddot{u}^0(t)$ and $p^0(t)$ respectively. The background model can be partitioned into interior domain, denoted by $\Omega$, and the exterior one denoted by $\Omega^+$. The $\Gamma$ is the boundary that separates interior and exterior domains. The kinematic quantities at any point in the interior domain will be denoted with the lower index $(i)$, at boundary $\Gamma$ with index $(b)$ and in the exterior domain with index $(e)$. Nodal points that belong to the boundary $\Gamma$ are labeled as boundary (b), nodes that are in the $\Omega^+$ domain and do not belong to the boundary $\Gamma$ are labeled as exterior (e), and the remaining ones as interior (i).

After partitioning of the whole domain into $\Omega$ and $\Omega^+$ one may write equations of motion (see eq.(45), eq.(46) and fluid mass balance (see eq.(47), eq.(48)) in $\Omega$ and $\Omega^+$ respectively

- Overall equilibrium in $\Omega$

$$
\begin{bmatrix}
M_{ii}^{\Omega} & M_{ib}^{\Omega} \\
M_{bi}^{\Omega} & M_{bb}^{\Omega}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_i \\
\ddot{u}_b
\end{bmatrix}
+ 
\begin{bmatrix}
K_{ii}^{\Omega} & K_{ib}^{\Omega} \\
K_{bi}^{\Omega} & K_{bb}^{\Omega}
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b
\end{bmatrix}
+ 
\begin{bmatrix}
C_{ii}^{\Omega} & C_{ib}^{\Omega} \\
C_{bi}^{\Omega} & C_{bb}^{\Omega}
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b
\end{bmatrix}
+ 
\begin{bmatrix}
C_{F,ii}^{\Omega} & C_{F,ib}^{\Omega} \\
C_{F,bi}^{\Omega} & C_{F,bb}^{\Omega}
\end{bmatrix}
\begin{bmatrix}
p_i \\
p_b
\end{bmatrix}
= 
\begin{bmatrix}
0 \\
P_b
\end{bmatrix}
$$

- Overall equilibrium in $\Omega^+$
Figure 1: Full model of subsoil and structure, and source of the loading $P_e(t)$

Figure 2: Background model

Figure 3: Reduced model
\[
\begin{bmatrix}
M^{\Omega^+}_{bb} & M^{\Omega^+}_{be} \\
M^{\Omega^+}_{eb} & M^{\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_b \\
\ddot{u}_e
\end{bmatrix}
+
\begin{bmatrix}
K^{\Omega^+}_{bb} & K^{\Omega^+}_{be} \\
K^{\Omega^+}_{eb} & K^{\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
\dot{u}_b \\
\dot{u}_e
\end{bmatrix}
+
\begin{bmatrix}
C^{\Omega^+}_{bb} & C^{\Omega^+}_{be} \\
C^{\Omega^+}_{eb} & C^{\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_b \\
\ddot{u}_e
\end{bmatrix}
+
\begin{bmatrix}
C^{\Omega^+}_{bb} & C^{\Omega^+}_{be} \\
C^{\Omega^+}_{eb} & C^{\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
\dot{p}_b \\
\dot{p}_e
\end{bmatrix}
= \begin{bmatrix}
-P_b \\
-P_e
\end{bmatrix}
\] (46)

- Fluid mass balance in $\Omega$

\[
\begin{bmatrix}
R^{F\Omega}_{ii} & R^{F\Omega}_{ib} \\
R^{F\Omega}_{bi} & R^{F\Omega}_{bb}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_i \\
\ddot{u}_b
\end{bmatrix}
+ \begin{bmatrix}
(C^{F\Omega}_{ii})^T & (C^{F\Omega}_{ib})^T \\
(C^{F\Omega}_{bi})^T & (C^{F\Omega}_{bb})^T
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b
\end{bmatrix}
- \frac{1}{\gamma^F}
\begin{bmatrix}
H^{F\Omega}_{ii} & H^{F\Omega}_{ib} \\
H^{F\Omega}_{bi} & H^{F\Omega}_{bb}
\end{bmatrix}
\begin{bmatrix}
p_i \\
p_b
\end{bmatrix}
- \begin{bmatrix}
M^{F\Omega}_{ii} & M^{F\Omega}_{ib} \\
M^{F\Omega}_{bi} & M^{F\Omega}_{bb}
\end{bmatrix}
\begin{bmatrix}
\ddot{p}_i \\
\ddot{p}_b
\end{bmatrix}
= \begin{bmatrix}
-Q^F_i + h_i^F \\
-Q^F_b + h_b^F
\end{bmatrix}
\] (47)

- Fluid mass balance in $\Omega^+$

\[
\begin{bmatrix}
R^{F\Omega^+}_{bb} & R^{F\Omega^+}_{be} \\
R^{F\Omega^+}_{eb} & R^{F\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_b \\
\ddot{u}_e
\end{bmatrix}
+ \begin{bmatrix}
(C^{F\Omega^+}_{bb})^T & (C^{F\Omega^+}_{be})^T \\
(C^{F\Omega^+}_{eb})^T & (C^{F\Omega^+}_{ee})^T
\end{bmatrix}
\begin{bmatrix}
\dot{u}_b \\
\dot{u}_e
\end{bmatrix}
- \frac{1}{\gamma^F}
\begin{bmatrix}
H^{F\Omega^+}_{bb} & H^{F\Omega^+}_{be} \\
H^{F\Omega^+}_{eb} & H^{F\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
p_b \\
p_e
\end{bmatrix}
- \begin{bmatrix}
M^{F\Omega^+}_{bb} & M^{F\Omega^+}_{be} \\
M^{F\Omega^+}_{eb} & M^{F\Omega^+}_{ee}
\end{bmatrix}
\begin{bmatrix}
\ddot{p}_b \\
\ddot{p}_e
\end{bmatrix}
= \begin{bmatrix}
Q^F_b + h_b^F \\
-Q^F_e + h_e^F
\end{bmatrix}
\] (48)

where vector of boundary forces is denoted by $P_b$ and vector of boundary fluxes by $Q^F_b$.

By summing eq.(45) and eq.(46) one gets the following global form of equation of motion
Same operation performed for eq.(47) and eq.(48) yields the following global form of the balance of fluid mass

\[
\begin{bmatrix}
  \dot{u}_i \\
  \dot{u}_b \\
  \dot{u}_c \\
\end{bmatrix}
= 
\begin{bmatrix}
  M^{\Omega}_{ii} & M^{\Omega}_{ib} & 0 \\
  M^{\Omega}_{bi} & M^{\Omega}_{bb} + M^{\Omega^+}_{bb} & M^{\Omega^+}_{be} \\
  0 & M^{\Omega^+}_{eb} & M^{\Omega^+}_{ee} \\
\end{bmatrix}
\begin{bmatrix}
  u_i \\
  u_b \\
  u_c \\
\end{bmatrix}
+ 
\begin{bmatrix}
  K^{\Omega}_{ii} & K^{\Omega}_{ib} & 0 \\
  K^{\Omega}_{bi} & K^{\Omega}_{bb} + K^{\Omega^+}_{bb} & K^{\Omega^+}_{be} \\
  0 & K^{\Omega^+}_{eb} & K^{\Omega^+}_{ee} \\
\end{bmatrix}
\begin{bmatrix}
  \dot{u}_i \\
  \dot{u}_b \\
  \dot{u}_c \\
\end{bmatrix}
+ 
\begin{bmatrix}
  C^{\Omega}_{ii} & 0 \\
  C^{\Omega}_{bi} & C^{\Omega}_{bb} + C^{\Omega^+}_{bb} \frac{T}{\gamma} \\
  0 & C^{\Omega^+}_{eb} \\
\end{bmatrix}
\begin{bmatrix}
  \dot{\Omega}_i \\
  \dot{\Omega}_b \\
  \dot{\Omega}_c \\
\end{bmatrix}
= 
\begin{bmatrix}
  0 \\
  0 \\
  0 \\
\end{bmatrix}
(49)
\]

\[
\begin{bmatrix}
  p_i \\
  p_b \\
  p_c \\
\end{bmatrix}
\]

If we decompose displacement \(\mathbf{u}_c\) and pressure \(\mathbf{p}_c\) vectors, in the exterior domain, into free field, \(\mathbf{u}_c^0\) and \(\mathbf{p}_c^0\), and residual ones, \(\mathbf{\hat{u}}_c\) and \(\mathbf{\hat{p}}_c\), as follows

\[
\mathbf{u}_c = \mathbf{u}_c^0 + \mathbf{\hat{u}}_c \\
\mathbf{p}_c = \mathbf{p}_c^0 + \mathbf{\hat{p}}_c
(50)
\]
then after substituting terms (51) and (52) into eq.(49), and eq.(50), we will get the following forms of the overall equilibrium and fluid mass balance

\[
\begin{bmatrix}
M_i^{\Omega} & M_i^{\Omega} & 0 \\
M_i^{\Omega} & M_i^{\Omega} + M_b^{\Omega} & M_i^{\Omega} \\
0 & M_e^{\Omega} & M_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b \\
\dot{u}_e \\
\end{bmatrix}
+ \begin{bmatrix}
K_i^{\Omega} & K_i^{\Omega} & 0 \\
K_i^{\Omega} & K_i^{\Omega} + K_b^{\Omega} & K_i^{\Omega} \\
0 & K_e^{\Omega} & K_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\bar{u}_i \\
\bar{u}_b \\
\bar{u}_e \\
\end{bmatrix}
= \begin{bmatrix}
u_i \\
u_b \\
u_e \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
C_i^{\Omega} & C_i^{\Omega} & 0 \\
C_i^{\Omega} & C_i^{\Omega} + C_b^{\Omega} & C_i^{\Omega} \\
0 & C_e^{\Omega} & C_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b \\
\dot{u}_e \\
\end{bmatrix}
+ \begin{bmatrix}
C_i^{\Omega} & C_i^{\Omega} & 0 \\
C_i^{\Omega} & C_i^{\Omega} + C_b^{\Omega} & C_i^{\Omega} \\
0 & C_e^{\Omega} & C_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\bar{u}_i \\
\bar{u}_b \\
\bar{u}_e \\
\end{bmatrix}
= \begin{bmatrix}
p_i \\
p_b \\
p_e \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
-R_i^{\Omega} & R_i^{\Omega} & 0 \\
R_i^{\Omega} & R_i^{\Omega} + R_b^{\Omega} & R_i^{\Omega} \\
0 & R_e^{\Omega} & R_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b \\
\dot{u}_e \\
\end{bmatrix}
+ \begin{bmatrix}
(C_i^{\Omega})^T & (C_i^{\Omega})^T & 0 \\
(C_i^{\Omega})^T & (C_i^{\Omega})^T + (C_b^{\Omega})^T & (C_i^{\Omega})^T \\
0 & (C_e^{\Omega})^T & (C_e^{\Omega})^T \\
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b \\
\dot{u}_e \\
\end{bmatrix}
= \begin{bmatrix}
p_i \\
p_b \\
p_e \\
\end{bmatrix}
\]

\[
\frac{1}{\gamma F} \begin{bmatrix}
H_i^{\Omega} & H_i^{\Omega} & 0 \\
H_i^{\Omega} & H_i^{\Omega} + H_b^{\Omega} & H_i^{\Omega} \\
0 & H_e^{\Omega} & H_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\dot{p}_i \\
\dot{p}_b \\
\dot{p}_e \\
\end{bmatrix}
= \begin{bmatrix}
-Q_i^{F} + h_i^{F} \\
0 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
M_i^{\Omega} & M_i^{\Omega} & 0 \\
M_i^{\Omega} & M_i^{\Omega} + M_b^{\Omega} & M_i^{\Omega} \\
0 & M_e^{\Omega} & M_e^{\Omega} \\
\end{bmatrix}
\begin{bmatrix}
\dot{p}_i \\
\dot{p}_b \\
\dot{p}_e \\
\end{bmatrix}
= \begin{bmatrix}
0 \\
-Q_e^{F} + h_e^{F} \\
\end{bmatrix}
\]

The $P_e$ and $Q_e$ terms can now be derived from eq.(46), and eq.(48) respectively, assuming that these are solved for a simpler problem that does not include the structure
and the overall equilibrium equation can be written as follows

$$
P_c = M^{\Omega b}_{e b} \dot{u}_b^0 + M^{\Omega c}_{e c} \dot{u}_c^0 + K^{\Omega b}_{e b} \dot{u}_b^0 + K^{\Omega c}_{e c} \dot{u}_c^0 + C^{\Omega b}_{e b} \dot{u}_b^0 + C^{\Omega c}_{e c} \dot{u}_c^0 + C^{\Omega \epsilon}_{e e} \dot{u}_c^0 +
$$

$$
P_{c e b} p_b^0 + C^{\Omega \epsilon}_{e e} p_e^0
$$

$$
- Q_c + h^{F}_{e c} = R^{F b}_{e b} \dot{u}_b^0 + R^{F c}_{e c} \dot{u}_c^0 + \left( C^{F b}_{e b} \right)^T \dot{u}_b^0 + \left( C^{F c}_{e c} \right)^T \dot{u}_c^0 -
$$

$$
\frac{1}{\gamma^{F}} H^{F b}_{e b} \dot{p}_b^0 - \frac{1}{\gamma^{F}} H^{F c}_{e c} p_e^0 - M^{F b}_{e b} p_b^0 - M^{F c}_{e c} p_e^0
$$

(55)

By substituting the $P_c$ term to the eq.(53) the following form of the right hand side term in the overall equilibrium equation is obtained

$$
P^{\epsilon f f} = \begin{cases} 
0 \\
-M^{\Omega b}_{e b} \dot{u}_b^0 - K^{\Omega b}_{e b} \dot{u}_b^0 - C^{\Omega b}_{e b} \dot{u}_b^0 - C^{\Omega \epsilon b}_{e b} p_e^0 \\
M^{\Omega b}_{e b} \dot{u}_b^0 + K^{\Omega b}_{e b} \dot{u}_b^0 + C^{\Omega b}_{e b} \dot{u}_b^0 + C^{\Omega \epsilon b}_{e b} p_b^0
\end{cases}
$$

(57)

and the overall equilibrium equation can be written as follows

$$
\begin{bmatrix}
M^{\Omega b}_{ii} & M^{\Omega b}_{ib} & M^{\Omega b}_{bb} & 0 \\
M^{\Omega b}_{bi} & M^{\Omega b}_{ii} + M^{\Omega b}_{bb} & M^{\Omega b}_{ib} & M^{\Omega b}_{be} \\
0 & M^{\Omega b}_{ib} & M^{\Omega b}_{bb} & M^{\Omega b}_{be} \\
K^{\Omega b}_{ii} & K^{\Omega b}_{ib} & K^{\Omega b}_{bb} & K^{\Omega b}_{be} \\
0 & K^{\Omega b}_{ib} & K^{\Omega b}_{bb} & K^{\Omega b}_{be} \\
C^{\Omega b}_{ii} & C^{\Omega b}_{ib} & C^{\Omega b}_{bb} & C^{\Omega b}_{be} \\
0 & C^{\Omega b}_{ib} & C^{\Omega b}_{bb} & C^{\Omega b}_{be} \\
C^{\Omega \epsilon b}_{ii} & C^{\Omega \epsilon b}_{ib} & C^{\Omega \epsilon b}_{bb} & C^{\Omega \epsilon b}_{be} \\
0 & C^{\Omega \epsilon b}_{ib} & C^{\Omega \epsilon b}_{bb} & C^{\Omega \epsilon b}_{be}
\end{bmatrix}
\begin{bmatrix}
\dot{u}_i \\
\dot{u}_b \\
\dot{u}_c \\
\dot{u}_b \\
\dot{u}_c \\
\dot{u}_b \\
\dot{u}_c \\
\dot{p}_i \\
\dot{p}_b \\
\dot{p}_e
\end{bmatrix} = P^{\epsilon f f}
$$

(58)

Similarly by substituting the $Q_c$ term to the eq.(54) the following form of the right hand side term in the fluid mass balance equation is obtained

$$
Q^{\epsilon f f} = \begin{cases} 
0 \\
h^{F b}_{e b} \dot{u}_b^0 - R^{F \Omega b}_{e b} \dot{u}_b^0 - \left( C^{F \Omega b}_{e b} \right)^T \dot{u}_b^0 + \frac{1}{\gamma^{F}} H^{F \Omega b}_{e b} p_e^0 + M^{F \Omega b}_{e b} p_b^0 \\
R^{F \Omega b}_{e b} \dot{u}_b^0 + \left( C^{F \Omega b}_{e b} \right)^T \dot{u}_b^0 - \frac{1}{\gamma^{F}} H^{F \Omega b}_{e b} p_b^0 - M^{F \Omega b}_{e b} p_b^0
\end{cases}
$$

(59)

and the fluid mass balance takes the form as follows
\[
\begin{bmatrix}
R_{ii}^F & R_{ib}^F & 0 & R_{be}^F & R_{ee}^F \\
R_{bi}^F & R_{bb}^F & 0 & R_{be}^F & R_{ee}^F \\
0 & 0 & R_{bb}^F & R_{be}^F & \right.
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_i \\
\ddot{u}_b \\
\ddot{u}_e 
\end{bmatrix} +
\begin{bmatrix}
\left(C_{ii}^F \right)^T & \left(C_{ib}^F \right)^T & 0 & \left(C_{be}^F \right)^T & \left(C_{ee}^F \right)^T \\
\left(C_{bi}^F \right)^T & \left(C_{bb}^F \right)^T & 0 & \left(C_{be}^F \right)^T & \left(C_{ee}^F \right)^T \\
0 & 0 & \left(C_{bb}^F \right)^T & \left(C_{be}^F \right)^T & \right.
\end{bmatrix}
\begin{bmatrix}
\ddot{p}_i \\
\ddot{p}_b \\
\ddot{p}_e 
\end{bmatrix} =
\begin{bmatrix}
-\dot{Q}_i + h_{F_i} \\
0 
\end{bmatrix} + \dot{Q}_{eff}
\]

(60)

It has to be mentioned that the DRM approach requires a linear response of the computational model in the exterior domain for both solid and fluid parts, although response in the interior domain can be nonlinear. As the method is implemented in the finite element framework we need to select at least a single row of finite elements, in the reduced model, that is treated as a boundary layer separating the interior and exterior domains.

5. Application of DRM method for solving practical 3D problems

The considered practical case concerns protection of a liquidated piped gas borehole located nearby newly constructed road, subject to excitation caused by a road vibratory rollers. To assess the influence of a selected set of vibratory rollers several numerical analyses had to be carried out for discrete 3D models of subsoil-piped borehole-diaphragm wall system (Fig.4), assuming different positions of the roller from the borehole axis. This kind of analyses need to be preceded by some field investigations that give information on shear and dilatational wave velocities in subsoil layers, eigenfrequencies for vertical and horizontal directions, and damping coefficients. Based on this information one may estimate material properties for soil layers including small strain moduli \(E\) and Poisson ratio \(\nu\). Additional tests were carried out to assess the effect of attenuation of surface accelerations caused by a given vibratory roller working at a given frequency. This measurements were used later on to estimate Rayleigh damping parameters. In this article the attention will be focused on case without 4m high road
embankment assuming that subsoil is not saturated.

Figure 4: Protection of a gas piped borehole

Figure 5: DRM scheme

5.1 Computational method

All simulations were carried out using DRM method and HHT integration scheme ($\alpha=-0.3$, $\beta=0.4225$, $\gamma=0.8$, $\theta = 0.5$). As it has been mentioned in the previous section, the DRM method consists of two steps. In the first step the free field motion, caused by a vertical load $F(t) = F|\sin(\theta t)|$, applied at the axis of an axisymmetric model, has to be computed. For given roller and assumed force amplitude, and frequency,
the free field motion is solved only once. In the second step the whole 3D system is analyzed, taking into account only limited volume of subsoil. The effective force term in this reduced model is computed based on the computed axisymmetric free field motion (see Fig.(5)). All layers of subsoil were discretized using 8 node BBAR brick elements, diaphragm walls and concrete walls using MITC 4 node shell elements and piped borehole was discretized using 2 node beam elements with half of the stiffness due to assumed symmetry boundary condition.

5.2 Material properties

Material properties of soil layers, piped borehole and concrete walls are given in Table 1. Geometrical and material characteristics of piped borehole, filled by a concrete, were estimated assuming that the axial and bending stiffness of the real and equivalent cross section of the beam are equal.

Table 1: Material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Descr.</th>
<th>$E$ [kPa]</th>
<th>$\nu$ [-]</th>
<th>$\rho$ [kg/m³]</th>
<th>$A$ [m²] or $h$ [m]</th>
<th>$I$ [m⁴]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-3.5m, Silty clay</td>
<td>99000</td>
<td>0.35</td>
<td>1600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3.5-17m, Stiff plastic silts</td>
<td>244000</td>
<td>0.35</td>
<td>1700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>17-29m, Stiff clays</td>
<td>1010000</td>
<td>0.35</td>
<td>1950</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Diaphragm wall, Concrete B20</td>
<td>29000000</td>
<td>0.2</td>
<td>2500</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Concrete wall, Concrete B20</td>
<td>29000000</td>
<td>0.2</td>
<td>2500</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Vibroisolation</td>
<td>2800</td>
<td>0.0</td>
<td>710</td>
<td>0.025</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Embankment</td>
<td>400000</td>
<td>0.35</td>
<td>1950</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Piped borehole</td>
<td>0</td>
<td>0.2</td>
<td>3437.5</td>
<td>0.141</td>
<td>$I_x=0.00636173$ $I_y=I_z=0.00318$</td>
</tr>
</tbody>
</table>

5.3 Damping properties

In the analyzed case we have the two basic sources of the damping i.e. radial damping in subsoil, that is naturally included in the axisymmetric free field model and hysteretic
damping in subsoil and in the structure.

![Graph showing the distribution of maximum vertical acceleration](image)

Figure 6: Distribution of maximum vertical acceleration for vibratory roller HAMM 3518 HT 23.56 Hz 331 kN

In all simulations common values of Rayleigh damping parameters were assumed for all soil layers ($\alpha = 0.1$ and $\beta = 0.000375$). In diaphragm and upper concrete walls logarithmic decrement of damping $\delta = 0.2$ was assumed, while for the vibroisolation, according to the specification of Sylomer ST 220 (thickness 25mm), loss coefficient $\eta = 0.13$. A comparison of measured and computed distribution of maximum vertical acceleration is shown in Fig.(6).

### 5.4 Reduced model

The DRM method allowed to reduce size of the model up to 250 000 DOF, for case without road embankment, and to 300 000 for case with embankment. In the considered case the exterior domain consists of a single layer of elements on vertical FE model walls (except symmetry plane) and on the bottom of the model. The boundary layer consists also of a single layer of elements directly adjacent to the external one. In order to change the distance of an excitation force from the structure it was enough to move the 3D model along global $x$ axis. The 3D discretization of the reduced model, without embankment, is shown in Fig.(7) and Fig.(8). The viscous boundaries are located on the external faces of the elements in the exterior domain. Their material properties are automatically inherited from the adjacent continuum.
5.5 Results

The distribution of maximum $a_x$ acceleration as a function of vibratory roller (Stavostroj 1500 D, 237 kN/35 Hz) location is shown in Fig. (9, 10, 11) for distances 100m, 80m and 60m respectively.

To visualize deformations resulting from DRM method in the standard Z_Soil output total and residual deformations are output in the interior and exterior domains respectively. A typical deformation pattern of the reduced 3D model is shown in Fig. (12). A strong distortion of elements in the boundary layer is typical for the DRM method.

![Diagram of a vibratory roller](image1)

Figure 7: aaa

![Diagram of DRM method](image2)

Figure 8: Protection of a gas piped borehole
Figure 9: Distribution of maximum $a_x(r)$ for load placed 100m from the borehole

Figure 10: Distribution of maximum $a_x(r)$ for load placed 80m from the borehole

Figure 11: Distribution of maximum $a_x(r)$ for load placed 60m from the borehole
6. Conclusions

A detailed derivation of the two-phase fully or partially saturated soil-structure interaction problem was given in the paper. Special attention was paid to the reduction of size of computational models using DRM approach proposed by Bielak [2], [10]. The practical example of protection of a piped gas borehole proved the method to be robust to deal with soil-structure interaction problems where the reduced model is usually 3D while the free field can be analyzed by a model with spatial dimension of same order (3D) or lower (2D/1D). The developed time history analysis module within the Z_Soil code is a complete tool to carry out complex dynamic analyses for soil-structure interaction assuming the soil to be single or two-phase, fully or partially saturated, medium.
References


Seismic safety of a buttress dam and appurtenant structures -
3D static and dynamic analyses

Aïssa Mellal
STUCKY SA, Switzerland

Keywords: buttress dam, hydraulic gates, seismic safety, stress analyses, stability analyses

Abstract
In this paper, the methodology and results of the seismic safety assessment of a buttress dam are presented. Three-dimensional finite element model of the dam, including appurtenant structures, metallic gates and foundation is prepared. Calibration and validation of the numerical model are carried out through comparisons between measured and calculated dam temperatures and displacements. Static analyses of the dam are first conducted to evaluate stresses and displacements under usual load cases, i.e. self weight, hydrostatic pressure and temperature gradient. Then, using site spectrum-compatible accelerograms, dynamic analyses are run, in combination with initial static loads. Calculated compressive and tensile stresses in the dam and appurtenant structures are compared to dynamic concrete strength. The stability of the dam against sliding and overturning is evaluated considering the maximal dynamic response. Finally, seismic safety of metallic gates is evaluated considering steel strength, jamming hazard and buckling of hydraulic jacks.

1. Introduction
In the framework of the Swiss seismic safety requirements on existing dams [1], a detailed study of a buttress dam near Rossinière, Switzerland has been carried out according to a methodology published by the Swiss Federal Office of Energy [2]. Main project phases are:
- Gathering of data in relation with geometry, geology and topography
- Preparation of a 3D numerical model of the dam, its appurtenant structures, hydro-mechanical equipments (gates) and foundation
- Calibration and validation of the constructed finite element (thermal and mechanical) using available measurements (temperatures, displacements and uplift pressures)
- Definition of the seismic load, including accelerograms to be applied to the dam
- Assessment of the seismic safety of the dam and its components in terms of strength and stability

2. Context and project description

Rossinière dam is located at the extremity of Vernex Lake, in a narrow corridor formed by sub-vertical rocky walls. The dam is 30 m high and has a crest length of 35 m. The volume of reservoir is about 2.9 million cubic meters. The dam structure is constituted of three buttresses, a footbridge and two central openings of 5 m width each, equipped with two flap gates and two outlet gates (Figure 1). Laterally, the dam is extended by two (left and right) wing buildings. According to the Swiss seismic norm classification, the dam belongs to Class I category for which highest safety criteria are required.

![Figure 1. Rossinière dam](image)

3. Numerical model

The analysis of the dam’s seismic safety is carried out using a numerical finite element model. Static and dynamic analyses are performed to represent dam’s initial state and behaviour during an earthquake. Numerical analyses are achieved using the finite element package Z_Soil 3D [3].

The geometry of the dam and its location require the definition of a three-dimensional model for a realistic representation of its mechanical behaviour. Furthermore, due to comparable dimensions, structural interactions may take place between hydraulic gates and dam’s walls during earthquake and it is therefore necessary to explicitly model and include the metallic gates in the whole finite element model.

The finite element mesh of the dam including lateral wing buildings, footbridge and hydraulic gates is shown on Figure 2. Detailed views of the gates and dam’s inner structure are presented in Figure 3. The finite element
mesh of the dam including wing buildings and the foundation required about 65’000 volumetric 8-nodes elements. Hydraulic gates were modelled using 3’658 shell elements, 354 beams and 4 truss elements.

Figure 2. Finite element model of Rossinière dam

![Figure 2](image)

Figure 3. Finite element mesh of dam’s gates and inner structure

Linear elastic behaviour was assumed for all materials used in the finite element model, i.e. concrete, rock and steel. In dynamic regime, an increase of 25% is assumed for the elastic modulus. Material parameters (Table 1) were defined using available laboratory test data and, when missing, from usual literature values for similar materials. Thermal material properties for concrete (Table 2) were determined after thermo-mechanical calibration of the model,
for thermal expansion coefficient, and from usual values for concrete.

Table 1. Elastic material parameters

<table>
<thead>
<tr>
<th></th>
<th>Mass concrete</th>
<th>Structure concrete</th>
<th>Rock</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific weight, $\gamma$ [kN/m$^3$]</td>
<td>24</td>
<td>25</td>
<td>0*</td>
<td>8'000</td>
</tr>
<tr>
<td>Elastic modulus, $E$ [GPa]</td>
<td>20</td>
<td>30</td>
<td>15</td>
<td>210</td>
</tr>
<tr>
<td>Poisson’s coefficient, $\nu$ [-]</td>
<td>0.2</td>
<td>0.2</td>
<td>0.25</td>
<td>0.27</td>
</tr>
</tbody>
</table>

* massless foundation assumption is considered for dynamic analyses

Table 2. Thermal characteristics of concrete

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Thermal expansion coefficient, $\alpha$ [1/°C]</td>
<td>$6.66 \times 10^{-6}$</td>
</tr>
<tr>
<td>Heat conductivity, $K$ [kJ/m/h/°C]</td>
<td>8</td>
</tr>
<tr>
<td>Heat capacity, $C^*$ [kJ/m$^3$/°C]</td>
<td>2'200</td>
</tr>
<tr>
<td>Thermal diffusivity, $\mu$ [m$^2$/h]</td>
<td>$36.36 \times 10^{-4}$</td>
</tr>
</tbody>
</table>

4. Model calibration and validation

The finite element model of the dam is validated, after a calibration of its parameters, through comparisons between measured temperatures and displacements, and corresponding calculated values.

First, a transient thermal analysis of the dam is performed. Depending on exposition to air or water, air or water temperature is applied on dam’s external surfaces (thermal boundary conditions) over a 12-years time span (1997 to 2009) and corresponding temperature fields in the dam are calculated. Figure 4 shows measured temperatures at four dam locations and corresponding calculated values. For all thermometers, a good agreement is noticed on the whole considered period.

A thermo-mechanical analysis is then carried out for the same time span considering as mechanical loads, in addition to dam’s self weight, the hydrostatic pressure corresponding to water level variations and thermal gradients obtained from previously calculated temperature fields. Dam’s crest calculated displacements are compared to measured displacements in all directions (river-stream, vertical and cross-stream). As shown on Figure 5, a very good agreement between measurements and calculated displacements is obtained.

The calculated temperatures and displacements compare well to measured temperatures and displacements over a long time period. It can be therefore
considered that the finite element model of the dam reasonably represents dam’s behaviour under thermal and mechanical loads. This model is thus retained to evaluate the dam behaviour under static and dynamic loads.

Figure 4. Calculated vs. measured temperatures in the dam

Figure 5. Calculated vs. measured displacements at dam’s crest
5. Eigen modes and frequencies

In dynamic time history analyses, the total mass is considered in the equation of motion, so there is no need to evaluate cumulative mass contribution of eigen modes. However, the determination of Rayleigh damping coefficients requires the definition of a frequency range for which damping is below critical damping. On the other hand, free vibration modes help to understand dam’s complex behaviour during an earthquake. Dam’s significant modes and corresponding mass contribution are evaluated through an eigen frequency analysis. Figure 6 shows the fundamental mode at full reservoir water level.

![Figure 6. Rossinière dam’s fundamental eigen mode (19.3 Hz)](image)

6. Seismic safety assessment

6.1 Initial static conditions

The dynamic response of the dam is evaluated considering the following initial static conditions: self weight, upstream and downstream hydrostatic pressure, silt pressure load and a temperature gradient corresponding to winter or summer load case (Figures 7-8). Uplift pressures (Figure 9) are considered only in global stability calculations, not in stress analyses.
Figure 7. Hydrostatic and silt pressure at initial state

Figure 8. Summer and winter temperature fields
6.2 Seismic load

6.2.1 Peak ground acceleration and accelerograms

According to the Swiss seismic hazard map [4], the intensity $I_{\text{MSK}}$ for 10’000 years return period at Rossinière dam site is 8.6, to which corresponds a horizontal PGA (peak ground acceleration) of 0.27 g. The vertical PGA is taken as 2/3 of the horizontal PGA. Horizontal and vertical response spectra are defined according to the Swiss norms [2]. Three sets of 3-components accelerograms, compatible with site response spectra, are then generated according to Simqke approach [5]. Figure 10 shows the 3 components (river-stream, vertical and cross-stream) of a generated accelerometer set and corresponding response spectra.

6.2.2 Hydrodynamic pressure

The hydrodynamic load resulting from oscillating mass water during earthquake is evaluated using (a) generalized Westergaard approach for river-stream motion and (b) mass of water between buttresses for cross-stream (lateral) motion.

6.2.3 Damping

Damping matrix $C$ is assumed as a combination of mass matrix $M$ and stiffness matrix $K$: $C = \alpha_0 M + \beta_0 K$, where $\alpha_0 = 5.236$ and $\beta_0 = 0.00026526$ are Rayleigh coefficients corresponding to a maximal critical damping of 5% in the frequency range 10-50 Hz.
6.3 Dynamic response

The dynamic response of the dam is evaluated using a direct time integration scheme, with a Hilbert-Hughes-Taylor algorithm (HHT, $\alpha = -0.3$). At each time step, displacement and stress fields are calculated. At any dam location, displacement and/or stress time histories can then be retrieved and analysed.

Figure 11 shows the envelope of principal stresses for “winter – full reservoir” load case. It corresponds to the time history of extreme stresses in the dam structure. An analogous envelope is obtained for “summer – full reservoir” load case.
To evaluate the extent of extreme tensile (or compressive) stresses, principal stress fields for different dam parts are plotted at time instants corresponding to peak values of stresses. Figure 12 shows the major principal stress field in the central dam buttress at time of peak tensile stress for “summer – full reservoir” and “winter – full reservoir” load cases.

Figure 12. Dynamic principal stress field S1: (a) summer, (b) winter

6.4 Concrete strength assessment

Extreme dynamic compressive and tensile stresses are evaluated in all dam structural parts. A maximum compressive stress of 3.8 MPa is reached for “summer – full reservoir” load case and is located on downstream side of the footbridge. The maximum tensile stress is 3.4 MPa and is obtained at the same location for “winter – full reservoir” load case. Both compressive and tensile
extreme stresses are below dynamic concrete compressive and tensile strength, estimated as 60 MPa and 5.6 MPa respectively. Therefore, dam’s concrete strength is sufficient to withstand an earthquake with 0.27g peak ground acceleration.

6.5 Global dam stability assessment

The stability of the dam against sliding and overturning during an earthquake is evaluated considering the following assumptions:

- Analysis of 3 buttresses with two half-openings
- Initial static loads: self-weight, hydrostatic pressure at full reservoir level, silt load, uplift pressure (100% of total pressure, evolution according to average measurements)
- Resisting lateral forces (3D) are neglected
- Earthquake: horizontal and vertical components
- Evaluation of stresses at time of maximal upstream-downstream crest displacement

Resultant forces and moments are determined by integration of normal and shear stresses on potential sliding surfaces. Sliding and overturning factors of safety are then calculated for each buttress individually and for the whole dam structure. As shown in Table 3, factors of safety are greater than 1 either for stability or overturning. Therefore, the global stability of the dam during the considered earthquake is satisfied.

<table>
<thead>
<tr>
<th></th>
<th>Sliding</th>
<th>Overturning</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Static</td>
<td>Dynamic</td>
</tr>
<tr>
<td>Left buttress</td>
<td>2.78</td>
<td>1.50</td>
</tr>
<tr>
<td>Central buttress</td>
<td>3.96</td>
<td>2.19</td>
</tr>
<tr>
<td>Right buttress</td>
<td>1.97</td>
<td>1.47</td>
</tr>
<tr>
<td>All buttresses</td>
<td>2.99</td>
<td>1.77</td>
</tr>
</tbody>
</table>

6.6 Hydraulic equipments (gates) assessment

From safety point of view, it is crucial that the normal operation of the hydraulic gates is guaranteed after an earthquake. In addition to steel strength assessment, jamming hazard of the gates as well as buckling hazard of hydraulic jacks during earthquake is evaluated.

6.6.1 Steel strength

Extreme dynamic stresses in hydraulic gates are compared to steel strength both in tension and compression. Figures 13 and 14 show the principal stress
fields in flap gates and outlet gates at time of maximum displacement which corresponds to fundamental mode of vibration. Extreme tensile and compressive stresses are well below the steel strength (about 350 MPa). The strength criterion of hydraulic gates is therefore satisfied.

Figure 13. Dynamic principal stress fields in flap gates (winter)

Figure 14. Dynamic principal stress fields in outlet gates (winter)
6.6.2 Jamming hazard of gates

During earthquake, the lateral motion of buttresses can potentially be asynchronous and the initial opening width reduced or increased. This may cause jamming of the gates between the buttresses if the initial gap between the gate and the buttress is completely closed. Therefore, it is necessary to evaluate for each gate, the evolution of the spacing between the gate and the buttress during the seismic motion.

Figure 15 shows the relative displacements of lateral buttresses during earthquake calculated at top and down borders of each gate. The highest space gap reduction is 0.3 mm for the flap gate and 0.12 mm for the outlet gate. These values are well below the 40 mm of available space between a buttress and a gate. Therefore, the risk of gate jamming during earthquake is excluded.

![Lateral relative displacements of buttresses](image)

Figure 15. Dynamic lateral relative displacements of buttresses (summer)

6.6.3 Buckling hazard of jacks

During earthquake, the efforts on hydraulic gates transmit compressive forces to the hydraulic jacks (Figure 16). If the normal force exceeds a critical value, buckling of the jack may occur. Therefore, it is necessary to evaluate for each jack, the evolution of the dynamic normal effort and compare extreme calculated values to the critical normal effort defined as:
\[ N_{cr} = -\frac{\pi^2 EI}{l_0^2} \]  \hspace{1cm} (1)

\( l_0 \) is a characteristic length, equal to the jack length (hinged supports); \( E \) is the elastic Young modulus of steel; and \( I \) is the moment of inertia of the jack’s rod.

Figure 16. Representation of normal efforts in hydraulic jacks

Maximum calculated normal efforts in the flap gate’s jack and in the outlet gate’s jack are respectively 420 kN and 200 kN (summer load case). The corresponding critical normal efforts are 1277 kN and 366 kN. Therefore, the risk of jack buckling during earthquake is excluded.

7. Conclusion

A 3D numerical finite element model of Rossinière dam including lateral wing buildings, hydromechanical equipment (gates) and foundation was developed to evaluate seismic safety of the dam according to Swiss norm requirements. Model preparation and analyses were carried out using FE software Z_Soil 3D. A large number of the program’s features were successfully used.

Calibration and validation of the model, through transient thermal and thermo-mechanical analyses, showed a very good agreement with measured temperatures and displacements over several years.

Dynamic stress analyses show that compressive and tensile stresses remain below concrete strength. The stability analyses of the dam indicate that neither sliding nor overturning may occur under the considered loads. Assessment of the metallic gates stresses and deformations during earthquake indicates that dynamic compressive and tensile stresses remain within steel strength domain and that the risks of gates’ jamming and jacks’ buckling are excluded. Therefore, the seismic safety of Rossinière dam is fulfilled.
8. Acknowledgement

The project was funded by Groupe E, Switzerland. The author wishes to thank the dam owner for providing all necessary drawings and data to complete this study.

9. References


Analysis of the as-is-state of old natural stone barriers in the Bernese Alps

Bernd Kister
Lucerne University of Applied Sciences and Arts, Horw, Switzerland

Keywords: natural hazards, torrent control structures, natural stone masonry, as-is-state of structures, rating systems for rock mass and natural stone masonry, structure-rock mass-interaction, potential failure mechanisms, 3D-FEM

Abstract
Within the framework of the revision of the natural hazard map concerning the area of the villages Schwanden, Hofstetten, Brienz and Brienzwiler in the Bernese Alps assessment and rating for the as-is-state of the 100-years old torrent control structures in the Lammbach rift valley had to be done. This paper shows the general course of action in the project which was done on the basis of the 5 step procedure of the PLANAT (PLAtform NATurgefahren). Special boundary conditions concerning the data collection of constructions and subsoil are discussed as well as characteristics of the different torrential barriers. For the visual inspection of the natural stone masonry of the torrent control structures the “Lucerne Rating System” has been used. For the check of the structural safety as well as the fitness for purpose of the structures simple 2D-models have been used just as well as 3D-FEM. Some of the results are presented here.

1. Introduction
The mountain torrents of the Brienz area are well known for their flooding and debris flows for hundreds of years. In the year 1896 disastrous debris flows occurred in the Lammbach rift valley which destroyed several houses in the village Kienholz, along with the track of the Swiss Federal Railway. The volumes of the last three incidents have been reported as approximately 300'000 m³ of material covering land along a shoreline length of 120 m with an average debris thickness of about 2.5 to 4.0 m.
Figure 1: The project area is located north of the Lake Brienz.

In consequence of those disastrous debris flows, torrent control measures had been installed in the Lammbach rift valley to raise the streambed and stabilize the slopes. For this purpose 20 barriers had been constructed in the period 1896 to 1913. Those barriers consist of natural stone masonry and have remarkable dimensions in part. The largest one, barrier IVa, has a span of 90 m and the visible height at the downstream face is still 13 m today (Figure 5).

Since the construction of those barriers no disastrous debris flow with origin at the Lammbach rift valley has ever reached the villages. But on the other hand up to now by guess 500’000 m$^3$ of debris has been accumulated behind the torrent control structures and further 500’000 m$^3$ debris are deposited at the slopes of the Lammbach rift valley. For this reason there is a potential for large debris flows, especially if one or more of the barriers will fail to work. The structural safety of the old torrent control structures is therefore of utmost importance.

2. Topography and Geology

The watershed of the Lammbach is placed at the southern slope of the Brienzerrgrat (Figure 2) and covers an area of approximately 3.2 km$^2$. The area ranges from approx. 2200 m altitude down to the sea level of the Lake Brienz at 578 m altitude.
Figure 2: The 6 mountain torrents of the Brienz area located at the southern slope of the Brienzergrat (Ryter, 2004).

The Brienzergrat consists of the cretaceous rocks of the so-called Wildhorndecke. At the ridge siliceous limestones appear. Downhill these limestones are replaced by the marly layers of the Valangien. The dark siliceous limestones are normally straticulate and alternate with layers of marl. Compression, minor folds and buckling is very common in this geologic structure (Bauer, 1971).

The dip direction (DD) of the bedding plane in the project area is SE and the dip (D) is 25° to 35°. Three main joint sets K1, K2 and K3 have been identified (Table 1).

<table>
<thead>
<tr>
<th>Joint set</th>
<th>DD</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1</td>
<td>242°</td>
<td>63°</td>
</tr>
<tr>
<td>K2</td>
<td>334°</td>
<td>70°</td>
</tr>
<tr>
<td>K3</td>
<td>171°</td>
<td>69°</td>
</tr>
</tbody>
</table>

Table 1: Dip Direction (DD) and Dip (D) of the three main joint sets in the project area
Figure 3: Left: Example of small layered limestone in the project area, right: Distribution of measured joints and great circles of joint sets

The bedding planes have slightly rough surfaces while the joint surfaces can be described as rough. The Geological Strength Index (GSI) is in the range of 35 to 45. The rock mass at the Lammbach rift valley is in general vulnerable for weathering and erosion (Haldimann, 1992; Dasen, 1951).

Due to the existing joint system the limestone degrades into cubical or block-shaped pieces with an edge length of 1 to 4 decimeters in the majority of cases. The marly parts of the rock mass decays into fine grained sediments. At the surface the fine-grained material is often eroded and only the blocky material is left. Therefore along the streambed and at the slopes as well as with depth the composition of the debris varies.

Figure 4: Streambed of the Lammbach above barrier IVa, the surface is covered by coarse grained and blocky material while the eroded slope in the background shows soil material with a high percentage of fine grained soil
3. **The Lammbach barriers**

The Lammbach barriers, constructed during the period 1896 to 1913, had been made of natural stone masonry. Natural stone masonry is of course a durable construction material but even such a material underlies weathering and aging. The lifespan of such constructions made of natural stonework in general is specified with 60 to 80 years, maximum 100 years (e.g. Rudolf-Miklau & Agerer, 2007). Therefore the structural safety and the fitness for purpose of the barriers had to be checked.

Additional to weathering and aging of the masonry, due to old plans there are some specifics concerning the construction of the barriers:

- footing with wooden sleepers (barriers Ia, Ib, IIIa)
- wooden piles at the footing (barrier II)
- arch constructions within the masonry (barriers Ie, II, III, V).

Since the construction of the barriers, at some of them modifications had been done. So the barriers I, Ic, II, IV, V, VII and VIII had been increased in height. The barriers III and IVa got partly a concrete slab on downstream face with pre-stressed anchors (see e.g. Figure 5).

![Figure 5: Barrier IVa with rehabilitation measures, middle part fixed by a concrete plate and prestressed anchors in 1976, buttresses and pre-stressed anchors installed at the barrier wings in 2000](image)

4. **Procedural method**

The Swiss National Platform for Natural Hazards (PLANAT) defines a 5 step procedure concerning the assessment and the effect of protective structures. The steps are named as follows:

1. evaluation of the basics
2. process assessment
3. arrangement assessment
4. outcome assessment
5. realization

The course of action used in the project follows this 5 step procedure. Focus will be set here to the steps 1 to 4. Step 5 is in this case the realization of the natural hazard map.

4.1 Evaluation of the basics

In the first step the available documentation has been analyzed, a geodetic survey has been done and the barriers’ as-is-state has been mapped. The last one has been done by using a special rating system, which has been developed at the Lucerne University of Applied Sciences and Arts in the last years (see e.g. Kister et al, 2008 a, b).

The new rating system for natural stone masonry has been developed based on our experience with rating systems in rock mechanics on one hand and our experience with natural stonework on the other. Combining both and additionally taking into account aspects of the surrounding area as topography, geology, groundwater and plant-cover, the assessment and rating of old natural stonework has been put on a less subjective base.

Therefore an assessment of the barriers made of natural stone masonry has to keep in mind

- the as-is state of the construction,
- the constructiveness and
- the properties of the surrounding area.

The as-is-state of the construction implies the as-is-state of the construction itself, as for example break-outs in the masonry or damage due to settlement, but also the as-is-state of the components stone and mortar and the status of the drainage. The constructiveness sums up data concerning the design and composition of the structure, such as ratio of masonry wall thickness to construction height, size of stones and thickness of joints. Properties of the surrounding area are plant cover and indication of slope instabilities for example.

Constructions made of natural stonework are partly large-sized and some times there is a change in stone size or even in bond type within a construction. During the life cycle of a structure large-scale changes may have happened, done with materials different to the original construction materials. In all these cases one has to divide the structure in so-called homogeneous segments. Homogeneous segments are characterized by the following features for example:

- only one type of bond exists
- the structure of the masonry varies only marginal
no significant changes in geometry

The evaluation of the basics includes also the determination of material parameters of the barriers as well as of the rock mass. To do this, it had to be taken into account, that the access to the barriers is only possible via hiking trail or by helicopter flight. Therefore easily manageable methods and non-destructive methods have been preferred for this work.

So for the determination of the uniaxial compressive strength of the rock as well as of the mortar a Schmidt-hammer, type L, has been used in the field. The determination of the uniaxial compressive strength values has been done by using the formulas of Deere & Miller (1966).

A small drilling equipment has been transported to the barriers IV and IVa by helicopter and at each of the both barriers a horizontal drilling has been executed. To confirm the results produced with the Schmidt-hammer some compressive tests on drill cores have been done additionally. Furthermore the drillings of the barriers have been inspected by a borehole camera.

Comparing the test results, it can be mentioned, that there is a difference in the compressive strength of the rocks used for the barriers. The compressive strength of the rock used for barriers IV and IVa was less than the compressive strength of the rocks used for the barriers Ia, Ib and Ic of the Lammbach rift valley. Figure 6 shows the results of Schmidthammer tests in comparison to the results of uniaxial compression tests.

Figure 6: Uniaxial compressive strength of the siliceous limestone, dark bars: frequency distribution of Schmidt-hammer results, pink area: band of test results of uniaxial compression tests done with cores derived from barriers IV, IVa and rock blocs of the area between barrier IV and barrier IVa, green area: band of test results of uniaxial compression tests done with cores derived from rock blocs of the area barrier I to barrier Ia
Due to the test results for the calculation two types of limestone have been defined:

- type I with a low uniaxial compressive strength $\sigma_d = 40 \text{ MN/m}^2$ and
- type II with a high uniaxial compressive strength $\sigma_d = 100 \text{ MN/m}^2$.

The rock mass parameters for those both types have been estimated using the Geological Strength Index (GSI) and the formulas of Hoek:

<table>
<thead>
<tr>
<th>rock mass parameters:</th>
<th>type I</th>
<th>type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>friction angle $\varphi_{RM}$</td>
<td>25 - 30</td>
<td>25 - 30</td>
</tr>
<tr>
<td>cohesion $c_{RM}$ [MPa]</td>
<td>1.0 - 1.5</td>
<td>2.5 - 3.5</td>
</tr>
<tr>
<td>uniaxial compression strength $\sigma_{ARM}$ [MPa]</td>
<td>4 - 5</td>
<td>10 - 12</td>
</tr>
<tr>
<td>Young’s modulus $E_{RM}$ [GPa]</td>
<td>2.5 - 4.5</td>
<td>4.0 - 7.5</td>
</tr>
</tbody>
</table>

Table 2: Rock mass parameters of the siliceous limestone type I and type II

The compressive strength $f_{d,M}$ of the natural stone masonry was evaluated to approximately 6 MN/m² by using the formula of Berndt (1996).

Another part of evaluation of the basics was to check the masonry status at the upstream face of the barriers and to verify some structural elements. This was done by excavating trial pits as spot samples at barriers IV and IVa. A walking excavator, the only construction machine which was able to reach the barriers IV and IVa on its own, has been used to do this work.

As a result of this work we can state a decomposition of mortar in the masonry joints on the upstream face which was predominantly less than 10 cm. At the same barriers the decomposition of mortar in joints at the downstream face often shows a greater depth, the maximum found at barrier IVa was 50 cm. The excavation of the trial pits eliminated also some lack of clarity concerning the thickness of the construction and the shape of the cross section.

4.2 Process assessment

In the second step the potential failure mechanisms and the hazard scenarios have to be detected and the impact forces acting on the constructions have to be determined. Figure 8 show some relevant load cases for damage or loss of effectiveness of barriers.
4.3 Arrangement assessment

The 3rd step of the PLANAT procedure deals with the estimation of the functional capability of the adopted measure. The structural safety as well as the fitness for purpose of the structures has to be checked. Hereby both, the structure-subsoil-interaction and weathering respectively aging, have to be taken into account.

An example for case C of Figure 8, loss of effectiveness due to aggradation
at the upstream side as well as at the downstream side, is barrier V. Today this barrier is covered by debris except for a very small part at the eastern barrier wing (Fig. 9). At the spillway the thickness of the debris layer is several meters (Fig. 10). Therefore barrier V has lost its fitness for purpose according to the primary design. But even in this status the barrier is of high importance for the complete torrent control system because in case of a large debris flow it limits the erosion of material to the level of the barrier’s crown.

Figure 9: a) completely covered area of the spillway of barrier V, b) visible part of barrier V.

Figure 10: Results of the photogrammetric survey in the Lammbach rift valley at the area of barrier V and barrier IVa, light blue line shows the surface in 1940 – the barriers V and IV are clearly visible as steps in the line, dark red line shows the surface in 2005 (Flotron AG, 2008).

Barriers III and IVa are examples for case B of Figure 8, increasing load at the upstream face of the barrier due to aggradation. Figure 11 shows the large debris cone acting on the eastern wing of barrier III. The results of the photogrammetric survey in the Lammbach rift valley in Figure 10 show the increase of material behind barrier IVa within a period of about 65 years.

It must be pointed out that, due to the geometry of both barriers on the one hand and the topography on the other hand, we have to deal with real 3D-
problems (see e.g. Figures 12 and 13). Therefore the FE-method has been chosen for the analysis of the structural safety of both barriers. Some of the results of the FE-calculations for barrier IVa will be presented in the subsequent sections.

Figure 11: Increasing load acting on barrier III due to aggradation.

Barrier IVa with a span of approximately 90 m is the largest one of the barriers in the Lammbach rift valley. At the spillway a barrier height of 19 m is reported, but there exist no plans dated from the construction phase during the period 1906 to 1912 or before. Two phases of reconditioning, in 1976 and in 2000, are documented.

In 1976 the middle part of the barrier was tighten with a concrete slab and pre-stressed anchors (see Figures 5, 12 and 13). This had to be done due to an approximately 4m x 4m break-out of the masonry with a depth of about 1 m and a crack which ran down the complete visible masonry starting at the eastern edge of the spillway and running down at least to the level of the debris in the streambed. That failure of the masonry is very poorly documented, no photos exist. There is just a sketch of the break-out and the crack in the barrier inserted in a drawing. Additional the trial pits at barrier IVa showed us some differences between the 1976- drawings and the real executed work of that reconditioning phase.
In 1992 a survey report stated cracks in the barriers western wing and a danger of bulging. Due to this in 2000 a second reconditioning phase, now for the barrier’s wings, had been started. Concrete buttresses with pre-stressed anchors had been constructed, 7 at the western barrier wing and 3 at the eastern barrier wing (see Figures 5, 12 and 13).

Before we could start with the analysis of the structural safety of barrier IVa with FEM, we had first to determine the geometry of barrier IVa. But as mentioned before there exist no original plans from the construction phase and for this reason no information about the foundation depth and the foundation bed of barrier IVa. In the drawings of the two reconditioning phases these information are missing too. Additional problems are given by those drawings because different shapes of cross section of barrier IVa had been used. Therefore the geometry of barrier IVa had to be reconstructed in a complicated puzzle using 2 old photographs, the information about the volume of natural stone masonry used for the construction, the drawings of the 2 reconditioning phases, the data of the geodetic survey and the data obtained with the trial pits. The result is given in Figure 13.

Based on the geometry of the Figures 12 and 13 a 3D-FE-mesh has been developed with the program Z-Soil which represents the body of barrier IVa as well as the surrounding area. Figure 14 shows the FE-mesh consisting of 25’160 3D-elements, 49 discrete anchor elements and 1’393 interface elements.
Figure 13: Geometrical model for barrier IVa
3D-Finite-Element-Mesh

3D-elements: 25'160
anchor elements: 49
interface elements: 1'393
number of materials 20

Figure 14: FE-mesh of barrier IVa and surrounding area.

For the calculations done for barrier IVa 22 “time steps” have been used with the program Z-Soil 3D to simulate the different phases starting with primary stress calculation, then construction of the barrier in 2 steps, backfill, increase of water level up to 9 m below spillway, installation of concrete plate and anchors at the middle part, installation of buttresses and anchors at both flanks, increase of water level up to spillway and as the last “time step”, loading due to debris flow.

For the load case “debris flow” a surface load has been applied which is equivalent to a soil layer with a 3 m thickness and the width of the spillway (surface load “debris flow” is shown in blue in Fig. 14). Up to now only very few information is available about the tractive force of debris flows. Therefore as a first approach the load vectors have been inclined by adding a horizontal component which is approximately 9% of the vertical load.

The calculations have been done using linear elastic and elastic-plastic material behavior for the barrier. Mohr-Coulomb failure criterion and a tension-cut-off model as well as the multi-laminate model have been used to simulate the masonry. For the subsoil and the securing mean (concrete slab, buttresses and anchors) elastic behaviour has been chosen.

The influence of different values for the tension-cut-off model concerning the plastic zones is shown in Figure 15. Without a tension-cut-off model the barrier behaves almost elastic and this results in large tensile stresses in the structure. But this is not a realistic behaviour for the masonry. With the tension-cut-off model the tensile stresses will be reduced, but it is linked with an increase of the plastic zones. The more the tension-cut-off parameter is reduced, the more the size of the plastic zones is enlarged. But even in the case, if the tension-cut-off parameter is set to the low value of 10 kN/m² the deformation of the barrier is limited.
Figure 15: Comparison of the plastic zones of a model without tension-cut-off and three models with tension-cut-off at different levels for time steps 6 and 22.

The calculations for barrier IVa show a deformation which was less than 10 cm in maximum (see e.g. Fig. 16). The check of the anchor load show a value which is nearby, but lower than the tolerable value. Therefore even if the water level rises up to the level of the spillway and a debris flow occur additionally the calculations show no failure of barrier IVa.
Figure 16: Deformation of barrier IVa at time step 22 (after activated load “debris flow”) for a reduced compressive strength $f_{LM}$ of the natural stone masonry

### 4.4 Outcome assessment

The outcome assessment, the 4th step of the procedure, handles the evaluation of uncertainties and the resulting risks. Uncertainties are given for example by incomplete project documentation of the construction as described before. To counteract those problems analysis with modified geometry may be done as it is shown in Figure 13 for example. The numerical method used here is also able to take into account the influence of variance of material parameters and loads.

### 5. Conclusions

The assessment of the 100-years old torrent control structures in the Lammbach rift valley is a very complex and ambitious task which necessitate the close collaboration of experts of different disciplines. As shown before not all barriers in the Lammbach rift valley show the fitness for purpose and the structural safety after a period of 100 years. But with the reconditioning done in the past the analysis of barrier IVa showed the structural safety of that barrier. And this results in a significant reduction of the mass of a potential debris flow, which may reach the inhabited area at Lake Brienz in case of a 300 year event.
Uncertainties are given especially by missing or ambivalent data concerning the barriers’ geometry and composition. Not all of those uncertainties can be eliminated within reasonable operating expense by exploration and investigation. Additional there is an incomplete knowledge concerning debris flows. Further research is needed to get a better understanding of the debris flow load cases and to reduce uncertainties in this domain.

Acknowledgements

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Furthermore the author is indebted to Mr. Jean-Luc Sarf for his assistance with Z-Soil 3Dmodelling

6. References


Numerical Modeling of Thermal, Filtration and Mechanical Phenomena in a Selected Section of a Gravity Dam

Aleksander Urbański*
*Cracow University of Technology
Department of Environmental Engineering
Institute of Geotechnics
ZACE Services Ltd

Keywords: concrete dam, monitoring, thermal effects in structures, filtration, displacements, stresses.

Abstract
The paper presents course and results of numerical simulations performed on a selected section of the Solina Dam. Thermal and water pressure fields variable in time were taken into account with their influence on displacement and stress states in the dam. Results of the analysis were compared with measurements.

1. Introduction

In the years 2007-2008 the author took part in a multi-disciplinary research group aimed at investigation and evaluation of the technical state and safety of the Solina Dam. Solina Dam, the greatest gravity dam in Poland, was built in 1965. It is located on San river in Bieszczady mountains, in the south-east of Poland.

The research which was carried out, consisted of: assessment of measurement records of displacements, pressures and temperature fields gathered in about 10 year time period, assessment of quality of concrete after 45 years. This was confronted with the results of the numerical simulation of the behavior of the selected section No 22, for which the record of data was the most complete. An overview, location and basic technical data, together with the archival drawing of the selected section (No 22) are given in the Fig.1.

Numerical analysis of transient temperature, water pressures fields and their influence on mechanical fields (displacement, stresses) was intended to verify that measurements of one field were consistent with another, and if safety of the dam was assured.

Analysis was performed with the use of Z_Soil.PC v. 2007.
2. Thermal analysis

2.1 Assumptions, data and computational schema

The aim of the analysis is to create an image of the space distribution and time evolution of temperature fields inside section 22 of the dam during the yearly cycle of climatic condition (temperature of water and air). Temperature results will be used in mechanical (static) analysis, where non-uniform distribution of the temperature may prove to be the crucial factor generating tensile stresses and subsequent cracking of the dam.

Temperature fields \( T(x,t) \) in the dam and its surroundings is described by the Fourier’s equation (1), together with proper boundary conditions (BC).

\[
(\lambda \ (T_{,x}))_{,x} = c \ T .
\]  

Following characteristics were assumed:
- heat conductivities \([\text{W/(m·°K)}]\):
  \( \lambda_1 = 1.8 \) for concrete (according to PN –91/B-02020), \( \lambda_2 = 3.0 \) for the bedrock,

Fig. 1. Solina Dam. Overview and basic hydrotechnical data. Section nr 22.
- heat capacities \([kJ/(m^3\cdot K)]\):
  \[c_1 = 2016\] and \[c_2 = 2362\], respectively.

Three types of BC are involved:

**Known temperature BC (1-st kind)**, \(T(x,t)=T_s(h,t)\), are applied there, where temperature on the boundary is imposed by the contact with an external medium having large heat capacity and known temperature \(T_s\), at the time instant \(t\), on depth \(h\). In the model of the section, this was:

1a – influence of water. Time records of water temperature measurements at points marked as: Tw601 (level 369.5m), Tw602 (l. 401.5), Tw603 (l. 412.0) were taken. For points between them linear interpolation has been employed.

1b – intact temperature in surrounding soil equal to average yearly temperature (7°C for the Bieszczady region).

1c – temperature in the zone of influence of the power station building (10°C)

**Adiabatic BC (2-nd kind)**, \(q_n=0\), are applied there, where heat flux is meaningless. In the model this was assumed on the lateral boundaries, and also
in the concave zones filled with water being in thermal equilibrium with the massive of concrete. Adiabatic conditions were applied also in control galleries assuming no air flux.

**Convective BC (3-rd kind),** \( q_{n} = \alpha_{c}(T \cdot T_{e}) \), on the surfaces where heat exchange takes place with the surrounding fluid (i.e. air) having known ambient temperature \( T_{e} \). In the model, convective BC were assumed on the downstream face and on the upstream face above the water surface. Convection coefficient (according to standard PN-91/B-02020 for concrete wall) \( \alpha_{c} = 23 \text{ [W/(m·°K)]} \).

External temperatures of water and air has been assumed as 10 yearly cycles given in the form:

\[
T_{e}(t) = T_{A}(1 - \Delta T/(2T_{A}) \cdot \cos(2\pi t/365))
\]

(2)

where \( T_{A} \) is yearly average temperature, \( \Delta T \) - is the amplitude of monthly averaged temperatures. These values were taken from the temperature data for the year 2006. In the last period of simulation (07.2006-07.2007) explicitly measured values of temperature have been imposed. Time histories of external temperatures are given in Fig. 3.

<table>
<thead>
<tr>
<th>Parameters of yearly temperature cycles</th>
<th>( T_{A} ) [°C]</th>
<th>( \Delta T ) [°C]</th>
<th>( \text{Tmin} ) [°C]</th>
<th>( \text{Tmax} ) [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>water on 369.5</td>
<td>6.5</td>
<td>0</td>
<td>6.5</td>
<td>6.5</td>
</tr>
<tr>
<td>401.5</td>
<td>9</td>
<td>6</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>412.0</td>
<td>13.5</td>
<td>22</td>
<td>2.5</td>
<td>24.5</td>
</tr>
<tr>
<td>Air</td>
<td>11</td>
<td>28</td>
<td>-3.0</td>
<td>25</td>
</tr>
</tbody>
</table>

Fig. 3. Time records of external temperatures of water and air
Fig. 4. FE mesh used in thermal, filtration and mechanical analysis: a) rough, b) dense.
First step of the simulation was to set the initial conditions by solving steady state problem at time $t=0$. Next, numerical integration with time step equal to $\Delta t_1=10$ days has been performed in 10 years time range and with $\Delta t_2=7$ days for the last period. The aim of using such prolonged procedure was to stabilize the response of the system submitted to periodic excitations, which at first few periods was perturbed by the influence of initial conditions. The reason for this discrepancy is that steady state solution differs substantially from the sought solution for the quasi-periodic state, particularly in points distant from the boundaries. Equation (1) is solved by FEM in the 3D domain representing the section and its subsoil. Backward Euler time integration schema has been adopted, see [1].

For each problem (thermal and filtration) two numerical model have been built: rough (app. 10000 nodes), Fig 4.a, and dense (app. 40000 nodes), Fig. 4.b. In this way a possibility of verification and assessment of the obtained results has emerged.

In FEM model kinematic constraints, see [1], are applied in the subsoil zone, allowing to perform of this multi-step simulation on PC -Pentium 4, 2.4GHz, 1MB RAM.

### 2.2 Results of thermal analysis

Fig. 5. shows obtained temperature time histories in 10 years simulation at selected (including some measurements) points. It is visible, that influence of initial conditions (steady state) is gradually vanishing (see t. h. for point No 20278 and 17632 ) and response becomes periodic. Practically, starting from cycle No 5, thermal response is acceptable as an input to mechanical analysis.

![Influence of initial condition (steady state) gradually vanishing](image)

Fig. 5. Temperature time histories at selected points in 10 years simulation.
Fig. 6. Temperature distribution inside the dam section: a) in the winter, b) in the summer.
Fig. 6 shows the temperature field at the vertical plane in the mid of the section. It concerns two representative moments of winter (Jan 2007) and summer (Jul 2007) periods.

Analysis indicates the appearance of the large temperature gradients in the zone of the downstream face. It is so due to large thermal inertia of concrete mass, which in the middle keeps temperature close to the yearly average, while the external surfaces are submitted to heating in summer and cooling in winter periods. Resulting thermal strains are possible source of different mechanical effects, which will be analyzed in chapter 4.

3. Analysis of the filtration through the dam

The goal of the filtration analysis is to create an image of the water pressure fields in the concrete section. Thus, flow through the dam subsoil is treated only in the approximate manner. Detailed analysis of the flow would require numerical model and set of data concerning whole structure and its surroundings. In the recent analysis of the flow in one section presence of internal cavities between neighboring sections, filled with water has been assumed.

Because of small variability of water levels analysis is limited to the steady state with level WG = 416.2 m. above sea level, only in last period of simulation (6 months) measured water levels were taken into account. Water level in internal cavities was kept constant WF= 364.5 m. above sea level, basing on observations.

In the analysis in order to evaluate the free surface, nonlinear filtration model of Van Genuchten has been applied. Seepage surface elements were used to model the leakage zone on the downstream face and also on the surface of internal cavities. Fig. 7. shows adopted BC for the filtration problem.

Fig. 8. shows the obtained pressure field in the middle of the section, Fig 9. - pressures in horizontal cross-section at 362.5 m. and 380.5 m. above sea level. Distribution of filtration pressures shows the draining role of internal cavities. Visual observation of the upper part of the upstream wall confirms the appearance of leakage zone.
Fig. 7. Boundary condition for filtration problem

\[ p(y,t) = -\gamma(h_0 + LTF(t) - y); \]
\[ h_0 = 1 \]

2. seepage b.c

3. \( q = 0 \)

Fig. 8. Filtration pressure in vertical section

Pressure fields

leakage zone

in the mid cross-section

in the side of section 22
4. Static analysis

4.1 Methodology and assumption

The goal of the analysis is to find displacements, strains and stresses in the section of the dam as well as its evolution in time. On the structure acts: gravity force, water pressures on the external surfaces, body forces from pressure gradients, internal temperature field. The following sequence of events has been considered:

1. \( t = 0 \), initial state, gravity load. Accompanying deformation is disregarded.
2. \( t = 1 \), water pressure up to the level \( WG = 416.4 \) m. above sea level.
3. \( 2000D < t < 3650D \), about 4 yearly cycles of imposed strains (basing on artificially created external temperatures using Eq. 2 and data in Fig. 3).
4. \( 3655D < t < 3826D \), state from 01.01.2007 to 13.07.2007, simulation under measured temperatures and variations of water level.

Results from precedent analyses i.e. temperature field increments \( \Delta T \), water pressure field \( p \) and saturation ratio \( S \) are used as input for subsequent static analysis. As both the input (temperature field) and the response of the media are time dependent, the formulation of the mechanical problem is based
on an incremental approach in time. It is worth noting that FE mesh used during static analysis (rougier) differs from these of thermal analysis (denser). Also the time integration schema of static analysis is different, as it covers shorter time (about last 5 years) than thermal analysis. Starting from the initial state, at each step of the analysis increments of thermal strains are imposed. These are evaluated in Gauss points of mechanical FE mesh basing on values of temperatures given at nodes of thermal FE mesh.

\[
\Delta \varepsilon_\theta^0(x,t) = \alpha \Delta T(x,t) \delta_\theta
\]  

Coefficient of thermal dilatancy is assumed to be constant and equal to: 
\[\alpha = 1.13 \cdot 10^{-5}[1/{^\circ}K].\]

If the thermal strains field is linearly variable in space then it fulfills compatibility equation and thus does not produce thermal stresses. In the case of the considered dam, field of imposed strains differs substantially from the linear one (see Fig. 6), thus appearing thermal stresses should be investigated.

Water level is kept constant at the begin of simulation, only in the last period is based on measurements.

In lateral surfaces of the numerical model no-tension elements have been introduced (Fig. 10), in order to simulate the presence of neighboring sections (in summer periods - compression, in winter - free deformation).

Two variants of concrete model were considered during computations:
- elastic (basic),
- elastic with creep (more realistic).

In elastic model with creep we have:

\[
\sigma^{ep+} = \sigma^e + D(\Delta \varepsilon - \Delta \varepsilon^e - \Delta \varepsilon^{cr})
\]  

\[
\varepsilon^{cr} = D_{\theta}^{cr} \sigma C(t,\tau)
\]

where \(D_{\theta}^{cr}(v) = \frac{1}{E}D(E,v)\) is elastic matrix for \(E=1\).

Creep function is of exponential type:

\[C(t,\tau) = A(1 - \exp(-\frac{1}{B}(t-\tau)))\]

Creep coefficient \(A\) has been estimated on Polish Code for Concrete Design (PN/B-03264, [7]). Humidity RH = 80% (outside), age of concrete in load application \(t_0=365\)d, and comp. strength B30, give factor \(\phi = 1.0\), which generates \(A = \phi/E = 3.268e-8[1/kPa]\). Retardation time, not given in [7], was assumed after Aleksandrowskiij, see [2], \(B=33.3D\).
Section 22. Loads and BC for mechanical analysis

Fig. 10. Loads due to water pressure. No-tension elements

4.2 Material data

Mechanical properties for concrete have been examined on samples taken from the structure, remaining material data (foundation) were assumed form archival data. They are given in Tab.1. Material zone distribution is given in Fig. 4.

Table 1.

<table>
<thead>
<tr>
<th>Nr</th>
<th>Material</th>
<th>Dead weight $\gamma$ [kN/m$^3$]</th>
<th>Young modulus $E$ [GPa]</th>
<th>Poisson’s ratio $v$ [-]</th>
<th>Thermal dilatancy $\alpha [1/°K] \times 10^{-5}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>23,55</td>
<td>34,22</td>
<td>0,17</td>
<td>1,13</td>
</tr>
<tr>
<td>6</td>
<td>Injected subsoil</td>
<td>26,00</td>
<td>9,20</td>
<td>0,30</td>
<td>1,30</td>
</tr>
<tr>
<td>8</td>
<td>Retention wall</td>
<td>10,10</td>
<td>11,47</td>
<td>0,13</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Sandstone</td>
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<td>12,00</td>
<td>0,30</td>
<td>1,0</td>
</tr>
<tr>
<td>11</td>
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<td>4,00</td>
<td>0,29</td>
<td>1,0</td>
</tr>
<tr>
<td>17</td>
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<td>0,37</td>
<td>1,0</td>
</tr>
<tr>
<td>19</td>
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<td>0,15</td>
<td>1,13</td>
</tr>
<tr>
<td>20</td>
<td>Silt-slates injected</td>
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<td>0,80</td>
<td>0,37</td>
<td>1,0</td>
</tr>
<tr>
<td>24</td>
<td>Sandstone-slates II</td>
<td>20,94</td>
<td>3,38</td>
<td>0,29</td>
<td>1,0</td>
</tr>
</tbody>
</table>
4.3 Results of static analysis

4.3.1 Deformation

One of the main goals of the analysis was to compare simulated displacements with corresponding values measured by the pendulum hanging in the vertical control gallery, particularly in the last period (01.2007-07.2007), after replacement of some part of the measurement equipment. Comparison of displacement amplitudes in horizontal direction (UX) is given in the Fig. 11.

Computations show that the main reason of observed deformation are yearly cycles of variation of temperature acting on the dam (water and air). Total displacements UX at the level of gallery G4 related to water level from 416.9 to 419.5 m a.s.l. disregarding temperature are equal to from 10 to 13 mm. Considering both influences (temperature changes and variable water level) UX= 22 mm (winter) and UX=10 mm (summer).

In the period of comparison (winter 2006/07- summer 2007) a very good coincidence of computed amplitudes of the relative displacements UX (Gi-G1, i=3,4) with the result of measurements by pendulum was obtained. It confirms correctness of the estimation of: material data such as elastic characteristics of concrete, thermal input (measured values) and its representation in numerical model (BC, convection coefficients) as well as pore pressures field.

![Fig. 11. Time history of relative displacements UX. Measurements and simulation](image-url)
4.3.2 Stresses

Fig. 13 shows the map of principal stresses $\sigma_1$ in winter obtained a) for elastic model with creep, b) for elastic model without creep. Fig. 14 shows principal stress crosses on the surface of the model (with creep) in periods: a) winter b) summer.

Stresses evaluated during simulations are strongly dependent from thermal effects. A particularly vulnerable part of the structure is the downstream wall, where in winter, large tensile stresses may appear, reaching the value $\sigma_{CR}=3.5\text{MPa}$ (considering creep) - see Fig. 13a. An analogous value for the elastic model of concrete, $\sigma_E=8.0\text{MPa}$, (Fig. 13b) should be regarded as non-realistic, because of the lack of relaxation phenomenon. Maximum tensile stresses appear close to the surface (1 m in depth), at height of about 20 m from the dam footing. They could be source of degradation (cracking) of the wall surface, which should be repaired during renovation, but does not create immediate risk for the structure safety. In the remaining zones/time stresses do not exceed tensile or compressive strength of concrete.
Fig. 13. Principal stress $\sigma_1$ in winter season: a) creep in concrete, b) no creep in concrete

$\sigma_{1\text{max}} \approx 3.5 \text{MPa}$

Fig. 14. Principal stress crosses with creep in concrete: a) in winter, b) in summer

$\sigma_{1\text{max}} \approx 8.0 \text{MPa}$
5. Final conclusions

The fundamental conclusion for the estimation of the safety of the dam, resulting from the numerical simulation of the multi-field problem, is that all observed phenomena are fully explicable and are related to external actions. Moreover, the safety of the dam is not endangered in the normal exploitation regime.

Once again, the thesis has been confirmed, see [3], [4], [5], [6], that main source of the observed stress state and deformation in a massive concrete dam is the influence of variable temperature of water and air in yearly climatic cycles.

Moreover it has been shown, similar like in [8], [9], that Z_Soil.PC code is efficient in solving multi-field, evolutionary 3D problems resulting from practical needs of hydraulic engineering. Aleksandrowkij

6. References

A 3D Push-Over Analysis of a Motorway Exit Bridge

Stéphane Commend*, Michel Capron**
* GeoMod ingénieurs conseils SA, Lausanne, Switzerland
** BG ingénieurs conseils SA, Lausanne, Switzerland

Keywords: bridge seismic assessment, displacement-based method, push-over

Abstract
This paper describes the 3D push-over analysis of a motorway exit bridge located near Neuchâtel (Switzerland) is described in this paper. The Z_SOIL.PC software [1] has the capability to run such analyses, both in 2D and 3D.

1. Introduction
Seismic assessment of existing structures through displacement-based methods has gained increasing attention in Switzerland in the last ten years [2]. The use of such methods tends to be less conservative than conventional replacement forces methods, leading to potential savings when deciding whether or not to intervene on the structure. In this paper, we describe the application of such a displacement-based method – the push-over approach – with Z_SOIL.PC on the seismic assessment of a motorway exit bridge located near Neuchâtel (Switzerland). In section 2, we briefly recall displacement-based methods. In section 3, we describe the 3D finite element model of the structure. We then compute the target displacement corresponding to seismic demand in section 4, and we define the capacity of deformation of the bridge’s piles in section 5. Finally, in section 6, we sum up the principal results of the study and draw some conclusions.

2. Displacement-Based Method
2.1 Seismic Assessment of an Existing Structure
According to the Swiss norm SIA CT 2018 [2], a compliance factor \( \alpha_{cd} \) is defined as the ratio between the capacity of deformation of the structural element to be assessed \( w_{cd} \) (see section 5) and the deformation due to seismic action \( w_d \) (equal to the target displacement returned by the push-over approach, see sections 2.2 and 4).
\[ \alpha_{\text{eff}} = \frac{w_{\text{bd}}}{w_d} \]  

\( \alpha_{\text{eff}} \) is then compared to reduction factors \( \alpha_{\min} \) and \( \alpha_{\text{adm}} \) in order to evaluate the necessity to intervene on the structure. Reduction factors depend on the class and the life-span of the structure. Typically, in Switzerland, for a structure of class II (medium importance), \( \alpha_{\min} = 0.25 \) and \( \alpha_{\text{adm}} \) ranges between 0.40 and 0.90 (0.76 for a life-span of 50 years). When \( \alpha_{\text{eff}} < \alpha_{\min} \), something has to be done immediately. When \( \alpha_{\text{eff}} > \alpha_{\text{adm}} \), no intervention is necessary. When \( \alpha_{\text{adm}} > \alpha_{\text{eff}} > \alpha_{\min} \), the intervention has to be conducted if “proportionate”.

2.2 The Push-Over Approach

The push-over approach is a nonlinear static method described in Eurocode 8 [3]. Its implementation in Z_SOIL.PC is described in detail in [4]. In short, force distribution (unitary or modal) is applied to the structure and monotonously increased. A capacity curve is obtained, drawing the total shear force at the base of the structure with respect to the top displacement (in our case, the bridge’s deck). This curve is then expressed for an equivalent single degree of freedom oscillator and it is bi-linearized, giving birth to the so-called capacity spectrum (Figure 1).

![Figure 1. Capacity spectrum](image-url)
The seismic action depends on the type of the structure, soil conditions and the zone of application. It is expressed as an acceleration-displacement response (ADRS) spectrum, or demand spectrum. The superposition of both the capacity and demand spectra leads to obtaining target displacement \( w_d^* \) for the single degree of freedom oscillator, and finally to target displacement \( w_d \) for the real structure (Figure 2). This target displacement represents the maximal horizontal displacement which will be experienced by the structure during an earthquake corresponding to the given ADRS spectrum.

![Graph showing demand spectrum and target displacement](image)

**Figure 2.** Demand spectrum and target displacement

### 3. 3D Finite Element Model of the Structure

An engineer sketch of the bridge is depicted in Figure 3. The corresponding 3D structural mesh composed of beam elements is shown in Figure 4.

![3D model of bridge](image)

**Figure 3.** Sketch
As the seismic assessment focuses on piers P2, P3, P8 and P13, only these four piers are modelled explicitly with nonlinear beam elements, introducing concrete compression and traction limits as well as steel reinforcement. The other piers are introduced as boundary conditions, and the bridge’s deck and
foundations are introduced as linear beam elements with correct inertia and area.

Figure 5 summarizes the differences between the real bridge and the model. In particular, no interaction between soil and structure is considered in this analysis: the piles supporting the piers are considered to be fixed at a depth of B/2 in the bedrock.

4. Target Displacement Computation with Z_SOIL.PC

The four under discussion piers have to be examined in both longitudinal and transversal directions (with +/- signs) for unitary and modal force distributions, which means that a total of 8 capacity spectra for each of the four piers will be created. As for the demand spectrum, it is defined in Z_SOIL.PC through an input screen where peak horizontal ground acceleration, structure importance factor, ground type and damping can be introduced according to Eurocode 8 definition. Figure 7 shows the output given by Z_SOIL.PC corresponding to the analysis of pier 2, in the longitudinal direction (+ sign), with a modal force distribution, as defined in the Z_SOIL.PC input screen shown in Figure 6. A target displacement of 1.22 cm is obtained in this case.

Remark: pile foundation flexibility is neglected in this computation, which means that for the pier’s seismic assessment, the displacement at the base of the pier is considered to be equal to zero. This assumption is conservative if both the top and the bottom of the pier move in the same direction during the earthquake.
5. Capacity of Deformation

5.1 Definition

According to SIA CT 2018 §6.2 [2], the pier’s capacity of deformation is defined as:

\[ w_{rd} = \theta_{\text{max}} L_v \]  

(2)

Where \( L_v = H_{\text{piw}} \times 2 \) is the shear length for a fixed-end beam (we assume here that the pier is fixed in the deck) and the rotation capacity \( \theta_{\text{max}} = 3 \theta_y \) in the first approximation. \( \theta_y \) is the yielding chord rotation and can be expressed as:

\[ \theta_y = \phi_y \frac{L_v}{3} \approx 2.1 e_{sk} h_b \frac{L_v}{3} \]  

(3)

In Eq. (3), the nominal curvature of plastification \( \phi_y \) is approximated as a function of steel plastification strain \( e_{sk} \) and rectangular beam height \( h_b \).
5.2 Shear force verification

Deformation-based methods cannot be applied to structures subjected to fragile failure mechanisms, like shear failure (SIA CT 2018 §6.2.15 [2]). In order to circumvent this limitation, a reduced compliance factor $\alpha_{\text{eff}, \text{red}}$ can be computed as follows:

$$
\alpha_{\text{eff}, \text{red}} = \frac{w_{\text{rd,red}}}{w_d}
$$

(4)

Where $w_{\text{rd,red}}$ takes into account the maximal shear resistance of the pier’s section $V_{rd}$ (see SIA 262, § 4.3.3.4.3 [5]), when nominal shear resistance $V_d^* > V_{rd}$ (see Figure 8):

$$
w_{\text{rd,red}} = w_{rd} V_{rd} V_d^* = w_{rd} \frac{A_{zd} \cdot s \cdot z \cdot f_{zd} \cdot \cot \alpha}{(M_{zd,1}^* + M_{zd,2}^*) / H}
$$

(5)

6. Summary and Conclusions

Given $w_d$ (Figure 5) and $w_{rd}$ (Eq. (2)) we can compute the compliance factor $\alpha_{\text{eff}}$ or $\alpha_{\text{eff,red}}$ (Equations (1) or (4)) – for each pier, each direction and each load distribution – and compare it with reduction factors $\alpha_{\text{min}}$ and $\alpha_{\text{adm}}$. Table 1 summarizes these factors for the four piers.
### Table 1. Compliance factors

<table>
<thead>
<tr>
<th></th>
<th>Longitudinal direction: $\alpha_{eff} = \frac{w_{Rd}}{w_{d}}$</th>
<th>Transverse direction: $\alpha_{eff} = \frac{w_{Rd,r\delta}}{w_{d}}$</th>
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<td></td>
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<td>6.65</td>
</tr>
<tr>
<td>Pier 13</td>
<td>11.44</td>
<td>10.11</td>
</tr>
</tbody>
</table>

Compliance factor $\alpha_{eff} > \alpha_{adm} = 0.76$ for each and every pier, and therefore the seismic assessment of the motorway exit is fulfilled.

It has been shown on other structures designed in the 1970s that applying the replacement forces method today can lead to the decision to reinforce the bridge. This is due to the fact that norms tend to be more restrictive nowadays than they were in the past, and so verification acceleration is greater than design acceleration.

As nonlinear elasto-plastic behavior of structural elements isn’t explicitly taken into account in the replacement forces method (it is hidden in behavior factor $q$), there is a reserve that can be tapped using displacement-based methods, therefore often yielding satisfactory compliance factors: reinforcement is not necessary in this case.

### 7. References

Conceptual design with respect to vibration for the next generation Swedish synchrotron radiation facility, MAX IV

Ulf Ekdahl*
*Ekdahl GeoDesign AB, Lomma, Sweden

Keywords: Ground vibration, dynamic analysis, non-linear elastic material model, ZSOIL.

Abstract
A new generation synchrotron light source which produces high brilliance x-Ray beams will be built in Sweden. Vibration may degrade the electron beam centre of mass stability, hence dramatically reducing the effective brilliance. It is therefore necessary to carefully measure and survey the vibrations existing on the MAX IV site. Dynamic analysis with respect to interaction between ground and building must be considered and vibration generated by heavy goods vehicles on adjacent roads must be possible to predict. Based on the existing traditional geotechnical investigation and documentation an anticipated geological model has been developed. With this model as a basis and also use of earlier gained experience about the soil in the region, geotechnical properties for the materials have been suggested. This geo model will then be verified and updated during both the design and construction stage, following the observational method according to Eurocode 7. The axle loads from heavy goods vehicles have been simulated with use of a Falling Weight Deflectometer (FWD) and vibration propagation has been measured with both accelerometers and geophones. This study shows that vibrations from heavy goods vehicles can be simulated through a FWD source and that ZSOIL can predict the vibration propagation with good precision using the suggested geological model and geotechnical material properties. ZSOIL has since been used in a conceptual design of the planned facility. The conclusions from this study is that an alternative design with use of stabilized soil and vibro isolation consisting of soft barrier material can reduce the vertical vibration level (RMS 1sec) by 78% compared with the currently design proposal in the system documents.

1. Introduction
MAX IV is planned to be the next generation synchrotron radiation facility
in Sweden, see Figure 1. It will replace the existing laboratory in Lund consisting of the MAX I, II and III storage rings. The main sources at MAX IV are two storage rings (1.5 GeV and 3 GeV) with state-of-the-art low emittance for the production of soft and hard x-rays. Since the quality of the measurement results from the MAX IV ring is dependent on the precision of the synchrotron light, very strict requirement regarding the vibration levels of the magnets are specified. The strict requirement is especially put on the mean vertical vibration level which must be less than 26 nm during one second in the frequency span of 5–100 Hz. The mean vibration level is calculated as a RMS value (Root Mean Square). Vibrations with frequencies lower than 5 Hz may be adjusted by an active calibration system. In the interval between 0–5 Hz vibration levels up to 260 nm are therefore allowed. Frequencies higher than 100 Hz may be neglected and probably have very low amplitudes since they are easily damped out in the structure.

Figure 1: 3D view of MAX IV, high-tech facility (http://www.maxlab.lu.se).

Due to the stiffness of the magnet foundations the vibrations are assumed to be basically the same at the bottom and at the top positions of a magnet foundation. The evaluation points of the vibrations were picked at the floor where the magnet foundations are to be placed.
Main ground-borne vibration sources are road traffic. The MAX IV complex is situated between heavily trafficked roads, see Figure 1. In this paper ground vibration from heavy goods vehicles is simulated with the FWD source, routinely used in Sweden to measure road pavement characteristics (e.g., pavement stiffness). Other possibly disturbing vibration sources as wind turbines and blasting activities exists but are supposed to have less impact than the traffic load. However, those sources will be studied at a later stage.

The vibration propagation has been measured with accelerometers and with more than 20 geophones located between 20 and 90 meter from the vibration source. This article shows that the chosen calculation model in ZSOIL, using the proposed geo model, predicts the size of the vertical amplitude at different distances from the vibration source very well.

In this paper the result from a conceptual design with use of ZSOIL is reported using the validated calculation model from the FWD test. The discussed construction has sufficient safety level to the specified vibration criteria.

2. Anticipated geological model and geotechnical parameters

On the basis of existing documentation an anticipated geological model has been compiled for the area just northeast of Lund, where MAX IV is planned to be constructed, see Figure 2.

![Anticipated geological model, MAX IV](image)

Figure 2: Anticipated geological model for the MAX IV complex, Sivhed [1].
2.1 The Soil

Two different tills are found in the area. These are probably separated by inter till sorted sediments and also occasionally sorted sediments below the lower till. The lower till rests directly on the bedrock (shale and mudstone) and is named Northeast Till. It is massive, homogenous and might be argillaceous. It has a thickness of up to 10 m or more. The upper till is called Low Baltic Clay Till. It has a thickness of probably less than 4 m. Between those two tills inter till sorted sediments might occur. They consist of sand, gravel, clay and silt. In order to identify the two types of till and possibly also intermediate inter till sediments, new continuous sampling with core drilling will be performed and supplemented by seismic survey.

The till is very dense. The stiffness of the soil is not a material constant as it depends on stress state and strain level. Critical soil parameters are void ratio, stress level, strain level and matric suction. The stiffness of the soil is preferable calculated using a non-linear stress-strain model, SwePave, which has been implemented in the computer code ZSOIL. The soil model is described by Ekdahl [2].

The assumed geotechnical parameters for the low Baltic Till Clay are shown in Table 1. The pore pressure is calculated assuming a ground water level situated 1 meter below the ground surface. Below the ground water surface the low Baltic Clay Till is supposed to be fully saturated.

For the Northeast Till the assumed geotechnical parameters are shown in Table 2. This till is unsaturated and the saturation degree is supposed to be equal to 76%. The suction in this till could be put equal to 500 kPa.

2.2 Soil Depth

On the basis of data from previous geotechnical investigations it is not possible to determine the soil thickness. The bedrock surface will be identified by complementary soil-rock drillings and by geophysical methods.

2.3 The bedrock

The bedrock surface is composed of Silurian shale and mudstone with varying degrees of hardness. Almost vertical dolerite dykes penetrate the Silurian rocks, close to the dolerite dykes argillites could be found. The bedrock surface is probably the Silurian Colonus shale. It is a shale or mudstone and close to the dolerite dykes transformed to an argillite. The Silurian rocks are rich in mica and carbonate and are more or less horizontally bedded. In the Silurian shale, hard, carbonate cemented beds occur. This means that there is a variation in density and also in seismic velocity. The bedrock is penetrated by almost vertical dolerite dykes running in a north-west–south-east to west-northwest–east-southeast direction. They vary in broad from some meters to tens of meters. Dolerites have higher density and seismic velocity than the surrounding shale.
The dykes run from the still active Hardeberga quarry (4.5 km south-east of the area) into or immediately south of the area. There could be a risk (probably very small) that the dykes act as pathways for energy, generated by bursting in the Hardeberga quarry. The bedrock surface might be flat, but can also exhibit a varying topography with higher areas, erosional remnants, in connection to for instance dolerite dykes. An indication of uneven bedrock surface is the fact that the MAX IV centre is located at a minor hill. This fact could reflect a variation in the bedrock surface. The bedrock surface will be identified by soil-rock drillings and geophysical methods. Probable geotechnical parameters for the Shale/Mudstone are shown in Table 3.

Table 1: Anticipated geotechnical parameters for the Low Baltic Clay Till.

<table>
<thead>
<tr>
<th>Description</th>
<th>Symbol</th>
<th>Unit</th>
<th>Mean value</th>
</tr>
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<td>Unit weight</td>
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<td>Deviatoric stress</td>
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<td>kPa</td>
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The dykes run from the still active Hardeberga quarry (4.5 km south-east of the area) into or immediately south of the area. There could be a risk (probably very small) that the dykes act as pathways for energy, generated by bursting in the Hardeberga quarry. The bedrock surface might be flat, but can also exhibit a varying topography with higher areas, erosional remnants, in connection to for instance dolerite dykes. An indication of uneven bedrock surface is the fact that the MAX IV centre is located at a minor hill. This fact could reflect a variation in the bedrock surface. The bedrock surface will be identified by soil-rock drillings and geophysical methods. Probable geotechnical parameters for the Shale/Mudstone are shown in Table 3.
Table 2: Anticipated geotechnical parameters for the Northeast Till.

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<th>Description</th>
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Table 3: Anticipated geotechnical parameters for the Shale/Mudstone

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<td></td>
<td>0.2</td>
</tr>
<tr>
<td>Young's modulus as a secant modulus</td>
<td>E</td>
<td>MPa</td>
<td>8300</td>
</tr>
<tr>
<td>Rock mass modulus</td>
<td>E</td>
<td>MPa</td>
<td></td>
</tr>
<tr>
<td>Shear wave velocity</td>
<td>v_s</td>
<td>m/s</td>
<td>1200</td>
</tr>
</tbody>
</table>
2.4 Stabilized Clay Till

An appropriate soil improvement method which could be used for the MAX IV project is soil stabilization. If the low Baltic Clay Till is stabilized with cement the geotechnical parameters suggested in Table 4 could be used.

Table 4: Anticipated geotechnical parameters for the low Baltic Clay Till stabilized with cement.

<table>
<thead>
<tr>
<th>Description</th>
<th>Symbol</th>
<th>Unit</th>
<th>Mean value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight</td>
<td>( \gamma )</td>
<td>kN/m(^3)</td>
<td>22</td>
</tr>
<tr>
<td>Strength parameters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>( f_c )</td>
<td>MPa</td>
<td>20</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>( f_t )</td>
<td>MPa</td>
<td>0.37</td>
</tr>
<tr>
<td>Dilatancy angle</td>
<td>( \psi )</td>
<td>degrees</td>
<td>10</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>( v )</td>
<td></td>
<td>0.2</td>
</tr>
<tr>
<td>Coefficient of earth pressure at rest</td>
<td>( K_0 )</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>( E )</td>
<td>MPa</td>
<td>4000</td>
</tr>
<tr>
<td>Shear wave velocity</td>
<td>( v_s )</td>
<td>m/s</td>
<td>879</td>
</tr>
</tbody>
</table>

2.5 InSitu condition

The variation of the seismic modulus and shear wave velocity with depth was estimated using the non-linear elastic material model, SwePave, as shown in Figure 3. Also shown are preliminary results from seismic measurements using the MASW method.

The saturation ratio for the initial state, which was calculated using Van Genuchten equation in ZSOIL, is shown in Figure 4. The calculated saturation ratio in the Northeast Till is 76%. In the low Baltic Clay Till the saturation ratio is 100%, except for the upper first meter and the bedrock.

In the Northeast Till, the calculated pore pressure for the initial state is -500 kPa (suction), which is the result of heavy preconsolidation. The pore pressure in the low Baltic Clay Till and the bedrock is hydrostatic corresponding with a ground water level situated 1 meter below the ground surface.

ZSOIL calculated elasticity modulus correspond with the stiffness distribution shown in Figure 3.
Figure 3: Calculated seismic modulus and shear wave velocity compared with measured shear wave velocity in the soil.

Figure 4: Generated saturation ratio for the initial state.
2.6 Rayleigh damping

The frequency range where the damping ratios are valid is put equal to 5–25 Hz. The damping ratio at small strain for concrete, soil, bedrock and stabilized material is set equal to 2%. The damping ratio for pavement material, see Figure 6, is set equal to 5% and the damping ratio for the barrier material, see Figure 16, is set equal to 10%.

3. Simulating heavy goods vehicles with FWD

Ground-borne vibration is caused by the interaction between the dynamic forces produced by tires of highway vehicles and pavement surface irregularities. Typical principal frequencies are in the range 8–16 Hz. On the adjacent highway E22 the number and the magnitude of single axle loads have been measured. In Figure 5 the single axle load measured during one week in October 2008 is reported. The maximum registered single axle load was 13.94 ton. The design basis for the wheel load is therefore chosen to 70 kN. The device selected to achieve this impact load is the Falling Weight Deflectometer (FWD). The device consist of a trailer-mounted impact device with associated deflection and load measuring transducers and a system processor and microcomputer for controlling the hydraulic lifting gear and recording measurements. The system operates by allowing a guided weight to fall from a predetermined height to generate a transient force on a plate in contact with the road surface. The size of the impact can be varied by altering the distance through which the moving mass falls or by changing the mass itself.

Figure 5: Measured heavy goods vehicle axle load on highway E22.
The FWD test was performed on the main road Odarslövsvägen in a position where the distance to the planned facility is the shortest, about 80 meters. Measurement of vibration was carried out jointly by staff from the Max-lab, Lund University, PEAB and the consultant company Tyréns. Max-lab performed measurements with accelerometers at distances 20, 40, 60 and 80 meter from the FWD source. PEAB performed measurements with one accelerometer and one geophone at a distance of 20 meters from the FWD test. Tyréns performed measurements with 24 geophones spaced 3 meter at distances 20-89 meter from the FWD source. The accelerometers record a signal which is proportional to acceleration while the geophones provide an output signal which is proportional to the vibration velocity. Measurement data from the accelerometers were integrated once to obtain the vibration velocity and twice to obtain displacement, while data from the geophones were integrated once to obtain displacement.

3.1 Dynamic analysis of the simulated FWD test in ZSOIL and comparison with the measured displacement values

The FWD test has been simulated in a two-dimensional axially symmetric model in ZSOIL for two phase fully or partially saturated material. This is done because it is essential to describe the stress state of the initial state as accurately as possible so that the stiffness can be calculated with the non-linear elastic material model. With this generated initial stress state a dynamic analysis has been done in ZSOIL using both the non-linear and the conventional linear elastic material model.

The assumed material properties for the pavement construction are shown in Figure 6. The measured peak load in the FWD test is 72 kN which correspond to a maximum contact pressure of 1018 kPa. The plate diameter in the FWD test is 300 mm. The measured load pulse is used as input when the dynamic analysis is performed in ZSOIL.

Measured and calculated dynamic load-displacement curve are shown in Figure 7 and the time history of center displacement is shown in Figure 8. The result of the non-linear elastic material model are in surprisingly good agreement with the measured displacements. In Figure 9 the calculated E-modulus in the soil when the peak force reached 72 kN is shown.
Figure 6: Dynamic simulation of the FWD test with ZSOIL.

Figure 7: Measured and predicted Load-Displacement curve at the FWD test.
According to the vibration criteria the vibrations should be presented as displacements on the accelerator foundation and should be averaged over 1 s (RMS 1 sec). The displacements are not allowed to be more than 260 nm in the frequency range 0–5 Hz and 26 nm in the frequency range 5–100 Hz.

Figure 8: Dynamic simulation of the FWD test with ZSOIL.

Figure 9: ZSOIL calculated Young’s modulus at peak FWD load, 72 kN, for the non-linear elastic material model.
In order to compare the measured and the predicted displacement values with the vibration criteria the displacements are presented as RMS 1 sec values. The measured values at the sensors and the predictions from ZSOIL dynamic analysis are shown in Figure 10. The maximum displacement amplitude of the absolute value is also shown in the same figure. There is excellent agreement between predicted and measured displacement. This means that the adopted geo model and used calculation model in ZSOIL can with sufficient accuracy describe the vibration propagation in the soil for the simulated heavy goods vehicles axle load with the FWD test.

Figure 11 shows measured and predicted integrated Fourier power spectra for measured displacement at a distance of 20 meters from the FWD source. No significant difference exists between the dynamic analysis with the linear elastic and the nonlinear elastic material model. There is good agreement with the measured spectra.
The comparison between measured and predicted displacements in both time and frequency domain at a distance of 80 meter from the road is reported in Figure 12. The planned building will be located in this area.

### 3.2 Comparison with measurements done by NGI [3]

NGI has been commissioned by the University of Lund to perform measurements of vibrations on the ground where MAX IV will be built. NGI conducted continuously, for one week, long-term logging of vibration at four points on the ground in September / October, 2009. In addition, vibrations were measured at different distances from the local road when a truck ran over a 1" plank to simulate the vibrations from heavy goods vehicles driving on the road. In Figure 13 the red curve show measured (50-percentile) integrated power spectral density of vertical ground motion versus cut frequency by NGI. The ZSOIL predicted Integrated Fourier Power Spectra is shown with the dotted light blue curve. The result is reported for a position situated 80 meter from highway E22 at a location where the building is planned to be built. The dotted green curve shows measurements done by Tyrens for Max-lab. Traffic load is simulated by a FWD source. Comparisons with measurements from other similar facilities are also shown in Figure 13. The comparison shows that the MAX IV site at Lund is at the upper part of what is measured in other locations. The comparison in the figure is based on the 50-percentile which can be considered as an average over the entire measuring period.
The vibration of a loaded truck driving on the small road crossing MAX IV site were registered, see Figure 14. The truck crossed one inch thick plank. The vibration data were taken 30 m from the plank. Blue curve show the measured power spectra. Predicted Fourier Power from calculated ZSOIL displacement data using linear elasticity is shown with the red dotted curve. The light blue dotted curve represents the result from non-linear elasticity using the SwePave model. The distance from the FWD source is 20 meter. The corresponding measured power spectra are shown with the green dotted curve. The predicted and measured power spectra for both the FWD source and the truck crossing a plank are very similar.
4. Conceptual design with respect to vibration

Since the presumed geo model and the calculation model used in ZSOIL have been validated with results from the FWD test and measurement of vibration propagation it was possible to move forward with a conceptual design and compare the calculated displacement RMS values with the vibration criteria.

The design according to the main contract documents has been studied, but also an alternative design solution has been analyzed. The design according to the system documents is shown in Figure 15. The concrete slab is 700 mm thick and conventionally reinforced.

![Figure 15: The design solution according to the system documents.](image1)

![Figure 16: The alternative design solution with stabilized soil and vibro isolation to bedrock.](image2)
In the alternative design, see Figure 16, the low Baltic Clay Till is stabilized with cement and thereafter outlaid and compacted in 0.3 meter thick layers, to an assumed thickness of 3.4 meter. The characteristic compressive strength should be 2 MPa. The upper two layers, thickness 0.6 meter, consist of crushed unbound granular base material which is stabilized with cement to a characteristic compressive strength of 4 MPa. Vibro isolation is constructed with the diaphragm wall technique. The wall is 0.5 meter wide and is filled with bentonite-cement slurry which gives undrained shear strength of 10 kPa and a small strain elasticity modulus of about 20 MPa. The concrete slab, 0.2 meter thick, is cast with steel fibre reinforced self compacted concrete. The concrete strength is C50/C60 and 40 kg/m³ steel fibre is used.

4.1 Prediction of vibration due to heavy goods vehicles with ZSOIL

The evaluation point for the vibrations was picked at the floor where the magnet foundations are to be placed. The vibration source is heavy goods vehicles with a single axle load of 14 ton. This is simulated as a FWD source with a max peak load of 70 kN. The FWD source, simulating heavy goods vehicle axle load is situated 80 meter from the building. The calculated vibrations at the evaluation point for different design solutions are reported in Figure 17.

![Vibration 80 meter from impact from FWD with peak force 70 kN](image)

Figure 17: ZSOIL calculated vibrations at the evaluation point situated 80 meter from the FWD source with a peak force of 70 kN.
Summary of calculated Eigen Frequency and mean vibration level (RMS 1 sec) for different design proposals are shown in Table 5. The design proposal according to the system document reduce the mean vibration amplitude level with only 13 % whereas the alternative design solution with stabilized low Baltic Clay Till and vibro isolation with soft barrier material to rock level reduce the mean vibration amplitude level with 81 %.

Table 5: Summary of calculated Eigen Frequency and mean vibration amplitude level with ZSOIL.

<table>
<thead>
<tr>
<th>Design proposal</th>
<th>In situ condition without structure</th>
<th>System Document</th>
<th>Alternative design without vibro isolation</th>
<th>Alternative design with vibro isolation to Northeast Till</th>
<th>Alternative design with vibro isolation to Rock Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eigen Frequency Mode 1 (Hz)</td>
<td>7.7</td>
<td>7.7</td>
<td>6.7</td>
<td>7.4</td>
<td>6.4</td>
</tr>
<tr>
<td>Mean Vibration Level (RMS 1 sec)</td>
<td>12.0</td>
<td>10.5</td>
<td>5.1</td>
<td>4.1</td>
<td>2.3</td>
</tr>
<tr>
<td>Reduction of vibration compared to the in situ condition (%)</td>
<td>-</td>
<td>13</td>
<td>58</td>
<td>66</td>
<td>81</td>
</tr>
</tbody>
</table>

4.2 Isolation efficiency of barrier material

According to Massarsch [4] the isolation efficiency of barrier material could be expressed with the energy transmission coefficient, $E_n$, which is defined as:

$$E_n = \frac{Z_1}{Z_1 + Z_2}$$

where $Z_1$ and $Z_2$ are the impedances of the soil and of the barrier, respectively.

The impedance $Z$ is defined as

$$Z = c\rho$$

where $c$ is the wave propagation velocity and $\rho$ is material density.
Assume for the Northeast Till that \( c = 500 \text{ m/s} \) and \( \rho = 2.2 \text{ t/m}^3 \). This means that \( Z_1 = 1100 \). Assume for the Barrier Material that \( c = 100 \text{ m/s} \) and \( \rho = 1.4 \text{ t/m}^3 \). This means that \( Z_2 = 140 \). The energy transmission coefficient is then calculated as \( E_n = 0.40 \). The relative vibration, \( \text{RMS}_r \), quotient (isolation effect) is calculated from ZSOIL displacement result with

\[
\text{RMS}_r = \frac{\text{RMS}_\text{after}}{\text{RMS}_\text{before}}
\]

this gives

\[
\text{RMS}_r = \frac{2.33}{5.13} = 0.45
\]

The isolation efficiency calculated from ZSOIL result expressed as the relative vibration quotient \( \text{RMS}_r \) gives slightly more conservative isolation efficiency then the formula for the energy transmission coefficient proposed by Massarsch.

5. Concluding remarks

Based on the conceptual design presented above, the following conclusions can be made relating to vibration from heavy goods vehicles:

- The soil model developed for the anticipated geological conditions and geotechnical parameters appears to be a good estimate of actual conditions since the calculated displacements are in good agreement with the measured displacements.
- The non-linear elastic material model SwePave, implemented in ZSOIL, gives more accurate prediction of the displacements compared to the usual linear elastic material model.
- For the dynamic analysis it is important to describe the stiffness variation in the soil as accurately as possible. Since the size of the stiffness is stress dependent a two phase model with full or partial saturated material should be used. The initial stress condition should be generated with the greatest caution.
- Vibrations from heavy goods vehicles can be simulated with the Falling Weight Deflectometer (FWD). The peak load in the FWD test is to be as large as the actually measured dynamic force from heavy goods vehicles on the adjacent highway E22.
- Soil improvement with the stabilizing technique in combination with diaphragm walls and soft barrier material works as an effective vibro isolation methodology.

The film, which can be accessed at the following link, is a brief summary of the investigation carried out:

http://youtu.be/5goWRPXIJRg
6. Acknowledgments

This conceptual design with respect to vibration for the MAX IV project hasn’t been possible to carry out without engaged contribution from personnel at Max-lab, Structural Mechanics at Lund University, Peab, Tyrens and SGU. The following persons are especially acknowledged; Brian Jensen, Mats Svensson, Nils Rydén and Ulf Sivhed.

I also wish to thank Th. Zimmermann and A. Truty at Zace Services LTD. for assistance with the dynamic analysis in ZSOIL.

7. References


Keywords: pile test, loading frame, load-displacement curve prediction, load and displacement control

Abstract
Finite element analyses with Z_SOIL.PC [1] are used in order to predict the behaviour of a pile test. The loading frame is introduced explicitly into a 3D model, while a simpler axisymmetric analysis is carried out simultaneously in order to compare force-displacement curves. These curves are then compared to the in situ measured load-displacement curve, and shown to be in good agreement.

1. Introduction

A pile test was performed in order to optimize the pile foundation of a 350 m long viaduct in construction for the H144 road in Switzerland.

Plane strain finite element analyses are unable to represent correctly the behaviour of a single pile loaded at its top. Usually, a force-driven axisymmetric analysis is recommended, but this model doesn’t take into account the loading frame used to transmit the force at the pile’s head, as well as the retaining piles. In the case discussed in this paper, the loading frame is composed of a jack, linked to four retaining piles – located at a certain distance of the test pile – through two steel beams (see Figure 1). The test pile is a 23 m deep cast-in situ “Vibrex” pile: a steel tube is vibrated into the ground until it reaches the required depth, then steel reinforcement is placed and the pile is filled with concrete and pulled out. In general, such a technique increases the pile’s frictional capacity.

All finite element analyses have been conducted prior to pile test execution. The goals of these calculations were:
- to estimate the bearing capacity of the pile in order to design the jack,
- to predict the load-displacement curve, and
- to show whether interaction between retaining and test piles was
significant, therefore meaning that the use of a 3D model can be of interest, compared to axisymmetric model where retaining piles cannot be taken into account.

2. Pile Test Setup and Geology

The pile test setup is illustrated in Figure 1. It is carried out in the valley of the Rhone river; the geology is mainly composed of deposits on a rather high depth: flood deposits, fluvial deposits and lacustrine. Soil parameters estimated by the geotechnician are given in Table 1. Three values for the Young modulus are given for each soil deposit, meaning that three finite element analyses (optimistic, reasonable, and pessimistic) will be carried out. Soils are modelled using the simple Mohr-Coulomb elastic-perfectly plastic constitutive law, as monotonic loading is involved in this problem. The plastic deformation follows a non-associated flow rule.

![Figure 1. Pile test view](image)

<table>
<thead>
<tr>
<th>Depth</th>
<th>E_pess</th>
<th>E_realistic</th>
<th>E_opt</th>
<th>γ</th>
<th>c</th>
<th>φ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flood deposits</td>
<td>0 → -2.35 m</td>
<td>2</td>
<td>3.5</td>
<td>5</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>Fluvial deposits</td>
<td>-2.35 → -17.15 m</td>
<td>25</td>
<td>50</td>
<td>75</td>
<td>21</td>
<td>1</td>
</tr>
<tr>
<td>Lacustrine</td>
<td>-17.15 m →</td>
<td>25</td>
<td>37.5</td>
<td>50</td>
<td>19</td>
<td>1</td>
</tr>
<tr>
<td>Pile</td>
<td>-</td>
<td>20'000</td>
<td>-</td>
<td>25</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
3. **2D Axisymmetric Analysis**

The 2D axisymmetric model is depicted in Figure 2. A steady state coupled analysis is carried on, with a water table located 3 meters under the surface. A force is applied at the top of the pile and increased regularly. As this is a driven pile, no interface elements are introduced in the model at the pile-soil interface, but a rather fine mesh is created at this interface in order to follow accurately the development of plastified zones.

Figure 3 illustrates the load-displacement curves at the top of the pile for 2D axisymmetric analyses of the three sets of soil parameters (realistic, optimistic, and pessimistic). It can be seen that the three responses are smooth and none of them gives a clear failure load before the applied load reaches a value of 10’000 kN.

Figure 4 depicts the stress level in the soil for $F = 6’000$ kN which is the load at which plastification occurs along the whole pile, and the corresponding vertical displacement field is shown in Figure 5.
Figure 3. Load-displacement curves for the 2D axisymmetric model

Figure 4. Stress level for F = 6'000 kN

140
Figure 5. Vertical displacement field for $F = 6'000$ kN

Figure 6. 3D mesh (1/4 due to symmetry)
4. **3D Analysis**

A 3D model is considered in order to check the interaction between the test pile and the retaining piles.

Due to symmetry, only a quarter of the loading frame is modelled. Figures 6 and 7 illustrate the 3D model including a quarter of the test pile, a 28 m deep retaining pile, the loading frame with the HEB 1000 steel beams and the jack.

![3D model: pile head and loading frame](image)

In order to reproduce the loading with the jack, the following “trick” is used: a thermal analysis is carried out first, increasing the temperature inside of the jack. Then, the corresponding mechanical analysis is driven, and the expansion of the jack is applied to the pile and drives it into the soil. The force applied is obtained by integrating vertical stresses at the top of the pile.

Figure 8 shows the evolution of stress level for increasing loads. Although plastification is shown to develop between the test and retaining piles for a high load level, the load-displacement curve obtained for the 3D model with realistic Young moduli almost coincides with the one obtained with the 2D axisymmetric analysis, as Figure 9 shows.
Figure 8. Stress level and deformed mesh for $F = 0 \text{ kN}$ up to 8'000 kN

Figure 9. Comparison of load-displacement curves for 2D axisymmetric and 3D models
5. **Influence on convergence tolerance**

At the beginning of our study, we didn’t have such a good correlation – as shown in Figure 9 – between 2D and 3D load-displacement curves. The problem was that unlike the axisymmetric case – which is a force-driven problem, applying the force with the jack in the 3D case makes it more like a displacement-driven problem: the jack expands and forces the pile into the ground. In the latter case, particular attention should be given to the selection of tolerance, as Figure 10 shows: 1% tolerance yields a totally different load-displacement response (green curve), while with a 100 times smaller tolerance, the “correct” answer is retrieved (red curve).

![Figure 10. Influence of tolerance on the response of a displacement-driven problem](image)

6. **Conclusions**

In situ measures are compared to the three axisymmetric load-displacement curves in Figure 11. A very good agreement is found between the realistic prediction and the actual in situ load-displacement curve.

The a priori finite element analysis also helped in the choice of the jack, showing that it had to be capable to apply a force of up to 6'000 kN to the test
Neither the 2D axisymmetric model nor the 3D model are shown to predict a clear failure load before the applied load reaches 10’000 kN. During the pile test, the jack broke before the pile for a load just over 6’000 kN...

Figure 11. In situ vs. FE model comparison

7. Acknowledgements

The pile test was financed by the “Service des Routes du Canton de Vaud”. The main contractor was Marti SA, the subcontractor for the pile test was Solexperts SA and the geotechnician was De Cérenville Géotechnique SA, and the civil engineering office was INGPHI SA.

8. References

3D Numerical Analysis of the Toppling of A Nearly Completed 13-Storey Building in Shanghai

Yin Ji*, Zhang Yaodong**
*Shanghai Geotechnical Investigations & Design Institute, Co., LTD
Shanghai, PR. China
** GeoEng Consultants (S) Pte Ltd
Singapore

Keywords:HSS model, 3D FEM, toppling of a building, surcharge load, bearing capacity of PHC pile, soil structure interaction

Abstract
3D geotechnical software zsoil.pc v2009 is employed to analyze the toppling of a building in Shanghai. The underground garage excavation on the south side of the building together with two stages of surcharge on the north side of the building, are considered in the FEM analysis. The hardening soil model with small strain stiffness (HSS) is used to simulate the stress-strain-strength relationship of the soil. The analysis results reveal that the additional horizontal force acting on foundation of the building induced by surcharge is about 3000T. The building moves towards the excavation laterally as much as about 13cm as a result of the additional horizontal force. There are critical piles of the building on both excavation and backfill sides which may fail first and trigger the progressive failure of other piles and eventually the toppling of the building.

1. Introduction
On June 27, 2009 at approximately 5:30 a.m. in Shanghai China, a nearly completed 13-storey building of an apartment complex toppled over as shown in Fig. 1. It can be observed that this typical overturning failure was triggered by the foundation failure. The super structure after collapse still amazingly retained its overall integrity while the foundation piles were broken into parts. This paper tries to investigate the possible collapse mechanism of the building through 3D FEM analysis based on available information. A typical section of the site, including the information about the building, foundation, soil layer, soil surcharge, excavation and river, is shown in Fig. 2.
2. Project overview

2.1 General information about the building and foundation

The 13 storey collapsed building was located on the north part of the site, adjacent to a river to its north. The building was supported by combined pre-stressed high strength concrete (PHC) pipe piles and strip foundation. There are a total of 114 numbers of PHC AB pipe piles toed into the silt soil layer \( \Omega \) -1-2. The pipe piles are 33m long, 400mm outer diameter and 80mm thickness. The concrete grade of the piles is C80. The design axial capacity of each pile is 1300kN. The layout of strip foundation and piles is shown in Fig. 3.

Fig. 1 The collapsed building (from www.eastday.com)

Fig. 2 Schematic diagram of general section map
2.2 Garage Excavation and Backfill

To the south of the building, an excavation of an underground garage was in progress before the accident. The excavation is about 4.9m deep from ground level and about 2~4m away from the building. The excavation was supported by a composite cement mixing pile and soil nail wall. The soil nails, anchored into the ground below the strip foundation of the building, were about 9m to 12m long at 1m spacing.

Based on the information collected, the soils from the garage excavation were backfilled on the north side of the collapsed building at two stages as shown in Fig. 4. The backfill soil at the first stage was about 3 to 4m high, 20 m away from the building and 10m away from the river flood protection wall. While the backfill soil at second stage was much higher to about 10m, abutting the first stage backfill soil on one side and in very close proximity to the collapsed building on the other side.

![Fig. 3 Layout of strip foundation and pipe piles of the building](image1)

![Fig. 4 Surcharge adjacent to the collapsed building](image2)
2.3 Ground conditions

The site is located in the shore of East China Sea and southeast front fridge of Yangtze River estuary. The ground generally is nearly flat. The geological profile of the site belongs to coastal plain, one of the four major geomorphic units in Shanghai region. The top 60.3m of ground can be divided into 7 layers based on their origins. The soil layers from ground surface to 25.9m ~29.8m deep were formed during the Holocene Epoch (Q4) while soil layers below were originated in the Pleistocene Epoch (Q3). A typical p-s curve obtained from static cone penetration test of the site is shown in Fig. 5.

![Fig. 5 Typical p-s curve of the site](image_url)

![Fig. 6 Relative stiffness characteristics of soil layer](image_url)
The CPT curve shows the soil layer 5-1 from 13m to 22m deep is stiffer than the upper soil. The stiffness difference of the two soil layers could be huge. If judging from CPT test results, the stiffness of lower soil layer 5-1 is about 8 (2.5/0.35Mpa) times larger than the that of the upper soft soil. The large stiffness difference will cause large shear force and moment of piles near the boundary as will be illustrated in Section 4. The pile-pile welded joint (weak link) just situates near the boundary as shown in Figure 6.

3. FEM model and parameters

3.1 Selection of FEM software

In this paper, 3D FEM analysis was carried out using the commercial 3D geotechnical software Zsoil.pc v2009, which was developed by Zace Services Ltd in collaboration with the Swiss Federal Institute of Technology in Lausanne. Over 1500 users all over the world have been using this powerful and versatile program with success to simulate almost all kinds of practical geotechnical problems encountered in design, consultancy and research. It offers a unified approach to numerical modelling of soil, rock and structure mechanics, above and underground structures, excavation and tunnelling, soil-structure interaction, stability, consolidation and underground flow etc. Furthermore, this program offers a number of built-in advanced constitutive soil models, supported by the profound theoretical foundation from Swiss Federal Institute. Especially, the HSS model adopted in this analysis is one of the advanced built-in soil models. The small strain stiffness of natural soil, which was very often neglected in most of numerical modelling, plays a crucial role in excavation construction and soil-structure interaction problems.

3.2 Modelling of soils

The HSS model was developed from the standard hardening Soil models by incorporating the nonlinear elasticity of small strain stiffness with hysteretic effect (Benz, 2006). The standard HS model was originally proposed by Vermeer (1978) and then enhanced by Schanz (1998) and Schanz et al. (1999). In p-q space, the HS model is formulated in the framework of multi-mechanism elasto-plasticity and consists of a hyperbolic shear yield surface for deviatoric strain hardening and an elliptic type cap yield surface for volumetric strain hardening. The yield surfaces of HS model in its principle stress space is shown in Figure 7.
The top 30m thick ground comprises of muddy and clayey soil, of which the permeability is as low as $10^{-7}$cm/s. Due to its low permeability, the dissipation of excess pore pressure induced during the construction is limited. Therefore, the undrained analysis was carried out for the study. The soil strength parameters from consolidated undrained shear tests (direct shear test) are adopted for the analysis, as shown in Table 1.

Table 1. Soil parameters for HSS model

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>$\gamma$ /kN/m$^3$</th>
<th>$w$ /%</th>
<th>$e$</th>
<th>$c$ /kPa</th>
<th>$\phi$ /Deg.</th>
<th>$E_s$ /kPa</th>
<th>$E_{50}$ /kPa</th>
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3.3 Modelling of Structures

The beam element with consideration of shear deformation is chosen for the deep strip foundation. The main structures of the building are also included in the model. The outer walls, shear walls and floors are simulated using the one layer shell elements. Composite section of layered and nonlinear material models are adopted for the beam and shell elements in which the compression and tensile strength and stiffness can be specified so that more realistic moment vs. axial force interaction and plastic hinges for beam and shell element could be modelled.

The PHC pipe piles are modelled using the embedded pile elements. The pile element consists of a beam element with shear friction interface and pile toe interface as shown in Figure 8. The shear interface allows to model relative movement between the pile and surrounding soil. The interface behaviour is governed by the standard ideal elasto-plastic Coulomb's strength criteria. Pile toe interface is also an ideal elasto-plastic normal spring which yields when...
specified limiting end bearing capacity is reached. In the analysis, the limiting shear resistance and end bearing capacity of pile were set and derived based on the values provided in the soil investigation report.

The pile head is assumed to be rigidly connected to the strip foundation to allow transfer of shear force, axial force and bending moment between the piles and strip foundation.

### 3.4 FEM model and construction sequence

The size of the 3D numerical model is 160m long, 96m wide and 45m high, which comprises of 35919, 1618 and 8839 numbers of 8-node brick continuum elements, beam elements and shell elements respectively. The boundary conditions of the model are that the side boundaries are laterally restrained while the base is restrained both laterally and vertically. The whole 3D FEM mesh is shown in Figure 9 and the structural elements within the FEM model are isolated in Figure 10.
Based on the actual construction progress on site, the construction stages in the FEM analysis with reasonable simplification are as follows,

Stage 0: Initial equilibrium state, corresponding to time=0;
Stage 1: Activate piles, foundations and super structures, corresponding to time=0 ~5;
Stage 2: Activate the first stage of surcharge, corresponding to time=5 ~10;
Stage 3: Activate retaining wall and excavate the underground garage, corresponding to time=6 ~12;
Stage 4: Activate the second stage of surcharge, corresponding to time=10 ~25.

To model the actual construction stage, Stage 4 starts when stage 3 is still unloading through proper control of EXF and LTF(existence and load-time functions).

### 4. Analysis Results

#### 4.1 Additional lateral force induced by backfill surcharge

The maximum additional lateral stress induced by the backfill surcharge, which is the difference of lateral stress after and before backfill, is about 90kPa, occurring at top 1~2m of ground, as shown in Fig. 11. The additional stress is reduced to about 30kPa at the top of soil layer 5-1. If assuming linear distribution of the additional lateral stress from top of ground to the top of soil layer 5-1, the average additional lateral force per meter run is about (90kPa+30kPa)/2×11.0m=660kN/m. Considering the length of the building of about 45m, therefore the total additional lateral force on the foundation of the building is estimated as 660kN/m×45m=29700kN, about 3000T.
4.2 Movement of the building

After completion of the second stage of backfill, the lateral movements on the west and south sides of the building are about 13cm and 5cm respectively, as shown in Fig. 12. The lateral movement of the building on the west side is about 8cm larger than the east side. This could be due to the fact that about 1/4~1/3 of the excavation on the east side was not completed while the excavation was reached final formation level on its western zone.

4.3 Displacement and forces of the piles

The displacement pattern and magnitude of the foundation piles under the additional lateral force are presented in Fig. 13. It is observed that lateral displacement of piles mainly occurs at top one third of pile length and becomes insignificant below. In terms of magnitude, the lateral movement reaches the maximum of about 12cm at the pile head towards the excavation, and gradually reduces along the pile to about 5cm at the top of the soil layer 15-1, implying the strong restrain from the underlying stiff soil layer.
Fig. 14(a) presents the bending moment diagram of the piles. On the backfill side, the maximum bending moment of about 131kN.m occurs on the middle pile (Pile B in Fig. 13). On the excavation side, the maximum bending moment of about 140kN.m takes place in the south-western corner pile (Pile A in Fig. 13). Significant bending moment exists at the pile head and the interface zone of soft and stiff soil layers. It is interesting to observe that the bending moments at the pile head and interface zone of soft and hard soil layers are of the same signs on the backfill side but of opposite signs on the excavation side.

On the backfill side, for the Pile B with the maximum bending moment at the pile head, its axial force at pile head is about 960kN. While on the excavation side, the axial force at pile head for the Pile A is about 1200kN. The axial force distribution along piles is presented in Figure 14 (b).
4.4 Study on pile behaviour

From the above analysis results, the most critical piles are the middle pile on the backfill side and south-western corner pile on the excavation side. For the middle pile adjacent to the backfill, it experiences the combined axial force and bending moment of 960kN and 131kN.m respectively. For the south-western pile next to the excavation, it takes the combined axial force and bending moment of 1200kN and 140kN.m, respectively. Based on the ultimate structural capacity of M-N interaction diagram from the pile specification, the critical piles are very close to the structural capacity. Any small incremental load may trigger the failure of the pile foundations. It is likely that these critical piles failed first which caused the progressive failure of other piles and eventually the toppling of the building.

5. Conclusions

The geotechnical program zsoil.pc v2009 was employed to investigate the likely failure mechanisms of a collapsed residential building. Based on the results of the analysis, the following conclusions may be drawn.

The total additional lateral force acting on the foundation of the building is estimated at about 3000T.

Under the unfavourable situation of excavation on one side and dumping of excavated soil on the other side, the building moved towards into the excavation with maximum lateral movement of about 13cm.

The maximum pile movement is about 12cm. The foundation pipe piles were retrained by the superstructure at the top and restrained by the competent soil layer at the bottom.

The south-western corner pile on the excavation side is the most critical pile. The maximum of axial force and bending moment on the pile are 1200kN and 140kN.m, respectively according to the analysis.

It is likely that the critical piles of the building failed first which triggered the progressive failure of other piles and eventually the toppling of the building.

6. References


On the numerical simulation of landslides and mudflows

Thomas ZIMMERMANN\textsuperscript{a,1}, Matthias PREISIG\textsuperscript{b}
\textsuperscript{a}Zace Services Ltd, Lausanne, Switzerland
\textsuperscript{b}Princeton University, Department of Civil and Environmental Engineering, USA

Keywords: Slope instabilities, landslides, mudflows, numerical modeling

Abstract
Slope instabilities landslides and mudflows, often caused by extreme weather conditions occur frequently; it is useful therefore to develop simulation tools allowing to better predict consequences of such events. In this paper we present a simple and robust model, based on the equations governing the flow of incompressible viscous fluids, and mixture theory. The model is fully documented elsewhere and only illustrated in this paper. At present, the model is 2-dimensional and does not include the initiation mechanism, which can already be simulated with ZSOIL. It can be viewed as a prototype of coming up new features in ZSOIL.PC.

1. Introduction
Large slope instabilities, landslides and debris flows, sometimes leading to severe accidents, are frequently observed nowadays. It seems useful under such circumstances to reexamine existing simulation techniques and to develop new modeling techniques aiming at a better understanding of the initiation, the development and the impact of such events.

Different approaches to the simulation of this type of events are possible and documented in the literature. The model we propose in this paper is original in its formulation and algorithms. It is robust and permits to follow slope failure events from initiation to deposition. The model was initially developed as a Ph.D thesis at the Swiss Federal Institute of Technology [1] and further improved [2] later on. The model makes the assumption of a 2-phase mixture composed of two viscous fluids. In its present version, it is 2-dimensional in space and assumes linear constitutive behavior, viscosity of each phase is the only constitutive parameter. The motion is described as Lagrangian but conceptually ALE [3]. Extension of the model to 3D and/or to more complex nonlinear constitutive formulations is straightforward.

\textsuperscript{1} Corresponding Author, email address: zimmermann@zace.com (Th. Zimmermann).
A landslide begins when a slope loses stability (Fig. 1), often in relation with a change of pore pressure and saturation in the ground. The mass involved in the instability can be determined with a program like ZSOIL [4]. This mass consists schematically of a „pseudo-solid“ phase and a fluid phase. The model presented in this paper permits to follow the motion of the two phases as they move, mix and sediment while the slope instability transforms into a landslide.

The proposed model is developed and validated in references [1, 2], this paper illustrates its potential with two illustrations: first the collapse of a lake shore into the reservoir, diffusion and sedimentation of solid material into the lake and induced waves are here of interest, second the impact of a two-phase mudflow on a rigid protection dam.

![Figure 1](image.png)

**Figure 1.** Definition of the unstable mass (left). Assumed initiation of motion (right)
red = maximum displacement amplitude, blue = zero displacement amplitude.

2. **Governing equations**

**Assumptions**
We make the assumption of a two-phase mixture composed of two incompressible viscous fluids with linear constitutive behavior. The mixture formed by the two phases has variable concentration of the two phases in space and time, without mass exchange. \( C_s \) and \( C_f \) are the respective concentrations of the phases, \( s \) for the solid, \( f \) for the fluid. Both phases remain of course fluids, but with potentially very different viscosities. Both phases envelop each other completely, are present everywhere in the analysis domain and occupy the the whole domain together. As a consequence pressure must be same in the two phases.
**Balance laws**

A rigorous derivation based on mixture theory is developed in [6]. Formulating a weak form and then a matrix form follows now classical methods. Details are presented in references [1, 2]. The final matrix equation has the usual form of an equation of motion, where \( M \) is a nonlinear mass matrix, \( K \) a viscosity matrix which includes momentum exchange between phases, also nonlinear, \( a \) is the vector of nodal accelerations of the material points of the two phases, \( v \) the vector of nodal velocities and pressures:

\[
M(x_{n+1}, C_{n+1})a_{n+1} + K(x_{n+1}, C_{n+1})v_{n+1} = F_{ext}^{n+1}
\]  \hspace{1cm} (1)

where \( n+1 \) is the time step index.

**Remarks:**

1) The incremental Lagrangian form is adopted here, which avoids the use of stabilized formulations necessary for Eulerian formulations when convection is important.

2) Incompressibility requires appropriate provisions to avoid locking phenomena and pressure oscillations. A stabilized formulation is adopted here and discussed in [1, 2].

**3. Numerical model**

**3.1 Time stepping**

Direct time-integration is used with a trapezoidal algorithm described next. Spatial discretization can be done with finite elements, updated at each time step, or with « meshfree methods ». At each time step, a prediction is made for the motion variables, essentially velocities in the two phases; the mesh is updated on this basis. Velocities are different in the two phases, the position update in the two phases therefore leads to distinct points for two initially coincident points, i.e.:

\[
x_{s}^{n+1} = x^{n} + \Delta d_{s}
\]  \hspace{1cm} (2)

\[
x_{f}^{n+1} = x^{n} + \Delta d_{f}
\]  \hspace{1cm} (3)
To avoid proliferation of nodes the solution can be interpolated on a new mesh, which can be fixed in space or mobile, e.g. corresponding to the motion of the mixture, but the same for the two phases.

The new mesh must be adapted to boundary conditions at current time $t_{n+1}$. For boundaries subject to Dirichlet type boundary conditions, prescribed velocities will be imposed on boundaries of the updated mesh, which must correspond with the position of the boundary condition at current time, a projection may be needed here. For boundaries subject to Neumann conditions, an update of the mesh based on the motion of the mixture can be done, based on the displacement increment, as illustrated in figure 2. Boundary conditions corresponding to the free surface are imposed naturally and do not require any additional action.

The update of nodal positions based on nodal motions of the two phases usually leads to a highly irregular distribution of nodes. An optimization of nodal positions may then be needed to maintain satisfactory aspect ratios of the elements, adaptivity can be introduced simultaneously.

$$x_{mixture}^{n+1} = x_{mixture}^n + C_x \Delta d_x + C_f \Delta d_f$$

The update of nodal positions based on nodal motions of the two phases usually leads to a highly irregular distribution of nodes. An optimization of nodal positions may then be needed to maintain satisfactory aspect ratios of the elements, adaptivity can be introduced simultaneously.

Finally, concentrations of the 2 phases must be redefined on the nodes of the new mesh. This problem is theoretically simple; it is sufficient, in principle, to define the total mass of each phase located in the vicinity of each node and to compute the corresponding concentrations. In reality, it appears difficult to formulate an algorithm which does not generate chaotic results. This point is discussed in details in [1, 2].

Dynamic equilibrium is then computed and a correction of the nodal variables and of the mesh position is performed.
3.1 Spatial discretization

Spatial discretization of the computational domain is straightforward for an Eulerian formulation, as long as the fluid remains within the same boundaries. For free-surface flows front-tracking is however needed. In addition, the domain that has to be discretized includes the envelope of the whole domain through which the fluid flows during the analysis. These reasons related to spatial discretization, together with the need for stabilization for convection dominated flows, led us to prefer a Lagrangian formulation.

Mesh distortion is the main problem associated with a Lagrangian method for fluid flow. Two main approaches were considered in this study: on the one hand, meshfree methods, which provide a means of constructing an approximation without the need for any information other than the nodal locations, and most meshfree methods also have the advantage of being insensitive with respect to uneven distributions of nodes; on the other hand, standard finite element methods in conjunction with appropriate remeshing procedures, which provide a means of exploiting the good performance of a more robust method. The latter is retained in the sequel.

3.1.1 Spatial discretization using finite elements

Delaunay triangulation can be used to generate a mesh of triangular finite elements. By using triangular finite elements, together with a rezoning and remapping strategy, the most desirable properties of meshfree methods can be combined with the assertion of good performance of a well-understood approximation technique. At each step, a new set of nodes is obtained from the computation of the velocity field and a new mesh can be created by Delaunay triangulation. Excessive mesh distortion leading to a breakdown of the computation can be avoided by periodically rezoning (re-creating) the set of nodes. The method also permits computing solutions on fixed meshes, in which case the solution on the updated mesh is mapped back onto the fixed (Eulerian) mesh after each step.

4. Implementation

The model is implemented in prototyping, object-oriented, freeware FEM_Object [7, 8] and uses graphical toolbox Cgal [9], in particular for mesh generations. In a context of prototyping and algorithmic evaluation, these choices proved optimal, a number of different algorithmic variants could thus be experimented.
5. Applications

Accuracy, performance and validation of FEM and NEM methods are compared on patch tests and simple boundary value problems in [1, 2, 10]. Discretization requirements are similar, since Voronoi cells and Delaunay triangulation come as by-products. The following applications are computed with the FEM formulation, but could be repeated with NEM. Two illustrations, which are representative of possible real world situations, are presented next.

5.1 Application: Collapse of a lake shore.

We consider here the case of a lake shore which collapses, diffuses solid material into the lake and finally sediments. The case is hypothetical. The unstable soil material consist of a two-phase material with a solid concentration of $C_s=0.9$, which collapses into a reservoir filled with a two-phase material with a solid concentration of $C_s=0.05$, i.e. essentially a fluid. Other mechanical properties are: $\rho_s=10$, $\rho_f=0$, $\mu_s=0$ and, for numerical reasons, $\mu_f=0.01$.

Geometry is described in Figure 3a. Solid material diffusion, sedimentation and a propagating wave are observed, as shown in Figure 3.

![Figure 3](image.png)

Figure 3. Collapse of lake shore, geometry, free-surface and sedimentation at different times (shown shaded)
4.2 Application: Impact of mudflow on obstacle.

This case is similar to the preceding one, the downhill propagation of a two-phase mudflow and its impact on an obstacle is analyzed. The flow is initialized with a sudden release of the mixture. Propagation of single and two-phase flow with the following properties are compared in figure 4:

2-phase mixture: \( \rho_s = 1000, \quad \rho_f = 500, \quad \mu_s = 100 \) and, \( \mu_f = 2, \quad K_{\text{drag}} = 1000, \quad C_s_{\text{initial}} = 0.5 \)

1-phase mixture: \( \rho = 750, \quad \mu = 51 \).

Solid concentrations can be read from color maps and force resultants are plotted as a function of time, with the hydrostatic solution as asymptote.

![Solid volume fraction](image1)

**Figure 4.** Two-phase mudflow impact on an obstacle. Flow at different times and resultant force on obstacle.
6. Conclusion

A simple and robust numerical model for the simulation of two-phase viscous mudflows is proposed in this paper. “Meshfree” NEM using domain integration and finite element implementations are compared in references [1, 2, 10] and further tests using nodal integration are discussed in [12], FEM results are presented in this paper. The model is shown to permit qualitative and quantitative assessment of natural phenomena like landslides and corresponding induced flows, with a reasonable accuracy. The model is extensively validated in references [1, 2, 10]. Two examples of possible real world applications illustrate its potential in this paper. The linear viscous two-dimensional model discussed here is easily extendable to three space dimensions and to more complex constitutive laws.

Acknowledgments

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References


