April 18, 2007

GeoMedia File: G1221

The Corporation of the District of Maple Ridge
11995 Haney Place
Maple Ridge, BC

Attention: Mr. Terry Fryer, P.Eng.

Re: **Design Brief**
Dike Repair Works
Sta. 1+000 to 3+060 Albion Dike
River Road, Opposite Tamarack Lane, Maple Ridge, BC

1.0 INTRODUCTION

GeoMedia Engineering Ltd. presents herein a design brief for the above referenced project. A 2.5 km section of the Albion Dike which is the subject of this report extends from Kanaka Creek Park to just short of 236 Street, in Maple Ridge, BC. The purpose of the design brief is to provide recommendations to enable emergency repairs of the dike to be carried out before the arrival of the 2007 freshet of the Fraser River, which is anticipated to peak by about May 15 of this year. Due to the emergency nature of this work, the preparation of this letter was based on limited information conducted during approximately 4 working days. As such, further design and a review of the recommendations of this report may be needed when construction of the dike is under way.

The dike was noted to be in a substandard condition in an assessment carried out by AMEC during 2004. Deficiencies noted along areas of the dike included a narrow crest width, critically low crest elevations, oversteepened landside and riverside slopes, and stands of large trees on landside and riverside slopes. Further deterioration of sections of the dike has occurred since then.

The District of Maple Ridge has now initiated work to restore the dike closer to a compliant condition. It is understood that these works will include:

1. Grubbing and stripping in order to remove unsuitable soils, slumping soils and tree bulbs from the dike slopes.
2. Flattening of landside dike slopes.
3. Repair of the riverside slopes around an oxbow of Kanaka Creek, where ongoing erosion has created a cut bank.
4. Repair of the most critically unstable and scoured areas along the riverside slopes along the Fraser River.

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2.0 TENTATIVE WORKS

General

- Dike crest elevations undulate throughout the alignment. Raising of portions of the dike crest to el. 6.3 m is proposed. This will be carried out within the budgetary constraints.

Sta. 1+000 to 1+040

- The dike crosses Terasen ROW 23263. It is understood that works within the Terasen ROW requires prior approval before works can proceed in this area. Proposed works include raising of the crest elevation with associated widening on the western side (waterside).

Sta. 1+040 to 1+240

- Riverside slopes are to be grubbed and stripped of trees and flattened. The Contractor is currently sourcing out high fines material for the riverside widening.
- The dike crest is to be widened to 4.0 m.

Sta. 1+240 to 1+580

- Landside slopes are to be grubbed and stripped of trees and flattened. The Contractor is currently sourcing out a well-graded 75 mm ± pit run sand and gravel for the landside widening. Tentative plans are to provide a non-woven filter cloth (6oz Propex 4553) over the prepared slopes prior to placement of the pit run sand and gravel.
- Riverside slopes along a 150 m ± section of the Kanaka creek oxbow are to be grubbed and stripped of trees and slumping soils and replaced with approved materials. Tentative plans are to provide a non-woven filter cloth (8 oz Propex 4510) over the prepared slopes followed by 300 mm minus shotrock, similar to that carried out during 2005 for an adjoining section of the dike. The resulting slopes will be approximately 1.5H:1V (Horizontal:Vertical) – with a toe berm for stability.
- The dike crest is to be widened to 4.0 m.

Sta. 1+580 to 3+060

- Landside slopes are to be grubbed and stripped of trees and flattened. The Contractor is currently sourcing out a well-graded 75 mm ± pit run for the landside widening. Tentative plans are to provide a non-woven filter cloth (6oz Propex 4553) over the prepared slopes prior to placement of the pit run sand and gravel.
- The dike crest is to be widened to 4.0 m.
- Riverside slopes which are critically unstable and scoured are to be repaired. Slopes are to be flattened to approximately 1.5H:1V to 2H:1V. Tentative plans are to provide a non-woven filter cloth (8 oz Propex 4510) over the prepared slopes followed by a

300 mm layer of 150 mm minus shotrock, followed by a 900 mm layer of riprap that is keyed into or launched onto the riverbed.

3.0 SEEPAGE

As groundwater flows through soil, it exerts a seepage force. This seepage force is directly proportional to the hydraulic gradient. The hydraulic gradient is the ratio of hydraulic head loss to the flow path length over which that loss occurs. If the hydraulic gradient, where the seepage flow daylights, exceeds a certain critical value, then a boiling condition will develop. Once this happens, seepage discharging will begin to erode the soil in a process called internal erosion. Internal erosion begins at the point of the seepage exit and can rapidly retrogress towards where the hydraulic head is high in a process called piping. As this occurs, the flow path length decreases and the hydraulic gradients increase. Therefore, the rate of piping also increases. Once the pipe reaches the impounded water, flow and erosion become so severe that the dike becomes breached and general failure occurs.

The critical gradient at which the piping process can be initiated by upward flow can be estimated as follows:

$$\mathbf{i_C = (G\text{-}1)/(1+e)}$$

where $\mathbf{i_C}$ is the critical exit gradient, **G** is the specific gravity of solids in the soil (typically 2.65) and **e** is the void ratio of the soil. Field measurements indicate that critical hydraulic gradients for soils can range from about 0.54 to 1.02 with 0.85 considered a typical value. A factor of safety of 3 or 4 is often considered appropriate, meaning that the maximum upward gradient may be in the order of 0.25.

Hydraulic gradients where seepage discharges laterally from the downstream dike, if too high, can lead to a retrogressive failure of the embankment fill. The seepage force, computed as the product of the hydraulic gradient and the unit weight of water, is the frictional drag exerted by groundwater movement through the soil. The seepage force required to cause soil erosion and retrogressive internal erosion depends on the type of soil and the slope gradient. For example, a uniform very fine sand would be much more susceptible to seepage erosion than would a well-graded sand and gravel. The benefit of the gravel component is that, while sand particles might be initially eroded by seepage discharge, the remaining gravel sizes would tend to form a filter to prevent the additional loss of sand. With a uniformly graded material, no such self-healing filter could form.

Over the week of April 9 to 13, 2007, hand pits were carried along the landside slopes and boreholes were carried out on the dike crest to address questions regarding the consistency of the dike fills and foundation soils. Those questions, the findings of the site investigations and a commentary on seepage through and under the dikes are outlined below:

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<u>What is the nature of the dike fills existing on the landside slopes?</u>

This question is of significance in order to select the gradation of the drainage layer to be placed onto the landside of the existing dikes.

Handpits were carried out at approximately 200 m spacings between Sta.1+600 to 3+060. Sieve analyses on selected soil samples indicate a wide variation of materials – from a sand&gravel (coarse pit run material) to a sand&silt with some gravel, to a silt with trace to some sand.

In relation to the bulk dike fill, the new drainage layer should be filter compatible and comparatively more pervious. This would serve to prevent the migration of fine soil particles from the base material into the filter material, as well as prevent the formation of excessive internal head losses which can result in internal erosion and piping. Note the following:

1. The permeability of selected materials should increase from the core out to the landside slope to readily transmit seepage without excessive head losses and to reduce seepage from emerging on the landside slope.

2. The gradation of selected material should be coarse and well-graded enough to reduce the potential for softening/slumping should seepage through an embankment merge on the landside slope. Poorly graded or uniform fine sands are materials to be avoided.

A graded filter designed for each soil type is possible. For example, a sandy silt or silty fine sand could be covered by a concrete sand with narrow gradation limits, followed by pit run sand, followed by pit run gravel. Constructability issues do not serve the tight time constraints of this emergency project, not to mention the practicability of this approach. Therefore, it is recommended that the drainage layer be a coarse well-graded pit run separated from the bulk dike fills by a non-woven geotextile fabric provided with a sufficient overlap. At this present time, sieve test results from a Maple Ridge gravel pit have been judged too gap-graded and fine for this purpose, and another gravel pit in Coquitlam is currently being sourced.

<u>What is the nature of the foundation soils underlying the dike?</u>

This question is of significance in order to address the potential for dike underseepage, and to arrive at methods in which to control underseepage.

A total of seven boreholes were carried out along the dike crest using a track mounted drilling vehicle to depths ranging from 6 to 9 m below the dike crest elevation. Based on the results of the widely spaced boreholes the following is noted:

1. The existing dike fills generally consisted of sand, sand&gravel, and silt. Borehole BH6 encountered a surface hogfuel layer.

2. The dike foundation soils consisted of soft silt and clayey silt in Borehole BH1 in the area of Terasen ROW 23263. The dike foundation soils encountered in the remaining boreholes (BH2 to BH6) generally encountered fine grained sand and silty sand.

Seepage control measures applicable for this project includes seepage berms possibly with pervious toe trenches on the landside of the dike which would be designed to prevent heaving of the upper landside soil layers and rupture of weak spots with a resulting concentration of seepage flow.

Seepage analysis was carried for five scenarios out using the 2-D finite element program Seep-W. In lieu of more detailed information including closely spaced boreholes and soil permeability testing that would have otherwise been gathered in non-emergency situations, the analysis relied more upon generalized soil profiles lumped into scenarios or what-ifs. The finite element analyses yielded estimates of seepage volumes and hydraulic gradients. Of particular interest in terms of the safety of the dikes under design flood conditions are exit gradients (where seepage daylights at the ground surface) at the downstream face and toe of the dikes, and in the foundation soils to some distance downstream of the dikes.

For all scenarios, seepage breakout on the downstream face of the dyke fill is predicted. Defence against seepage induced sloughing and erosion at the landside would be provided by:

- The coarse nature of the well-graded, clean 75 mm minus sand and gravel placed on the landside dike slopes, which would offer more resistance to sloughing than a uniform sand.
- The 3H:1 V slope on the landside dike slopes.
- The vegetation cover of grasses on the dike slopes.

Scenario 4 shows a dike underlain by a soil of high permeability, with a relatively thin upper stratum of low permeability soil. Excessive head loss and high hydraulic gradients are shown. Seepage control can be made with the use of a pervious toe trench, shown as Scenario 5. Further review of actual site conditions, in combination with revision of the toe trench size and flattening of landside slopes may be required during construction. This is considered important between approximately Sta.1+240 to 1+580, where the existing dike rises some 3 m above the landside ground surfaces.

Due to the inhomogeneous soil conditions noted in the test holes, the presence of permeable granular soil strata under the dike, the possible presence of roots in the dike body and the uncertainty of accurately modeling the exit gradients at the landside toe of the dike, it is recommended that the dike is afforded a 24 hour high water patrol during the time of flooding.

Monitoring should be carried out in order to highlight signs of excessive seepage emanating from the landside of the dike. Where heavy seepage is resulting in sand boils, piping or displacement of soil particles, two methods may be employed. The first method is to ring the sand boils with sand bags. The second method, which should be immediately implemented if boiling is widespread, is to lay a seepage berm of sand over the area as soon as possible. This blanket may initially be placed to a thickness of 1 m and may extend several metres from the landside toe of the dike. Further reference is to be made to **Guidelines for Management of Flood Protection Works in British Columbia** (Ministry of Environment, Land and Parks, 1999).

Further to the above, seepage control measures that could also be implemented includes placement of waterside impervious layers. Although this is recommended for the dike widening from approximately Sta.1+040 to 1+240, this approach is not appropriate for the oxbow of Kanaka Creek where global stability is a concern, much of the fill placement will take place under water, and relatively steep slopes (1.5H:1V) may be necessary to keep the fill volumes within budgetary constraints. It is expected that the dike repair works in this area will include removal of unsuitable materials and placement of a non-woven geotextile and zones of 300 mm minus crushed rock and 50 kg class riprap to form a 1.5 H:1V waterside slope. This is generally similar to what was carried out for the 2005 emergency works.

4.0 RIPRAP ARMOUR

Fraser River

Document EM 1110-2-1601 entitled **Hydraulic Design of Flood Control Channels** (USACE, 1991) indicates a relationship of D_{30} of riprap to permissible water flow. The attached calculations based on this procedure, indicates the following:

- a 2H:1V slope armoured with 100 kg class riprap (MoTH **Standard Specifications for Highway Construction**, Section 205) would withstand average water flow velocities of 2.5 m/s. An upgrade in riprap class would be required for higher velocities.

- a 1.5H:1V slope armoured with 100 kg class riprap would withstand average water flow velocities of 2.0 m/s. An upgrade in riprap class would be required for higher velocities.

For large contracts including permanent works on the entire riverside section of dike, it is recommended that the D_{100}, D_{85}, D_{50}, D_{15}, as well as the W_{100}, W_{85}, W_{50} and W_{15} be provided to clearly define engineering requirements.

Document EM 1110-2-1601 indicates that the minimum layer thickness of the rock layer is $1.5xD_{50}$ or $1.0xD_{100}$ of the riprap. The minimum required layer thickness for a 100 kg class riprap would be approximately 1.5×450mm plus an additional thickness of 300 mm giving consideration to impact by ice debris, which totals approximately 0.9 m in thickness (coincidentally $2 \times D_{50}$ of the 100 kg class riprap). It is recommended that the riprap be underlain by 300 mm of 150 mm minus shotrock which, in turn, is underlain by a non-woven filter cloth (8 oz Propex 4510) placed over the prepared slopes.

Document EM 1110-2-1601 indicates that the toe protection may be provided by either 1) extending the toe to the maximum scour depth or 2) placing launchable stone defined as stone that is placed along expected erosion area at an elevation above the zone of scour. The stone will be undermined and will slide down the slope, arresting the erosion. The launching apron is normally 5T along the top and 1.5T thick, where 'T' is the layer thickness in metres.

Kanaka Creek

Pending further review of the flow velocities of the Kanaka Creek, it is recommended that a non-woven filter cloth (8 oz Propex 4510) over the prepared slopes followed by a 300 mm minus riprap, followed by a 400 mm thick layer of 50 kg class riprap. The resulting slopes will be approximately 1.5H:1V (Horizontal:Vertical) and will likely include a toe berm to satisfy global stability requirements. A typical section of riverside works on the Kanaka Creek oxbow is attached (with toe berm not shown).

It is recommended that the rock toe of all waterside dike slopes be reviewed during periods of low water and reconstructed to recognized standards in areas where deficiencies are detected.

5.0 SLOPE STABILITY

Conditions that require slope stability analysis include end of construction, steady seepage from full flood stage and sudden drawdown. Computer software Slide 5.0 was used to model the steeper riverside sections along the Kanaka Creek where the highest and steepest permanent slopes may occur.

Steady Seepage From Full Flood Stage

This case will result when the water remains high long enough for the embankment to become saturated and a condition of steady seepage occurs. The results of the analyses suggested that the landside dike slopes have a Factor of Safety in the order of FS = 1.2 for the steady seepage case. This is lower than the accepted recognized Factor of Safety of 1.4. It is important that the dike drainage fills be clean and well-graded, and that they be compacted in place.

Sudden Drawdown

This case will result when the river rapidly drops before the slope can adequately drain and lower the line of saturation. The results of the analyses suggested that the riverside dike slopes have a Factor of Safety in the order of FS = 1.1 for the sudden drawdown case. This is an acceptable recognized Factor of Safety. This Factor of Safety was achieved using a 2 m wide and 1 m thick launching apron at the toe of the riprap layer.

Actual calculated Factors of Safety for both cases will be reviewed once the works progress. Some amendment to the proposed works may be required to achieve acceptable Factors of Safety.

6.0 VEGETATION

To meet recognized standards for a permanent dike structure, it is recommended that the trees, root bulbs, organic soils, and other unsuitable soils be removed from the dike in accordance with Figure 3 and 4 of **Environmental Guidelines for Vegetation Management on Flood Protection Works to Protect Public Safety and the Environment** (BC Ministry of Environment, 1999). If existing trees are left in place, the risk of piping through the dike structure may be greater than compared to a structure built to recognized standards.

7.0 TERASEN GAS ROW CROSSING

A short section of the dike crosses Terasen ROW near Station 1+040. Subsurface information from Borehole 1 advanced outside the south ROW boundary encountered the following soil conditions:

- 0 – 1.2 m: Sand fill, fine to medium grained sand, trace silt, some gravel, loose, light brown.
- 1.2 – 3.1m: Silt fill, trace to some sand, low plasticity, stiff, light brown, moist.
  - below 1. 8 m, firm.
  - below 2.4 m, soft

  Moisture contents from this layer ranged from 37 to 41%
- 3.1 – 9.2 m: Silt, some clay, trace sand, stiff, light brown, moist.
  - below 3.4 m, firm,
  - below 4.3 m, very soft.
  - below 6.1 m, clayey with occasional sandy lenses.

  Moisture contents from this layer ranged from 38 to 42%

Settlement analysis was carried out using FoSSA 2.0. This program calculates the stress distribution under embankments with the resulting consolidation settlements. The results of the

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analysis indicate that settlements would be in the order of 0.07 m or 7 cm with a crest increase of 0.5 m and a dike widening of 1.8 m in the Terasen ROW.

8.0 IMPORTANT FUTURE CONSIDERATIONS

The design brief provides recommendations for upgrading the dike before the 2007 Fraser River freshet. If future works down the road result in significant raising and landside widening of the dike, it should be recognized that the landside drainage layer would be buried deeper within the core of the dike. Future designs may require that the coarse drainage layers be shifted or 'rolled over' closer to the landside of the much larger dike.

Consideration should be given to shifting the entire dike alignment some distance landward to avoid pushing significant volumes of fill into the river when the riverside slopes are repaired.

9.0 CLOSURE

Recommendations presented herein are based on the geotechnical evaluation of the findings of limited test hole information and a reconnaissance of the site. The material in this design brief reflects GeoMedia's best judgement in light of the information available to GeoMedia at the time of preparation of the report. If conditions other than those are noted during subsequent phases of the project, GeoMedia should be notified and given the opportunity to review and revise the current recommendations, if necessary.

This report has been prepared for the exclusive use of The District of Maple Ridge and its consultants for the specific application to the development described within this report. Any use which a third party makes of this report, or any reliance on or decisions made based on it are the responsibility of such third parties. GeoMedia accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report. We appreciate the opportunity to be of service to you. If you have any questions regarding the contents of this report, or if we can be of further assistance to you on this project, please call the undersigned.

Yours truly,

GeoMedia Engineering Ltd.

Darryl Grandberg, P.Eng.
Geotechnical Engineer
dg/

Reviewed by:

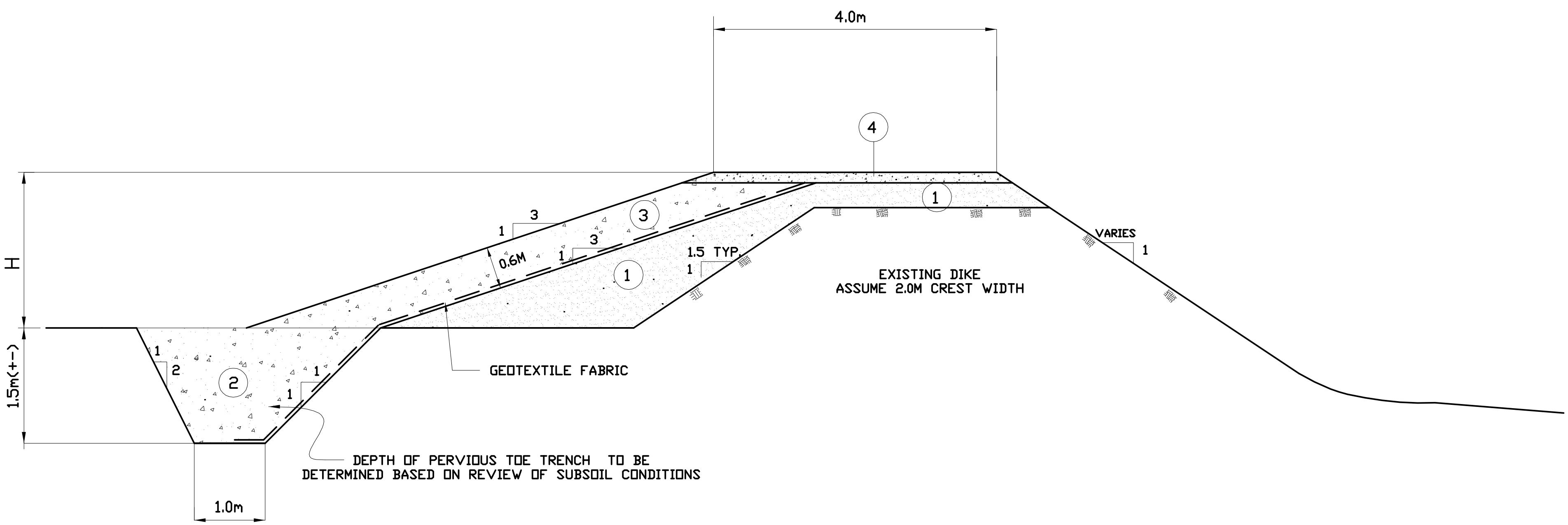
Mike Eivemark, P. Eng.
Senior Engineer

Appendix A

Drawings


Sta 1+000
Sta 1+600
Sta 2+000
Sta 3+000
Kanaka Way
Tamarack Ln
105 Ave
Lougheed Hwy
River Rd
McKay Ave
236 St
Fishermans Rd
© 2007 Navteq
© 2007 Europa Technologies
Image © 2007 DigitalGlobe
©2007 Google™
Pointer 49°11'36.88" N 122°34'39.61" W elev 5 m
Streaming |||||||||| 100%
Eye alt 1.83 km

1. GENERAL DIKE FILLS - FILTER COMPATIBLE WITH EXISTING DIKE FILLS WITH THE SAME OR HIGHER PERMEABILITY
2. SAND AND GRAVEL PERVIOUS TOE TRENCH
3. SAND AND GRAVEL DRAINAGE LAYER - DRESSED WITH TOPSOIL AND HYDROMULCH
4. HIGH FINES BASE COURSE 150mm THICK



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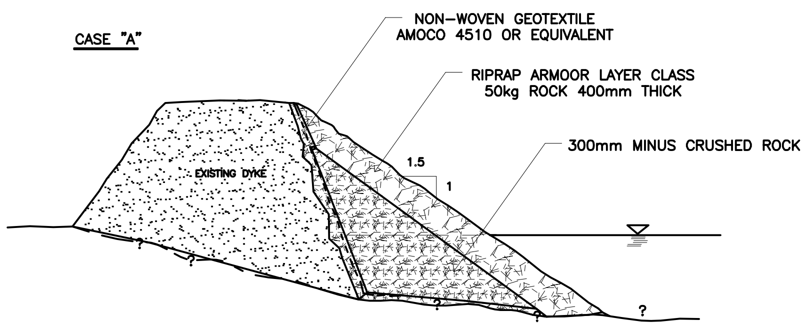
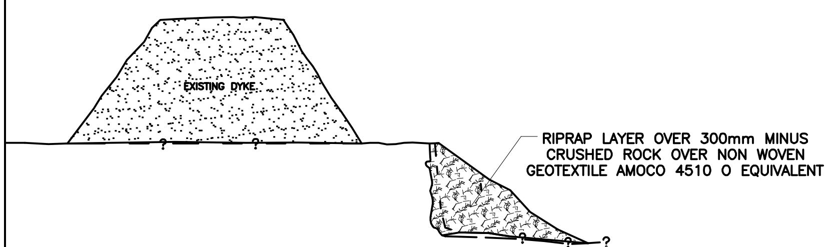
DESIGN	DG	CLIENT	THE DISTRICT OF MAPLE RIDGE	PROJECT NO.	G1221
DRAWN	TM	TITLE	TYPICAL DIKE CROSS-SECTION	DRAWING NO.	G1221-1
CHECKED	ME	PROJECT	ALBION DIKE UPGRADE	DATE	04 APRIL 07
DATE ISSUED:		LOCATION	ALBION DIKE - STA 1+240 TO 3+060	ISSUE/REVISION	

CASE "A"



CASE "B"



Notes:

1 Responsibilities

1.1 The Geotechnical Engineer shall visit the site at regular intervals to verify that the construction of the dike is consistent with the intent of the design.
1.2 The Contractor is responsible for coordinating and scheduling construction review and material testing by the geotechnical engineer with a minimum of 24 hrs notice.
1.3 The Contractor shall report to the Geotechnical Engineer any conditions inconsistent with the design drawings. Note that a resolution due to such inconsistencies may possibly result in a delay of construction.
1.4 The Contractor shall confirm that the works do not interfere with buried pipes of other structures buried within the existing dike.
1.5 The dike shall be constructed to written instructions and not to scaled dimensions if such a discrepancy exists.

2 Materials

2.1 New fills placed on the dike are to be approved by the geotechnical engineer with respect to filter compatability with other materials, gradation, compaction or any other applicable material property.

3 Construction

3.1 Riverside slopes shall be stripped and cleared of soft soils, organic soils, roots, root bulbs, and any other unsuitable materials. The prepared subgrade for placement of new fill shall be approved by the geotechnical engineer.
3.2 The method of removal of soft sediments or other unsuitable materials below the water line shall be approved by the relevant approving authorities. Overexcavation or displacement of soft sediments are options at this site.

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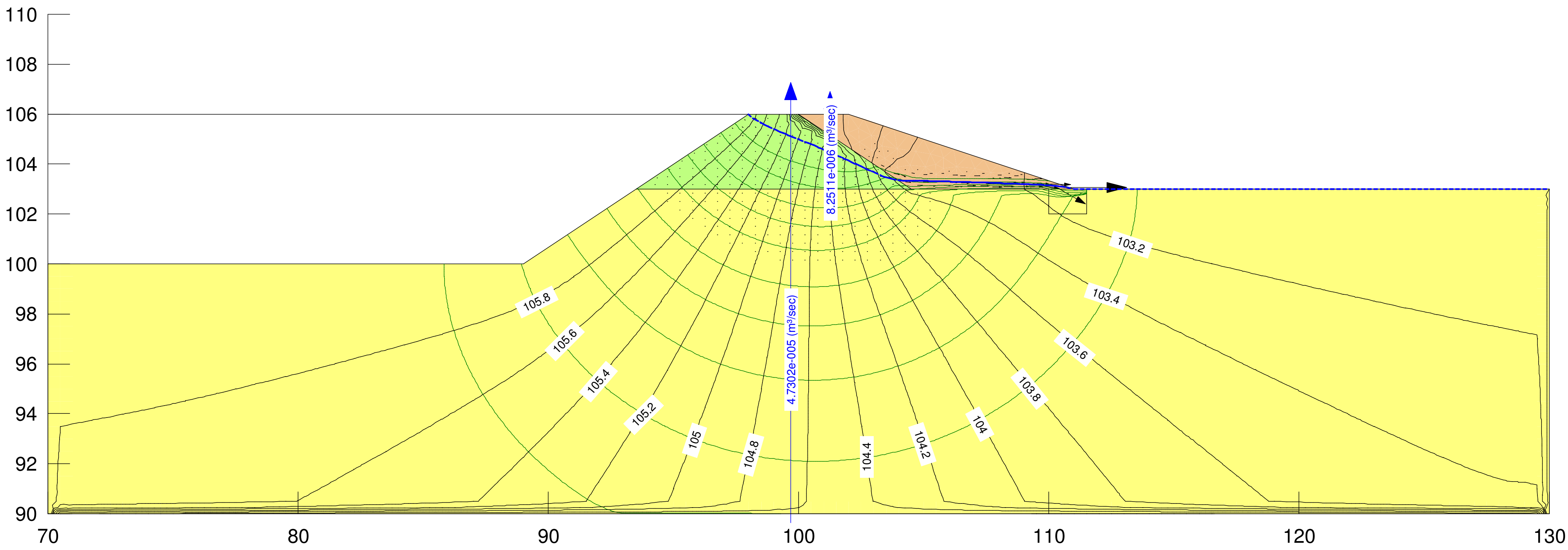
DESIGN	DG	CLIENT	THE DISTRICT OF MAPLE RIDGE	PROJECT NO.	G1221
DRAWN	TM	TITLE	TYPICAL DIKE CROSS-SECTION	DRAWING NO.	G1221-2
CHECKED	ME	PROJECT	ALBION DIKE UPGRADE	DATE	04 APRIL 07
DATE ISSUED:		LOCATION	ALBION DIKE - STA 1+220 TO 1+380	ISSUE/REVISION	

Appendix B

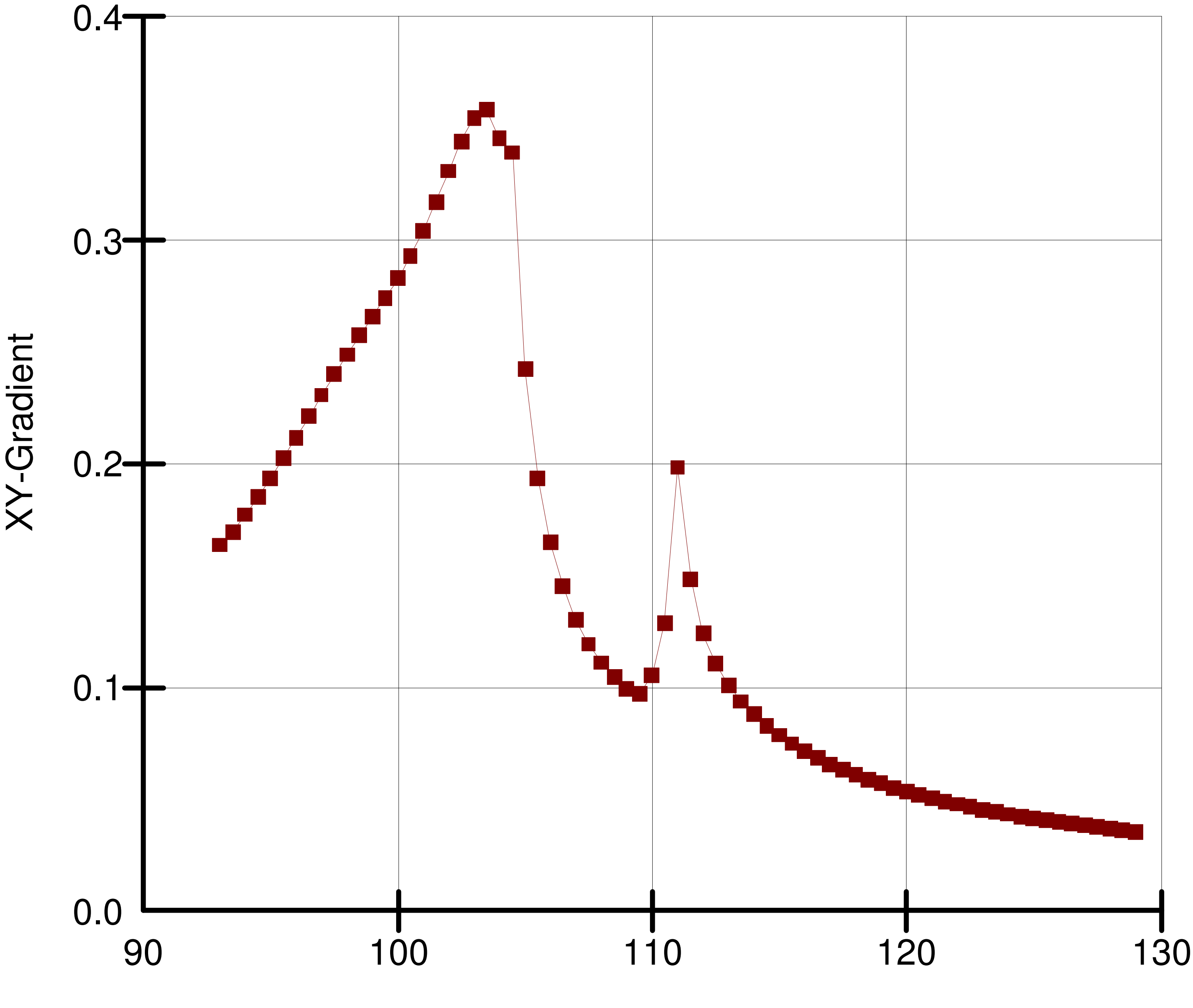
Seepage Analyses

Scenario 1

Kdyke = Kfoundation = 1x10-5
Kdrainage layer = 1x10-3



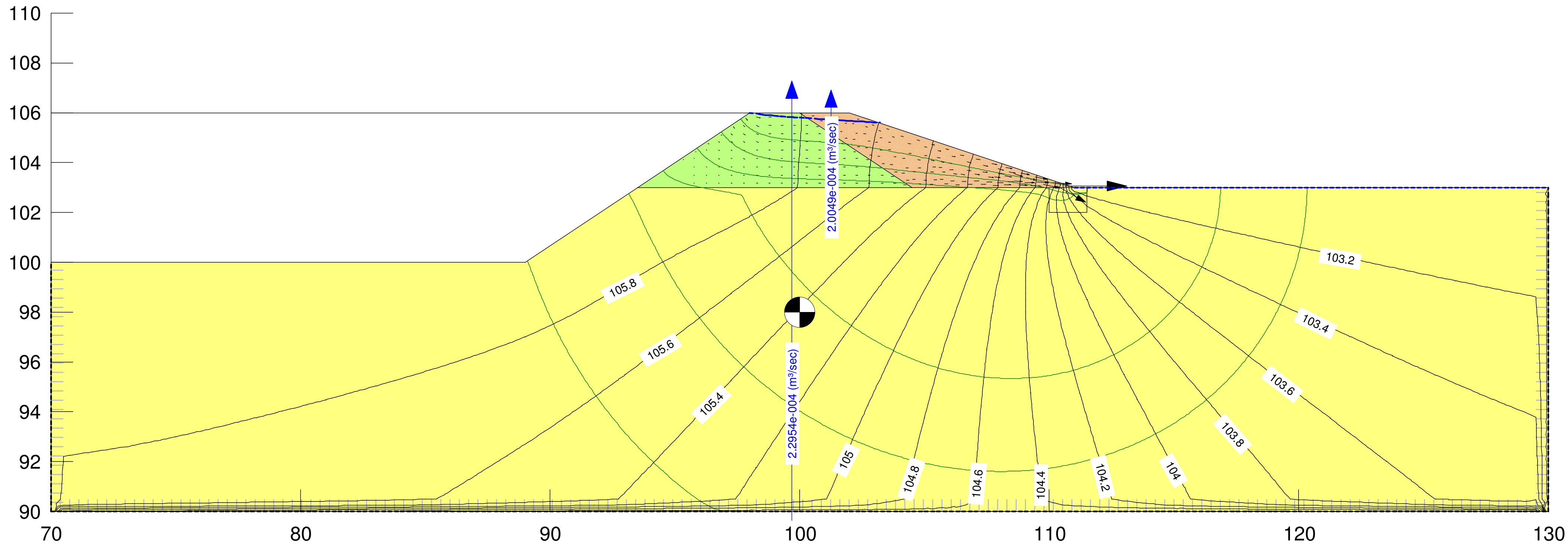
Scenario 1   XY-Gradient

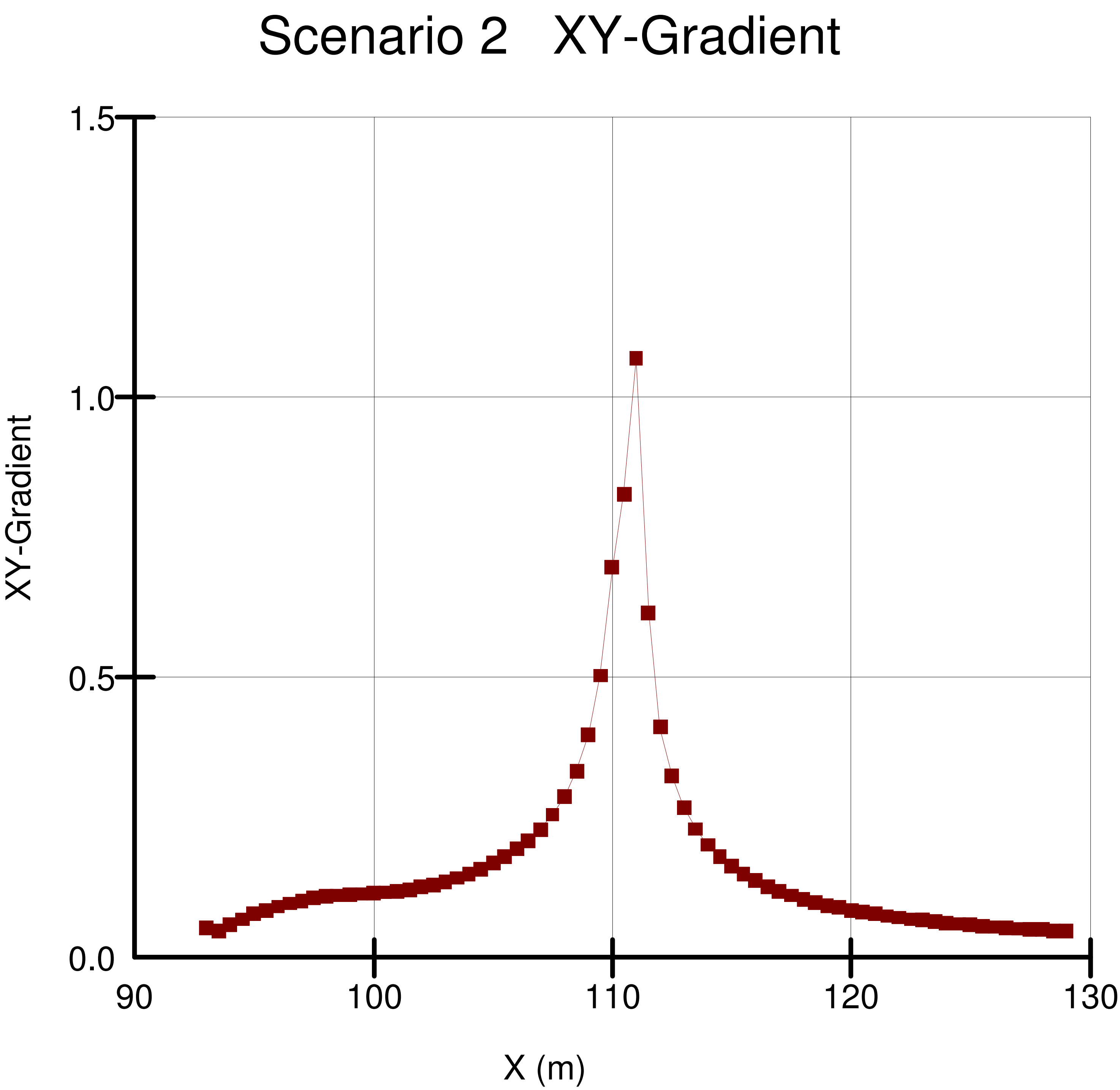


Scenario 2

Kdyke >> Kfoundation

K dyke = 1x10-3 m/s
Kfoundation = 1x10-5 m/s
Kdrainage layer = 1x10-3 m/s

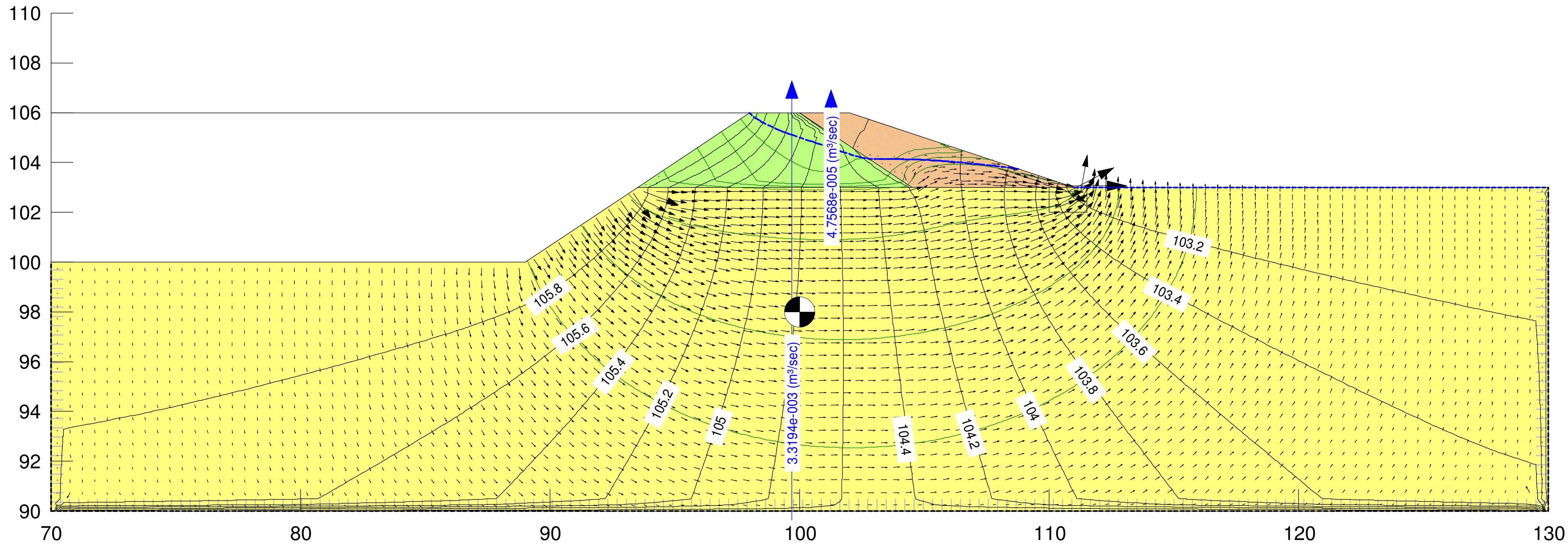


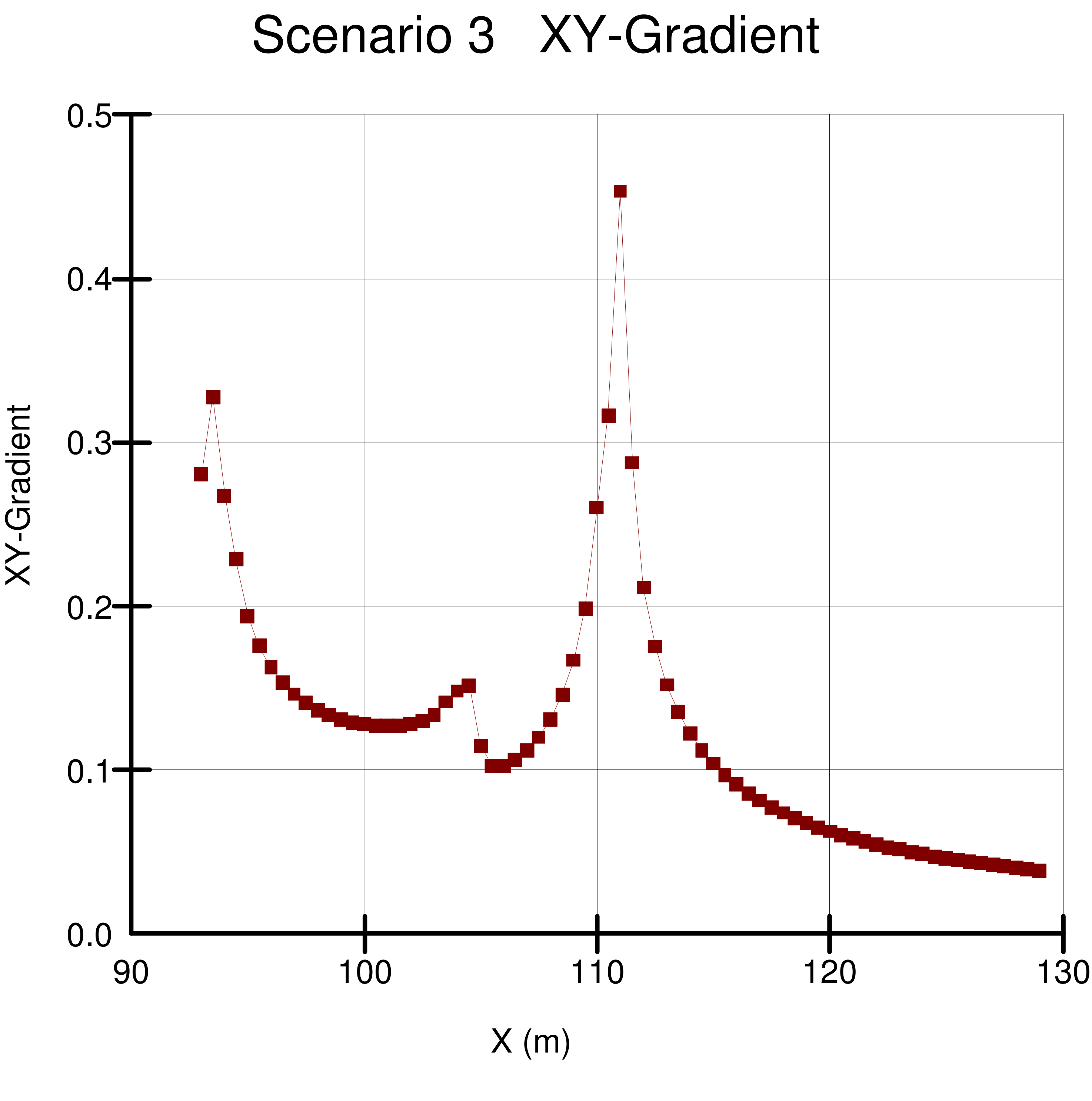

Scenario 2 XY-Gradient
XY-Gradient
1.5
1.0
0.5
0.0
90
100
110
120
130
X (m)

Scenario 3

Kfoundation >> Kdike

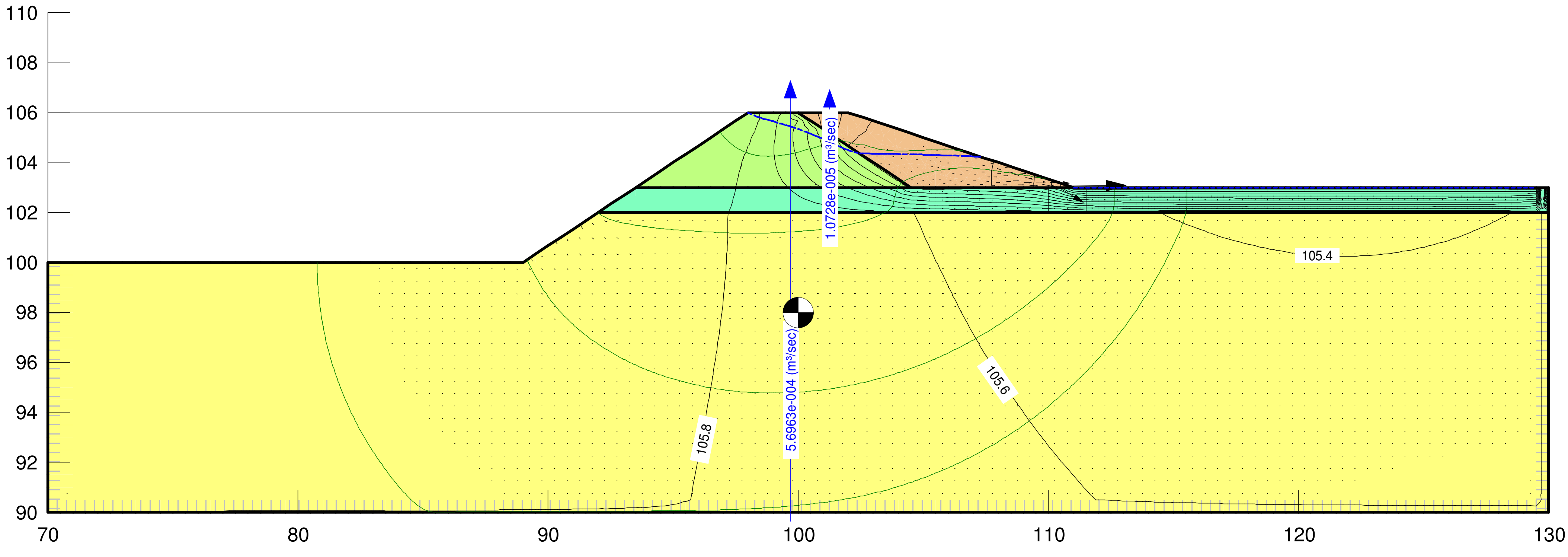
K dyke = 1x10-5 m/s
Kfoundation = 1x10-3 m/s
Kdrainage layer = 1x10-3 m/s



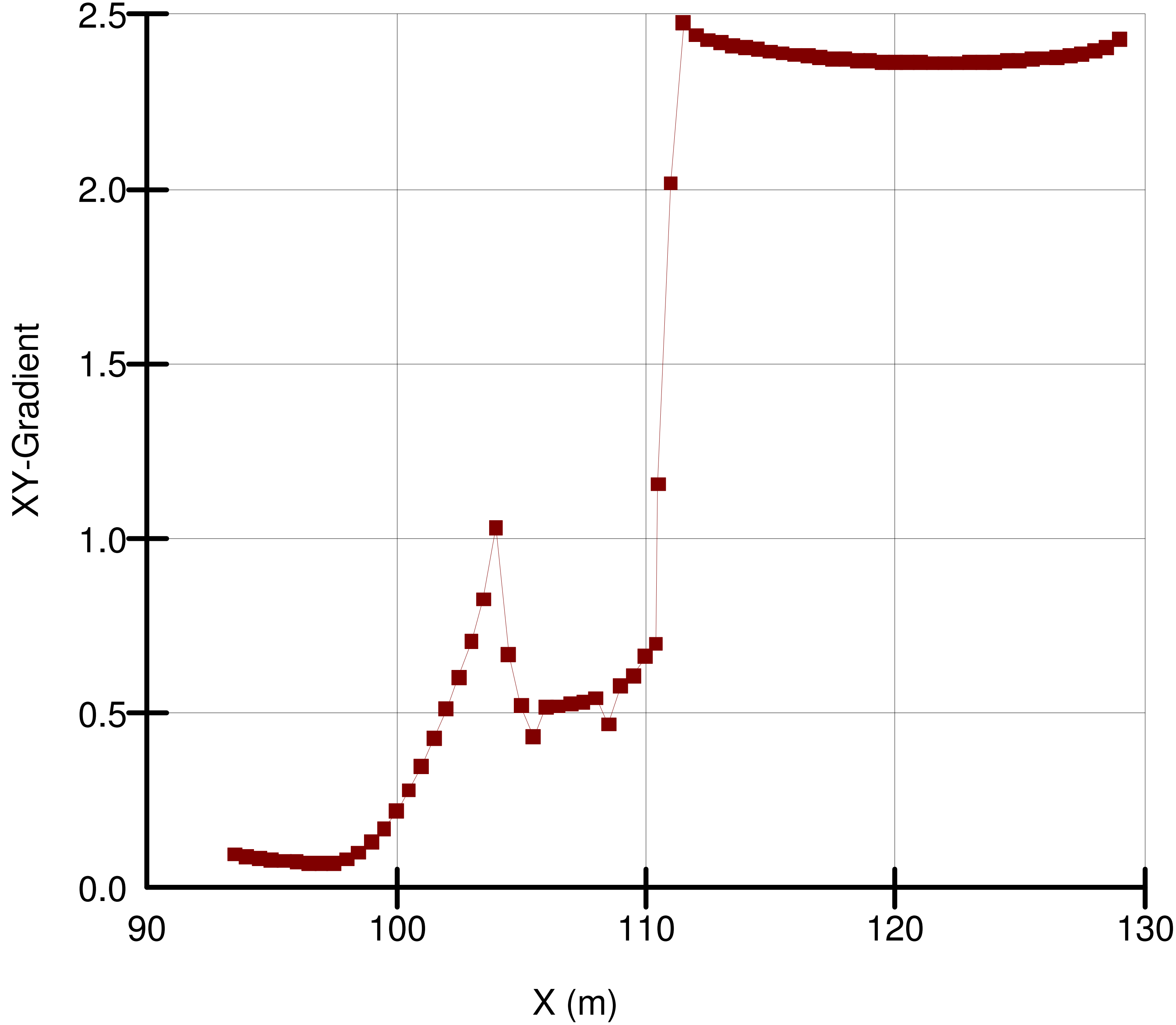

Scenario 3 XY-Gradient
XY-Gradient
0.0
0.1
0.2
0.3
0.4
0.5
90
100
110
120
130
X (m)

Scenario 4

Kdyke = 1x10-5
Kfoundation = 1x10-3
Kdrainage layer = 1x10-3
Ktop stratum = 1x10-5



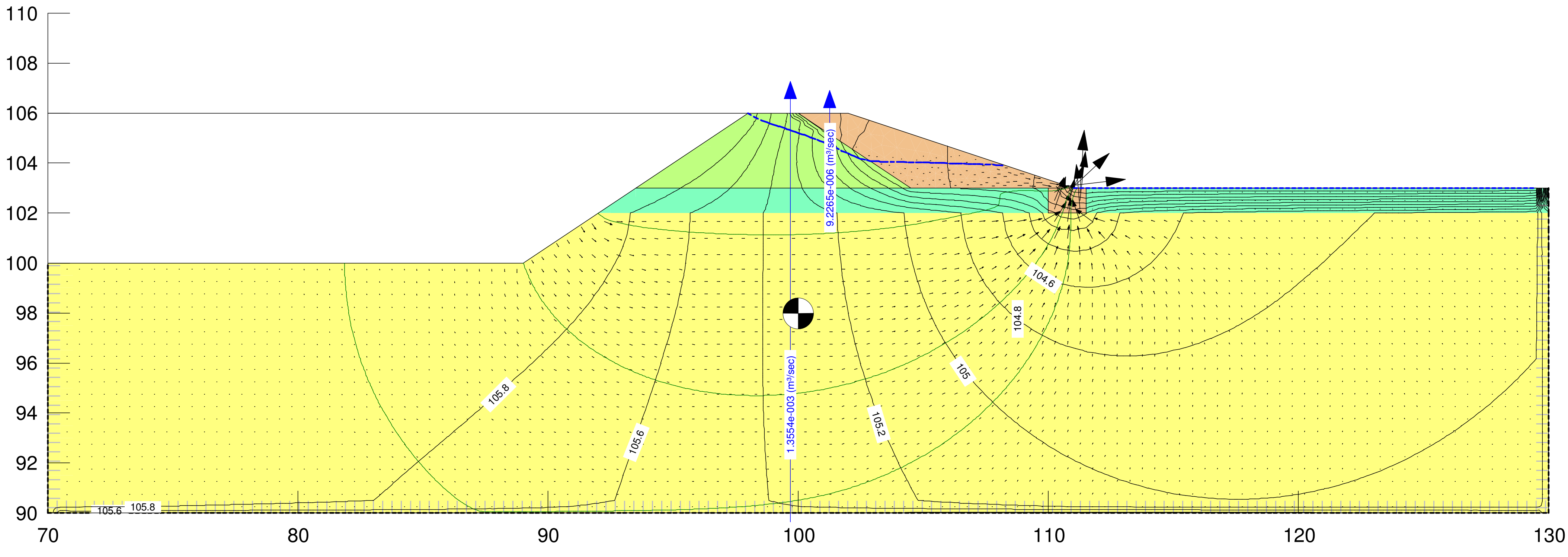
X-Y Gradient - No Pervious Toe Trench

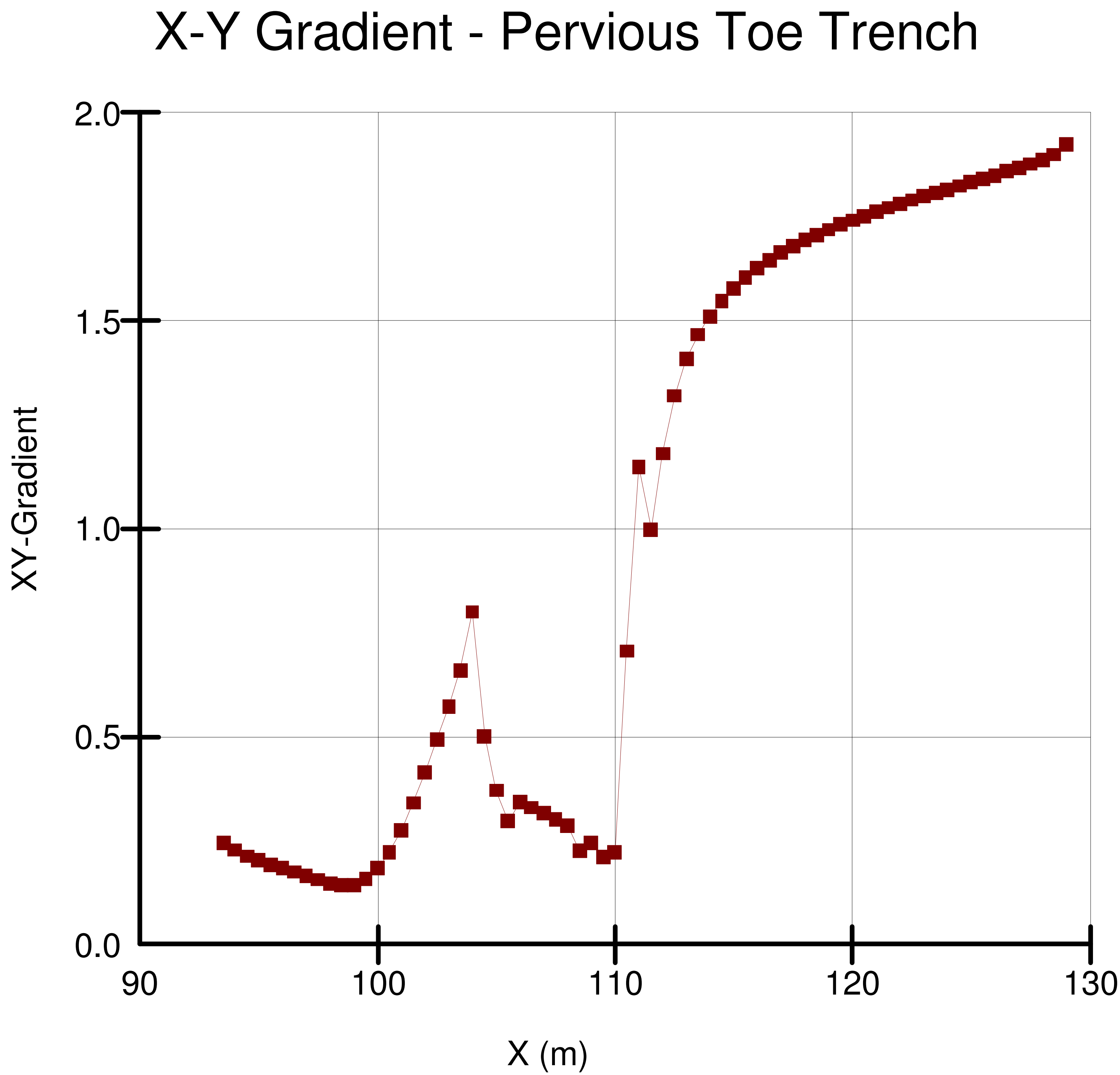


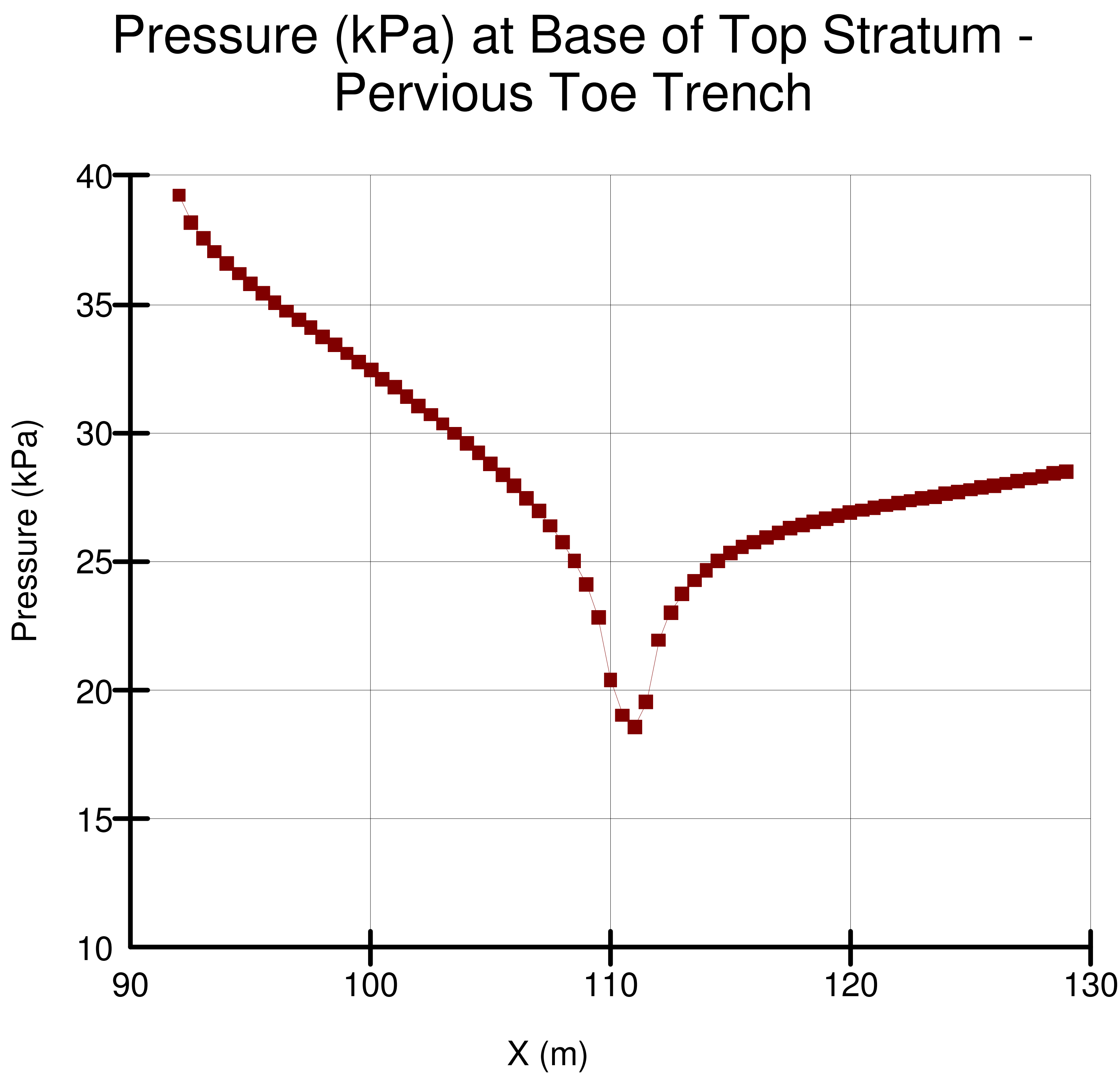
Scenario 5

Pervious Toe Trench

Kdyke = 1x10-5
Kfoundation = 1x10-3
Kdrainage layer = 1x10-3
Ktop stratum = 1x10-5




X-Y Gradient - Pervious Toe Trench
XY-Gradient
X (m)
0.0
0.5
1.0
1.5
2.0
90
100
110
120
130


Pressure (kPa) at Base of Top Stratum -
Pervious Toe Trench
Pressure (kPa)
X (m)
40
35
30
25
20
15
10
90
100
110
120
130

Appendix C

Riprap Armour Calculations

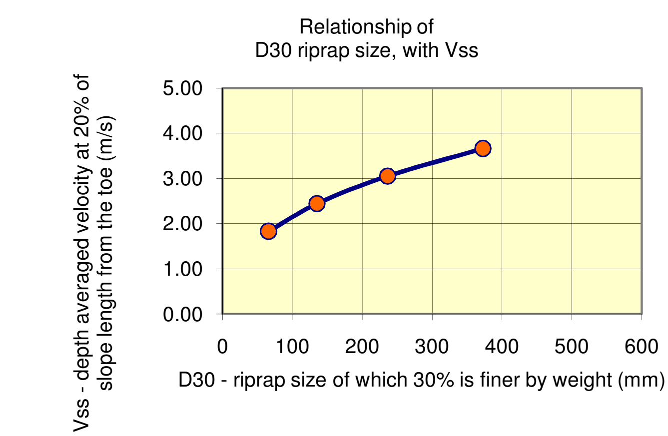
GeoMedia Engineering Ltd.
Stone Size Relations
Reference: U.S. Army Corps of Engineers, "Hydraulic Control of Flood Control Channels - Change 1", 1994

The basic relation for representative stone size in straight and curved channels applicable for slopes of 1.5H:1V, or flatter, is:
$D_{30}=S_f C_s C_v C_t d\ [\ (\gamma w/(\gamma s-\gamma w)^{1/2}\quad V_{ss}/(K_1 g d)^{1/2}\ ]^{2.5}$

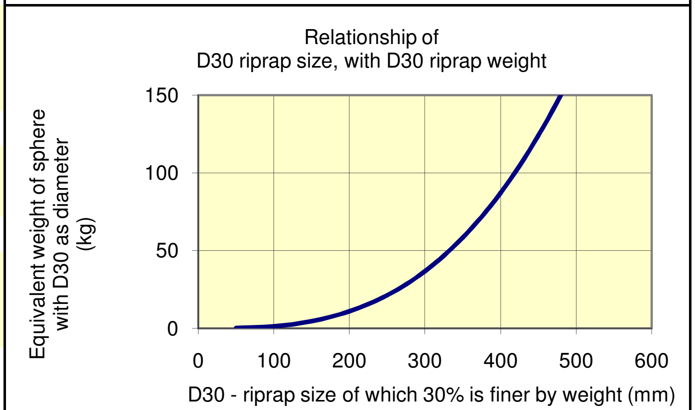
Where:
D_{30} = riprap size of which 30% is finer by weight.
S_f = safety factor. The minimum is 1.1.
C_s = stability coefficient for incipient failure. D_{85}/D_{15}= 1.7 to 5.2
= 0.3 for angular rock, 0.375 for rounded rock.
C_v = vertical velocity distribution coefficient.
= 1.0 for straight channels.
= 1.283 - 0.2 log (R/W) outside of bends (1 for R/W>26)
C_T =thickness coefficient.
= 1.0 for thickness of D_{100} or 1.5 $D_{50(max)}$ whichever is greater.
d = local depth of flow, length (same location as V).
γw = unit weight of water, weight/volume.
γs = saturated surface unit weight of stone.
V_{ss} = local depth averaged velocity over the slope at a point of 20 percent of the slope length from the toe of the slope.
K_1 = side slope correction factor.
g = gravitational constant.

Where:
K_1 = $(1-\sin^2\theta/\sin^2\phi)$
V_{ss} = V_{avg}*(1.74-0.52*log(R/W))

Where:
θ = angle of side slope with horizontal
ϕ = angle of repose of riprap (normally 40 degrees)
V_{ave} = average velocity at the upstream end of the bend.



		Case 1	Case 2	Case 3	Case 4
D_{30}	(mm)	66	135	236	373
D_{50}	(d_{85}/d_{15}=1.7)	79	161	282	445
	(d_{85}/d_{15}=5.2)	114	234	409	646
V_{ss}	(m/s)	1.83	2.44	3.05	3.66
V_{ave}	(m/s)	1.5	2	2.5	3
S_f		1.1	1.1	1.1	1.1
C_s		0.3	0.3	0.3	0.3
C_v		1.083	1.083	1.083	1.083
C_T		1	1	1	1
d_{total}		3.0	3.0	3.0	3.0
d for V_{ss}		2	2	2	2
γw	(kg/m^3)	1000	1000	1000	1000
γs	(kg/m^3)	2600	2600	2600	2600
g	(m/s^2)	9.81	9.81	9.81	9.81
K_1		0.718	0.718	0.718	0.718
R	C/L radius	5000	5000	5000	5000
W	water width	500	500	500	500
$\cot\theta$	of slope	2	2	2	2
$\sin\theta$	of slope	0.447	0.447	0.447	0.447
ϕ	of riprap	40	40	40	40



GeoMedia Engineering Ltd.
Stone Size Relations
Reference: U.S. Army Corps of Engineers, "Hydraulic Control of Flood Control Channels - Change 1", 1994

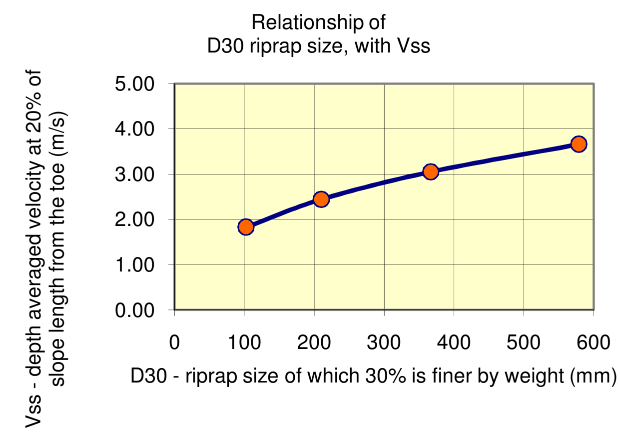
The basic relation for representative stone size in straight and curved channels applicable for slopes of 1.5H:1V, or flatter, is:
$D_{30}=S_f C_s C_v C_t d\,[\,(\gamma w/(\gamma s-\gamma w))^{1/2}\quad V_{ss}/(K_1 g d)^{1/2}\,]^{2.5}$

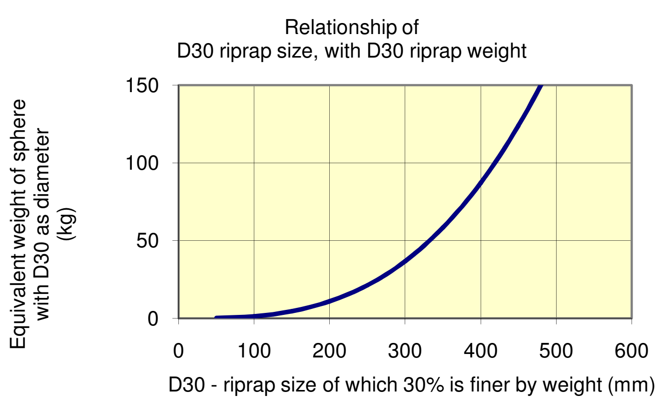
Where:
D_{30} = riprap size of which 30% is finer by weight.
S_f = safety factor. The minimum is 1.1.
C_s = stability coefficient for incipient failure. D_{85}/D_{15}= 1.7 to 5.2
= 0.3 for angular rock, 0.375 for rounded rock.
C_v = vertical velocity distribution coefficient.
= 1.0 for straight channels.
= 1.283 - 0.2 log (R/W) outside of bends (1 for R/W>26)
C_T =thickness coefficient.
= 1.0 for thickness of D_{100} or 1.5 $D_{50(max)}$ whichever is greater.
d = local depth of flow, length (same location as V).
γw = unit weight of water, weight/volume.
γs = saturated surface unit weight of stone.
V_{ss} = local depth averaged velocity over the slope at a point of 20 percent of the slope length from the toe of the slope.
K_1 = side slope correction factor.
g = gravitational constant.

Where:
K_1 = $(1-\sin^2\theta/\sin^2\phi)$
V_{ss} = $V_{avg}*(1.74-0.52*\log(R/W))$

Where:
θ = angle of side slope with horizontal
ϕ = angle of repose of riprap (normally 40 degrees)
V_{ave} = average velocity at the upstream end of the bend.

		Case 1	Case 2	Case 3	Case 4
D_{30}	(mm)	102	210	367	579
D_{50}	(d_{85}/d_{15}=1.7)	122	251	438	691
	(d_{85}/d_{15}=5.2)	177	364	635	1002
V_{ss}	(m/s)	1.83	2.44	3.05	3.66
V_{ave}	(m/s)	1.5	2	2.5	3
S_f		1.1	1.1	1.1	1.1
C_s		0.3	0.3	0.3	0.3
C_v		1.083	1.083	1.083	1.083
C_T		1	1	1	1
d_{total}		3.0	3.0	3.0	3.0
d for V_{ss}		2	2	2	2
γw	(kg/m^3)	1000	1000	1000	1000
γs	(kg/m^3)	2600	2600	2600	2600
g	(m/s^2)	9.81	9.81	9.81	9.81
K_1		0.505	0.505	0.505	0.505
R	C/L radius	5000	5000	5000	5000
W	water width	500	500	500	500
cotθ	of slope	1.5	1.5	1.5	1.5
sinθ	of slope	0.555	0.555	0.555	0.555
ϕ	of riprap	40	40	40	40



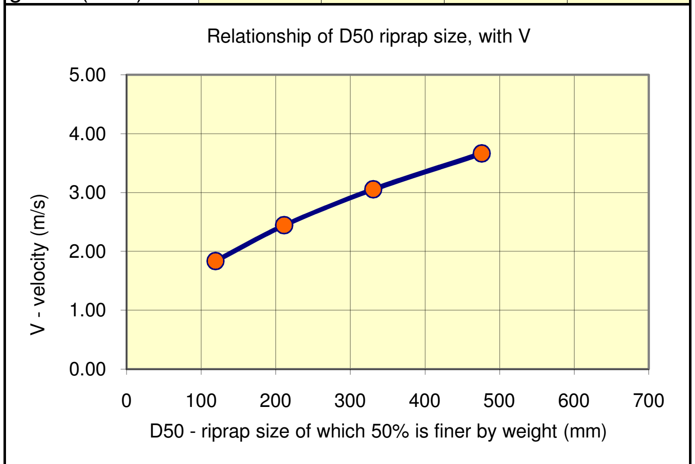


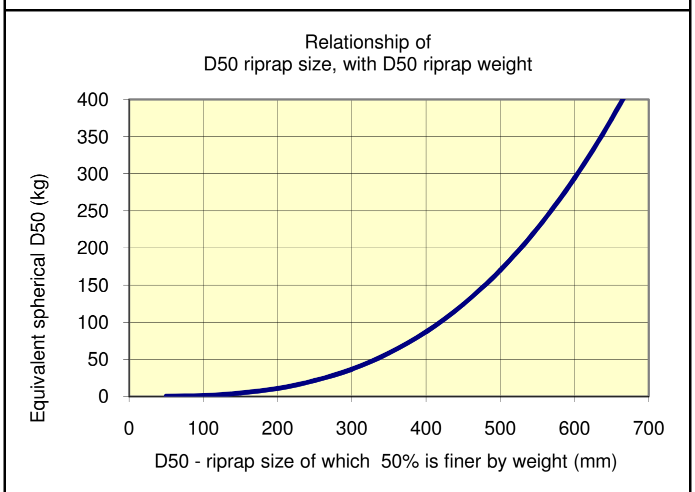
GeoMedia Engineering Ltd.
Stone Size Gradations
Reference: U.S. Army Corps of Engineers, EM 1110-2-1601, June 94

Isbach's equation for determination of D50(min) for riprap is:
D_{50}=(V/C*(2γs*(γs-γw)/γw)^$^{-0.5}$

Where:
D_{50} = riprap size of which 50% is finer by weight.
V = velocity (average)
C = Isbach coefficient
γw = unit weight of water, weight/volume.
γs = saturated surface unit weight of stone.
g = gravitational constant.

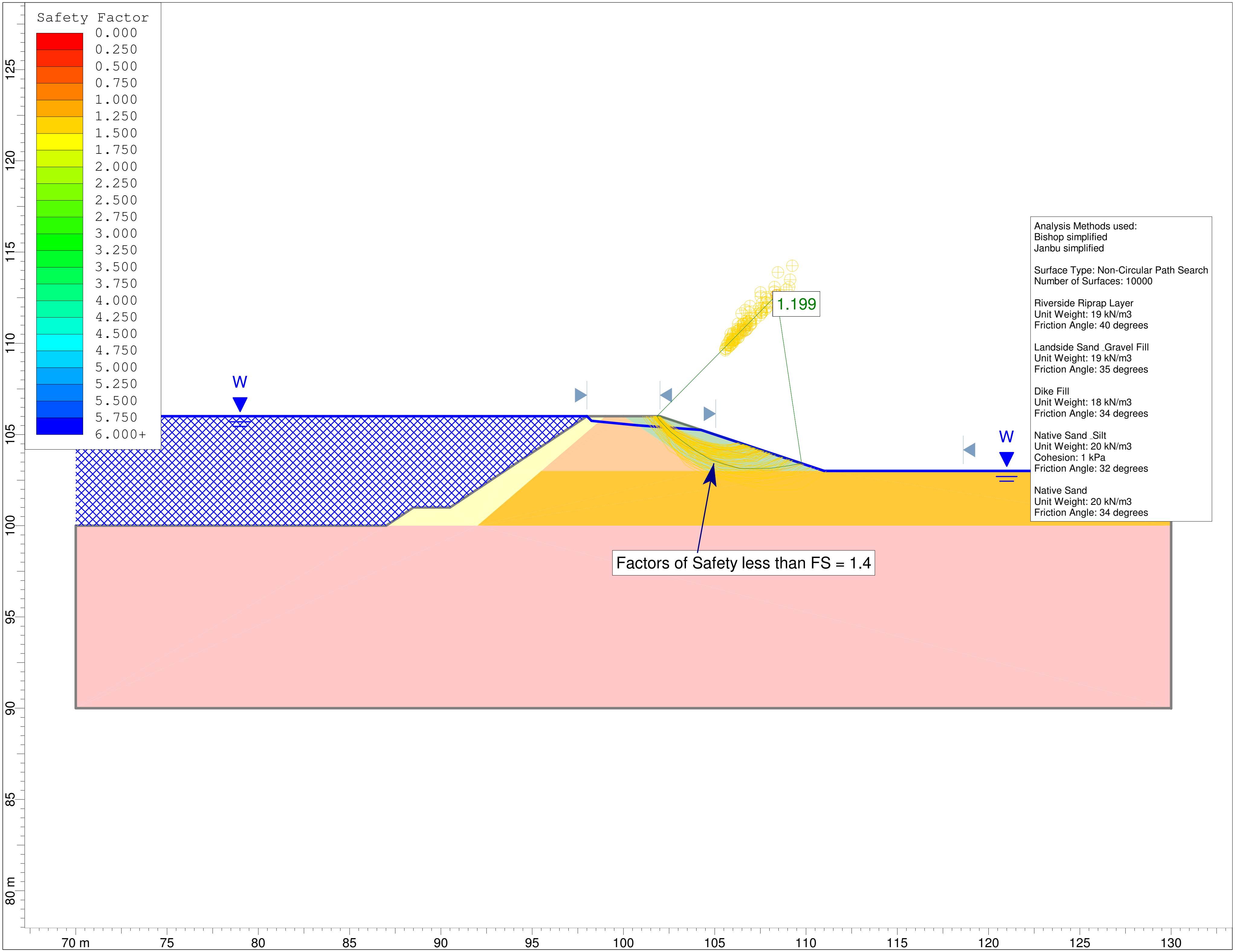
		Case 1	Case 2	Case 3	Case 4
D_{50}	(mm)	119	212	330	476
V	(m/s)	1.83	2.44	3.05	3.66
C		1.20	1.20	1.20	1.20
γw	(kg/m^3)	1000	1000	1000	1000
γs	(kg/m^3)	2650	2650	2650	2650
g	(m/s^2)	9.81	9.81	9.81	9.81



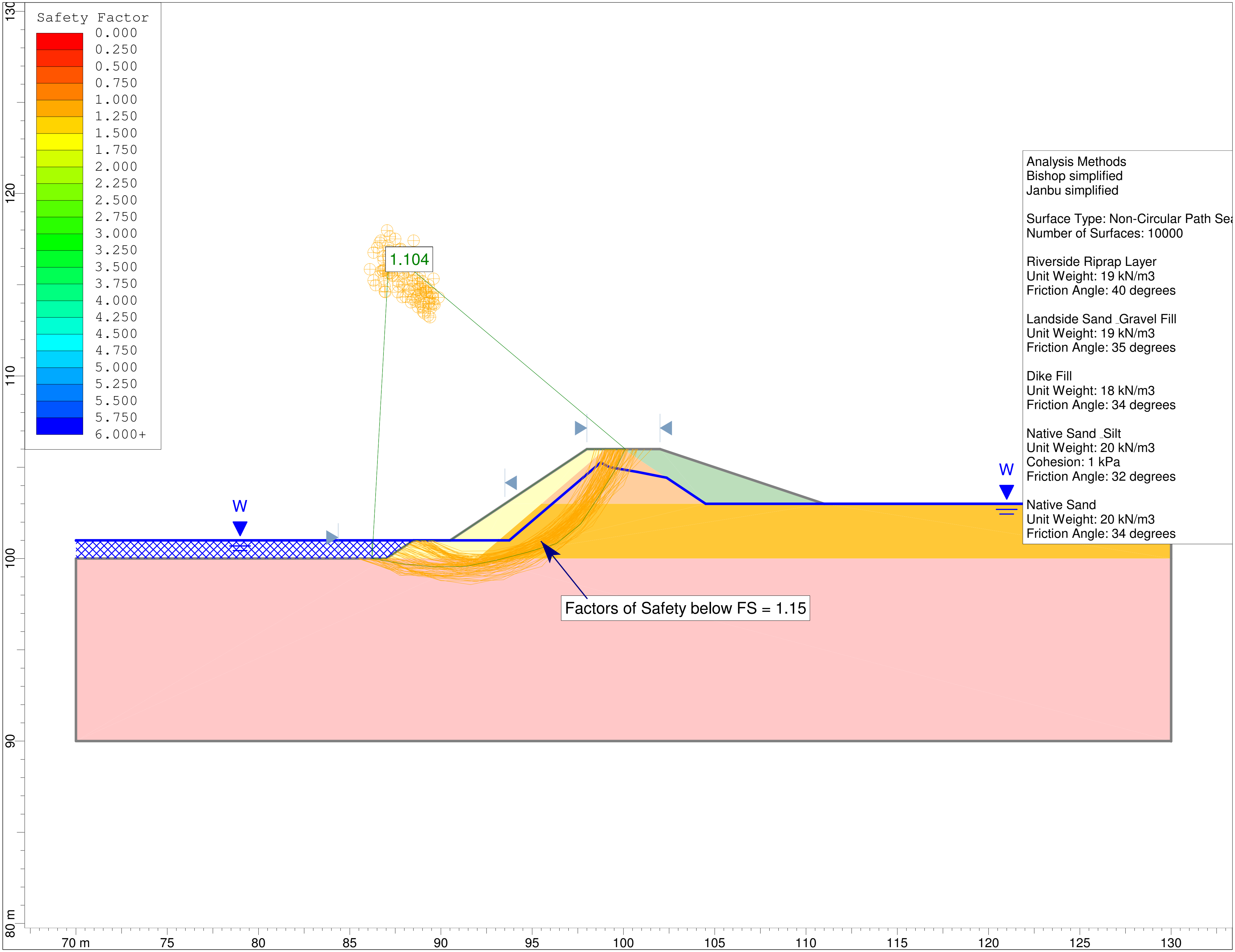


Appendix D

Slope Stability Analysis



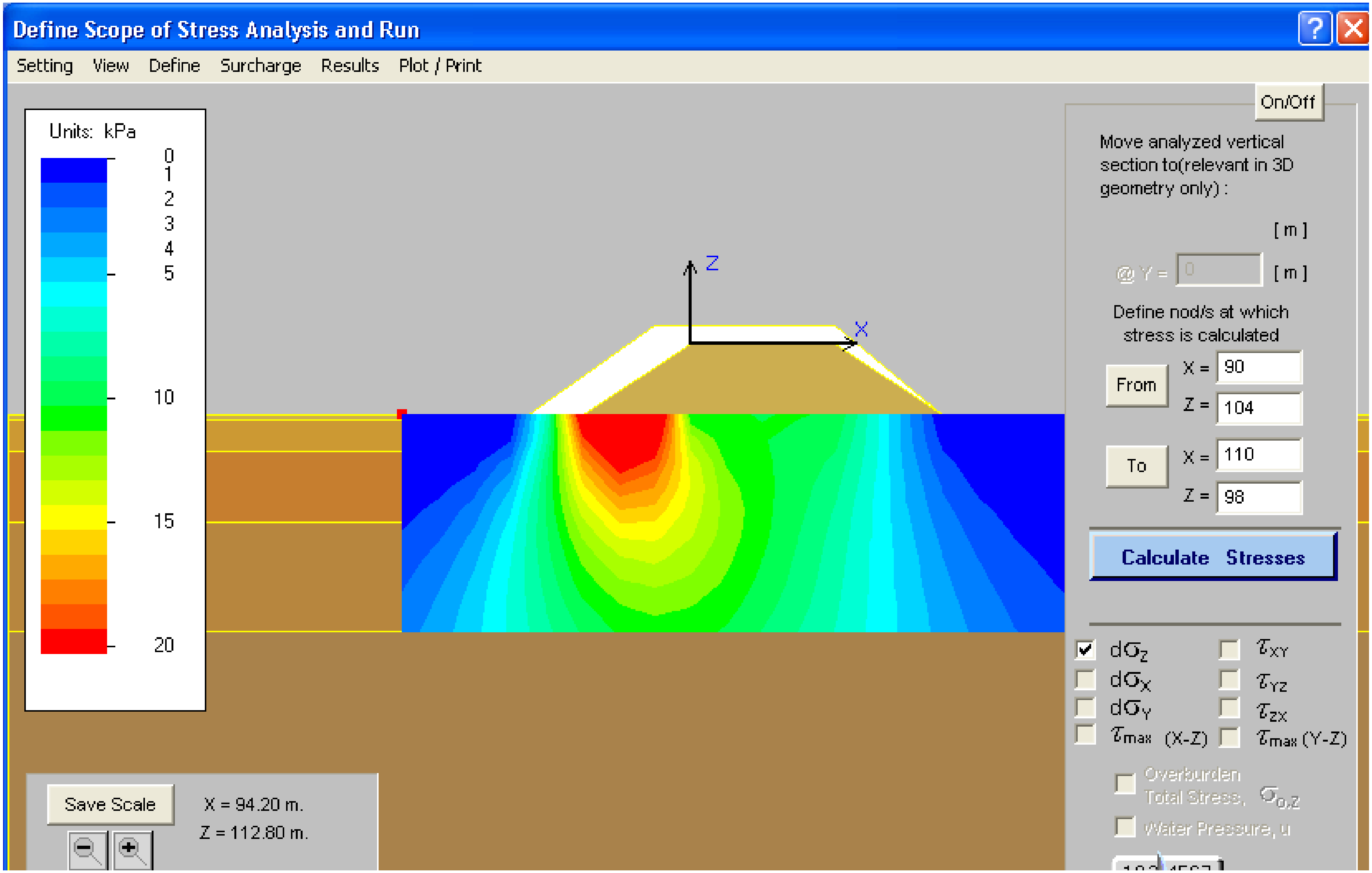
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
W
W
1.199
Factors of Safety less than FS = 1.4
Analysis Methods used:
Bishop simplified
Janbu simplified
Surface Type: Non-Circular Path Search
Number of Surfaces: 10000
Riverside Riprap Layer
Unit Weight: 19 kN/m3
Friction Angle: 40 degrees
Landside Sand _Gravel Fill
Unit Weight: 19 kN/m3
Friction Angle: 35 degrees
Dike Fill
Unit Weight: 18 kN/m3
Friction Angle: 34 degrees
Native Sand _Silt
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 32 degrees
Native Sand
Unit Weight: 20 kN/m3
Friction Angle: 34 degrees
70 m
75
80
85
90
95
100
105
110
115
120
125
130
80 m
85
90
95
100
105
110
115
120
125



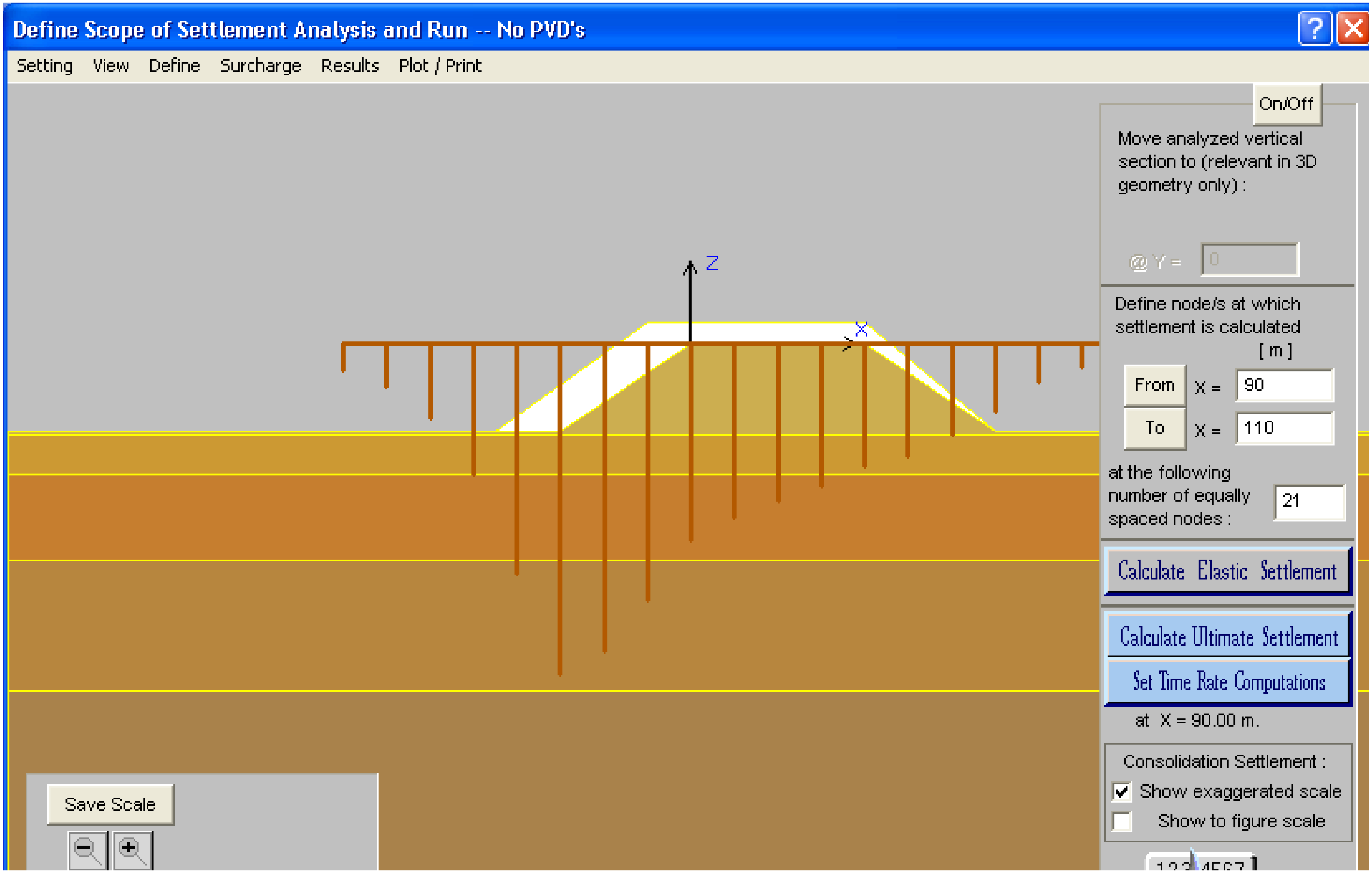
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.104
W
W
Factors of Safety below FS = 1.15
Analysis Methods
Bishop simplified
Janbu simplified
Surface Type: Non-Circular Path Sea
Number of Surfaces: 10000
Riverside Riprap Layer
Unit Weight: 19 kN/m3
Friction Angle: 40 degrees
Landside Sand _Gravel Fill
Unit Weight: 19 kN/m3
Friction Angle: 35 degrees
Dike Fill
Unit Weight: 18 kN/m3
Friction Angle: 34 degrees
Native Sand _Silt
Unit Weight: 20 kN/m3
Cohesion: 1 kPa
Friction Angle: 32 degrees
Native Sand
Unit Weight: 20 kN/m3
Friction Angle: 34 degrees
130
120
110
100
90
80 m
70 m
75
80
85
90
95
100
105
110
115
120
125
130

Appendix E

Settlement Analysis – Terasen ROW Crossing



Define Scope of Stress Analysis and Run
Setting
View
Define
Surcharge
Results
Plot / Print
Units: kPa
0
1
2
3
4
5
10
15
20
Z
X
On/Off
Move analyzed vertical section to(relevant in 3D geometry only) :
[m]
@ Y =
0
[m]
Define nod/s at which stress is calculated
From
X =
90
Z =
104
To
X =
110
Z =
98
Calculate Stresses
$d\sigma_Z$
$d\sigma_X$
$d\sigma_Y$
τ_{max} (X-Z)
τ_{XY}
τ_{YZ}
τ_{ZX}
τ_{max} (Y-Z)
Overburden Total Stress, $\sigma_{o,Z}$
Water Pressure, u
Save Scale
X = 94.20 m.
Z = 112.80 m.


Define Scope of Settlement Analysis and Run -- No PVD's
Setting
View
Define
Surcharge
Results
Plot / Print
Z
X
On/Off
Move analyzed vertical section to (relevant in 3D geometry only) :
@ Y =
0
Define node/s at which settlement is calculated [m]
From
X =
90
To
X =
110
at the following number of equally spaced nodes :
21
Calculate Elastic Settlement
Calculate Ultimate Settlement
Set Time Rate Computations
at X = 90.00 m.
Consolidation Settlement :
Show exaggerated scale
Show to figure scale
Save Scale

INPUT DATA -- FOUNDATION LAYERS -- 5 layers

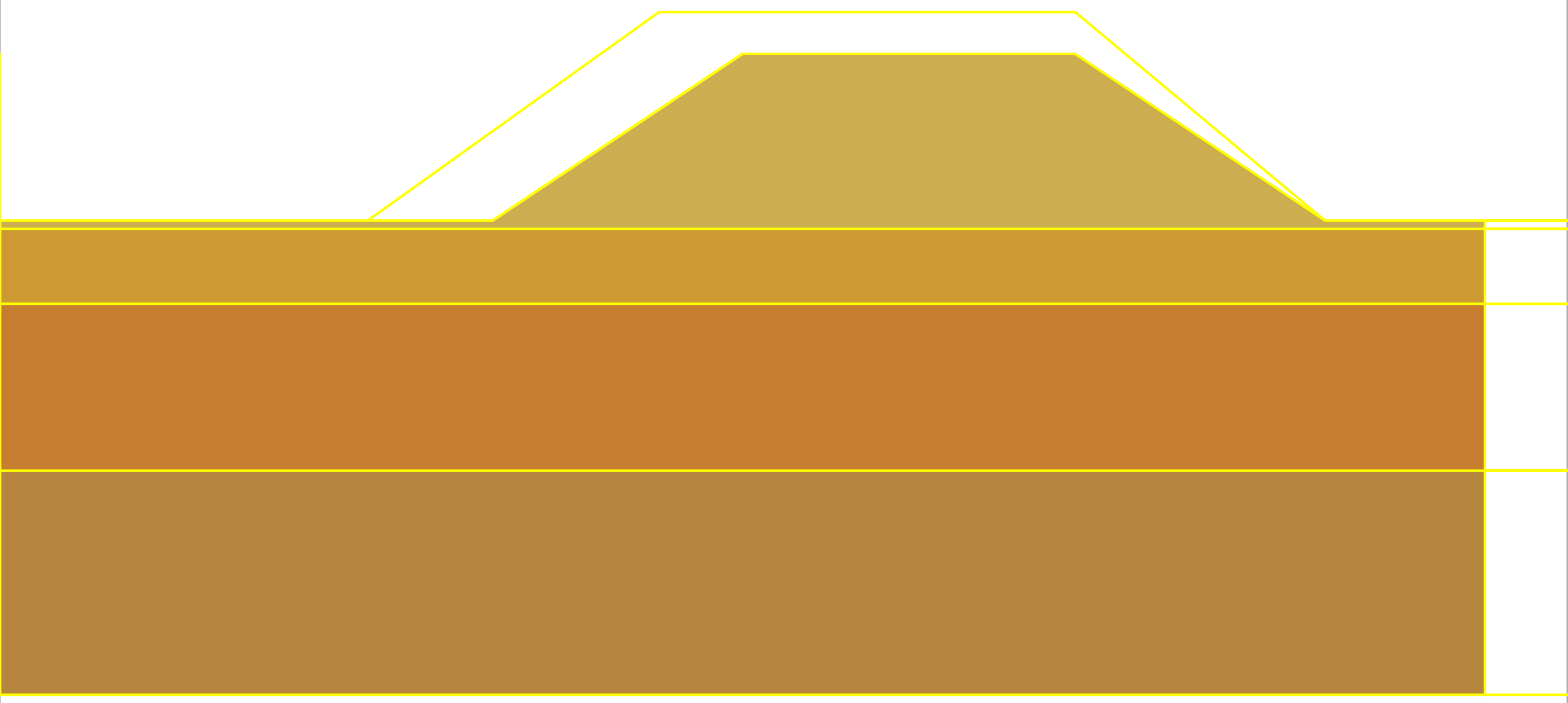
	Wet Unit Weight, γ [kN/m³]	Poisson's Ratio μ	Description of Soil
1	20.00	0.30	
2	20.00	0.30	
3	20.00	0.30	
4	20.00	0.30	
5	20.00	0.30	

INPUT DATA -- EMBANKMENT LAYERS -- 1 layers

	Wet Unit Weight, γ [kN/m³]	Description of Soil
1	20.00	

Version 2.0 FoSSA Version 2.0

DRAWING OF SPECIFIED GEOMETRY



Version 2.0 FoSSA Version 2.0

INPUT DATA FOR CONSOLIDATION –– $\alpha = 1/2$

Layer #	Underging Consolidation [Yes/No]	OCR = Pc / Po	Cc	Cr	e0	Cv [m ²/day]	Drains at :
1	No	N/A	N/A	N/A	N/A	N/A	N/A
2	No	N/A	N/A	N/A	N/A	N/A	N/A
3	Yes	1.00	0.30	0.08	1.00	0.0200	Top & Bot.
4	Yes	1.00	0.40	0.10	1.00	0.0200	Top & Bot.
5	No	N/A	N/A	N/A	N/A	N/A	N/A

ULTIMATE SETTLEMENT, Sc

Node #	X [m.]	Y [m.]	Original Z [m.]	Settlement Sc [m.]	Final Z * [m.]
1	90.00	0.00	104.00	0.01	103.99
2	91.00	0.00	104.00	0.01	103.99
3	92.00	0.00	104.00	0.02	103.98
4	93.00	0.00	104.00	0.03	103.97
5	94.00	0.00	104.00	0.05	103.95
6	95.00	0.00	104.00	0.07	103.93
7	96.00	0.00	104.67	0.07	104.60
8	97.00	0.00	105.33	0.06	105.28
9	98.00	0.00	106.00	0.04	105.96
10	99.00	0.00	106.00	0.04	105.96
11	100.00	0.00	106.00	0.04	105.96
12	101.00	0.00	106.00	0.03	105.97
13	102.00	0.00	106.00	0.03	105.97
14	103.00	0.00	105.33	0.03	105.31
15	104.00	0.00	104.67	0.02	104.65
16	105.00	0.00	104.00	0.02	103.98
17	106.00	0.00	104.00	0.01	103.99
18	107.00	0.00	104.00	0.01	103.99
19	108.00	0.00	104.00	0.00	104.00
20	109.00	0.00	104.00	0.00	104.00
21	110.00	0.00	104.00	0.00	104.00

*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.