YAYASAN PENDIDIKAN DAYANG SUMBI
INSTITUT TEKNOLOGI NASIONAL
LEMBAGA PENELITIAN DAN PENGABDIAN KEPADA MASYARAKAT
Jl. PHH Mustapa 23, Bandung 40124 Indonesia, Telepon: +62-22-7272215 ext 157, Fax:022-720 2892
Web site: http://www.itenas.ac.id., e-mail: lpp@itenas.ac.id

SURAT KETERANGAN

MELAKUKAN KEGIATAN PENGABDIAN KEPADA MASYARAKAT

INSTITUT TEKNOLOGI NASIONAL

No. 056/C.02.01/LPPM/II/2021

Yang bertanda tangan di bawah ini,

Nama : Iwan Juwana, S.T., M.EM., Ph.D.
Jabatan : Kepala
Unit Kerja : LPPM-Itenas
JL. P.K.H. Mustafa No.23 Bandung

Menerangkan bahwa,

No.	Nama	NPP	Jabatan
1	Dr.techn. Indra Noer Hamdhan, S.T., M.T.	20030105	Tenaga Ahli
2	Dr. Ir. Imam Aschuri, M.T.	940102	Tenaga Ahli
3	Ikhya, S.T., M.T., M.M.	20020107	Tenaga Ahli
4	Desti Santi Pratiwi, S.T., M.T.	20190804	Tenaga Ahli
5	Badriana Nuranita, S.T., M.T.	20200201	Tenaga Ahli
6	Rika Hernawati, S.T., M.T.	20160401	Tenaga Ahli

Telah melakukan kegiatan Pengabdian kepada Masyarakat sebagai berikut:

Nama Kegiatan : NGC Webinar Series #1 Pemodelan Numerik Geoteknik dalam Perencanaan Infrastruktur
Tempat : Webinar/*Zoom Meeting*
Waktu : 23 September 2020
Sumber Dana : RKAT Prodi Teknik Sipil Tahun 2020

Demikian surat keterangan ini dibuat untuk dapat dipergunakan sebagaimana mestinya.

Bandung, 03 Februari 2021

Lembaga Penelitian dan Pengabdian kepada Masyarakat (LPPM) Itenas
Kepala,

Itenas
LPPM

Iwan Juwana, S.T., M.EM., Ph.D.
NPP. 20010601

LAPORAN KEGIATAN
PENGABDIAN KEPADA MASYARAKAT

NGC WEBINAR SERIES #1

PEMODELAN NUMERIK GEOTEKNIK DALAM PERENCANAAN INFRASTRUKTUR

Ketua Tim :

Dr.techn. Indra Noer Hamdhan, ST., MT.

Anggota Tim :

Dr. Imam Aschuri, ST., MT.

Ikhya, ST., MT., MM.

Desti Santi Pratiwi, ST., MT.

Badriana Nuranita, ST., MT.

Rika Hernawati, ST., MT.

Program Studi Teknik Sipil

Fakultas Teknik Sipil dan Perencanaan

Institut Teknologi Nasional

Bandung

2020

HALAMAN PENGESAHAN

Judul	:	NGC Webinar Series #1 Pemodelan Numerik Geoteknik dalam Perencanaan Infrastruktur
Ketua Tim Pengusul		
Nama	:	Dr. techn. Indra Noer Hamdhan, ST., MT.
NIDN	:	0422087303
Jabatan/Golongan	:	Lektor / III D
Jurusan/Fakultas	:	Teknik Sipil / Fakultas Teknik Sipil dan Perencanaan
Bidang Keahlain	:	Rekayasa Geoteknik
Alamat Kantor	:	Jl. PHH Mustafa No. 23 Bandung
Lokasi Kegiatan		
Wilayah Mitra	:	Provinsi Jawa Barat
Desa/Kecamatan	:	-
Kota/Kabupaten	:	Kota Bandung
Provinsi	:	Jawa Barat
Jarak PT ke Mitra	:	-
Luaran	:	-
Waktu Pelaksanaan	:	23 September 2020
Total Biaya	:	Rp 10.000.000

Bandung, 30 Septermber 2020

Mengetahui,
Dekan Fakultas Teknik Sipil dan Perencanaan

itenas
FTSP

(Dr. Soni Darmawan)

Ketua Tim Pengusul

(Dr. techn. Indra Noer Hamdhan)

Disahkan Oleh
Ketua LP2M,

Iwan Juwana, S.T., M.EM., Ph.D.
NIP: 20010601

LAPORAN KEGIATAN

Tujuan

Tujuan dari kegiatan Webinar mengenai pemodelan numerik geoteknik ini adalah untuk memberikan wawasan, gambaran serta wadah diskusi kepada para tenaga-tenaga kerja yang berfokus pada bidang geoteknik terkait analisis menggunakan model numerik.

Deksripsi Pelaksanaan Kegiatan

Waktu pelaksanaan Webinar Pemodelan Numerik Geoteknik dalam Perencanaan Infrastruktur ini adalah selama 1 (satu) hari pada tanggal 23 September 2020 selama kurang lebih 2 (dua) jam. Jumlah peserta yang mengikuti Webinar ini yaitu sebanyak 1386 orang yang tercatat pada formulir pendafataran melalui Google form. Adapun peserta yang mengikuti webinar ini berasal dari banyak instansi, seperti dari Kementerian PUPR, Kementerian Perhubungan, Direktorat Bina Teknik Jalan dan Jembatan, Balai Geoteknik, Terowongan, dan Struktur, Instansi Akademisi, Kontraktor maupun Konsultan yang bergerak di bidang geoteknik di Indonesia.

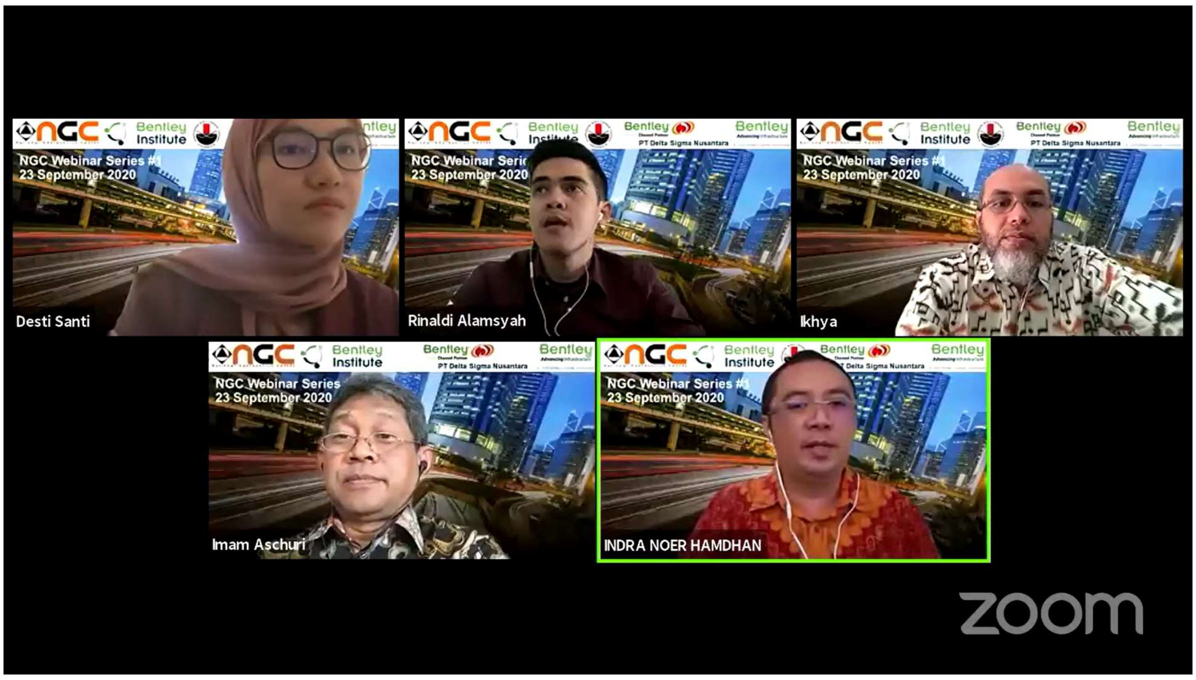
Tujuan dari kegiatan pengabdian kepada masyarakat berupa kegiatan webinar mengenai pemodelan numerik dalam perencanaan infrastruktur pada bidang ilmu geoteknik ini terpenuhi dimana para peserta memahami materi yang telah disampaikan secara daring. Adapun pokok materi yang disampaikan yaitu:

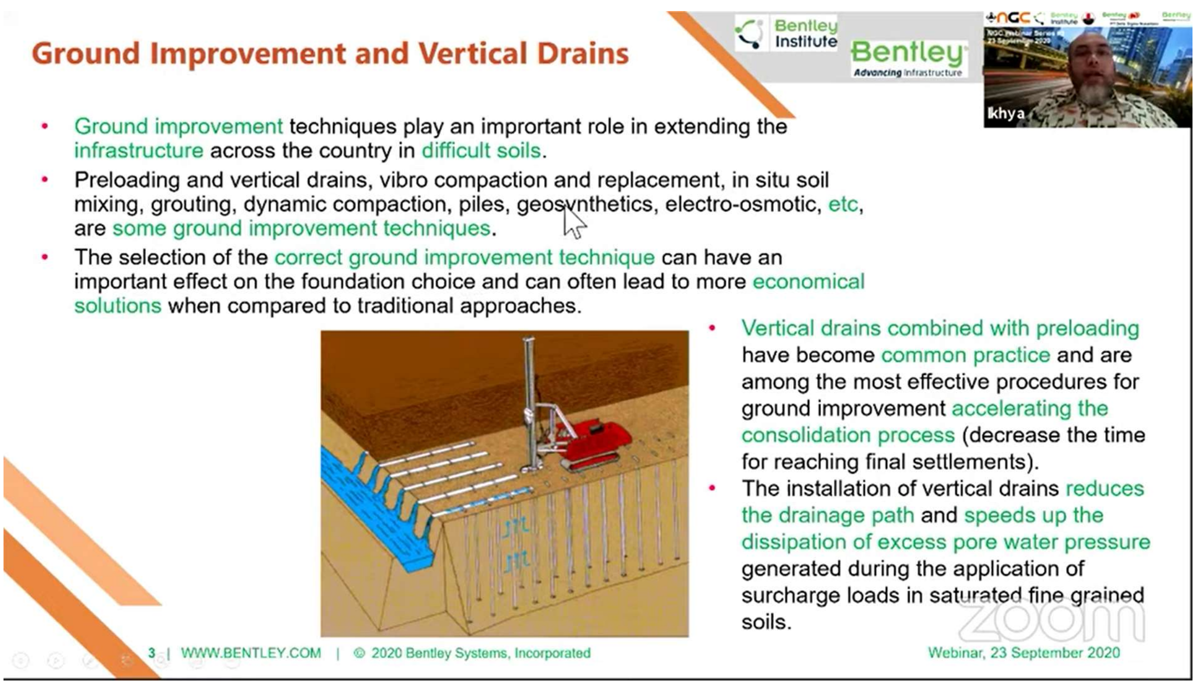
1) Pemodelan dan analisis stabilitas lereng
    - Pendahuluan
    - Analisis stabilitas lereng sederhana menggunakan model Mohr Coloumb
    - Perbandingan FEM dan LEM dalam analisis stabilitas lereng
    - Stabilitas lereng menggunakan PLAXIS 2D
    - Pemodelan stabilitas lereng untuk pekerjaan pembangunan jalan menggunakan PLAXIS 2D
2) Pemodelan dan analisis perbaikan tanah dengan *preloading* dan PVD
    - Tanah lunak di Indonesia
    - Perbaikan Tanah dan PVD
    - Instalasi dari PVD
    - Parameter dari PVD
    - Pemodelan PVD menggunakan PLAXIS 2D

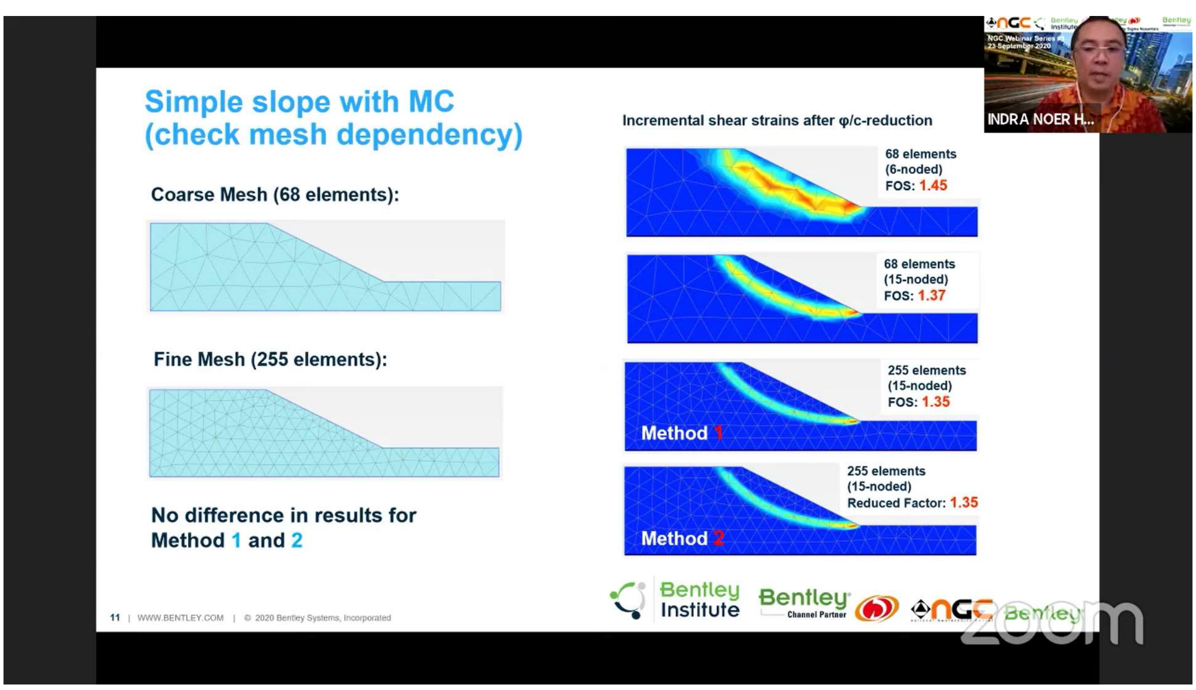
Para peserta yang mengikuti webinar ini akan diberikan sertifikat dalam bentuk elektronik sertifikat.

BUKTI KEGIATAN

- Foto-foto Kegiatan







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18/09/2020 16:04:07	Ferry Abdur Rahman	Itenas	081224096204	ferryabdurahman@gmail.com
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18/09/2020 21:08:02	Rinaldi Alamsyah	Institut Teknologi Nasional	087825111744	alamsyahrinaldi@gmail.com
19/09/2020 7:33:28	ibadurrokhman ali	Pupuk Kaltim	081281766225	ibadurrokhman.ali@pupukkaltim.com
19/09/2020 7:36:15	Muhammad Rais	Karyawan swasta	081250778873	rais_d@yahoo.com
19/09/2020 8:10:48	Aldi Tri Amdani	Institut Teknologi Nasional Bandung	087772582176	alditriamdani@gmail.com
19/09/2020 8:10:52	Arinal Ghalibaldi	Institut Teknologi Nasional Bandung	082115073434	arinalghalibaldi@gmail.com
19/09/2020 8:10:54	Nella Evelyn Enjelina	Insitut Teknologi Nasional	085721707202	nellaevelyn15@gmail.com
19/09/2020 8:10:58	Farid Rizwansyah	Institut Teknologi Nasional	082319226292	faridrizwansyah4@gmail.com
19/09/2020 8:11:23	Muhamad Abdan syakuro Gunawan	ITENAS	082112863843	abdan914@gmail.com
19/09/2020 8:11:30	Gianda Marsedi	Institut Teknologi Nasional Bandung	085316297557	giandamarsedi@gmail.com
19/09/2020 8:11:33	Reza Nugraha	Institut Teknologi Nasional Bandung	081220103535	rezanugraha55@gmail.com
19/09/2020 8:11:44	Niski Wirdatunnisa	Institut Teknologi Nasional	081214154450	wirdatunnisaniski@gmail.com
19/09/2020 8:11:45	Riana Alifya	Institut Teknologi Nasional	085846286045	rianaalifya.ra@gmail.com
19/09/2020 8:11:48	Alvhireal Bella Vheatrieze Analkhis	Institut Teknologi Nasional	087727634794	alvhirealb@gmail.com
19/09/2020 8:11:53	Dio Arya Erlangga	Mahasiswa ITENAS	089656376668	dioaryaa@gmail.com
19/09/2020 8:11:59	Obih Hidayat	Institut Teknologi Nasional	082240870527	obihhidayat12@gmail.com
19/09/2020 8:12:04	selviana rheina effendi	itenas	082120781947	rheinaeffendi123@gmail.com
19/09/2020 8:12:15	Rafli M Fauzan Farhan	ITENAS	083149811178	raflyfaudzan7@gmail.com
19/09/2020 8:12:32	Fikri Akbar Kusnaedi	Institut Teknologi Nasional	087760654766	fikriakbarkim@gmail.com
19/09/2020 8:13:21	Abelardo Elguna Novharieza Gunawan Putra	ITENAS	085220540203	abelardoelguna@gmail.com
19/09/2020 8:13:29	PANJI PUTRA PRATAMA	Institut Teknologi Nasional	082126758584	pratama,putra1311@gmail.com
19/09/2020 8:13:36	Ibnu Adhitomo	Institut Teknologi Nasional Bandung	082213334847	ibnuadhitomo98@gmail.com
19/09/2020 8:13:51	Aldy Adriawan	Itenas	087708439677	aldy,adriawan@gmail.com
19/09/2020 8:14:19	Nur Salfianty A,D	Institut Teknologi Nasional	081285911542	nsalfianty@gmail.com
19/09/2020 8:14:47	Alya Salma Hanifah	ITENAS	081221220026	almahanifah@yahoo,com
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19/09/2020 8:14:53	Bagus Maulana Pratikno	Intitut Teknologi Nasional Bandung	087821611604	bagus.maulana13@gmail.com
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19/09/2020 8:18:10	DELINAISMIE NURRENSYAH	ITENAS BANDUNG	085974921334	delinaismie27@gmail.com
19/09/2020 8:19:06	Fitri Mustika Dewi	Institut Teknologi Nasional	082240488877	fmustikadewi@gmail.com
19/09/2020 8:19:06	Salma Dwiyanda	ITENAS	082127005496	salmadwiyandaa@gmail.com
19/09/2020 8:19:14	Muhammad Ilham	Institut Teknologi Nasional	081212427252	ilhammuhammad188@gmail.com

Slope Stability Analysis with PLAXIS 2D

Dr. techn. Indra Noer Hamdhan

PT Delta Sigma Nusantara

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MoU ITENAS and Bentley Systems

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Institut Teknologi Nasional

Bentley
Advancing Infrastructure

MEMORANDUM OF UNDERSTANDING

BETWEEN

INSTITUT TEKNOLOGI NASIONAL

AND

BENTLEY SYSTEMS INTERNATIONAL LIMITED

MEMORANDUM OF UNDERSTANDING

BETWEEN

INSTITUT TEKNOLOGI BANDUNG

AND

BENTLEY SYSTEMS INTERNATIONAL LIMITED

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Bentley Institute
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    a) Knowledge sharing for students and student Ambassador program;

    b) Knowledge sharing for staff and Staff Ambassador program;

    c) International Bentley Webinars;

    d) Bentley conference participations;

    e) ITENAS to organize session and events via ITENAS-Bentley BIM Laboratory formation; and

    f) any other areas of co-operation to be mutually agreed upon by the Parties.

(Source: https://www.mynewlab.com/services/bim-laboratory-furniture-design/

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YAYASAN PENDIDIKAN DAYANG SUMBI

INSTITUT TEKNOLOGI NASIONAL

Jl. PKH. Hasan Mustapa No. 23 Bandung 40124 Indonesia, Telepon: +62-22-7272215, Fax: +62-22-7202892
Website: http://www.itenas.ac.id., e-mail: rektorat@itenas.ac.id

KEPUTUSAN
REKTOR INSTITUT TEKNOLOGI NASIONAL
Nomor: 0207/A.17.01/Rektorat/Itenas/XII2019

tentang:

Pendirian Pusat Studi Geoteknik Nasional Itenas

....

REKTOR INSTITUT TEKNOLOGI NASIONAL:

Menimbang :
1. Bahwa salah satu misi Institut Teknologi Nasional adalah mengembangkan Penelitian dibidang Teknologi.
2. Bahwa untuk mewujudkan misi Institut Teknologi Nasional tersebut perlu dibentuk Pusat Studi Geoteknik Nasional Itenas sebagai suatu wadah yang dapat mendorong berkembangnya penelitian dalam bidang Geoteknik.

Mengingat :
1. Peraturan Pemerintah No. 30 Tahun 1990:
2. Undang-Undang No. 2 Tahun 1989;
3. Keputusan Menteri Pendidikan dan Kebudayaan;
   a. No. 021 1 IU11982 Tanggal 26 Juni 1982;
   b. No. 0180/0/1985 juncto No. 0180i0/1985 Tanggal 20 April 1985;
4. Keputusan Dewan Pengurus Yayasan Pendidikan Dayang S~unbi:
   a. No. 01 KepIDSi1972 Tanggal 14 Desember 1972;
   b. No. 07Kpts/YPDS/11/2012 tanggal 6 Febniari 2012.

MEMUTUSKAN

Menetapkan :

Pertama : Pendirian Pusat Studi Geoteknik Nasional Itenas

Kedua : Pusat Studi Geoteknik Nasional Itenas berada di bawah koordinasi Lembaga Penelitian dan Pengabdian Kepada Masyarakat yang bertanggungjawab kepada Rektor.

Ketiga : Anggaran pengembangan Pusat Studi Geoteknik Nasional Itenas menjadi bagian dari Rencana Kegiatan dan Anggaran Tahunan Lembaga Penelitian dan Pengabdian Kepada Masyarakat.

Keempat : Surat Keputusan ini mulai berlaku sejak tanggal ditetapkannya dengan ketentuan bahwa akan diperbaiki sebagaimana mestinya apabila dikemudian hari ternyata terdapat kekeliruan dalam Surat Keputusan ini.

Ditetapkan di : Bandung
Pada tanggal : 16 Desember 2019
Rektor,

itenas
INSTITUT TEKNOLOGI NASIONAL

Dr. Imam Aschuri, Ir., M.T.

Tembusan :
1. Yth. Ketua Yayasan Pendidikan Dayang Sumbi ;
2. Yth. Para Wakil Rektor;
3. Yth. Para Dekan Fakultas;
4. Yth. Ketua LPPM;
5. Yth. Para Ketua Jurusan.

NGC
National Geotechnics Center

Pusat Studi Geoteknik Nasional (*National Geotechnics Center*) Itenas

15.12.2019

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Bentley Institute

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Geotechnical Engineering	SoilVision 2D/3D Suite, gINT Professional
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Structural Analysis and Detailing	STAAD.Pro, STAAD Advanced Concrete Design, STAAD Foundation Advanced, RAM Structural System, RAM Connection, RAM Elements, RAM Concept, MicroStran, ProStructures

Hydraulics and Hydrology	OpenFlows WaterGEMS, OpenFlows WaterCAD, OpenFlows SewerGEMS, OpenFlows SewerCAD, OpenFlows HAMMER, OpenFlows CivilStorm, OpenFlows StormCAD, OpenFlows PondPack, OpenFlows FlowMaster
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Pipe Stress and Vessel Analysis	AutoPIPE
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2020 BENTLEY CHANNEL PARTNER AGREEMENT

BENTLEY SOLUTIONS RESELLER

INTERNATIONAL

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Contents

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Introduction

Dr. techn. Indra Noer Hamdhan

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Introduction

Definition of safety factor obtained by FEM

(available = characteristic value)

$$\eta_{fe} = \frac{\tan\varphi_{available}}{\tan\varphi_{failure}} = \frac{c_{available}}{c_{failure}}$$

Basically 2 possibilities to obtain factor of safety:

1: Calculation with characteristic parameters > automatic reduction of strength parameters of soil until equilibrium is no longer achieved in numerical analysis

Some FE-codes do this automatically > strength-reduction technique

2: Calculation with reduced parameters > perform *new* calculation with different factors until equilibrium is no longer achieved in numerical analysis

Some codes do this automatically

see also: e.g. Griffiths (1980), Naylor (1981), Brinkgreve & Bakker (1991), Matsui & San (1992)

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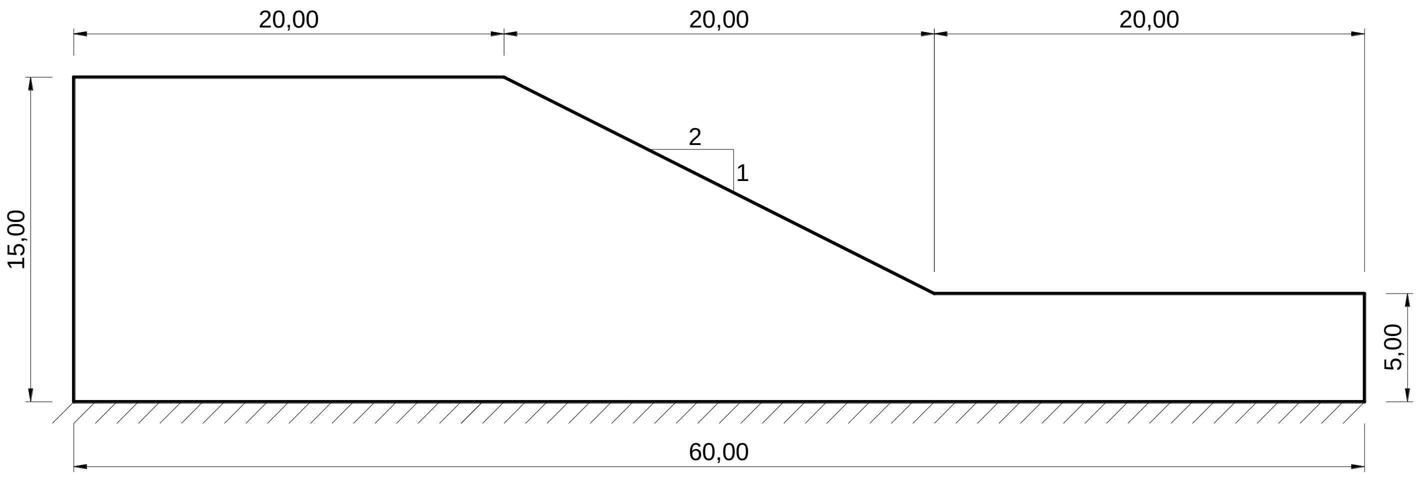
Simple slope with MC (check mesh dependency)

Homogeneous, drained soil layer

Inclination of slope: 2:1



Soil parameters (Mohr-Coulomb failure criterion):

φ' = 20.0 [°] , c' = 10.0 [kN/m²]

E = 10^5 [kN/m²] , ν = 0.3 [-] , γ = 20.0 [kN/m³]

Example from: Griffiths and Lane, Slope stability analysis by finite elements, Geotechnique 49, 387-403, 1999

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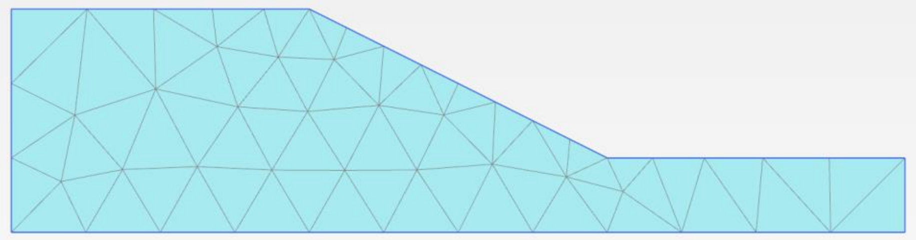
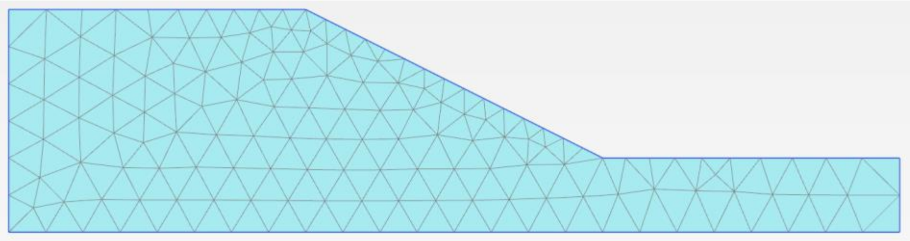
Simple slope with MC (check mesh dependency)

Coarse Mesh (68 elements):

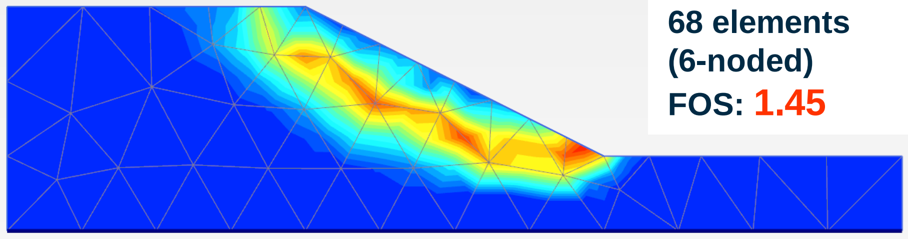


Fine Mesh (255 elements):

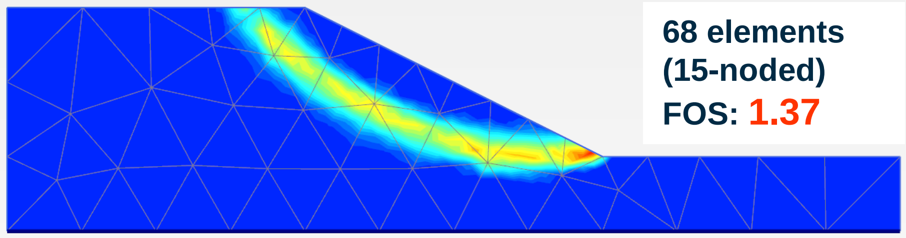


No difference in results for Method 1 and 2

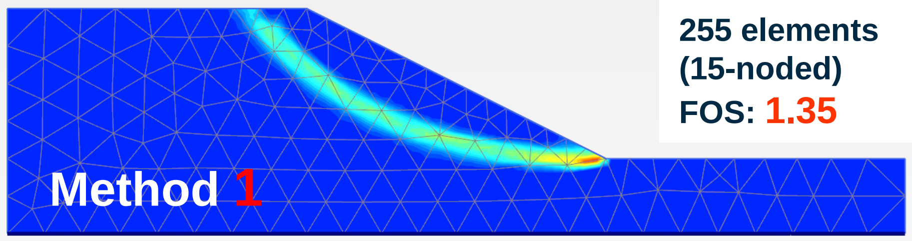
Incremental shear strains after φ/c-reduction



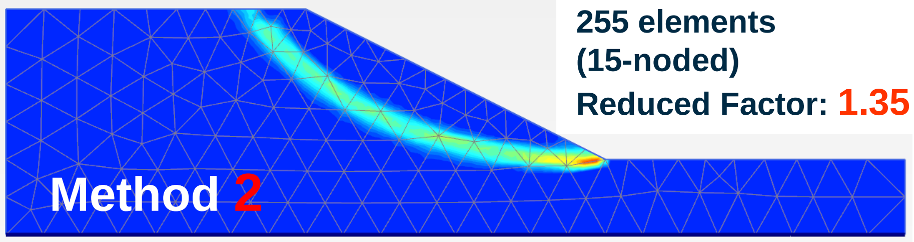
68 elements
(6-noded)
FOS: 1.45



68 elements
(15-noded)
FOS: 1.37



255 elements
(15-noded)
FOS: 1.35



255 elements
(15-noded)
Reduced Factor: 1.35

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FEM vs LEM in Slope Stability Analysis

Dr. techn. Indra Noer Hamdhan

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FEM vs. LEM

Advantage of FEM vs. LEM for Slope Stability Analysis

1. In the **FEM**, failure occurs naturally through the zones within the soil mass wherein the shear strength of the soil is not capable to resist the applied shear stress, so there is **no need to make assumption about the shape or location of the failure surface**.

2. There is no theory of slices in the **FEM**, so **no need to make assumption about slide side forces**. The FEM maintains overall equilibrium until failure is reached.

3. As long as the compressibility data of soils is available, the **FEM** will **provide deformations** result at the working stress levels.

4. The **FEM** is capable to check **the progressive failure** up to and including shear failure.

Source: Griffiths and Lane, Slope stability analysis by finite elements, Geotechnique 49, 387-403, 1999

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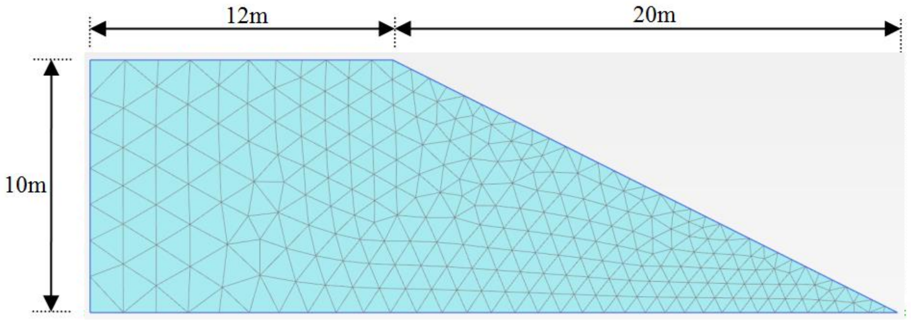
FEM vs. LEM

FEM

(1) Homogeneous slope with no foundation layer

Geometry and mesh:

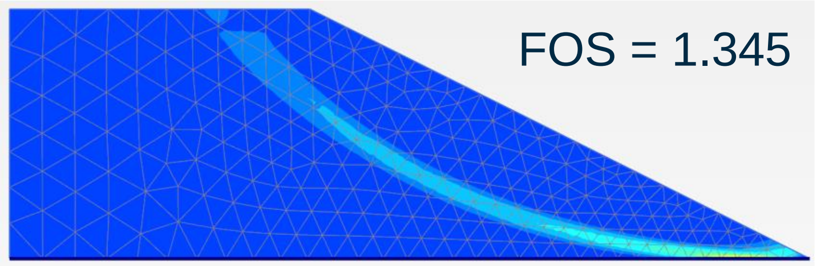


Soil parameters with Mohr-Coulomb model:

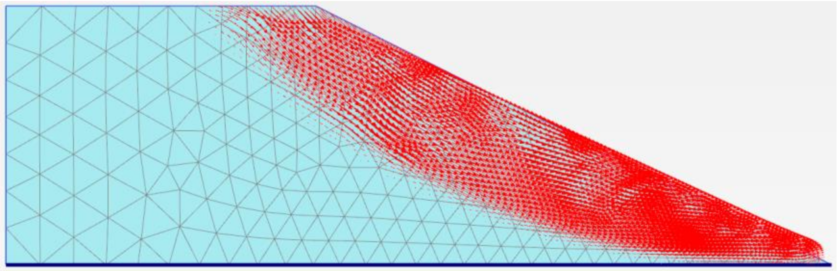
Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (effective shear strength)	c'	[kPa]	10
Friction angle (effective shear strength)	ϕ'	[º]	20

Failure mechanism:

Incremental strains:



Incremental displacement:



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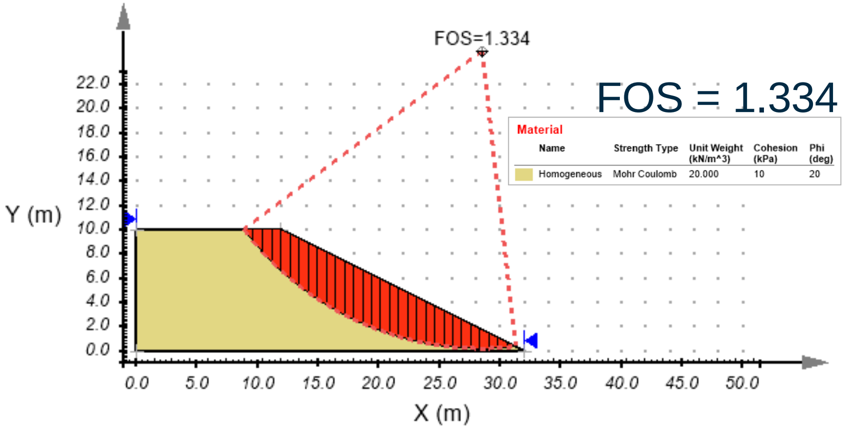
FEM vs. LEM

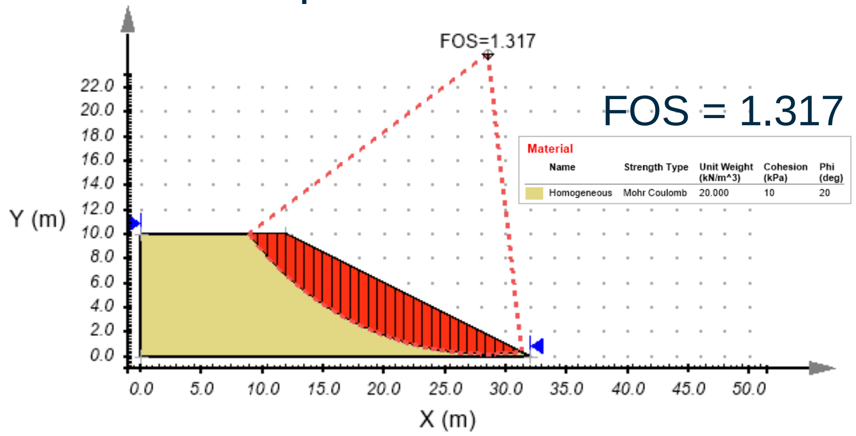
LEM

(1) Homogeneous slope with no foundation layer

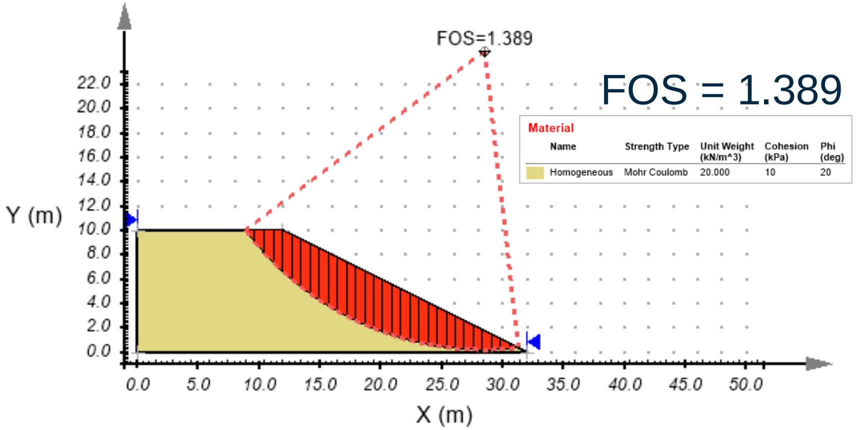
Ordinary Method of Slice:



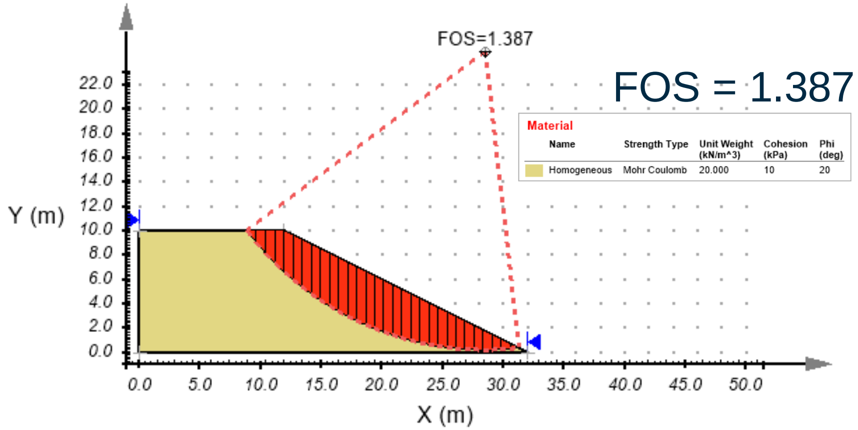
Janbu's Simplified Method :



Bishop's Simplified Method :



Morgenstern and Price Method :



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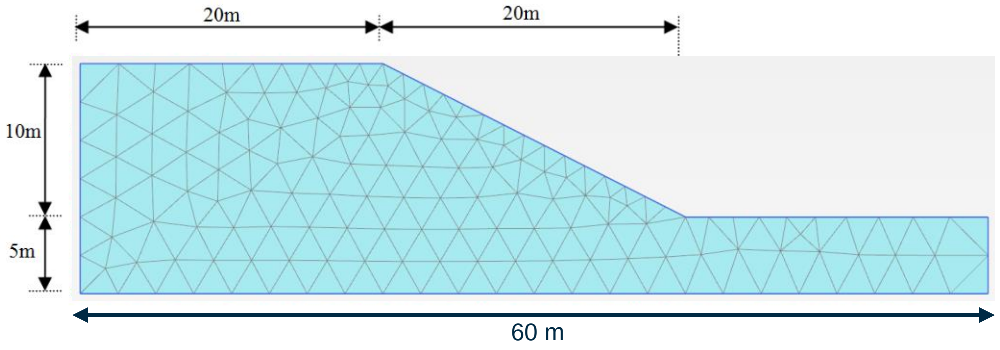
FEM vs. LEM

FEM

(2) Homogeneous slope with foundation layer

Geometry and mesh:

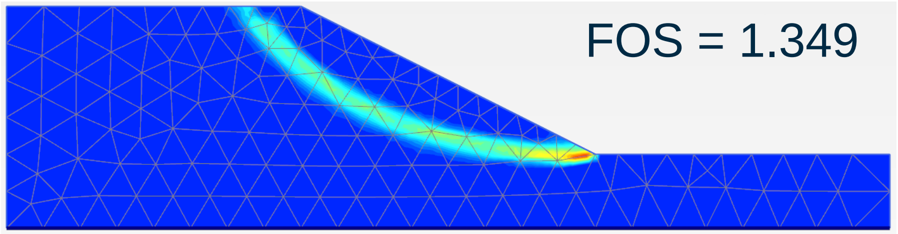


Soil parameters with Mohr-Coulomb model:

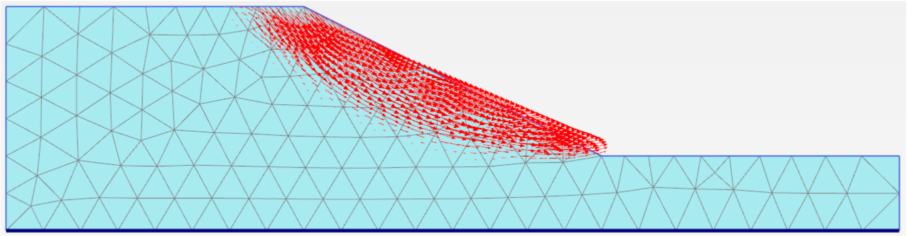
Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (effective shear strength)	c'	[kPa]	10
Friction angle (effective shear strength)	ϕ'	[º]	20

Failure mechanism:

Incremental strains:



Incremental displacement:



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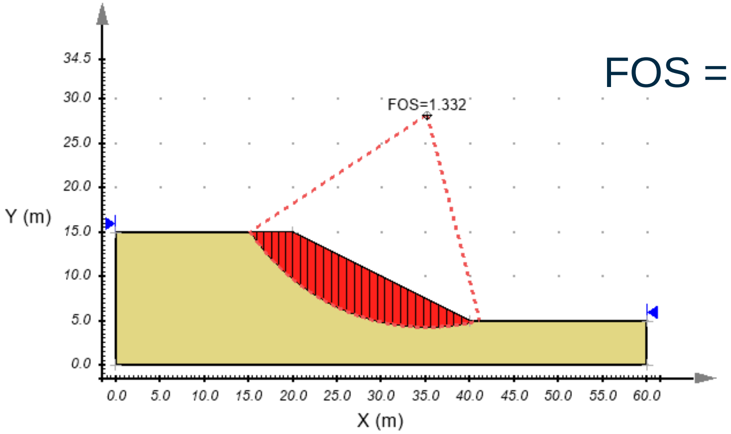
FEM vs. LEM

(2) Homogeneous slope with foundation layer

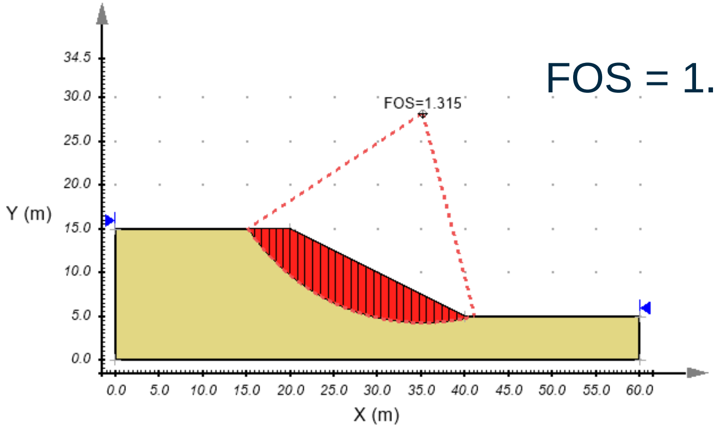
LEM

Ordinary Method of Slice:



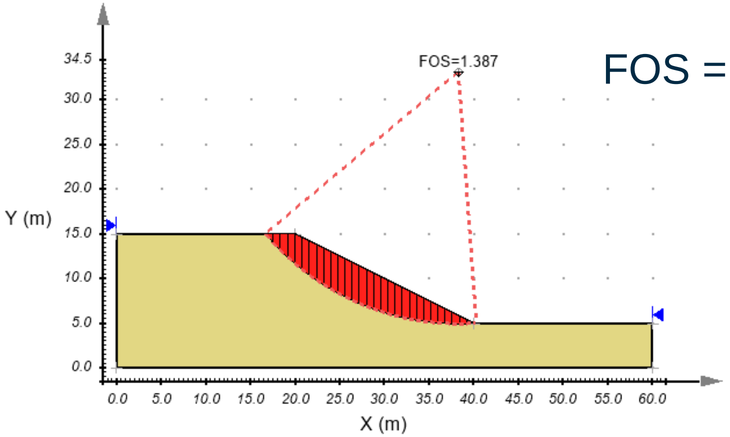
FOS = 1.332

Janbu's Simplified Method :



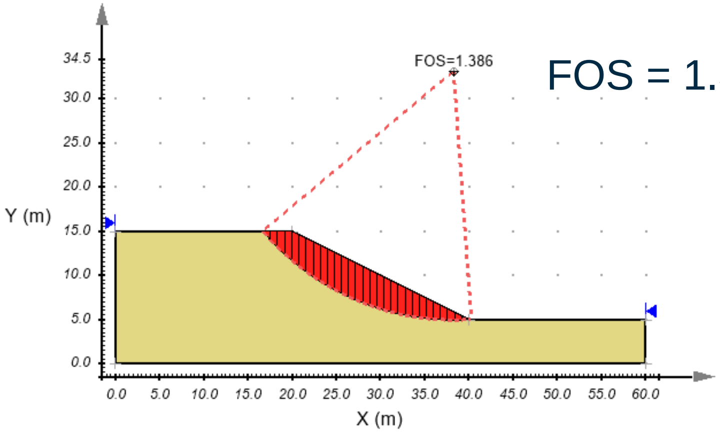
FOS = 1.315

Bishop's Simplified Method :



FOS = 1.387

Morgenstern and Price Method :



FOS = 1.386

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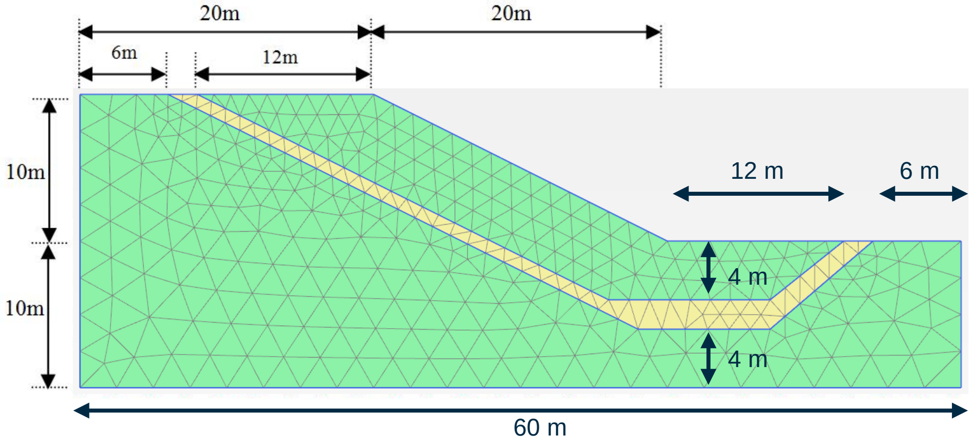
FEM vs. LEM

FEM

(3) An Undrained Clay Slope with a Thin Weak Layer

Geometry and mesh:



Soil parameters with Mohr-Coulomb model:

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (undrained shear strength)	c_{u1}	[kPa]	50
Friction angle (undrained shear strength)	ϕ_u	[º]	0

The analysis are carried out using a constant value of undrained shear strength of soil (cu_1) and five different values of undrained shear strength of the thin layer (cu_2) with ratio cu_2/cu_1 equal to **1**, **0.8**, **0.6**, **0.4**, and **0.2**.

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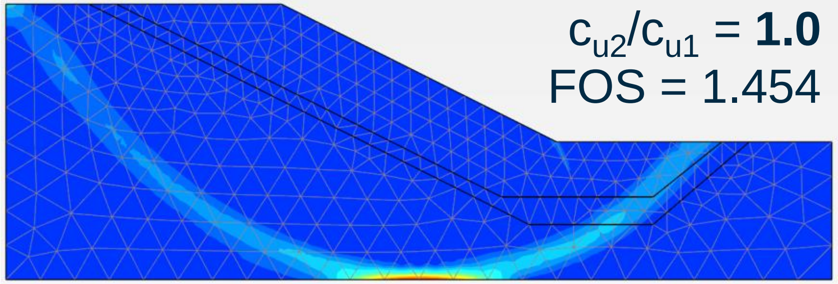
FEM vs. LEM

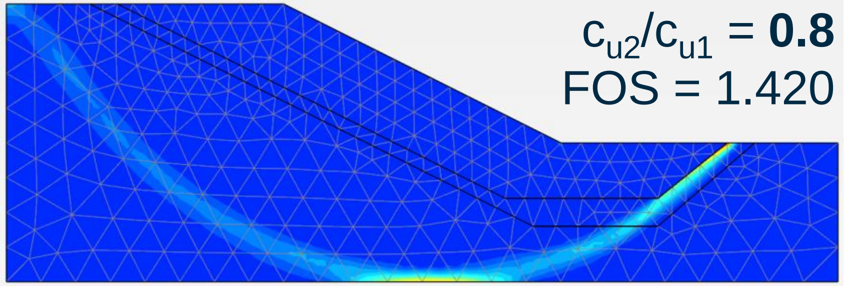
FEM

(3) An Undrained Clay Slope with a Thin Weak Layer

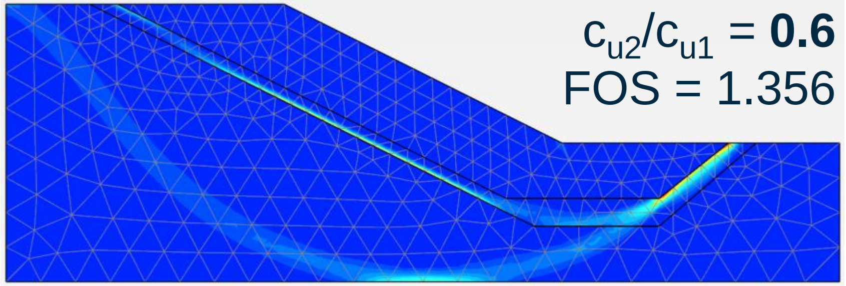
Failure mechanism (incremental strains):



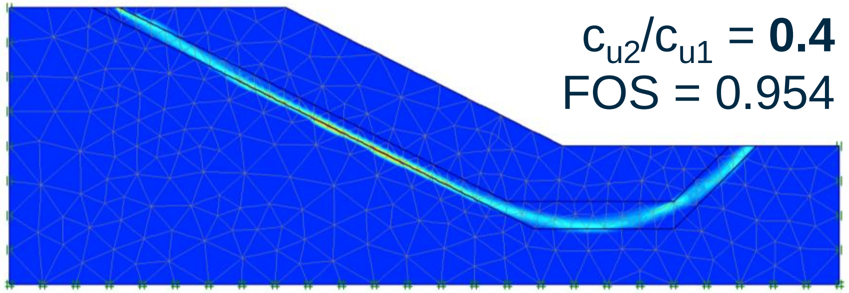
c_{u2}/c_{u1} = **1.0**
FOS = 1.454



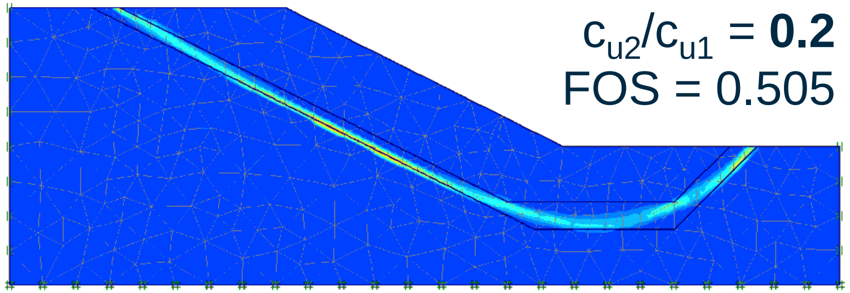
c_{u2}/c_{u1} = **0.8**
FOS = 1.420



c_{u2}/c_{u1} = **0.6**
FOS = 1.356



c_{u2}/c_{u1} = **0.4**
FOS = 0.954



c_{u2}/c_{u1} = **0.2**
FOS = 0.505

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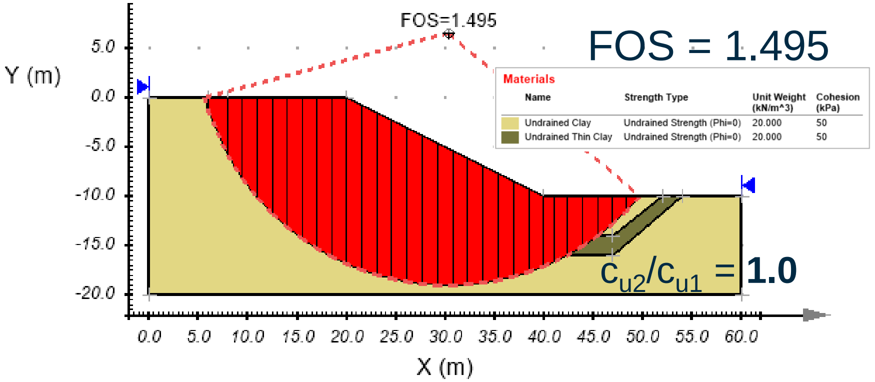
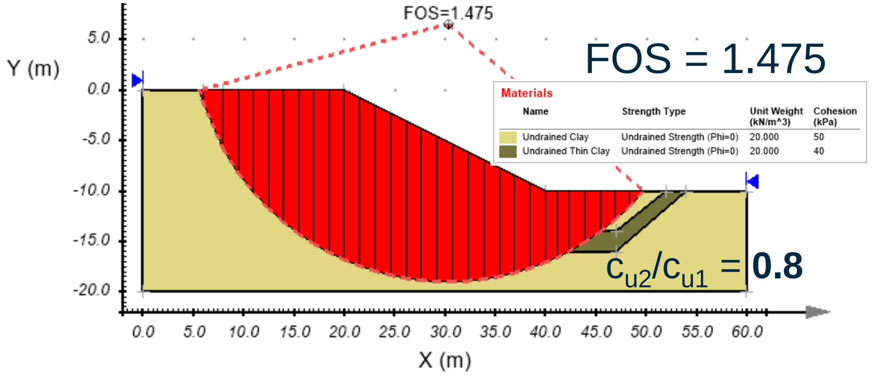
FEM vs. LEM

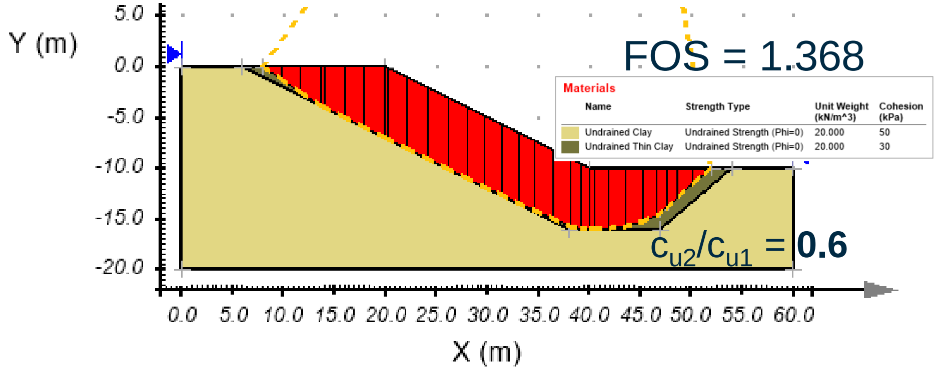
LEM

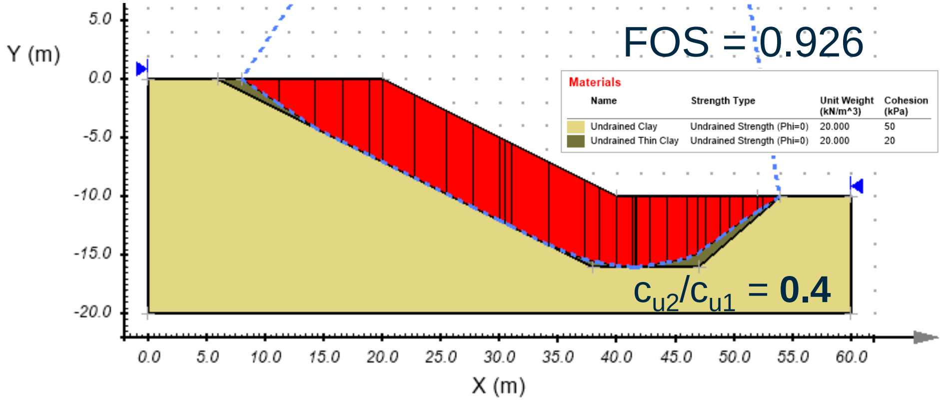
(3) An Undrained Clay Slope with a Thin Weak Layer

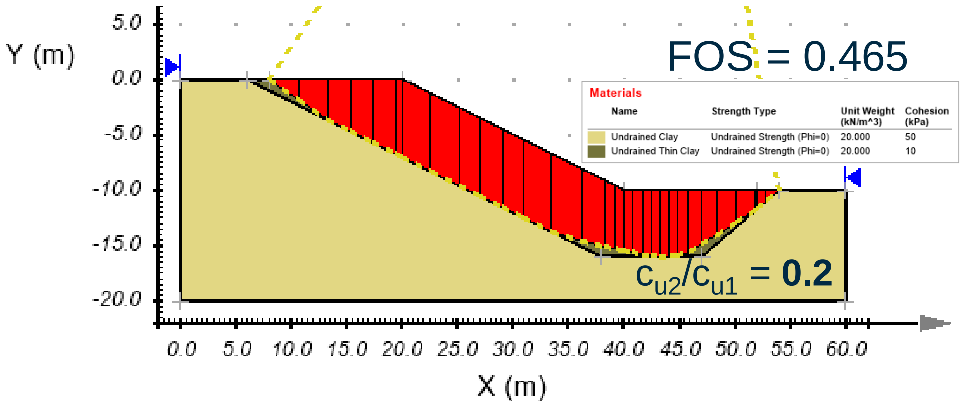
Morgenstern and Price Method :











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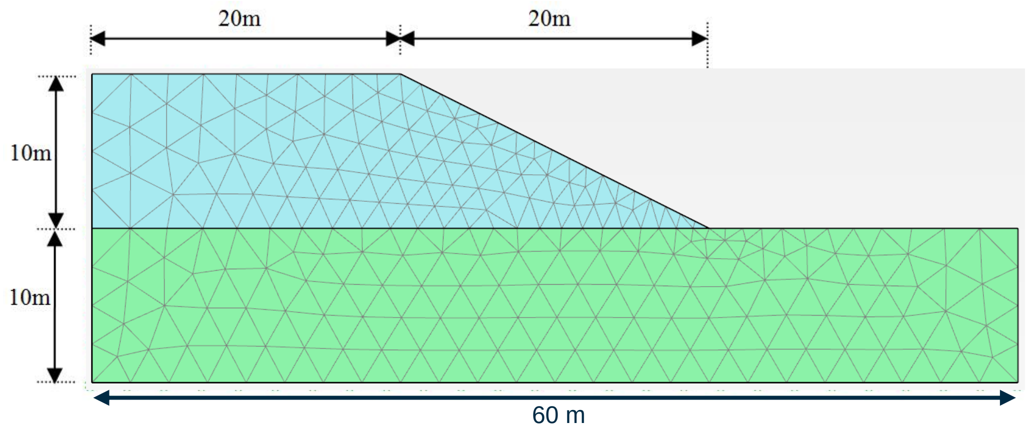
FEM vs. LEM

FEM

(4) An Undrained Clay Slope with a Weak Foundation Layer

Geometry and mesh:



Soil parameters with Mohr-Coulomb model:

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (undrained shear strength)	c_{u1}	[kPa]	50
Friction angle (undrained shear strength)	ϕ_u	[º]	0

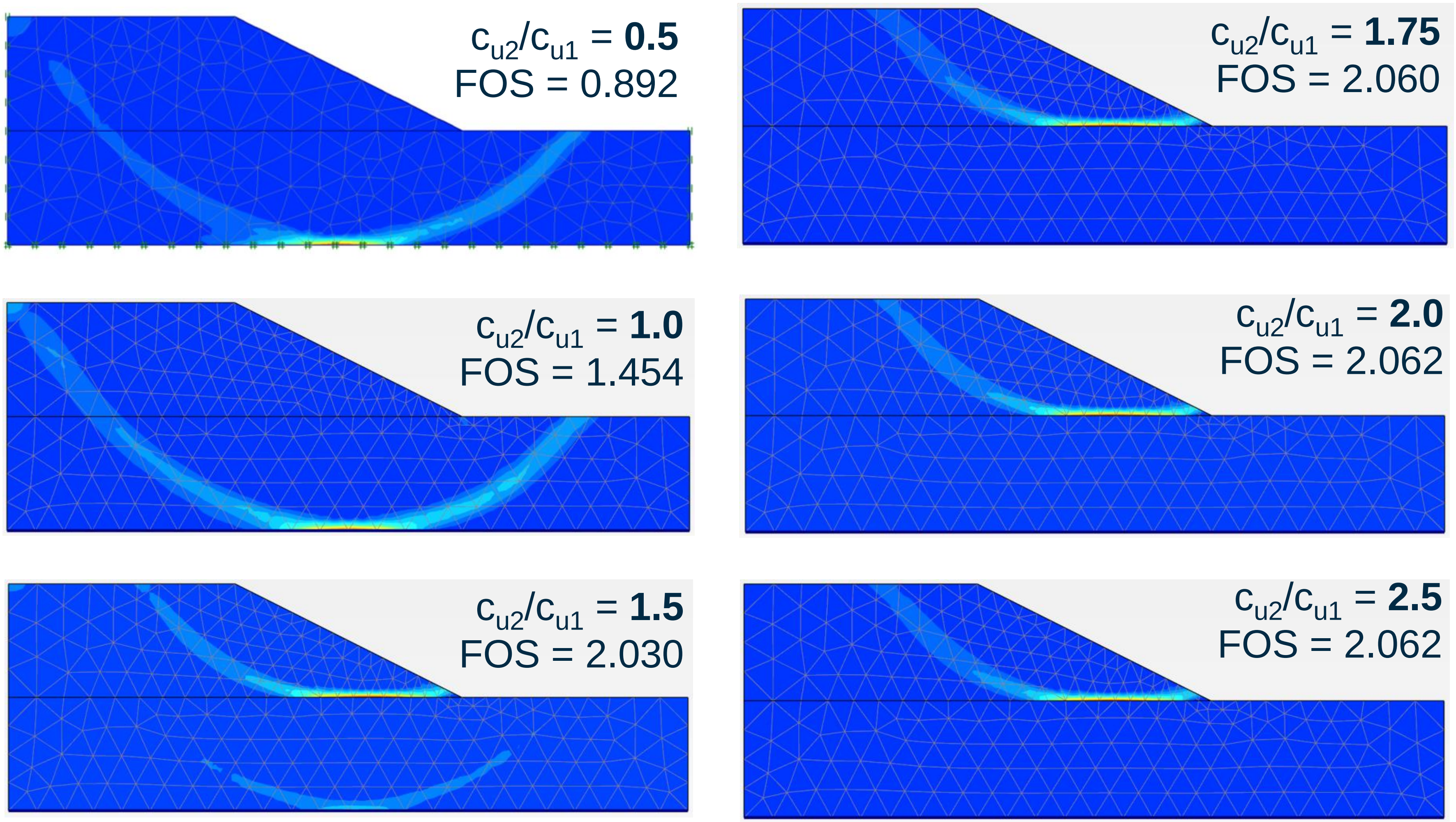
The analysis are carried out using a constant value of undrained shear strength of soil (cu_1) and six different values of undrained shear strength of the foundation layer (cu_2) with ratio cu_2/cu_1 equal to **0.5**, **1.0**, **1.5**, **1.75**, **2.0** and **2.5**.

FEM vs. LEM

(4) An Undrained Clay Slope with a Weak Foundation Layer

FEM

Failure mechanism (incremental strains):



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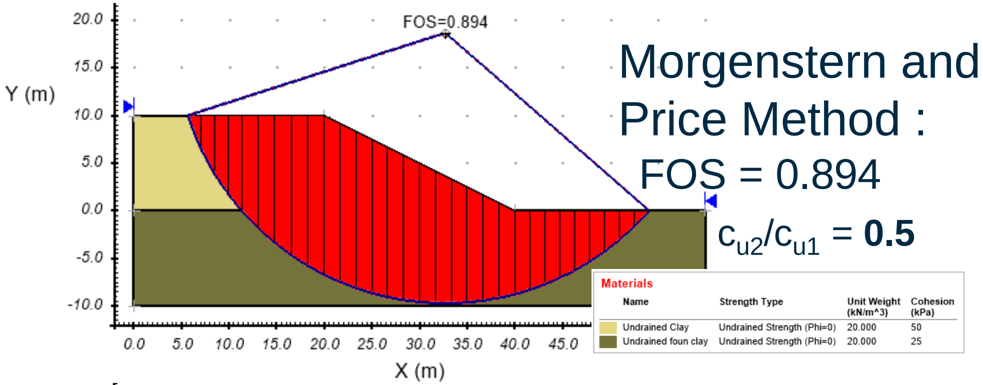
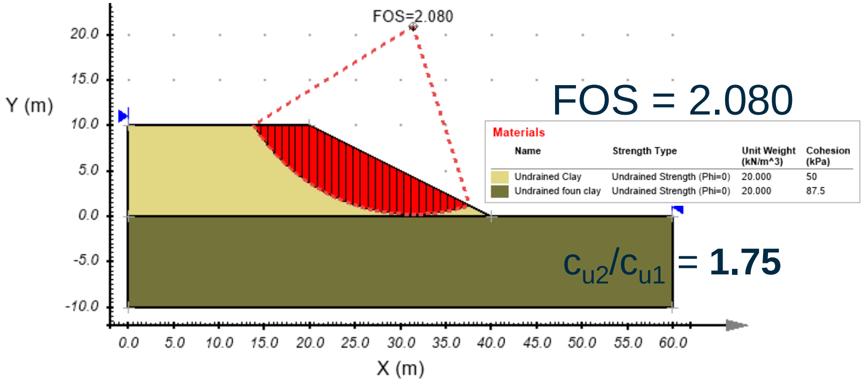
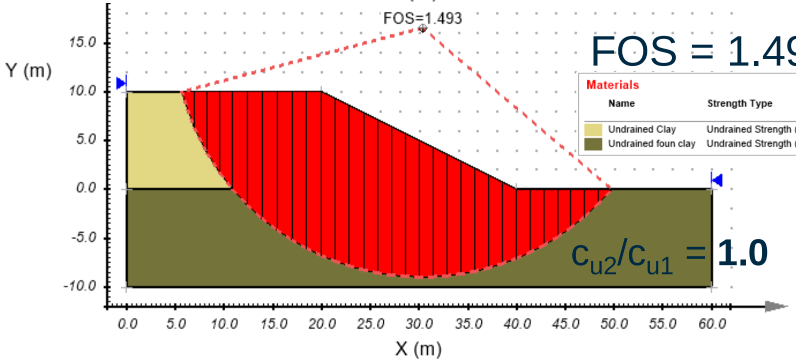
FEM vs. LEM

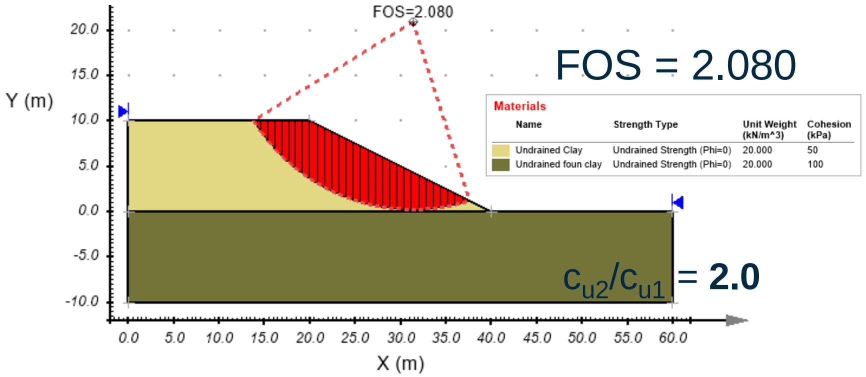
LEM

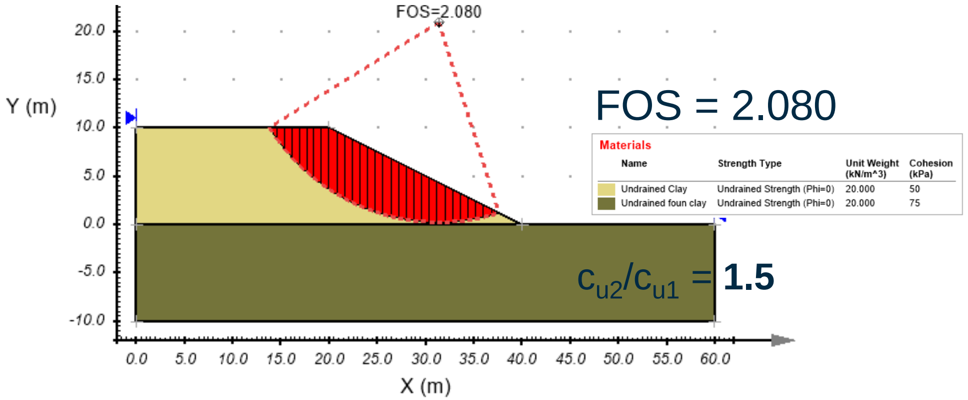
(4) An Undrained Clay Slope with a Weak Foundation Layer

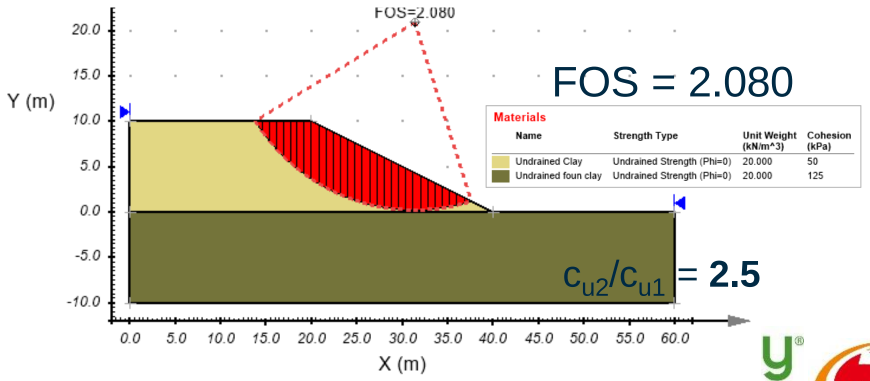












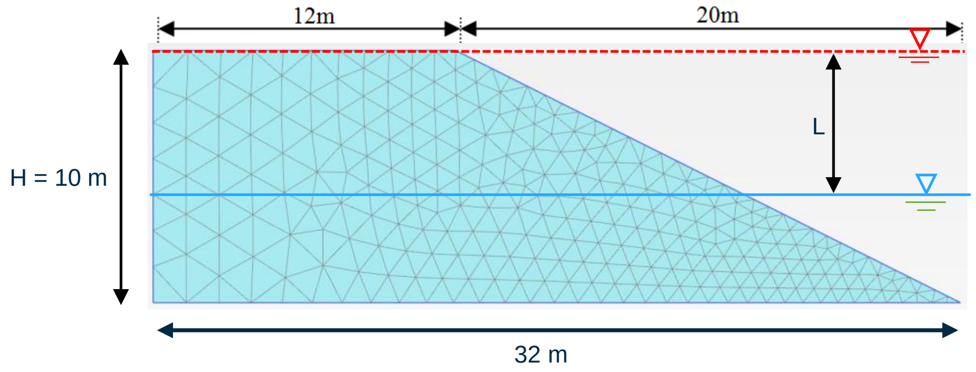
Channel Partner

FEM vs. LEM

FEM

(5) Homogeneous Slope with Horizontal Free-Surface

Geometry and mesh:



Soil parameters with Mohr-Coulomb model:

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (effective shear strength)	c'	[kPa]	10
Friction angle (effective shear strength)	ϕ'	[º]	20

In this analysis, a slope with different drawdown ratio, L/H which has been varied from 0.0 (slope completely submerged with water level at the crest of the slope) to 1.0 (water level at the toe of the slope) were considered

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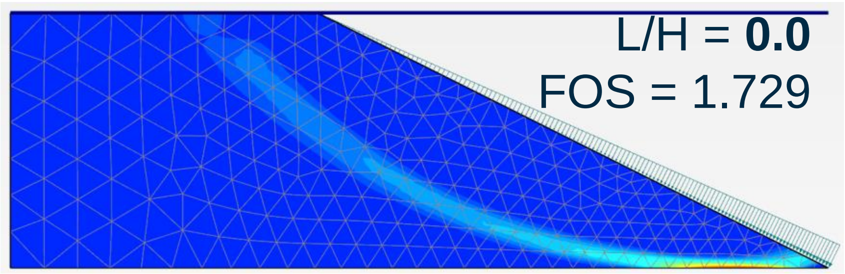
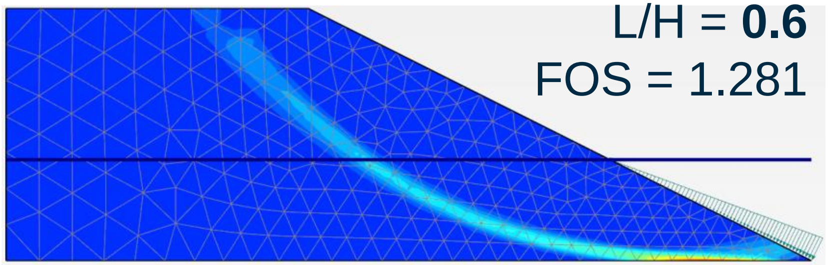
FEM vs. LEM

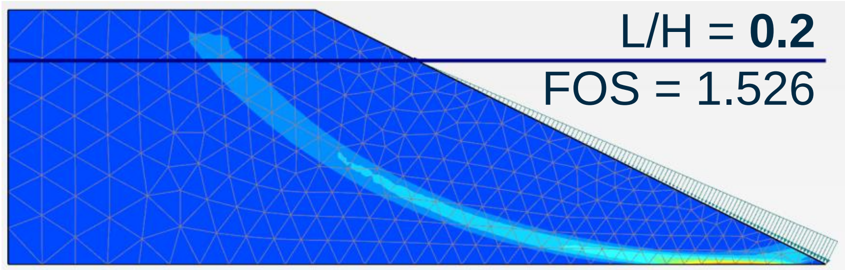
FEM

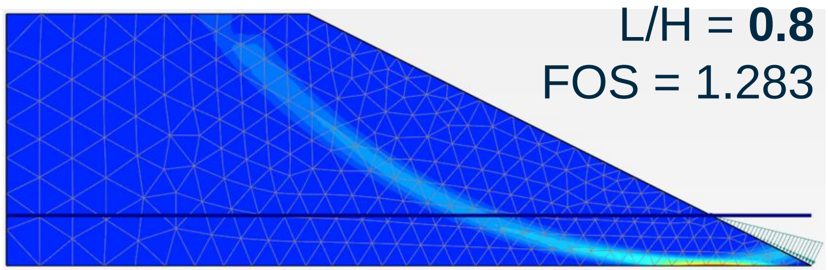
(5) Homogeneous Slope with Horizontal Free-Surface

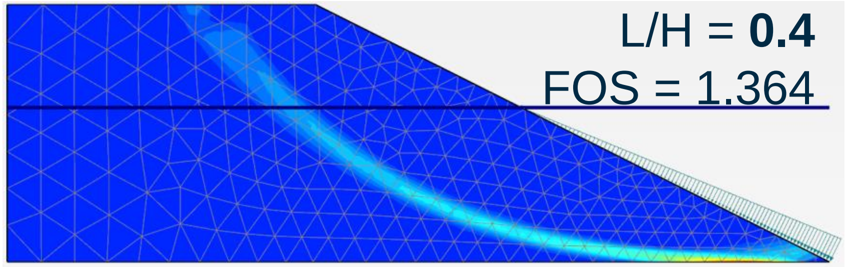
Failure mechanism (incremental strains):

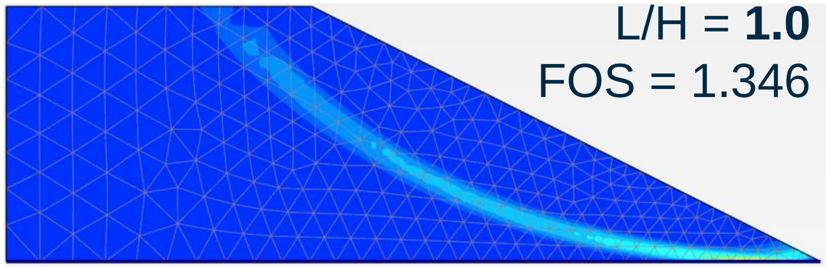












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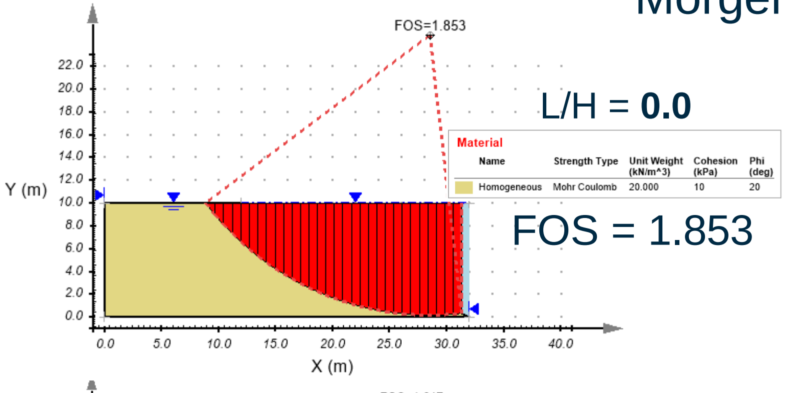
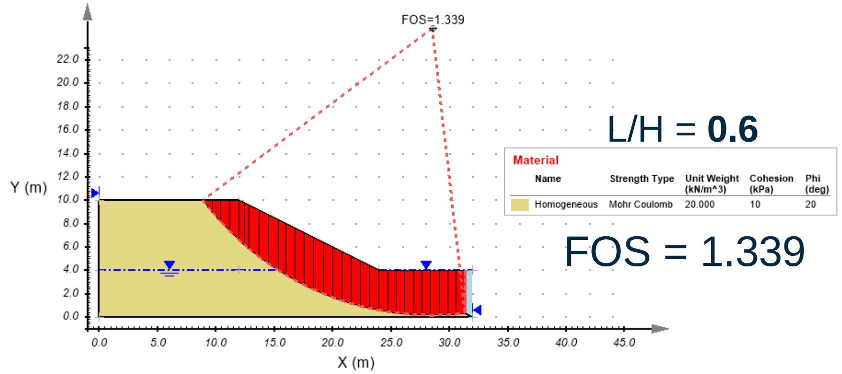
FEM vs. LEM

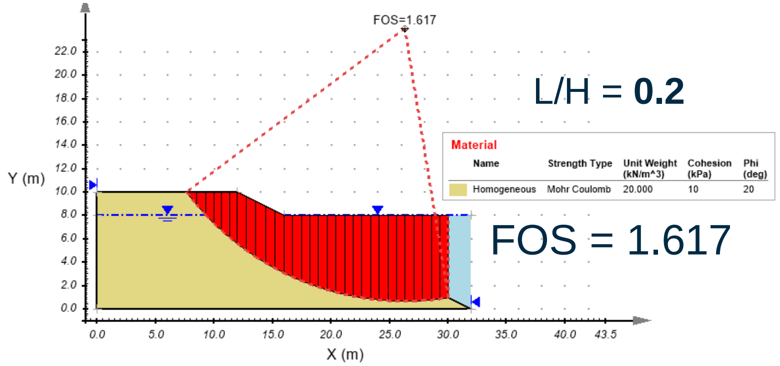
LEM

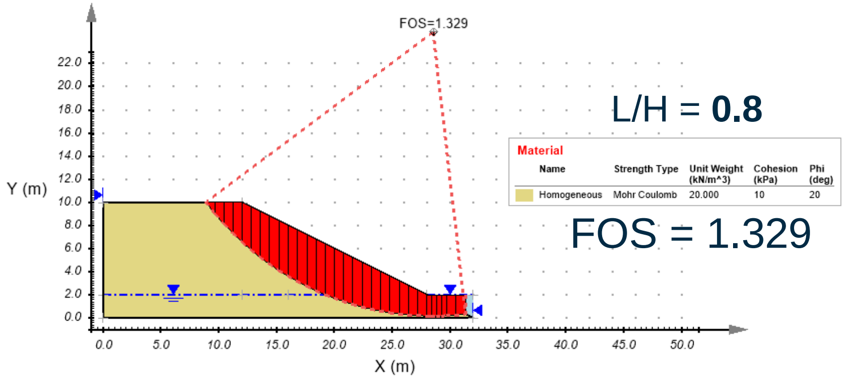
(5) Homogeneous Slope with Horizontal Free-Surface

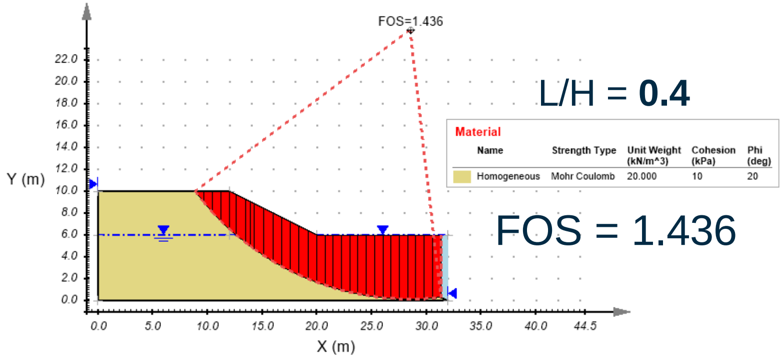
Morgenstern and Price Method :

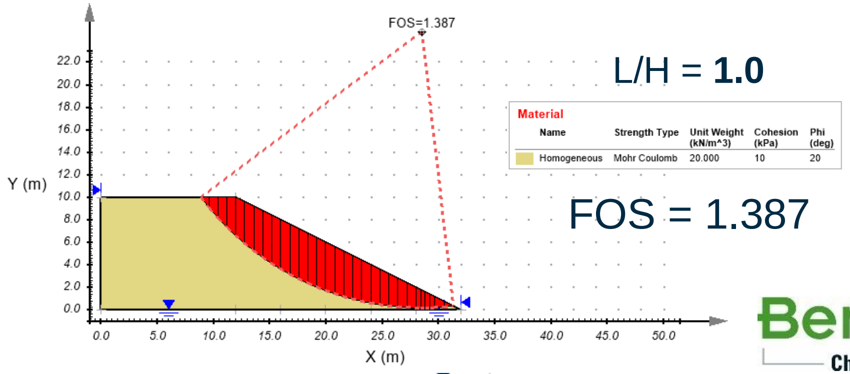












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Slope Stability Analysis using Plaxis 2D

Dr. techn. Indra Noer Hamdhan

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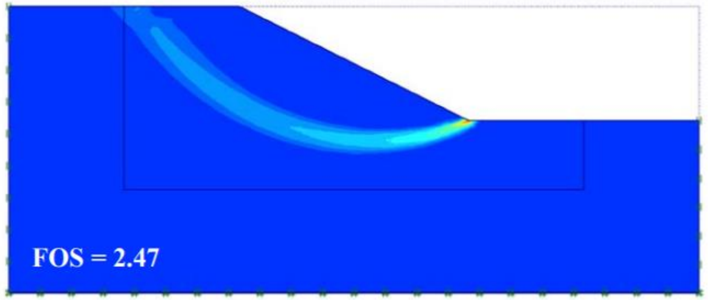
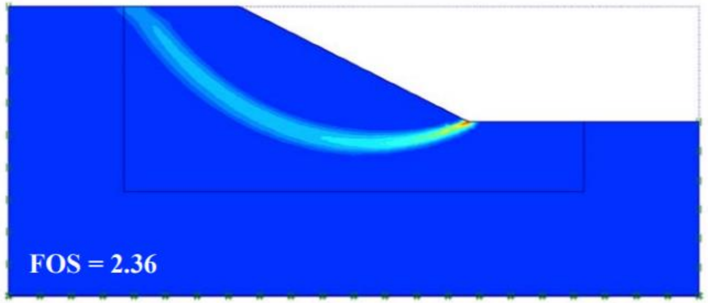
Slope stability analysis considering undrained behaviour

(Hamdhan & Schweiger, 2012)

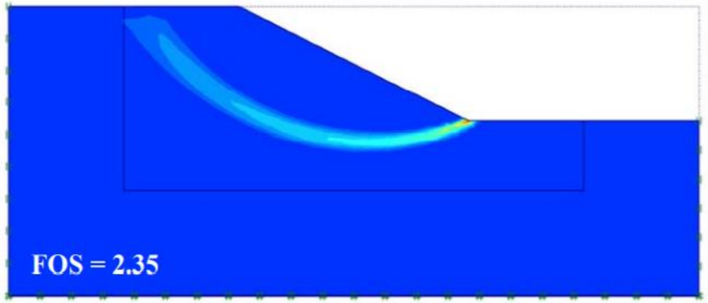
Undrained A



Undrained B



Undrained C



Soil Parameters:

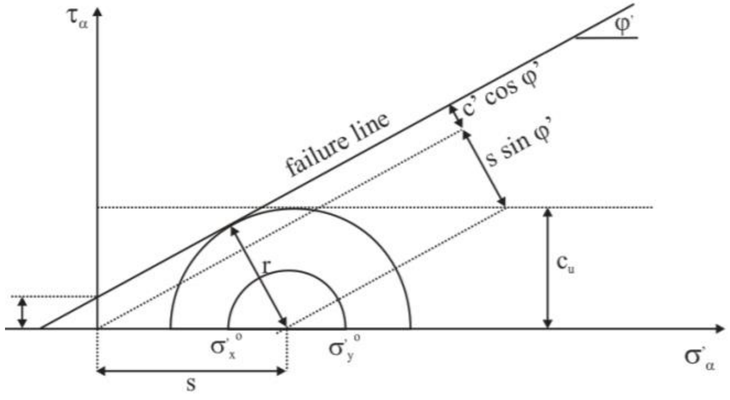
Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective young's modulus	E'	[kPa]	7500
Effective poisson's ratio	ν'	[-]	0.35
Cohesion (effective shear strength)	c'	[kPa]	20
Friction angle (effective shear strength)	ϕ'	[°]	20

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective young's modulus	E'	[kPa]	7500
Effective poisson's ratio	ν'	[-]	0.35
Cohesion (undrained shear strength)	c_u	[kPa]	18.79 *) with Δc = 5.67
Friction angle (undrained shear strength)	ϕ_u	[kPa]	0

*) Calculated from Equation 35

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective young's modulus	E_u	[kPa]	9375
Effective poisson's ratio	ν'	[-]	0.495
Cohesion (undrained shear strength)	c_u	[kPa]	18.79 *) with Δc = 5.67
Friction angle (undrained shear strength)	ϕ_u	[kPa]	0

*) Calculated from Equation 35



Mohr Circle for evaluating undrained shear strength

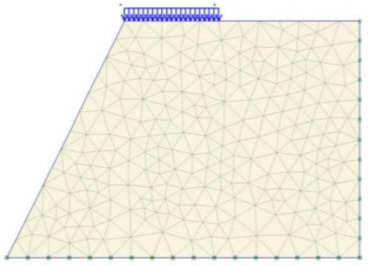
$$c_u = \tfrac{1}{2}\left(\sigma'^{o}_{xx} + \sigma'^{o}_{yy}\right)\sin\varphi' + c'\cos\varphi'$$

$$c_u = \tfrac{1}{2}\left(K_o + 1\right)\sigma'^{o}_{yy}\sin\varphi' + c'\cos\varphi'$$

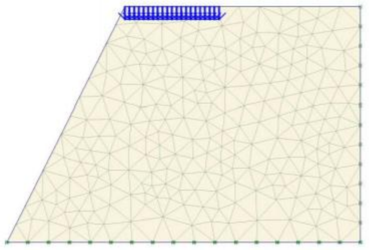
Stability analysis with FEM: strain softening vs. strength reduction approach

(Hamdhan & Schweiger, 2011)

Geometry Model:



b) Load input in MC.



a) Prescribed displacement input in ML.

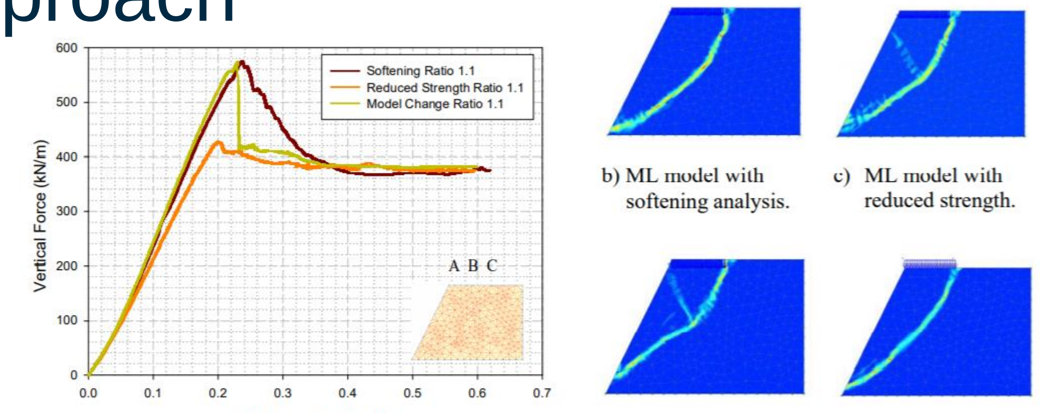
Soil Parameters:

Table 1: Input Parameters for Mohr Coulomb (MC) Model

Description	***Symbol***	**Unit**	**Value**
Unit weight	γ	kN/m^3	20
Young modulus	E	(kPa)	10000
Poisson's ratio	ν	(-)	0.35
Cohesion	c	(kPa)	30
Friction angle	φ	(°)	35

Table 2: Input Parameters for Multilaminate (ML) Model

Description	**Symbol**	**Unit**	**Value**
unit weight	γ	kN/m^3	20
Reference oedometer modulus	$E_{oed_{ref}}$	(kPa)	10000
Reference young modulus for unloading and reloading	$E_{ur_{ref}}$	(kPa)	30000
Poisson's ratio	ν'	(-)	0.25
Peak cohesion	c'_{peak}	(kPa)	30
Peak friction angle	φ'_{peak}	(°)	35
Reduced cohesion (ratio of 1.1)	$c'_{res1.1}$	(kPa)	27.27
Reduced friction angle (ratio of 1.1)	$\varphi'_{res1.1}$	(°)	32.48
Reduced cohesion (ratio of 1.3)	$c'_{res1.3}$	(kPa)	23.08
Reduced friction angle (ratio of 1.3)	$\varphi'_{res1.3}$	(°)	28.31
Softening rate parameters for cohesion	$m_{soft,\,c}$	(-)	2.75
Softening rate parameters for friction angle	$m_{soft,\,\varphi}$	(-)	0.064



a) Vertical force vs. total displacement

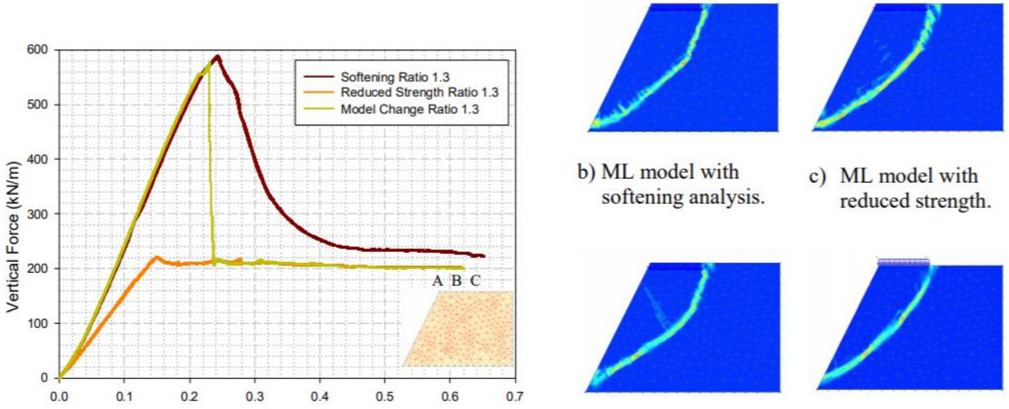
b) ML model with softening analysis.

c) ML model with reduced strength.

d) ML model with peak strength and reduction in 1 step

e) MC model with load based on softening analysis.

Figure 5: Analysis of slope stability with ratio peak / reduced strength equal to 1.1.



a) Vertical force vs. total displacement

b) ML model with softening analysis.

c) ML model with reduced strength.

d) ML model with peak strength and reduction in 1 step

e) MC model with load based on softening analysis.

Figure 6: Analysis of slope stability with ratio peak / reduced strength equal to 1.3.

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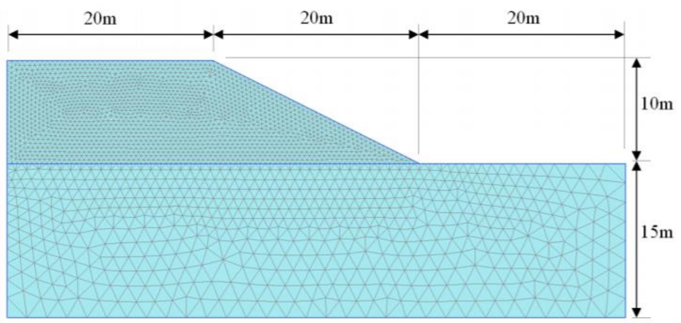
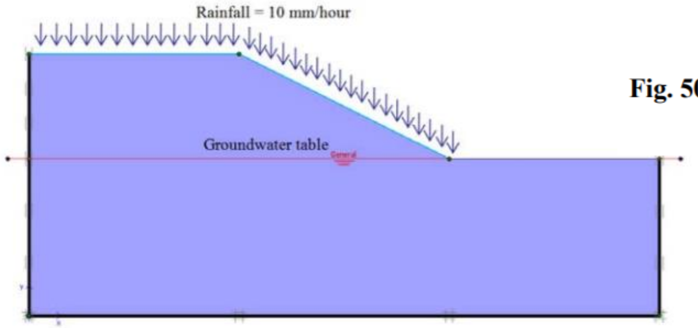
Slope Stability Analysis of Unsaturated Soil with Fully Coupled Flow-Deformation Analysis

(Hamdhan & Schweiger, 2011)

Model geometri:



Boundary Conditions:



Soil Water Characteristic Curve:

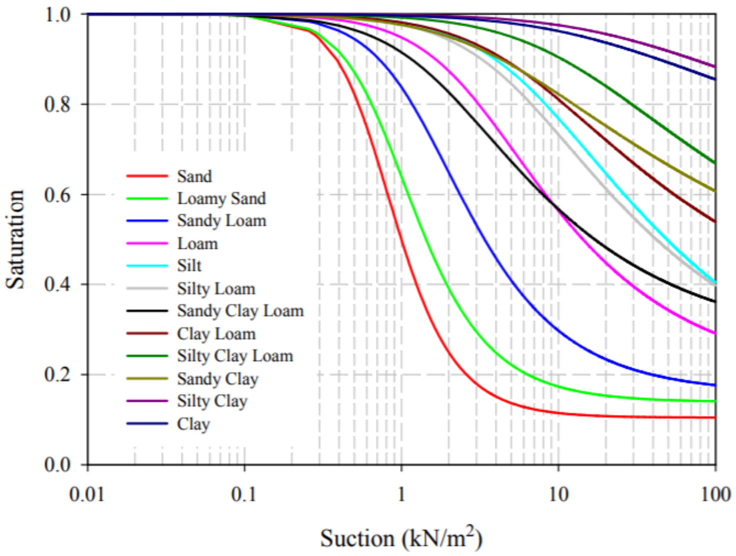


Fig. 50: Soil Water Characteristic Curves (SWCC) for USDA series with Van Genuchten Models

Initial Conditions:

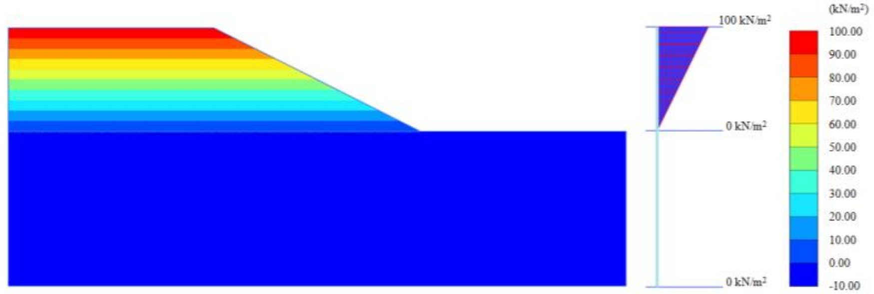


Fig. 68: Initial conditions: suction

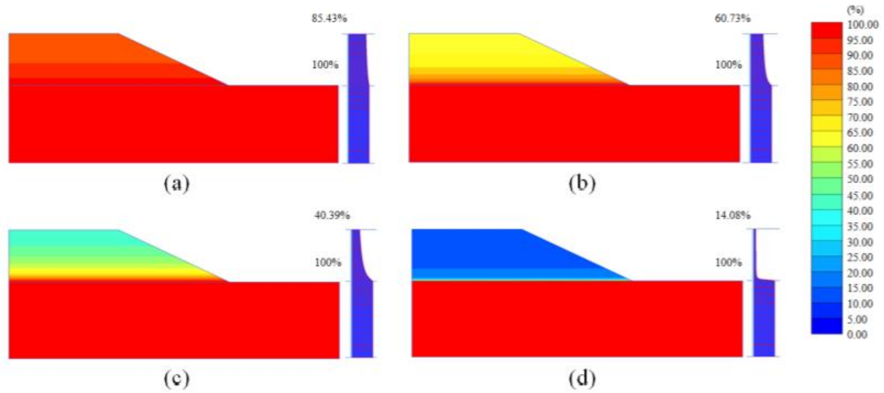


Fig. 69: Initial conditions: degree of saturation (a) Clay, (b) Sandy Clay, (c) Silt, and (d) Loamy Sand

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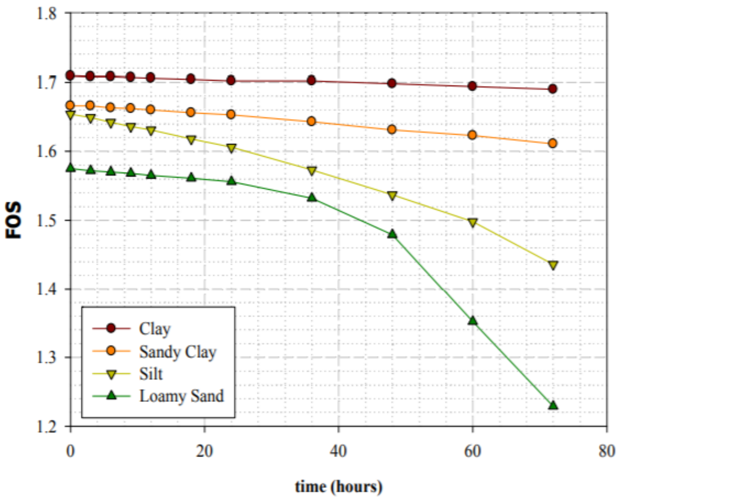
Slope Stability Analysis of Unsaturated Soil with Fully Coupled Flow-Deformation Analysis

(Hamdhan & Schweiger, 2011)



Change of FOS with time for unsaturated soil slope subjected to rain infiltration

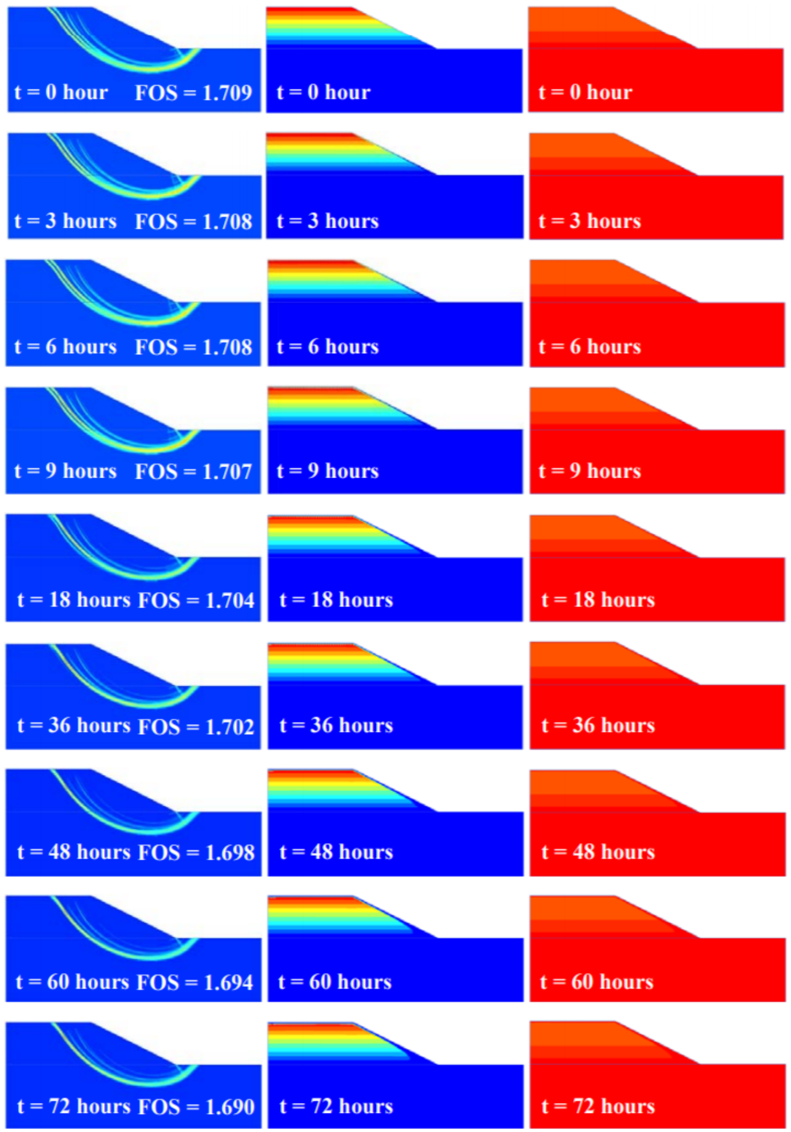


Fig. 71: Failure mechanism, change of suction and saturation during infiltration for clay slopes

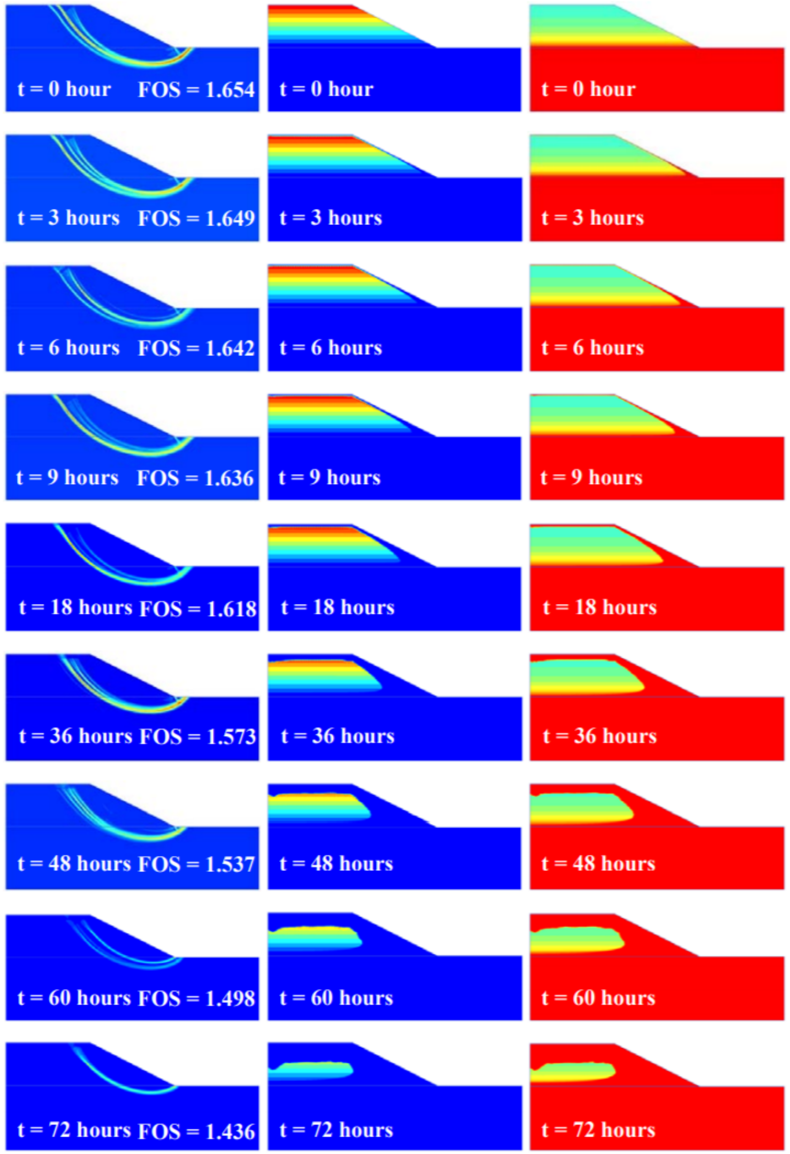


Fig. 73: Failure mechanism, change of suction and saturation during infiltration for silt slopes

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Analisis Stabilitas Lereng pada Perkuatan Lereng dengan Dinding Mechanically Stabilised Earth (MSE Wall)

(N.H., Risnanda & Hamdhan, 2019)

Tabel 4. Parameter Dinding MSE

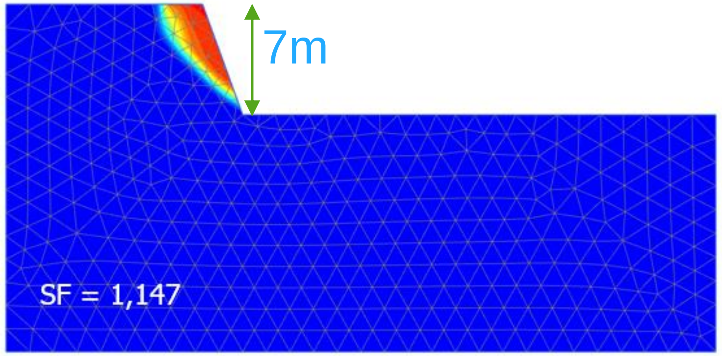
Keterangan	Material Type	*Normal Stiffness* EA [kN/m]	*Flexural Rigidity* EI [kN/m²/m]	*Eqivalent Thickness* d [m]	*Weight* w [kN/m/m]	*Poisson's Ratio*	*Model*
Concrete Panel Facing	*Elastic*	4,2 x 10^7	78.500	0,15	3,6	0,15	*Plate*

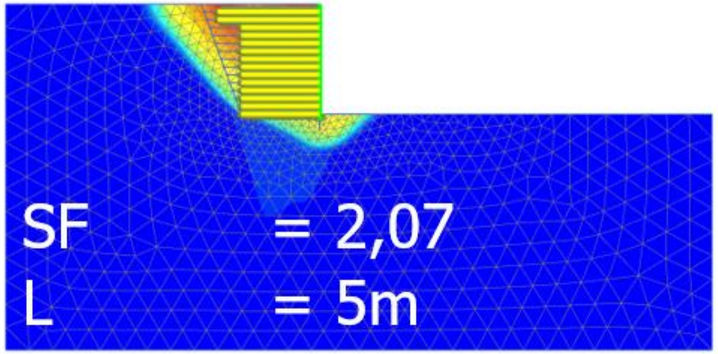
Tabel 3. Parameter Tanah Kondisi Eksisting dan Parameter Tanah Timbunan (*Backfill*)

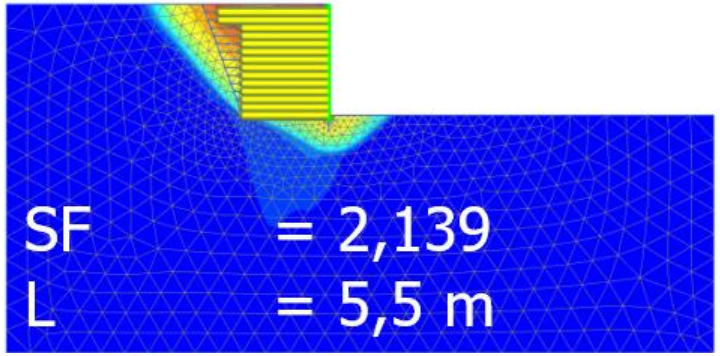
Keterangan	γ sat [N/m³]	γ unsat [kN/m³]	C' [kN/m³]	Φ' [°]	φ [°]	E [kN/m²]	ν
Tanah Eksisting	16	17	11	37	7	10000	0,3
Tanah Timbunan	21	22,7	14	39	9	20000	0,3

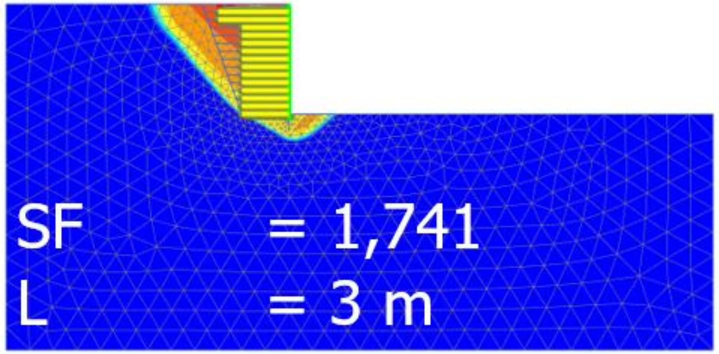
Tabel 5. Parameter Geogrid

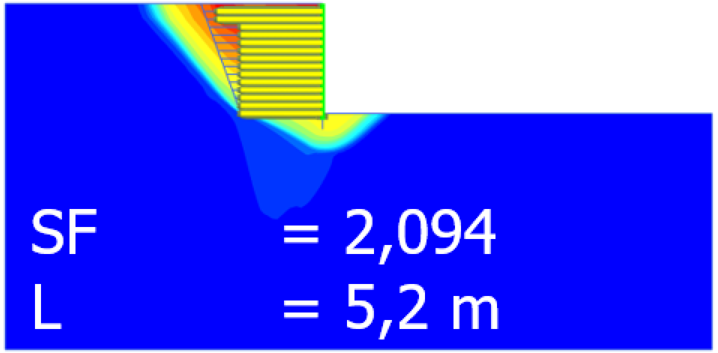
Keterangan	Material Type	*Tensile Modulus @2% Strain* [kN/m]	*Elongation at Break* [%]	*Normal Stiffness* EA [kN/m]	R_{inter}
Mirafi HP380a	*Elastic*	500	10	935,28	0,77

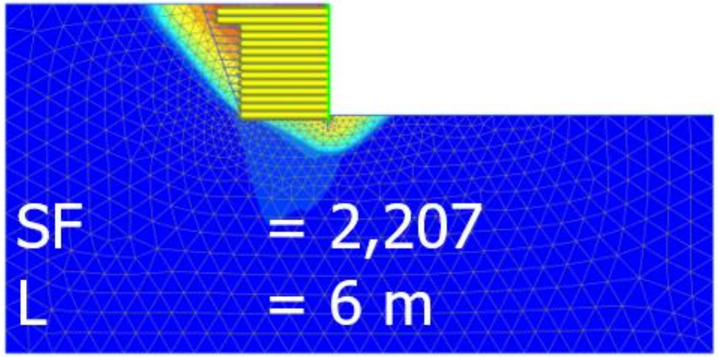












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Modeling Slope Stability for a Road Construction Project

Dr. techn. Indra Noer Hamdhan

Bentley Channel Partner

National Geotechnics Center

Bentley Institute

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Modeling Slope Stability for a Road Construction Project

On the North Island of New Zealand a new road section has to be constructed along the shoreline of a tidal bay, see figure 1.

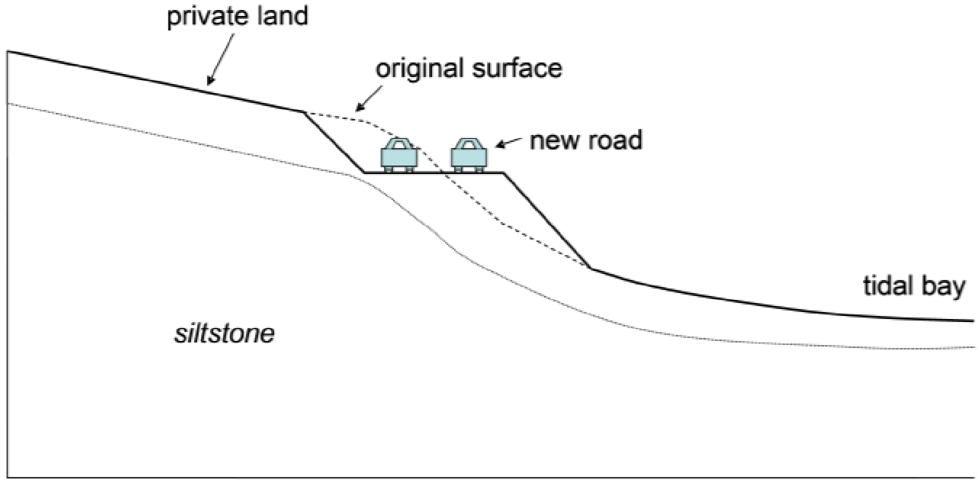

Figure 1: Situation overview for the newly constructed road

The new road therefore had to be constructed along the steeper gradient just next to the shoreline of the tidal bay. The hillside is mainly siltstone, weathered at the surface but intact at certain depth. Construction will take place in summer when the ground water level is low. However, in winter the hillside side almost fully saturates due to heavy rainfall, which has a significant influence on the stability.

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Modeling Slope Stability for a Road Construction Project

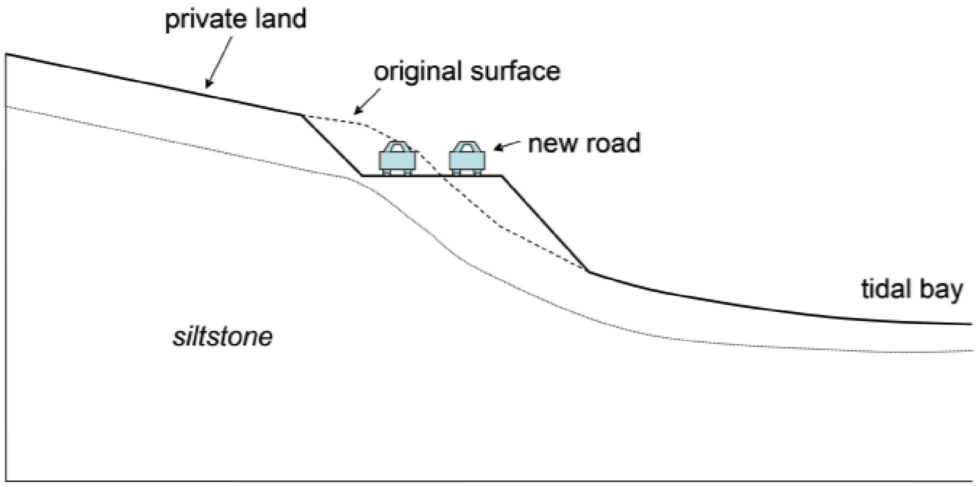

Figure 1: Situation overview for the newly constructed road

For the construction of the new road part of the slope was excavated. The excavated material is crushed and mixed with sand and gravel to make fill material to support the road. During the first winter after the road construction the road started to tilt towards the tidal bay and after assessing the winter situation the factor of safety was considered too low. The decision was taken to stabilize the fill and hillside below the road using so-called launched soil nails: long steel reinforcement bars that are shot with high speed into the ground.

Modeling Slope Stability for a Road Construction Project

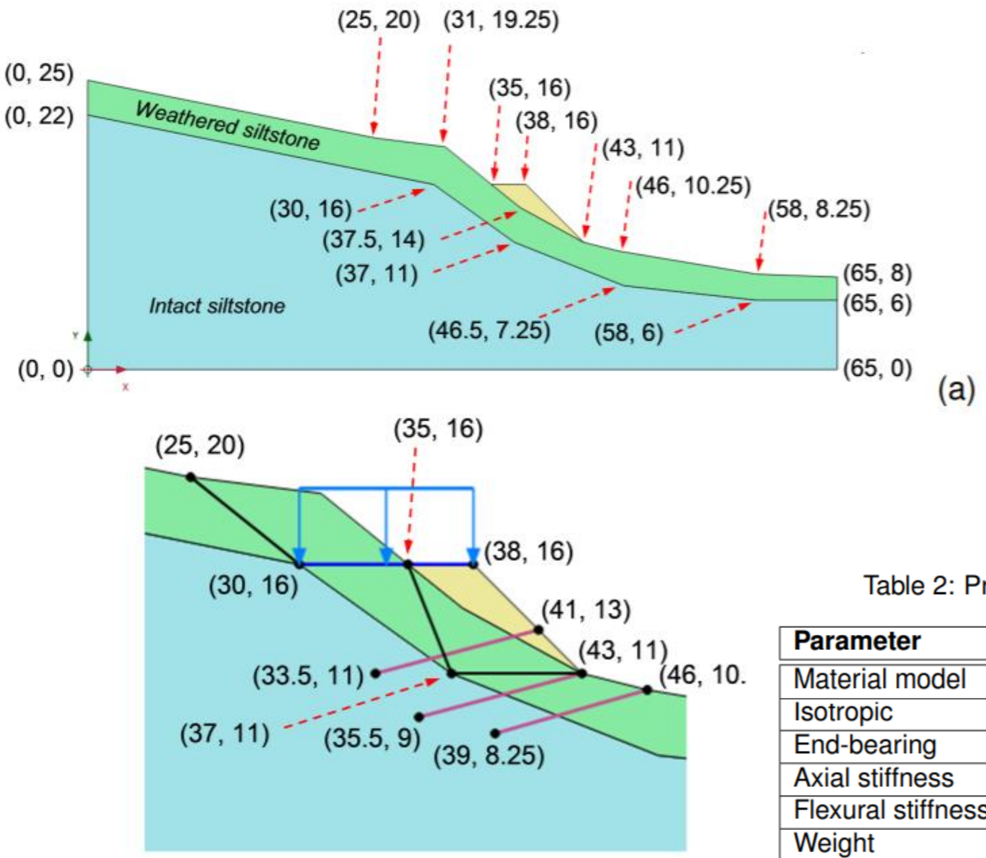

Table 1: Soil material set parameters

Parameter	Symbol	*Intact siltstone*	*Weathered siltstone*	*Reinforced fill*	Units
Material model	Model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	
Type of behaviour	Type	Drained	Drained	Drained	
Dry weight	γ_{unsat}	16.0	16.0	19.0	kN/m^3
Wet weight	γ_{sat}	17.0	17.0	21.0	kN/m^3
Young's modulus	E'	12000	12000	20000	kN/m^2
Poisson's ratio	υ'	0.3	0.3	0.3	–
Cohesion	c'_{ref}	12	10	8	kN/m^2
Friction angle	φ'	35	19	30	$^\circ$
Dilatancy angle	ψ	0	0	0	$^\circ$
Permeabilities	k_x, k_y	$1{\cdot}10^{-3}$	0.01	0.1	m/d
Tension cut-off	Tension cut-off	Disabled	Enabled	Enabled	

Table 2: Properties of the road surface (plate)

Parameter	Symbol	*Road surface*	Unit
Material model	Model	Elastic	–
Isotropic		Yes	–
End-bearing		No	–
Axial stiffness	EA_1, EA_2	$2.5{\cdot}10^5$	kN/m
Flexural stiffness	EI	500	kNm^2/m
Weight	w	3.0	$kN/m/m$
Poisson's ratio	ν	0.0	–

Table 3: Properties of the soil nails (embedded pile rows)

Parameter	Symbol	*Grout body*	Unit
Modulus of elasticity	E	$2.1*10^8$	kN/m^2
Material weight	γ	60	kN/m^3
Pile type	Pile type	Predefined	-
Predefined pile type	Predefined pile type	Massive circular pile	-
Diameter	Diameter	0.032	m
Spacing	$L_{spacing}$	1.0	m
Skin resistance	$T_{top,max}, T_{bot,max}$	1000	kN/m
Base resistance	F_{max}	0	kN
Interface stiffness factor		Default values	-

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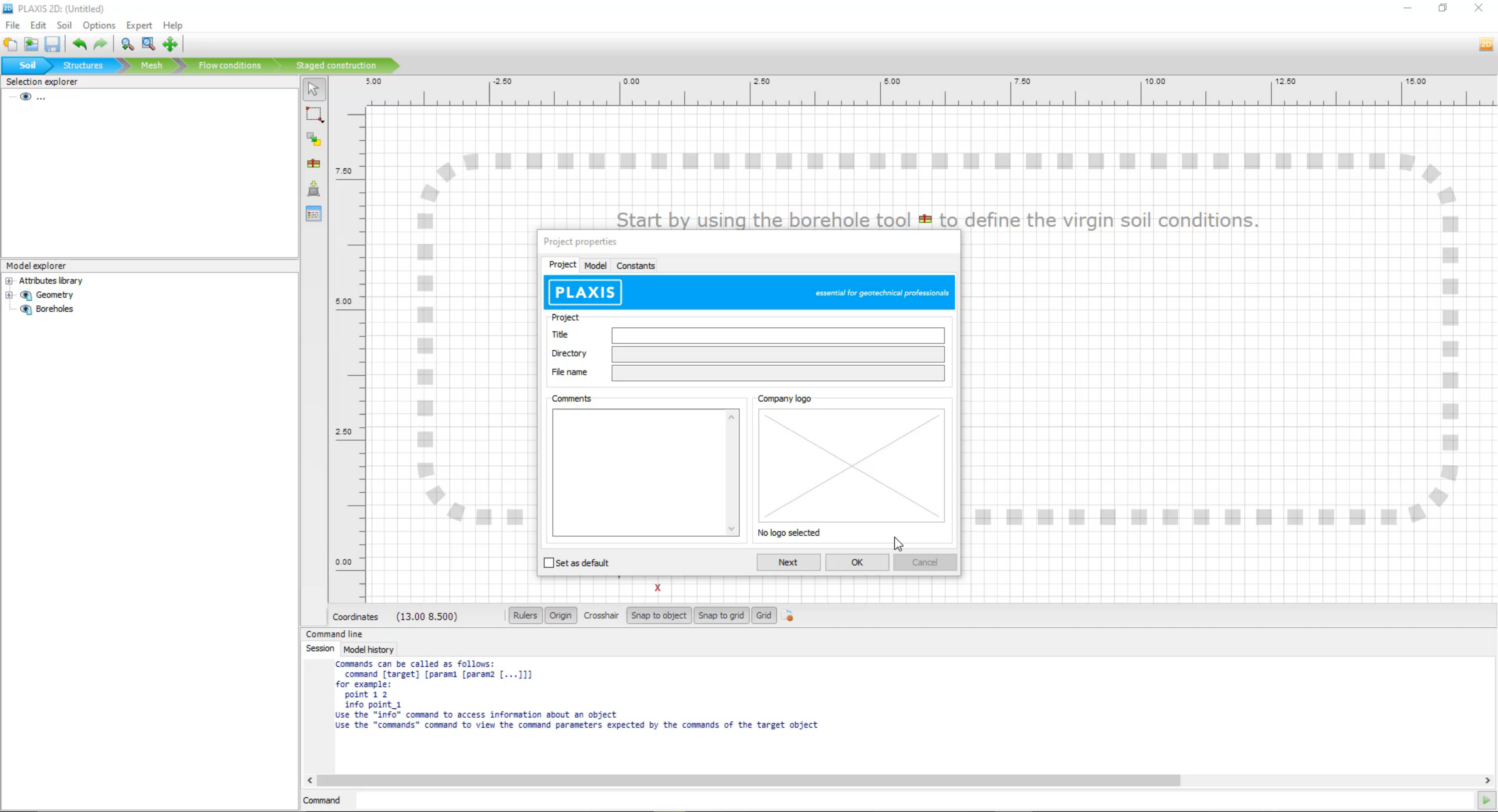
PLAXIS 2D: (Untitled)
File Edit Soil Options Expert Help
Soil
Structures
Mesh
Flow conditions
Staged construction
Selection explorer
Model explorer
Attributes library
Geometry
Boreholes
Start by using the borehole tool to define the virgin soil conditions.
Project properties
Project
Model
Constants
PLAXIS
essential for geotechnical professionals
Project
Title
Directory
File name
Comments
Company logo
No logo selected
Set as default
Next
OK
Cancel
Coordinates
(13.00 8.500)
Rulers
Origin
Crosshair
Snap to object
Snap to grid
Grid
Command line
Session
Model history
Commands can be called as follows:
command [target] [param1 [param2 [...]]]
for example:
point 1 2
info point_1
Use the "info" command to access information about an object
Use the "commands" command to view the command parameters expected by the commands of the target object
Command

Terima kasih

Dr. techn. Indra Noer Hamdhan

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Modelling of Ground Improvement (PVD) with PLAXIS 2D

Ikhya, ST., MT., MM.

Civil Engineering Department
National Institute of Technology (Itenas)
Bandung, Indonesia

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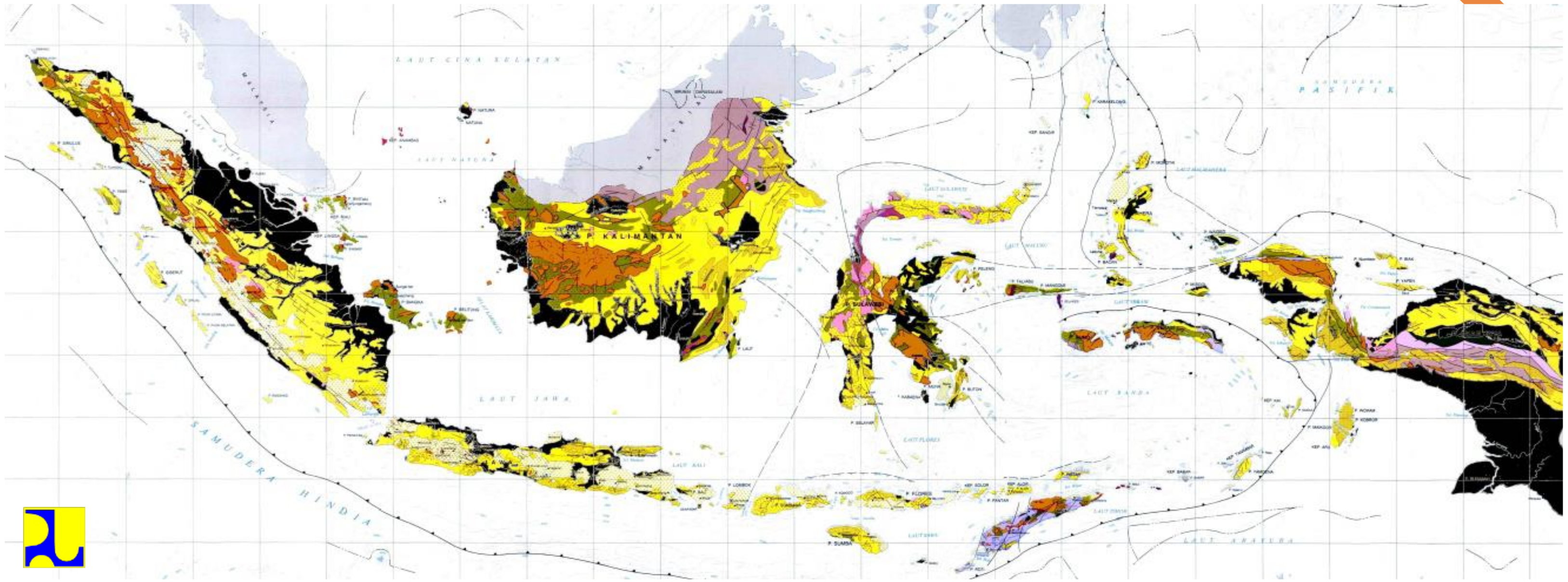
Soft Soil and Soft Soil in Indonesia



- Soft soil deposits (<u>soft clays, organic soils, and peats</u>) are normally characterized by low shear strength, high compressibility and low coefficient of permeability. Characteristics which make them a difficult soil for engineering .
- Soft Soil deposits can be found throughtout world and are also prevailing in many parts of Indonesia.
- Indonesia possesses one of the longest coastal-line in the world with many of them having soft soil deposits.
- The area of soft soil deposits in Indonesia is around 30% of Indonesia's total land area (60 million-ha with 20 million-ha is peats) with varying depth (black area and some white area on the map).

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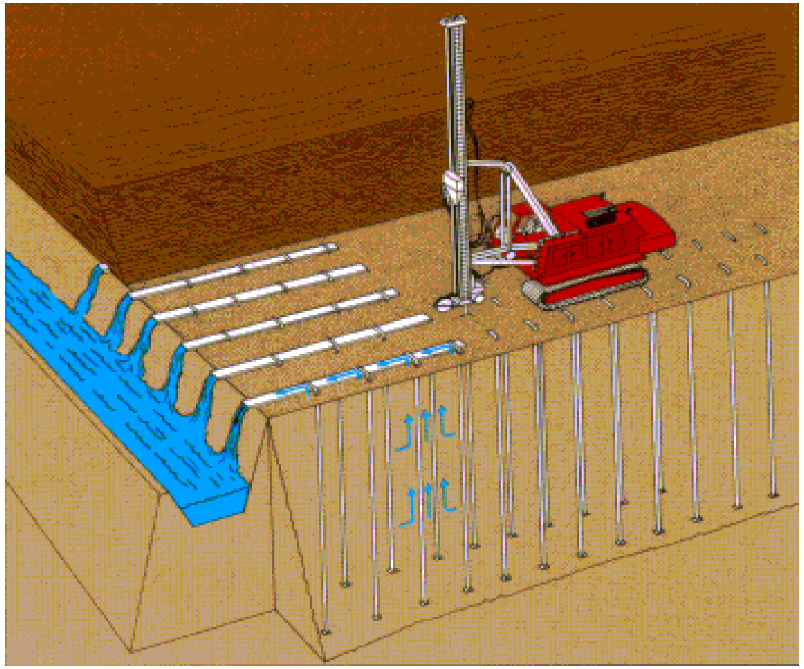
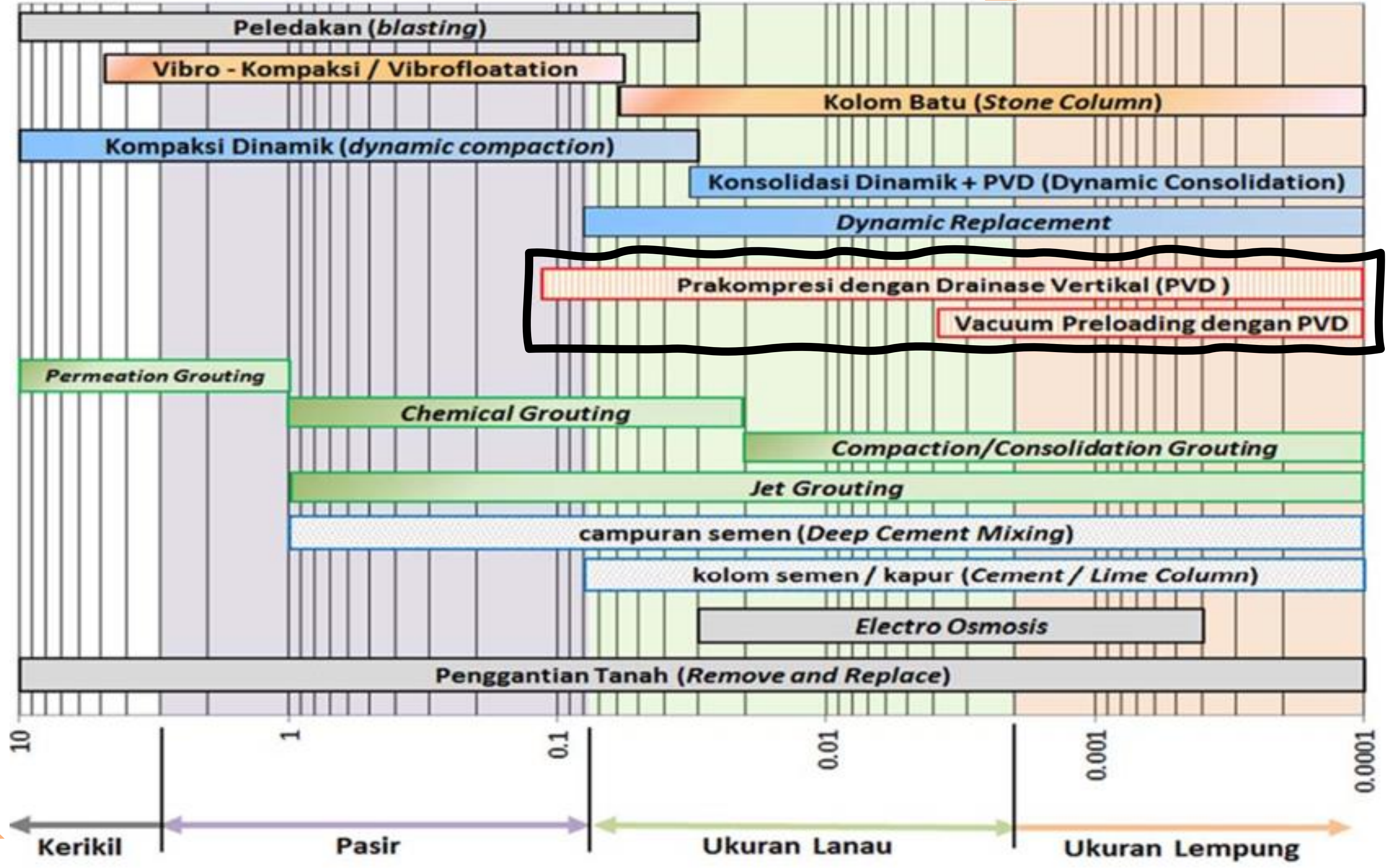
Ground Improvement and Vertical Drains

- Ground improvement techniques play an improtant role in extending the infrastructure across the country in difficult soils.
- Preloading and vertical drains, vibro compaction and replacement, in situ soil mixing, grouting, dynamic compaction, piles, geosynthetics, electro-osmotic, etc, are some ground improvement techniques.
- The selection of the correct ground improvement technique can have an important effect on the foundation choice and can often lead to more economical solutions when compared to traditional approaches.

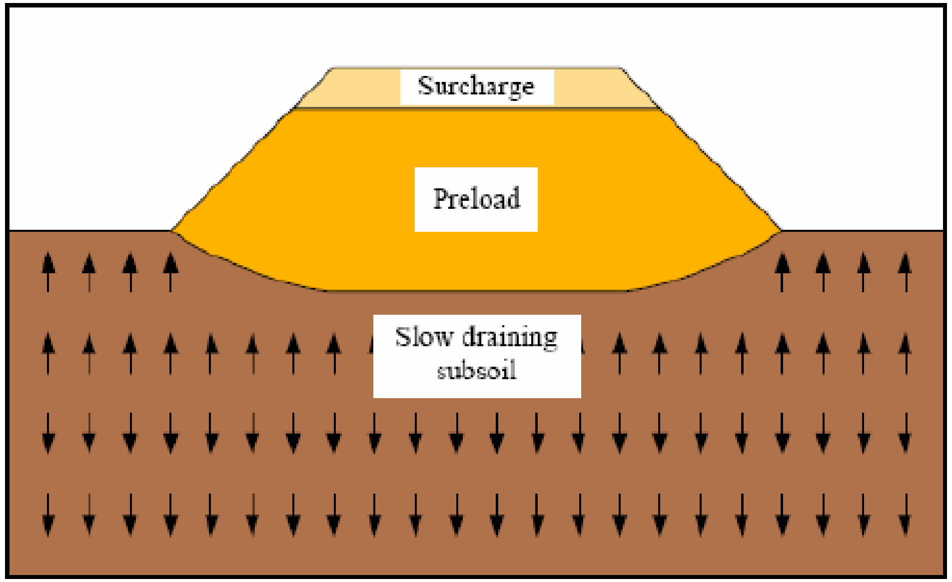


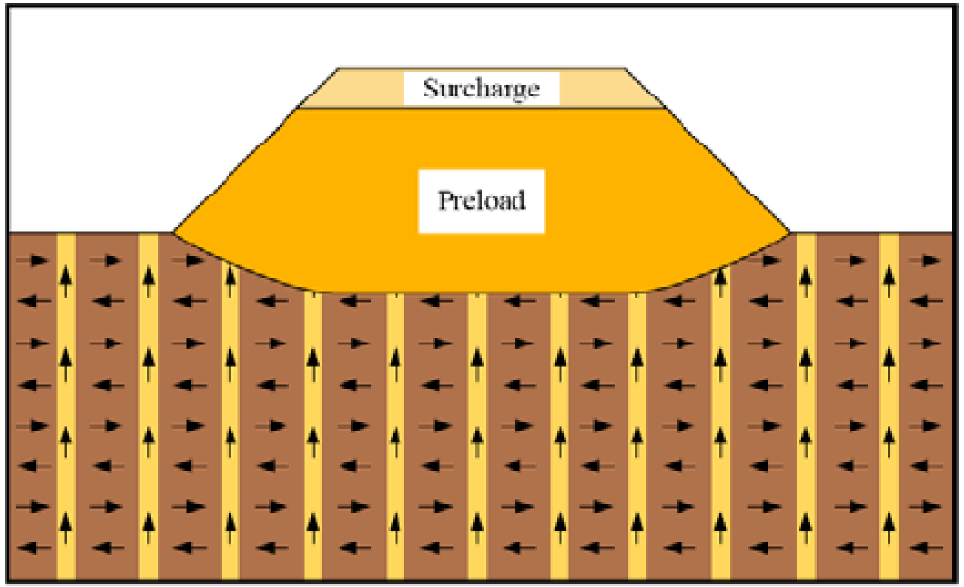
- Vertical drains combined with preloading have become common practice and are among the most effective procedures for ground improvement accelerating the consolidation process (decrease the time for reaching final settlements).
- The installation of vertical drains reduces the drainage path and speeds up the dissipation of excess pore water pressure generated during the application of surcharge loads in saturated fine grained soils.

Ground Improvement and Vertical Drains

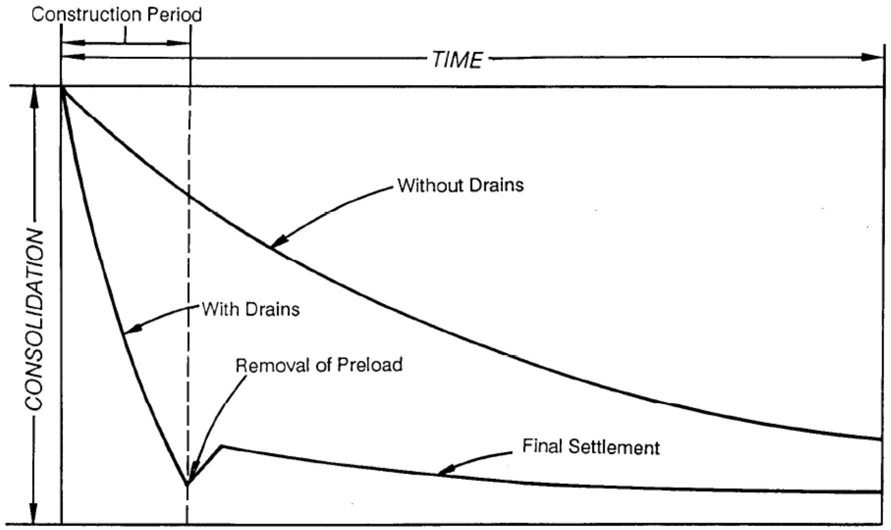


Vertical Drains

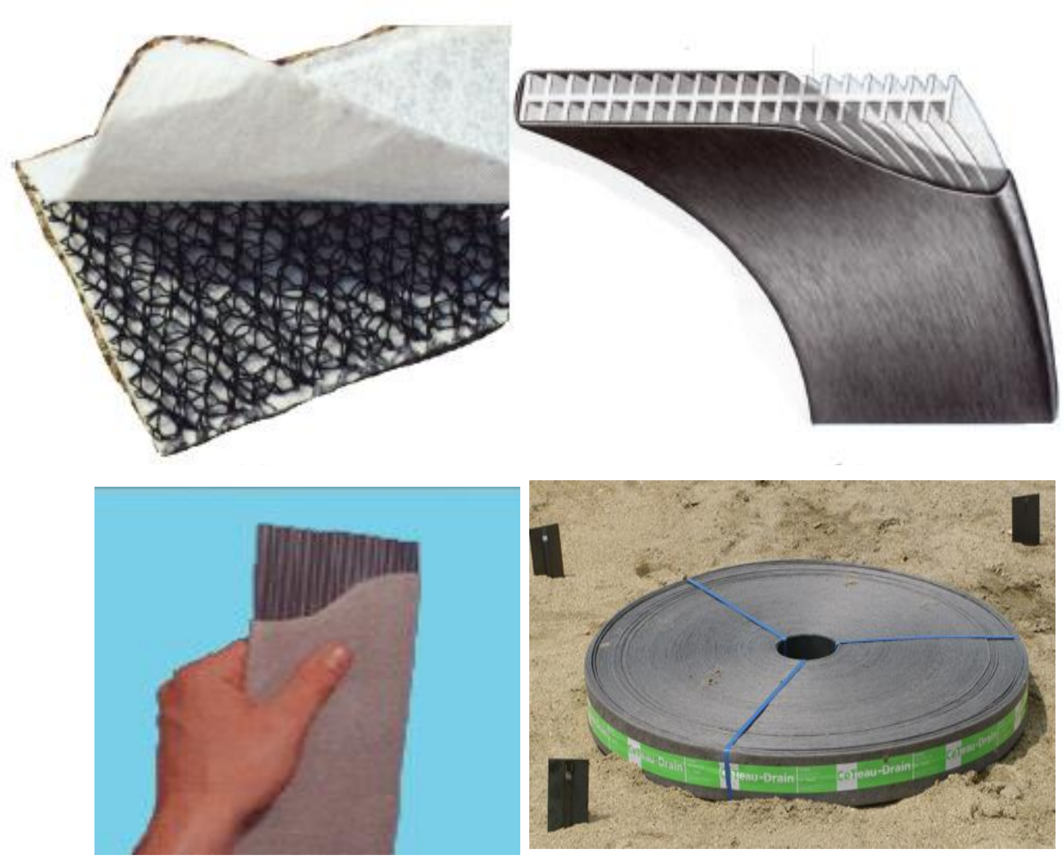


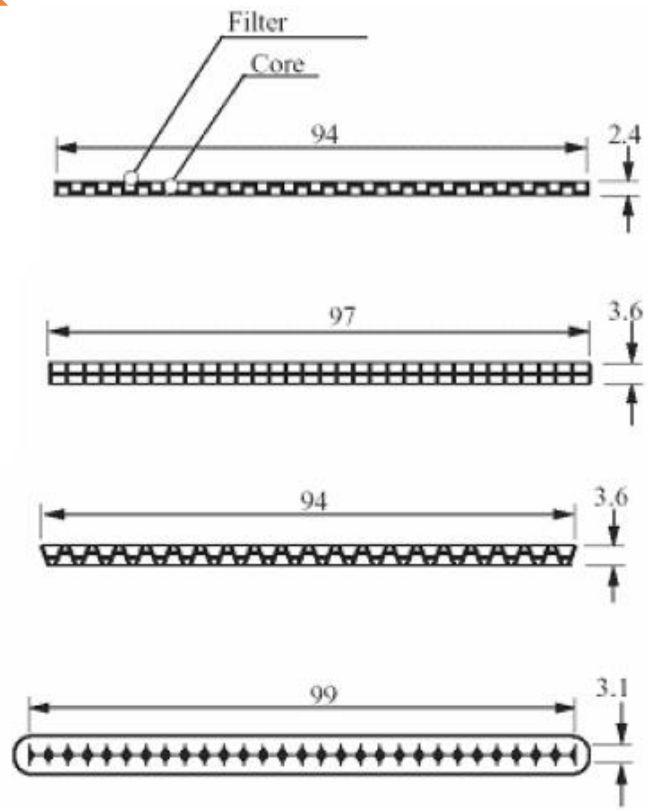


- The installation of vertical drains provides a shorter drainage path and a faster dissipation of excess pore water, thereby resulting in faster development of settlements and a quicker gain of strength due to consolidation.



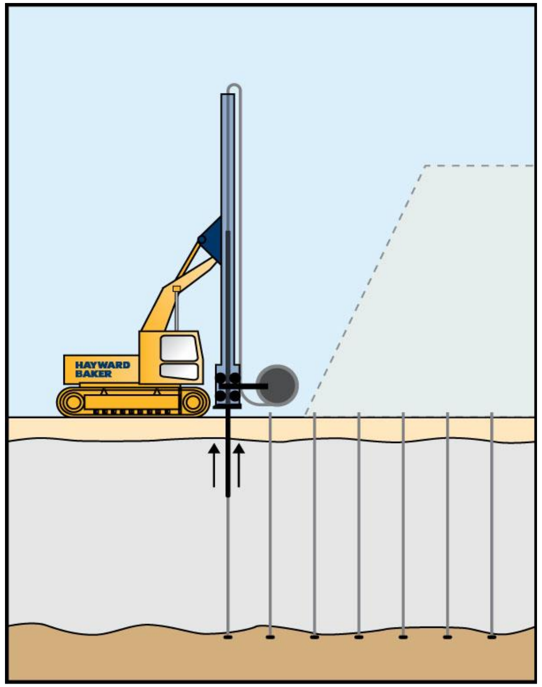
Prefabricated Vertical Drains (PVD)

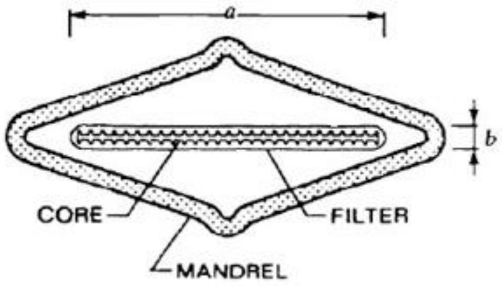




- Prefabricated vertical drains (PVD) are typically 10 cm wide, 3 to 9 mm thick, and packed in rolls.
- The drain consists of a geotextile filter wrapped on a plastic strip with molded channels that allow water to travel to the ground surface, relieving excess subsurface pore water pressure.
- The geotextile filter prevents soil particles from entering the channels and clogging the drain.

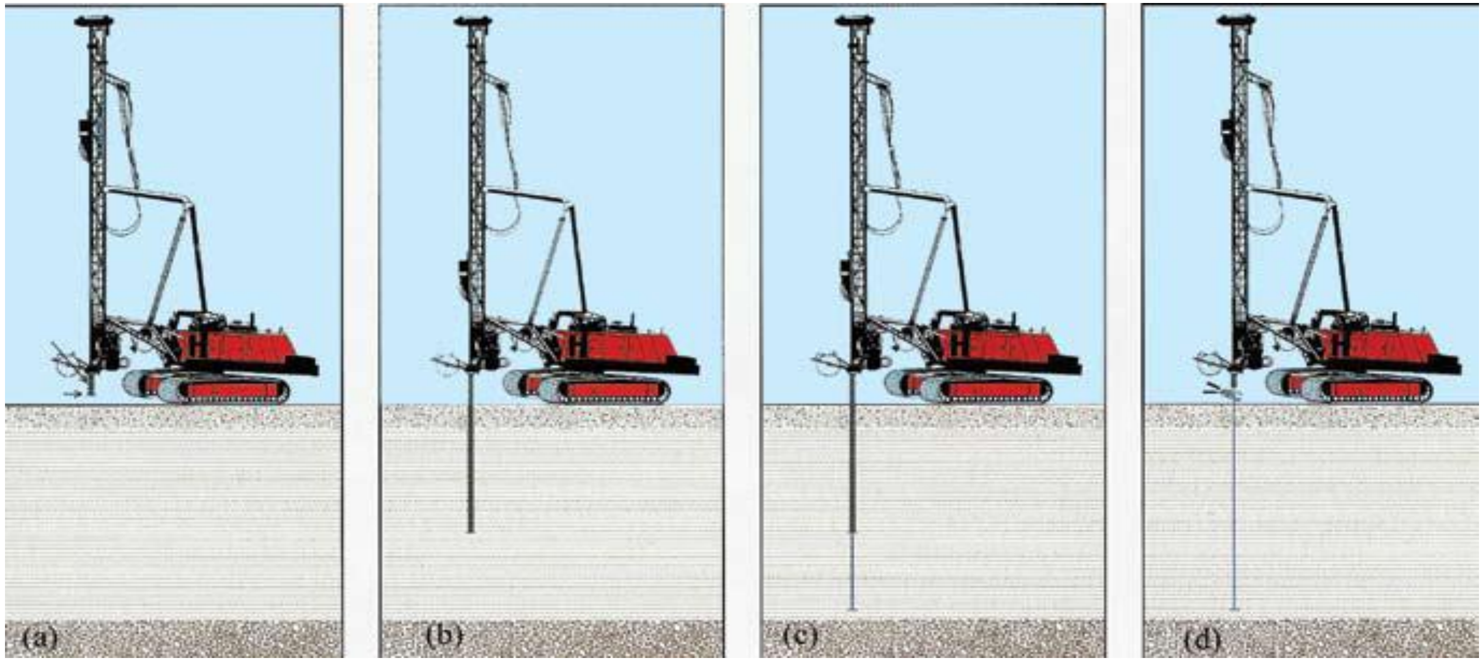
Installation of Prefabricated Vertical Drains (PVD)





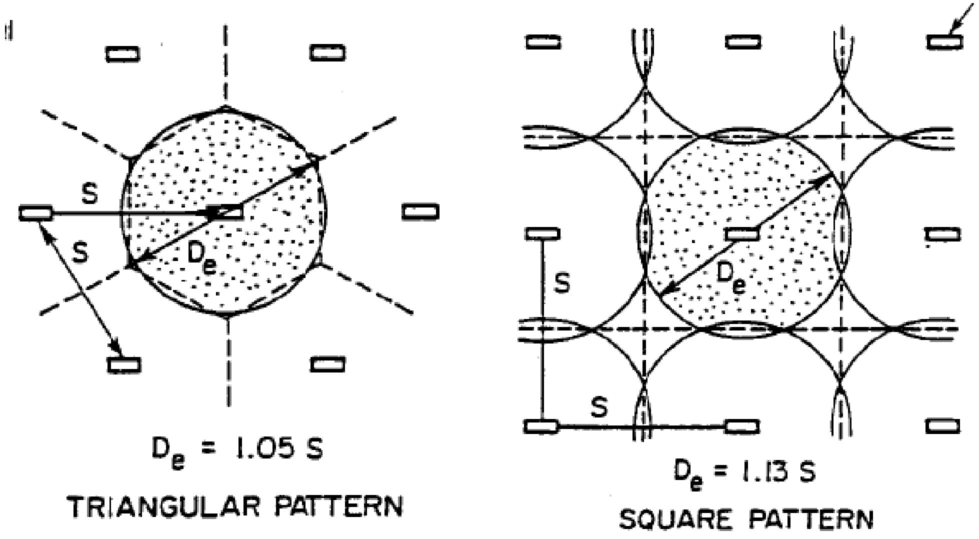
- The installation equipment consists of the drain spools, mandrel, anchor and PVD delivery route excavator or crane.
- The mandrel cross-sectional area is typically 60 - 80 square cm.
- The installation force is typically provided by vibratory methods, static methods or a combination of these methods depending on the soil conditions.
- The majority of depths for PVD applications are between 5 - 40 m, however, drains can be installed up to 60 m.

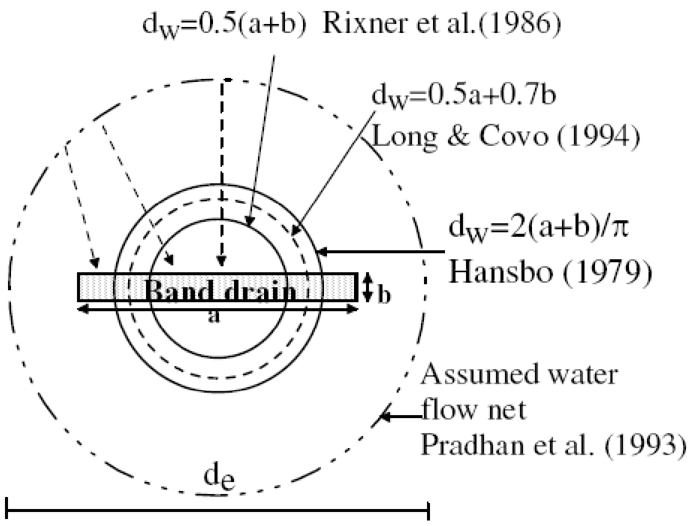
Installation of Prefabricated Vertical Drains (PVD)



- The wick drain is threaded through the mandrel which protects it from damage as it is driven through the soil.
- After reaching the target depth, the drain is anchored to keep it in place while the mandrel is withdrawn.
- The drain is then cut approximately 10 to 30 cm above ground, and a new anchor fastened to the wick at the bottom of the mandrel in preparation for the next installation point.
- The drains layout usually consists of triangular or square grid patterns and center-to-center spacing is usually in the range of 0.5 - 2.5 m.

Vertical Drains Parameters



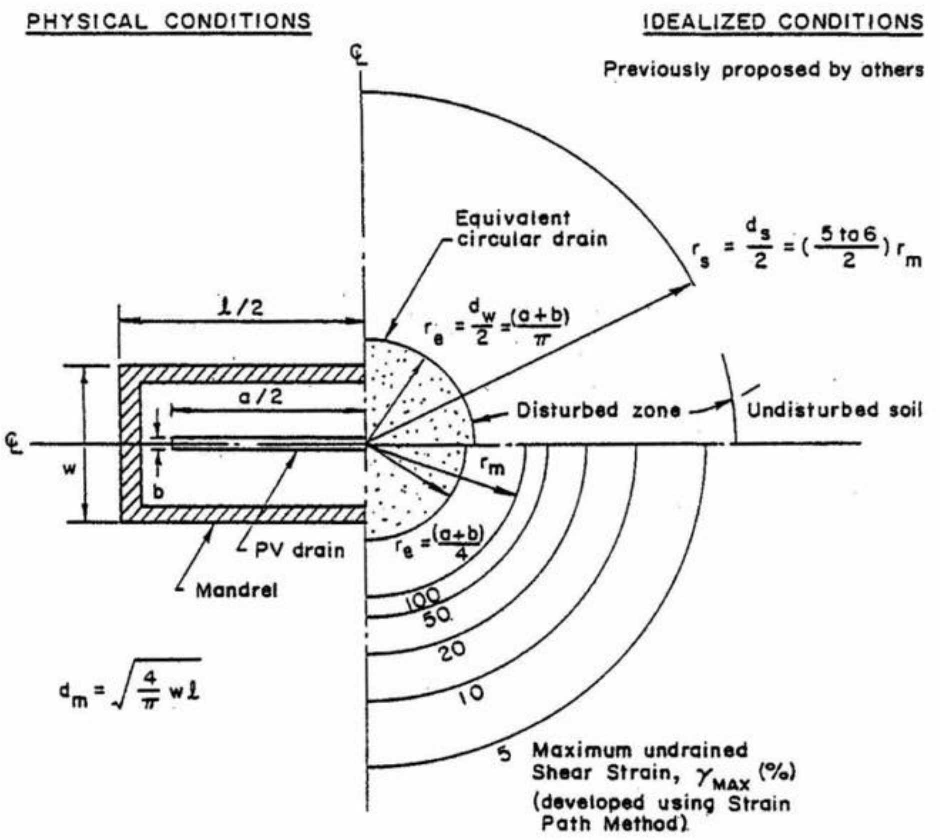


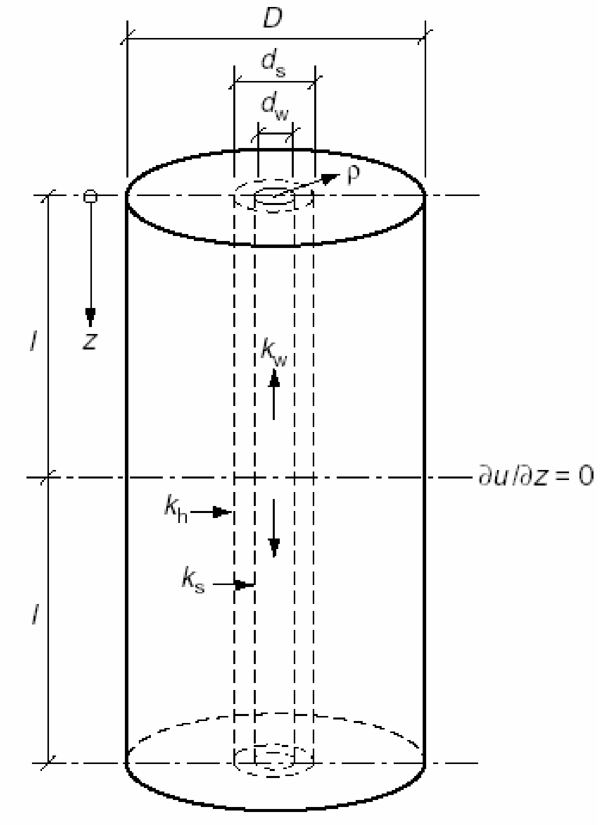
Authors	Smear zone radius
Holtz and Holm (1973); Hansbo (1986), (1987); Bergado et al. (1991), (1993b)	$2\ r_{m,eq}$
Jamiolkowski et al. (1983)	$(2.5\text{ to }3)\ r_{m,eq}$
Mesri et al. (1994)	$(2\text{ to }4)\ r_{m,eq}$
Hansbo (1997); Chai and Miura (1999); Hird and Moseley (2000)	$(2\text{ to }3)\ r_{m,eq}$
Sathananthan and Indraratna (2006)	$2.5\ r_{m,eq}$

S = center-to-center spacing,
De = equivalent influence zone diameter,
rm = equivalent mandrel radius,

axb = dimension of drain,
dw = equivalent drain diameter,
rs = equivalent smeared zone radius.

Vertical Drains Parameters



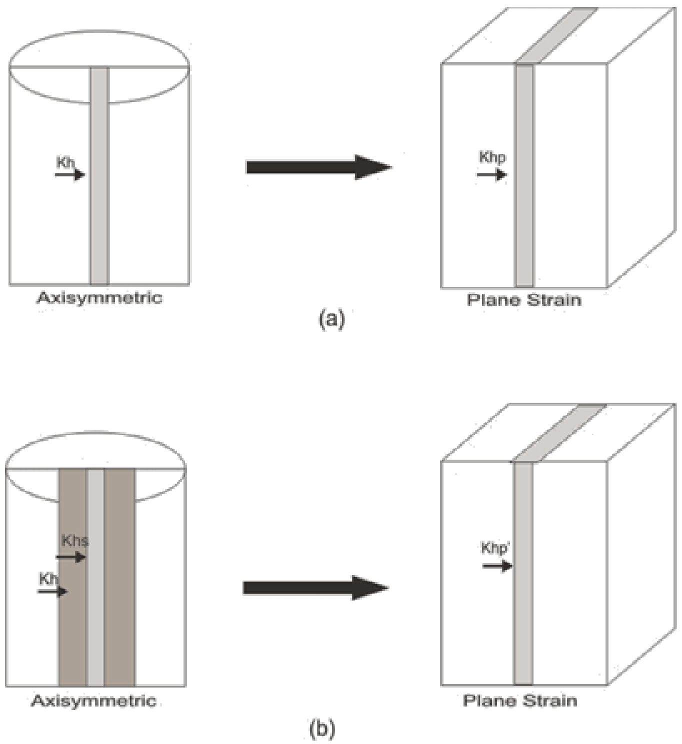


axb = dimension of drain,
$\mathbf{r_e}$ = equivalent drain radius,
$\mathbf{r_s}$ = equivalent smear zone radius,
$\mathbf{r_m}$ = equivalent mandrel radius,

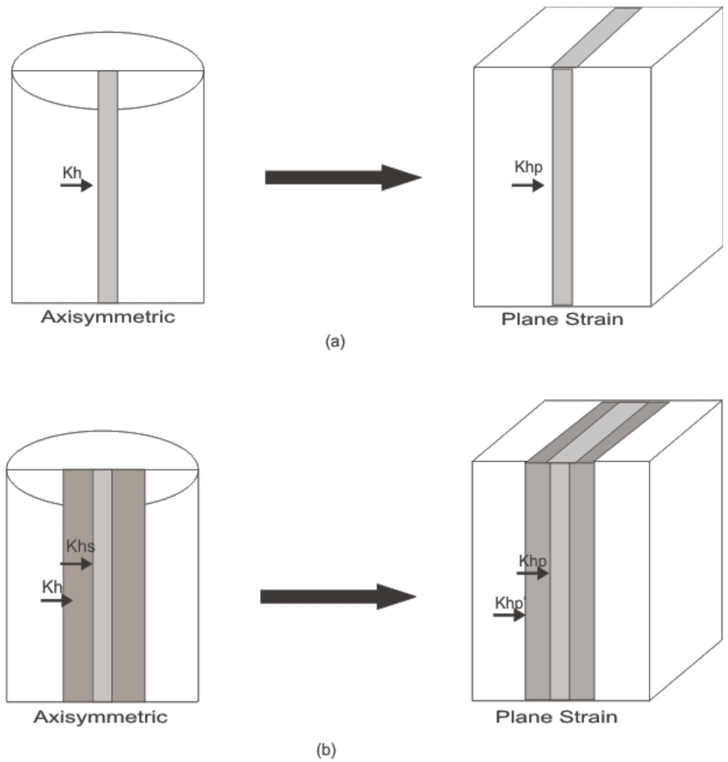
lxw = dimension of mandrel,
$\mathbf{d_w}$ = equivalent drain diameter,
$\mathbf{d_s}$ = equivalent smear zone diameter,
$\mathbf{d_m}$ = equivalent mandrel diameter,

D = diameter of soil cylinder,
$\mathbf{k_w}$ = vertial permeability of the drain,
$\mathbf{k_h}$ = horizontal permeability of soil,
$\mathbf{k_s}$ = horizontal permeability of smear zone.

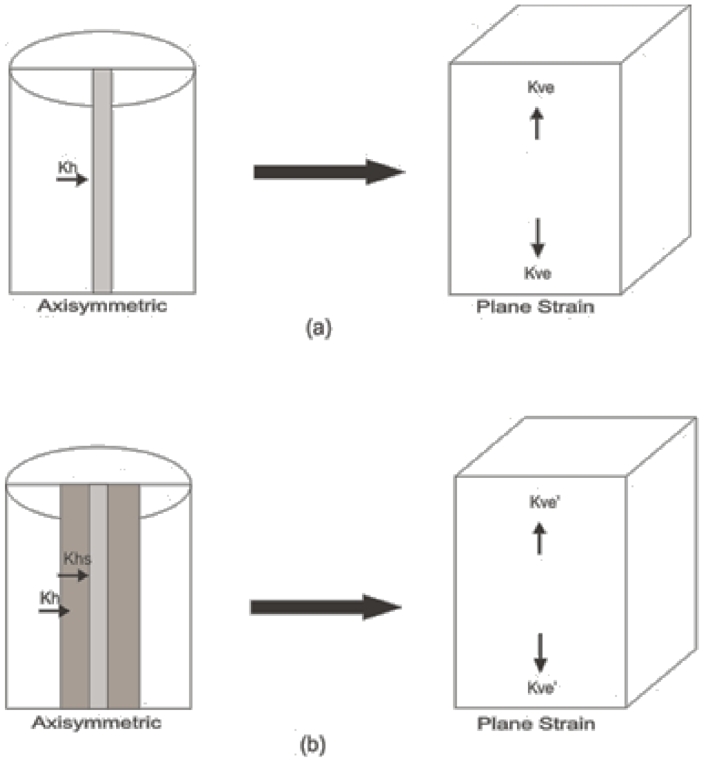
Conversion of an axisymmetric unit cell into plane strain condition



Hird et al. (1992)



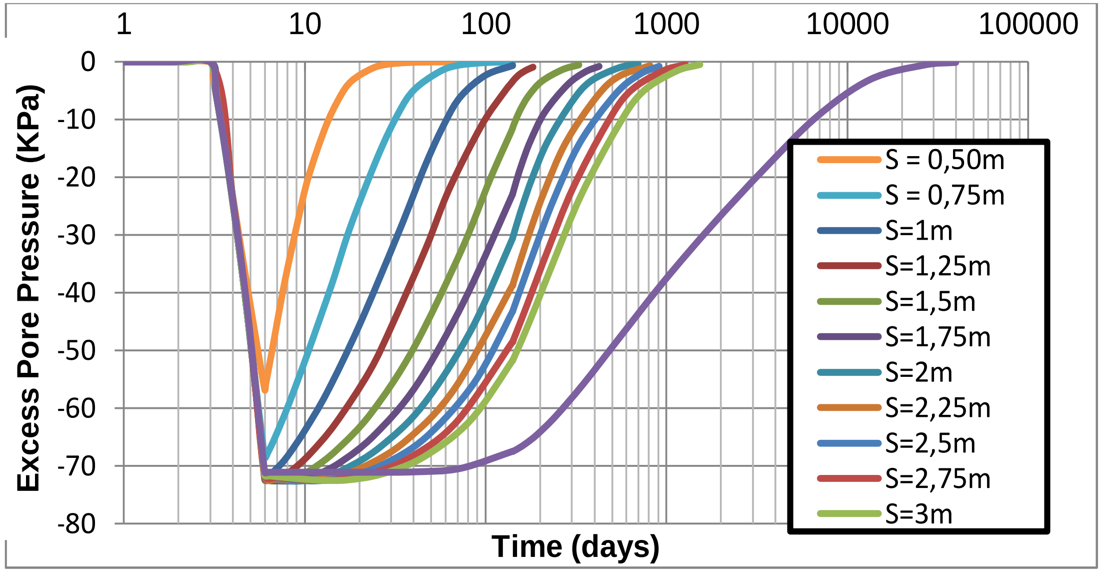
Indraratna et al. (1997)



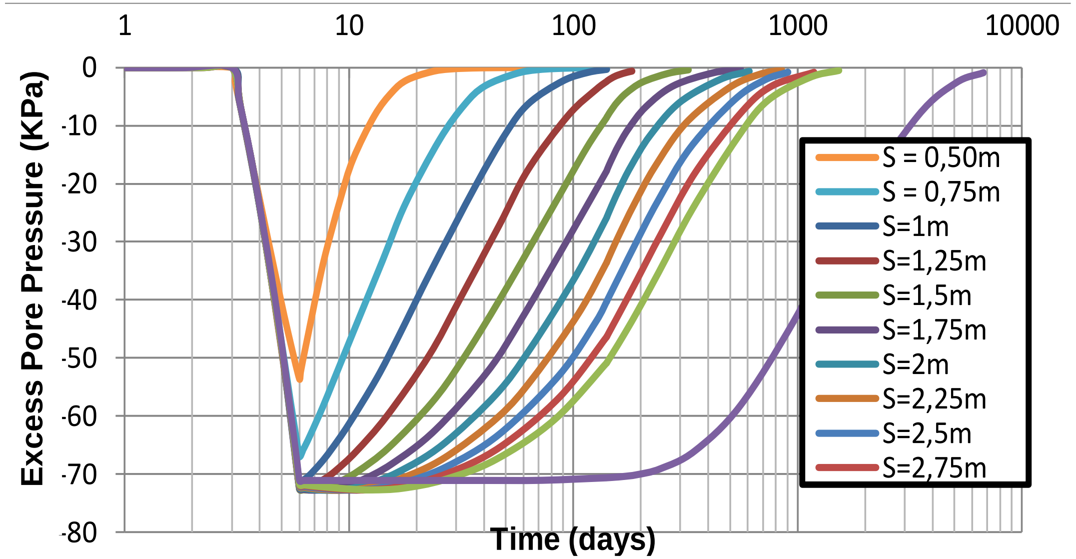
Chai et al. (2001)

Transforming the in-situ 3D unit-cell axisymmetric condition into equivalent 2D multi-drain plane strain condition

Influence of PVD Spacing (s)

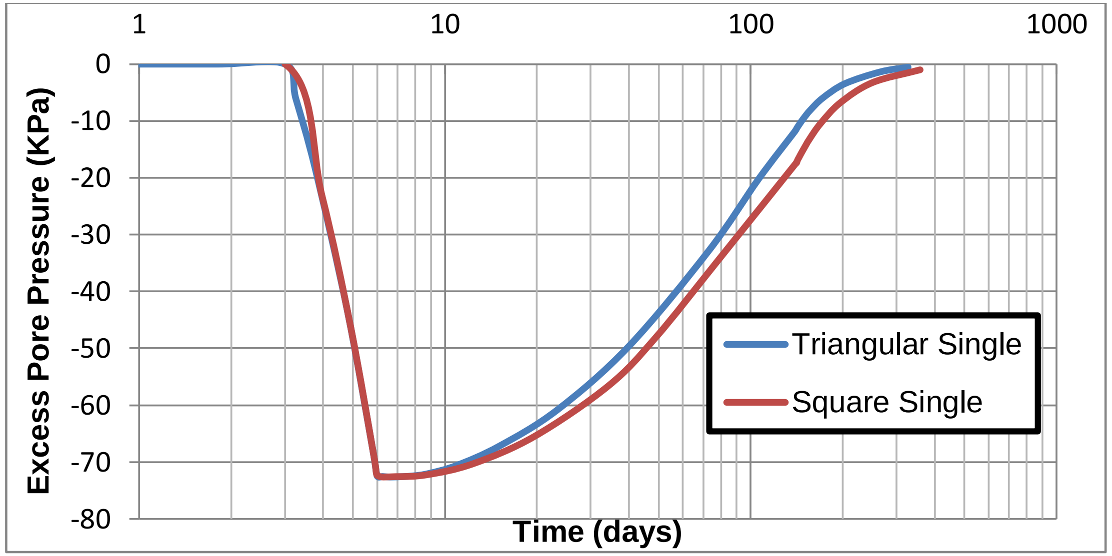


Single Drained
S= 0.5m - 3.0m

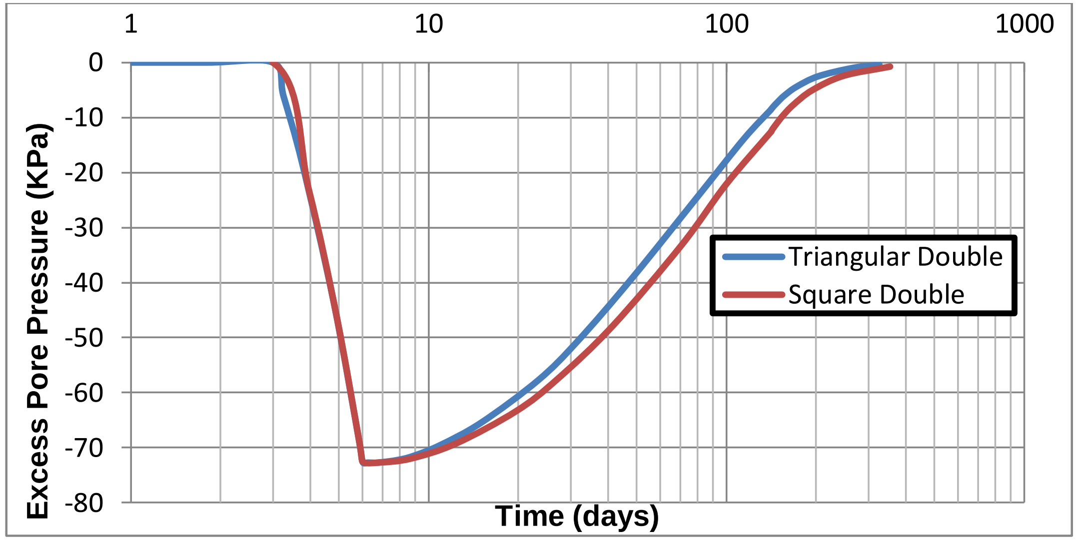


Double Drained
S= 0.5m – 3.0m

Influence of PVD Configuratioan (de) Triangular and Square Patterns



Single Drained



Double Drained

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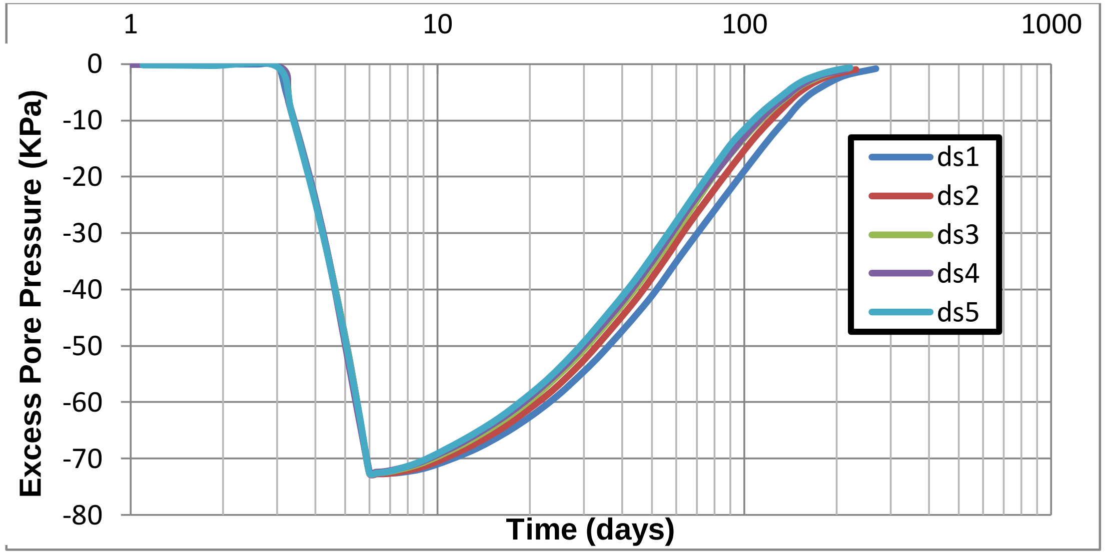
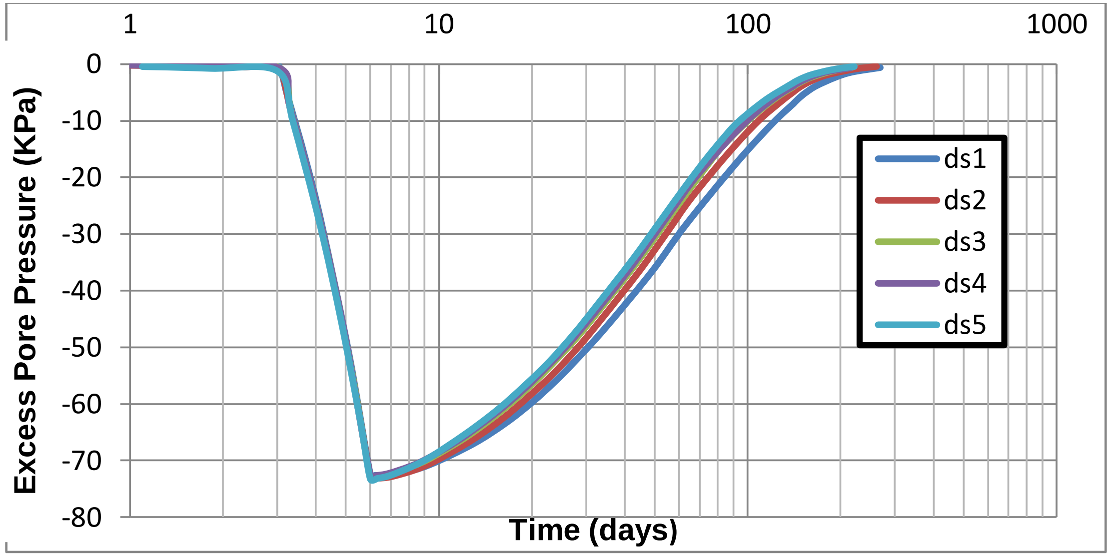
Influence of Smear Zone Diameter (ds)



Single Drained
ds1= 99 mm
ds2= 240 mm
ds3= 360 mm
ds4= 396 mm
ds5= 480 mm

Double Drained
ds1= 99 mm
ds2= 240 mm
ds3= 360 mm
ds4= 396 mm
ds5= 480 mm



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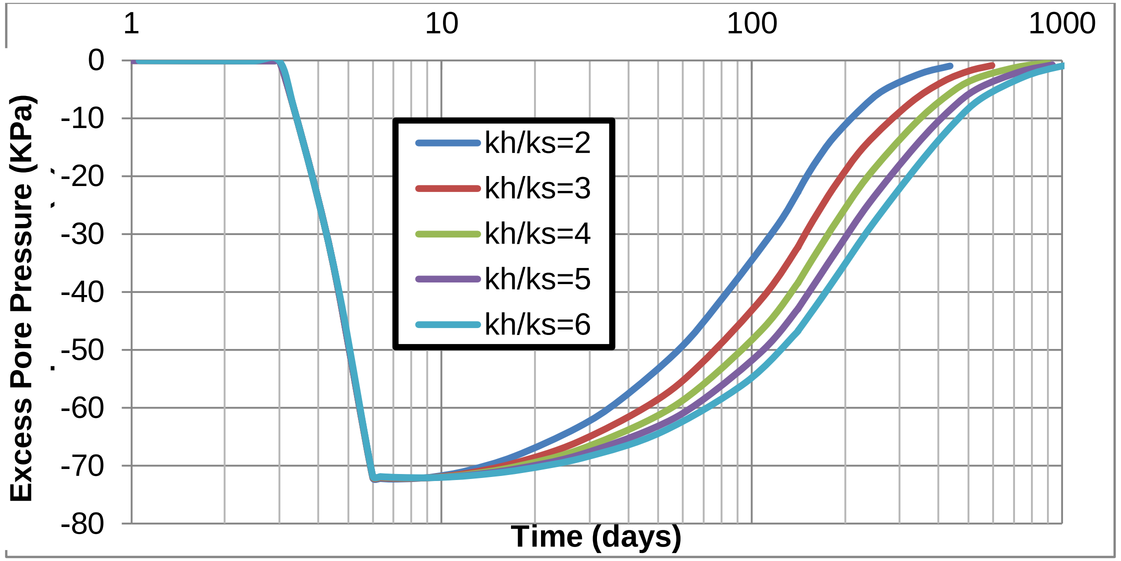
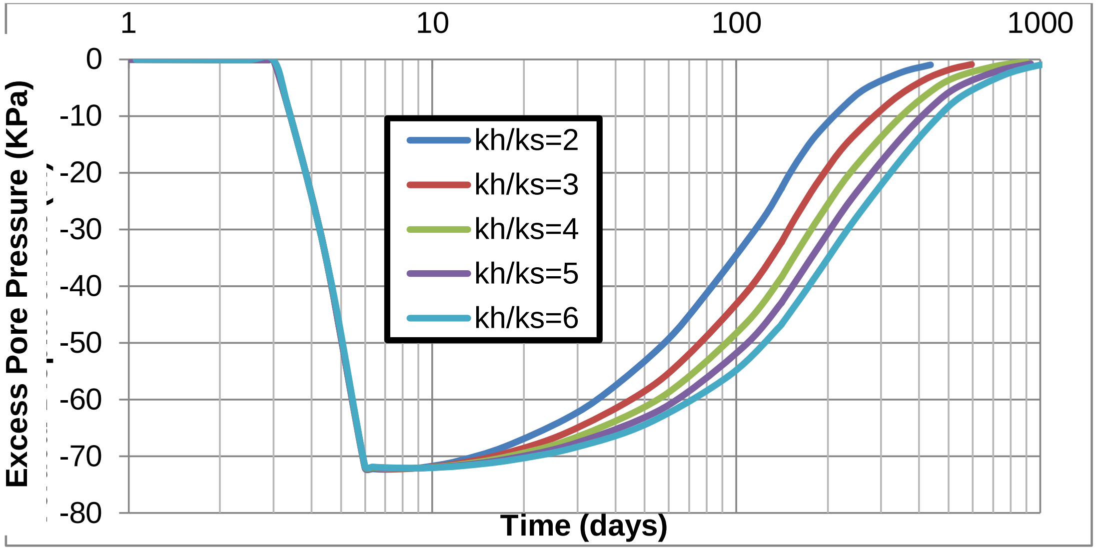
Influence of Smear Zone Permeability (ks)

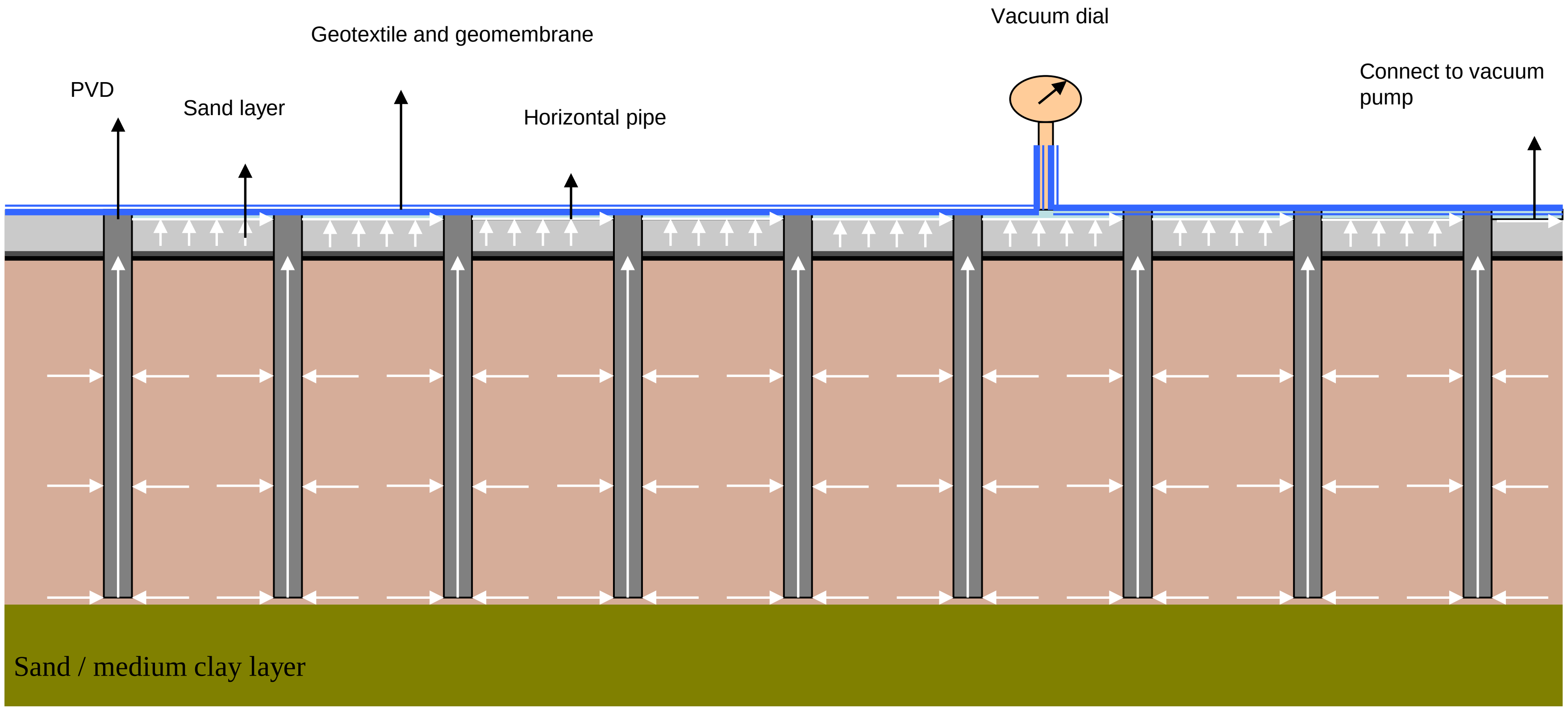


Single Drained



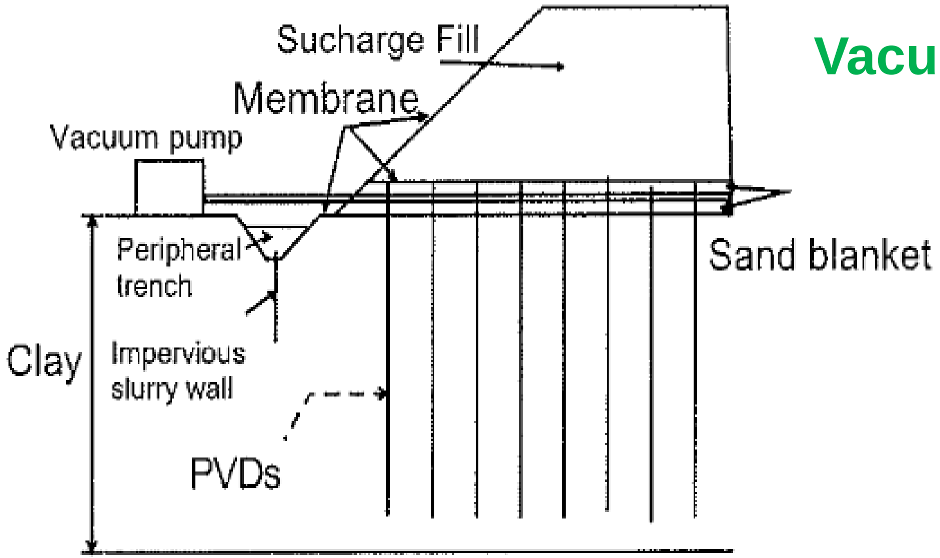
Double Drained

PVD with Vacuum Preloading

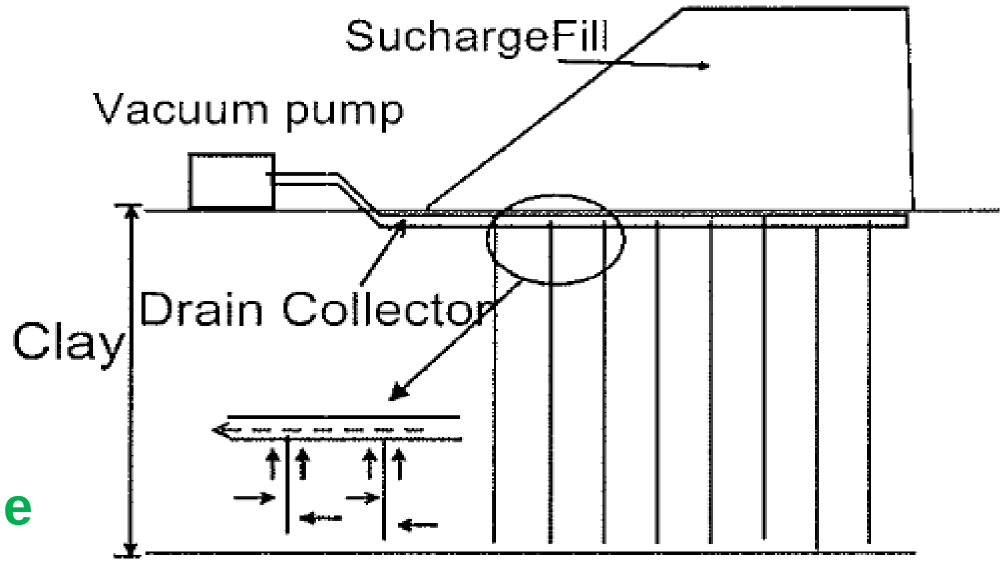


- Kjellman first introduce a vacuum preloading to accelerate consolidation in 1952 and a small field test was perform.
- In this method the soil load is replace by atmospheric pressure

PVD with Vacuum Preloading



Vacuum with Membrane



Vacuum without Membrane

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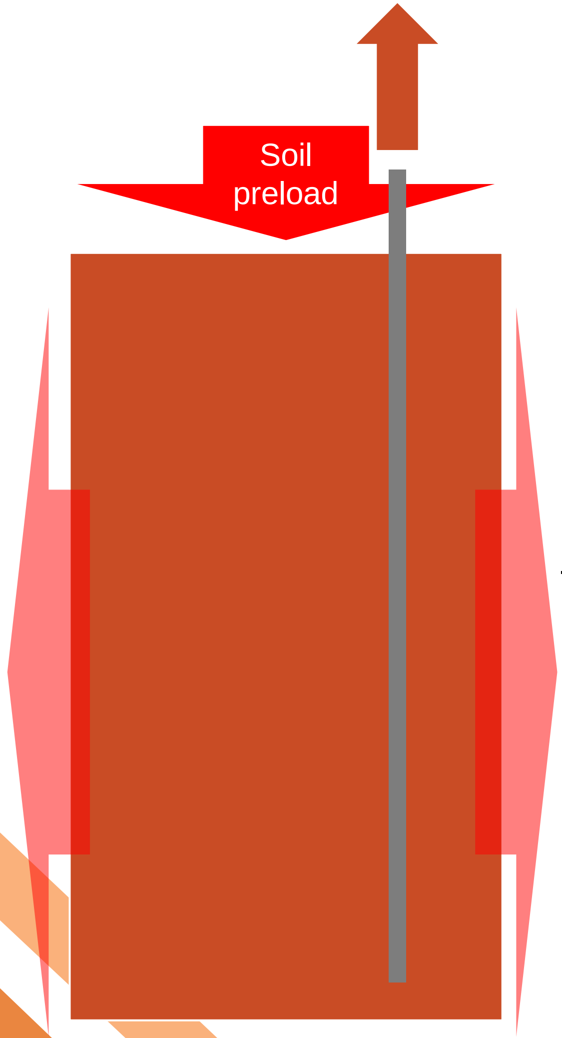
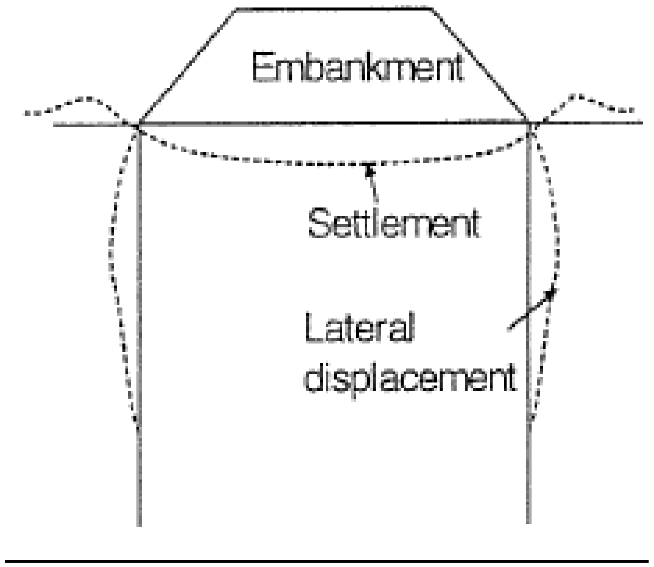
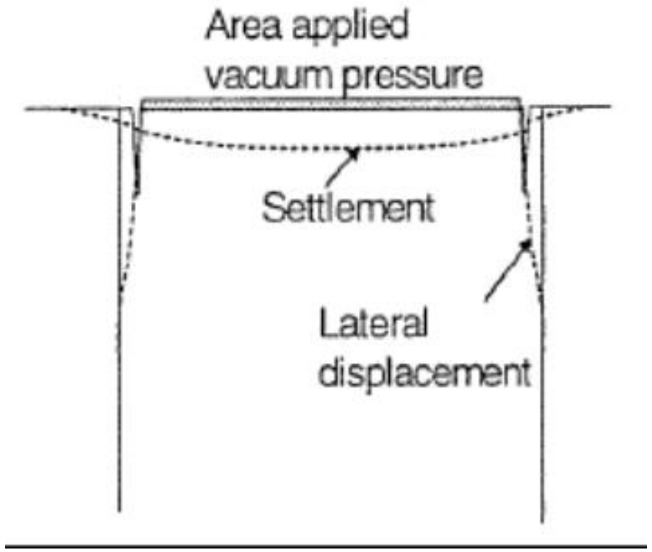
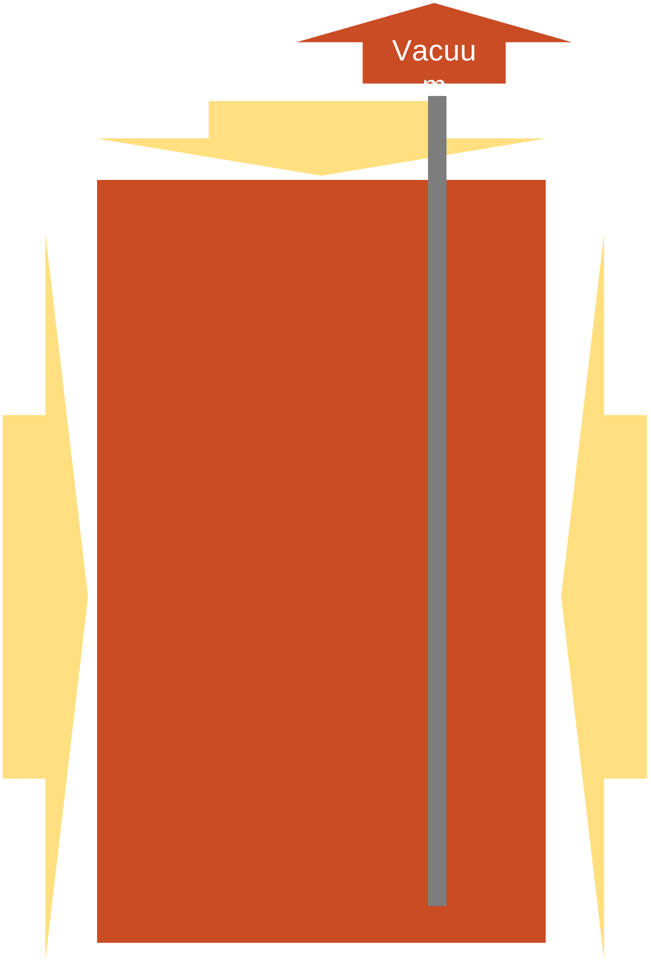
Vacuum vs Non Vacuum





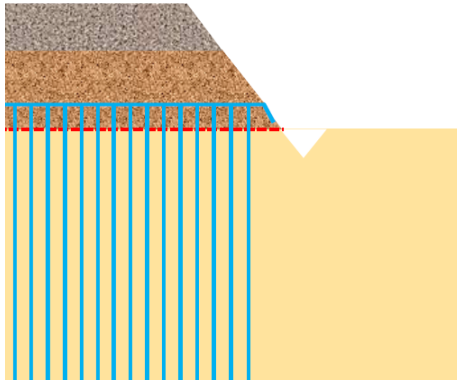


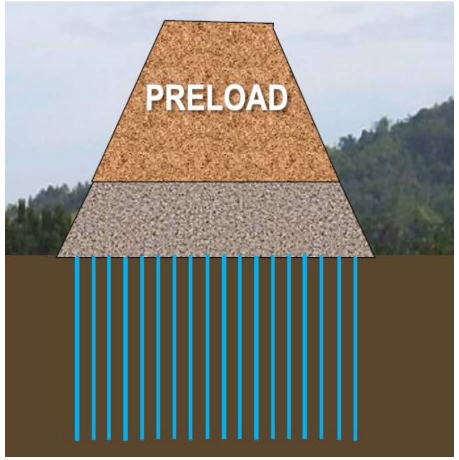


Vacuum vs Non Vacuum

PVD with Conventional Soil Prelading

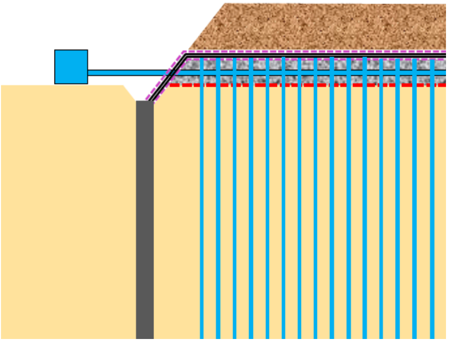
1. Higher Surcharge
2. Lower Stability
3. Greater Lateral Movement
4. Many heavy equipment
5. Longer Construction Time





PVD with Vacuum Preloading

1. Lower or no Surcharge
2. Better Control of Stability
3. Less Lateral Movement
4. Less heavy equipment
5. Shorter Construction Time



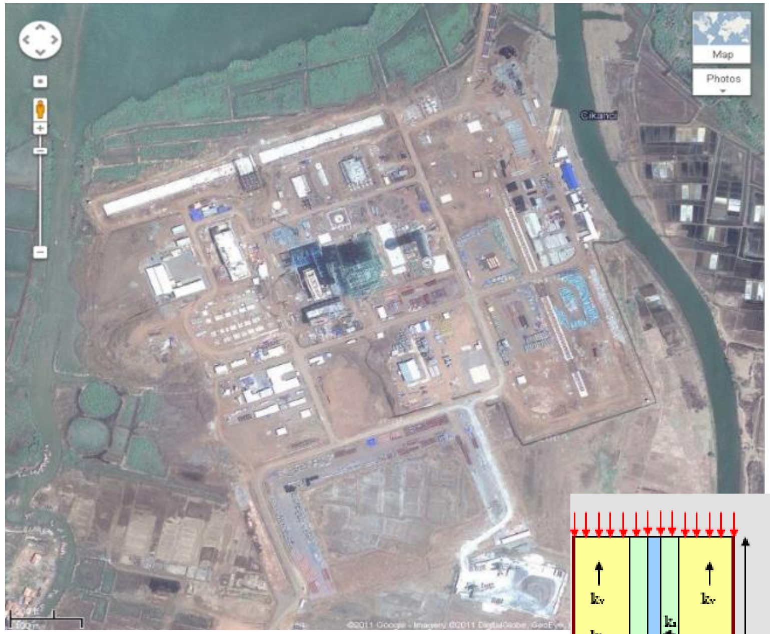
Case Study 1
Partially Penetraton of Vertcal Drains
(Floating PVD)
Cirebon Power Plant Project

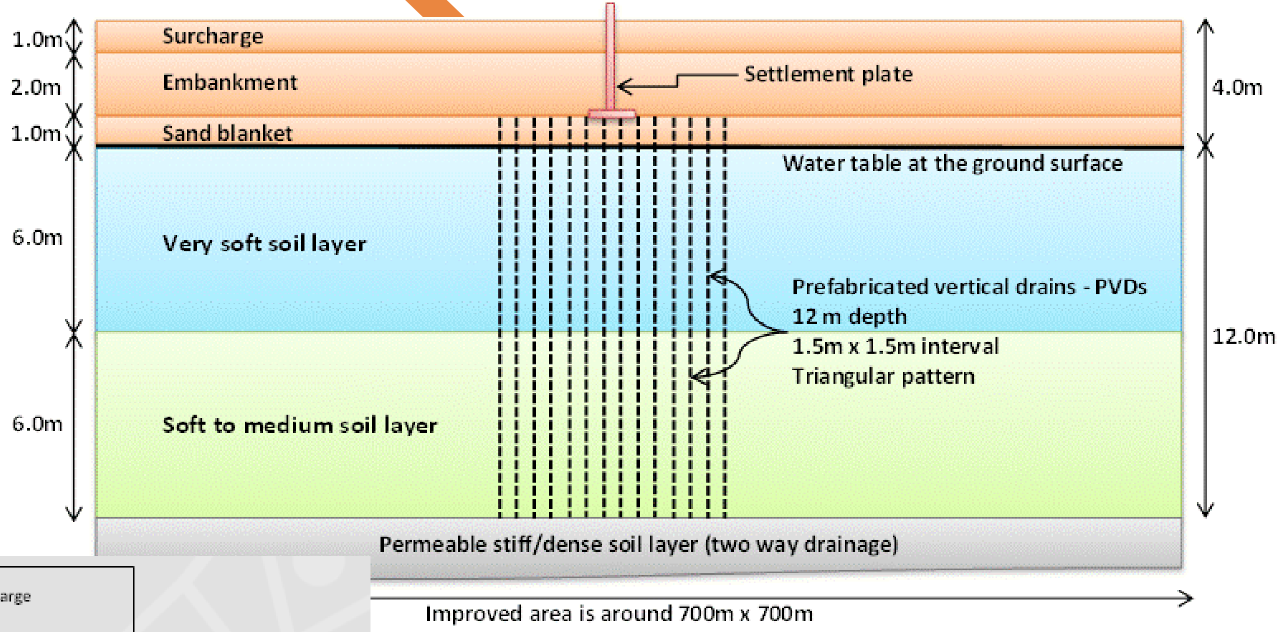
Ikhya Ikhya 1) and Helmut F. Schweiger 2)

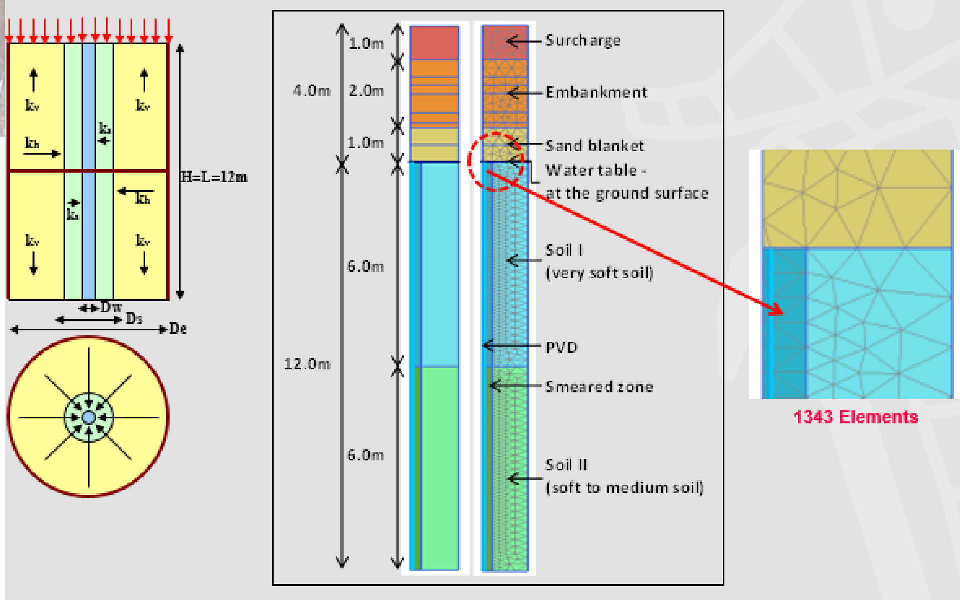
1) Civil Engineering Department
National Institute of Technology (Itenas)
Bandung, Indonesia

2) Computational Geotechnics Group
Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology

Case study 1: Cirebon Power Plant Project Partially Penetration of Vertical Drains (Floating PVD)

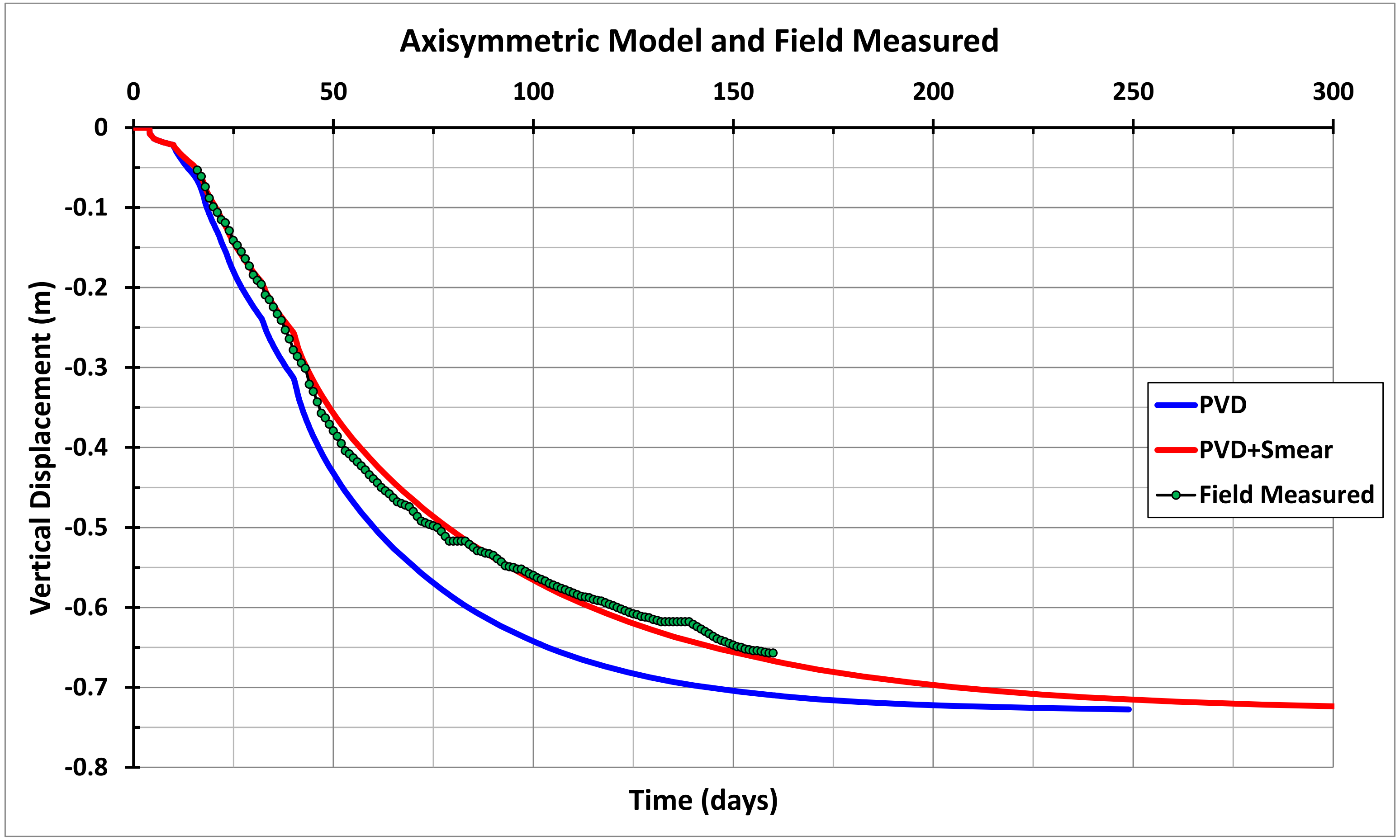






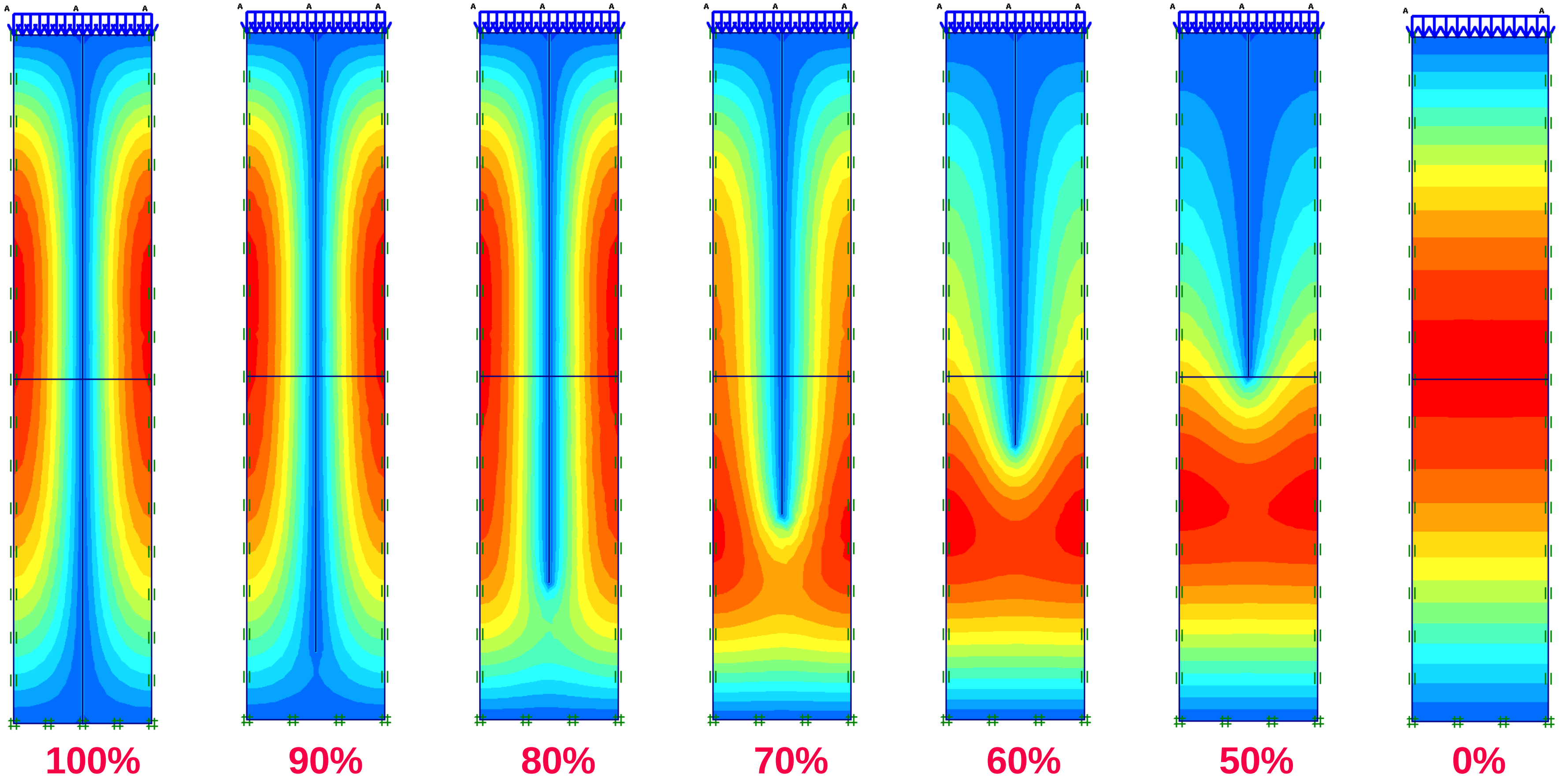
A good agreement between the field measurement result and the numerical model prediction in the 2D axisymmetric condition at Cirebon Power Plant Project.

Case study 1: Cirebon Power Plant Project Comparison field measurement and numerical model - Vertical Displacement



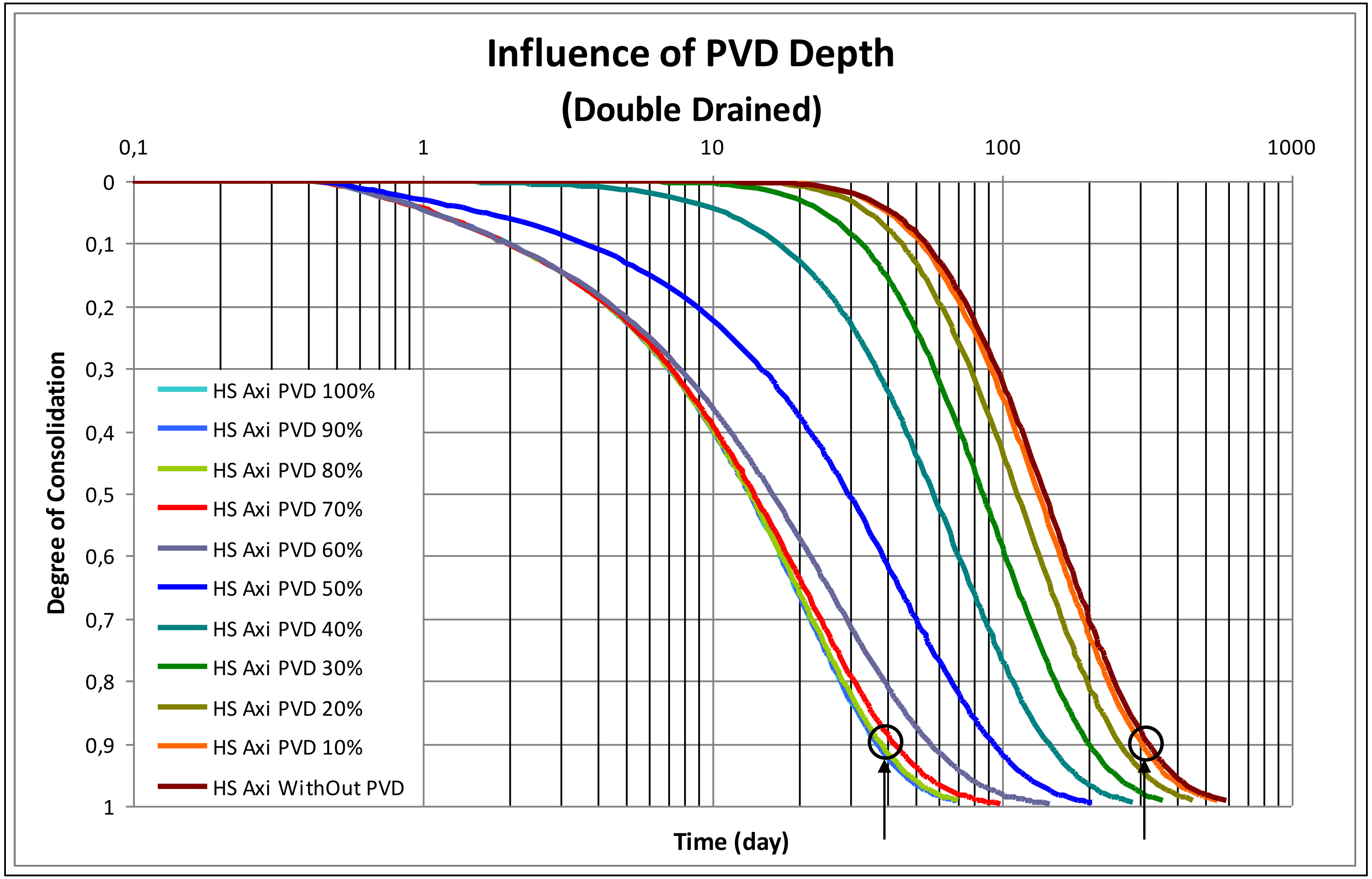
A good agreement between the field measurement result and the numerical model prediction in the 2D axisymmetric condition at Cirebon Power Plant Project.

Case study 1: Cirebon Power Plant Project Influence of PVD Penetration Depth (Double Drained)

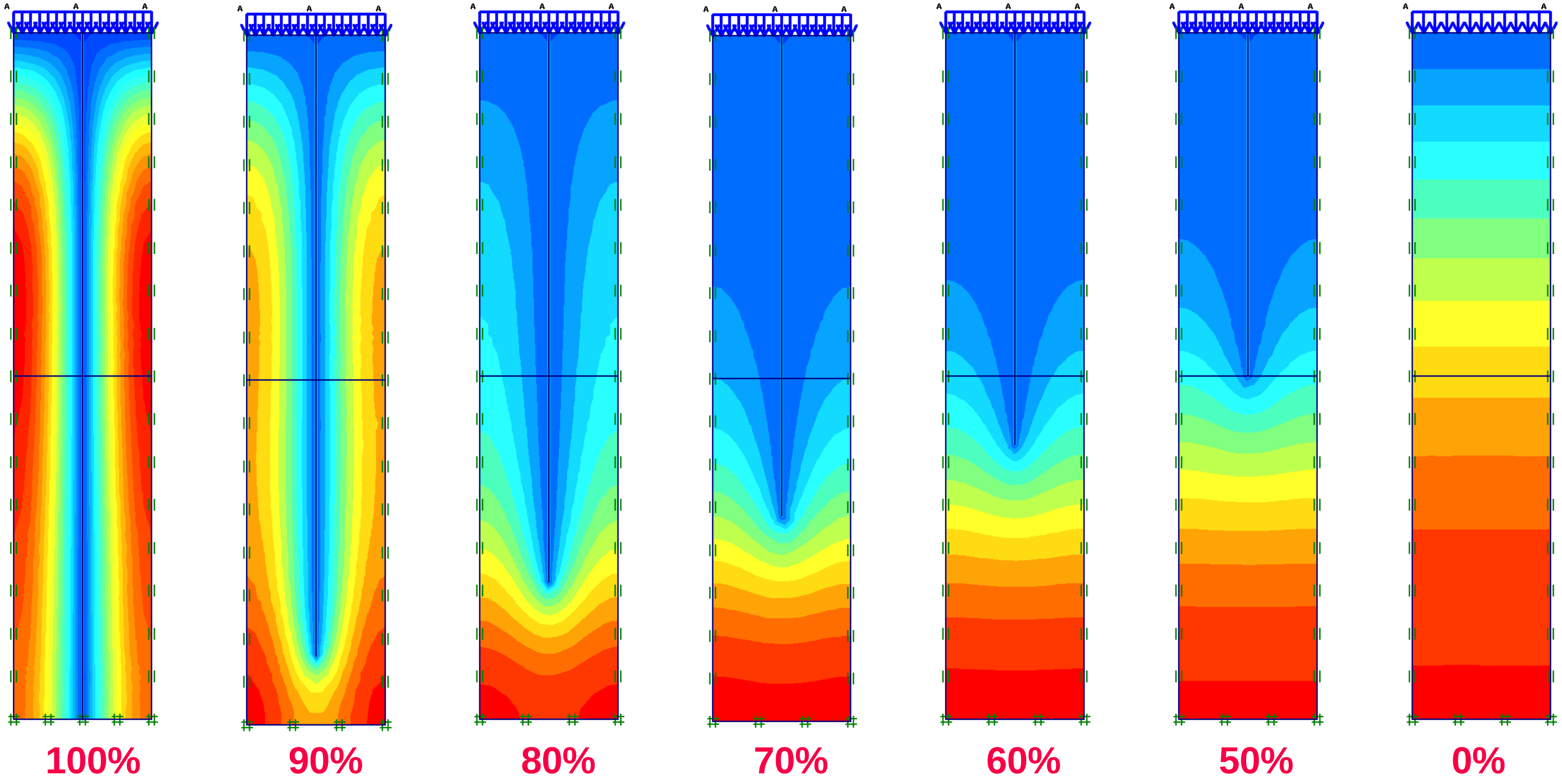


(De=2B=2m, Hsoil=10m, P=100KPa, GWT=0m, Double Drained)

Case study 1: Cirebon Power Plant Project Degree of Consolidation for Different PVD Penetration Depth (Axi)

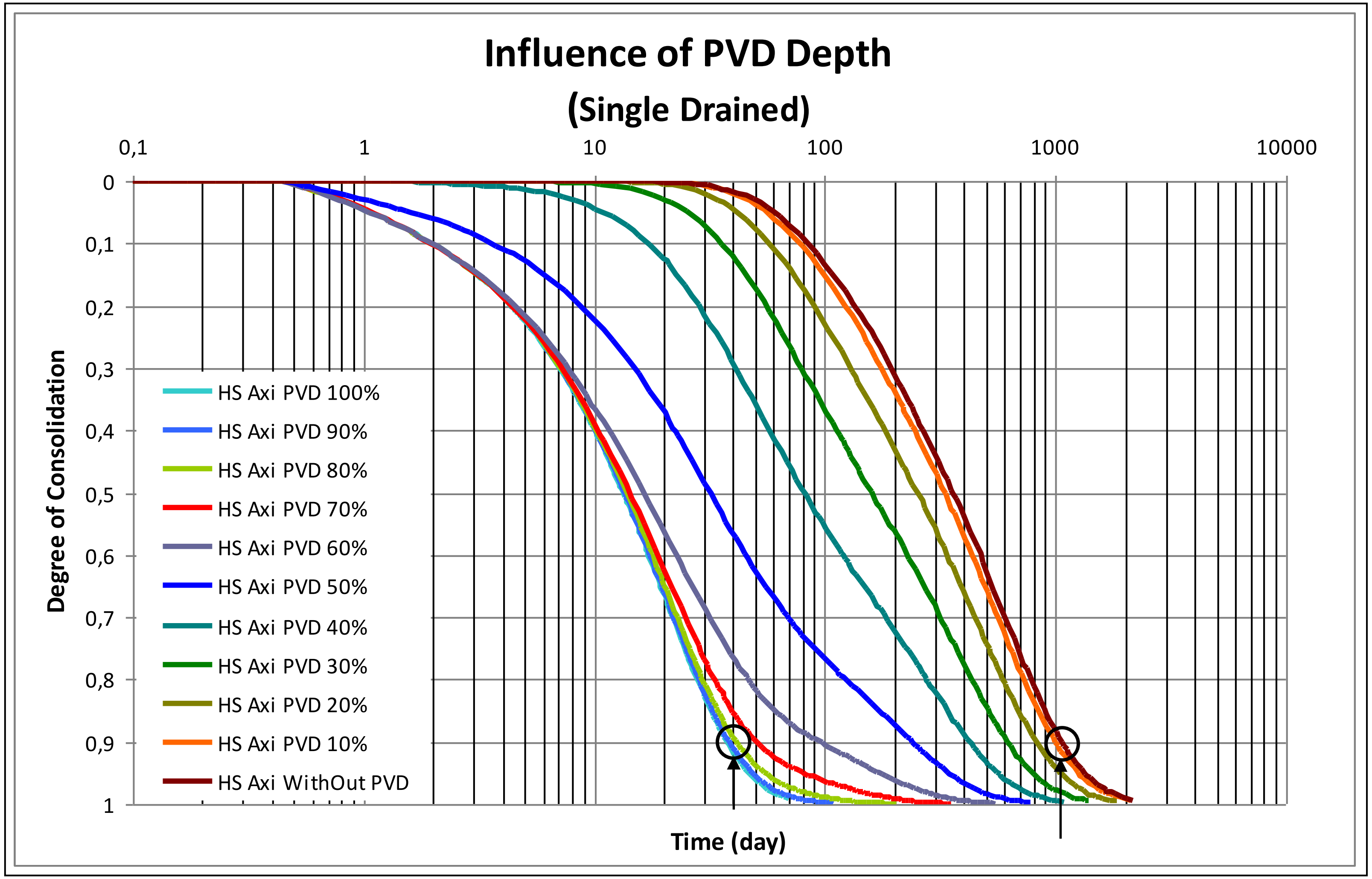


Case study 1: Cirebon Power Plant Project Influence of PVD Penetration Depth (Single Drained)



(De=2m, Hsoil=10m, P=100KPa, GWT=0m, Single Drained)

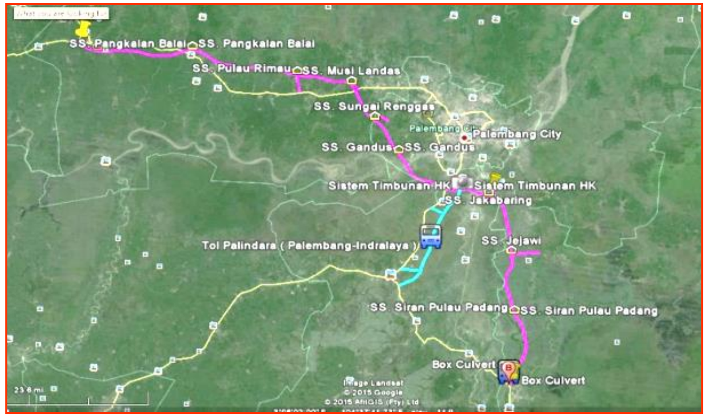
Case study 1: Cirebon Power Plant Project Degree of Consolidation for Different PVD Penetration Depth (Axi)



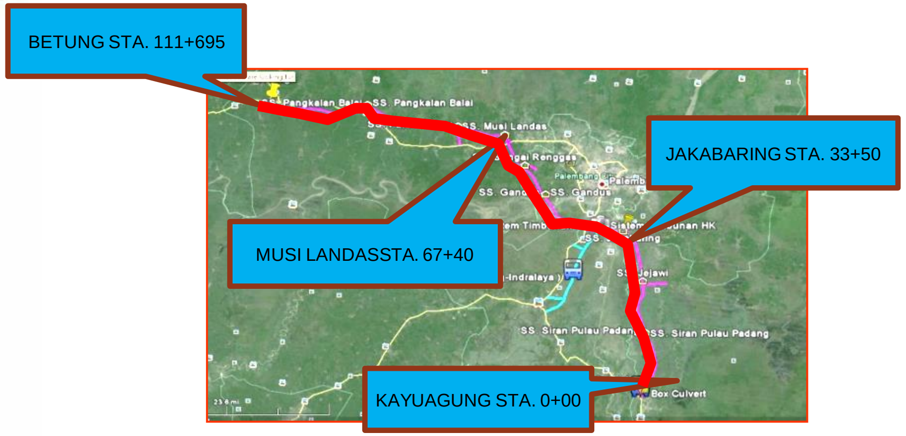
Case Study 2
PVD with Vacuum Preloading
(Vacuum vs Non Vacuum)
Kayu Agung – Palembang – Betung
(Kapalbetung) Toll Road project

Ikhya and Muhammad Yoke Syahputra

Civil Engineering Department
National Institute of Technology (Itenas)
Bandung, Indonesia



Case study 2: Kapalbetung Toll Road Project PVD with Vacuum Preloading (Vacuum vs Non Vacuum)

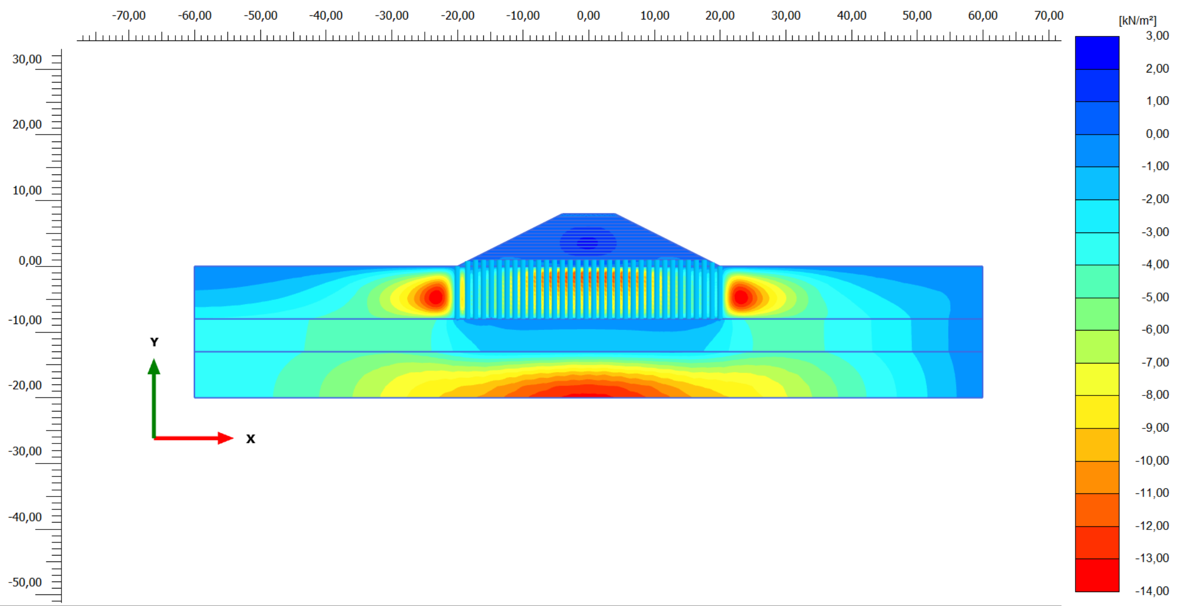


Case study 2: Kapalbetung Toll Road Project
PVD with Conventional Soil Preloading

PVD + Preloading

STA. 17+500 – PVD + Preloading

No	Depth (m)	Soil Model	Soil Layer	γ_{sat} (kN/m^3)	c' (kPa)	ϕ' (deg)	E' (kPa)	E_{50} (kPa)	E_{oed} (kPa)	E_{ur} (kPa)	K_o^{NC}	kx (m/day)	ky (m/day)
1		Mohr Coulomb	Fill Material	16	30	27	30000	-	-	-	-	0.01	0.01
2	0 - 8 m	Hardening Soil	Soft CLAY	16	7	20.0	-	1000	1000	3000	0.658	0.00423	0.00423
3	0 - 8 m	Hardening Soil	Soft CLAY (ps)	16	7	20.0	-	1000	1000	3000	0.658	0.00998	0.00998
4	8 - 13 m	Mohr Coulomb	SAND	17	0.001	30.0	12000	-	-	-	-	0.01	0.01
5	13 - 20 m	Mohr Coulomb	Clayey SILT	17	23	17.0	17000	-	-	-	-	0.004	0.004

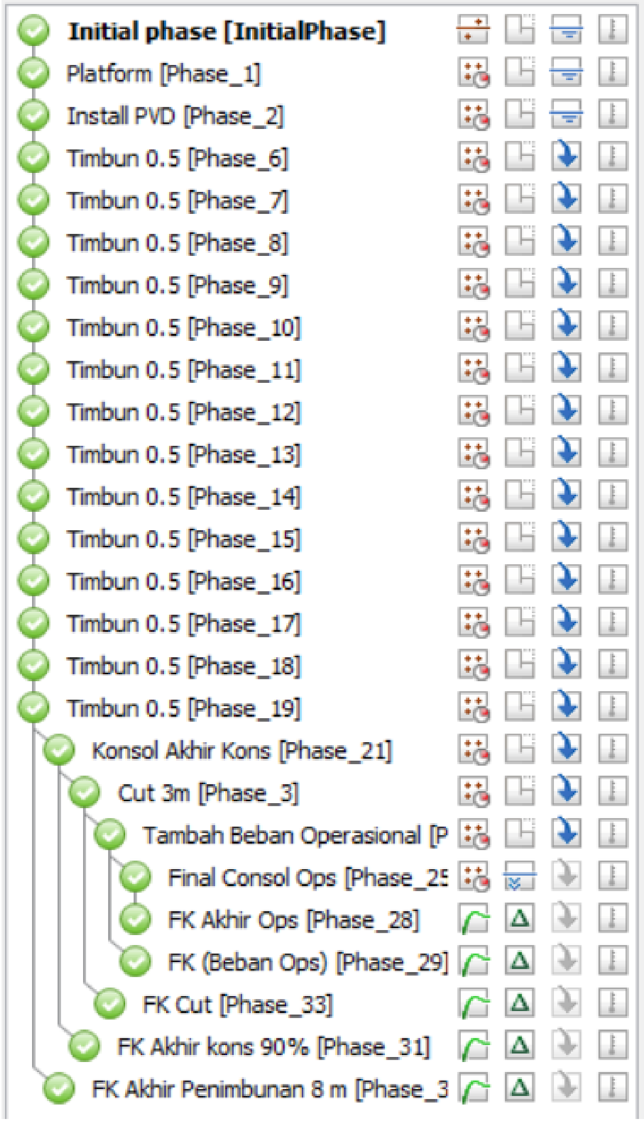


Excess pore pressures p_{excess} (Pressure = negative) (Time 112,0 day)

Maximum value = 2,234 kN/m² (Element 375 at Node 13686)

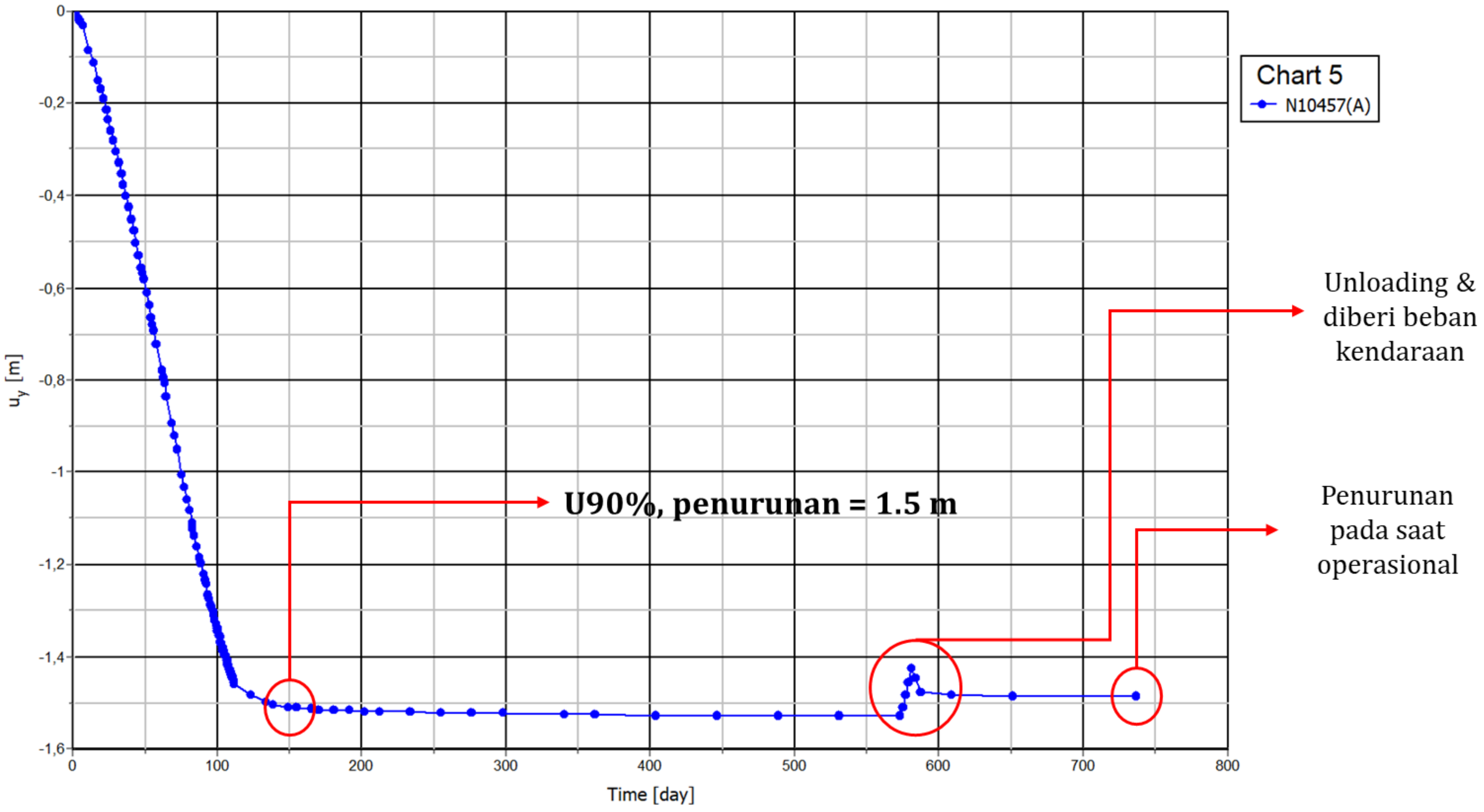
Minimum value = -13,87 kN/m² (Element 1030 at Node 1137)

Stage Construction :



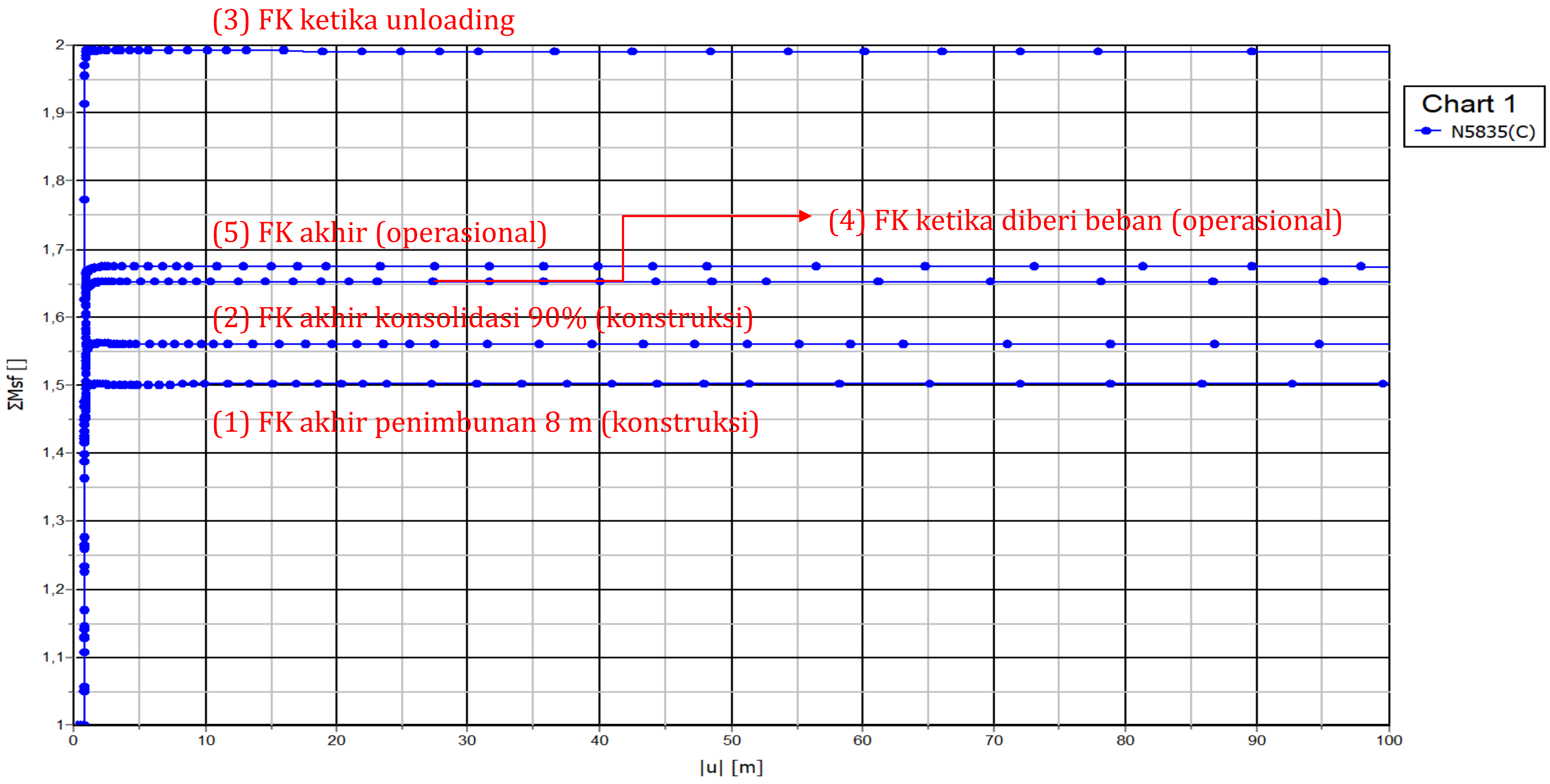
Case study 2: Kapalbetung Toll Road Project
PVD with Conventional Soil Preloading

STA. 17+500 – PVD + Preloading



Case study 2: Kapalbetung Toll Road Project PVD with Conventional Soil Preloading

STA. 17+500 – PVD + Preloading



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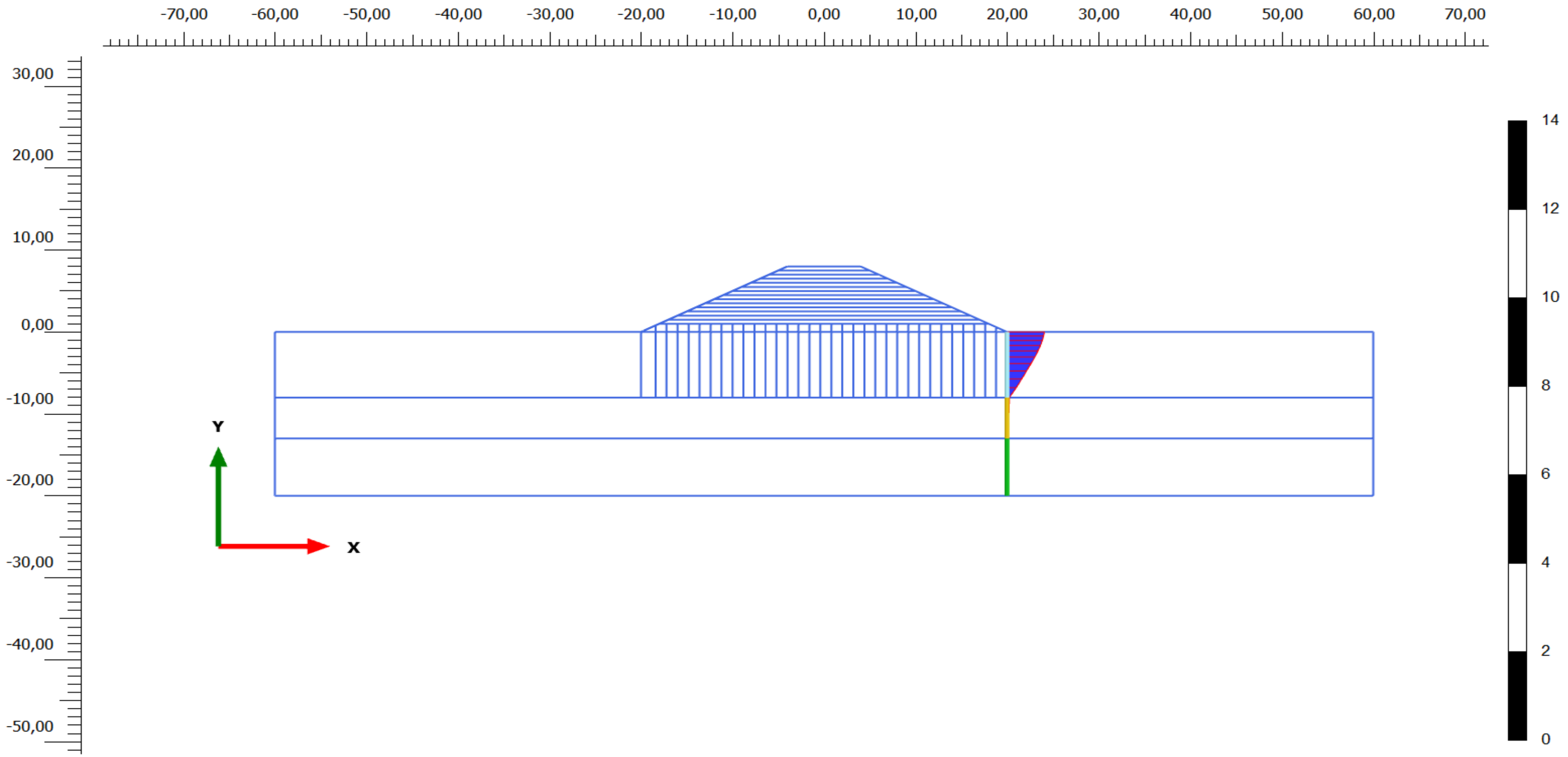
Case study 2: Kapalbetung Toll Road Project PVD with Conventional Soil Preloading

STA. 17+500 – PVD + Preloading



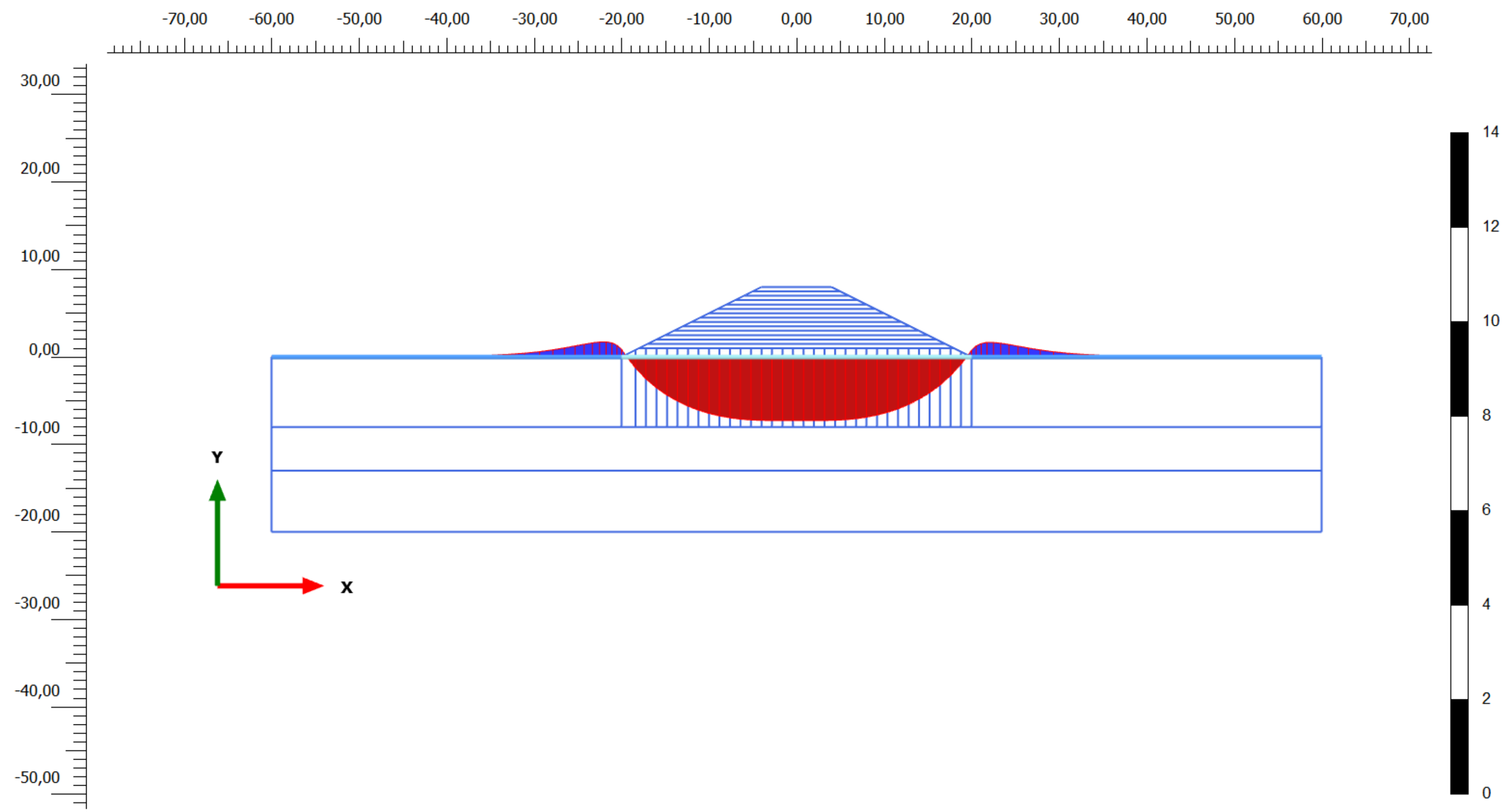
Total displacements u_x (scaled up 5,00 times) (Time 112,0 day)

Maximum value = 0,8164 m

Minimum value = 0,000 m

Case study 2: Kapalbetung Toll Road Project
PVD with Conventional Soil Preloading

STA. 17+500 – PVD + Preloading



Total displacements u_y (scaled up 5,00 times) (Time 112,0 day)

Maximum value = 0,3451 m

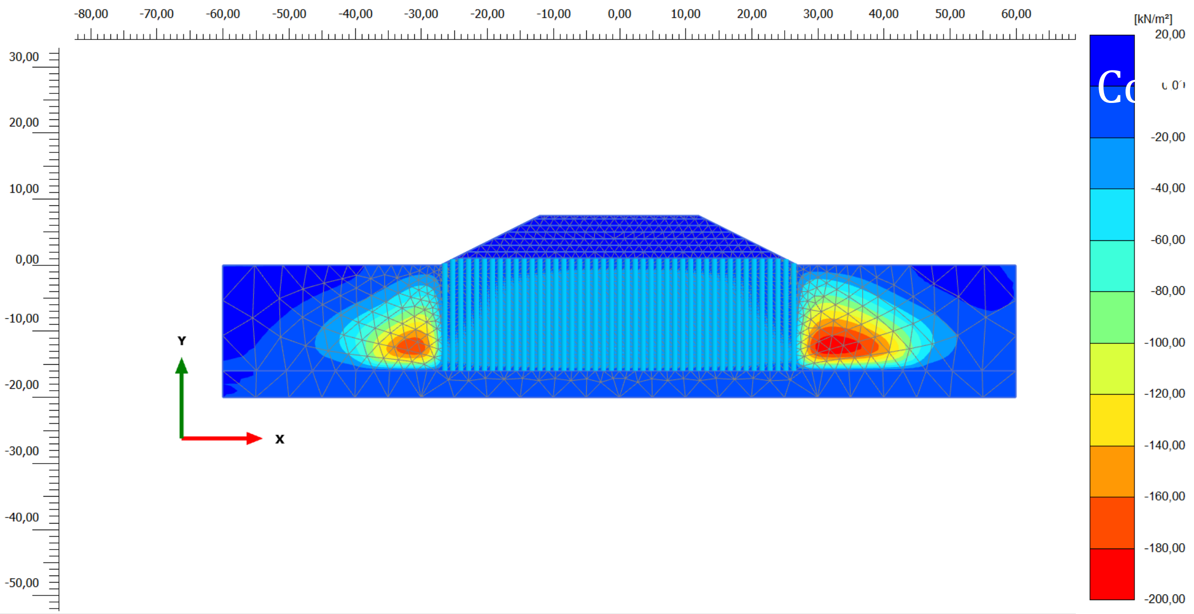
Minimum value = -1,453 m

Case study 2: Kapalbetung Toll Road Project PVD with Vacuum Preloading

STA. 31+500 – PVD + Vacuum

PVD + Vacuum

No	Depth (m)	Soil Model	Soil Layer	γ_{sat} (kN/m^3)	c' (kPa)	ϕ' (deg)	E' (kPa)	E_{50} (kPa)	E_{oed} (kPa)	E_{ur} (kPa)	K_o^{NC}	kx (m/day)	ky (m/day)
1		Mohr Coulomb	Fill Material	16	30	27	30000	-	-	-	-	0.01000	0.01000
2	0 - 16 m	Hardening Soil	Soft CLAY	16	8	22.0	-	1200	1200	3600	0.625	0.00423	0.00423
3	0 - 16 m	Hardening Soil	Soft CLAY (ps)	16	8	22.0	-	1200	1200	3600	0.625	0.00998	0.00998
4	16 - 20 m	Mohr Coulomb	SAND	17	0.001	30.0	12000	-	-	-	-	0.01000	0.01000

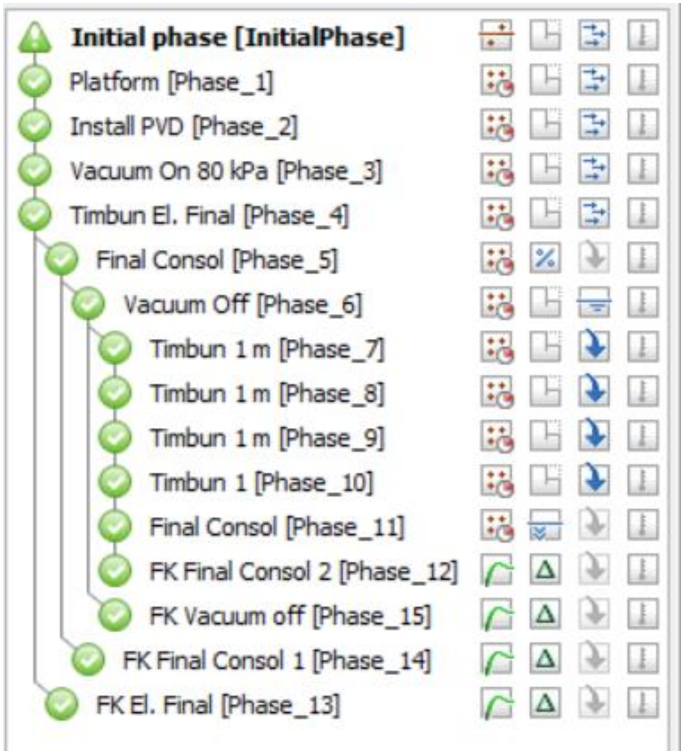


Excess pore pressures p_{excess} (Pressure = negative) (Time 35,00 day)

Maximum value = 5,991 kN/m² (Element 849 at Node 26)

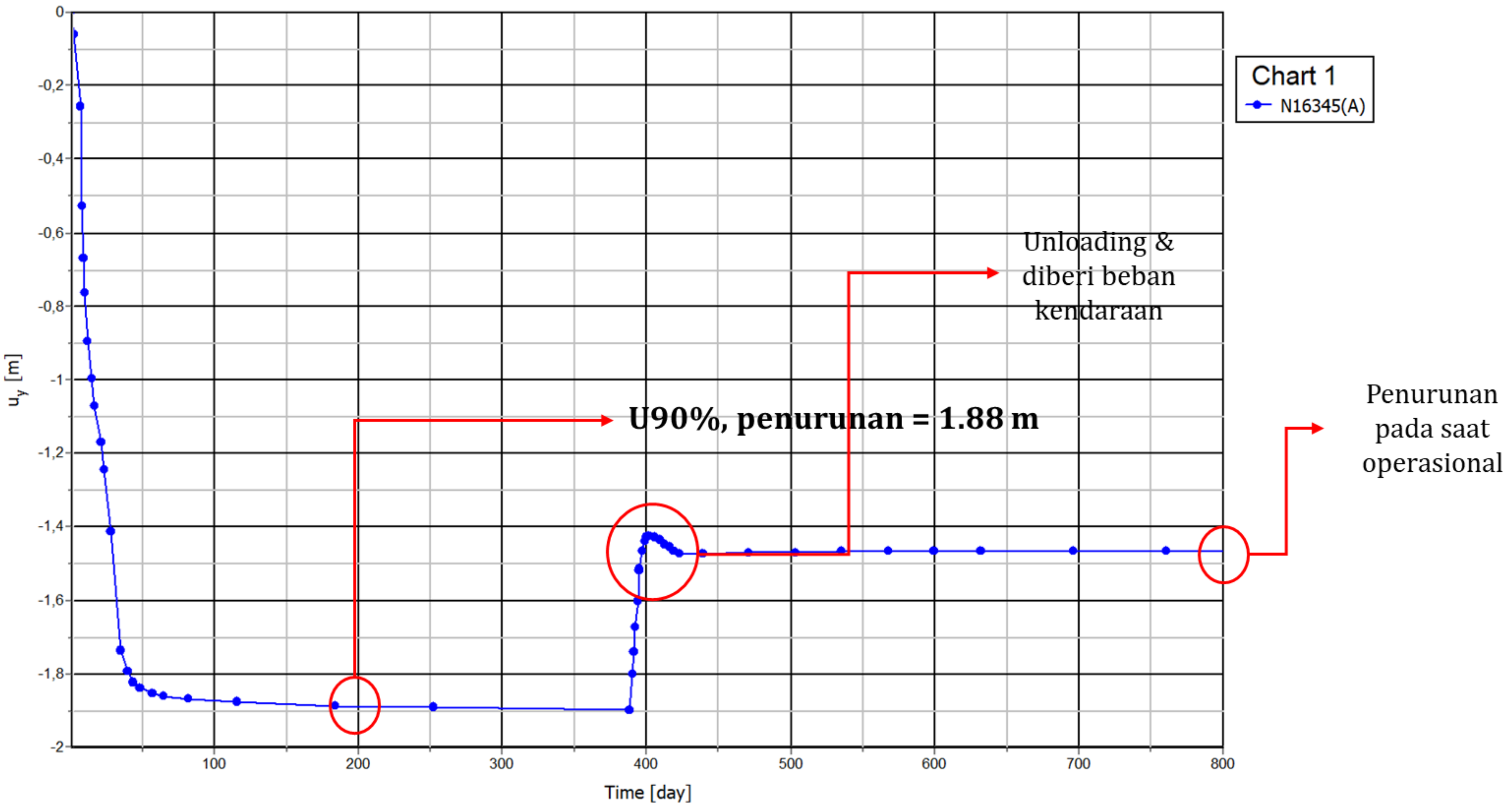
Minimum value = -195,4 kN/m² (Element 2324 at Node 7149)

Stage Construction :



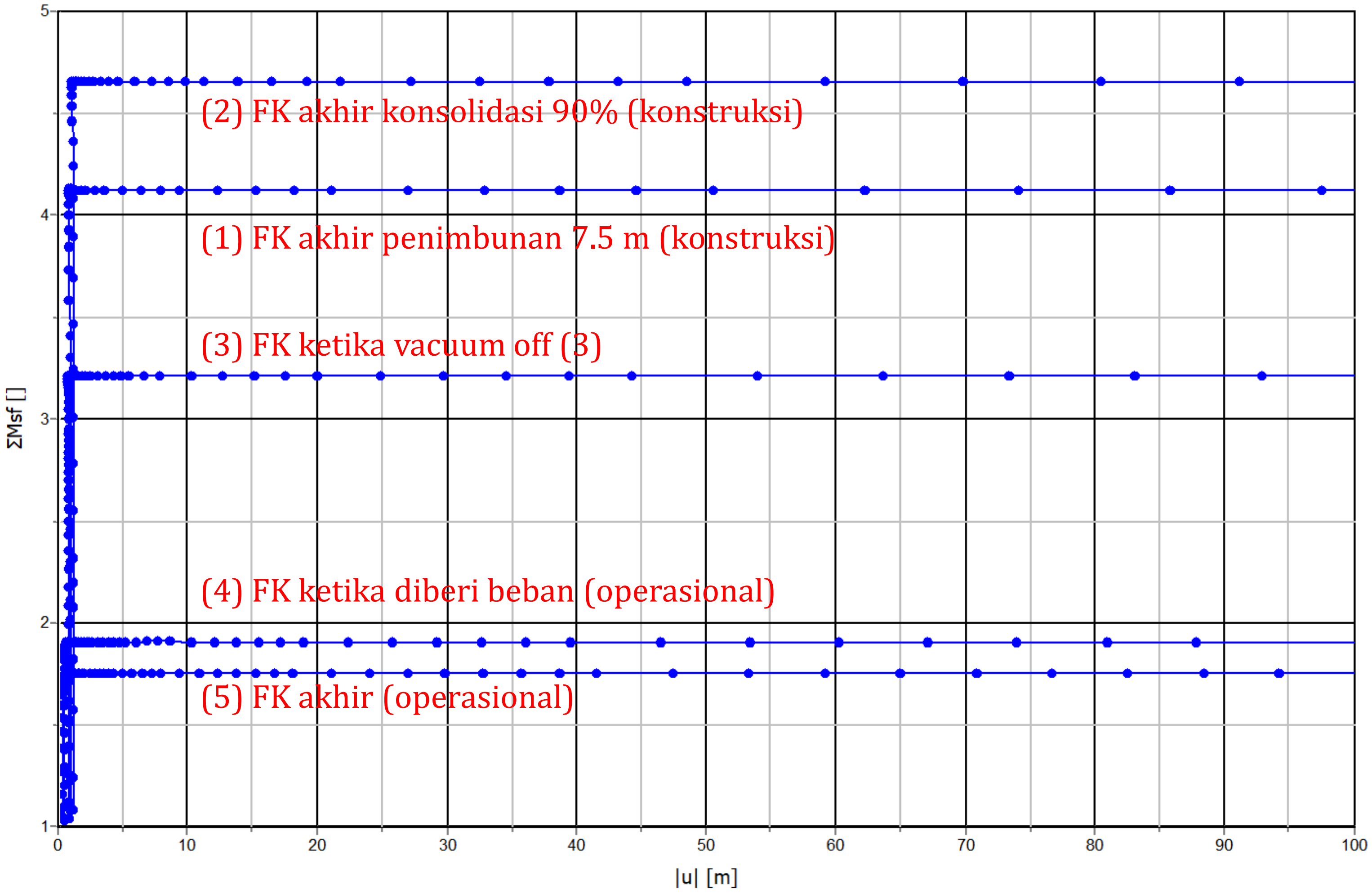
Case study 2: Kapalbetung Toll Road Project
PVD with Vacuum Preloading

STA. 31+500 – PVD + Vacuum



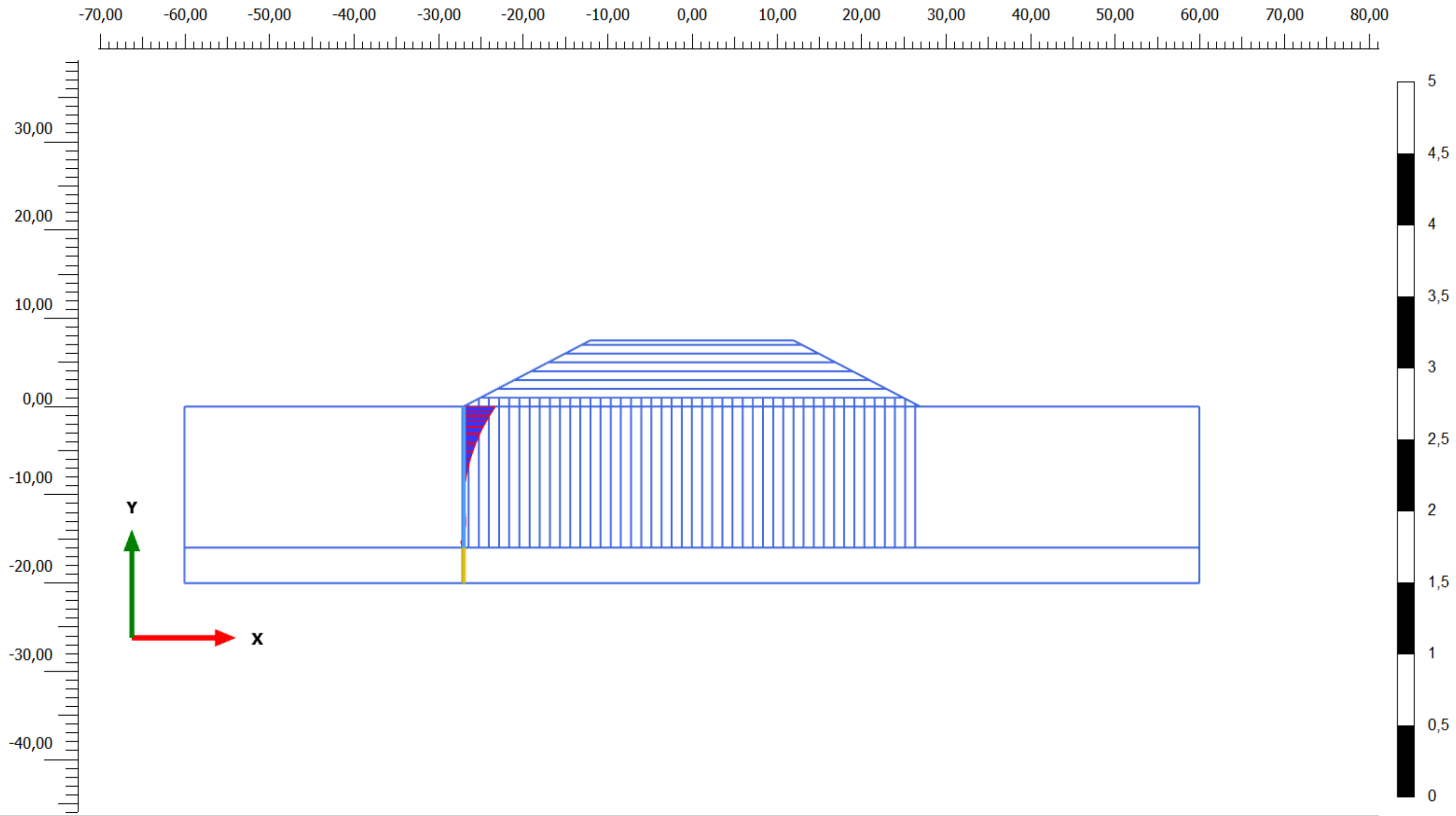
Case study 2: Kapalbetung Toll Road Project PVD with Vacuum Preloading

STA. 31+500 – PVD + Vacuum



Case study 2: Kapalbetung Toll Road Project
PVD with Vacuum Preloading

STA. 31+500 – PVD + Vacuum



Total displacements u_x (scaled up 15,0 times) (Time 35,00 day)

Maximum value = 0,2522 m

Minimum value = -0,02146 m

Conclusion

- Numerical methods provide a less restrictive analysis compared to analytical methods, such as: advanced soil models, multiple drain analysis, multi layer subsoil condition, staged construction, non uniform load distribution (lateral displacement and stability analysis), variation of drains configuration (spacing and floating), effects of a sand mat and reinforcement with geosynthetic, etc.
- The equivalent 2D plane strain approach is sufficient to conduct multi-drain analysis in large projects, where the application of a 2D plane strain model is efficient both from a computational time point of view and with respect to the accuracy of the predictions (compared with a full 3D model and 2D Axisymmetric model).
- For floating PVDs, the drain length can be reduced to 70% (double drained) and 80% (single drained) of the entire soft clay thickness without significantly affecting the consolidation process.
- The Advantages Vacuum Consolidation Method compare with conventional soil preloading are Lower or no surcharge, better control of stability, less lateral movement, less heavy equipment, shorter construction time.

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...Thank You...

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INSTITUT TEKNOLOGI NASIONAL

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Rabu, 23 September 2020

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Itenas LPPM

Iwan Juwana, ST.,M.EM., Ph.D.

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