ANNEXE K

Geotechnical Closure Report for Voorspoed Open Pit

Report prepared for

DBCM (Pty) Ltd Voorspoed Mine

DE BEERS GROUP

Report No. 544921 – FINAL REPORT

Report prepared by

srk consulting

January 2024

Geotechnical Closure Report for Voorspoed Open Pit

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SRK Project Number Report No. 544921

January 2024

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LIST OF ABBREVIATIONS

AA	: Anglo-American
APEC	: Asia-Pacific Economic Corporation
DBCM	: De Beers Consolidated Mines
ICMM	: International Council on Mining and Metals
SRK	: SRK Consulting (South Africa) (Pty) Ltd
J&W	: Jones & Wagener (Pty) Ltd
CFRD	: concrete faced rockfill dam
FRD	: Fines Residue Deposit
WRD	: Waste Rock Dump
ALARP	: As Low As Reasonably Practicable
BB	: break-back distance (in metres)
DAC	: Design Acceptance Criteria
QRA	: quantitative risk assessment
RSG	: Relative Stability Guideline
RRR, or R^3	: Residual Risk Rating
SCC	: Slope Condition Class
TSR	: Target Stability Reliability
CKBX	: Country Rock-Kimberlite Breccia
DOL	: Dolerite
KBBX	: Kimberlite-Basalt Breccia
KCBX	: Kimberlite-Country Rock Breccia
UVK	: Undifferentiated Volcaniclastic Kimberlite
VO	: Volksrust Formation
VR	: Vryheid Formation
VRM	: Fine-grained mudstones of the Vryheid Formation
VRSSC	: Inter-bedded shale, sandstones and conglomerates of the Vryheid Formation
VRVS	: Varved shale
c	: cohesion (in kPa)
CC	: coefficients of correlation between cohesion c and friction angle ϕ
CV	: coefficient of variation
ϕ	: friction angle (in degrees)
FoS	: Factor of Safety
JRC	: Joint Roughness Coefficient
PMP	: Probable Maximum Flood
PoF	: Probability of Failure (in %)
SRF	: Strength Reduction Factor
UCS	: Unconfined compressive strength (in MPa)

DISCLAIMER

1 BACKGROUND & TERMS OF REFERENCE

1.1 Introduction

De Beers Consolidated Mines (Pty) Ltd ("DBCM") Voorspoed Mine appointed SRK Consulting (South Africa) (Pty) Ltd ("SRK") to assist with the compilation and finalisation of a Geotechnical Closure Report for the Voorspoed Open Pit. Voorspoed Mine is located about 36 km north of Kroonstad, on Road S156 just off the R76 in the Free State Province of South Africa.

The principal objective of this geotechnical closure study is to address the recommendation that *"a geotechnical study must be conducted to determine the pit stability for all the scenarios outlined in the geochemical report"* as prescribed in a DMRE letter dated 29/09/2023 (DMRE Ref.: FS 30/5/1/2/3/2/1 (12) EM).

1.2 Scope of Work completed

Three pit remediation scenarios were outlined in the geochemical report (Golder, 2020):

- Option 1 – Pit lake fills naturally by groundwater and rainfall only.
- Option 2 – Pit lake filling enhanced with engineered surface water catchment.
- Option 3 – Complete backfilling of pit.

To address the DMRE recommendation *that "a geotechnical study must be conducted to determine the pit stability for all the scenarios outlined in the geochemical report"*, the following Scope of Work was completed as presented in this Report:

- Review and compilation of applicable geotechnical risk and consequence acceptance criteria for closure of the Voorspoed pit based on recent guidance provided in this regard by Carter et al. (2022, 2023).
- Review the suitability of the Geotechnical Model developed for operational design, for use in post-closure design processes.
- Conduct additional 2D analyses using Rocscience's Slide2 software, to complement limit equilibrium analysis work completed to date:
  - Critically review the impact of higher groundwater tables on pit stability as the pit lake rises to the predicted long-term stable pit lake level, and the potential impact of climate change scenarios (i.e., more frequent high intensity storm events, resulting in periodic transient pore pressure fluctuations long-term).
  - Analysis of a re-oriented SSE design section to provide an estimate of additional long-term- break-back likely to occur behind the SSE1 failure area.
  - Review of the potential impact of seismicity on post-closure stability of the Voorspoed pit, for long-term seismic design and dynamic load scenarios.
- Comment on long-term erosion and slope degradation in the Voorspoed pit, based on:
  - Material degradation susceptibility affecting strength properties.
  - An evaluation of break-back rates on different design sections and in different geologies, based on annual LiDAR scan data.
  - Estimate the very long-term pit slope profile based on our understanding of geological materials encountered in the pit and a high-level evaluation of the erodibility and dispersivity of these materials.
- Review and comment as appropriate on the Jones & Wagener ("J&W") stormwater management report prepared in 2022 for Voorspoed.

That said, and with reference to the Golder 2020 options to be evaluated in this geotechnical study, the following should be noted in terms of the work presented in this Report:

- **Option 1 – Pit lake fills naturally by groundwater and rainfall only**
  Option 1 is the Base Case used in this study. The geotechnical risk, design acceptance and consequence criteria presented in Section 3, and the stability assessment and break-back extent delineation work in Section 4 and most of the worst case/special case scenarios considered, applies to this Option 1.
- **Option 2 – Pit lake filling enhanced with engineered surface water catchment**
  Although Option 2 is just a 'special case' variation of Option 1, it warranted separate analysis, with the results thereof presented in Section 4.3.3. That said, given that it is a variation of Option 1, the Baseline Option 1 results and specifically the acceptable break-back limits for static load conditions presented in Section 4.2.3, also applies to this Option 2.
- **Option 3 – Complete backfilling of pit**
  In this Option 3, there is no possibility of slope instability given that the pit walls will be fully buttressed to surface, which did not warrant any further geotechnical assessment or stability analysis (i.e., there is no credible mode of slope instability if the pit is completely backfilled).

1.3 Project team & Statements of Capability

The project team comprised the following SRK key staff:

- Mr Gerhardus Keyter Pr Eng | AREC: Principal Geotechnical Engineer
- Mr Luis-Fernando Contreras Pr Eng: Associate Principal Geotechnical Engineer
- Mr Peter Shepherd Pr Sci Nat: Partner and Principal Hydrologist
- Ms Sharla Coetsee Pr Eng: Principal Geotechnical Engineer
- Mr Desmond Mossop Pr Sci Nat | COMREC: Partner and Principal Engineering Geologist

Statements of Capability for Mr Keyter and Mr Contreras are included in Appendix A as reference.

1.4 Statement of SRK independence

Neither SRK nor any of the authors of this Report have any material present or contingent interest in the outcome of this Report, nor do they have any pecuniary or other interest that could be reasonably regarded as being capable of affecting their independence or that of SRK.

SRK furthermore has no prior association with De Beers regarding the mineral assets that are the subject of this Report, and SRK has no beneficial interest in the outcome of the technical assessment being capable of affecting its independence.

SRK's fee for completing this Report is based on its normal professional daily rates plus reimbursement of incidental expenses, and the payment of that professional fee is not contingent upon the outcome of this Report.

1.5 Pertinent legal requirements, international standards & industry best practices

A thorough review of pertinent legal requirements for mine closure in South Africa as applicable to geotechnical aspects and the stability of open pit mines, together with an overview of pertinent international standards and industry best practices in this regard, was carried out to ensure compliance with relevant Acts and Regulations, and with national and international standards and industry best practice listed in the Bibliography in Section 7.

2 VOORSPOED MINE GEOTECHNICAL MODEL (SRK, 2013)

2.1 Geotechnical Model

2.1.1 Local geology & structure

The Voorspoed Kimberlite (Group II) is Lower Cretaceous (131 Ma) in age and is one of six kimberlite pipes located in what is known as the Kroonstad Kimberlite Cluster.

The Voorspoed pipe was 12.5 ha at surface before mining commenced, with 6 ha of that having been basaltic in nature. The pipe comprised a complex kimberlite facies and breccia architecture for which the principal rock types are detailed in Figure 2-1, these are: the Undifferentiated Volcaniclastic Kimberlite ("UVK"), Kimberlite-Country Rock Breccia ("KCBX"), Country Rock-Kimberlite Breccia ("CKBX"), and Kimberlite-Basalt Breccia ("KBBX") which was found along the pipe margins. Basalt rafts and talus breccias were located on the south-east margin of the pipe (Fulop et al., 2017; Howarth et al., 2013).

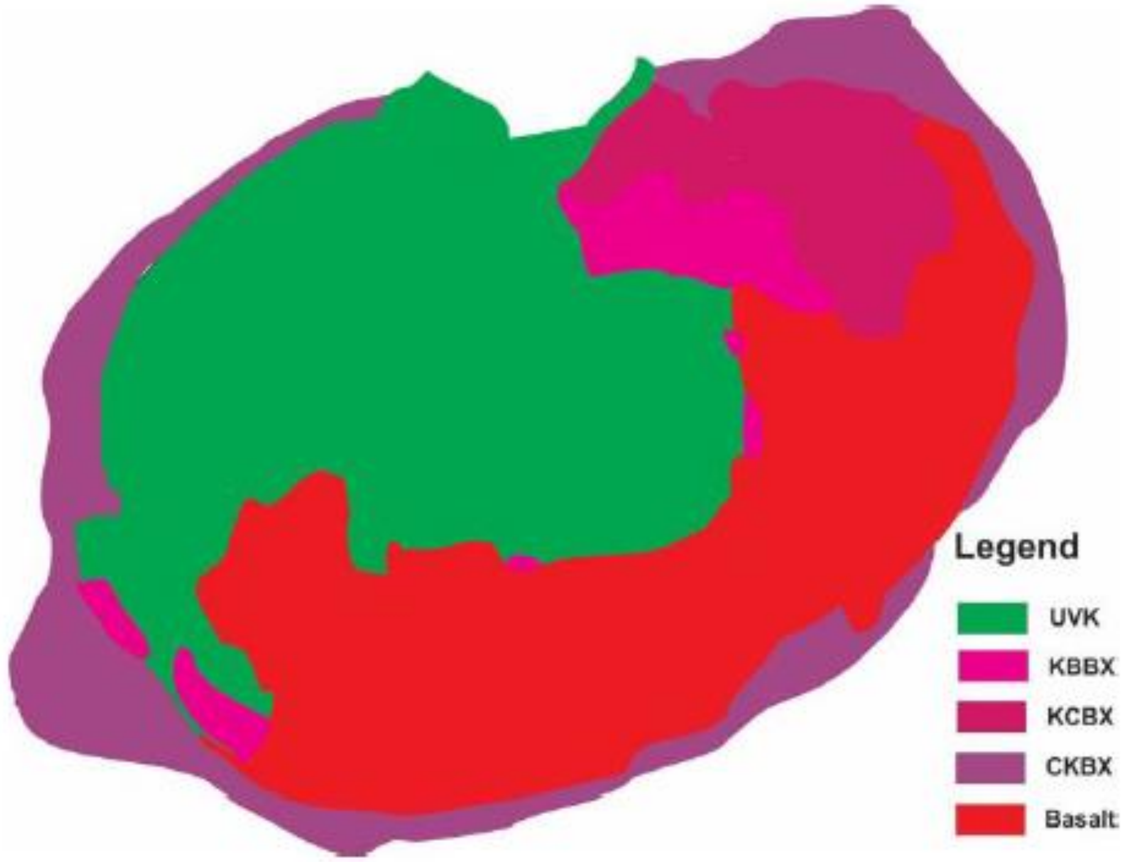


Figure 2-1: Kimberlitic facies types of the Voorspoed pipe (Fulop et al., 2017)

The Voorspoed kimberlite intruded karoo mudrocks and shales up to 150 m thick of the upper Ecca Group of the Karoo Supergroup, underlain by between 150 to 400 m of shales, carbonates and quartzites of the Transvaal Supergroup. The pipe is typified by steep sides, with the country rock being highly brecciated at the contact, with brecciation becoming more gradational further away from the pipe margins (Howarth et al., 2013).

The Voorspoed country rock sedimentary package is described in detail by SRK (2013) and comprised the following Formations of the Upper Ecca Group of the Karoo Supergroup:

- Volksrust Formation ("VO"): These are fine-grained, deep-water sediments that are dominated by a massive but occasionally finely laminated carbonaceous mudstone.
- Vryheid Formation ("VR"): These are aerial and subaerial deltaic deposits that are either fine-grained mudstones ("VRM") or inter-bedded shale, sandstones and conglomerates ("VRSSC").
- There is also a varved shale ("VRVS") unit, lower in the sequence.
- Finally, the VR was intruded by several dolerite sills ("DOL") of variable thickness.

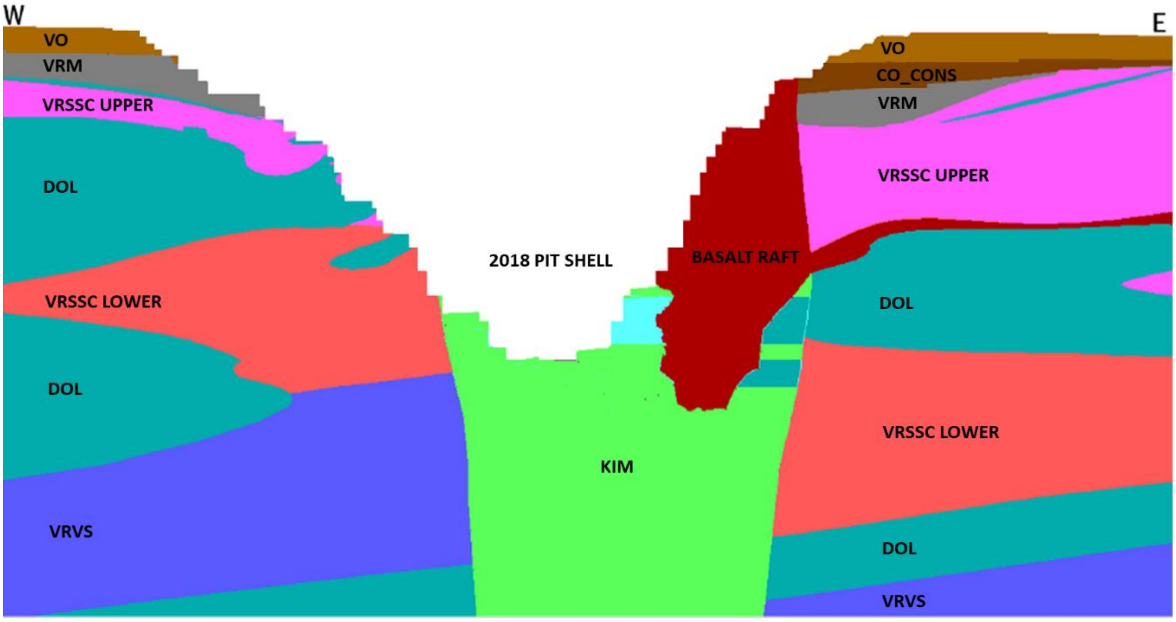


Figure 2-2: Geological cross section (view north) illustrating the principal sedimentary units of the country rock, the location of the kimberlite pipe and basalt raft (SRK, 2013)

Several major geological structures furthermore intersected the country rock and kimberlite of the Voorspoed pit.

2.1.2 Pit design sectors & ground control districts

Operational ground control districts in the Voorspoed pit were delineated based on the geology, structure in terms of bedding orientation, and major fault sub-division. This was then further refined based on the orientation of the pit slope to represent specific pit slope design sectors as illustrated in Figure 2-3 (SRK, 2013). The lithologies were segregated into two categories, namely:

- **Soft rocks** with uniaxial compressive strength ("UCS") of less than 50 MPa in which bench to multi-bench slope instabilities occurred during mining, and since; these included the VO and VRM mudstones, the VRSSC Upper and the VRVS.
- **Hard rocks** with UCS above 50 MPa which were generally stable, with falls of ground comprising key blocks and loose rock posing the main hazard to the mining operation; these included the Kimberlites, the VRSSC Lower, the Basalt, and the Dolerites.

Double benching was only employed in the more competent Basalt, Dolerite, CKBX and Kimberlites as indicated in Figure 2-3.

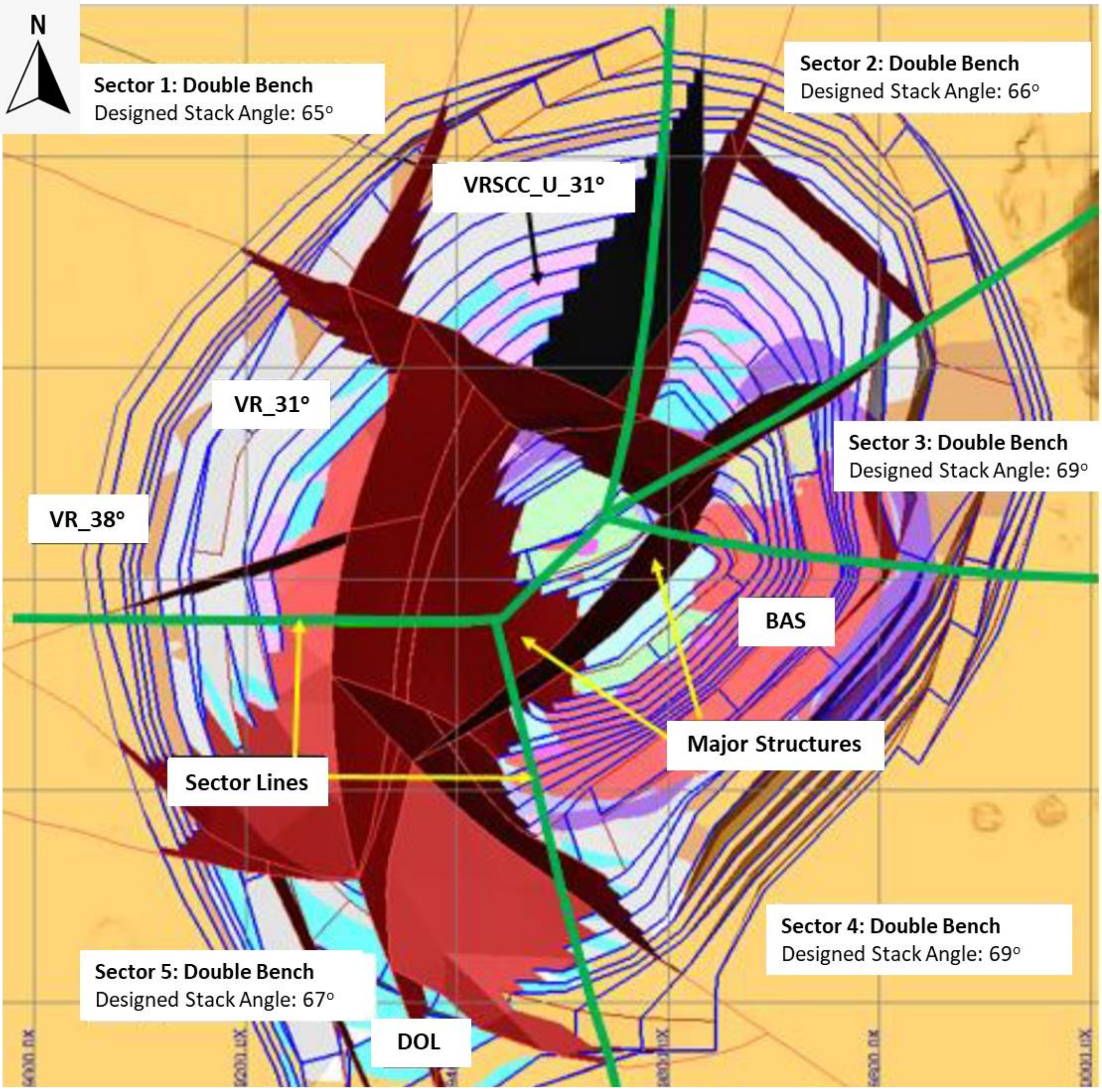
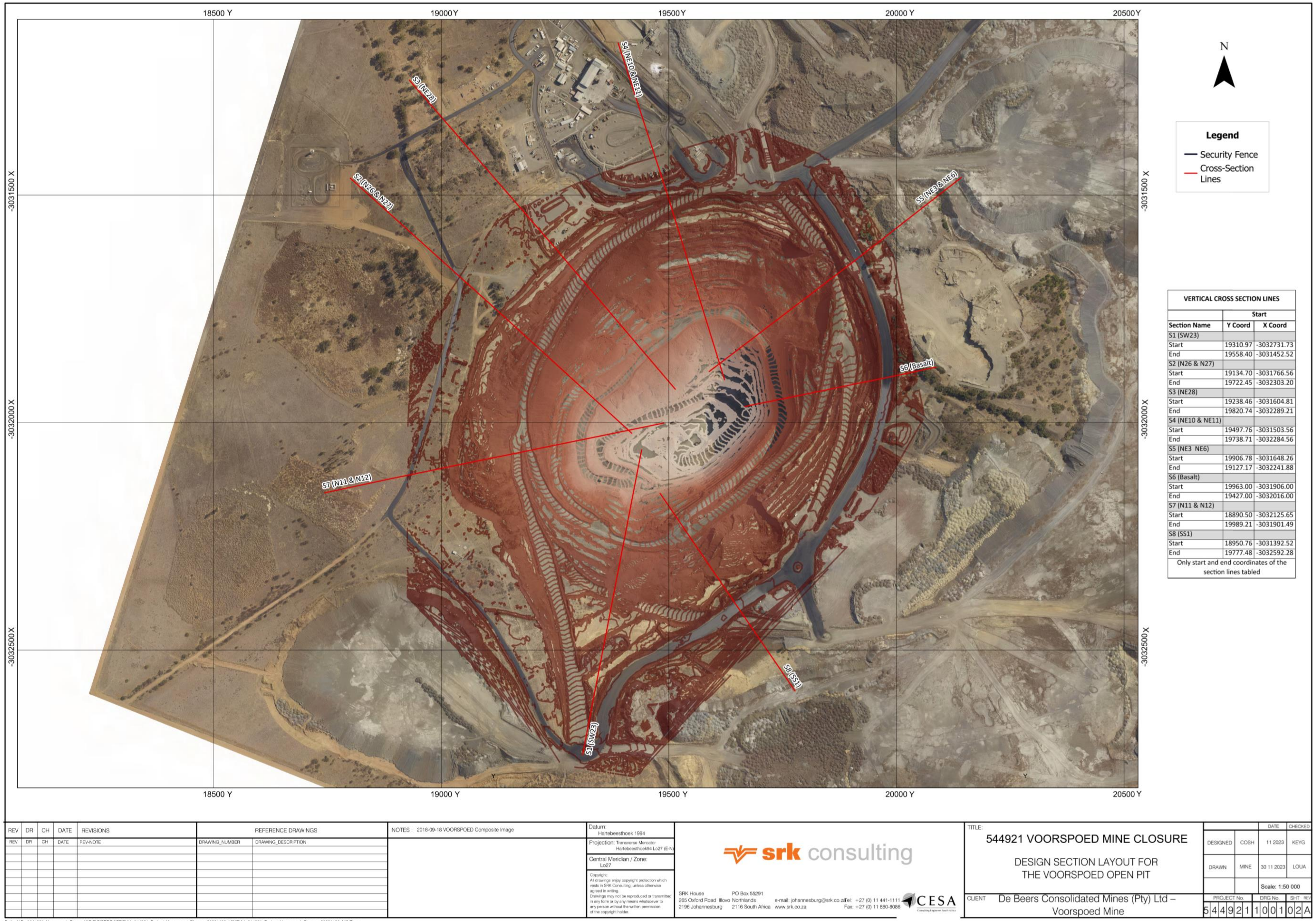


Figure 2-3: Plan view of the Cut 3 design with the major structures, sector domains & multi-bench- stack angles for both the double benches & weaker rock masses (Voorspoed Mine COP, 2016)

Figure 2-4 shows the location of pit slope design section lines used for the design of the Voorspoed open pit initially, as well as subsequently for pit closure as presented in this Report.

The most recent operational geotechnical hazard and monitoring plan for the Voorspoed open pit, is presented in Figure 2-5. Note that significant portions of the pit slopes are progressively deforming and are unstable. **Consequently, inadvertent access into all areas of elevated geotechnical risk around the open pit, is actively prevented by a well-maintained security fence which was constructed when mining ceased in 2018**. Risk mitigation measures implemented since include regular inspection of pit highwalls and crest areas, stormwater management around the pit, and maintenance of the security fence. Also, Mr Gerhard Keyter Pr Eng | AREC, a Principal Geotechnical Engineer with SRK, has been legally appointed under Regulation 22.14.1(8) of the Mines Health & Safety Act (Act 29 of 1996) since 2020, and regularly visits Voorspoed to participate in pit inspections, and to evaluate additional mitigation measures that may be required in areas of active break-back.



VERTICAL CROSS SECTION LINES		
	Start	
Section Name	Y Coord	X Coord
S1 (SW23)		
Start	19310.97	-3032731.73
End	19558.40	-3031452.52
S2 (N26 & N27)		
Start	19134.70	-3031766.56
End	19722.45	-3032303.20
S3 (NE28)		
Start	19238.46	-3031604.81
End	19820.74	-3032289.21
S4 (NE10 & NE11)		
Start	19497.76	-3031503.56
End	19738.71	-3032284.56
S5 (NE3 NE6)		
Start	19906.78	-3031648.26
End	19127.17	-3032241.88
S6 (Basalt)		
Start	19963.00	-3031906.00
End	19427.00	-3032016.00
S7 (N11 & N12)		
Start	18890.50	-3032125.65
End	19989.21	-3031901.49
S8 (SS1)		
Start	18950.76	-3031392.52
End	19777.48	-3032592.28
Only start and end coordinates of the section lines tabled		

Figure 2-4: Plan view of Voorspoed Mine showing the location of geotechnical section lines used for pit design, and subsequent closure analysis

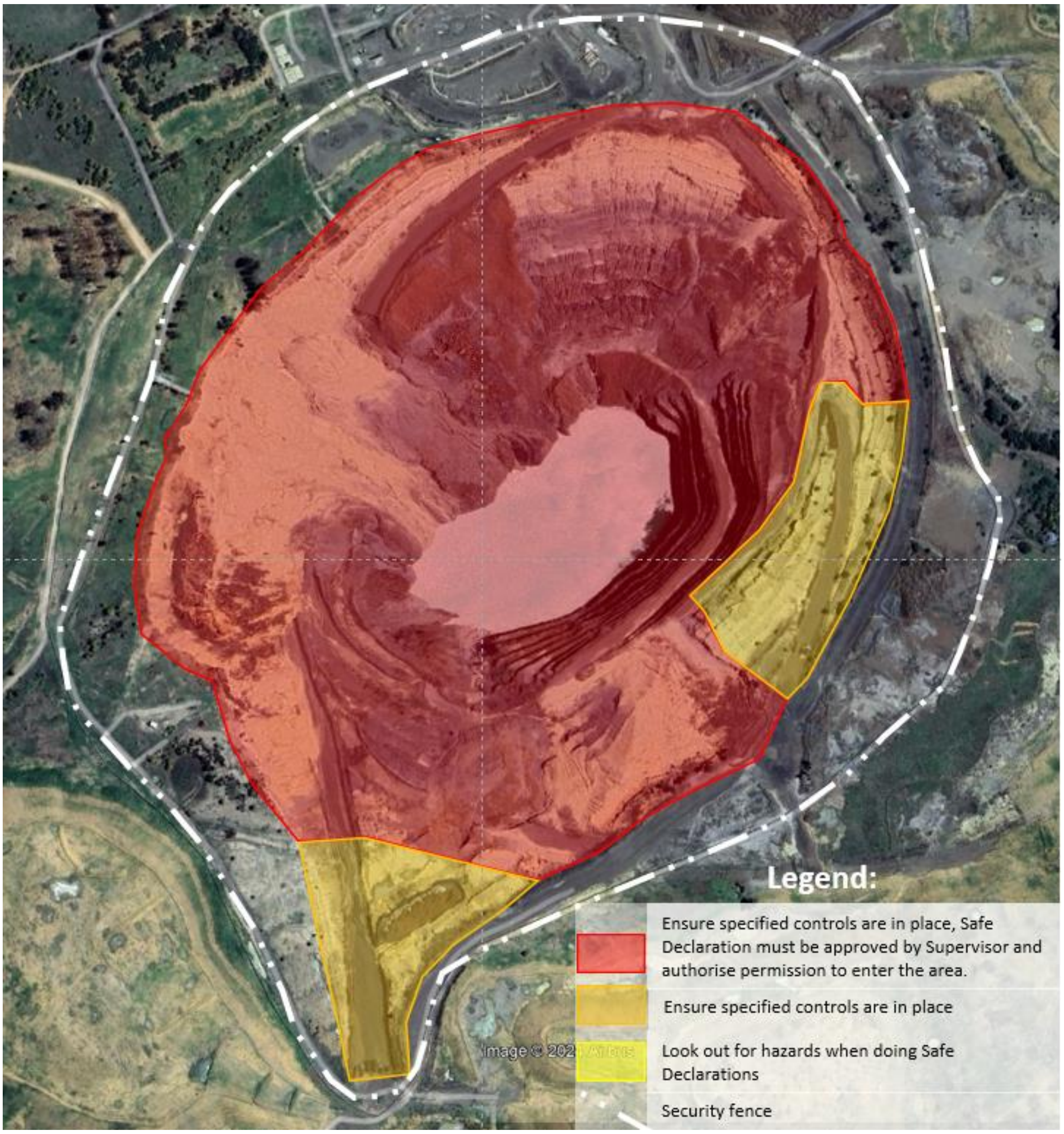


Figure 2-5: Voorspoed Mine, Pit Hazard & Monitoring Plan (January 2024) (not to scale)

2.1.3 Suitability of Geotechnical Model for use in post-closure design

Following the above detailed review of the Geotechnical Model developed for operational design (SRK, 2013) and considering anticipated mechanisms of slope degradation and the possible extent thereof, as well as the scale and depth of potential slip surfaces/instability that are likely to develop in the long-term, it was concluded that this Geotechnical Model, augmented by key supplemental model scenarios and sensitivity assessments, is generally suitable for review of the long-term stability of the pit slopes at Voorspoed post-closure.

2.1.4 Geotechnical design parameters used in Voorspoed pit closure review

Geotechnical design parameters and material properties reported by Itasca (2015) and the adjusted bedding strength parameters using the Barton-Bandis criterion assuming a Joint Roughness Coefficient ("JRC") of 2 as summarised in Table 4-1 were used in the Slide2 limit equilibrium analyses for pit closure as presented in Section 4 of this Report. These post-closure slope stability analyses also considered variability in strength parameters and the shape of the slip surfaces, and the following should also be noted:

- The Mohr-Coulomb strength parameters listed in Table 4-1 were based on the equivalent Mohr-Coulomb parameters calculated from the Hoek-Brown strength criterion for low confinement stresses (σ_{3max} < 1 MPa) and then adjusted based on results of the back-analysis of past failures and expert judgement.
- The variability in strength parameters was assumed to conform to a normal distribution with a coefficient of variation ("CV") of 30% for cohesion ("*c*") and 15% for friction angle ("ϕ"). These assumptions are based on judgement and are considered appropriate for the sensitivity analysis of break-back distance. Figure 2-6 shows what this variability means in terms of the cohesion and friction angle of the VRM and VRSSC upper units, which generally control the stability of most of the Voorspoed slopes.

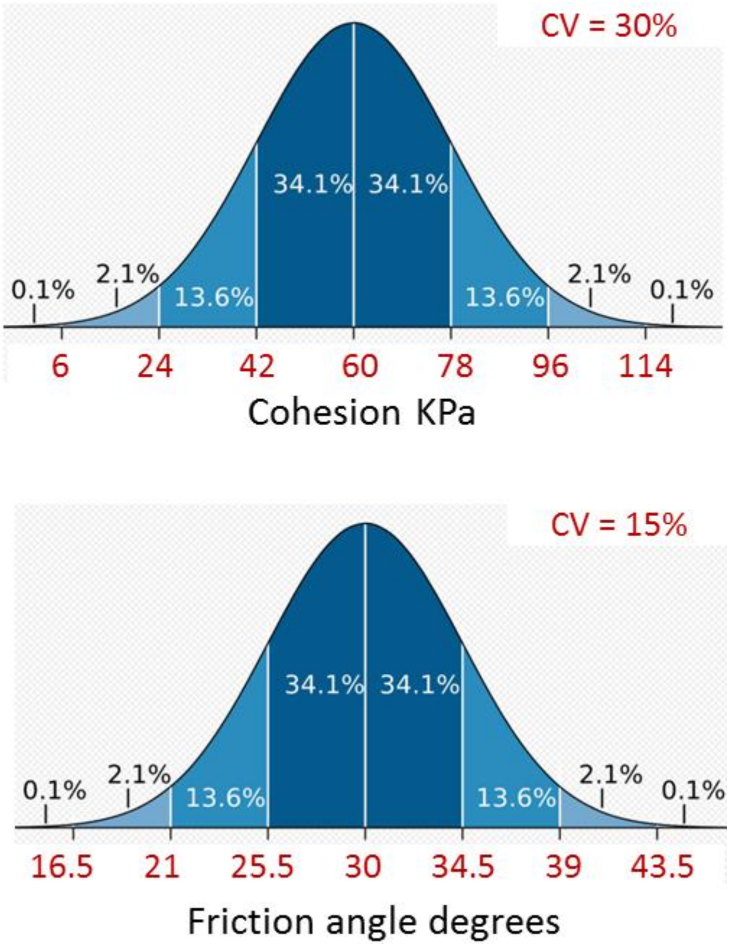


Figure 2-6: Assumed variability of strength parameters in VRM & VRSSC upper

- Coefficients of correlation ("CC") between cohesion *c* and friction angle ϕ were conservatively assumed to be zero when evaluating likely break-back distances in these post-closure stability analyses.

Table 2-1: Geotechnical design parameters for the Voorspoed open pit

Unit ID	Lithology	Lithological Code	Material Parameters			Coefficient of Variation	
			Friction Angle, ϕ (deg)	Cohesion, *c* (kPa)	Unit Weight, γ (kN/m^3)	15% std dev§, ϕ (deg)	30% std dev§, *c* (kPa)
1	Volksrust mudstone	VOM	12	30	22.4	1.8	9.9
2	Vryheid mudstone	VRM	30	60	26	4.5	18
3	Vryheid shale/sandstone/... conglomerate	VRSSC UPPER	30	60	26	4.5	18
4	Vryheid diamictite	VRSSC LOWER	60	1120	27.6	9.1	336.6
5	Vryheid varved shale	VRVS	27	10	26		
6	Dolerite	DOLERITE	60	4100	29.3	9	1231
7	Kimberlite contact breccia	CKBX (Anisotropic)*	23	45	25.51		
8	Kimberlite contact breccia	CKBX (SW23)**	21	1015	25.51	3.2	305
9	Kimberlite contact breccia	CKBX Contact	25	10	26		
10	Basalt	BASALT	58	1510	26.09	8.7	453
11	UVK kimberlite	UVK (DVK)***	52	1530	25.7	7.8	460
12	OVK kimberlite	OVK	52	1530	25.51	7.8	459.9
13	KBBX kimberlite	KBBX	52	1530	25.8	7.8	459.9
14	RXVK kimberlite	RXVK	48	820	25	7.2	246.0
15	Buttress	BUTTRESS	38	30	20	5.7	9
16	Failed material	FAILED MATERIAL	36	0	20	5.4	0
17	Fault	FAULT	27	5	26	4.1	1.5

Notes:

§ std dev = standard deviation.

* N27 back analysis.

** Calculated from Strength Reduction Factor (“SRF”) with in-situ Itasca properties (2015).

2.2 Hydrogeology (extract from Golder, 2020)

A conceptual hydrogeological model was developed for Voorspoed by Golder (2020) and used to develop a post-closure geochemical pit lake model. From a hydrogeological perspective:

- The filling period conditions are depicted in Figure 2-7, while the conceptualised conditions for the 200-year post-closure steady-state model are shown in Figure 2-8.
- At closure, the Voorspoed pit will begin to fill due to inflows from deep groundwater, shallow groundwater, surface runoff, and direct rainfall.
- The water balance modelling shows that the pit lake will take approximately 180 years to reach a steady-state water level (Golder, 2020) during what is termed the “filling period”, up until the year 2200 approximately.
- Hydrogeochemical processes will continue for a further 200 years after the “filling period” until steady-state conditions are achieved during what is termed the “steady--state period”, which will continue up until the year 2400 approximately.
- The closure period begins at start of the filling period and continues several years into the steady-state period, allowing time for the pit lake to physically and chemically stabilised.

Water sources that have been identified to have a direct impact on the Voorspoed pit post-closure include:

- Groundwater infiltration from shallow groundwater aquifers and from seepage from the Waste Rock Dump (“WRD”).
- Groundwater infiltration from deep groundwater aquifers which is unimpacted by surface dump seepage.
- Direct rainfall.
- Wall rock flushing from the pit walls above the lake water level.
- Surface runoff inflows from surface catchments adjacent to the Voorspoed pit.
- Seepage from the Fines Residue Deposit (“FRD”) towards the Voorspoed pit, in particular.

That said, seepage inflows are expected to be relatively insignificant given the low permeability of the country rock masses at Voorspoed. Given that annual evaporation in the Voorspoed region far exceeds annual precipitation, it is estimated that it will take up to 200years for an ultimate steady-state pit lake level to develop.

Golder (2020) provided three hydrogeological model scenarios as tabulated in Table 2-2 for which the following modelling assumptions and criteria have been applied:

- Option 1 – Pit lake filling: Pit lake levels 10years and 200years after closure formed respective Baselines for analysis and evaluation of the stability of the Voorspoed slopes post-closure, with the 10year lake level constituting a more short-term, less stable pit slope Baseline, and the 200year lake level a more stable, long-term Baseline.
- Option 2 – Pit lake filling with engineered catchment[1]: In this Scenario, the stability of pit slopes were evaluated for a pit lake level 30 m higher than the 200-year Baseline in Scenario 1, and with phreatic surfaces in the pit slopes adjusted accordingly.
- Option 3 – Complete backfilling of pit: For this Scenario, there is no credible possibility of slope instability given that the pit walls will be fully buttressed to surface, and this did not warrant any further assessment or analysis.

[1] Note: A similar scenario can result from very high seasonal rainfall in some years, in the long-term post-closure.

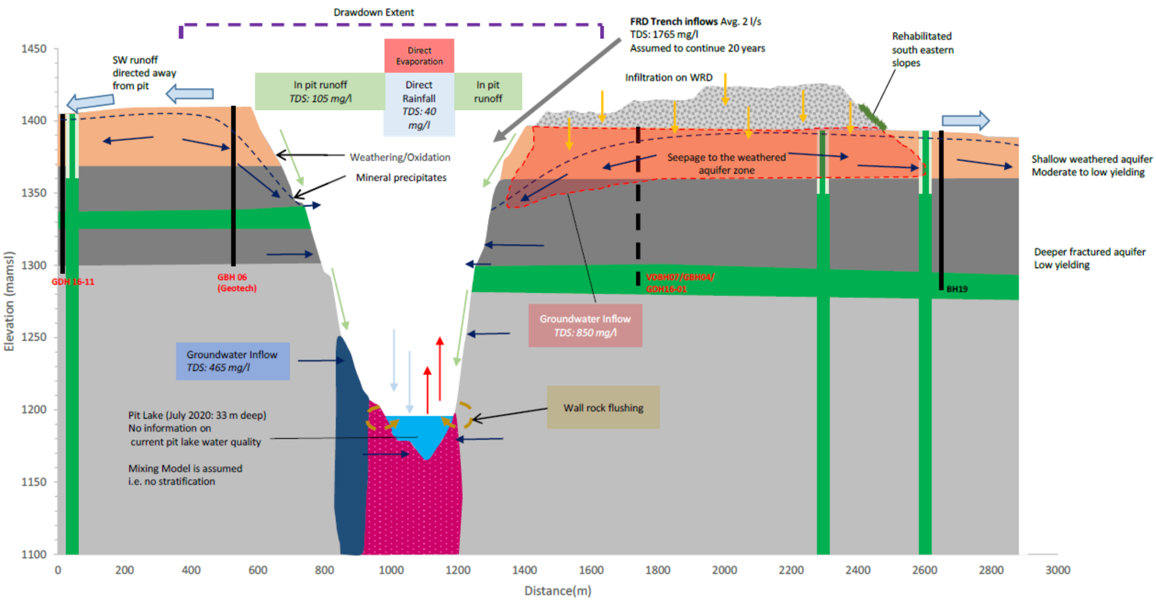


Figure 2-7: Conceptual hydrogeological model of the Voorspoed pit lake at closure (Golder, 2020)

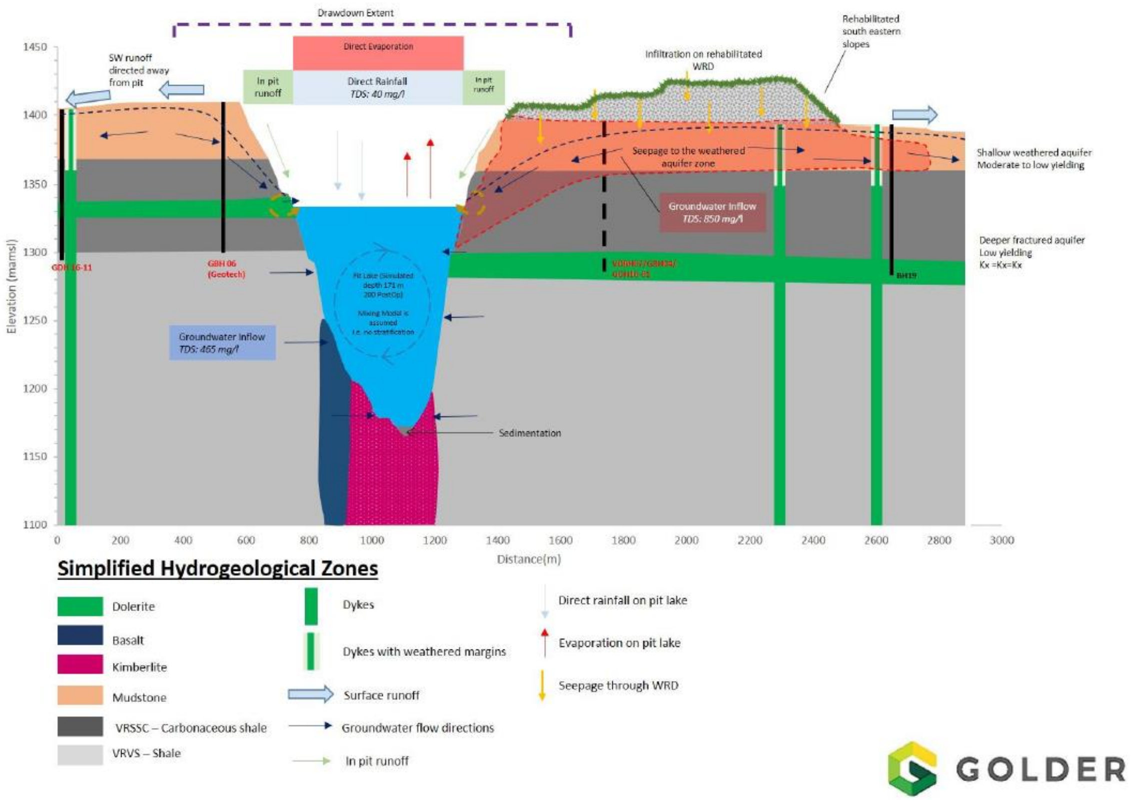


Figure 2-8: Conceptual steady-state hydrogeological model of the Voorspoed pit lake in 2220 (Golder, 2020)

Table 2-2: Golder (2020) hydrogeological modelling options

ID	Scenario	Description
1	Pit lake filling	The pit lake fills naturally in this scenario. All surrounding surface water is assumed to be diverted away from the pit area. Inputs to the pit thus include rainfall on the pit footprint, and groundwater ingress. Output from the pit was losses to evaporation. In this scenario, it was demonstrated that the pit would remain a groundwater sink due to the high evaporation compared with rainfall and groundwater ingress.
2	Pit lake filling with engineered catchment	The second scenario considered the pit lake which receives ingress from rainfall on the pit footprint, groundwater inflows and losses via evaporation, as well as additional surface water inflows generated from an engineered catchment measuring 406.2 ha. It is assumed that the catchment will be smoothed and grassed, and accordingly surface runoff was calculated using the SCS method and assigning a curve number of 79. In this scenario, it was demonstrated that the final pit lake water level would be 20 to 30 m higher than that simulated for Scenario 1. The pit would however remain a sink because of the high evaporation.
3	Backfilling of the pit	A third scenario was simulated where the pit was assumed to be backfilled. In this scenario, the pit is no longer expected to remain a sink and pit water is thus expected to migrate away from the pit area to the adjacent aquifer zones.

Groundwater levels and phreatic surfaces used in the models and analysis to evaluate the stability of the Voorspoed pit slopes, were based on a mine-wide phreatic surface developed by Piteau (2018), with adjustments made as appropriate to align with rising pit lake levels post-closure as discussed above, and also to evaluate the potential impact of transient water pressures nearer surface and above the phreatic surface, during periods of prolonged rainfall. Phreatic surfaces used in the analyses are presented in Appendix B for reference.

2.3 Seismicity

In the absence of a detailed seismic hazard study specifically for Voorspoed Mine, published seismic hazard maps were used to select appropriate seismic design loads (i.e., peak ground accelerations, or “PGAs”), benchmarked against seismic design loads derived during detailed design of two major rockfill dams being constructed in Southern Africa.

The hazard ranking and typical seismic design criteria is provided in Table 2-3, which suggests a design earthquake with a 2,475year return period for passive closure of open pits where the consequences of largescale pit instability is considered ‘high’. However, considering that the consequences of any large-scale instability at Voorspoed post-closure, would be ‘moderate’ to ‘low’ given the mine’s remote location, the existing pit exclusion zone with security fence, and the resultant lack of public risk exposure or risk to major infrastructure, this design criteria for a ‘high consequence’ event as adopted in this Report, is considered conservative.

Table 2-3: Typical seismic design criteria for open pits & waste rock dumps (modified from 2020 Global Industry Standard on Tailings Management)

Consequence Classification	Seismic Design Acceptance Criteria ("DAC") & Suggested Thresholds	
	Return Period	Applicable Scenario/s[1]
Low (Operating Basis Earthquake, "OBE")	None to 1/145	Operations level for pits
Moderate	1/475	Common threshold for building structures
Significant	1/975	Active closure, DAC threshold
High	1/2,475	Passive closure, Residual Risk Rating (R^3) for open pits
Very high	1/5,000	Passive closure, Relative Stability Guideline ("RSG") for dumps
Extreme (Maximum Design Earthquake, "MDE")	1/10,000	Passive closure, RSG for tailings and large water dams

Notes: 1 - Nominal open pit and WRD function descriptors, may not be representative of actual consequence classification, use site specific consequence assessment or regulatory guidance.

That said, the seismic hazard map published by Midzi et al. (2020) indicates a horizontal acceleration of approximately 0.07 g (see Figure 2-9) in the Voorspoed area, for earthquakes with a 475-year return period.

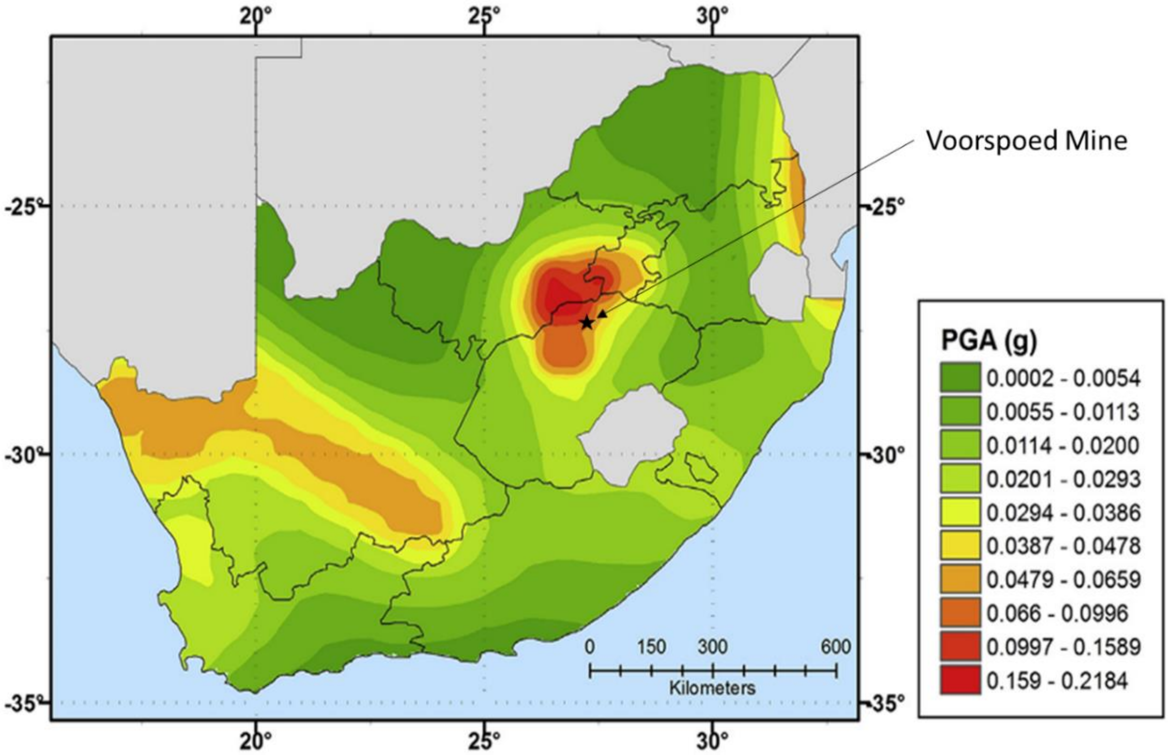


Figure 2-9: Distribution of mean PGA values in South Africa, for a 10% probability of exceedance in 50 years (i.e., for a return period of 475 years) (from Midzi et al., 2020)

To derive a DAC for a seismic event with 2,475-year return period in accordance with Table 2-3, this 0.07 g horizontal acceleration for a seismic event with a 475-year return period, was compared with design values derived in detailed seismic studies for the design of two large rockfill dams currently being constructed in Southern Africa, as a benchmarking exercise – these were:

- Ngwadini Dam, a 50 m high concrete faced rockfill dam ("CFRD") being constructed near Durban in KwaZulu Natal:
  - 475-year return: 0.042 g ± 0.016 g
  - 2,475-year return: 0.107 g ± 0.035 g
  - 10,000-year return: 0.199 g ± 0.064 g
- Polihali Dam, a 165 m high CFRD, being constructed as part of Phase II of the Lesotho Highlands Water Project:
  - 475-year return: 0.047 g ± 0.018 g
  - 2,475-year return: 0.121 g ± 0.036 g
  - 10,000-year return: 0.223 g ± 0.048 g

From these benchmark studies, it is noted that the horizontal acceleration for an event with a 2,475-year return period, conservatively, is about three times that of an event with a 475-year return period. Applying a factor of three to the Midzi et al. (2020) values for Voorspoed, gives a horizontal acceleration of 0.21 g.

In an update to the CSIRO Guidelines for Open Pit Slope Design (Read et al., 2009) currently being developed to also guide pit closure designs, it is recommended that two-thirds of this horizontal acceleration (i.e., $^{2}/_{3}$ x 0.21 g = 0.14 g) be used as design seismic coefficient when employing limit equilibrium analysis techniques in programs such as Rocscience's Slide2 (personal communication, de Graaf (2024)). A vertical acceleration equal to a sixth of that (i.e., $^{1}/_{6}$ x 0.14 g = 0.023 g) was similarly adopted.

The above seismic design values are considered conservative, for the following reasons:

- As noted earlier, given the 'low' to 'moderate' consequences of large-scale pit slope instability at Voorspoed post-closure.
- The current seismic signature for the area is influenced by the fact that Voorspoed is located within the general footprint of the Witwatersrand Gold Basin, with several deep underground operating gold mines located in and around towns such as Welkom and Virginia to the south, and Klerksdorp and Carletonville to the north. As a result of these deep underground mining activities and associated mining induced seismicity, the Witwatersrand Basin carries a higher earthquake risk than most of the rest of South Africa. However, the level of mining induced seismicity in the region is bound to reduce over the next several decades as remaining gold deposits become too deep to economically exploit, and notwithstanding possible temporary increases in seismic risk locally in the region, where shaft pillar extraction or similar activities are undertaken towards the end of the Life of Mine of remaining deep underground gold mining operations.

3 GEOTECHNICAL RISK & DESIGN ACCEPTANCE CONSEQUENCE CRITERIA FOR PIT CLOSURE

Generally accepted design acceptance criteria for pit slope closure are described by Carter et al. (2022), with the process and procedures for conducting a stability evaluation taking into consideration the parameters assigned for Voorspoed, can be best summarised as a 5-step approach as set out in the following subsections.

3.1 Step 1: Initial screening & options analysis

The initial screening for defining a Residual Risk Rating ("RRR", or "R^3") for an open pit to be closed and the associated options analysis, is shown schematically in Figure 3-1 in terms of R^3 and Target Stability Reliability ("TSR").

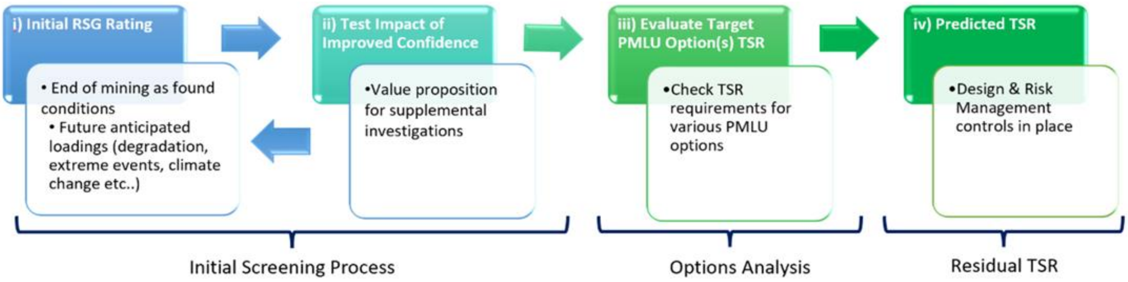


Figure 3-1: Initial screening process, options analysis & residual TSR flow chart (Carter et al., 2022)

The purpose of this screening approach is to ensure that all eventualities in terms of pit slope conditions, both current and future, are managed with appropriate design and risk mitigation controls in place.

The different pit slope design sectors at Voorspoed are expected to behave in a similar manner post-closure and as such, the geotechnical risk and design acceptance consequence criteria for pit closure adopted for Voorspoed as presented in the next subsections, made no distinction between the different pit design sectors.

3.2 Step 2: Selecting an appropriate Design Confidence Rating

Design Confidence Ratings varies from very high (VH=0), high (H=1), medium (M=2), low (L=3) to very low (VL=4), with a description of these confidence ratings given in Table 3-1. For Voorspoed, the level of design confidence is generally good, given that it is supported by a good understanding of geotechnical and hydrogeological conditions in the pit slopes. However, as the exposed pit slopes will continue to weather and erode post-closure, this does introduce some model and design uncertainty, and a Design Confidence Rating of medium (M=2, baseline) to low (3, long-term) was adopted for Voorspoed in this Report for the purposes of pit closure.

3.3 Step 3: Selecting an appropriate Failure Consequence Rating

Failure Consequence Ratings in Table 3-2, varies from very low (VL=1), low (L=2), medium (M=3), high (H=4) to very high (VH=5). The consequence of slope failure at Voorspoed post-closure, is deemed relatively low (since a pit exclusion approach with a security fence has been implemented already), and a Failure Consequence Rating of very low (VL=1) to low (L=2) was adopted for Voorspoed in this Report for the purposes of pit closure.

Table 3-1: Design Confidence Rating descriptions (de Graaf, person. comms dated 12/12/2023)

Category		Geotechnical Closure Model Confidence I – Very Low Design & Operational verification result in high closure model uncertainty	II - Moderate Design & Operational verification result in moderate closure model uncertainty	III - Very High Design & Operational verification result in low closure model uncertainty
Site Investigation	Geological/ Structural/ Rockmass Model	High or unknown susceptibility to degradation Limited Site & Laboratory Investigation Empirical shear strength parameters Potential progressive failure from strain weakening materials	Moderate confidence structural/geological models Moderate susceptibility to degradation Site investigations & Laboratory Investigation to support operations Shear strength parameters based on laboratory programs with limited verification	High confidence structural/geological models Low susceptibility to degradation Verification of models based on operational knowledge base Validated or known conservative Shear Strength Parameters
	Hydrogeological model	Conceptual hydrogeological model with limited field data		Conservative Pore Pressures
Failure Mode Recognition		Potential instability mechanisms not defined/poorly understood Designs based on empirical parameters or literature	Potential instability mechanisms defined Designs based on material parameters based on laboratory testing, including some variability Peak Shear Strength Parameters (Waste Rock dumps) Empirical rockfall and failure runout models (waste rock dumps)	High confidence of potential instability mechanisms. Design including sensitivity analysis to understand influence of change of conditions Residual Shear Strength Parameters (Waste Dumps) Run-out Modelling for Rockfall & runout models (waste rock dumps)
Geochemical				
Design parameter selection				
Performance		History of significant slope deformation indicating poor knowledge of shear strength or hydrogeological conditions Unknown design or poorly documented operational knowledge base	Knowledge Basis consisting of Initial operation reconciliation and model verification, reasonably documented operational history and monitoring records.	History of limited slope deformation, or back analysis of deformation used to inform design modifications Knowledge base consisting of Long-term slope performance history with active slope monitoring database

VL = 4	L = 3	M = 2	H = 1	VH = 0

Table 3-2: Failure Consequence Rating descriptions (de Graaf, person. comms. dated 12/12/2023)

	Severity	Consequence of Failure on Rest of System	Degree of Intervention Required	Open Pit Slope/Waste Rock Landform Slope Failure or Rockfall Consequence
VL = 1 L = 2	Low (Slight/Minor)	*No cascading consequences, or consequences does not affect global stability*	*Integrity maintained, no intervention or maintenance required up to monitor or localised maintenance.*	*Natural Environment* Limited or no incremental (to inherent impact) adverse effects to the natural environment. Impacts are limited to low conservation value and remediation is possible without intervention. *Water:* No or Short-term degradation of surface/groundwater quality within the local water catchment. (i.e., *water table displaced by mine dewatering rebounds to natural water levels within operational timeframe or No ARD or material prone to chemical weathering material exposed above water table)* *Health & Safety:* First Aid Case - A reasonable person can identify any hazard that could pose a threat to safety. Not dissimilar to the safety risk associated to public access national parks. *Social & Cultural:* No impact to heritage, recreation, community, or cultural assets. *Infrastructure & Economics:* No impact to significant infrastructure
M = 3	Moderate	*Failure may have cascading consequences that do not result in global instability*	*Intervention or maintenance required to limit impact of cascading consequences*	*Natural Environment:* Loss of significant natural environment, flora or fauna that recovers without intervention. *Water:* Medium term degradation of surface/ground water quality within the local water catchment *Health & Safety:* Medical Treated Case - A reasonable person cannot identify that a hazard may pose a threat to safety but no credible outcome that could lead to serious injury or loss of life beyond what is accepted in anecdote natural environment (i.e., Natural park in proximity to site). *Social & Cultural:* Recoverable damage to items of cultural significance *Infrastructure & Economics:* Temporary loss of important infrastructure and economic repairs can be made.
H = 4	High	*Global instability, localised runout/area of influence*	*Intervention or maintenance required to maintain functionality*	*Natural Environment:* Extensive damage or loss to the natural environment, flora or fauna. Areas of National or greater significance. (i.e., National parks). Remediation requires intervention or will result in significantly altered ecosystem. *Water:* Long term degradation of surface/groundwater quality across multiple water catchments or loss of flow from creek system *Health & Safety:* Loss of life or serious injury is possible. *Social & Cultural:* National heritage or community facilities or cultural assets damaged. *Infrastructure & Economics:* Critical public infrastructure damaged, significant impact to local community.
VH = 5	Very High (Severe)	*Global instability, widespread runout/area of influence*	*Remediation not possible*	*Natural Environment:* Permanent/Extensive damage or loss to the natural environment, flora or fauna. Areas of National or greater significance. (i.e., National parks). Remediation requires intervention or will result in significantly altered ecosystem. *Water:* Permanent degradation of surface/groundwater quality across multiple water catchments or loss of flow from creek system *Health & Safety:* Multiple loss of life or serious injury is possible. *Social & Cultural:* Significant National heritage or community facilities or cultural assets destroyed. *Infrastructure & Economics:* Critical public infrastructure permanent loss with repairs not practicable.

Notes: *Assigned consequence should reflect the most likely outcome. If assigning consequence with consideration of the worst case or a combination of discrete outcomes, this must be declared. Community impacts must be determined through meaningful engagement with stakeholders and may include a consideration of health, loss of access/destruction of traditional lands, housing, destruction/damage of farmland, harm to livestock, damage to water or soil resources, impacts to trapping and fishing, loss of animals, overall cultural impact, and employment. Reputation, legal aspects, and economics are not considered in this consequence table as they are considered site - and corporation specific. It may be necessary to assess these aspects on a site - specific basis.*

3.4 Step 4: Evaluating existing pit wall conditions

Existing pit wall conditions are characterised by a Slope Condition Class ("SCC") as detailed in Figure 3-2 using SCC classes and ratings that vary from V (VH=5), IV (H=4), III (M=3), II (L=2) to I (VL=1). The existing pit slopes at Voorspoed are still actively sloughing and breaking back, and as a result, an SCC of V (VH=5) was adopted for Voorspoed in this Report for pit closure purposes, notwithstanding that parts of the Voorspoed pit may fall in an SCC of IV (H=4) – such as e.g., the inter-ramp slopes in Basalts.

Existing Pit Wall Conditions				
SCC	**Likely PoF (%)**	**Likely FoS***	**Assessed Reliability Index, β**	**Likely Long-Term Reliability**
V	> 10	≤ 1.2	< 1.0	Slopes with uncontrolled rockfall risk, and undesirable risk of failure
IV	5 – 10	1.2 – 1.5	1.0 – 1.2	Standard design reliability, slopes without rockfall control
III	1.5 – 5	1.5 – 1.8	1.2 – 1.5	Good design reliability, unvegetated slope, but with rockfall protection
II	0.5 – 1.5	1.8 – 2.0	1.5 – 2.5	Design reliability to routine civil design standards or higher incl. rockfall protection & berm drainage
I	< 0.5	> 2.0	> 2.5	Very high reliability, specific scenarios may require controls that far exceed credible hazard (e.g., vegetated slope to full highway design standards)

Figure 3-2: Existing pit wall conditions & suggested closure strategy, updated from Carter et al. (2022) (de Graaf, person. comms. dated 12/12/2023)

3.5 Step 5: Calculate RRR (or R^3) for Voorspoed open pit closure

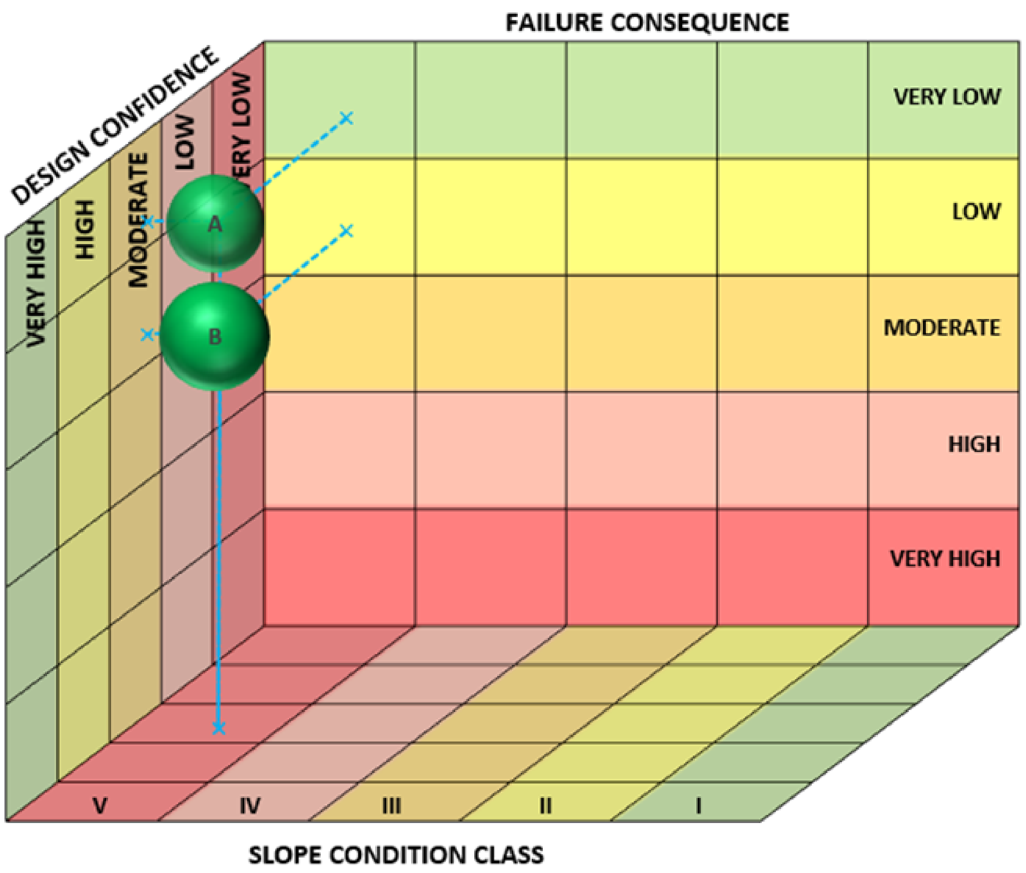
The Target Stability Reliability RRR (or R^3) for the Voorspoed pit is obtained by multiplying the three component ratings, namely the Design Confidence Rating, the Failure Consequence Rating, and the Slope Condition Class (or SCC), with the results tabulated in Table 3-3 and shown diagrammatically in Figure 3-3.

Table 3-3: Target Stability Reliability RRR Score for Voorspoed Mine

Rating Combination	Design Confidence Rating	Failure Consequence Rating	Slope Condition Class, SCC	RRR, or R^3
1	M = 2*	VL = 1*	VH = 5*	10*
2	L = 3			15
3	M = 2*	L = 2*		20*
4	L = 3			30

Note: * - These are considered baseline ratings

As can be seen in this Table, the R^3 score for Voorspoed, for the different rating ranges adopted, varies from 10 to 30 – with a base case $R^3 \leq 20$ for Voorspoed, for post-closure. The corresponding remedial actions in terms of this R^3 score for Voorspoed, can then be read off Table 3-4, based on which the Voorspoed R^3 level is considered 'low' with static FoS > 1.2 and PoF < 3.0% proposed as TSR criteria.



INITIAL CONDITION RANKINGS			
Slope Condition Class Rating	Design Confidence Rating	Failure Consequence Rating	ESR RRR Score
V = 5	VL = 4	VH = 5	80-100
IV = 4	L = 3	H = 4	60-80
III = 3	M = 2	M = 3	40-60
II = 2	H = 1	L = 2	20-40
I = 1	VH = 0	VL = 1	<20

SLOPE SECTOR	Slope Condition Class Rating	Design Confidence Rating	Failure Consequence Rating	Initial RRR Score
A	V	M	VL	10
B	V	M	L	20

Figure 3-3: Diagrammatic representation of base case RRR scores derived for Voorspoed pit closure

Table 3-4: Required remedial actions, updated from Carter et al. (2022) (de Graaf, person. comms. dated 12/12/2023)

<table>
<tr><th>R^3
Score</th><th>R^3
Level</th><th>Suggested TSR Static FoS*, **
& PoF</th><th>Recommended Actions</th></tr>
<tr><td>$R^3 > 80$</td><td>Extreme</td><td>FoS > 2.0
PoF < 0.5%</td><td rowspan="2">Comprehensive and extremely rigorous analysis required to develop a thorough understanding of failure mechanisms and behaviour to support implementation of extremely robust controls. Detailed numerical crest break-back and/or runout analyses required to establish permanent exclusion zones. Likely requires detailed supplementary field and laboratory investigations. Detailed evaluation of feasibility of alternatives/trade-off studies to reduce R^3 and achieve As Low As Reasonably Practicable (“ALARP”) objectives. Very high design reliability required.
FoS > 2.0 needed unless quantitative risk assessment (“QRA”) undertaken.</td></tr>
<tr><td>$60 < R^3 \leq 80$</td><td>Very High</td><td>1.8 < FoS ≤ 2.0
PoF < 0.75%</td></tr>
<tr><td>$40 < R^3 \leq 60$</td><td>High</td><td>1.5 < FoS ≤ 1.8
PoF < 1.0%</td><td>Rigorous analysis necessary so that thorough understanding of mechanisms is gained, so robust controls can be implemented. Detailed numerical runout analyses to establish permanent exclusion zones. May require supplementary field and laboratory investigations. Trade-off studies to assess feasibility of reducing R^3 and achieving ALARP objectives.
High design reliability required, FoS > 1.5</td></tr>
<tr><td>$20 < R^3 \leq 40$</td><td>Moderate</td><td>1.3 < FoS ≤ 1.5
PoF < 1.5%</td><td>Moderate level of reliability of all critical inputs required to support stability analysis. Standard analysis sufficient (empirical approaches), good understanding needed of scenario so that appropriate controls can be implemented. Empirical break-back and/or runout analyses to establish permanent exclusion zones. May require supplementary field and laboratory investigations. Confirm alignment with ALARP objectives.
Good design reliability required, FoS > 1.3</td></tr>
<tr><td>$R^3 \leq 20$</td><td>Low</td><td>1.2 < FoS ≤ 1.3
PoF < 3.0%</td><td>Evaluate reliability of all critical inputs to support stability analysis. Apply good practice, guidance and hierarchy of controls, may require targeted investigations to address areas where knowledge is lacking or unreliable. Confirm alignment with ALARP objectives.
Fair design reliability required, FoS > 1.2</td></tr>
</table>

* For relinquishment and passive closure objectives: One class lower for active closure or ongoing ‘care and maintenance’ objectives (FoS 1.2 minimum)

** For seismic design follow risk-based decision tree approach.

Application of the 'As Low As Reasonably Possible' or ALARP principle in terms of the RSG or the R^3 score is shown schematically in Figure 3-4. This approach improves risk mitigation during post-closure planning and design as follows:

- The proposed Relative Stability Guideline (RSG) concept (which was subsequently renamed Residual Risk Rating, R^3) provides a more effective way of communicating pit slope stability conditions and the consequences of slope instability, transparency is improved and a base case is provided for evaluation of post-closure risk and required remediation and/or pertinent mitigation.
- Importantly, the proposed RSG/R^3 approach must be applied case-by-case and in consultation with relevant stakeholders.
- Benchmarking against key geotechnical guidelines should promote industry discussion and standardisation.

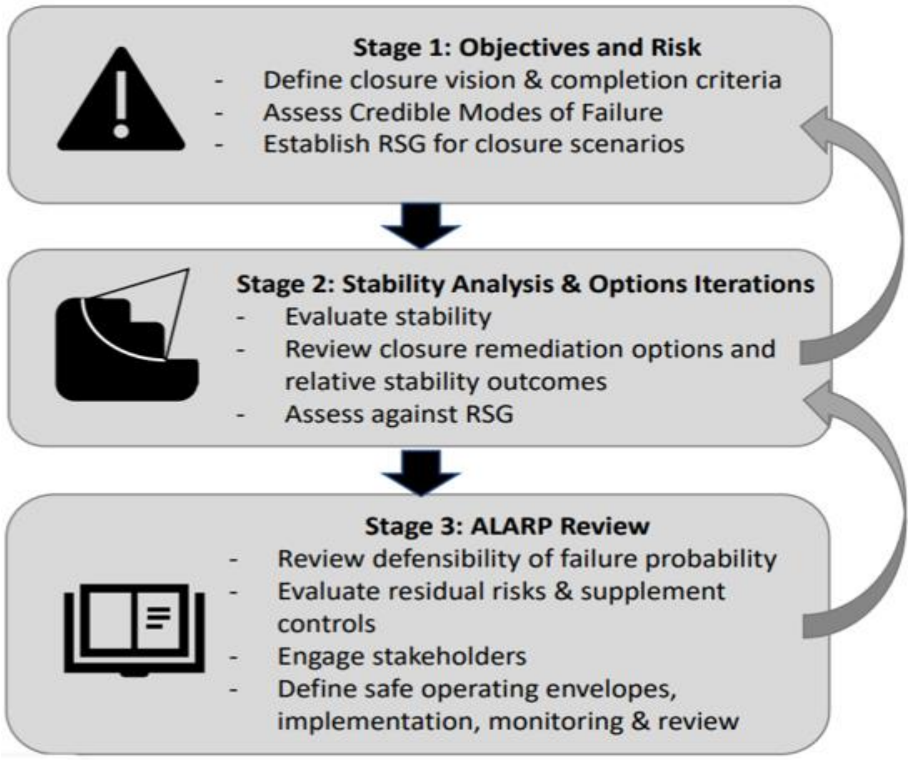


Figure 3-4: RSG/R^3 pathway to ALARP risk mitigation & management for open pit closure

In summary, the pit closure classification for Voorspoed shows that:

- The Voorspoed pit carries a low R^3 score of ≤ 20.
- Applicable TSR for determining safe break-back limits are: 1.2 < FoS ≤ 1.3 and/or PoF < 3.0%. Or to word it differently, a break-back limit with an FoS of between 1.2 and 1.3 and/or a PoF of less than 3.0%, would be acceptable from a geotechnical risk and design acceptance consequence point of view.
- Actions recommended for Voorspoed on this basis include the following:
  - Evaluate reliability of all critical inputs to support stability analysis.
  - Apply good practice, guidance and hierarchy of controls.
  - May require targeted investigations to address areas where knowledge is lacking or unreliable.
  - Confirm alignment with ALARP objectives.

- Fair design reliability required, FoS > 1.2.
- Indicative stakeholder position: Limited to no concern.
- Suggested controls:
  - Freely allowed, i.e., freehold on a property, no legal restriction.
  - Incidental superficial monitoring to no monitoring required.

As a final check of the TSR criteria adopted for Voorspoed as presented above, Kirsten (1983) provided PoF design acceptance guidelines with commentary on service life, public liability and minimum surveillance requirements as tabulated in Table 3-5. These recommendations by Kirsten (1983) were based on a detailed review including several back-analyses of the stability of slopes and of earth and rock-fill embankments and dams. The current condition of the slopes is highlighted in red, given that several slopes in the pit are actively breaking back currently.

Table 3-5: PoF design acceptance guidelines, Kirsten (1983)

	Design criteria			Aspects of natural situation	
PoF (%)	Serviceable life	Public liability	Minimum surveillance required	Frequency of slope failures	Frequency of unstable movements
50–100	None	Public access forbidden	Serves no purpose	Slope failures generally evident	Abundant evidence of creeping valley sides
20–50	Very very short-term	Public access forcibly prevented	Continuous monitoring with intensive sophisticated instruments	Significant number of unstable slopes	Clear evidence of creeping valley sides
10–20	Very short-term	Public access actively prevented	Continuous monitoring with sophisticated instruments	Significant instability evident	Some evidence of slow creeping valley sides
5–10	Short-term	Public access prevented	Continuous monitoring with simple instruments	Odd unstable slope evident	Some evidence of very slow creeping valley sides
1.5–5	Medium-term	Public access discouraged	Conscious superficial monitoring	No ready evidence of unstable slopes	Extremely slow creeping valley sides
0.5–1.5	Long-term	Public access allowed	Incidental superficial monitoring	No unstable slopes evident	No unstable movements evidence
<0.5	Very long-term	Public access free	No monitoring required	Stable slopes	No movements

That said, the RSG/R[3] break-back limit at PoF < 3.0%, will be located at a position far enough back from the crests of the current pit slopes, where there will be no ready evidence of instability behind this limit line post-closure. Furthermore, the only mitigation measures required- behind this break-back limit line comprise (i) discouraging public access, and (ii) conscious superficial monitoring. It follows that the geotechnical risk and design acceptance criteria adopted, are largely aligned with findings of this detailed review by Kirsten (1983).

4 SLOPE STABILITY ASSESSMENT & BREAK-BACK EXTENT DELINEATION FOR PIT CLOSURE

4.1 General

The pit slope stability analyses for pit closure were carried out using Rocscience Slide2 limit equilibrium software:

- Slide2 is commonly used for evaluating the safety factor and/or probability of failure, of both circular or non-circular failure surfaces in soil or rock slopes.
- The program analyses the stability of slip surfaces using vertical slice or non-vertical slice limit equilibrium methods. For the Voorspoed pit closure analyses, vertical slices were used.
- Slide2 also includes groundwater pressures in the analyses, through a defined phreatic surface, a groundwater coefficient R_u, or through finite element groundwater seepage analysis for both steady-state and transient conditions. For the Voorspoed pit closure analyses, phreatic surfaces were used to define steady-state groundwater pressures, and a groundwater coefficient Ru was used to model transient groundwater pressures.

Stability analyses for Baseline static load conditions (see Section 4.2) were carried out using probabilistic methods to account for uncertainty in rockmass strengths, and two groundwater scenarios were considered for the Baseline analyses, namely an interim groundwater regime about 10years after mining stopped, and an ultimate steady-state pit lake and groundwater regime 200years after mine closure.

Following these Baseline analyses, the following ‘special’ or ‘worst case’ scenarios are considered reasonably foreseeable and credible changes that may occur, also when considering the Voorspoed Geotechnical Model, and anticipated mechanisms of slope degradation and instability that may develop in the long-term post-closure:

- **Seismic loading**: Stability of the Voorspoed pit slopes post-closure during seismic loading, by an earthquake with a 1:2,475 year return period (see Section 2.3).
- **Option 2 (Golder, 2020)**: Stability of the Voorspoed pit slopes post-closure for a pit lake level higher than the 200year projected pit lake level (Golder, 2020) because of additional stormwater runoff from the surrounding area being directed into the Voorspoed pit (see Section 4.3.3).
- **Climate change**: Stability of the Voorspoed pit slopes post-closure during a period of prolonged rainfall resulting in transient groundwater pressures developing in the pit highwalls above the steady-state phreatic surface (see Section 4.3.4).

This was followed by a review of long-term erosion and degradation of the Voorspoed pit slopes post-closure (see Section 4.4).

4.2 Baseline analysis results for static load conditions

4.2.1 Using probabilistic methods in slope stability analyses

The objective of employing probabilistic methods in the post-closure stability analyses was to account for uncertainty in rockmass strength parameters used to characterise the different rock units when estimating the long-term break-back distance from the pit crest.

In a probabilistic analysis, a Monte Carlo simulation is used in Slide2 to statistically generate material parameters in terms of defined means, standard deviations, distribution types, and coefficients of variation. The resultant Probability of Failure ("PoF") is defined as the probability (as a %) that the Factor of Safety FoS < 1 (i.e., the slope is unstable) for the critical slip surface.

Finally, note that the approach adopted in the probabilistic evaluation presented here, was based on calculating the PoF for at least three critical slip surfaces at various distances back from the pit crest – typically 50, 100 and 150m, with the results then used to determine break-back distances for PoF values of 5% and 1% respectively through interpolation.

4.2.2 Summary of Baseline Slide2 models completed

The stability analyses were conducted for the eight design cross-section lines shown in Figure 2-4. Design section slope geometries were based on the 2018 pit geometry and the most up-to-date information on geological, structural, groundwater and geotechnical conditions as summarised in Section 2 of this Report.

Table 4-1 provides a summary of the modelling approach adopted in Slide2 for the Baseline analyses for static load conditions, while the slip surface shape and search approach adopted is schematically illustrated in Figure 4-1.

Table 4-1: Slide2 modelling approach summary for Baseline analyses completed

Model aspect/parameter	Description
Design sections analysed	S1, S2, S3, S4, S5, S6, S7, and S8
Break-back distances analysed	Typically 3 (No), at 50, 100 and 150 m from the crest
Groundwater scenarios considered	
Short-term pit lake level	10-years after mining stopped – interim state[1]
Long-term lake level	200-years post-closure, ultimate steady-state
Slope stability analyses	
Methods employed	Janbu corrected, Spencer
Slip surfaces:	Non-circular
Search method:	Cuckoo with surface altering optimisation
Strength parameters	
Failure criterion used	Mohr-Coulomb
Coefficient of variation of cohesion	30%
Coefficient of variation of friction angle	15%
Correlation coefficient between cohesion and friction angle	0
Monte Carlo simulation	
Sampling method employed	Latin hypercube
Number of simulations per model run	6,000

Notes: 1 – The pit lake level after 10years is a much lower pit lake level compared to that at ultimate steady-state (after 200years), and therefore represents a worst-case scenario given that overall stability conditions generally improve in all pit design sectors at Voorspoed as the pit lake rises long-term.

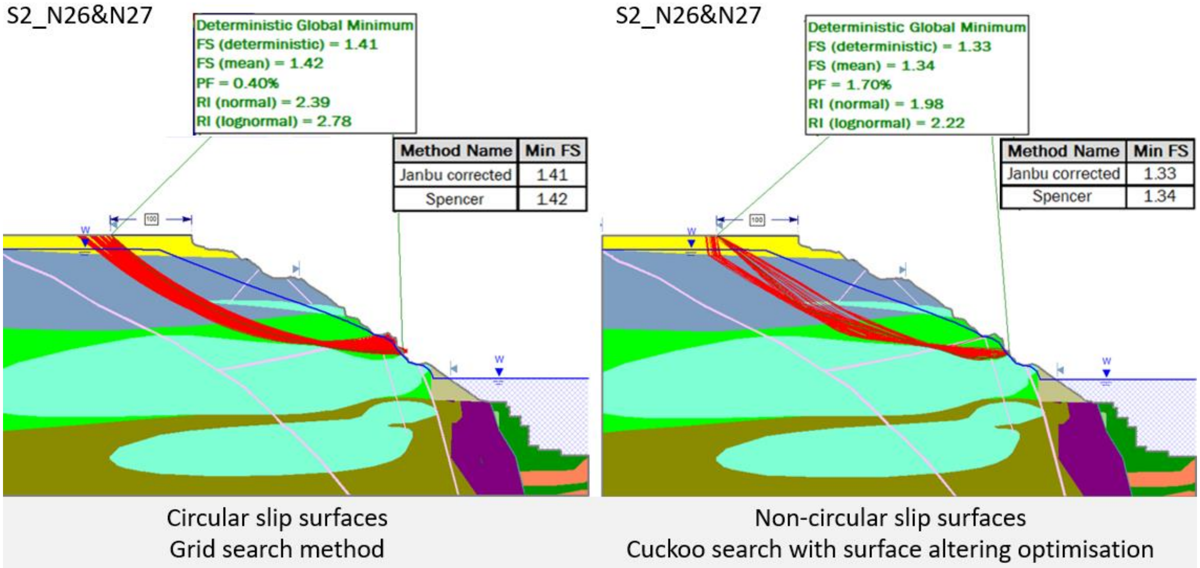


Figure 4-1: Schematic illustration of slip surface shape & search method approach adopted in Slide2 in Baseline analyses for static load conditions

4.2.3 Baseline analysis results for static load conditions

Baseline analysis results for static loading conditions and for pit lake levels 10-years and 200-years after mine closure, with phreatic surfaces as determined by Piteau (2018) are tabulated in Table 4-2. **Note that break-back ("BB") distances as -indicated in this Table, were measured from the 2018 pit crest position on the various design sections. Interpolated BB distances for the required PoF limits (1% and 5%) and FoS limits (1.3 and 1.5) are listed in the columns on the far right in this Table.** These break-back limits are shown schematically in Figure 4-2 for PoF of 1% and 5% respectively, and in Figure 4-3 for FoS of 1.3 and 1.5 respectively (also see cross-reference to these Figures at the bottom of the far right columns in Table 4-2) in relation to the current security fence position, and superimposed on the 2018 LiDAR image of the Voorspoed pit. Note that these break-back limit lines between design section lines are best estimates, based on pit crest geometry, local geology and precedent experience.

From these Figures, the following are noted in relation to the TSR criteria adopted for Voorspoed pit closure as presented in Section 3.5:

- The BB limits for a PoF of 1% and 5% respectively, plot outside the current security fence in several areas. The TSR PoF limit of 3%, which will plot in-between the PoF 1% and 5% BB limits in Figure 4-2, will therefore also fall outside the existing security fence in these areas.
- Similarly, the BB limits for an FoS of 1.3 and 1.5 respectively, plot outside the current security fence in several areas. The TSR FoS limit of 1.2, which will plot just inside the FoS 1.3 line in Figure 4-3, will therefore also fall outside the security fence in these areas.
- It follows that the security fence will require repositioning in areas where the predicted TSR break-back limits extend beyond the current security fence line, to appropriately mitigate the associated risks.

The Baseline analysis results from Slide2 as discussed and presented above, are included in Appendix C.

Table 4-2: Summary of Baseline stability analysis results for static loading conditions & pit lake levels 10-years and 200-years after mine closure respectively

Design Section (see Figure 2-4)	Years after Pit Closure	Interpolated break-back distances behind 2018 Voorspoed pit crest position: BB distance (m)		BB distance (m)	
		PoF 5%	PoF 1%	FoS 1.3	FoS 1.5
S1 (SW23)	10	0	0	0	0
	200	0	0	0	0
S2 (N26&N27)	10	0	127	52	>150
	10	57	63	52	59
	200	-	-	-	-
	200	55	59	51	56
S3 (N28)	10	160	222	179	>230
	200	0	94	0	>100
S4 (NE10&NE11)	10	44	150	60	>150
	200	0	67	0	>100
S5 (NE3&NE6)	10	-	-	-	-
	10	93	113	81	111
	200	-	-	-	-
	200	93	113	81	112
S6 (Basalt Raft)	10	-	-	-	-
	10	15	29	3	18
	200	-	-	-	-
	200	15	29	3	18
S7 (N11&N12)	10	-	-	-	-
	10	73	100	68	107
	200	73	100	68	107
S8 (SSE2)	10	-	-	-	-
	10	97	119	65	92
	200	106	121	62	93
		See Figure 4-2		See Figure 4-3	

Note: The greyed values in this Table correspond to 200-year cases where the minimum FoS is known from the 10-year case because the critical slip surface is independent of the pit lake level. These values are included in the Table because they are used as points of reference for interpolated estimation of the break-back distances for the PoF 5% and 1% TSR limits.

A further set of Baseline analyses were carried out as a check on the above, in which phreatic surfaces were adjusted on four of the design section lines to better account for the 200-year after mine closure pit lake level, local geology and pit geometry. The results of these additional Baseline analyses are tabulated in Table 4-3 and shows FoS values well in excess of the FoS > 1.2 TSR limit adopted for Voorspoed.

Table 4-3: Additional Baseline analysis results for optimised phreatic surfaces, for pit lake levels 200-years after mine closure

Design Section	Slip surface search area	FoS at Security Fence	BB (m) at FoS=1.5	BB (m) at FoS=1.3
S3 (N28)	wide – whole crest area	1.50	140 m	52 m
	narrow – just back from security fence	1.38		
S4 (NE10&NE11)	wide – whole crest area	1.64	88 m	47 m
	narrow – just back from security fence	1.40		
S7 (N11&N12)	wide – whole crest area	2.25	27 m	20 m
	narrow – just back from security fence	1.83		
S8 (SSE2)	wide – whole crest area	1.61	93 m	62 m
	narrow – just back from security fence	1.53		

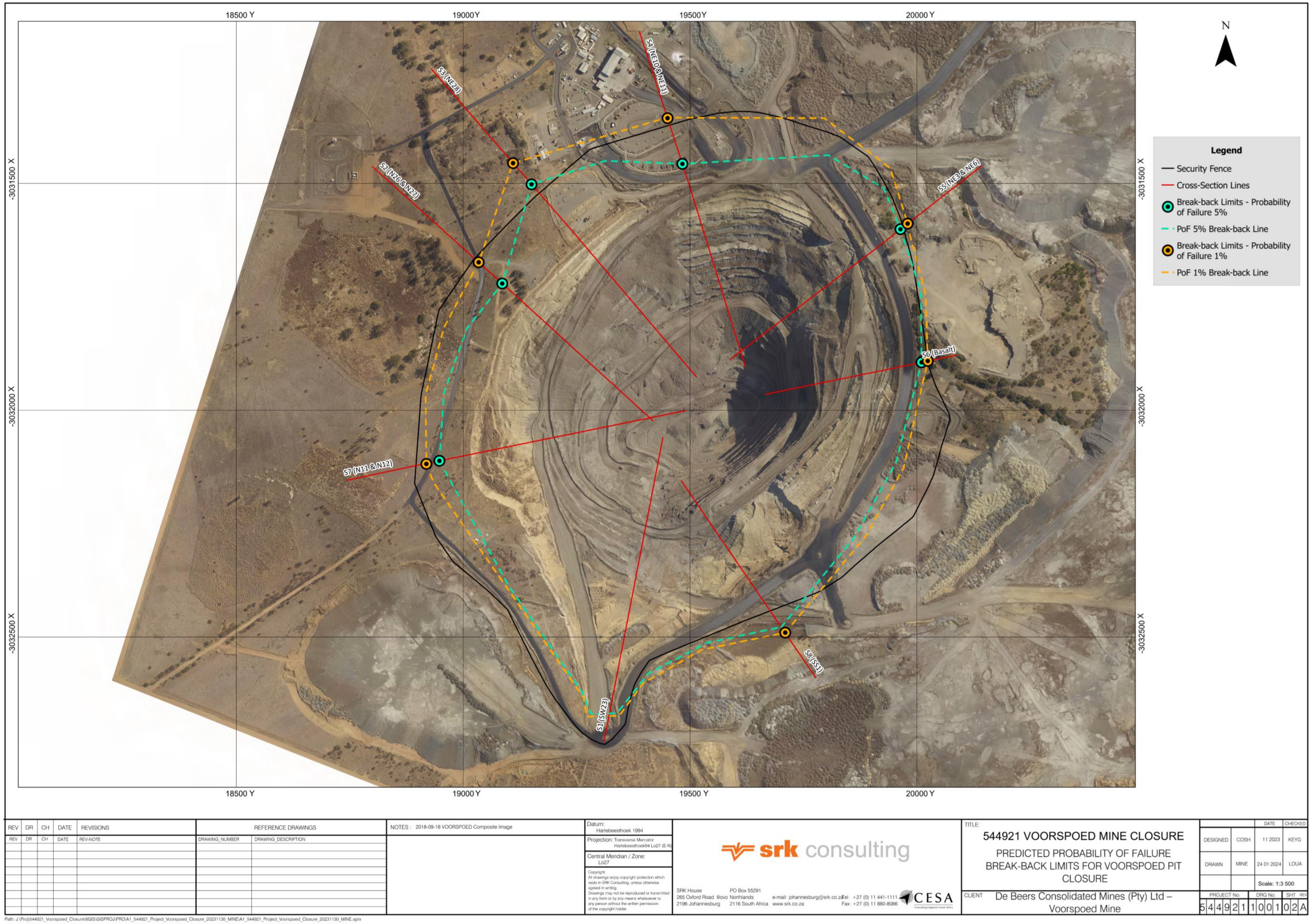


Figure 4-2: Break-back lines for PoF limits of 1% and 5% respectively, from Baseline analyses for static load conditions after Voorspoed mine closure

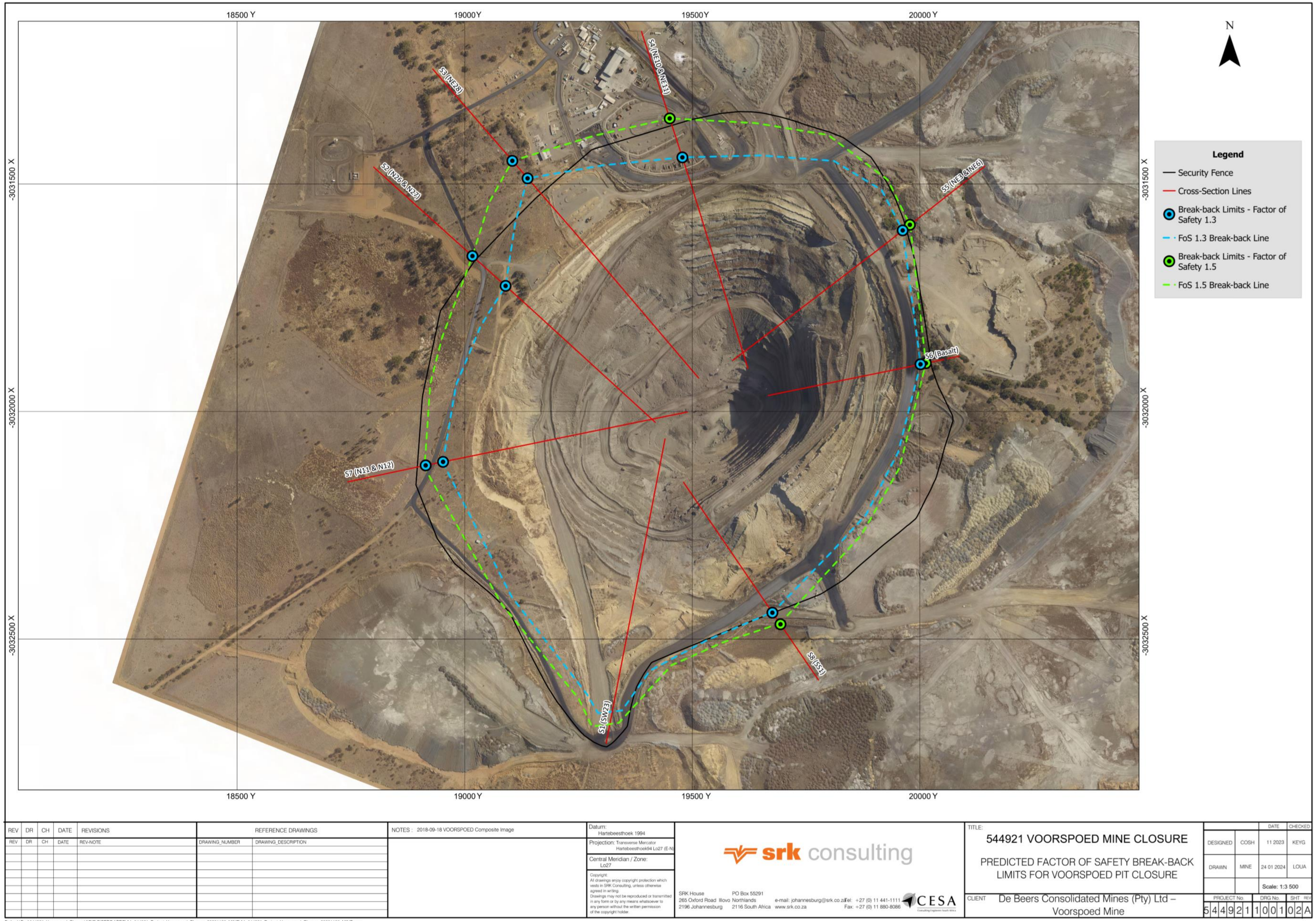


Figure 4-3: Break-back lines for FoS limits of 1.3 and 1.5 respectively, from Baseline analyses for static load conditions after Voorspoed mine closure

4.3 Analysis of Special Cases

4.3.1 Introduction

The stability of the Voorspoed pit slopes were also evaluated for several Special Cases as described in the following subsections. These generally represent extreme loading cases that the slope may temporarily be subjected to – for example:

- A large earthquake event which results in dynamic loading of the pit slopes.
- Transient pore pressures that may develop in the pit slopes following a period of extreme, prolonged storm rainfall.

As such and as is commonly done in engineering, the acceptability criteria adopted for these Special Cases was based on a determination of the FoS at the security fence position, with an FoS = 1.0 for slip circles extending up to the security fence position deemed acceptable as that indicates that the slope up to the security fence position will remain stable during the worst-case loading scenarios considered.

4.3.2 Worst-Case – Scenario 1: Seismic loading

Seismic coefficients for a design earthquake event with a 1:2,475-year return period as discussed in Section 2.3 were applied as a Worst Case in Slide2 models for four select design sections, with the results as tabulated in Table 4-4.

Table 4-4: Summary of Worst Case – Scenario 1 seismic loading results

Design Section	Design seismic coefficients applied	FoS at Security Fence	BB (m) at FoS=1.5	BB (m) at FoS=1.3
S3 (N28)	Horizontal seismic coefficient k_h = 0.140 Vertical seismic coefficient k_v = k_h/6 = 0.023	0.97	> 220 m	> 220 m
S4 (NE10&NE11)		1.06	> 200 m	> 200 m
S7 (N11&N12)		1.40	136	81
S8 (SSE2)		1.14	> 218 m	165 m

Based on the results in Table 4-4, the following is concluded:

- The security fence is generally located far enough away from the edge of the pit crest, to ensure stability of the fence in the event of a large earthquake.
- For Design Section S3, the fence line in this position will only be marginally stable in this extreme scenario and therefore will locally require repositioning outwards to ensure the above acceptability criterion is met. However, this is in an area where the fence have to be extended already in terms of predicted BB limits in Baseline analyses completed for static load conditions (see Section 4.2.3) and no additional mitigation is therefore required because of the results from this seismic analysis.

The Slide2 model results for these worst-case seismic loading scenarios as discussed and presented above, are included in Appendix D for reference.

4.3.3 Special Case – Scenario 2: Higher pit lake elevation (Option 2 – Golder, 2020)

The stability of the Voorspoed pit slopes post-closure for a pit lake level higher than the 200year projected pit lake level (Option 2 - Golder, 2020) because of additional stormwater runoff from the surrounding area being directed into the Voorspoed pit, was evaluated in Slide2 models for four select design sections, with the results as tabulated in Table 4-5.

Table 4-5: Summary of Special Case - Scenario 2: Higher pit lake elevation

Design Section	Pit lake level	FoS at Security Fence	BB (m) at FoS=1.5	BB (m) at FoS=1.3
S3 (N28)	The pit lake level 200-years after mine closure, was raised by a further 30 m	1.59	97	-
S4 (NE10&NE11)		1.80	59	7
S7 (N11&N12)		2.47	35	19
S8 (SSE2)		1.54	110	65

Based on the results in Table 4-5, the following is concluded:

- The existing security fence is located far enough away from the edge of the pit crest, to ensure stability of the fence in the event of a higher pit lake elevation developing in the long-term, whether because of additional runoff from the surrounding area being directed into the Voorspoed pit, or because of extreme storm rainfall in some years.

The Slide2 model results for these special case raised pit lake scenarios as discussed and presented above, are included in Appendix D for reference.

4.3.4 Worst-Case – Scenario 3: Transient groundwater conditions during periods of prolonged rainfall

The stability of the Voorspoed pit slopes post-closure for transient groundwater pressure conditions above the steady-state phreatic surface because of prolonged and heavy rainfall, was evaluated in Slide2 models for four select design sections, with the results as tabulated in Table 4-6.

Table 4-6: Summary of results transient groundwater pressure scenarios

Design Section	Transient pore pressure condition[1]	FoS at Security Fence	BB (m) at FoS=1.5	BB (m) at FoS=1.3
S3 (N28)	R_u = 0.15 (high transient pressure)	1.48	150 m	50 m
	R_u = 0.30 (extreme transient pressure)	1.47	150 m	60 m
S4 (NE10&NE11)	R_u = 0.15 (high transient pressure)	1.61	91 m	54 m
	R_u = 0.30 (extreme transient pressure)	1.55	100 m	65 m
S7 (N11&N12)	R_u = 0.15 (high transient pressure)	2.20	20 m	20 m
	R_u = 0.30 (extreme transient pressure)	2.25	20 m	20 m
S8 (SSE2)	R_u = 0.15 (high transient pressure)	1.50	125 m	92 m
	R_u = 0.30 (extreme transient pressure)	1.30	> 125 m	125 m

Note: 1 – Recent work indicates a typical R_u value of 0.1 for modelling high transient groundwater pressures in most rock masses; the analysis results presented in this Table are therefore very conservative.

Based on the results in Table 4-6, the following is concluded:

- The existing security fence is located far enough away from the edge of the pit crest, to ensure stability of the fence in the event of high/extreme transient groundwater pressure conditions developing in the Voorspoed pit slopes above the ultimate steady-state phreatic surface because of prolonged and heavy rainfall.

The Slide2 model results for these worst-case transient groundwater pressure scenarios as discussed and presented above, are included in Appendix D for reference.

4.4 Review: Long-Term Erosion & Slope Degradation

4.4.1 Introduction

Predicting erosion rates and resultant extent of erosion long-term, and weathering/rheological processes acting in pit slopes following closure, in complex geological terrane comprising several different lithologies as exposed in the Voorspoed pit slopes, is very challenging due to the multitude of variables involved and uncertainty in how these may change over time. As a result, reviews of long-term erosion and slope degradation such as this, is usually based on the following:

- A review of the weatherability of the different lithologies exposed in the pit slopes.
- A review of historical break-back in different parts of the pit, and in different pit slope lithologies,

Consequently, resultant predictions are typically limited to qualitative engineering judgements.

These aspects are reviewed and discussed in the following subsections insofar it pertains to the Voorspoed pit slopes.

4.4.2 Weatherability of lithologies exposed in the Voorspoed pit slopes

The Karoo mudrocks exposed in the Voorspoed pit slopes, and more specifically the VRM and VRSSC upper units, are expected to continue degrading because of weathering and erosional processes, even after the slopes have reached stable configurations because of ongoing, long-term break-back. That said, by the time that manmade slopes in weak Karoo mudrocks reaches a flat enough, quasi-stable configuration, vegetation (initially grasses and shrubs, followed later by trees) starts establishing on the slopes, with the plant roots providing reinforcement to sloughed material and weathered faces, which starts countering erosion rates.

Ultimately this will lead to a stable landform, provided stormwater management systems and controls remain place around the pit crest, to prevent concentrated runoff from entering the pit. These weak Karoo mudrocks are known to be dispersive, and highly erodible, so dongas will rapidly form, with vegetation washed out or disturbed. This will trigger local undercutting and break-back, which is likely to grow and extend into adjacent areas on affected pit slopes.

To reach a point of quasi-stability, both from a slope stability and erosion point of view, and for good vegetation cover to establish on the pit slopes in Karoo mudrocks, typically takes decades/a century or more. A case in point being Kimberley Big Hole (see Figure 4-4), where mining stopped in the early 1900s, yet break-back processes and erosion are still ongoing to this day on the Big Hole slopes – albeit that the rate of break-back because of slope instability has reduced significantly over time, with no major slope instability/highwall sloughs recorded at the Big Hole for the last several decades.

Figure 4-4: Photo of Kimberley Big Hole (Source: G. Keyter, Sep 2023)

That said and with reference to Figure 4-4, note that a more competent capping layer at surface, comprising a dolerite sill typically 10 to 15 m thick, provides a level of protection against surface erosion, which has slowed down erosion rates at the Big Hole significantly. However, at Voorspoed, such a more competent capping layer does not exist, and higher rates of erosion are therefore expected, especially in those lithologies that are more weatherable and dispersive such as the VRM and VRSSC upper units.

Again, as noted in the introduction above, predicting the degree of weatherability and associated erosion rates for the different lithologies at Voorspoed, is very challenging, except qualitatively so - i.e., higher erosion rates are expected in the soft rocks (e.g., the VRM and VRSSC upper units) whereas erosion rates in the hard rocks (e.g., in the kimberlites and basalts) are expected to be low. That said, an evaluation of historical break-back in the different lithologies currently exposed in the Voorspoed pit slopes, provides some indication of what to expect in the longer term.

4.4.3 Evaluation of historical break-back in Voorspoed pit slopes

Annual LiDAR scans of the Voorspoed pit were used to generate annual pit surfaces through which 2D vertical cross-sections were generated along a selection of design section lines (see Appendix E). These 2D vertical cross-sections were georeferenced in CAD in terms of x- and y-coordinates (i.e., horizontal distance) and elevation, to allow accurate measurement, year-on-year, of slope angles in different lithologies exposed in the pit.

That said, given that the Voorspoed slopes are still actively breaking back, three angles were typically measured in a lithology where possible, year-on-year, where possible:

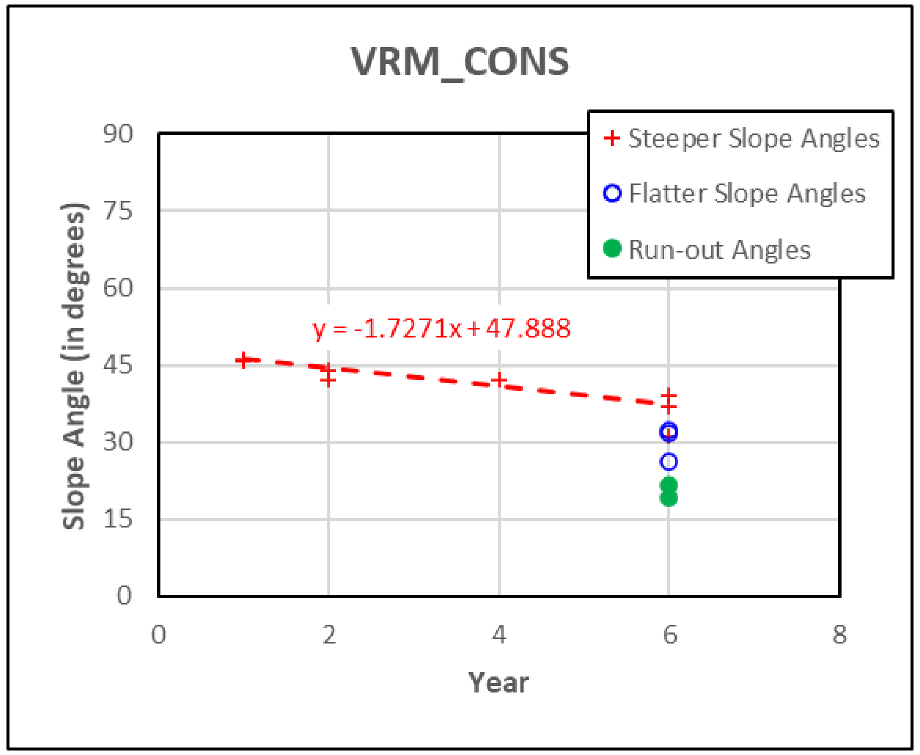
- Steeper Slope Angles: These measurements were typically taken higher up in a slope in a specific lithology, in what generally constitute the steeper back scar of a slope that is actively breaking back.
- Flatter Slope Angles: These measurements were typically taken lower down on a slope in a specific geology, in what generally constitute flatter, stabler parts of the slope below areas of active break back higher up the slope.
- Run-out Angles: These measurements were taken where what appears to be either failed or eroded material has run out at the base of a slope in a specific geology. Note that these angles are not necessarily the angle of repose of the material (although it could be under certain circumstances).

The measurements are presented graphically in Figure 4-5 with 2018 constituting Year 1, 2019 constituting Year 2 and so forth. Based on these Figure, the following conclusions can be made:

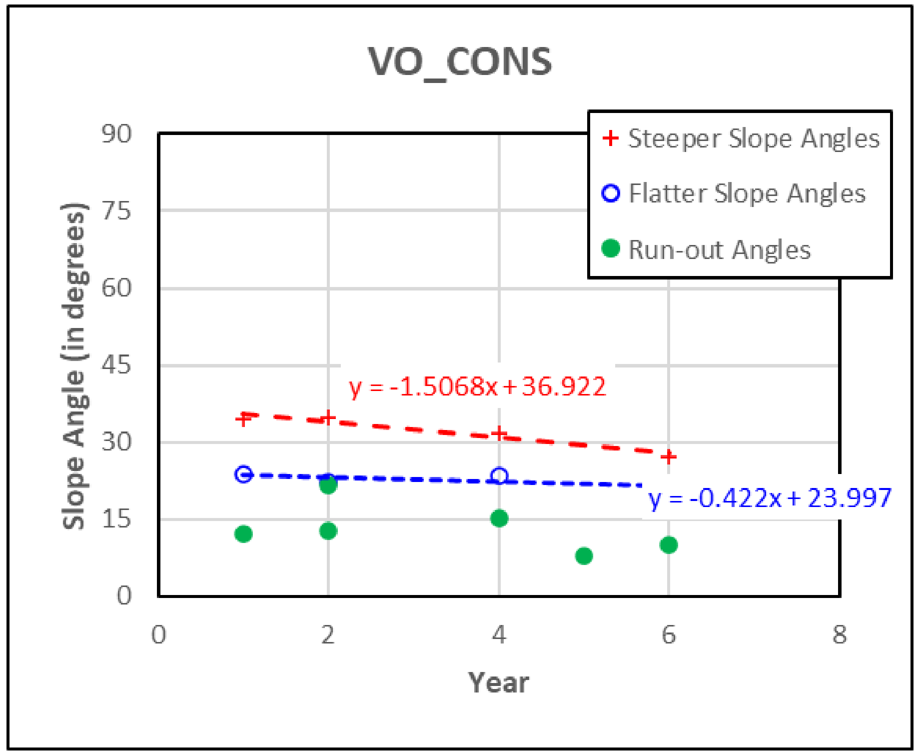
- Active rates of break-back measured in the Karoo mudrocks are as follows:
  - VRM_CONS: ~1.7 degrees/year
  - VO_CONS: ~1.5 degrees/year
  - VO_RIP: ~0.4 - 0.6 degrees/year
- Active rates of break-back in the Basalt are as follows: ~1.1 degrees/year (although it must be noted that the section of slope in basalt was stable followed by local crest breakout in Year 6 (2023) which resulted in overall flattening of the basalt slope which is measured from toe to crest).
- Steeper Slope Angles measured higher up the slopes, reduces at a faster rate than the Flatter Slope Angles measured lower down the slopes, and tend towards the Flatter Slope Angles over time.
- Flatter Slope Angles measured in slopes in Karoo mudrocks typically vary between 23 and 30 degrees, with a minimum of 18 degrees and a maximum of 32 degrees.
- The much flatter Run-out Angles of failed/eroded material at the base of a slope, typically varies between 10 and 20 degrees, with a minimum of ~8 degrees and a maximum of ~22 degrees.

From precedent experience, vegetation on the pit slopes will only start establishing itself once slope angles have failed back and eroded to angles of 15 to 18 degrees, and active break-back and erosion will continue, albeit at reduced rates over time, until such quasi-stable condition is reached (probably a few/several decades from now). Note that such quasi-stable, vegetated configuration, with a 15 degrees overall slope angle, formed the basis for setting out the current security fence line when it was constructed in 2018.

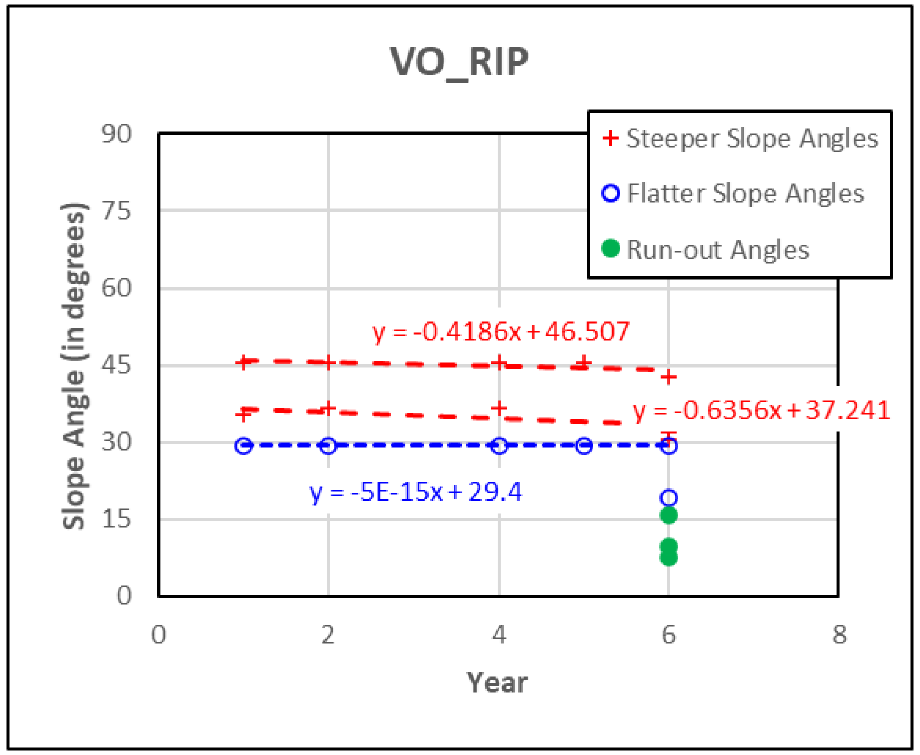
Based on this evaluation of historical break-back rates in the Voorspoed pit, and precedent experience elsewhere, it is not expected that break-back and erosion will extend beyond the current security fence position – provided stormwater runoff around the Voorspoed pit is controlled such that no concentrated runoff into the pit occurs, causing local erosion, and triggering more break-back than currently expected.



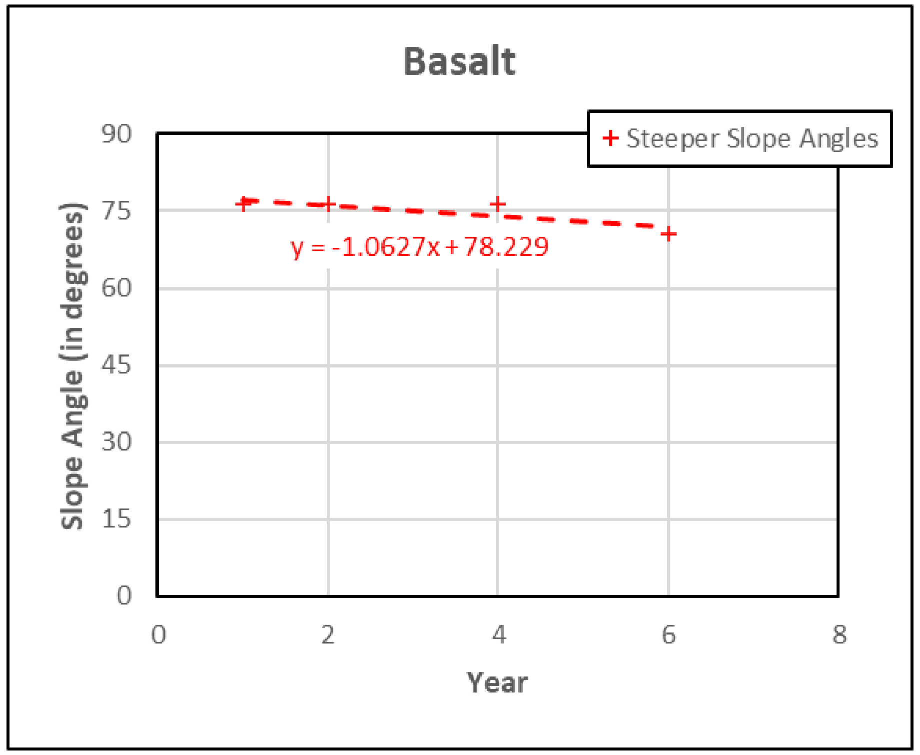
(a) Measured angles in VRM_CONS



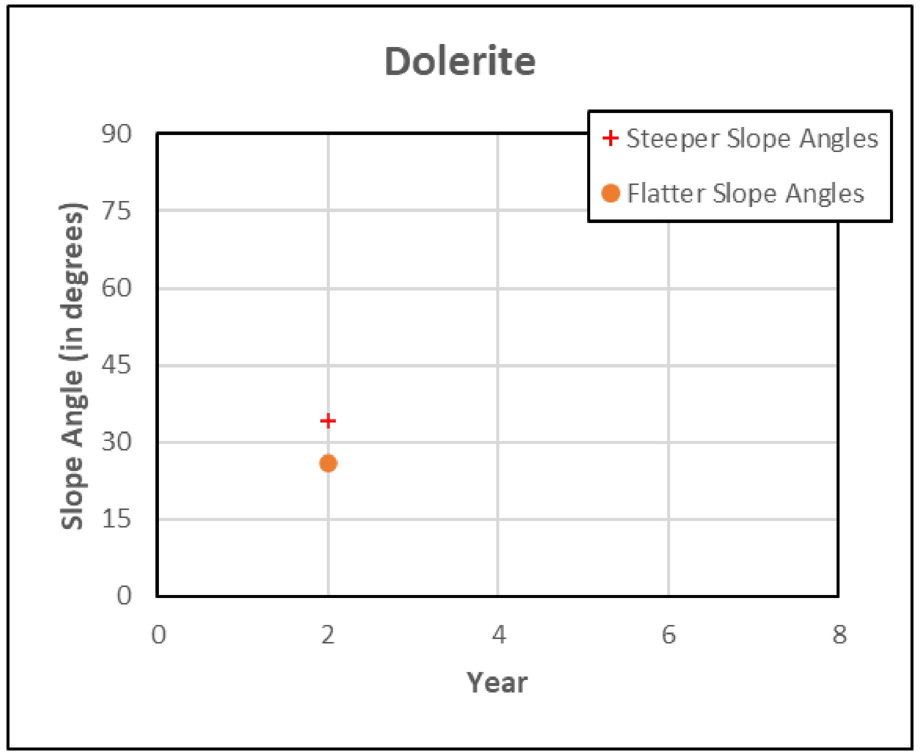
(b) Measured angles in VO_CONS



(c) Measured angles in VO_RIP



(d) Measured angles in Basalts



(e) Measured angles in Dolerites

Figure 4-5: Measured break-back angles in different lithologies exposed in Voorspoed pit slopes

5 STORMWATER MANAGEMENT

J&W reviewed post-closure stormwater management requirements at Voorspoed Mine in a report titled *'Voorspoed Mine Consolidated Storm Water Management Plan - Draft Report'* (Reference: J&W Report No. Jw355/22/J767 - Rev B, 2022). This J&W report provided a good synopsis of requirements to manage stormwater runoff water around the Voorspoed pit. In addition, the designs meet the necessary closure requirements in terms of pertinent South African legislation, as well as applicable Anglo-American ("AA") Standards, Procedures & Guidelines.

Conceptual engineering designs have been prepared and included in the J&W report for the proposed engineering works to manage stormwater around the Voorspoed pit. These designs are summarised below:

- An emergency spillway from the FRD to the south is proposed, which will then flow into a canal to convey any overspill to the Voorspoed pit.
- An abandonment bund and trench should be constructed to the west of the pit outside of the break-back zone, to divert stormwater away from the open pit.
- That said, areas of surface water ponding were identified within the anticipated break-back zone, and the crest of the pit in such areas of water ponding should be sloped such that surface water flows away from the pit, to minimise surface runoff flows into the open pit which can cause erosion and trigger additional break-back and associated slope instability. Specific areas to be reshaped included:
  - The surface area immediately north of the open pit, which lies within the break-back zone. This area must be reshaped and made free-draining to minimise depression points (where water currently ponds). Runoff water from this area will flow into the proposed emergency spillway canal from the FRD to the pit.
  - To the south-east of the open pit, a paddock system is proposed to manage stormwater in the area between the open pit and rehabilitated WRD. The paddock system has been designed to fall outside the break-back zone and to accommodate inflows resulting from the Probable Maximum Flood ("PMP") event.

Key items that will need to be addressed during detailed design are as follows:

- Where the proposed emergency spillway canal and abandonment bund intersect, the design of the bund and trench will need to be altered to allow them to end at the emergency spillway canal thereby preventing the trench from discharging into the canal.
- Mitigation measures identified in the J&W report that are critical to the erosion protection requirement and to meet long-term post-closure objectives, these mitigation measures will need to be constructed as per the design.
- The borrow area (i.e., basalt stockpile) at the south-eastern corner of the FRD Phase 2, which will be removed and used within the plant area rehabilitation.
- Further clarity may be required to understand how much water will be discharged to the open pit from the FRD, and the frequency of such discharges once the proposed spillway has been constructed through the FRD wall.

The J&W proposed layout of the emergency spillway, abandonment wall, paddock area and reshaping area around the Voorspoed open pit is presented in Figure 5-1.

Finally, it is recommended that additional work be carried out during detailed design to determine whether the FRD spillway canal can be discharged over more competent material (e.g., over the Basalt), rather than taking the shortest route over the edge of the north slope in Karoo mudrocks as currently proposed by J&W. This will reduce the potential for undercutting of the northern pit slope.

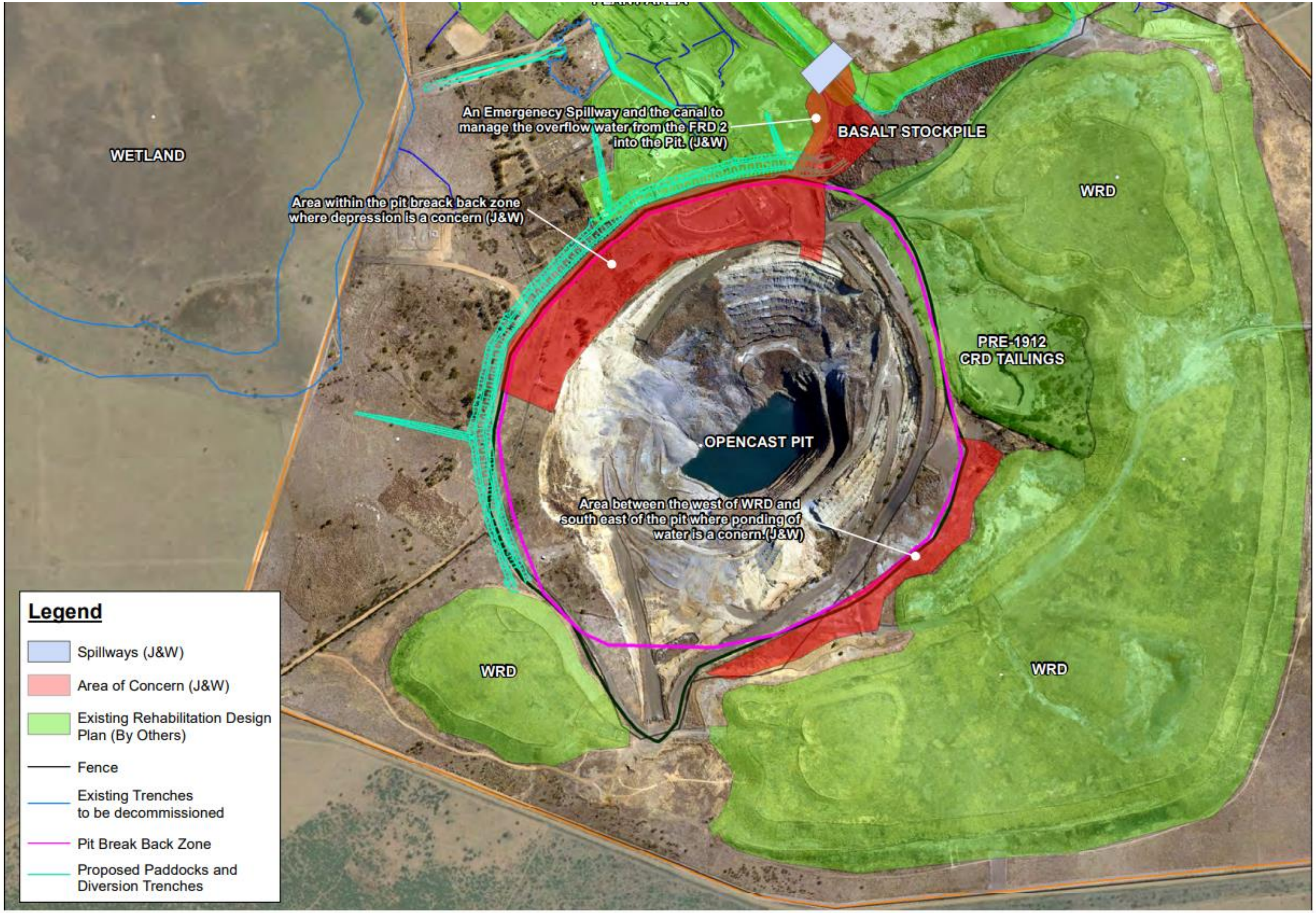


Figure 5-1: Proposed stormwater controls around the Voorspoed pit (J&W, 2022)

6 CONCLUSIONS & RECOMMENDATIONS

A detailed and comprehensive geotechnical closure study has been carried out to address the recommendation that *"a geotechnical study must be conducted to determine the pit stability for all the scenarios outlined in the geochemical report"* as prescribed in a DMRE letter dated 29/09/2023 (DMRE Ref.: FS 30/5/1/2/3/2/1 (12) EM), with the study findings and conclusions presented in this Report:

- The geotechnical study includes detailed Baseline stability analyses of the Voorspoed pit slopes, for static load conditions and different pit lake elevations, as well as several worst-case conditions and special scenario sensitivity assessments. The results of these analyses and reviews, support the Base Case pit closure design, which is Option 1 in the geochemical report (Golder, 2020).
- Separate stability analyses are included as a Special Case for Option 2 (Golder, 2020), which in effect constitutes a variation of Option 1.
- In Option 3 (Golder, 2020) there is no possibility of any slope instability given that the pit walls will be fully buttressed to surface, which did not warrant any additional geotechnical assessment or stability analysis.

The Voorspoed Geotechnical Model and its various components (i.e., local geology, structure, hydrogeology, seismic) were reviewed in detail, and it was concluded that with the supplemental scenarios and sensitivity assessments for extreme loading events undertaken, it is generally suitable to review pit slope stability at Voorspoed post-closure given anticipated mechanisms of slope degradation and the possible extent thereof, as well as the scale and depth of potential slip surfaces/instability that are likely to develop in the long-term.

The seismic setting of the Voorspoed area was reviewed, and appropriate seismic design criteria derived for use in stability assessments of the Voorspoed pit slopes, for post-closure design.

Appropriate geotechnical risk, design acceptance and consequence criteria were adopted for post-closure design of the Voorspoed open pit, with due consideration given to existing pit wall conditions in the pit, an evaluation of design confidence levels for post-closure design, and the potential consequences that may result from a pit slope failure at Voorspoed.

The current security fence was constructed around the Voorspoed pit in 2018, and located at a distance from the edge of the pit which was based on early assessments of possible long-term break-back and erosion that will lead to quasi-stable vegetated slopes. That said, considering historical break-back rates in the Voorspoed pit and precedent experience, the security fence is currently located at an appropriate distance from the edge of the pit to ensure adequate risk mitigation and limitation of public access in the short to medium term. That said, the Baseline analyses completed for this geotechnical closure study, show that break-back to the TSR limits adopted in this study, in several places around the Voorspoed pit, will extend to beyond the current security fence line. It follows that the security fence will require extension in areas where the predicted TSR break-back limits extend beyond the current security fence line.

On this basis it is concluded that either of Options 1 or 2 in the geochemical report (Golder, 2020) can be adopted for post-closure design, with a slightly lower level of risk (in terms of pit slope instability) attached to Option 3 – provided that the following is diligently implemented and maintained:

- Firstly, it must be noted that the current open pit area including most of the pit crest areas, is not safe given ongoing sloughing and break-back of the slopes, which is exacerbated by ongoing erosion given that no vegetation has established yet on the current slopes. An exclusion zone therefore must be maintained, using a well-maintained security fence, signposted as required, to control access into this high-risk area.
- Furthermore, the current security fence should be extended in several areas as outlined in this Report to ensure long-term exclusion and mitigation as required.
- Stormwater runoff around the Voorspoed pit should be adequately managed to prevent excessive erosion of the Voorspoed pit slopes, with appropriate engineering controls implemented as per J&W recommendations.
- The security fence and all stormwater management systems and controls implemented, should be well-maintained to control access and exposure to unstable ground, high-risk pit areas, and the pit lake.
- Regular inspections of the pit area, of the security fence, and stormwater management systems and controls, should be carried out at set intervals, to identify repair and maintenance requirements as appropriate.
- Finally, should a major subsidence/instability develop, additional mitigation options will need to be reviewed, with appropriate responses as necessary to address associated hazards and risks.

The stormwater controls identified by J&W (2022) are considerate adequate in terms of South African legislative requirements and Anglo-American standards for long-term pit closure purposes. That said, there may be an opportunity to further enhance the proposed FRD canal by discharging over more competent geology and/or ensuring that the drop structure is robust to mitigate erosion and undercutting which could accelerate crest backbreak and erosion.

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APPENDICES

APPENDIX A: STATEMENTS OF CAPABILITY

Statement of Capability – Gerhardus J Keyter Pr Eng | AREC

B Eng (Civil), MSc DIC | FSAICE, MSAIMM, MSANIRE, MISRM

Principal Geotechnical Engineer

I, Gerhardus Johannes Keyter, Pr. Eng. (Reg. No. 20100120), am a Professional Geotechnical Engineer and Principal with SRK Consulting (South Africa) (Pty) Ltd ("SRK"), working from SRK House, 265 Oxford Road, Illovo 2196, in the City of Johannesburg, Gauteng Province, Republic of South Africa.

I graduated from the University of Pretoria in South Africa with a Bachelor of Engineering Degree in Civil Engineering (cum laude) in 1992, and with a Master of Science Degree in Soil Mechanics in 1994 from the Imperial College of Science, Technology and Medicine in London, England. I have also obtained South African Chamber of Mines' Certificates in Rock Mechanics ("COMREC") in Surface Mining in 2018, and in Advanced Rock Engineering ("AREC") in 2019.

I am a Fellow of the South African Institution for Civil Engineering ("SAICE") and a Member of the Southern African Institute for Mining and Metallurgy ("SAIMM"), the South African National Institute of Rock Engineering ("SANIRE") and the International Society of Rock Mechanics ("ISRM"). I am also affiliated with the International Society for Soil Mechanics and Geotechnical Engineering ("ISSMGE") and served as the Chairperson of the South African National Committee on Tunnelling ("SANCOT") from October 2021 until September 2023.

I have practiced my profession continuously since 1992, and was engaged in the following during this time:

- Geotechnical design, engineering optimisation, and review work for several major civil engineering infrastructure projects and for numerous mining operations in South Africa, Botswana, Burkina Faso, the Democratic Republic of Congo, Eritrea, Ghana, Guinea, Lesotho, Malawi, Mali, Mozambique, Namibia, Tanzania, Zambia, and Zimbabwe, as well as for projects located in Australia, Brazil, Chile, China, India, Panama, and Russia.
- Managed the Geotechnical Department at De Beers Venetia Mine in South Africa from 2003 to 2004.
- Worked as Tunnel and Cavern Designer on the design of the Main Underground Works of the Ingula Pumped Storage Scheme in South Africa, which included the design of the excavation and support of the main power caverns with spans of up to 26m in carbonaceous shales.
- Managing Partner and Principal with GeoStable SA cc, from October 2009 up to May 2016.
- Chief Design Engineer of the Polihali Diversion Tunnels (LHDA Contract No. 3022) from September 2016 up to February 2019.
- Senior Geotechnical/Mining Engineer (October 2017 - February 2019) and Acting Chief Design Engineer (October 2018 - February 2019) of the Polihali Transfer Tunnel (LHDA Contract No. 3007).
- Compiled mandatory Codes of Practice to mitigate geotechnical risks and related accidents for several open pit mines, quarries, and underground massive mining operations in South Africa.
- Carried out numerous detailed risk assessments of geotechnical hazards and geotechnical audits/inspections of site operations at several major civil engineering infrastructure projects, and for various open pit mines, quarries, and underground massive mining operations in Southern Africa.
- Carried out both 2D and 3D numerical modelling work, and reviewed numerical models used in the design of several major civil engineering infrastructure and numerous mining projects in South Africa and elsewhere in the world.
- Carried out geotechnical engineering investigations, design and review work for slimes dams, waste rock dumps, and grouted cut-off curtains in dam embankments, as well as dam safety inspections.

I was furthermore awarded the SAICE Geotechnical Division's Distinguished Service Award in November 2023, for a substantial service contribution made to the geotechnical engineering industry in South Africa over a period of almost 30 years.

I am a Competent Person to do geotechnical engineering design and review work because of my experience and qualifications, and in terms of the Engineering Profession Act (Act 46 of 2000) of South Africa. I furthermore fully subscribe to the Engineering Council of South Africa's ("ECSA") Code of Conduct for Registered Persons as published in Board Notice 41 of 2017 in Government Gazette No. 40691.

I have worked as a Principal Geotechnical Engineer for SRK since March 2019.

Signed in **RANDBURG** on **03 JANUARY 2024**:

SRK Consulting - Certified Electronic Signature
srk consulting
900003/51/45292/Other
5114-7290-3843-KEYT-03/01/2024
This signature has been printed digitally. The Author has given permission for its use for this document. The details are stored in the SRK Signature Database

Gerhardus J. Keyter Pr Eng | AREC
B Eng (Civil), MSc DIC | FSAICE, MSAIMM, MSANIRE, MISRM
Principal Geotechnical Engineer, SRK Consulting

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e-mail: gkeyter@srk.co.za

Statement of Capability – Luis Fernando Contreras Pr Eng | SAIMM

Civil Engineer, MSc Engineering Geology | PhD Geotechnical Engineering,
FIEAust, CPEng (Civil and Geotechnical Eng.), NER, RPEQ
Associate Principal Geotechnical Engineer

I, Luis Fernando Contreras, Pr. Eng. (Reg. No. 20120311), am a Professional Geotechnical Engineer and Associate Principal with SRK Consulting (South Africa) (Pty) Ltd ("SRK"), working from 13/21, Fenton Street, Fairfield, QLD4103, in the City of Brisbane, Queensland, Australia.

I graduated from the Universidad Nacional de Colombia in Bogota with a Civil Engineering Degree in 1979, with a Master of Science Degree in Engineering Geology in 1992 from the International Institute for Aerospace Survey and Earth Sciences (ITC) in Delft, The Netherlands and with a PhD in Geotechnical Engineering in 2020 from the University of Queensland in Brisbane, Australia.

I am a Fellow of the Institution of Engineers Australia (FIEAust), a Chartered Engineer of the same institution in the fields of Civil and Geotechnical Engineering (CPEng), registered in the National Engineering Register of Australia (NER), a registered Professional Engineer of Queensland (RPEQ), a registered Professional Engineer in South Africa (Pr Eng) and a Member of the Southern African Institute for Mining and Metallurgy (SAIMM).

I have received the following awards for publications in the field of geotechnical engineering: ARMA Best Research Paper Award, US Rock Mechanics Symposium, San Francisco, 2008. SAIMM Gold Medal, Best Paper SAIMM Journal, 2015. UQ Dean's Award for Outstanding Higher Degree by Research Theses 2020. AusIMM Best Paper Award, Mine Waste and Tailings Conference 2023.

I have practiced my profession continuously since 1979, and was engaged in the following during this time:

- Geotechnical design, engineering optimisation, and review work for several major civil engineering infrastructure projects in South America, Central America and the Caribbean as a Senior Geotechnical Engineer at Integral Consulting Engineers in Medellin, Colombia. Main projects included large hydroelectric power plants with high earth and rockfill dams and deep underground cavern systems.
- Project management and geotechnical design of tunnel projects for the infrastructure of mine operations as a Principal Geotechnical Engineer at SRK Consulting Chile in Santiago, from March 2000 to December 2003.
- Partner, Director and Manager of the Geotechnical Department at SRK Consulting Chile in Santiago from January 2004 to June 2007.
- Carried out numerous quantitative risk evaluation studies for the expansion of mining projects and open pit geotechnical design.
- Project management, geotechnical design and risk evaluation studies for open pit mines in Africa and South America as a Principal Geotechnical Engineer at SRK Consulting South Africa in Johannesburg, from July 2007 to March 2010.
- Project management, geotechnical design, risk evaluation studies and review work for open pit mines in Africa and South America as a Corporate Consultant in Geotechnical Engineering at SRK Consulting South Africa in Johannesburg, from April 2010 to September 2015.
- Worked in diverse geotechnical studies including site investigations, rock-fill and earth-fill dam design, instrumentation design and rock mechanics analysis of geotechnical structures. Had experience covering different stages of project development from conceptual and feasibility studies to detail design, preparation of tender documents and specialised advisory during construction.
- Completed a research study on the use of Bayesian methods to treat uncertainty in slope design as part of a PhD programme and has implemented this methodology for the strength characterisation of rock wastes and tailings.

I am a Competent Person to do geotechnical engineering design and review work because of my experience and qualifications, and in terms of the Engineering Profession Act (Act 46 of 2000) of South Africa. I furthermore fully subscribe to the Engineering Council of South Africa's ("ECSA") Code of Conduct for Registered Persons as published in Board Notice 41 of 2017 in Government Gazette No. 40691.

I have worked as an Associate Principal Geotechnical Engineer for SRK since March 2022.

Signed in **BRISBANE** on **26 JANUARY 2024**:

Luis Fernando Contreras Pr Eng | SAIMM
Civil Eng., MSc, PhD | FIEAust, CPEng, NER, RPEQ
Associate Principal Geotechnical Engineer, SRK Consulting

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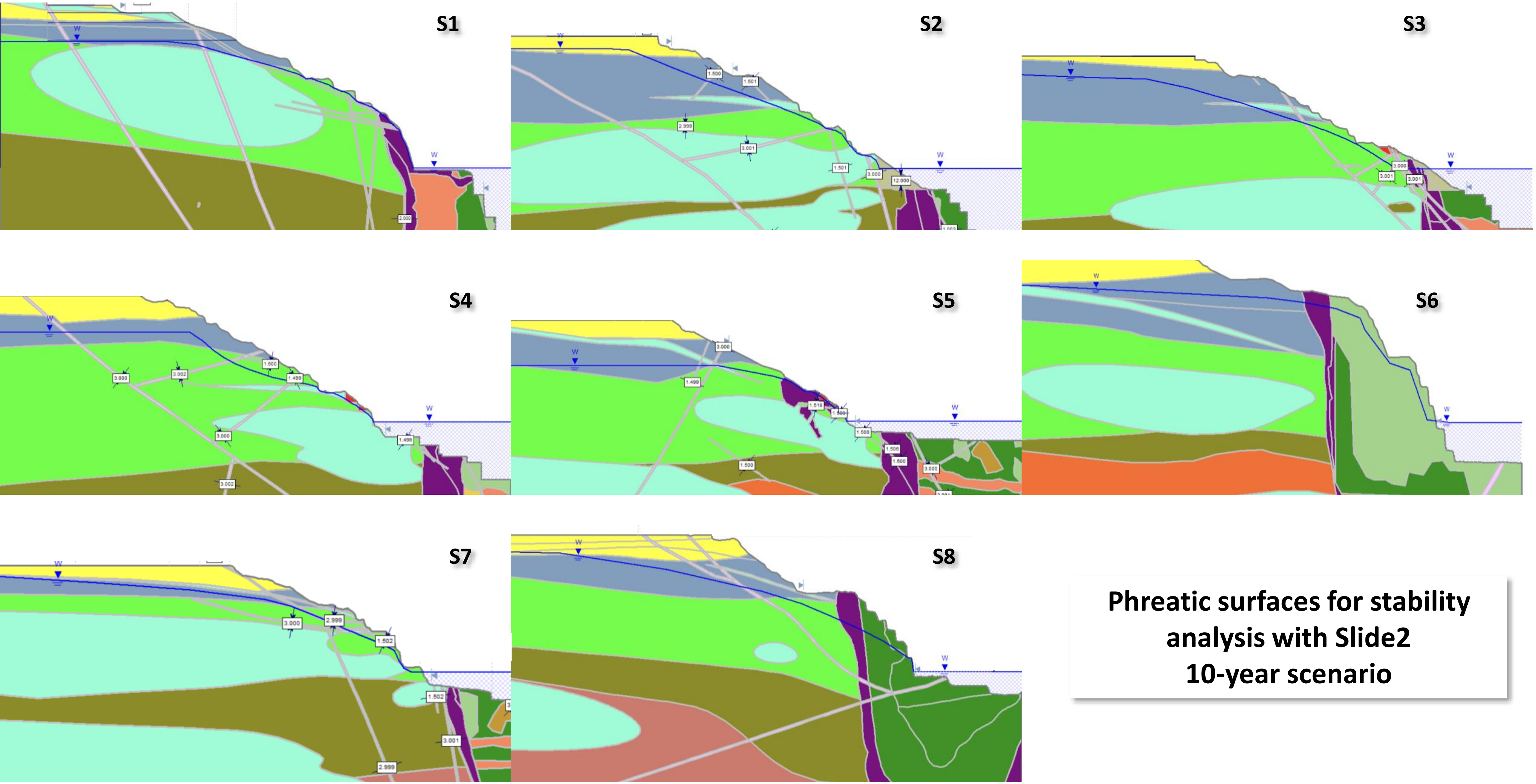
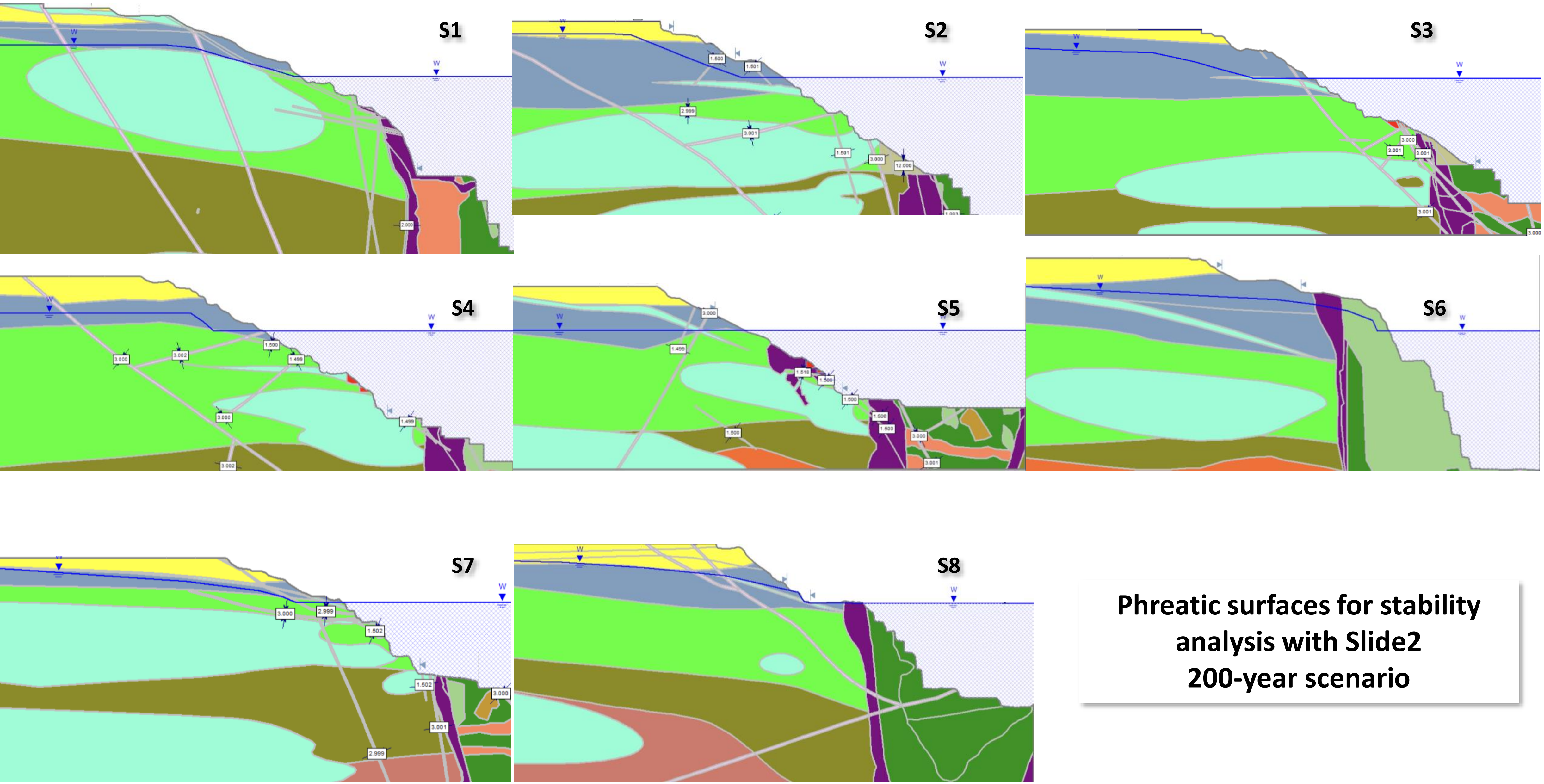
e-mail: lfcontreras@srk.com.au

APPENDIX B: PHREATIC SURFACES USED FOR PIT CLOSURE DESIGN



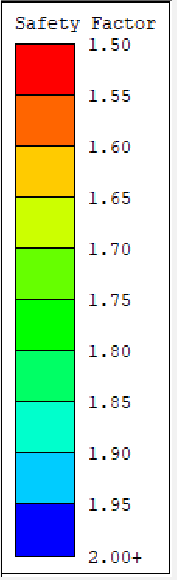
Phreatic surfaces for stability analysis with Slide2
10-year scenario



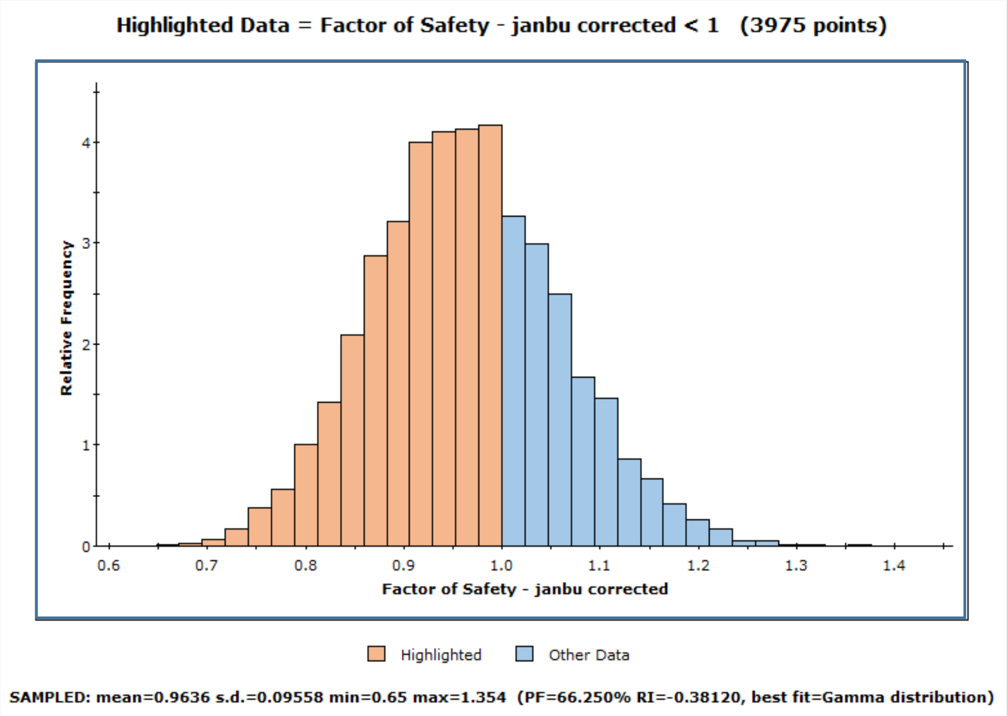
Phreatic surfaces for stability analysis with Slide2
200-year scenario

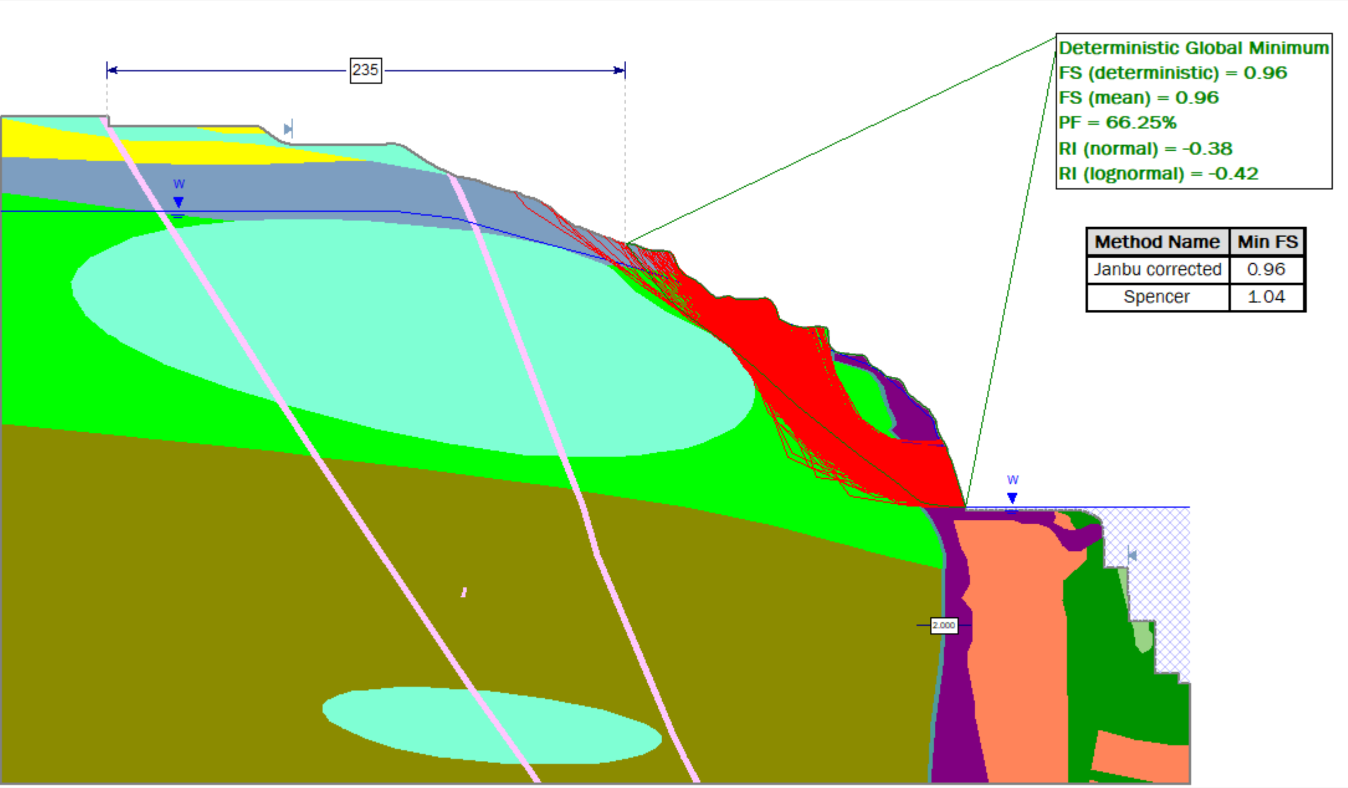
APPENDIX C: BASELINE – SLOPE STABILITY ANALYSIS RESULTS

Section S1_SW23

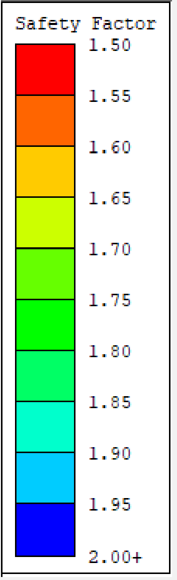


Section S1_SW23 – 10 years – break-back: -235m

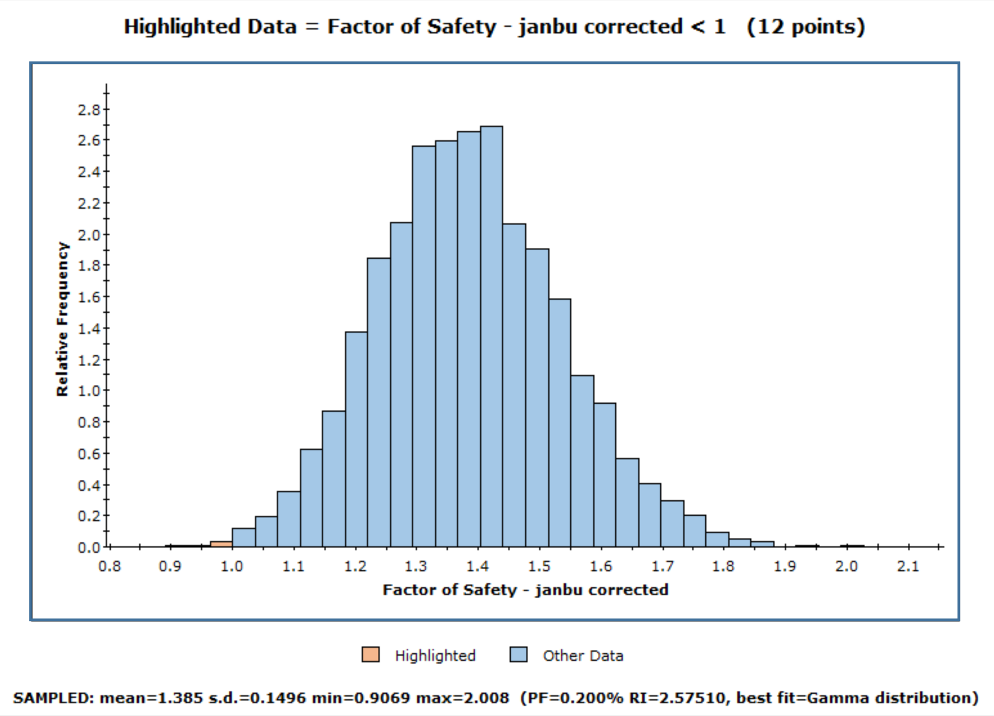


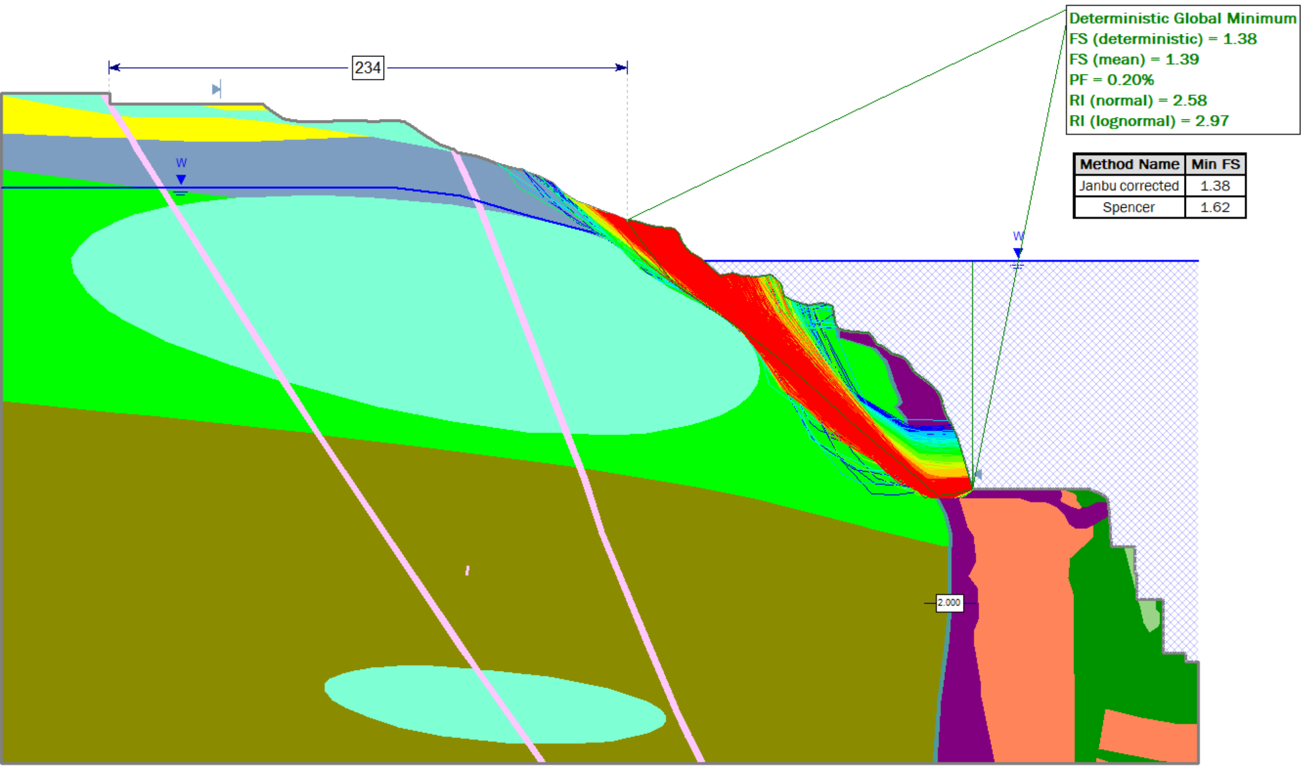


Probability FS ≤ 1.0 = 66.3%



Section S1_SW23 – 200 years – break-back: -234m

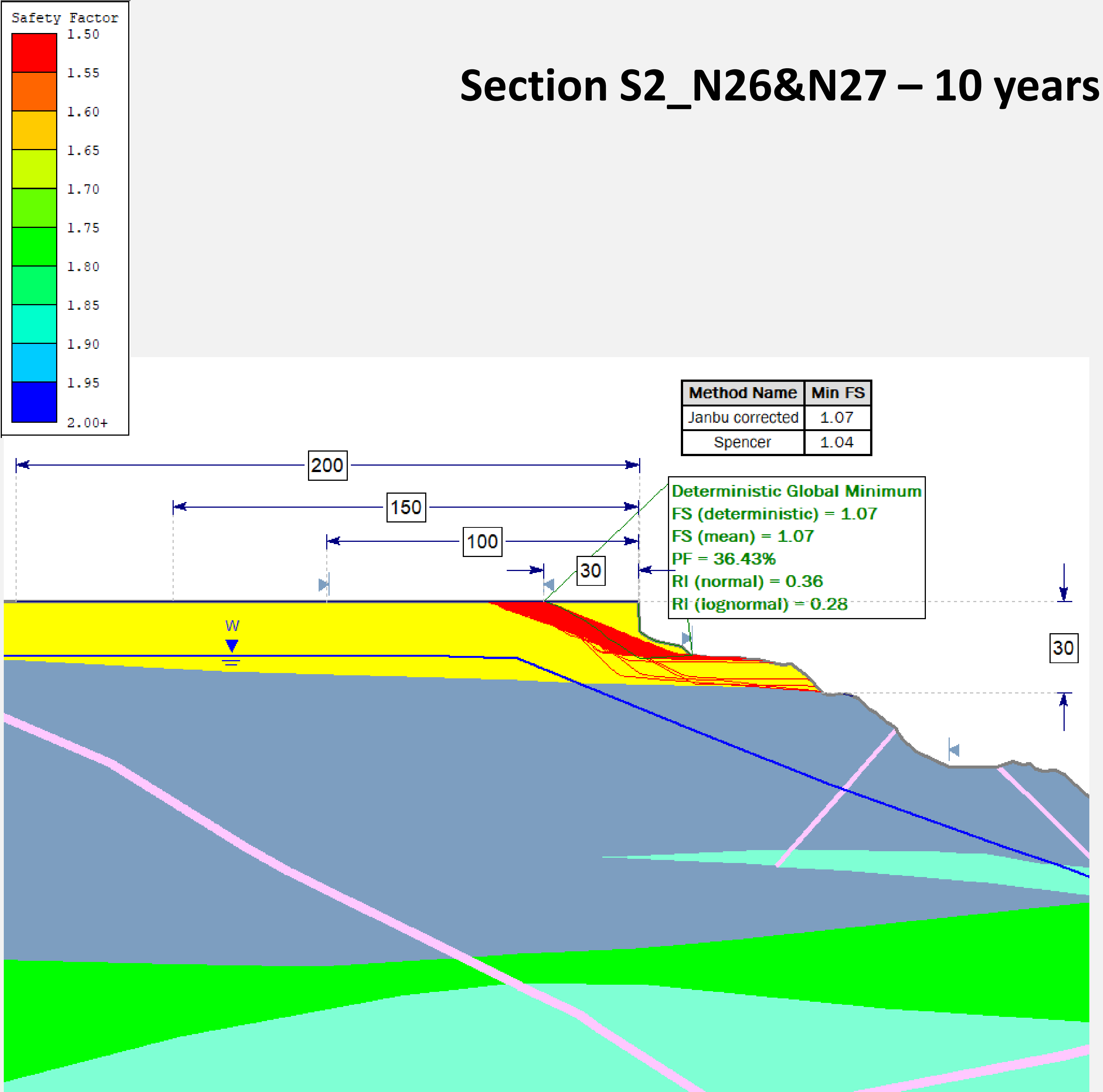


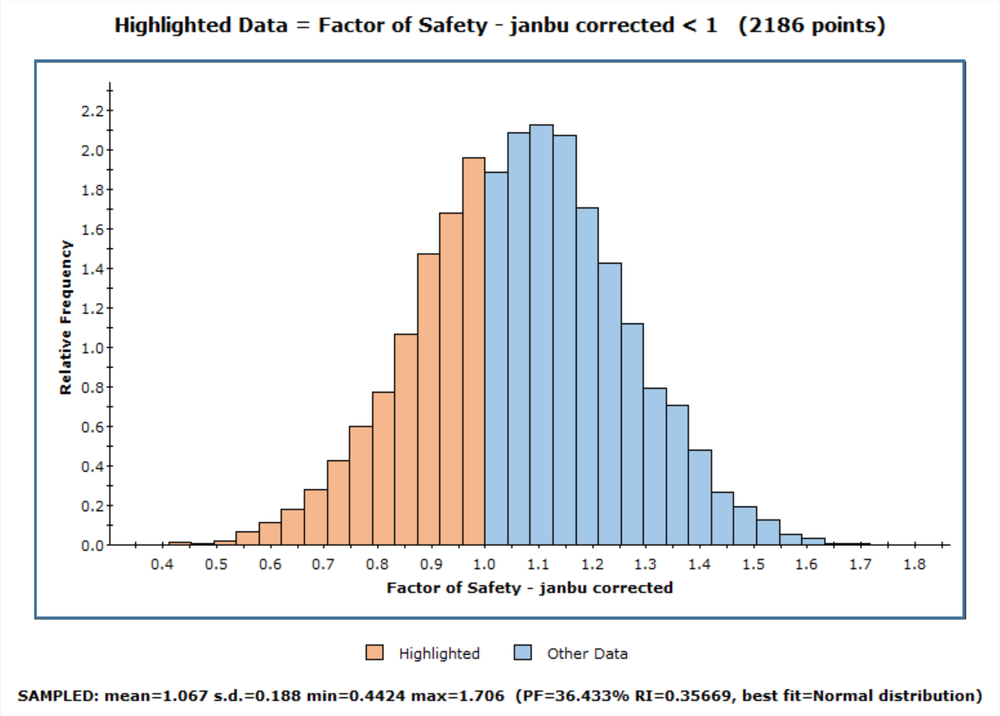


Probability FS ≤ 1.0 = 0.2%

Section S2_N26&N27

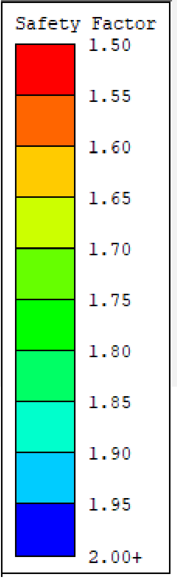
Section S2_N26&N27 – 10 years – break-back: 30m

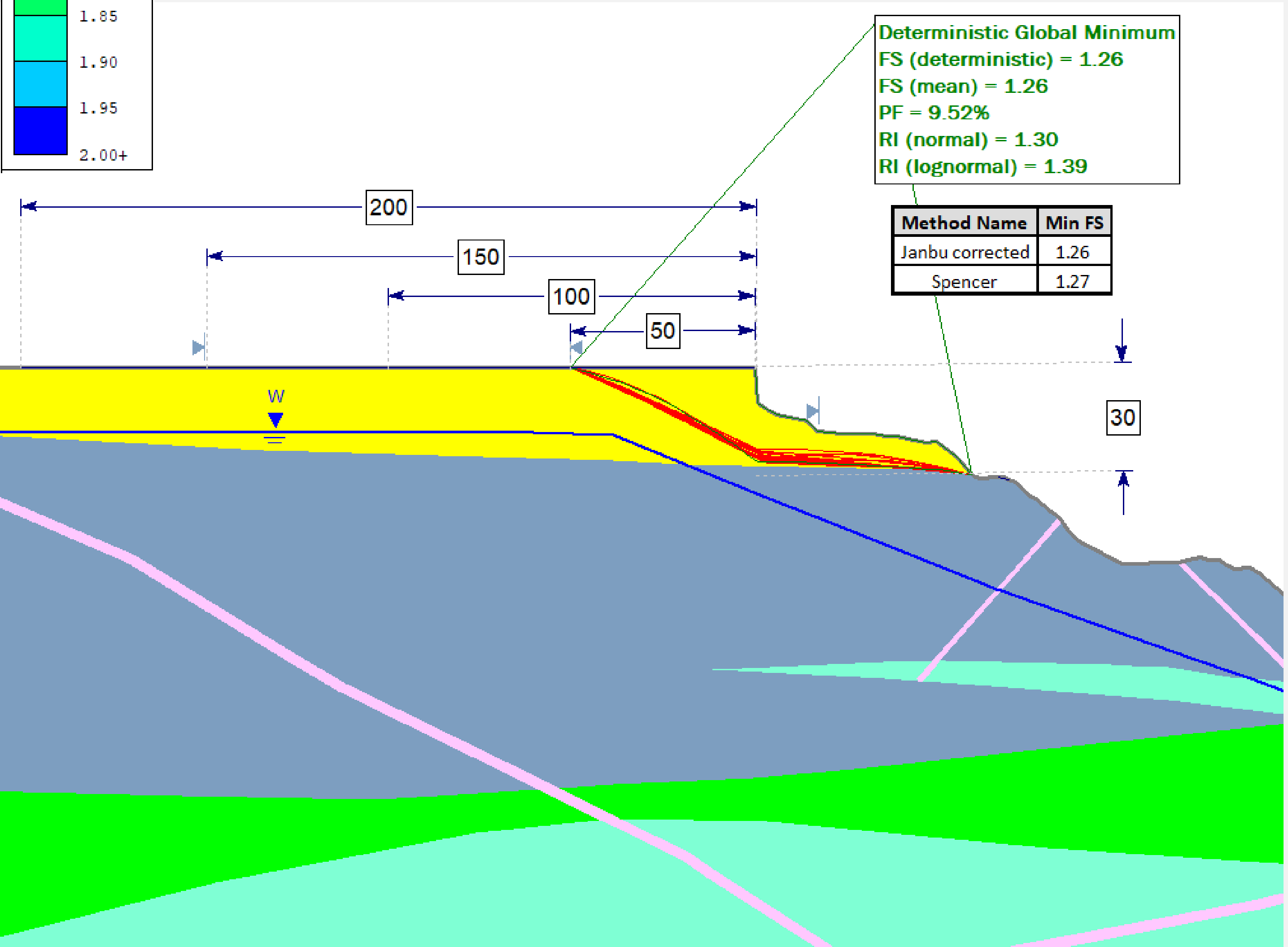


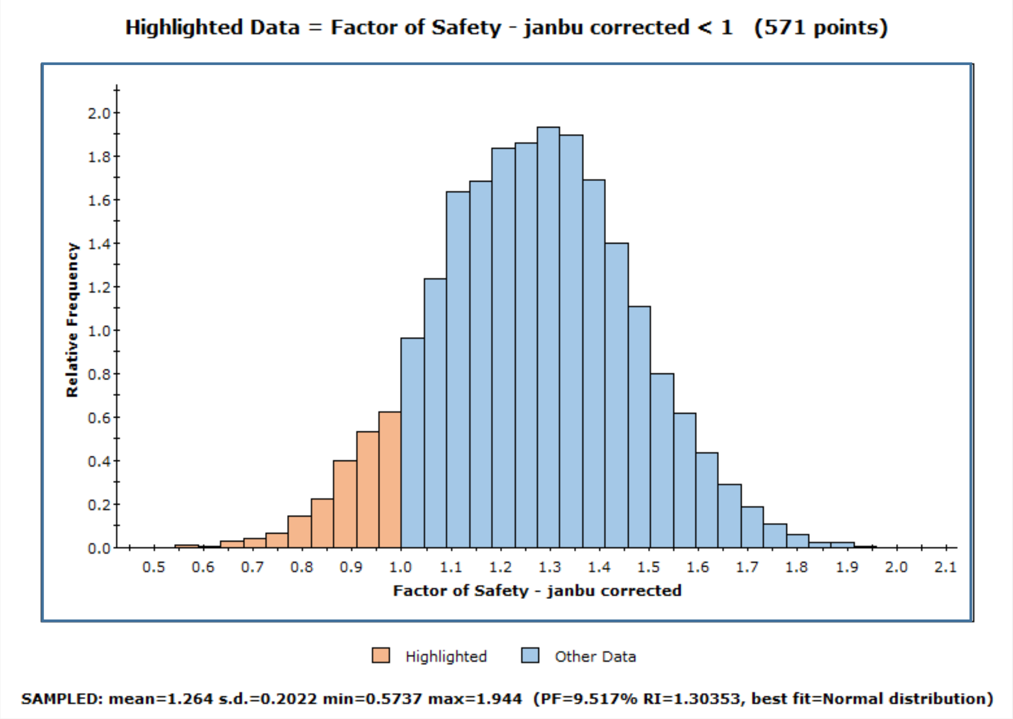


Probability FS ≤ 1.0 = 36.4%

Section S2_N26&N27 – 10 years – break-back: 50m

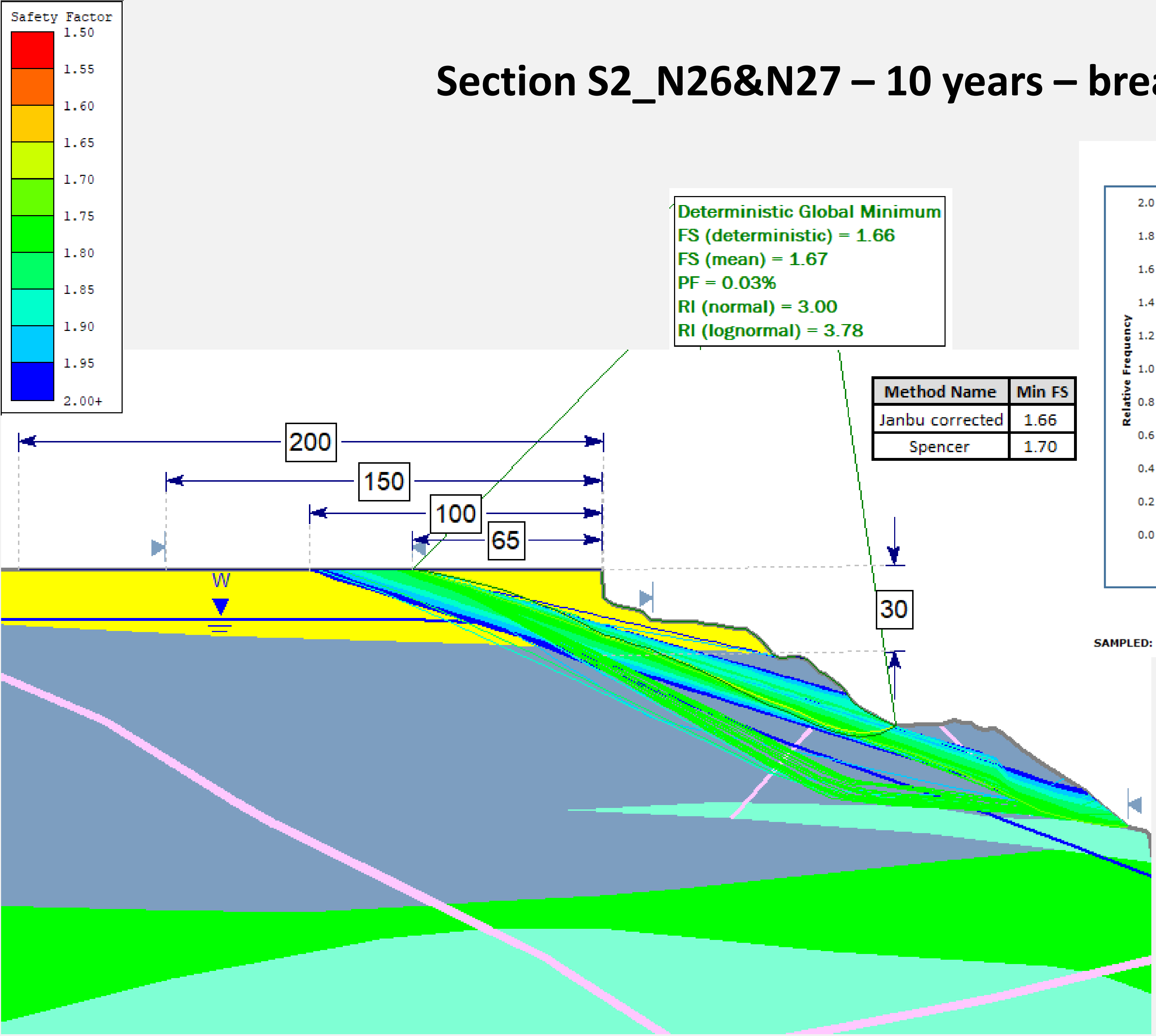


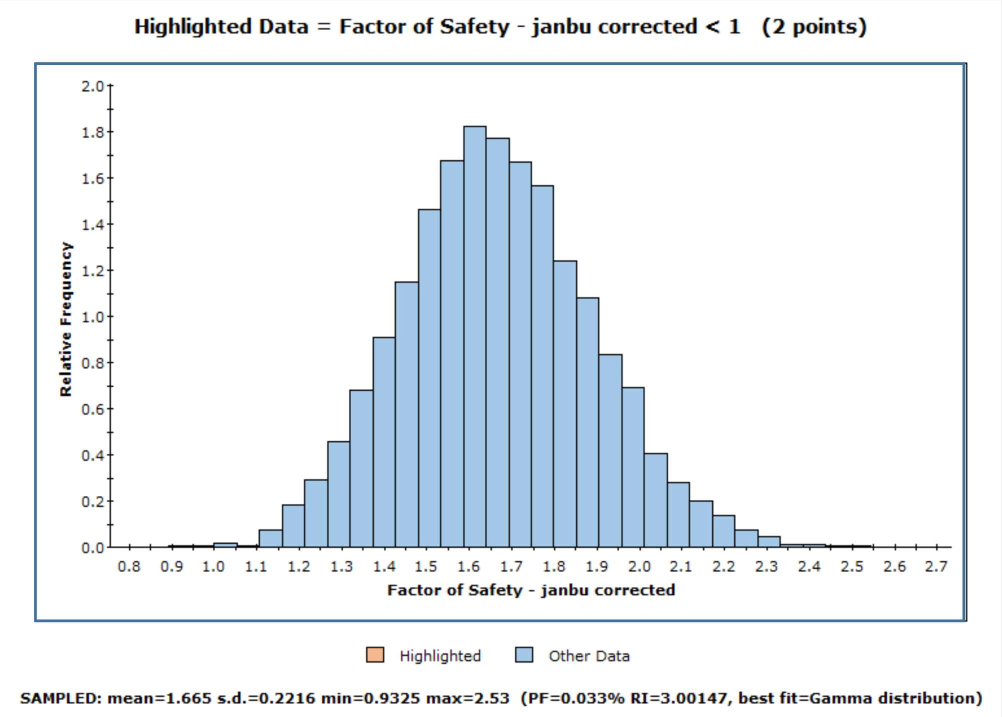




Probability FS ≤ 1.0 = 9.5%

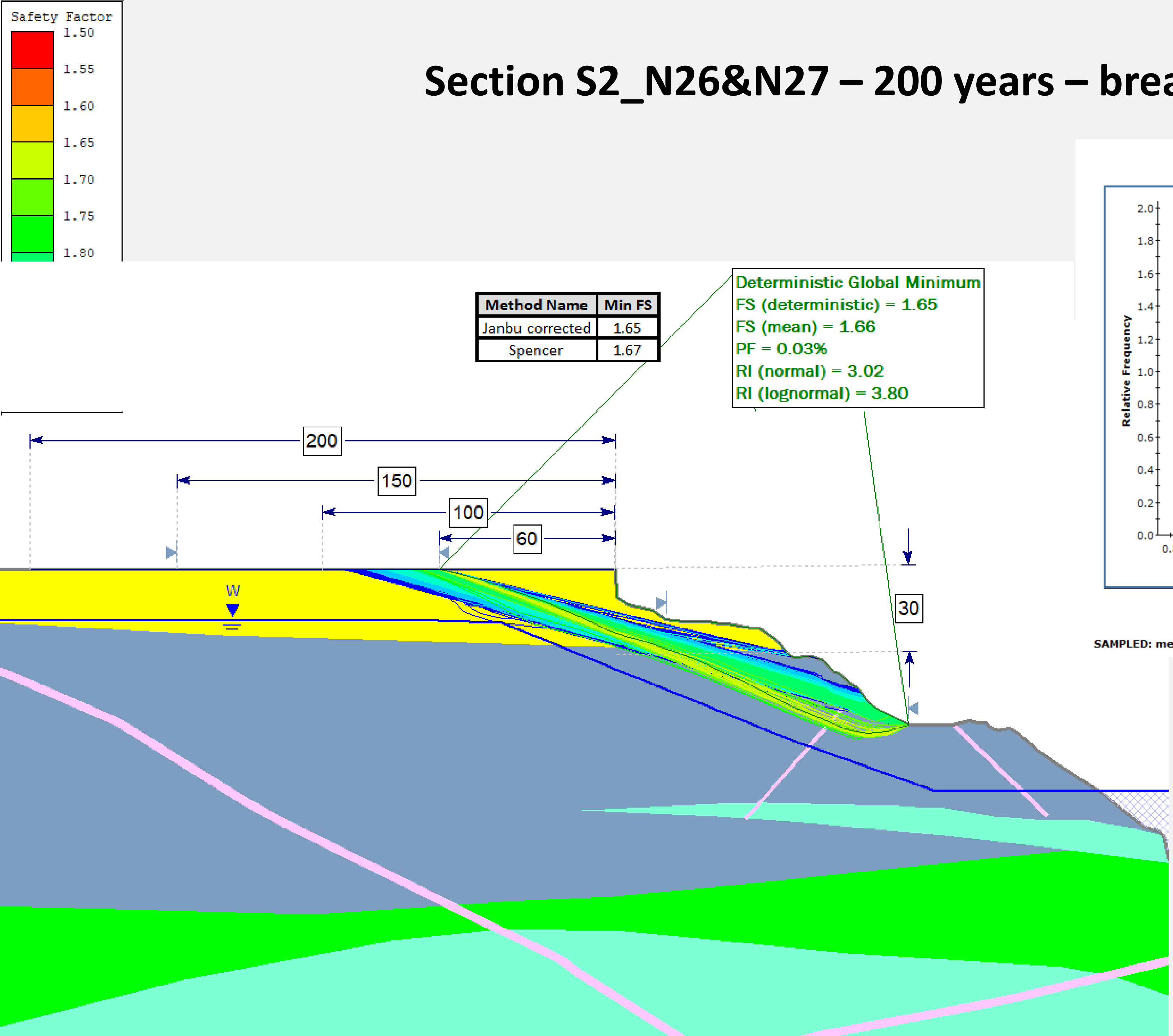
Section S2_N26&N27 – 10 years – break-back: 65m

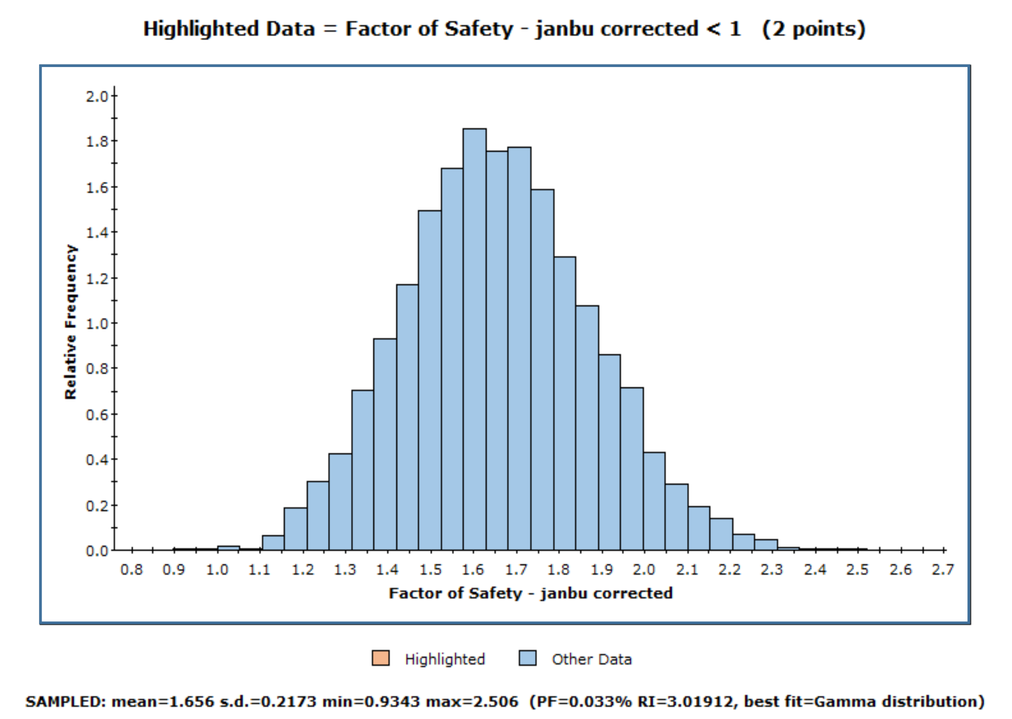




Probability FS ≤ 1.0 = 0.0%

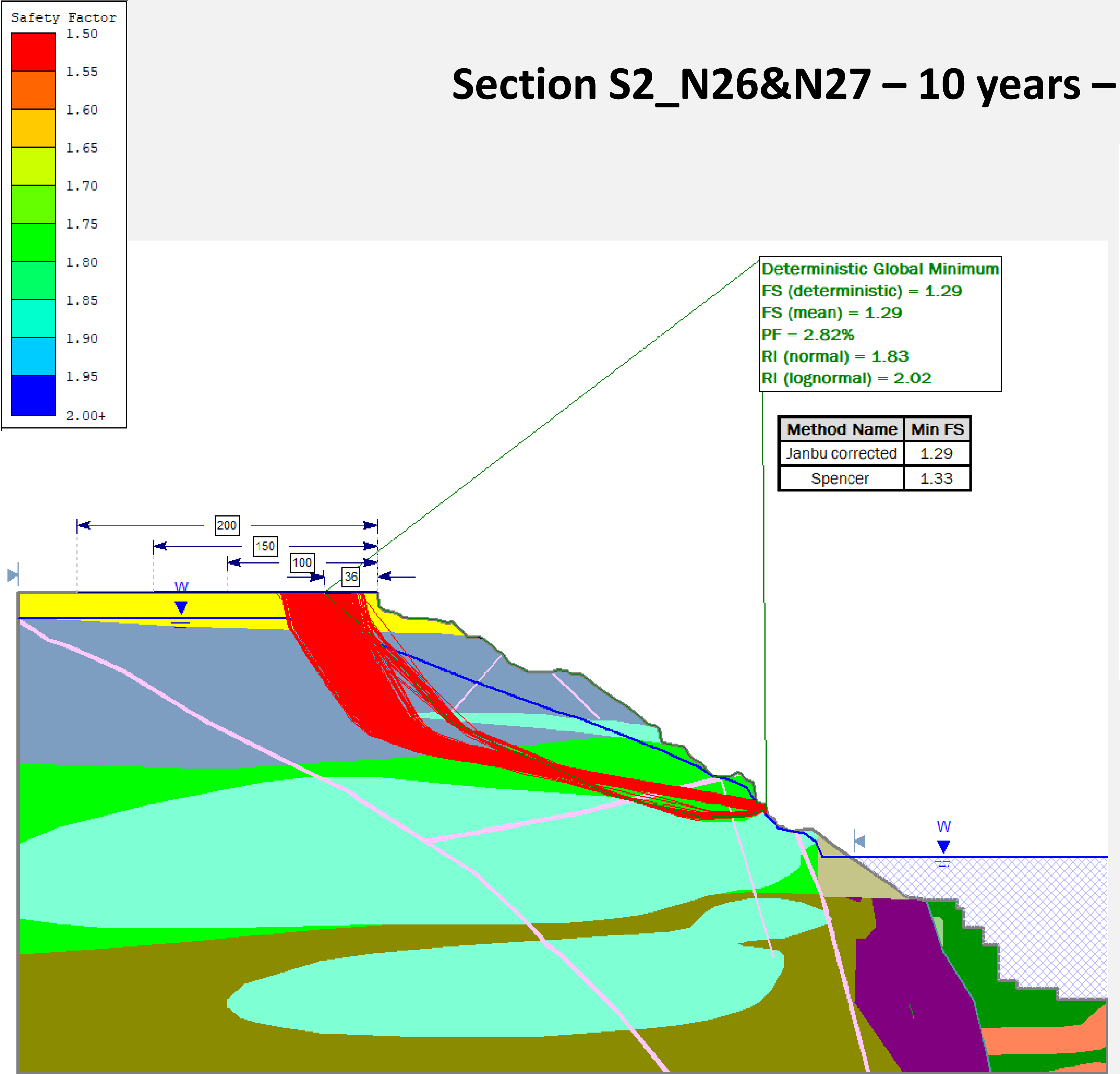
Section S2_N26&N27 – 200 years – break-back: 60m

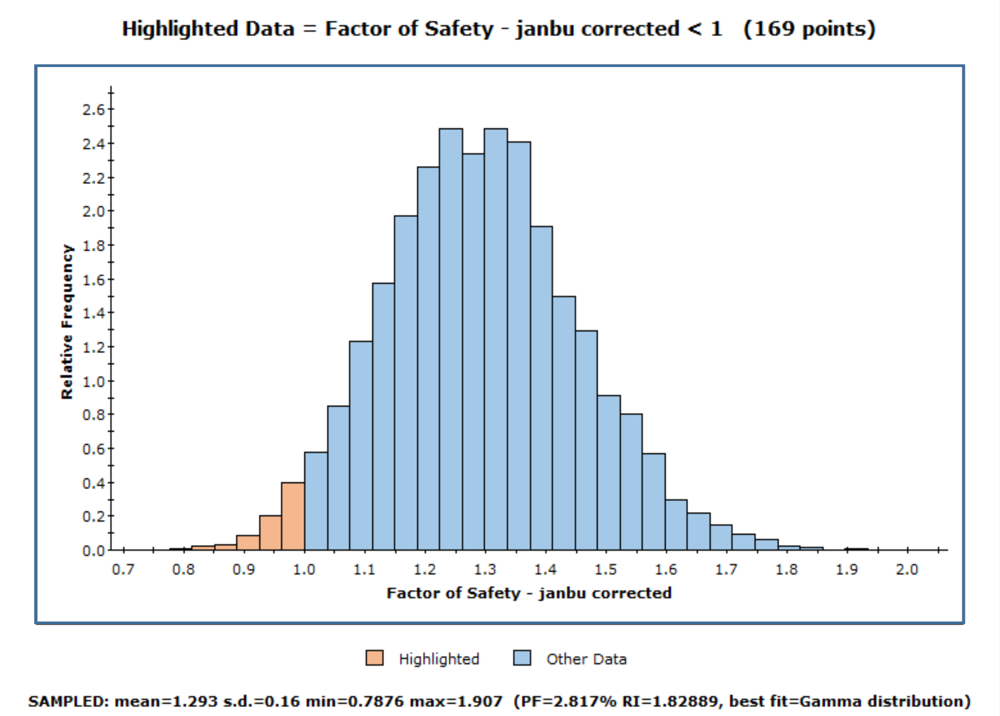




Probability FS ≤ 1.0 = 0.0%

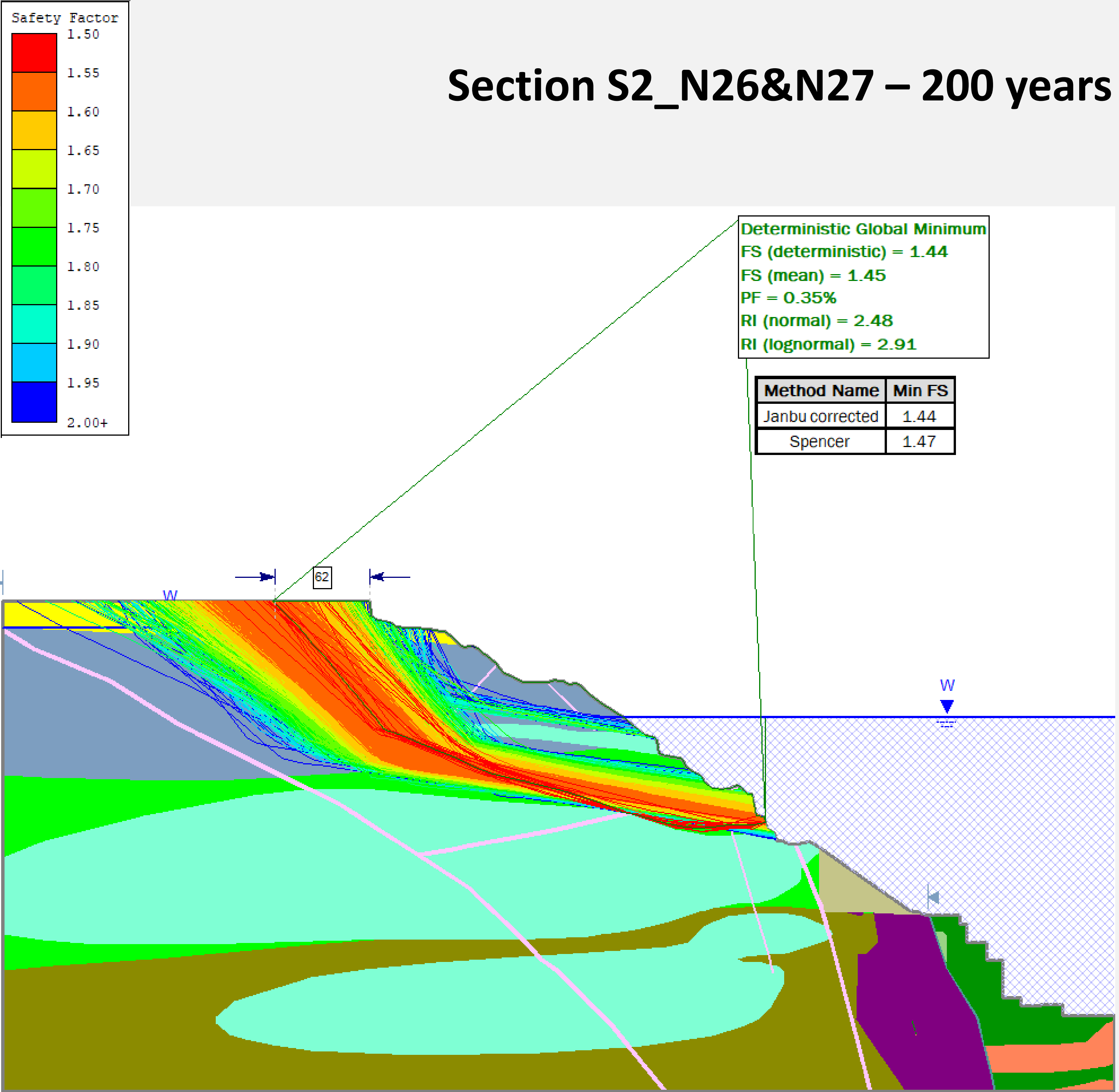
Section S2_N26&N27 – 10 years – break-back: 36m

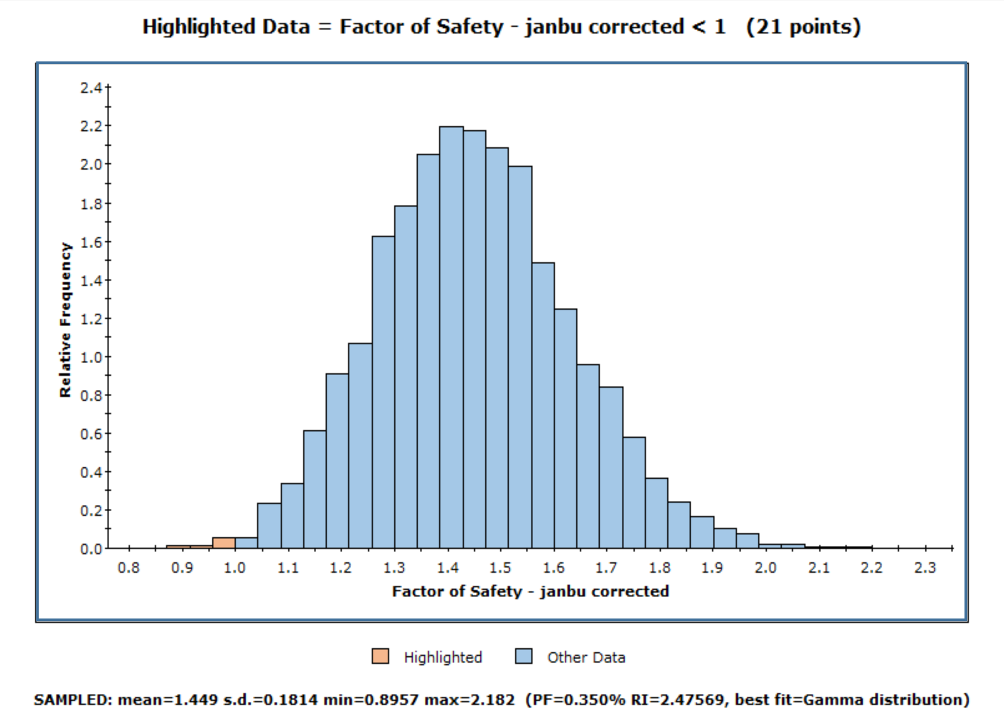




Probability FS ≤ 1.0 = 2.8%

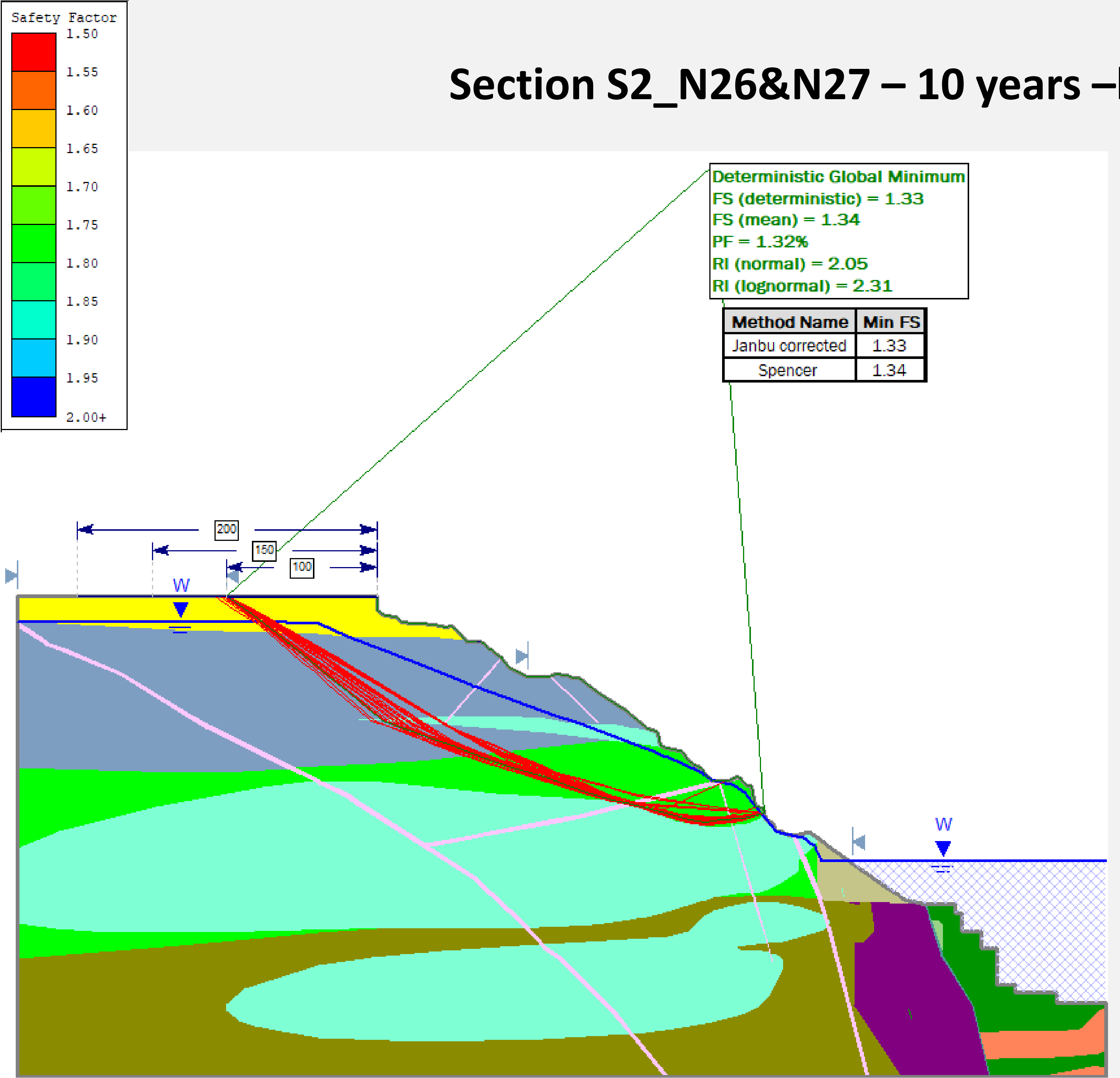
Section S2_N26&N27 – 200 years – break-back: 62m

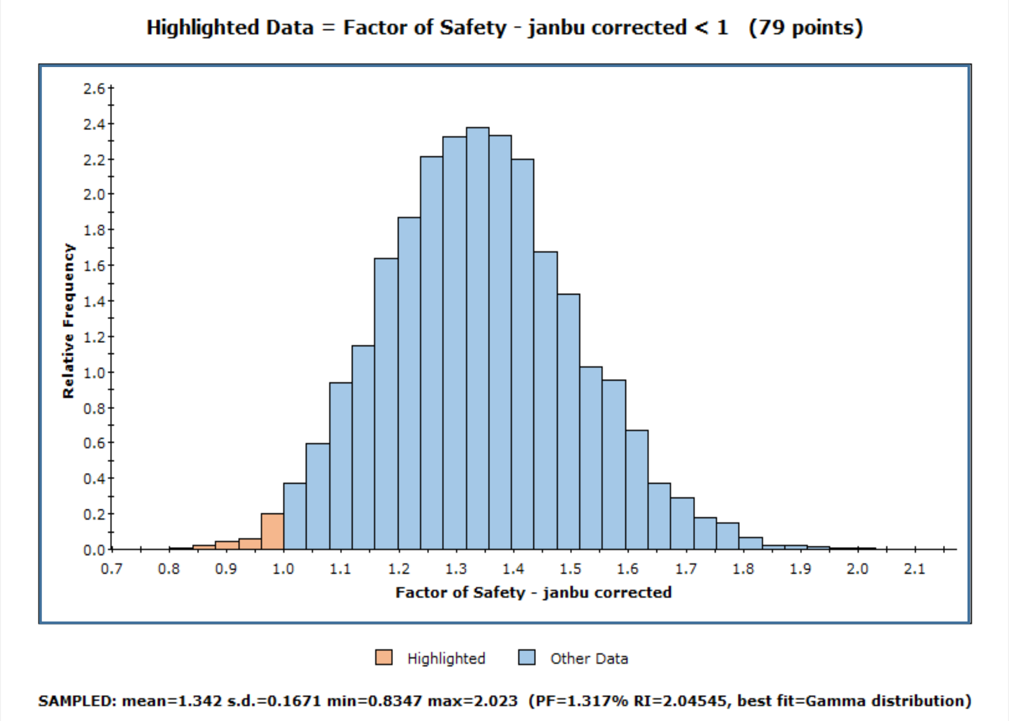




Probability FS ≤ 1.0 = 0.4%

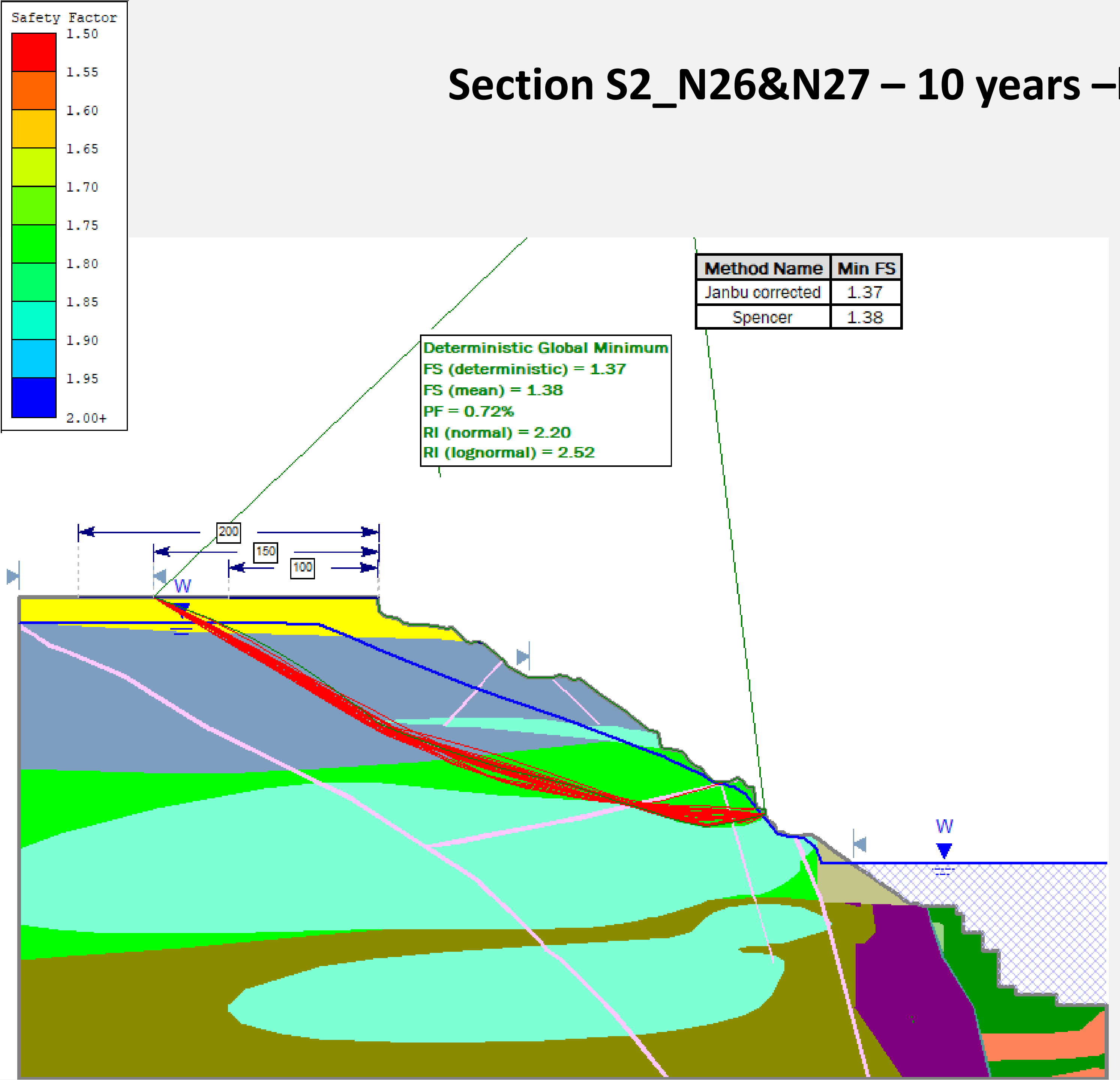
Section S2_N26&N27 – 10 years –break back: 100m

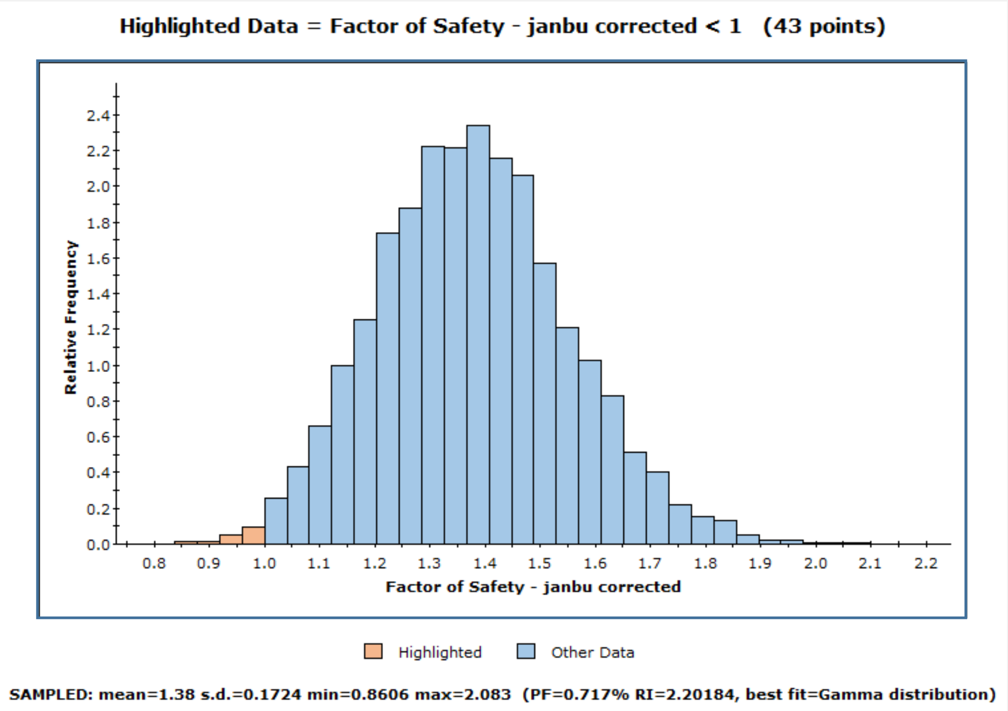




Probability FS ≤ 1.0 = 1.3%

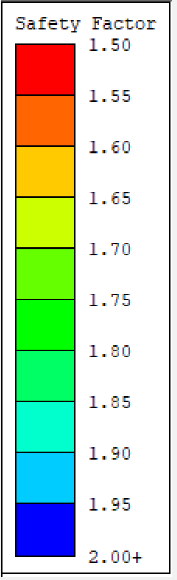
Section S2_N26&N27 – 10 years –break-back: 150m



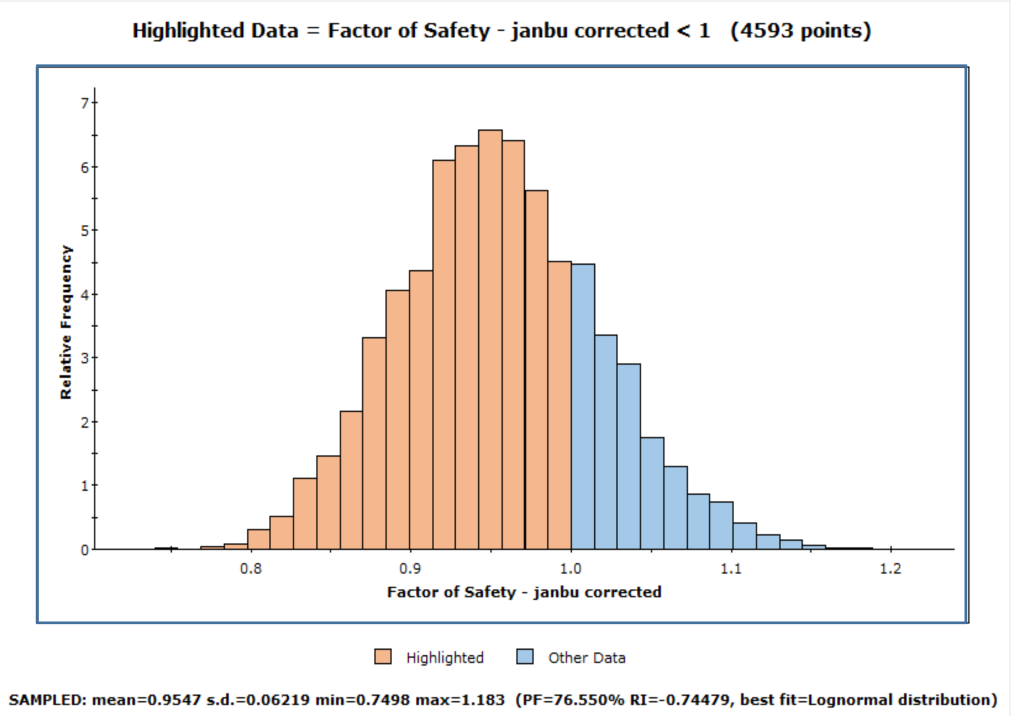


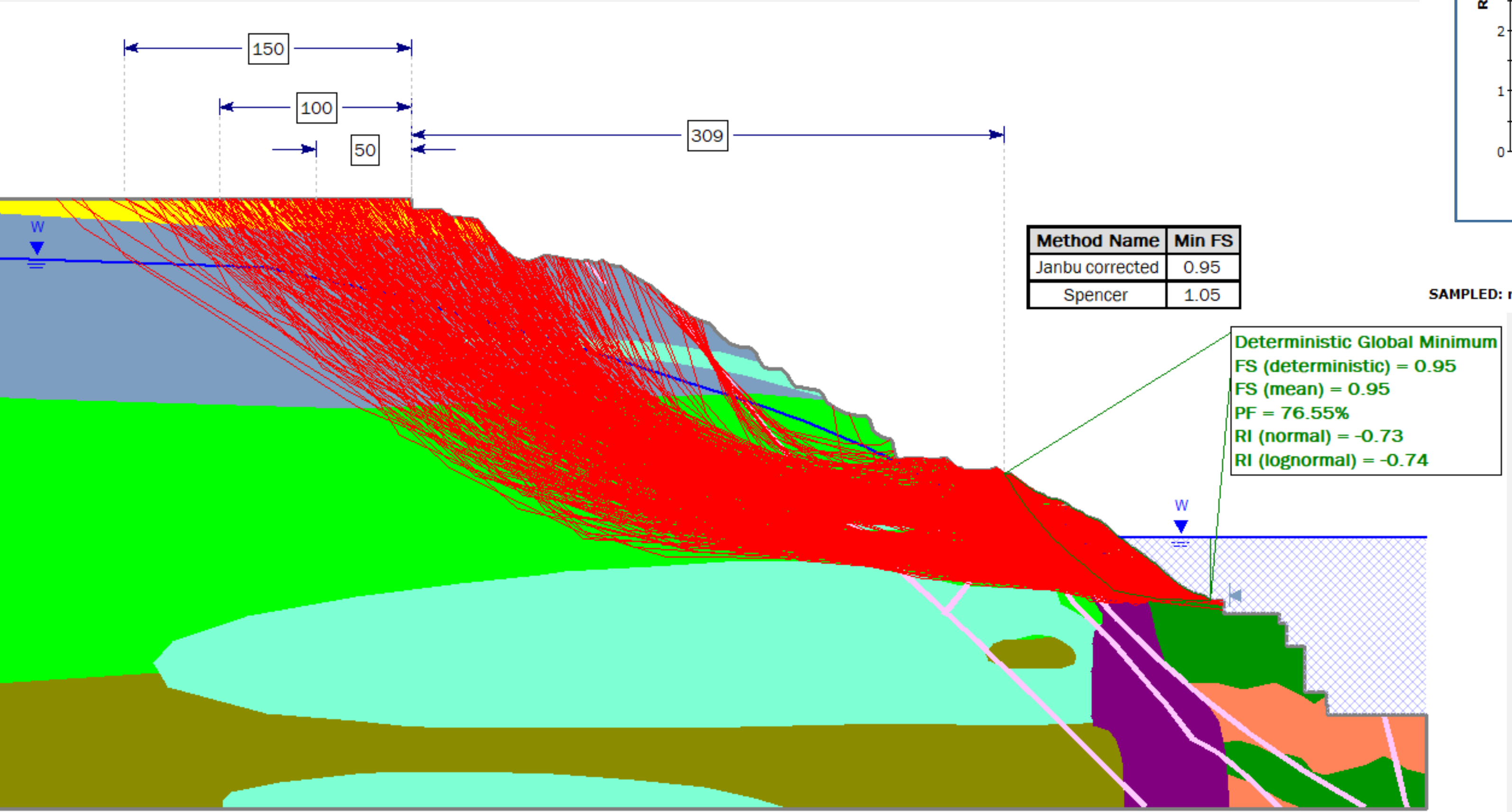
Probability FS ≤ 1.0 = 0.7%

Section S3_N28



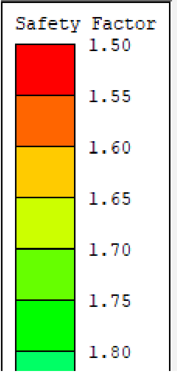
Section S3_N28 – 10 years –break-back: -309m

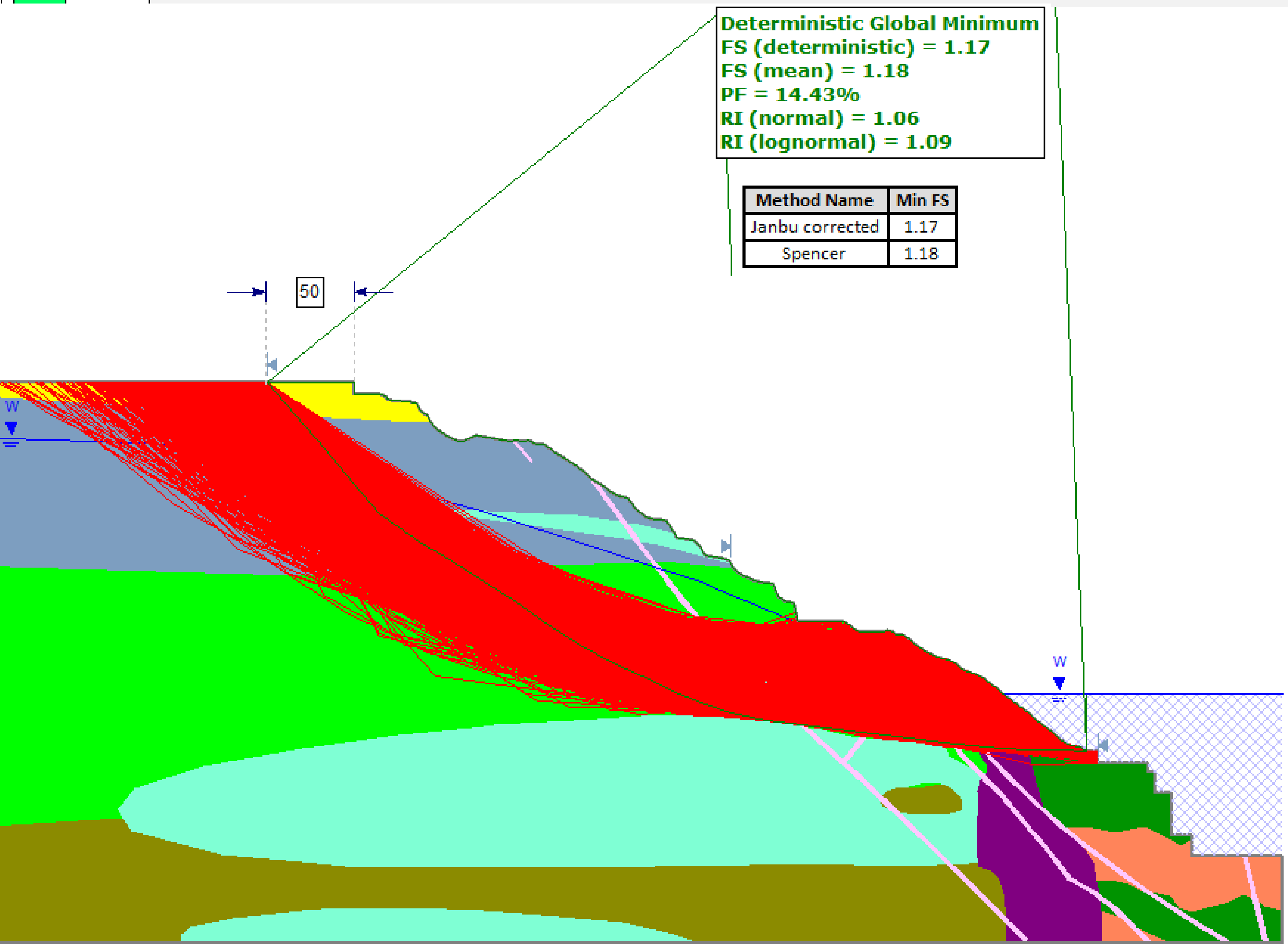


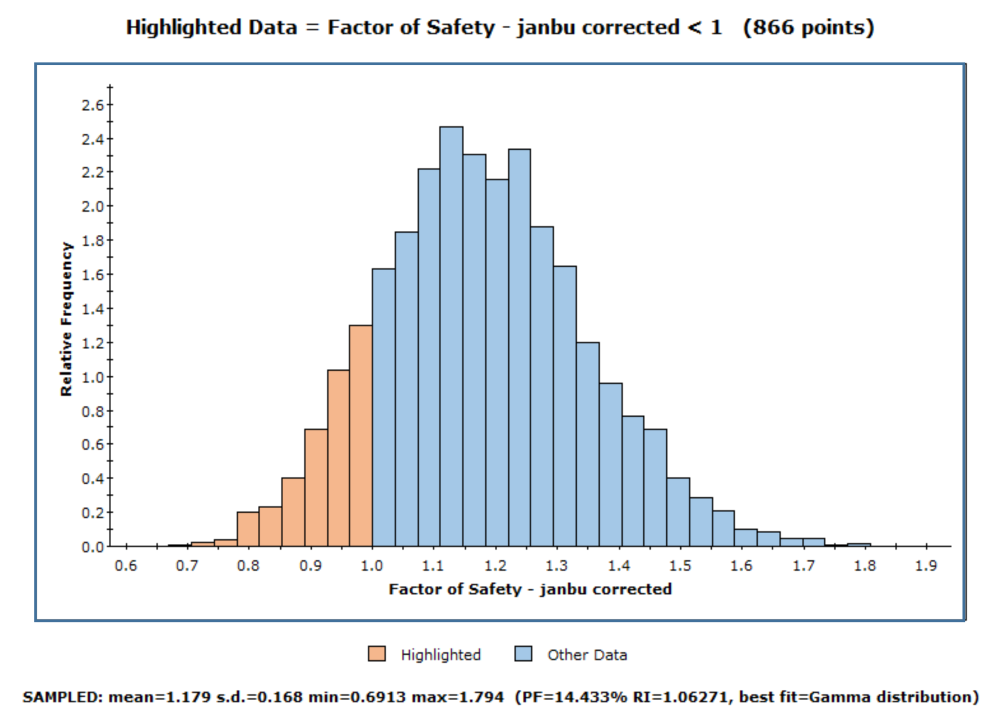


Probability FS ≤ 1.0 = 76.6%

Section S3_N28 – 10 years –break-back: 50m

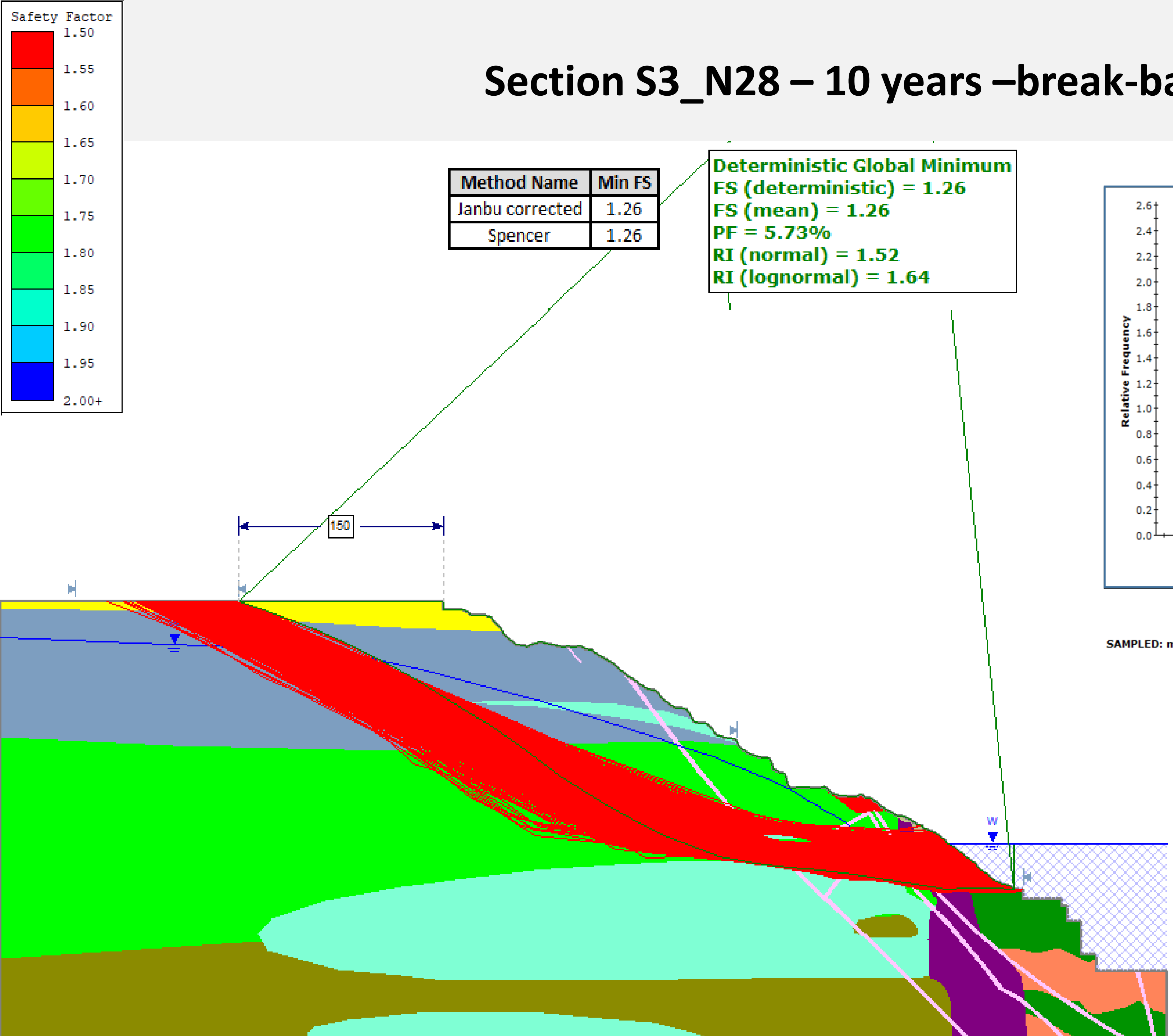


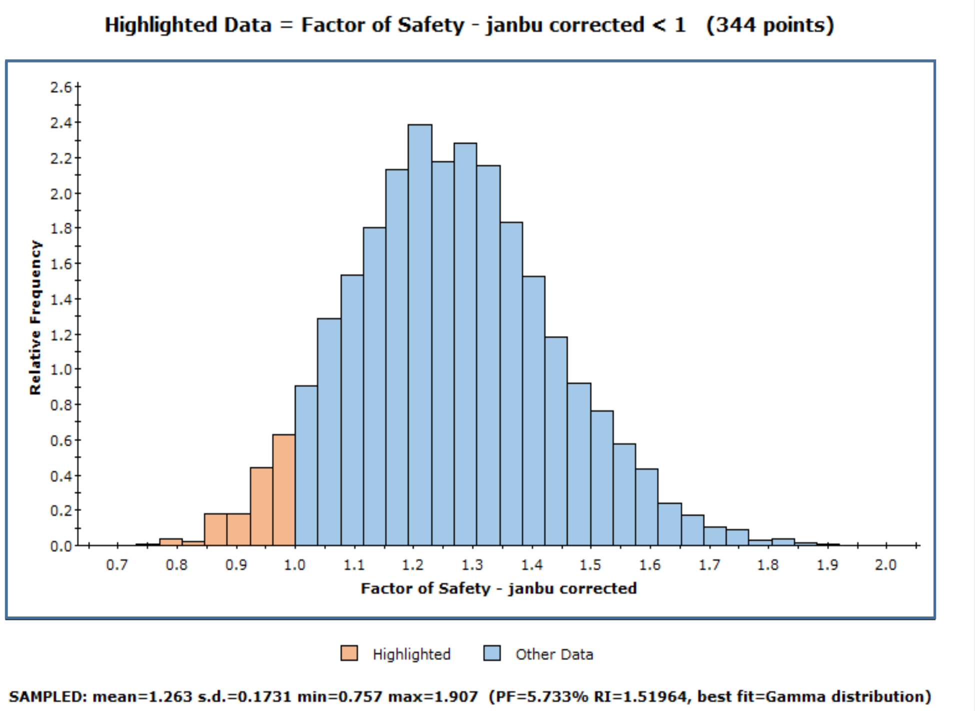




Probability FS ≤ 1.0 = 14.4%

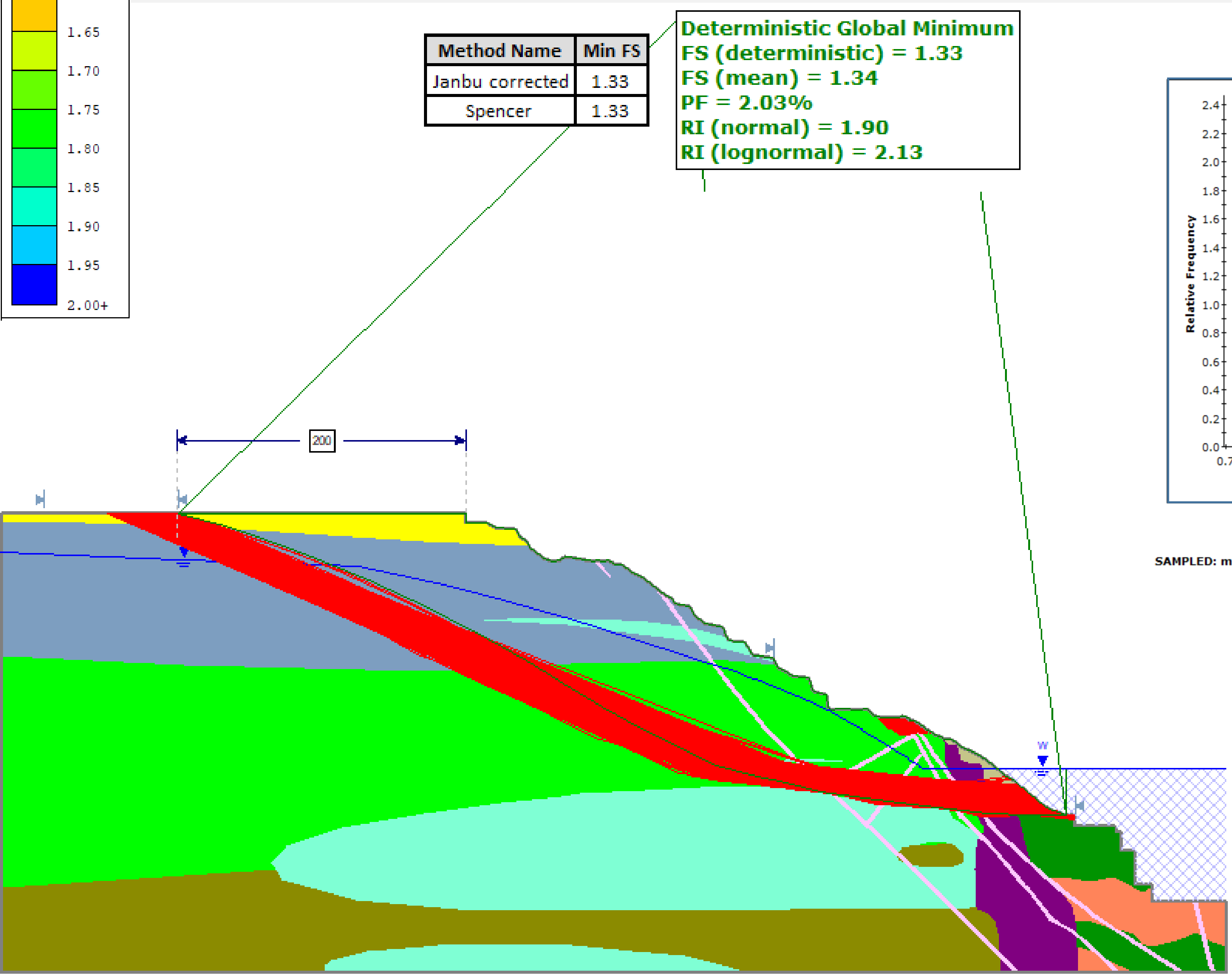
Section S3_N28 – 10 years –break-back: 150m

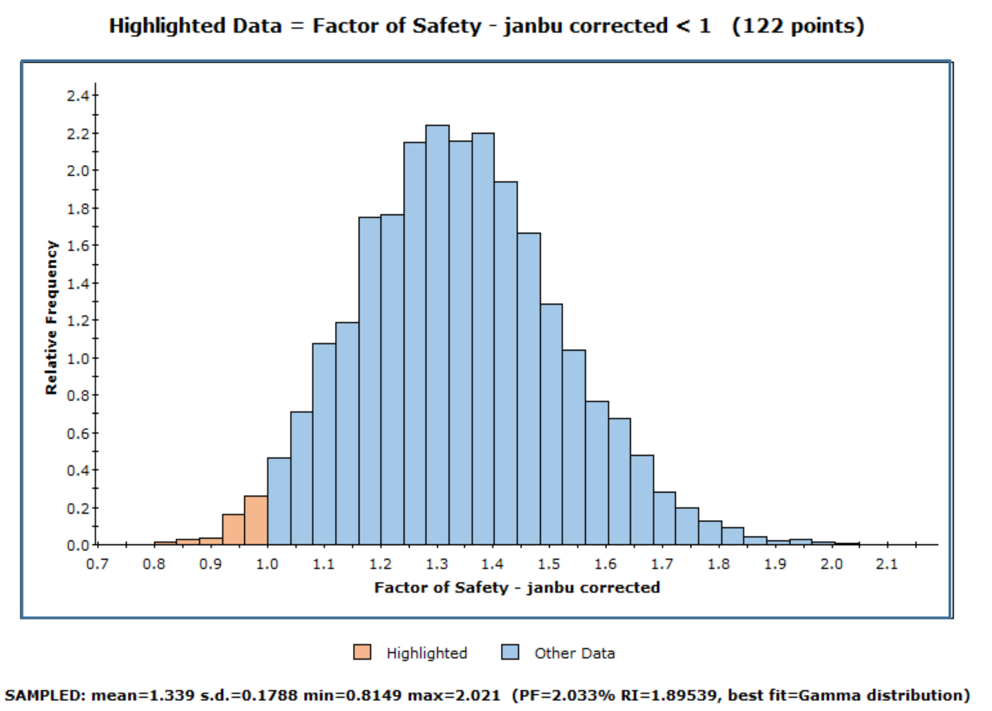




Probability FS ≤ 1.0 = 5.7%

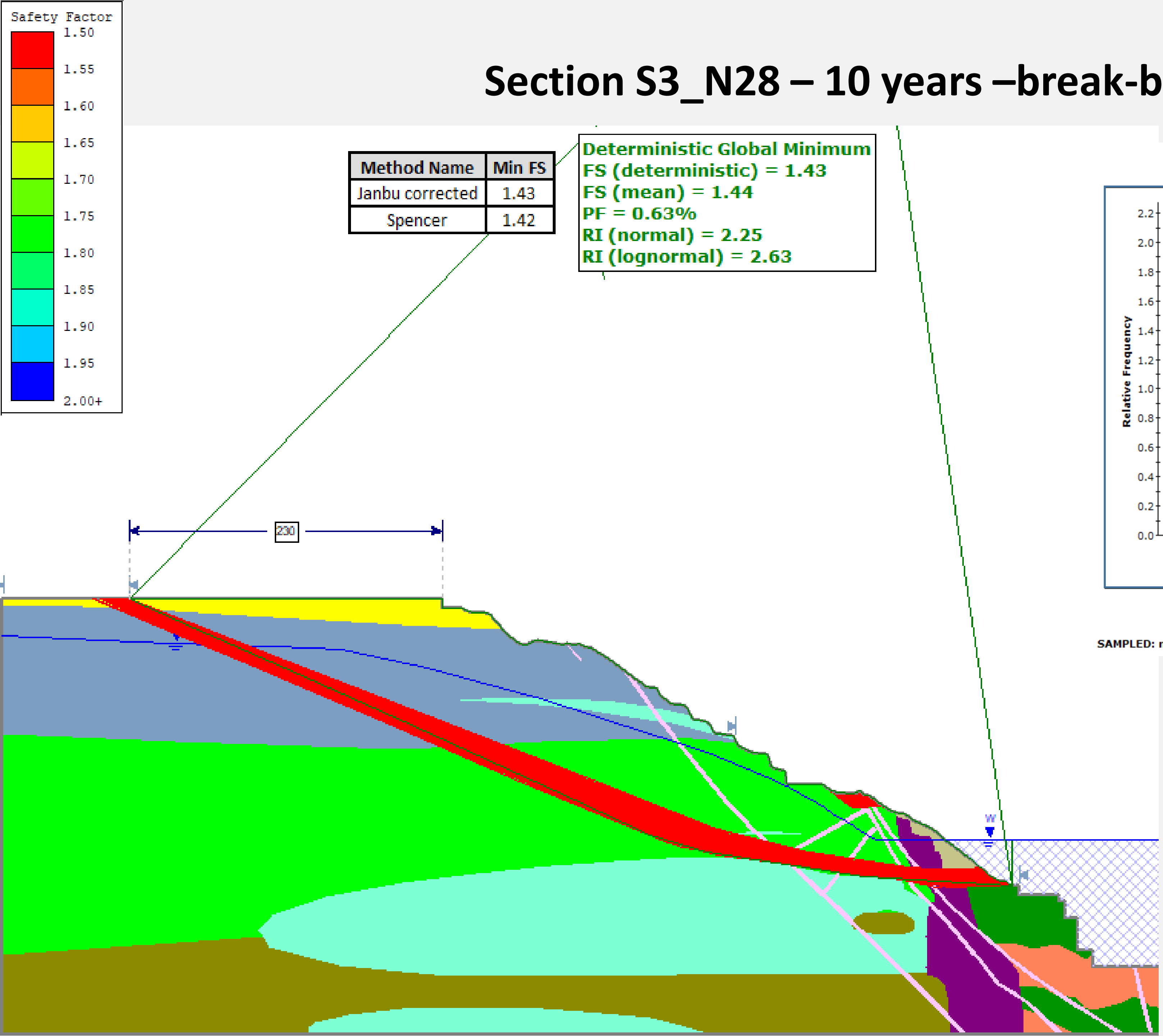
Section S3_N28 – 10 years –break-back: 200m





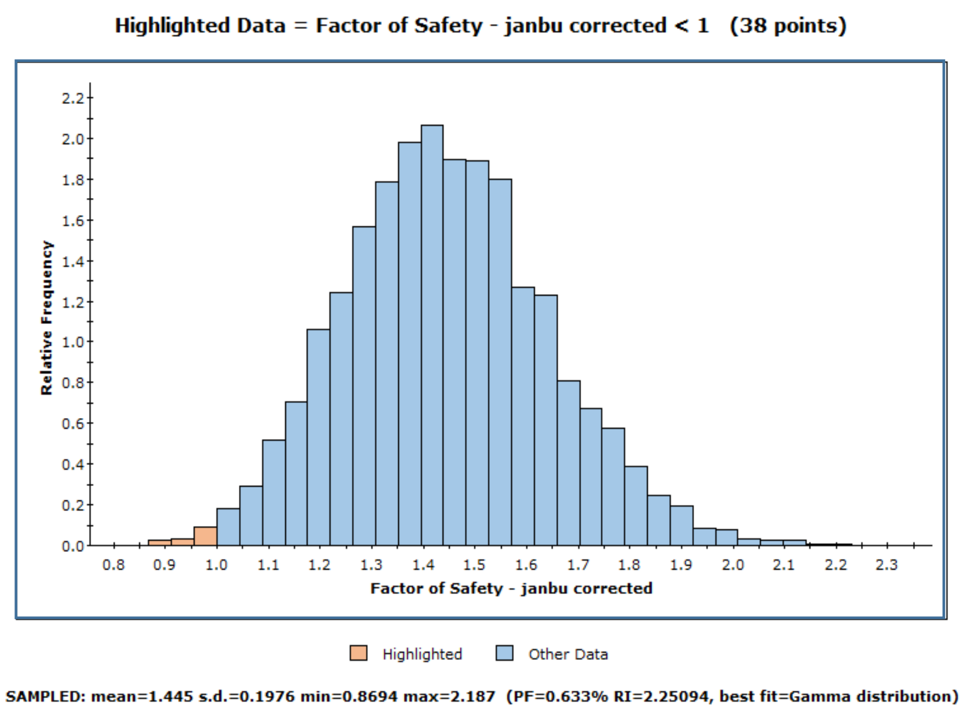
Probability FS ≤ 1.0 = 2.0%

Section S3_N28 – 10 years –break-back: 230m



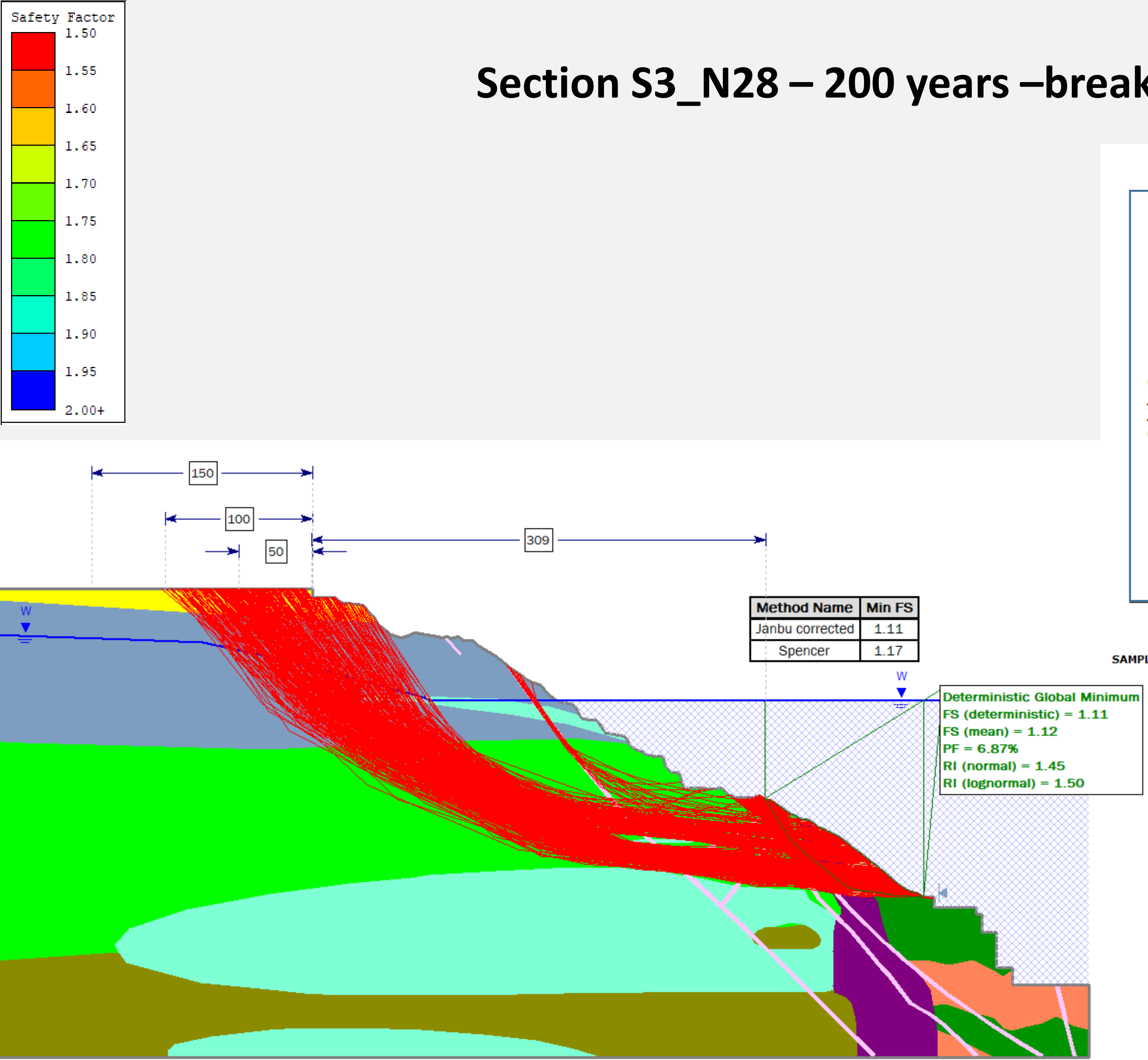
Method Name	Min FS
Janbu corrected	1.43
Spencer	1.42

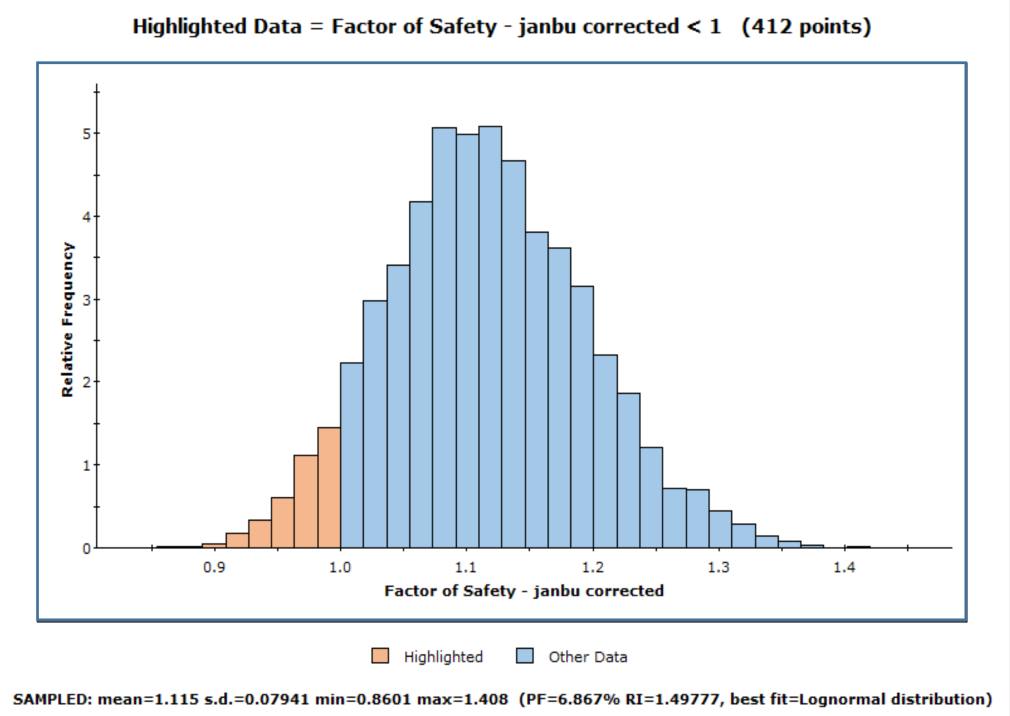
Deterministic Global Minimum
FS (deterministic) = 1.43
FS (mean) = 1.44
PF = 0.63%
RI (normal) = 2.25
RI (lognormal) = 2.63



Probability FS ≤ 1.0 = 0.6%

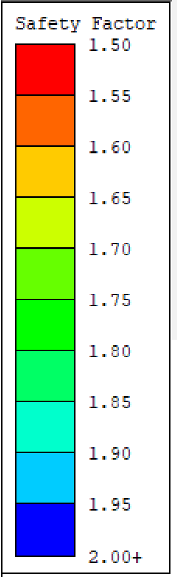
Section S3_N28 – 200 years –break-back: -309m

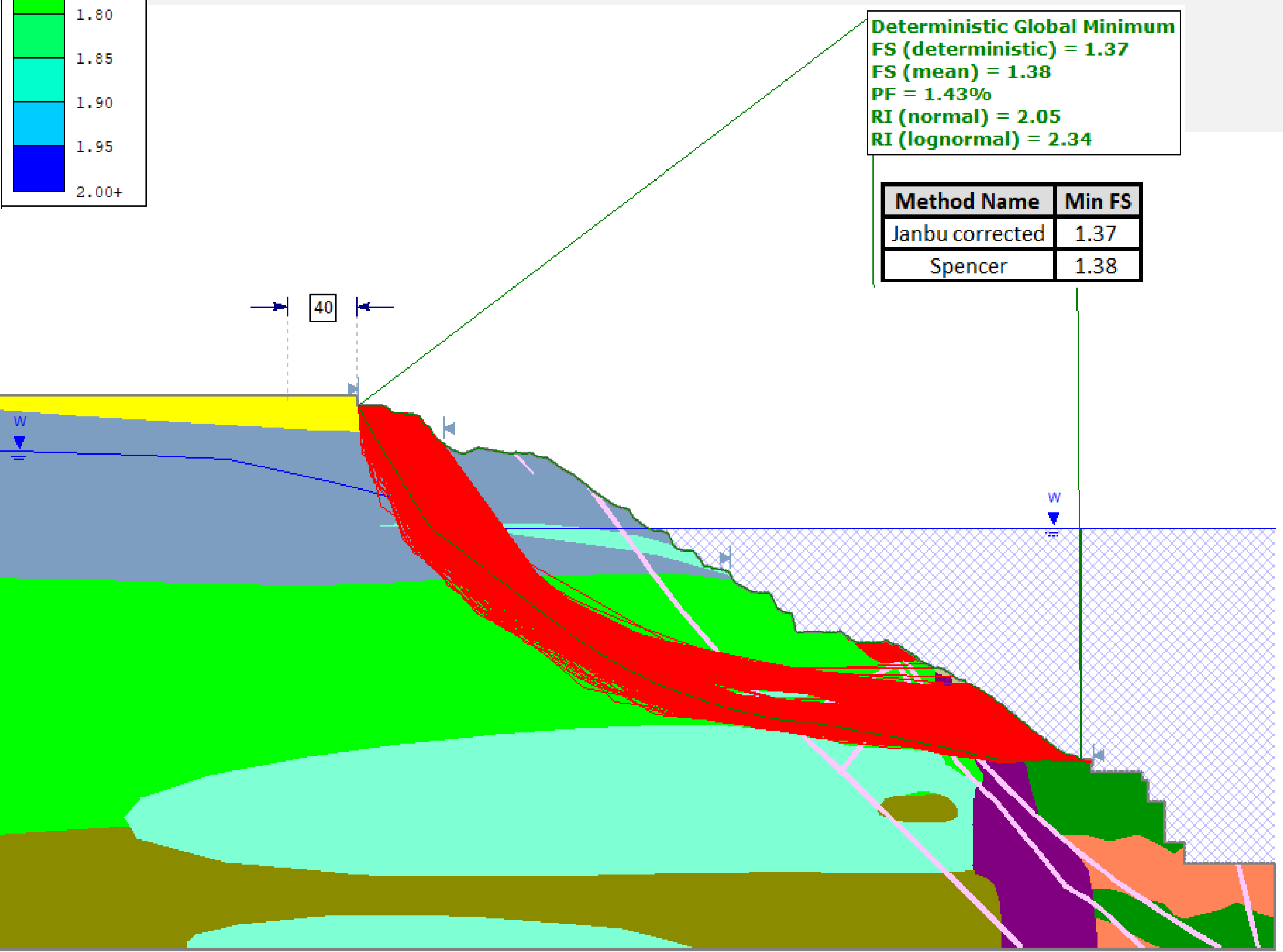




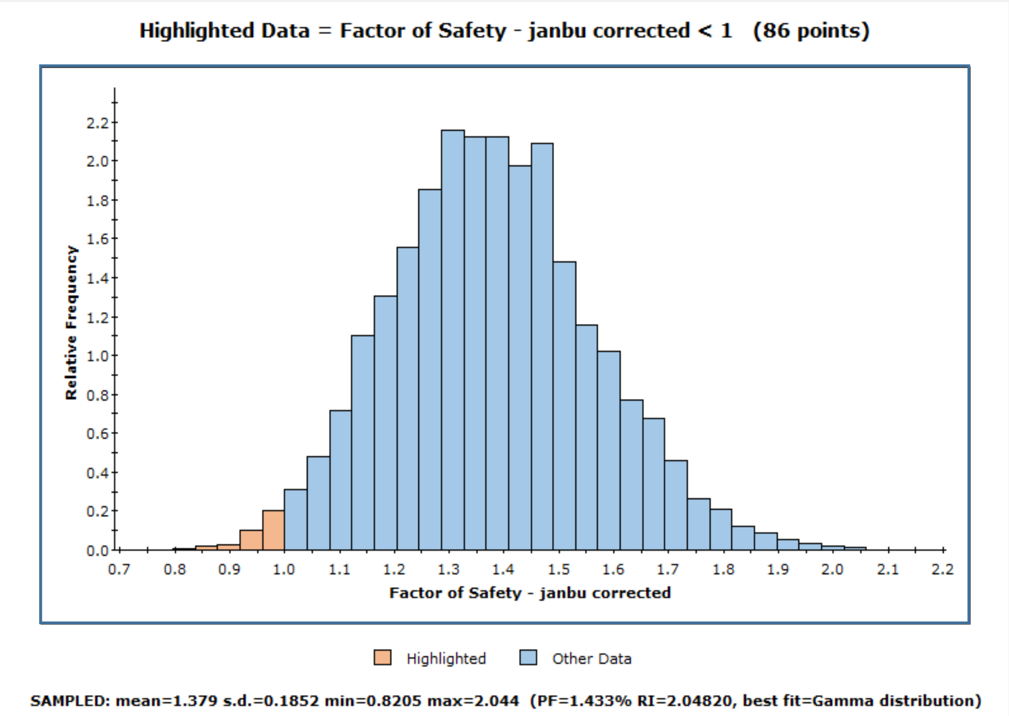
Probability FS ≤ 1.0 = 6.9%

Section S3_N28 – 200 years –break-back: 0m



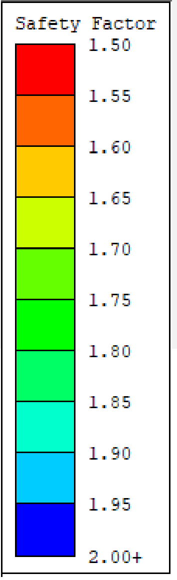


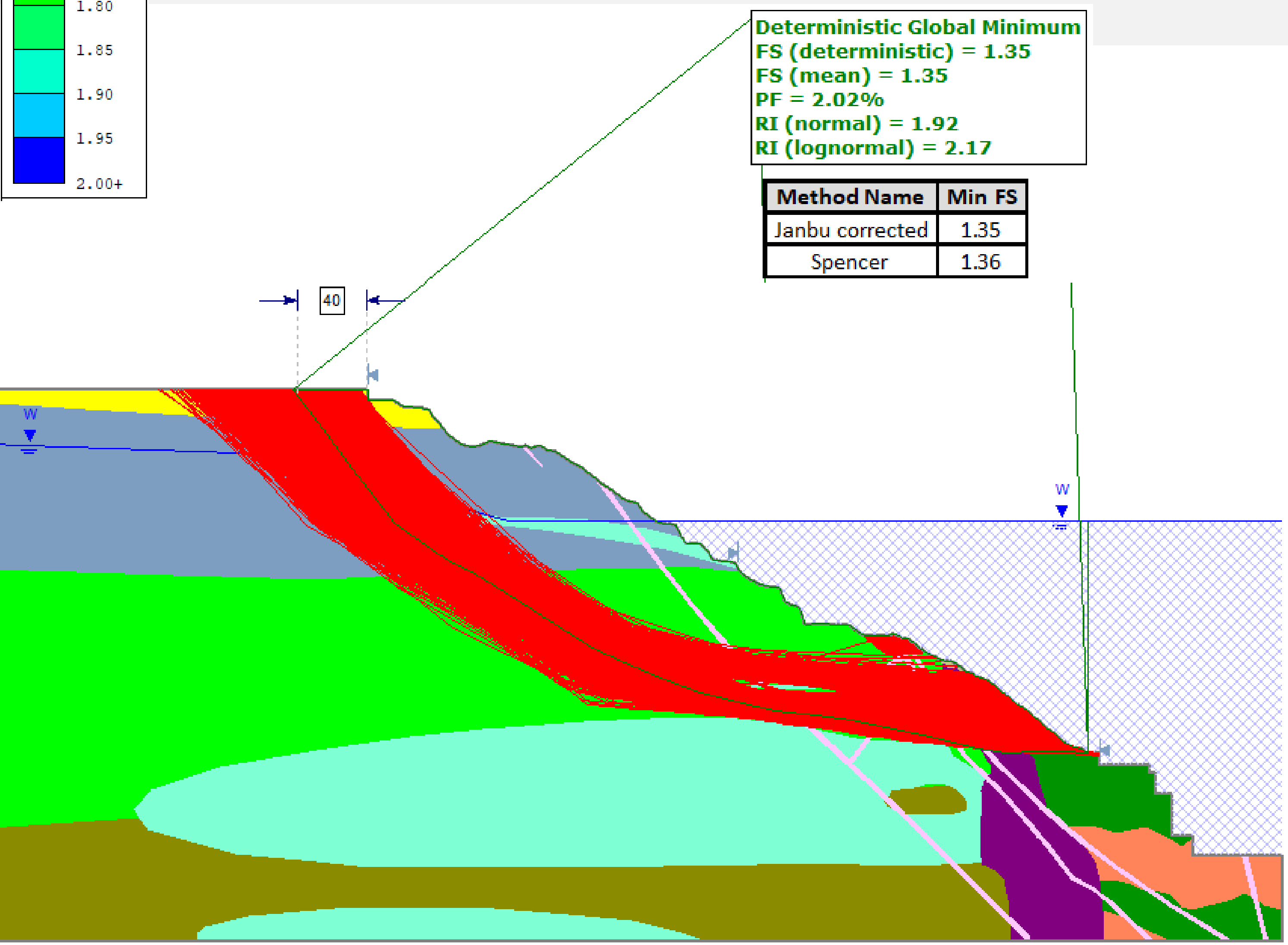
Method Name	Min FS
Janbu corrected	1.37
Spencer	1.38

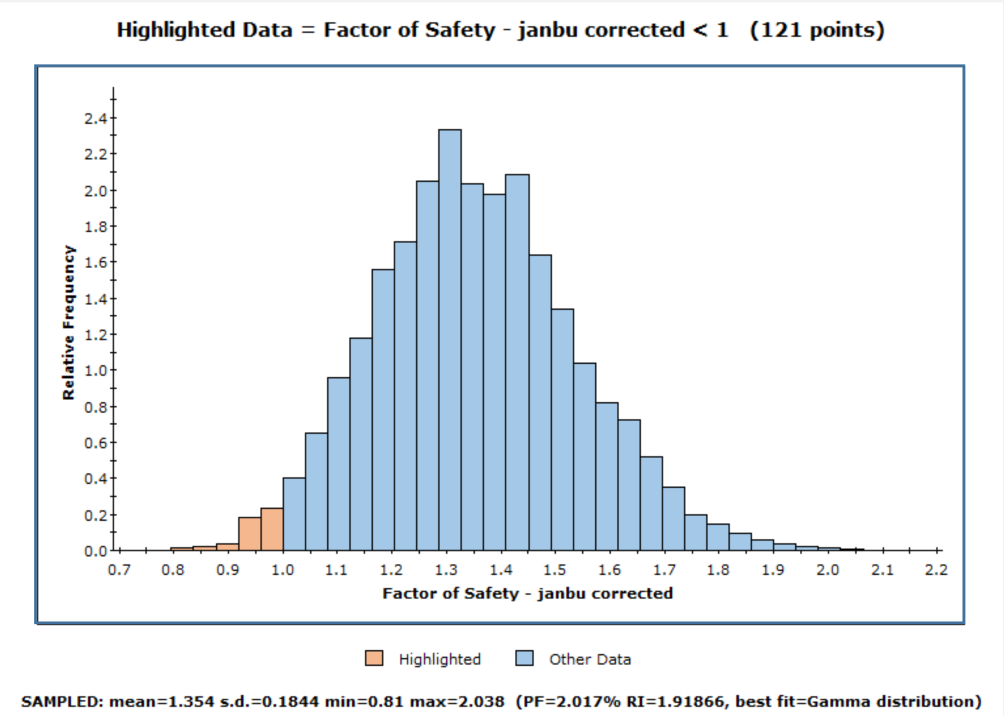


Probability FS ≤ 1.0 = 1.4%

Section S3_N28 – 200 years –break-back: 40m

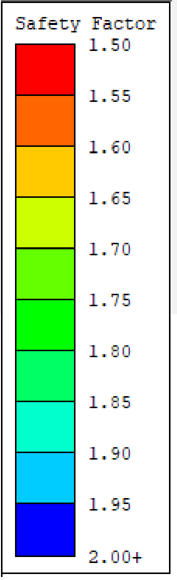


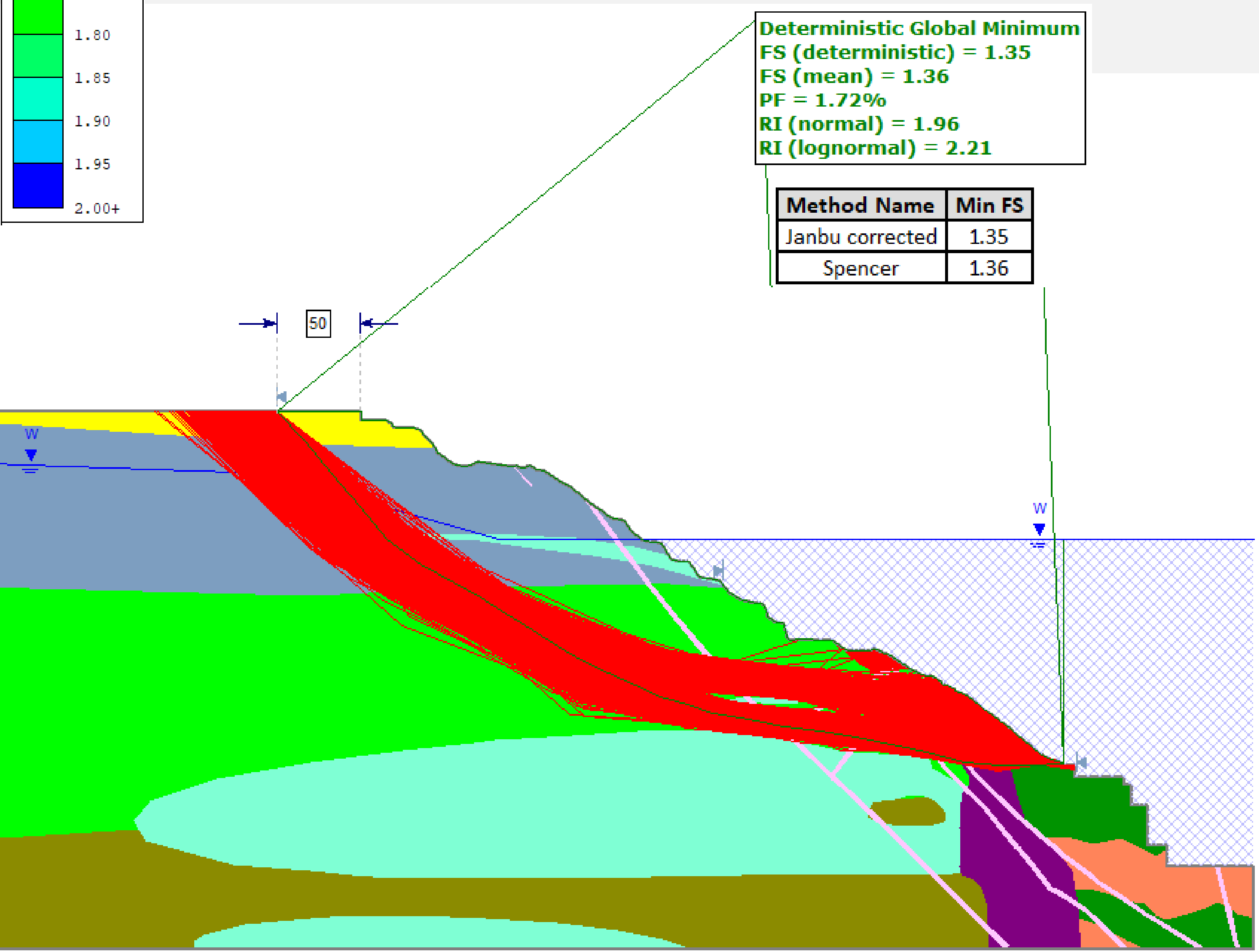


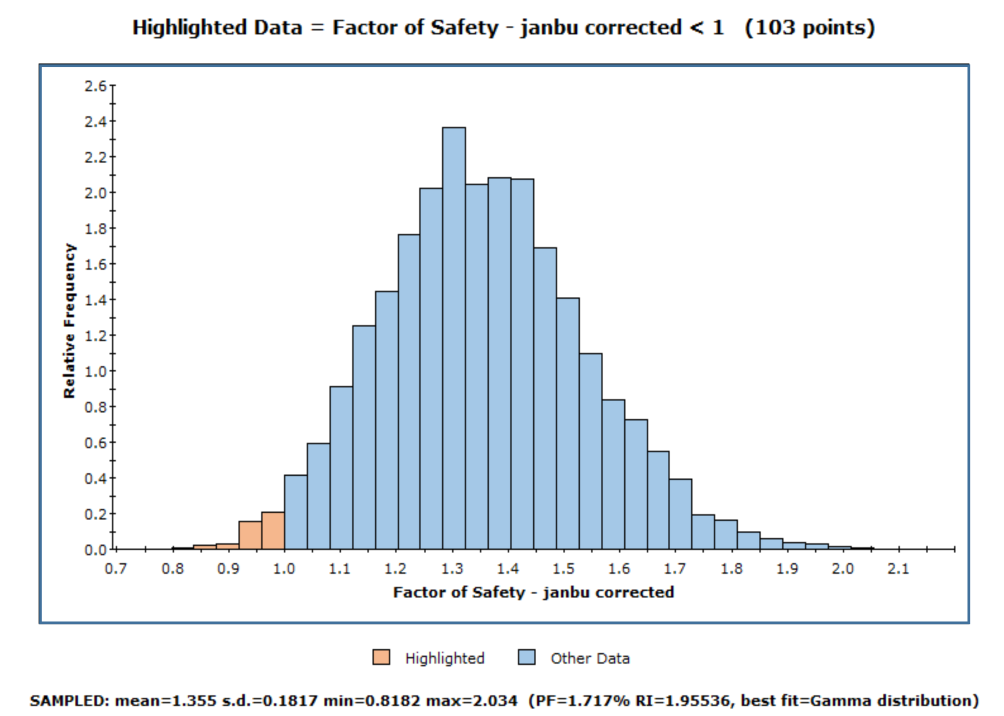


Probability FS ≤ 1.0 = 2.0%

Section S3_N28 – 200 years –break-back: 50m

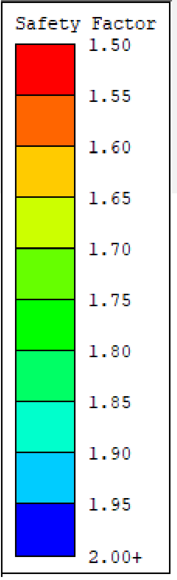


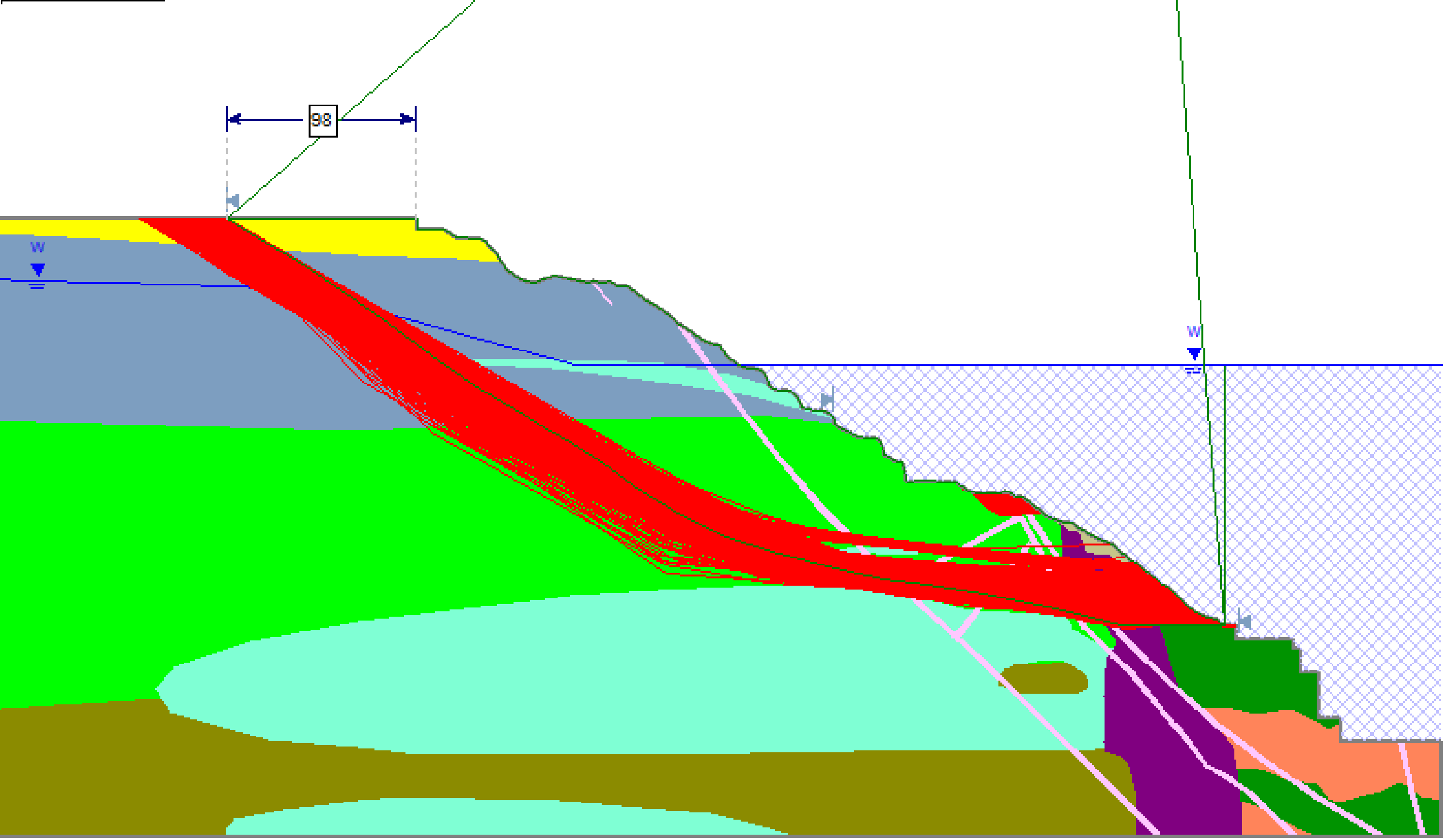


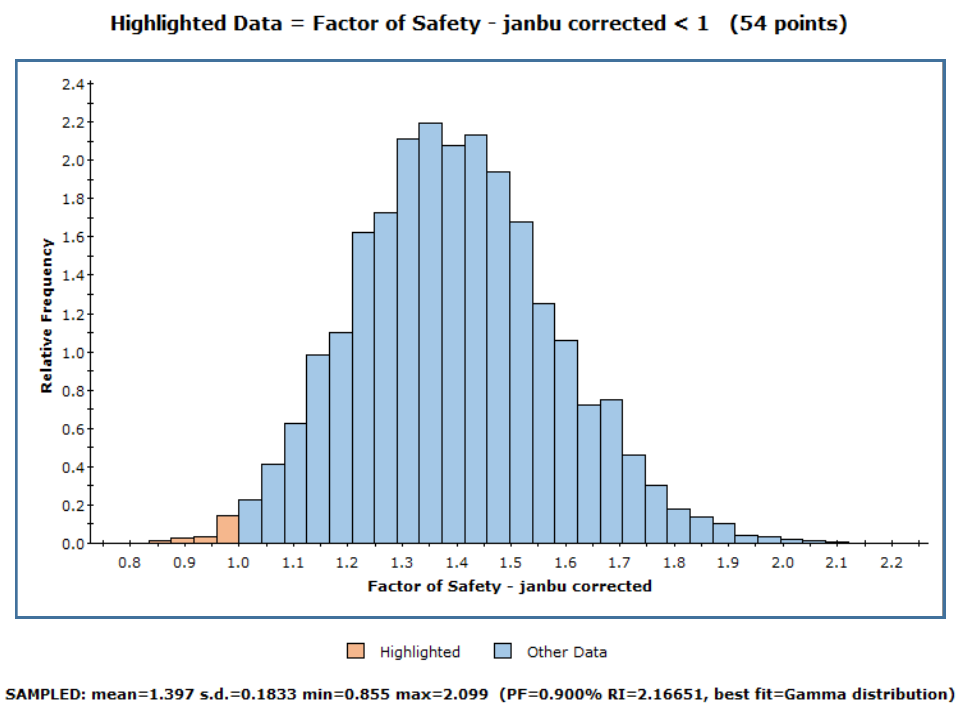


Probability FS ≤ 1.0 = 1.7%

Section S3_N28 – 200 years –break-back: 100m



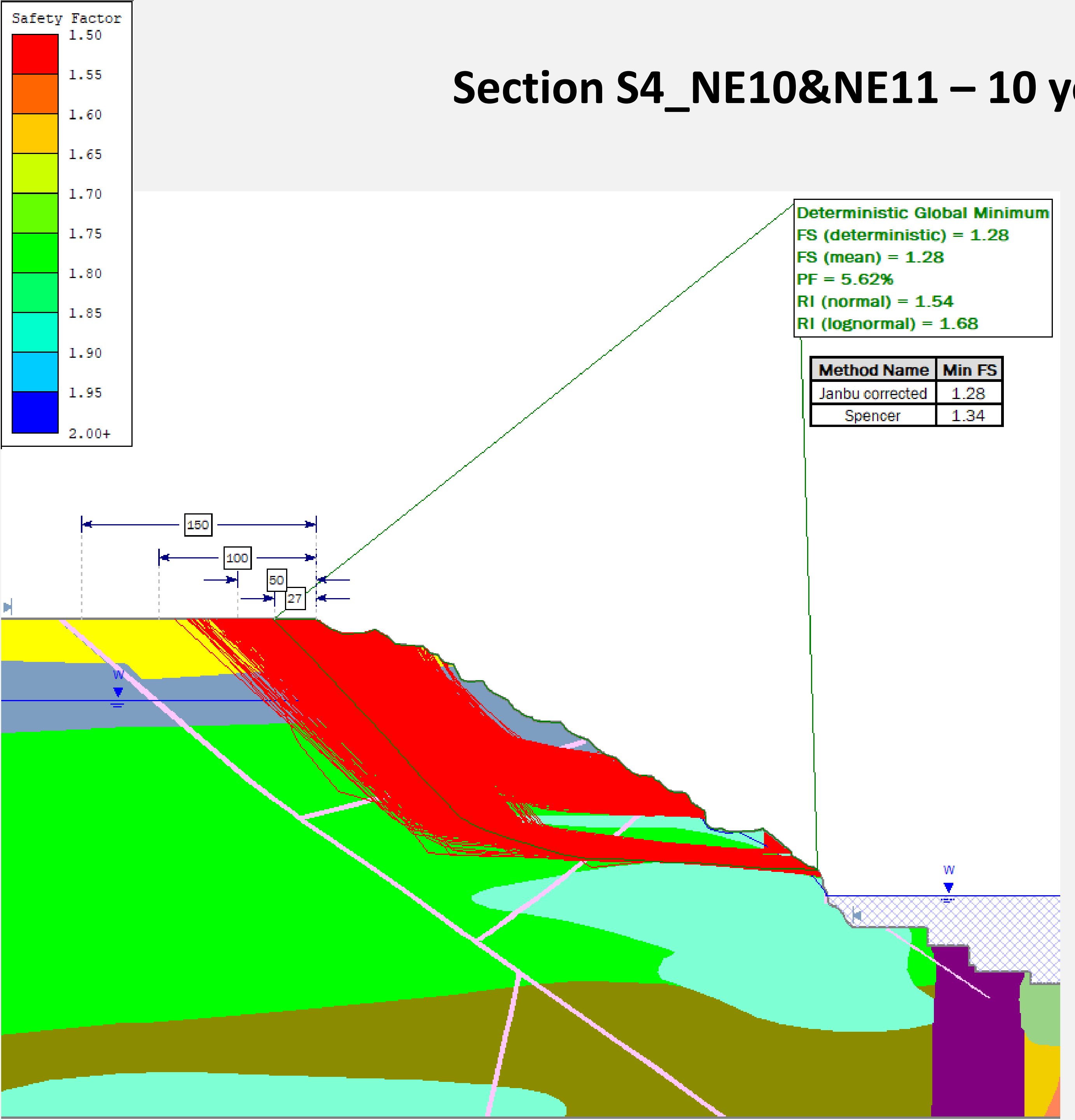


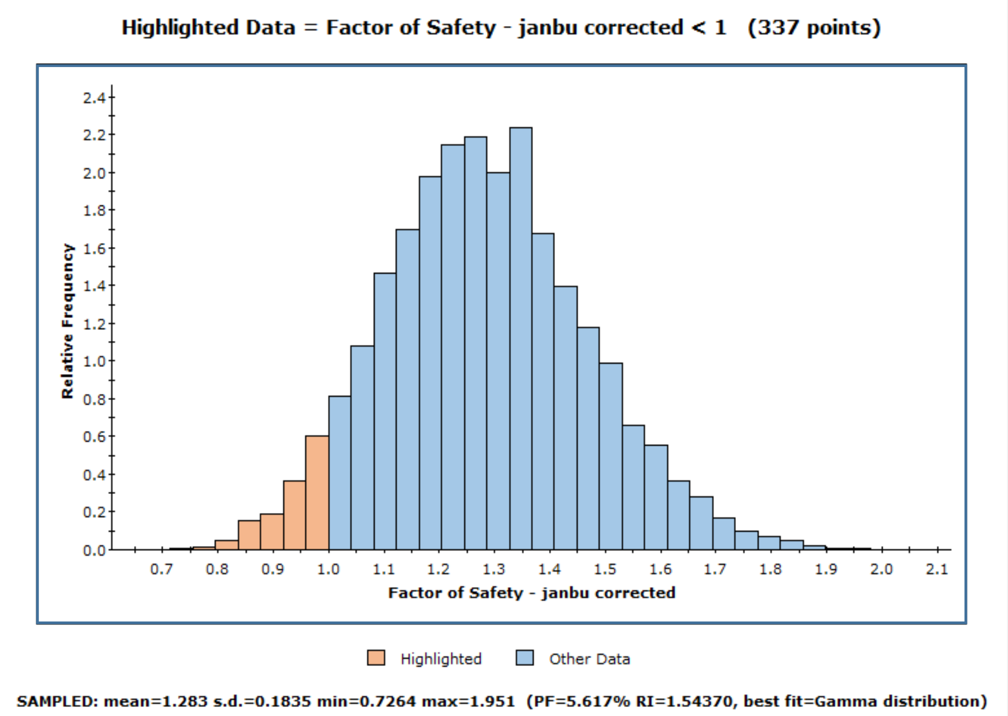


Probability FS ≤ 1.0 = 0.9%

Section S4_NE10&NE11

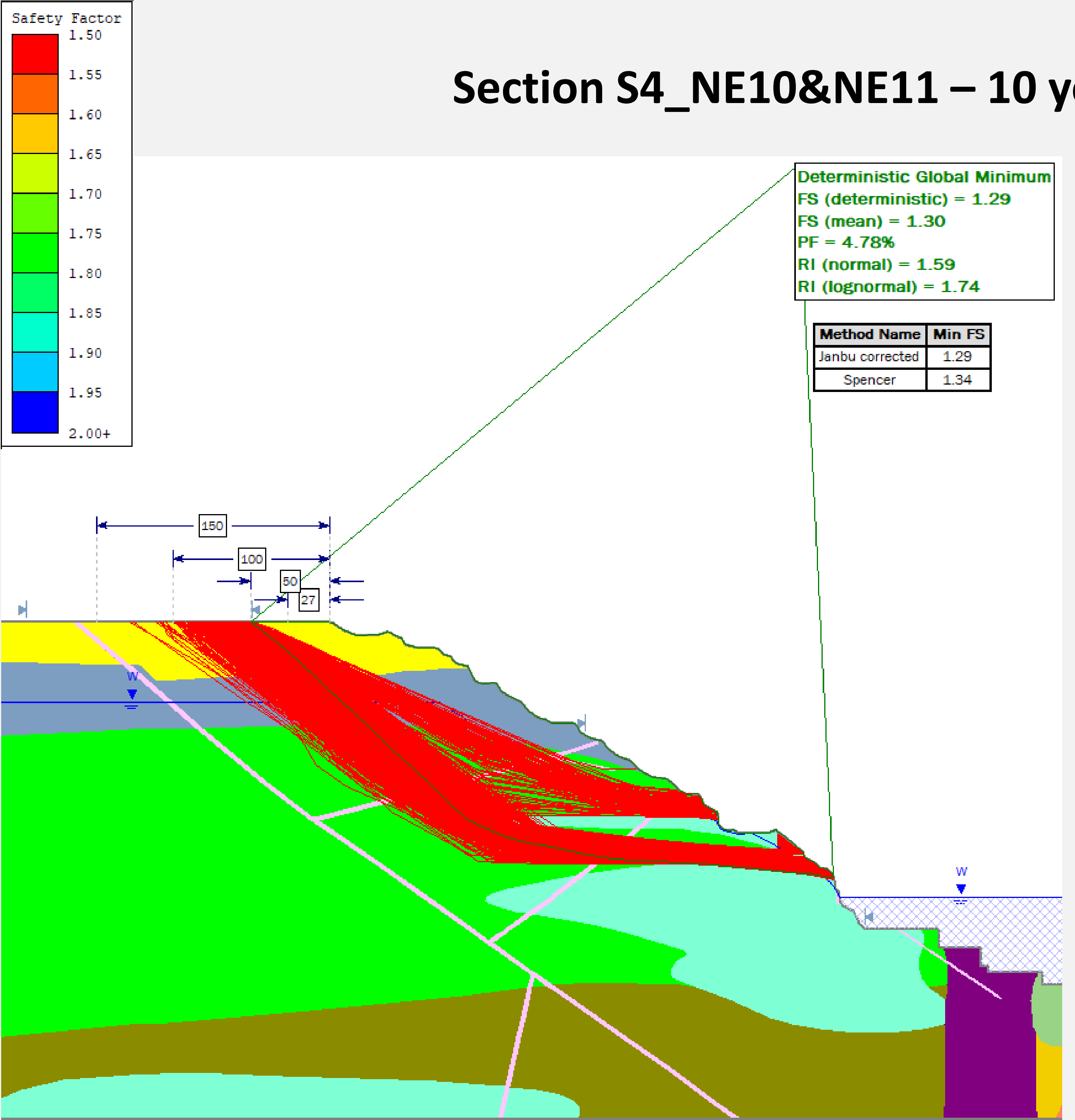
Section S4_NE10&NE11 – 10 years –break-back: 27m

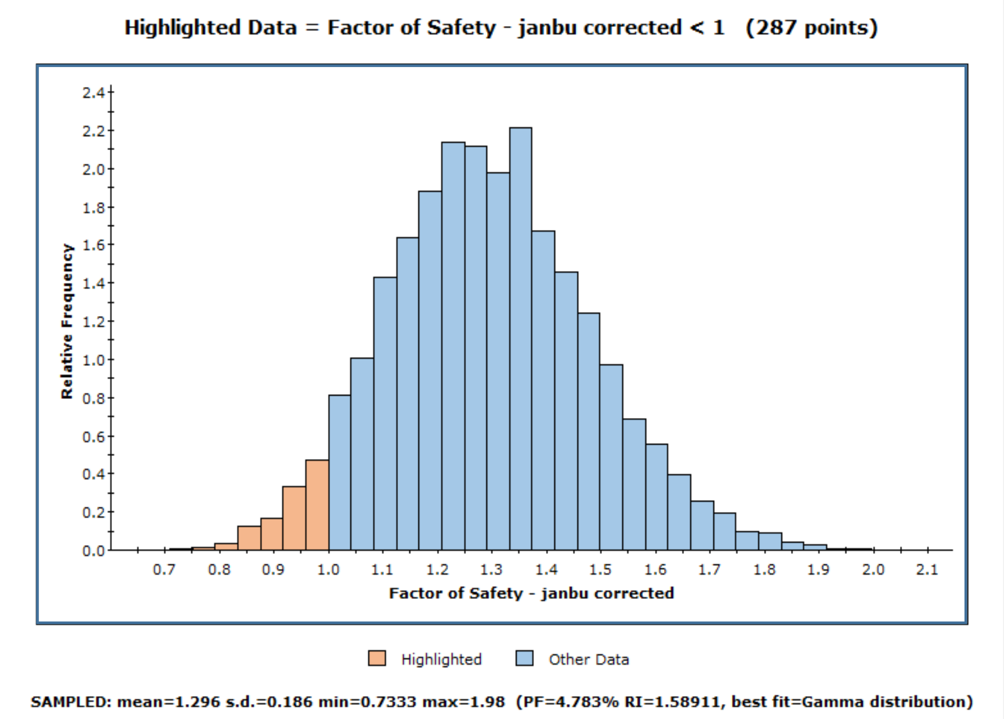




Probability FS ≤ 1.0 = 5.6%

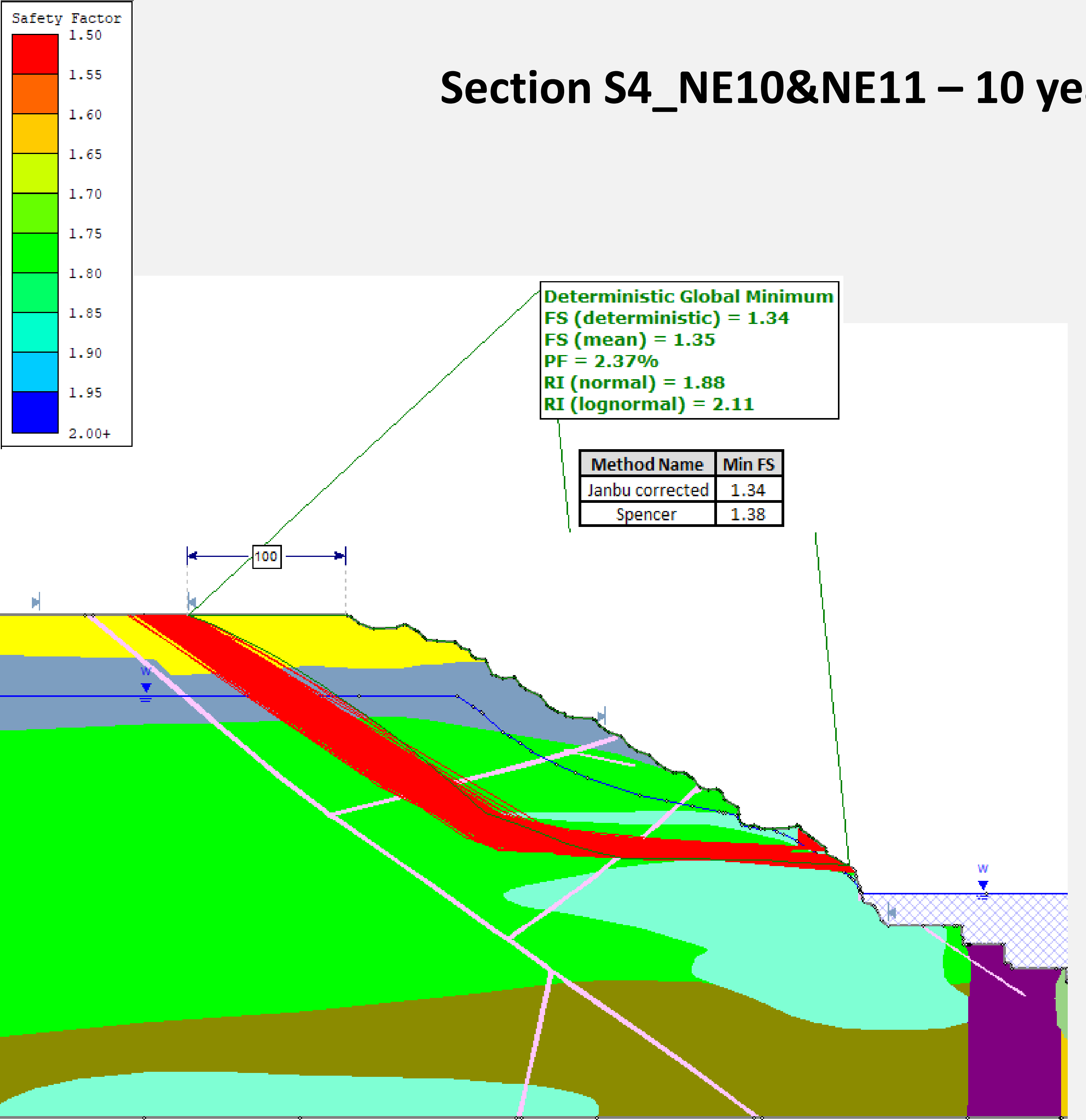
Section S4_NE10&NE11 – 10 years –break-back: 50m

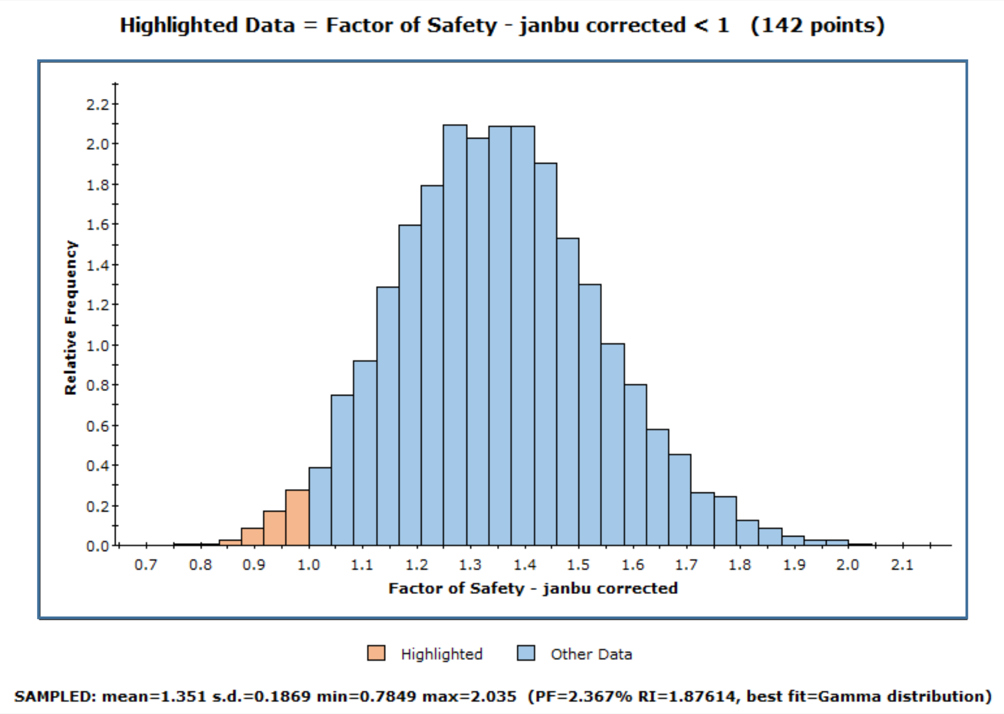




Probability FS ≤ 1.0 = 4.8%

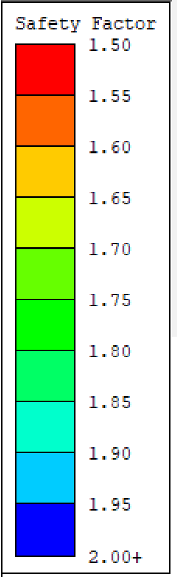
Section S4_NE10&NE11 – 10 years –break-back: 100m

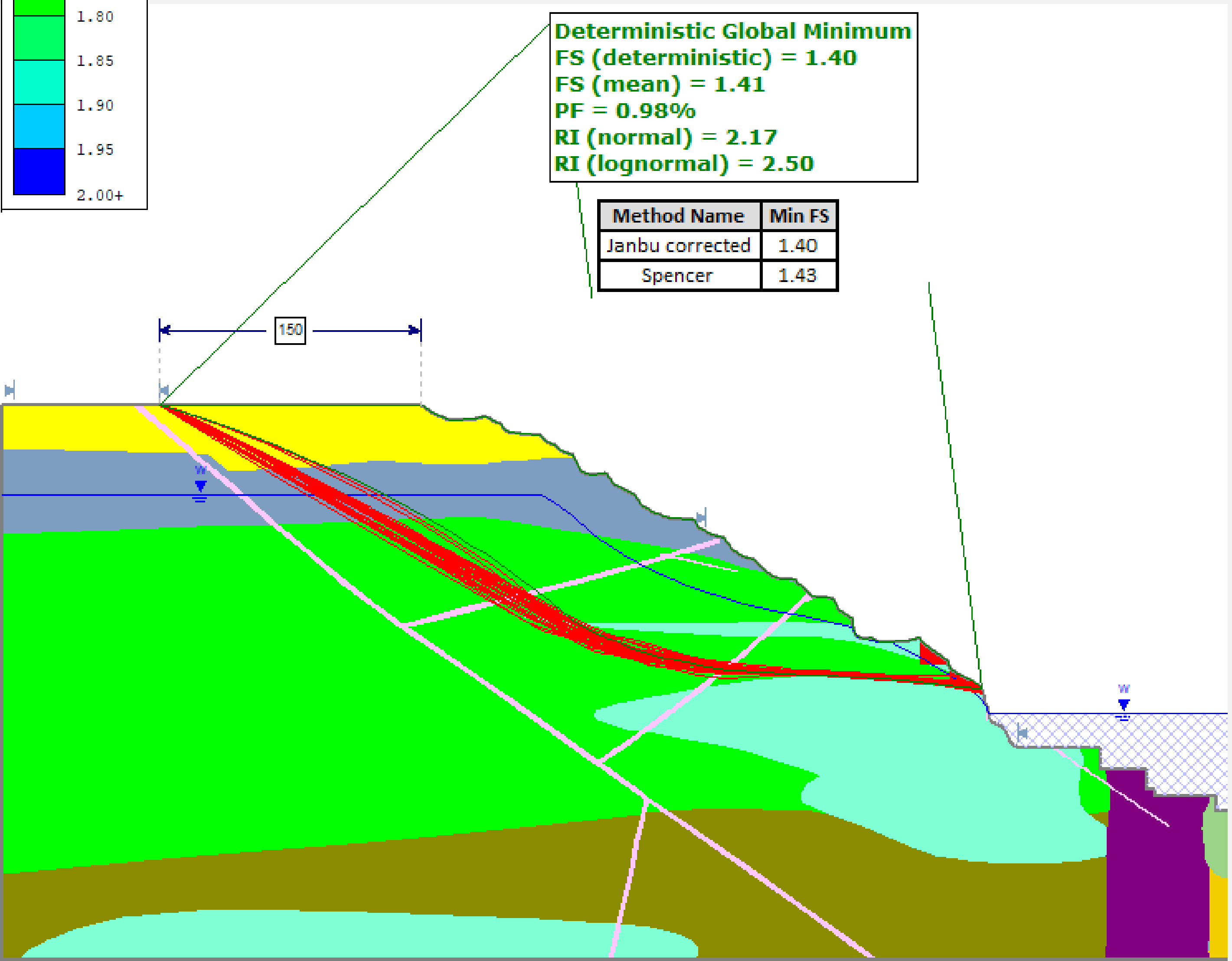


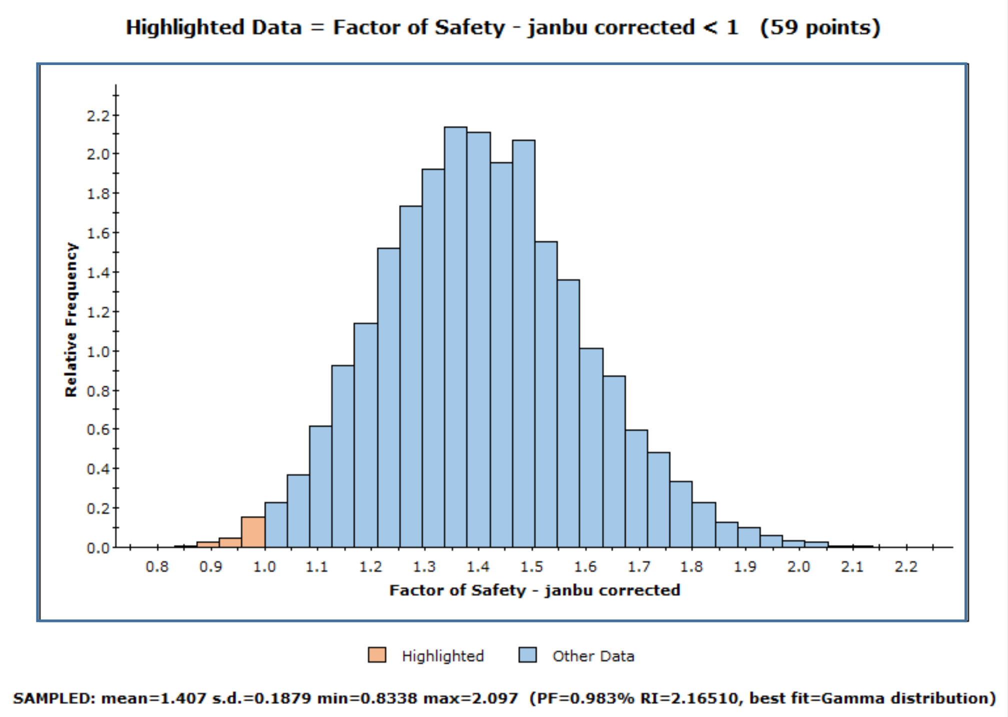


Probability FS ≤ 1.0 = 2.4%

Section S4_NE10&NE11 – 10 years –break-back: 150m

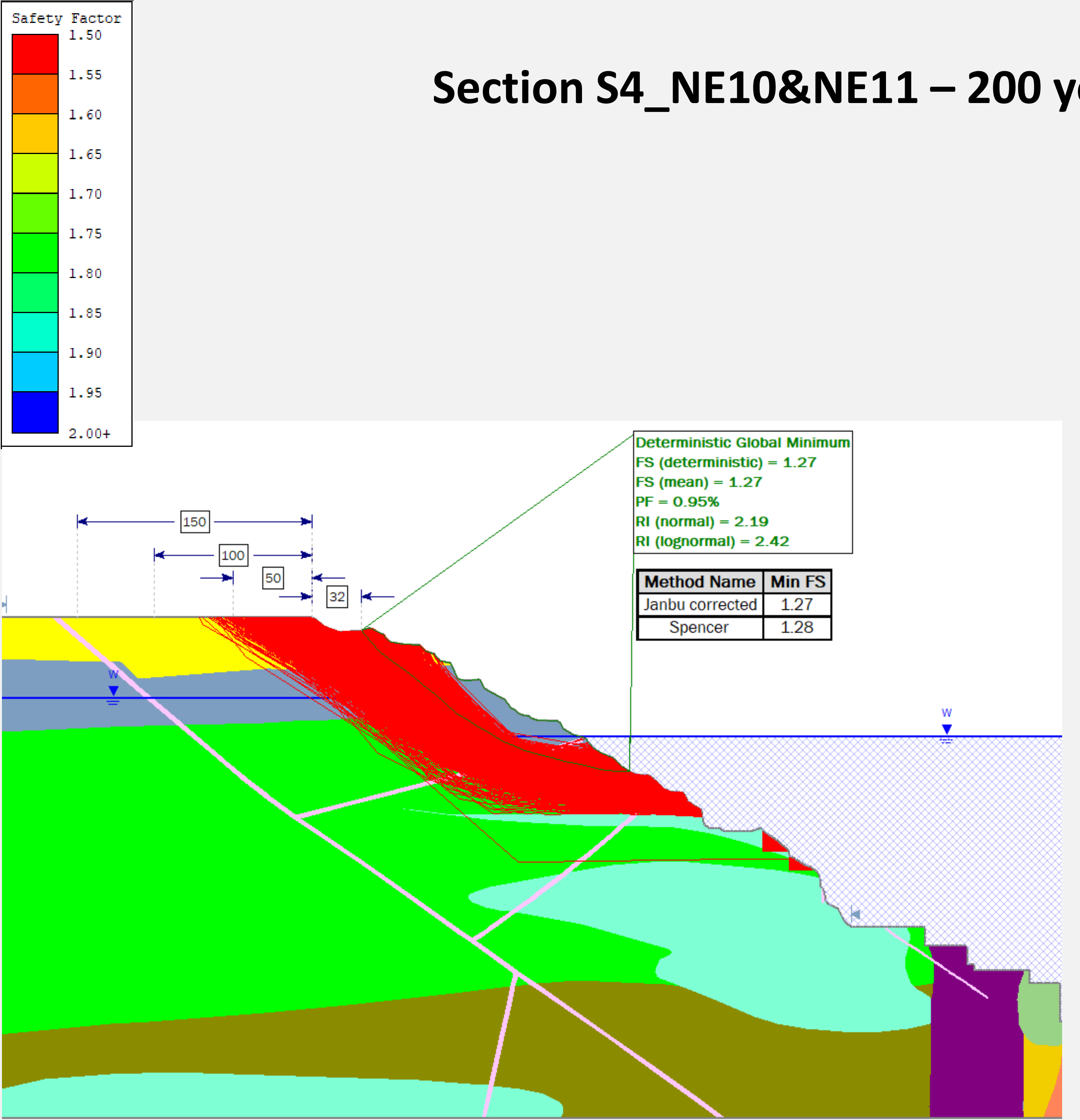


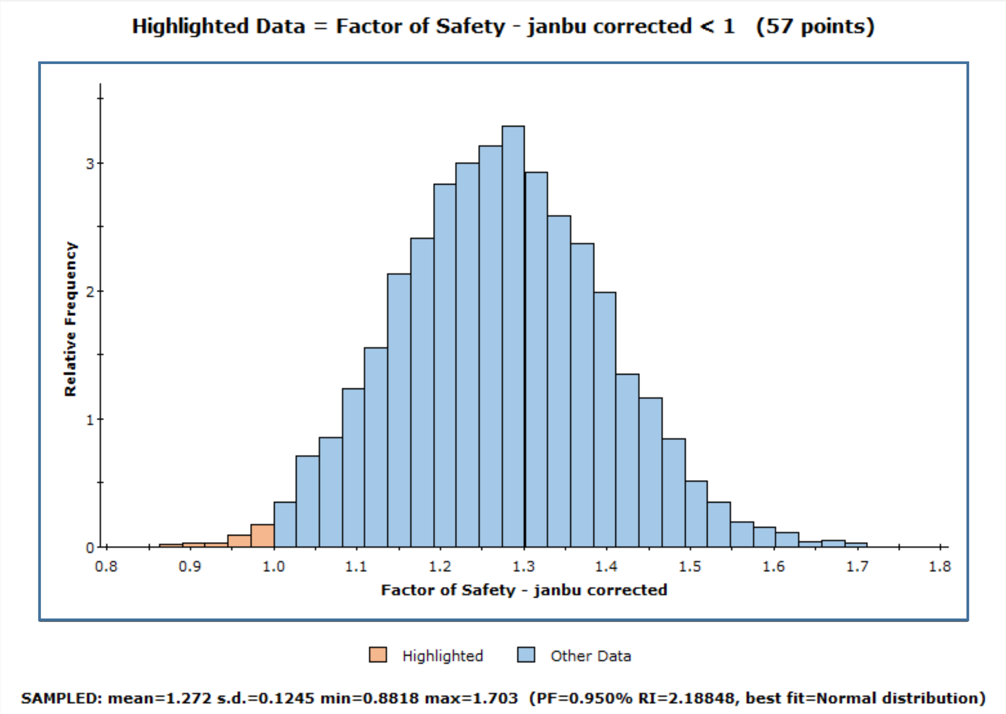




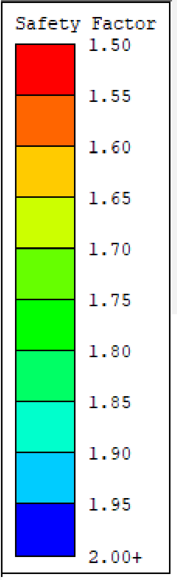
Probability FS ≤ 1.0 = 1.0%

Section S4_NE10&NE11 – 200 years –break-back: -32m

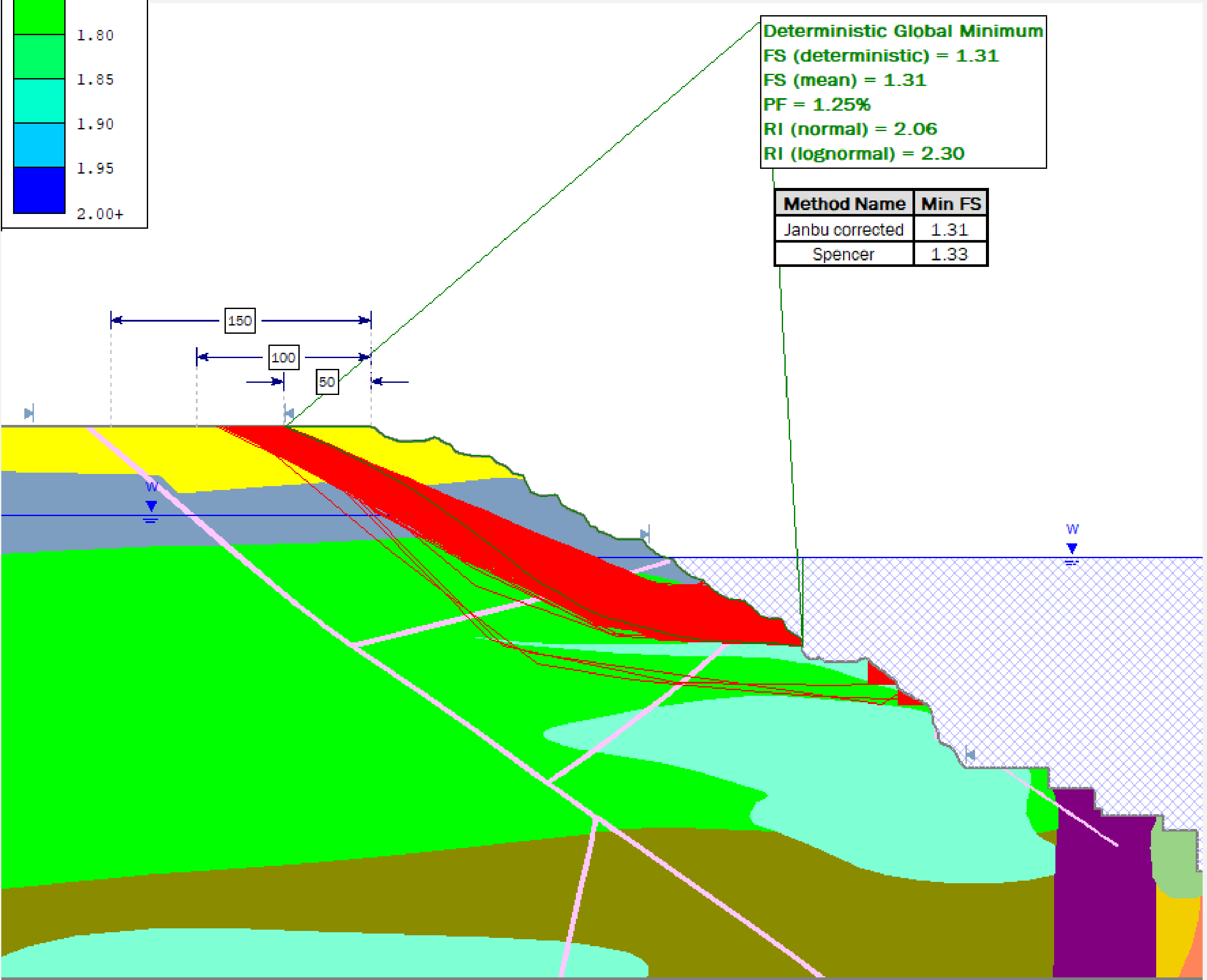


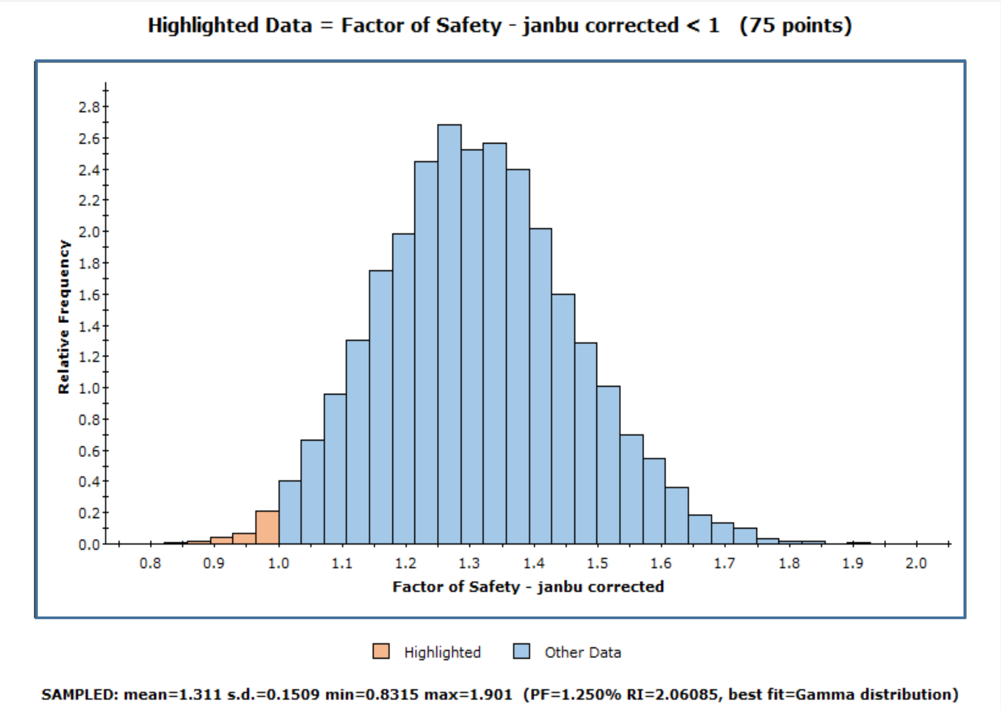


Probability FS ≤ 1.0 = 1.0%



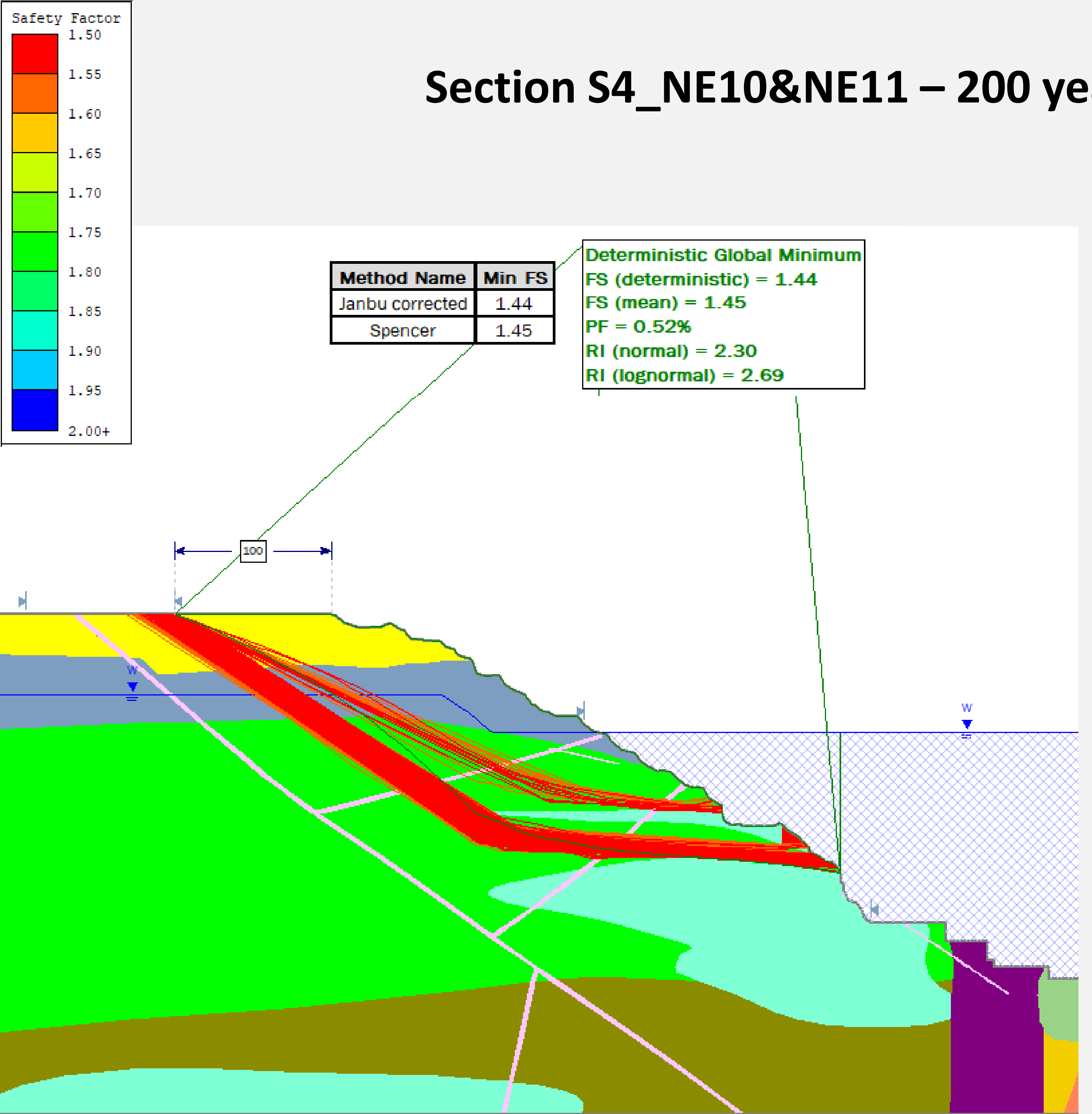
Section S4_NE10&NE11 – 200 years –break-back: 50m

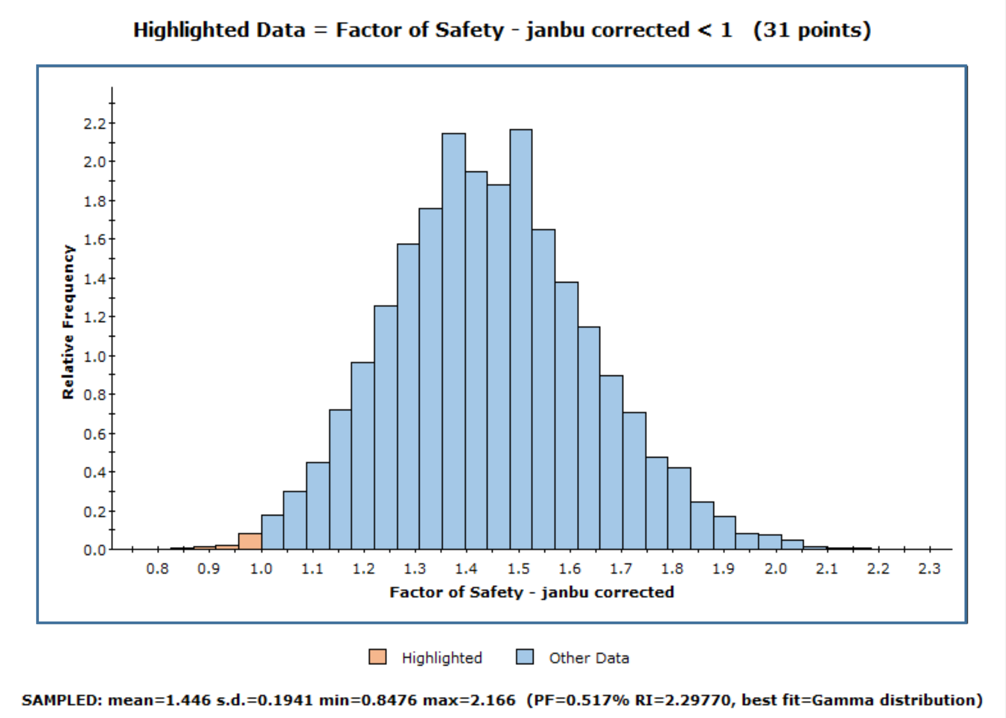




Probability FS ≤ 1.0 = 1.3%

Section S4_NE10&NE11 – 200 years –break-back: 100m

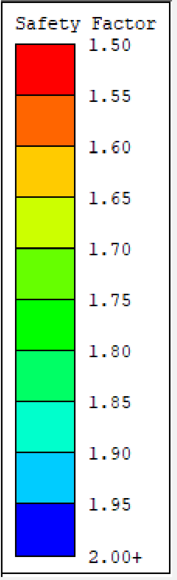


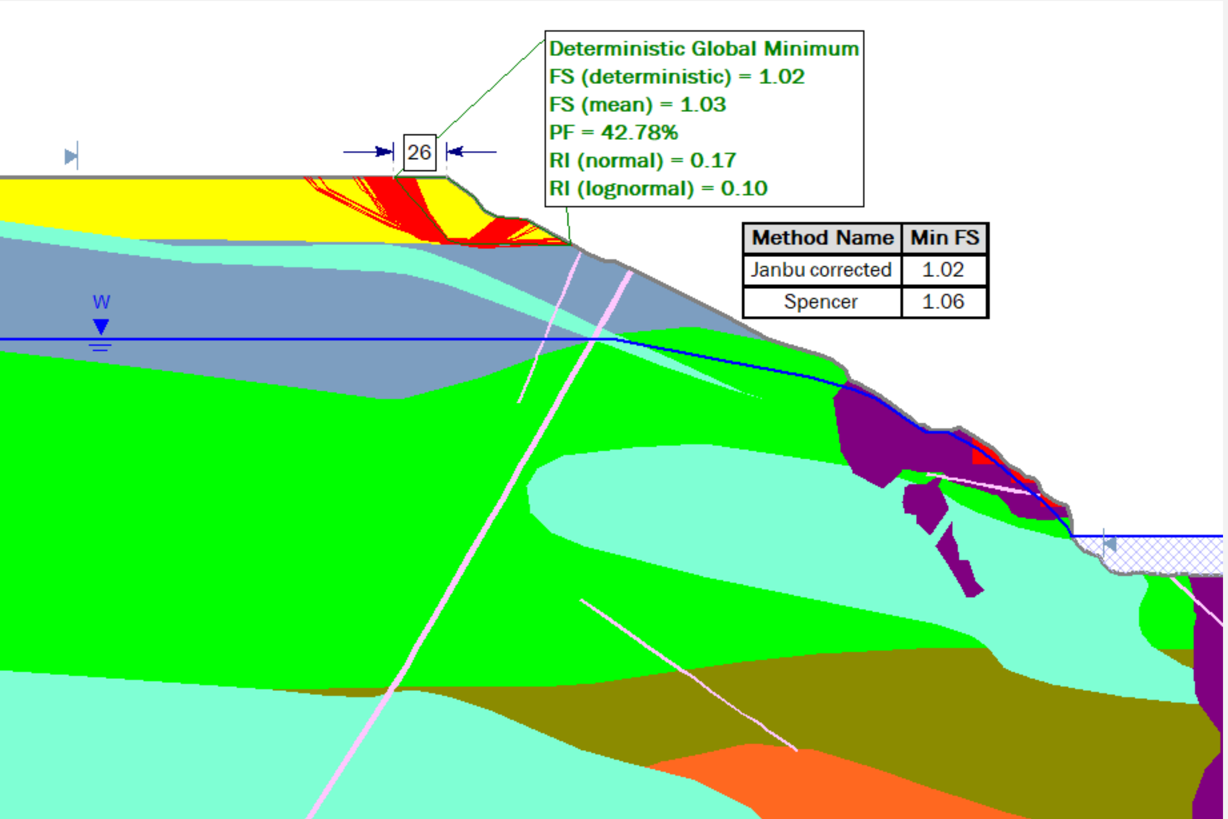


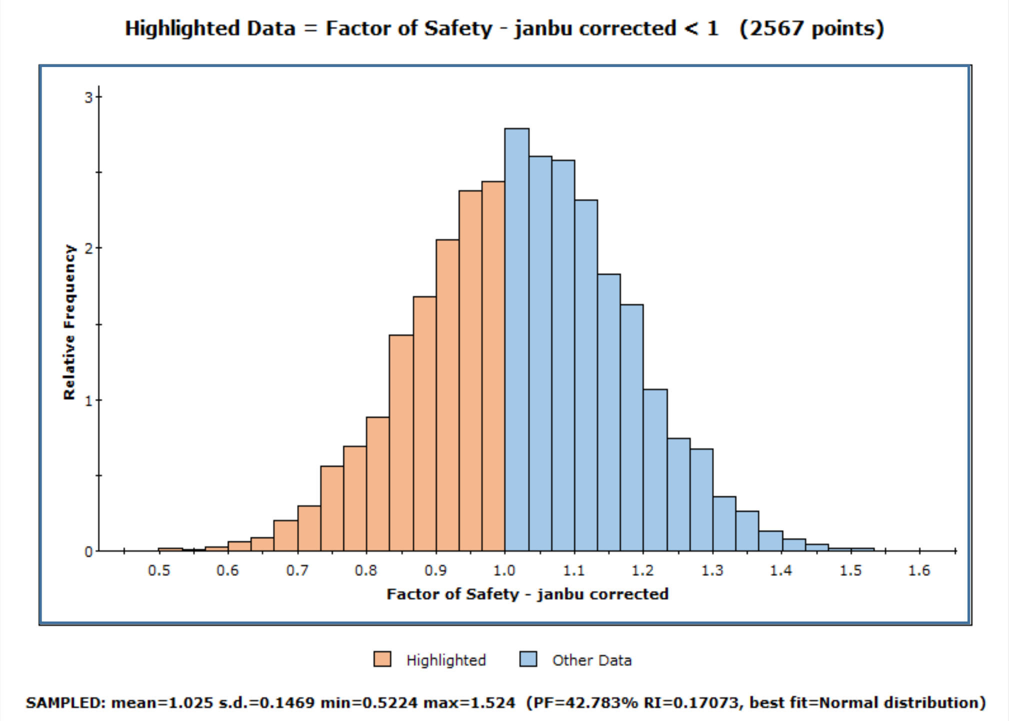
Probability FS ≤ 1.0 = 0.5%

Section S5_NE3&NE6

Section S5_NE3&NE6 – 10 years –break-back: 26m

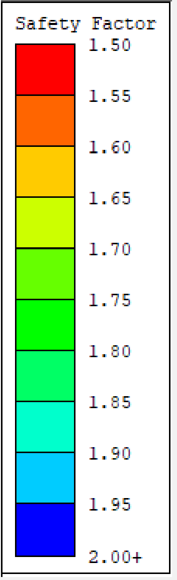


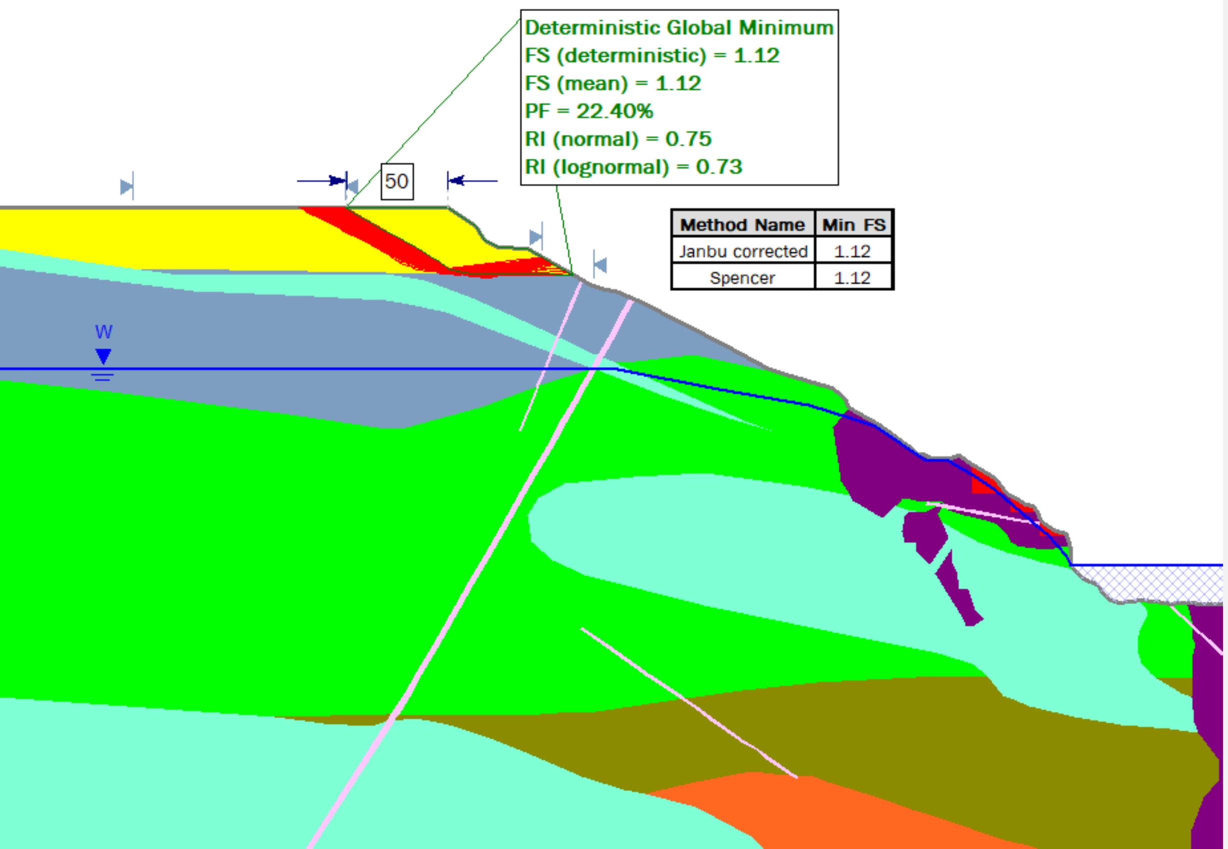


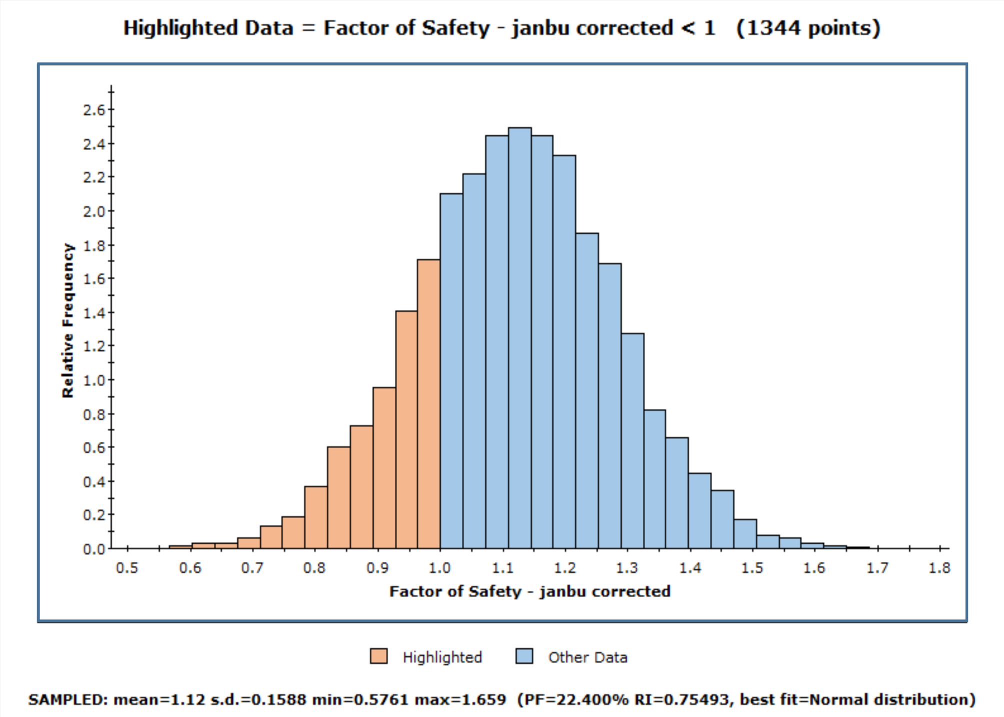


Probability FS ≤ 1.0 = 42.8%

Section S5_NE3&NE6 – 10 years –break-back: 50m

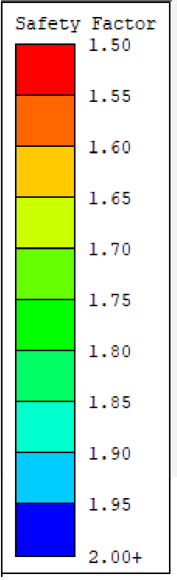


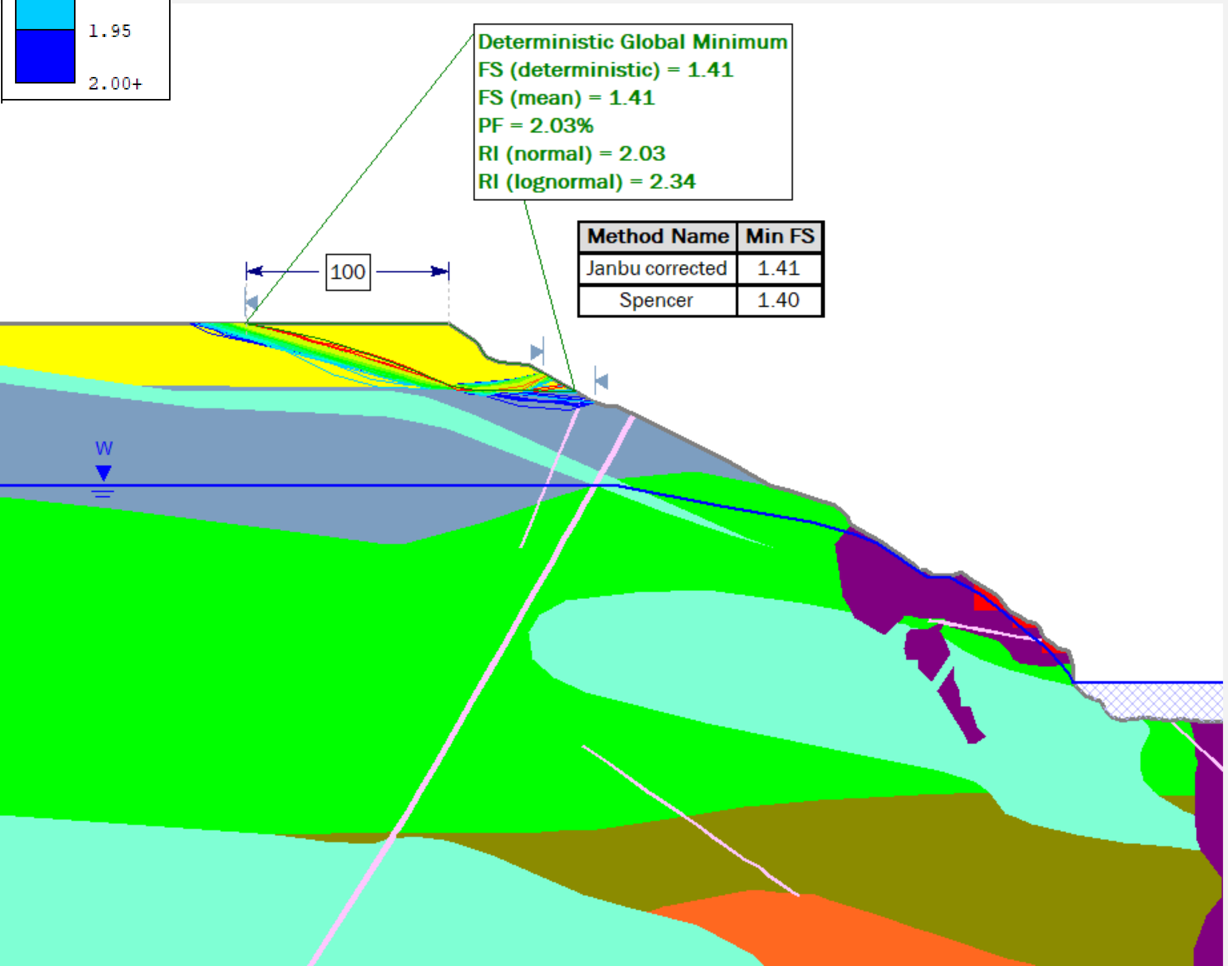


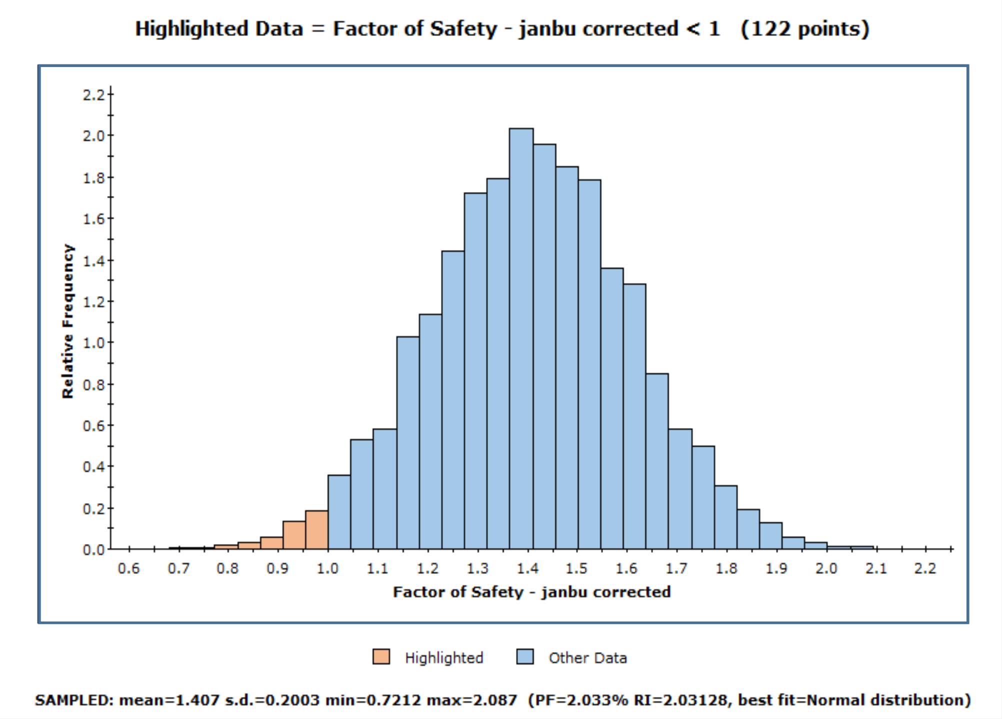


Probability FS ≤ 1.0 = 22.4%

Section S5_NE3&NE6 – 10 years –break-back: 100m

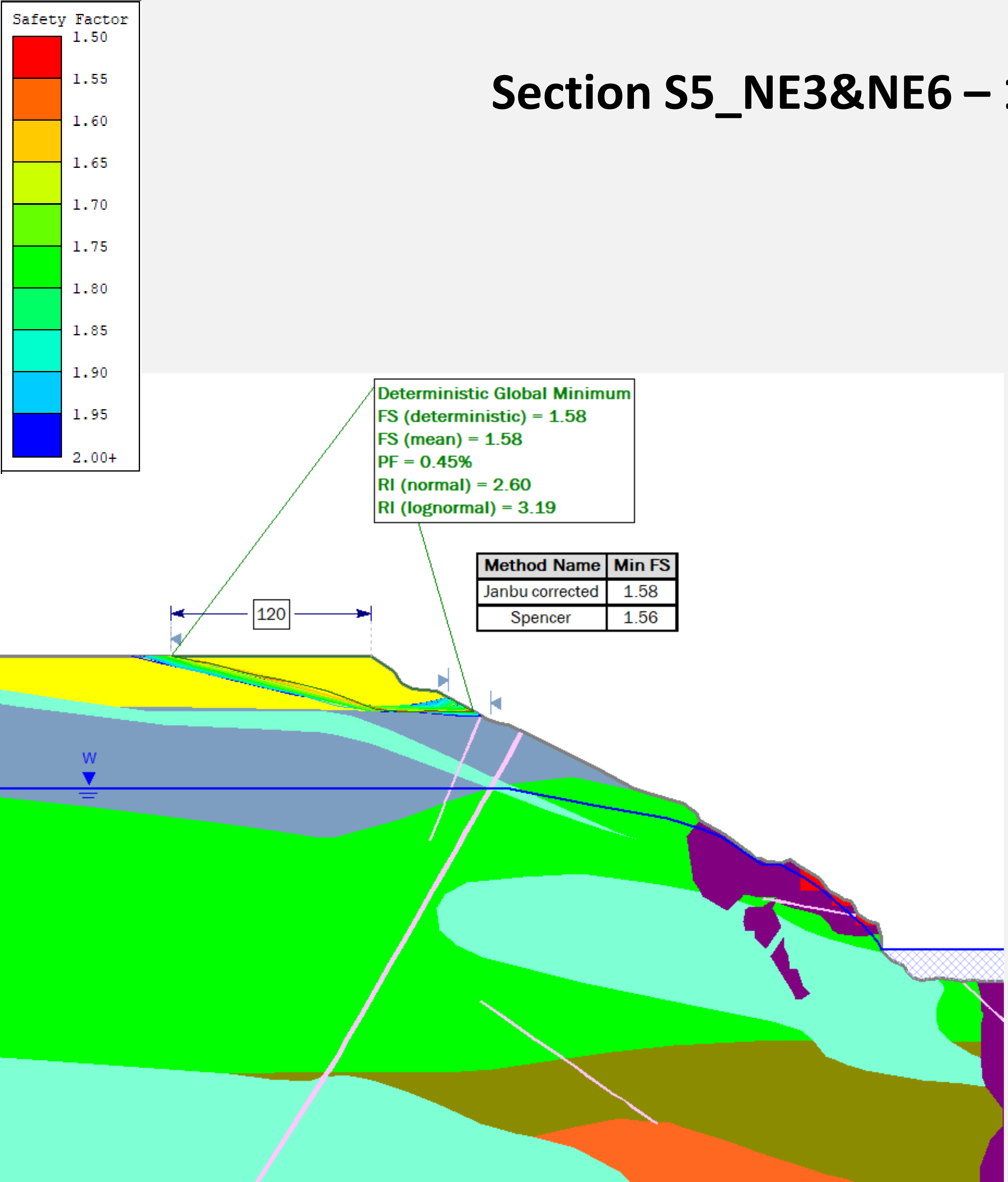




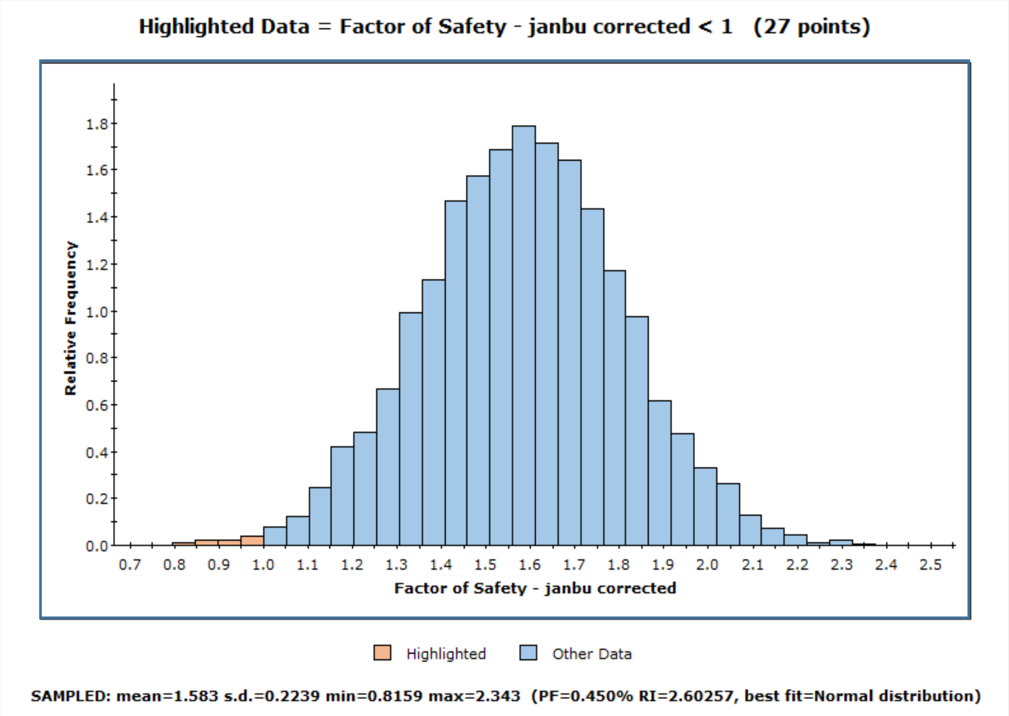


Probability FS ≤ 1.0 = 2.0%

Section S5_NE3&NE6 – 10 years –break-back: 120m

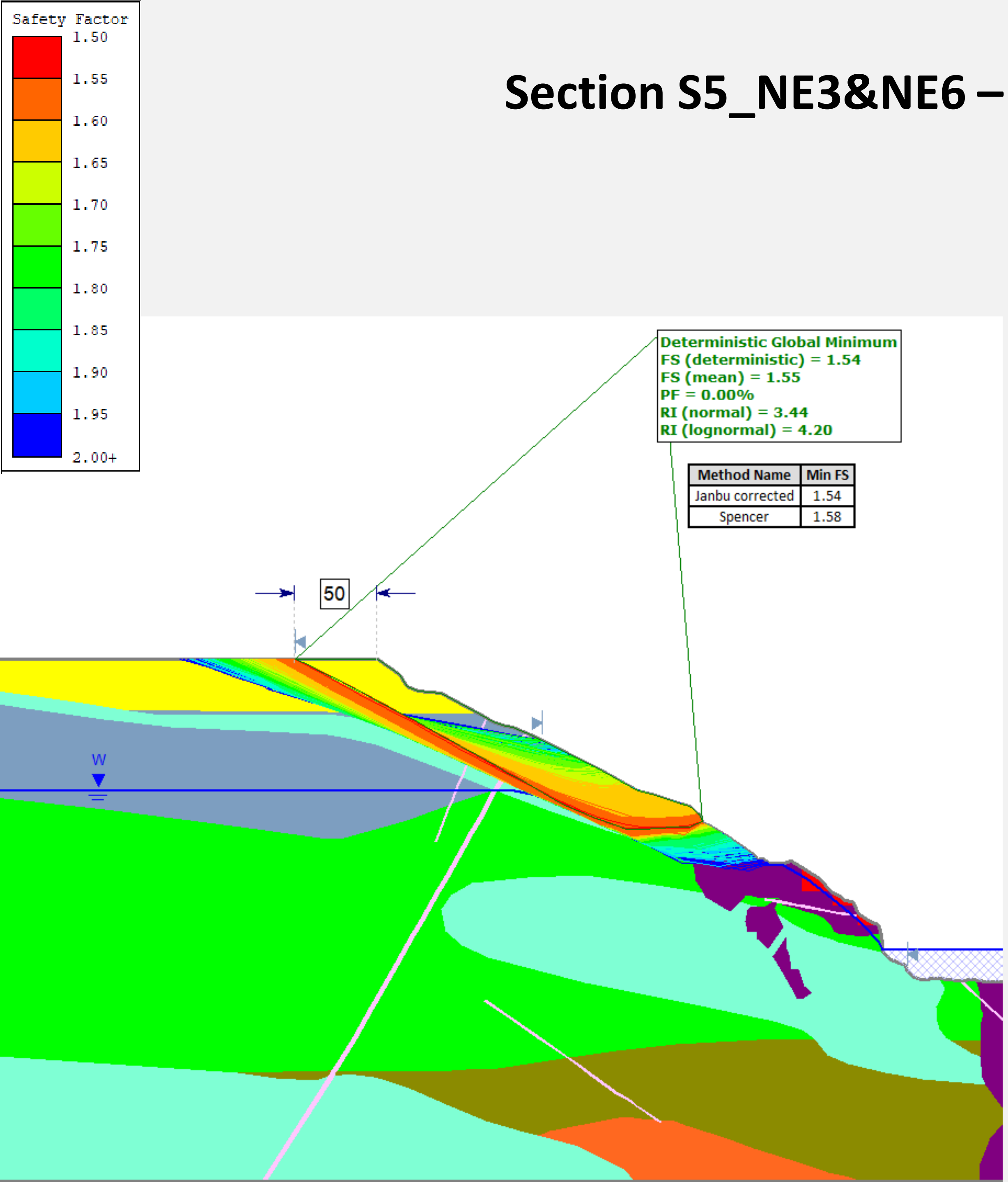


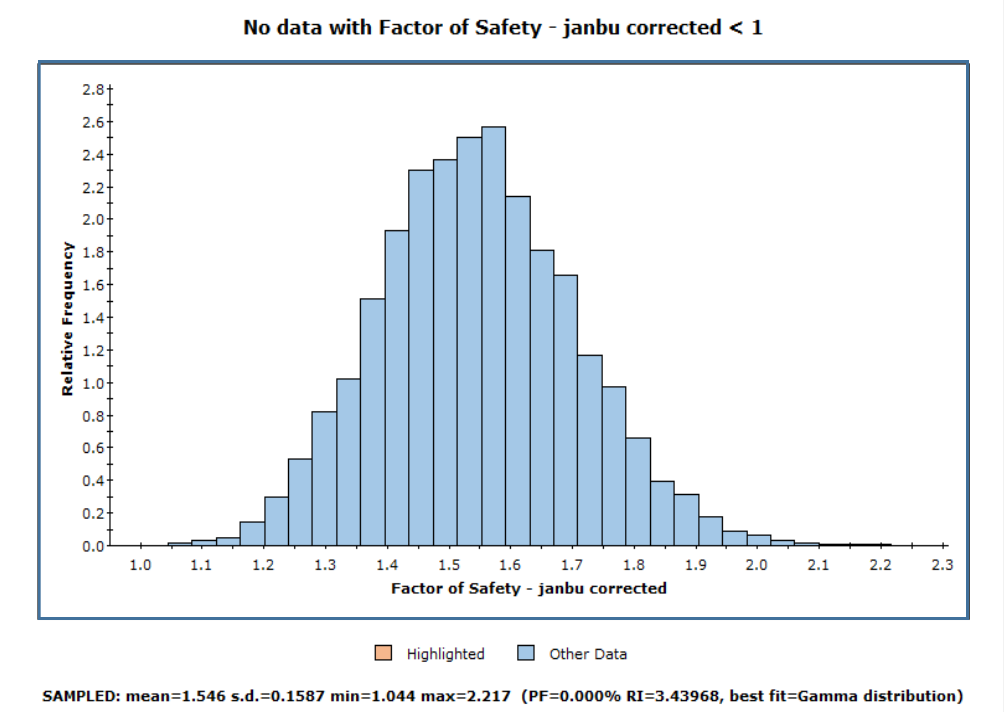
Method Name	Min FS
Janbu corrected	1.58
Spencer	1.56



Probability FS ≤ 1.0 = 0.5%

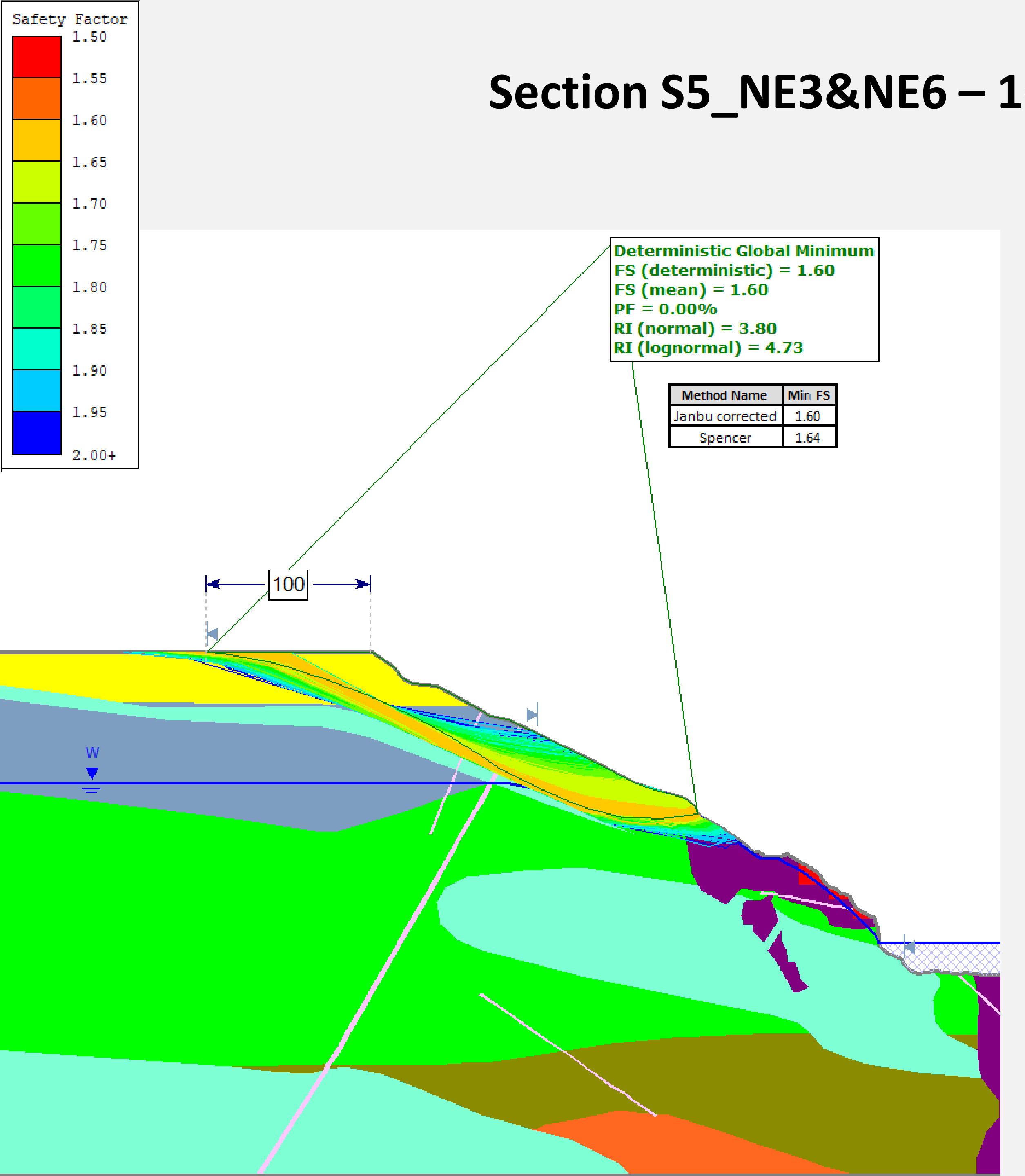
Section S5_NE3&NE6 – 10 years –break-back: 50m

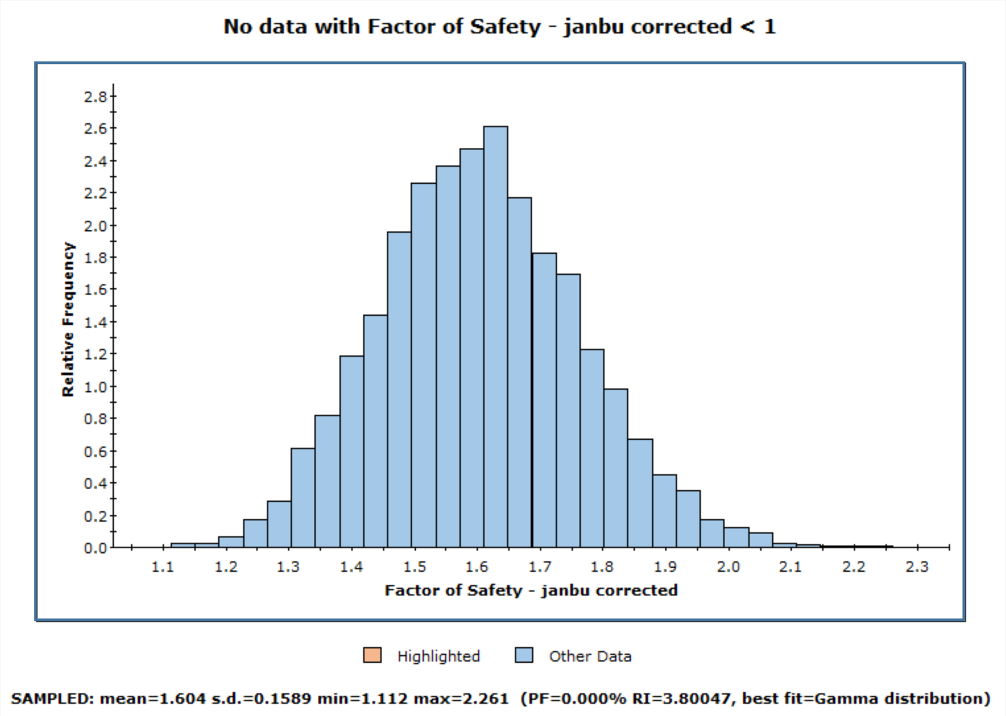




Probability FS ≤ 1.0 = 0.0%

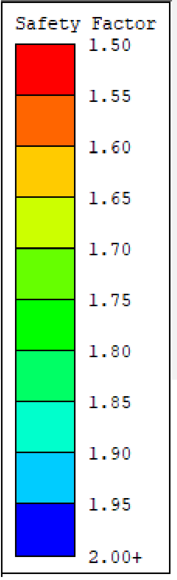
Section S5_NE3&NE6 – 10 years –break-back: 100m

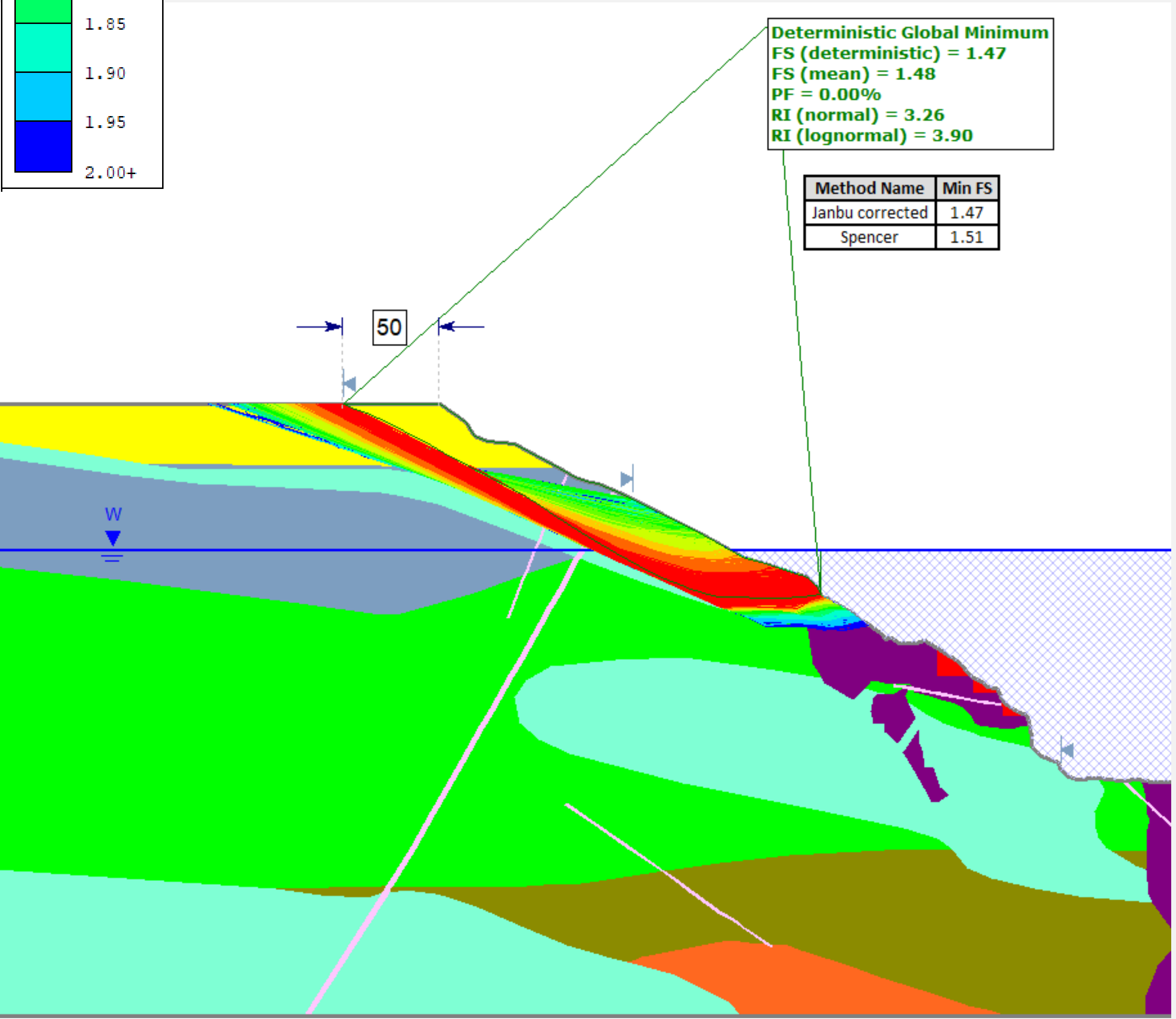


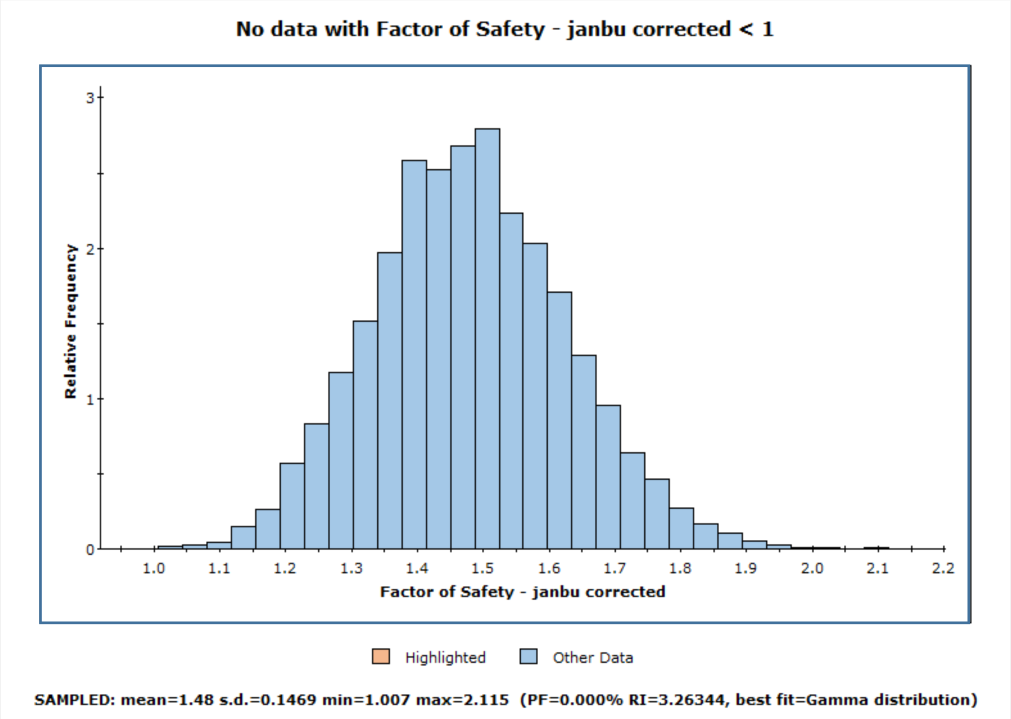


Probability FS ≤ 1.0 = 0.0%

Section S5_NE3&NE6 – 200 years –break-back: 50m

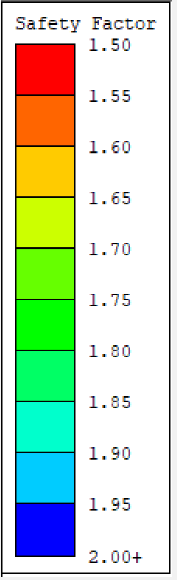




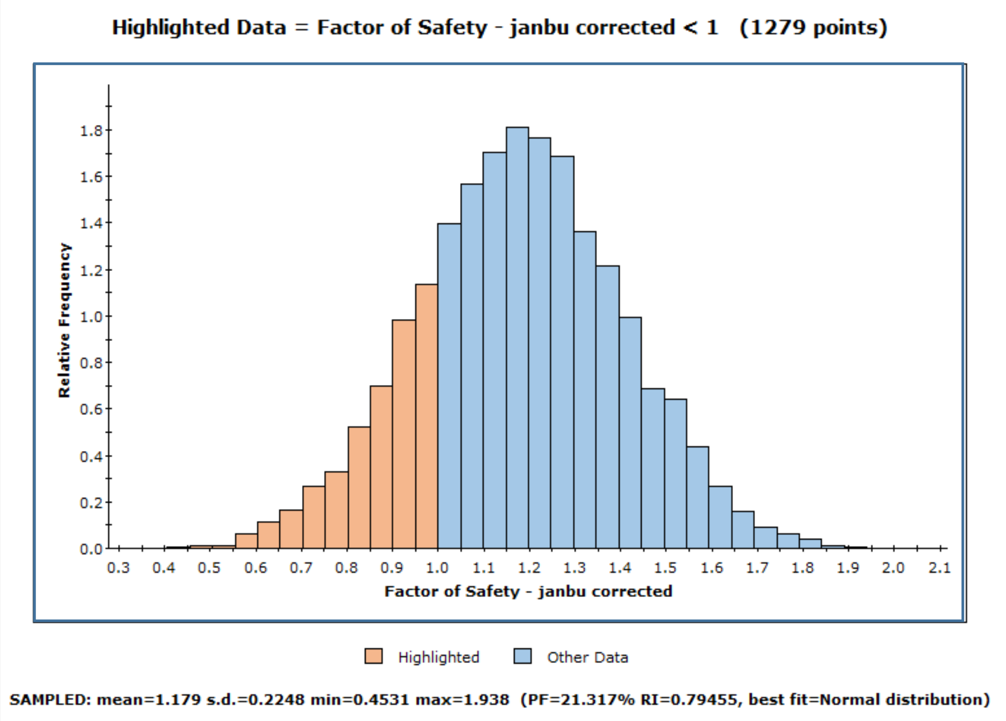


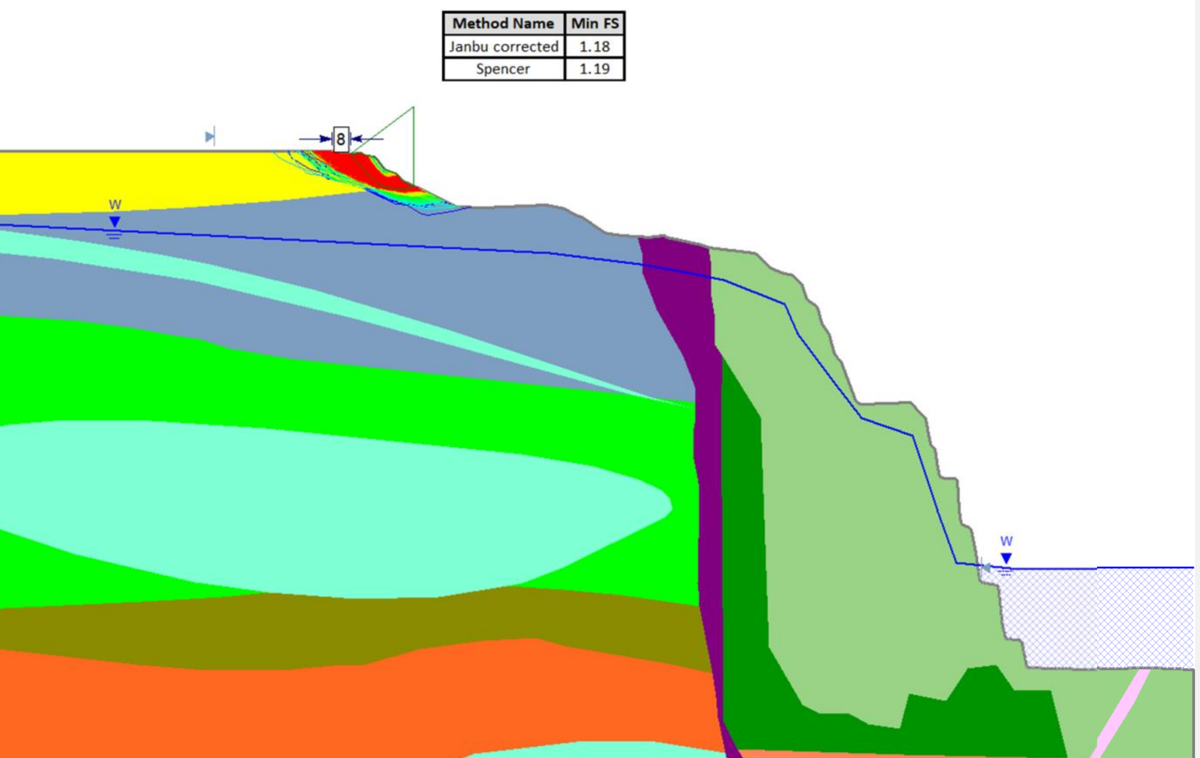
Probability FS ≤ 1.0 = 0.0%

Section S6_Basalt Raft



Section S6_Basalt Raft – 10 years –break-back: -8m

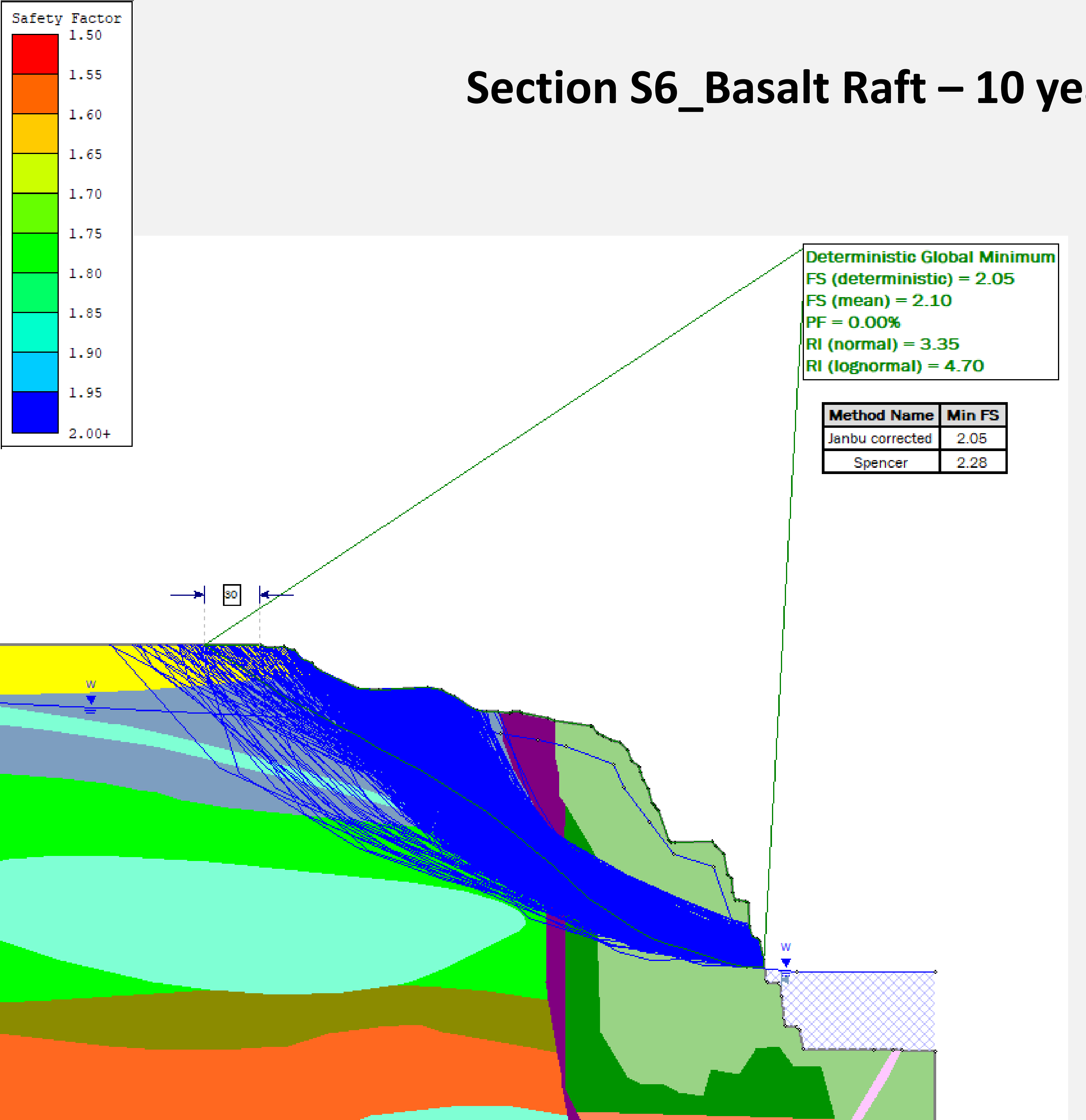




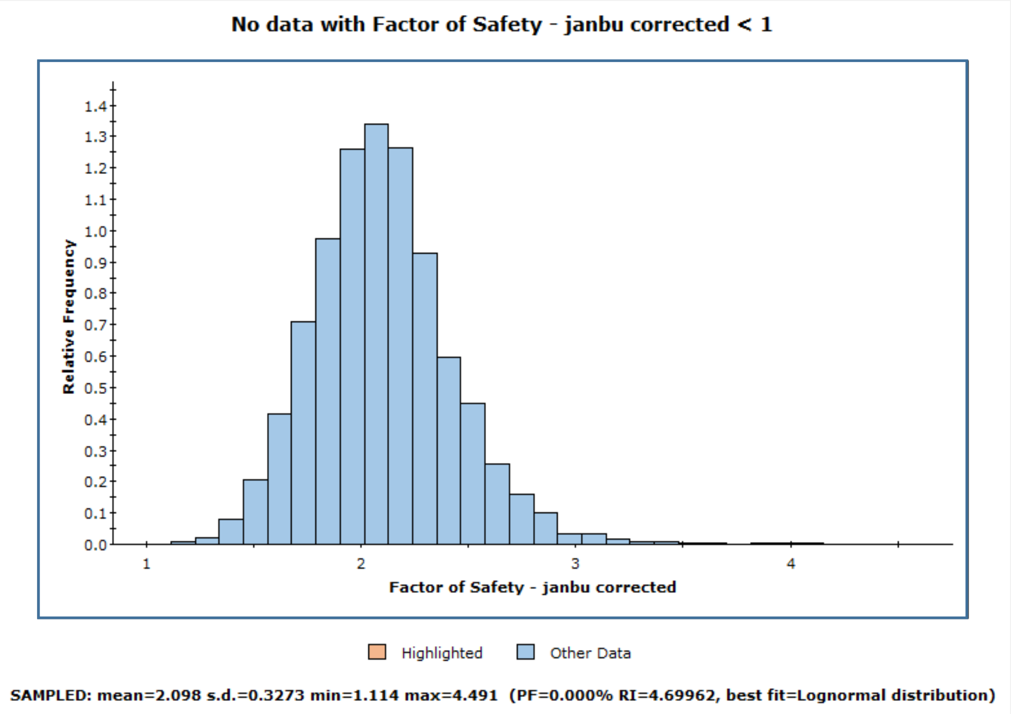
Method Name	Min FS
Janbu corrected	1.18
Spencer	1.19

Probability FS ≤ 1.0 = 21.3%

Section S6_Basalt Raft – 10 years –break-back: 30m

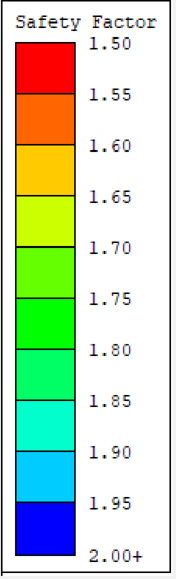


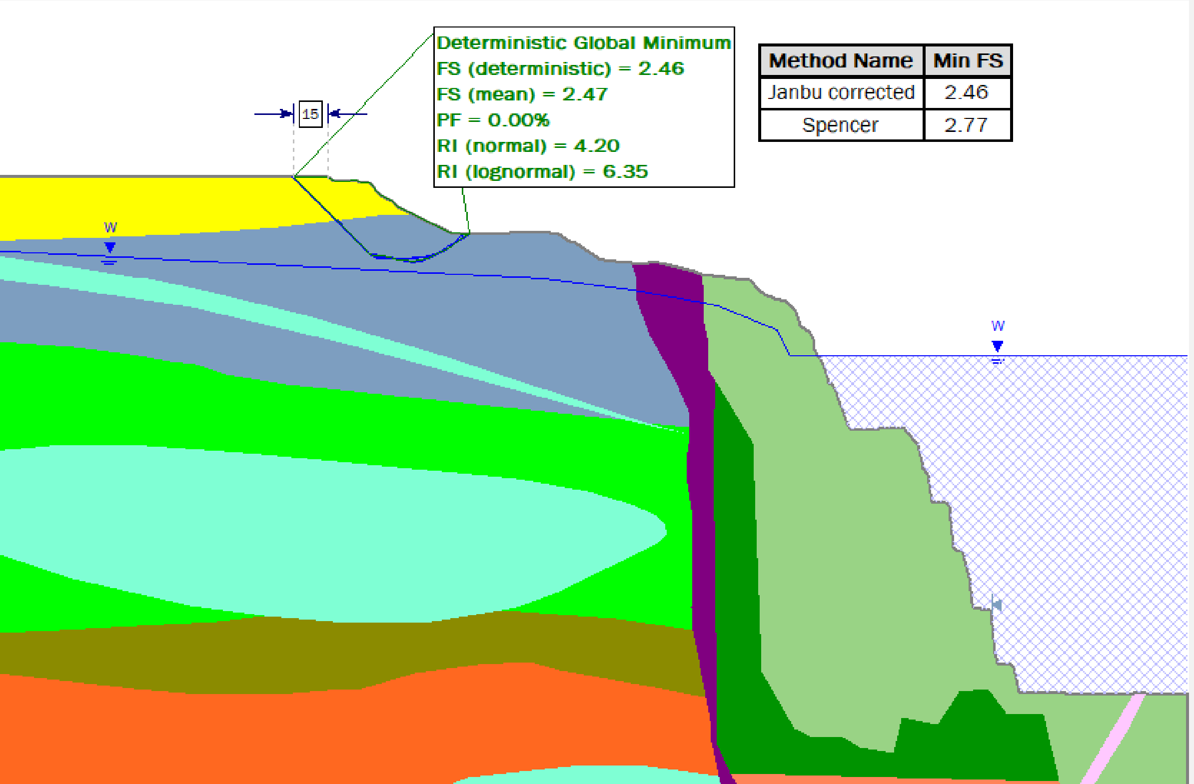
Method Name	Min FS
Janbu corrected	2.05
Spencer	2.28



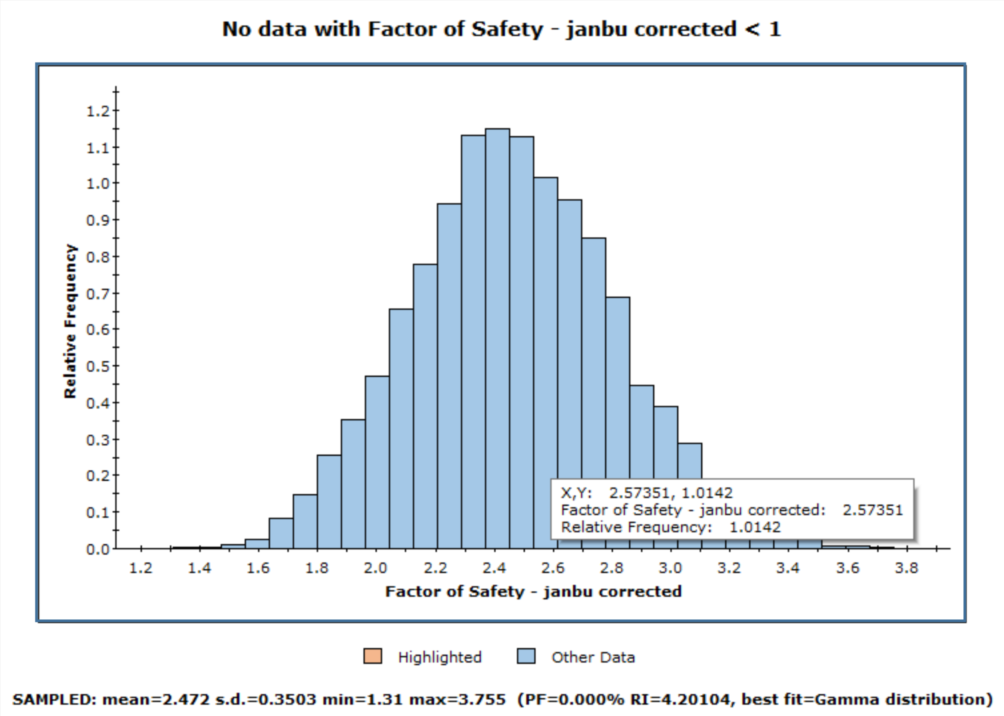
Probability FS ≤ 1.0 = 0.0%

Section S6_Basalt Raft – 200 years –break-back: 15m ovl



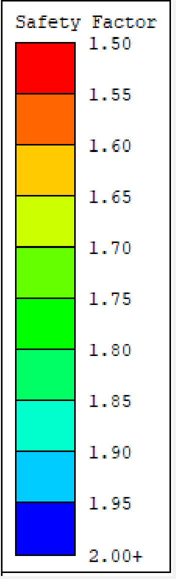


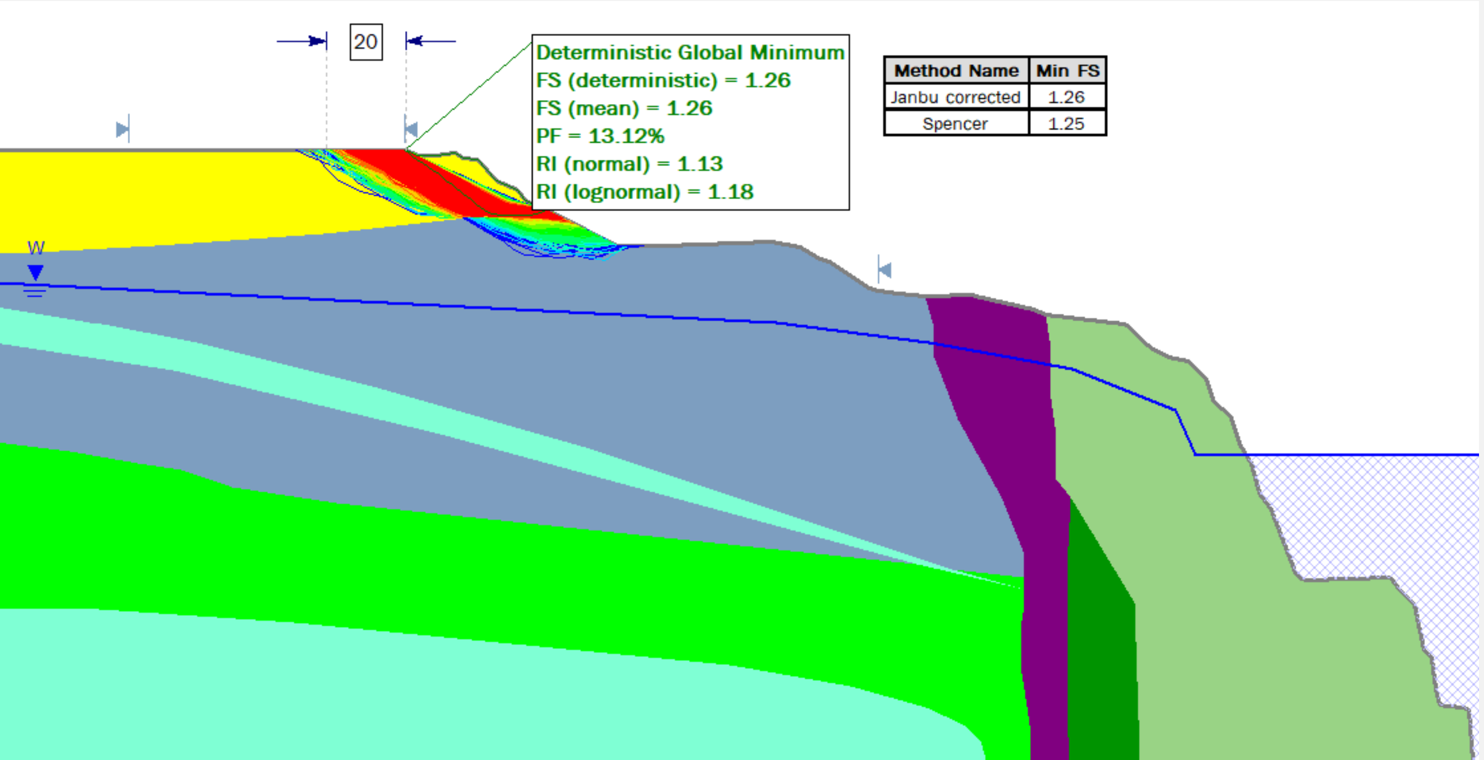
Method Name	Min FS
Janbu corrected	2.46
Spencer	2.77

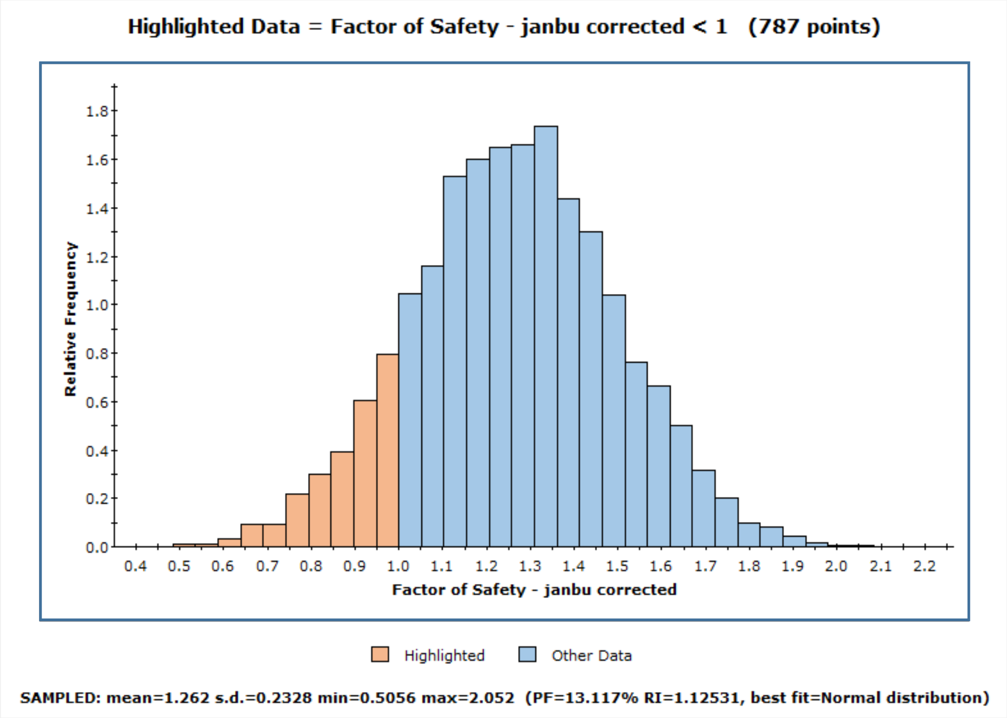


Probability FS ≤ 1.0 = 0.0%

Section S6_Basalt Raft – 200 years –break-back: 0m

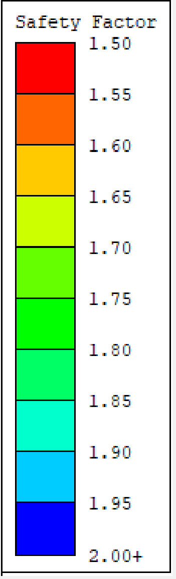


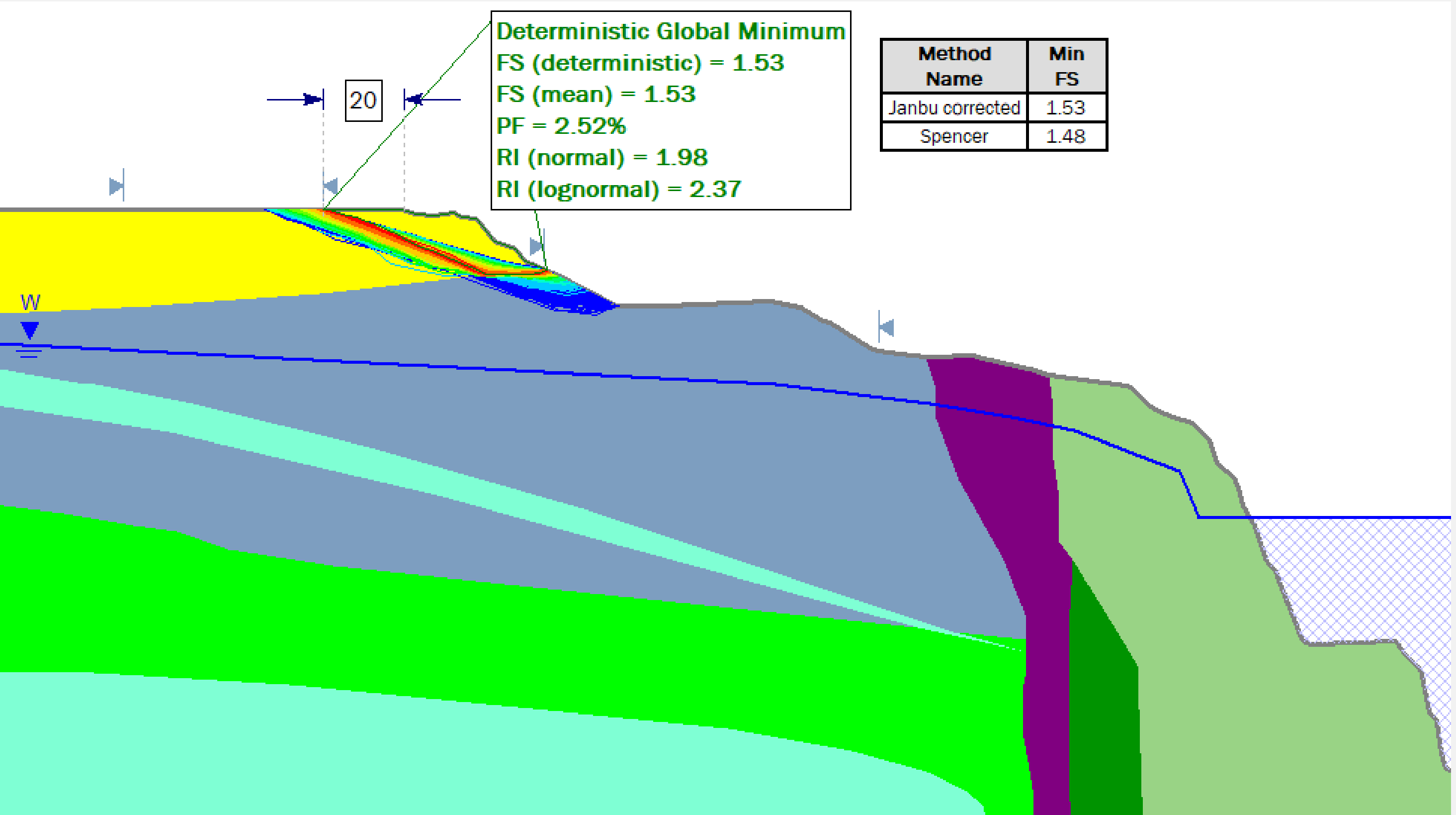


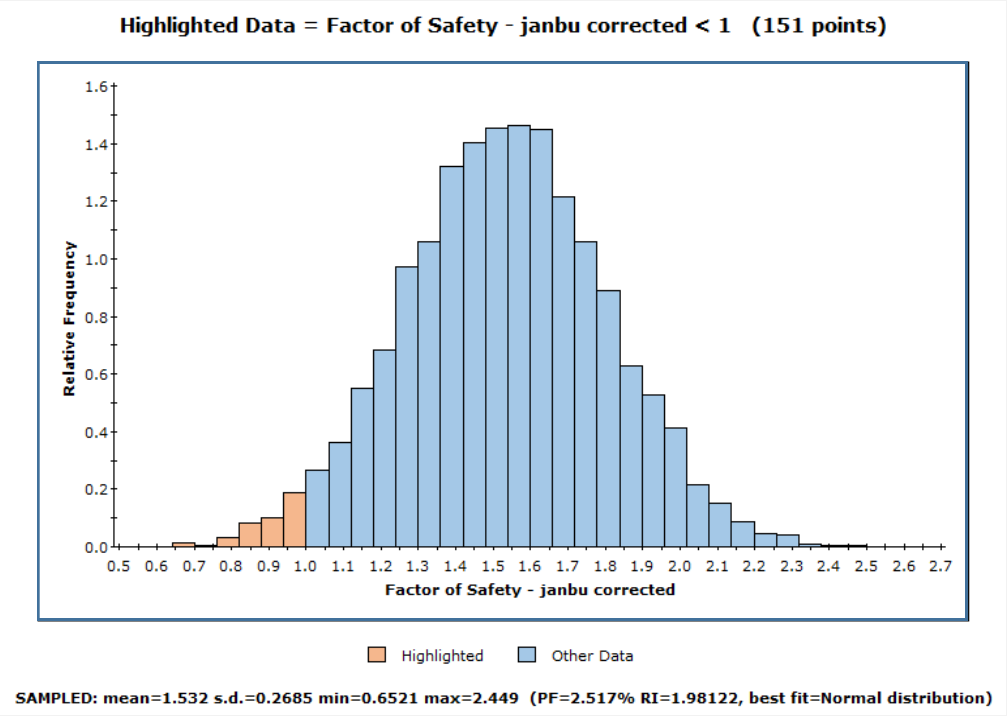


Probability FS ≤ 1.0 = 13.1%

Section S6_Basalt Raft – 200 years –break-back: 20m

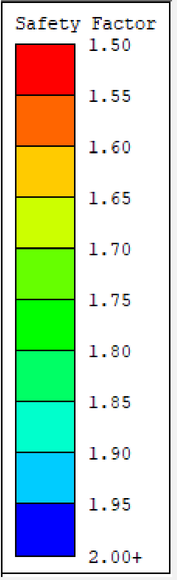


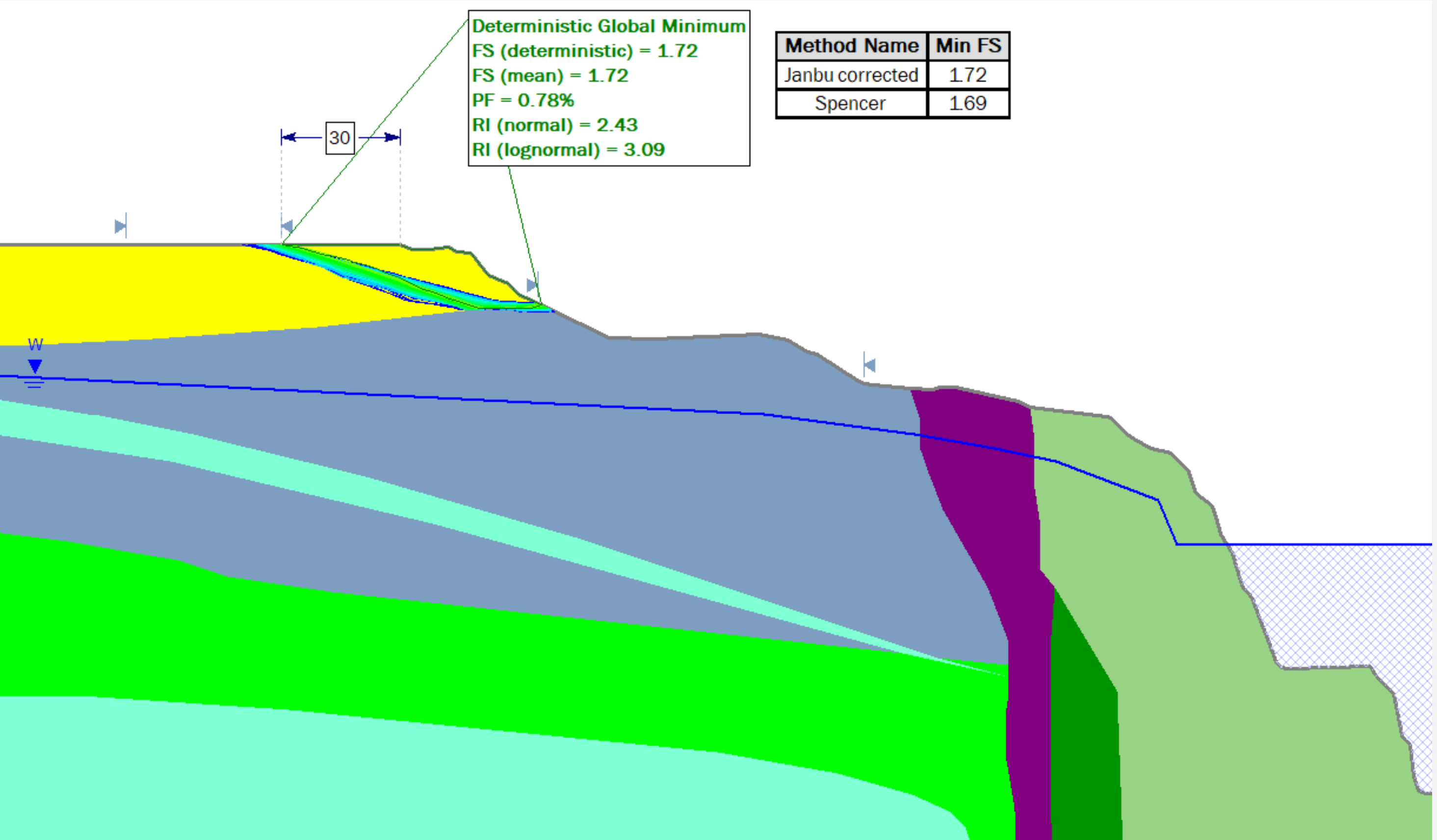


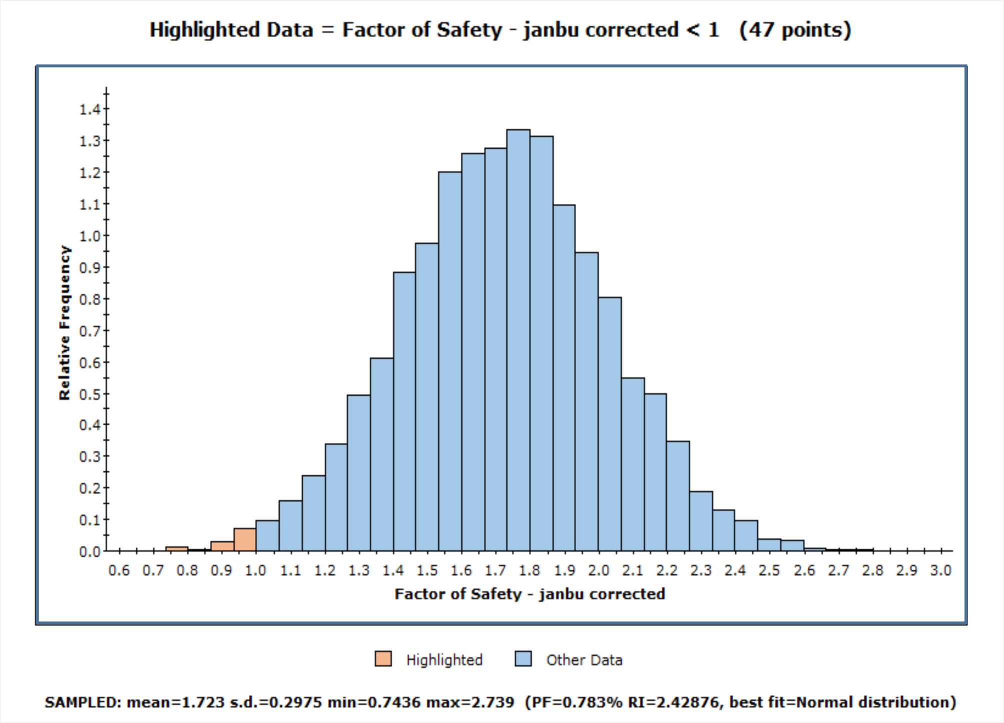


Probability FS ≤ 1.0 = 2.5%

Section S6_Basalt Raft – 200 years –break-back: 30m

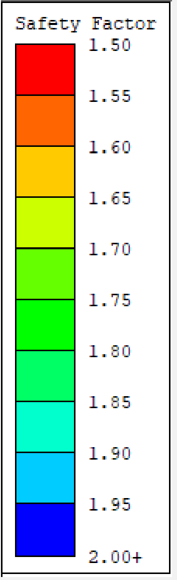




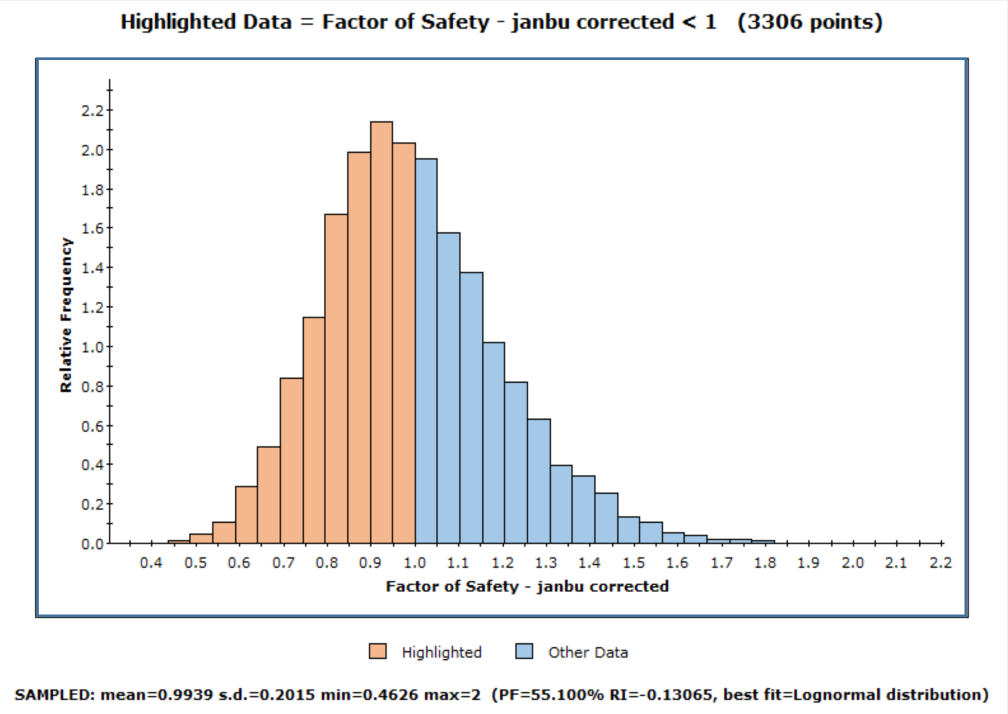


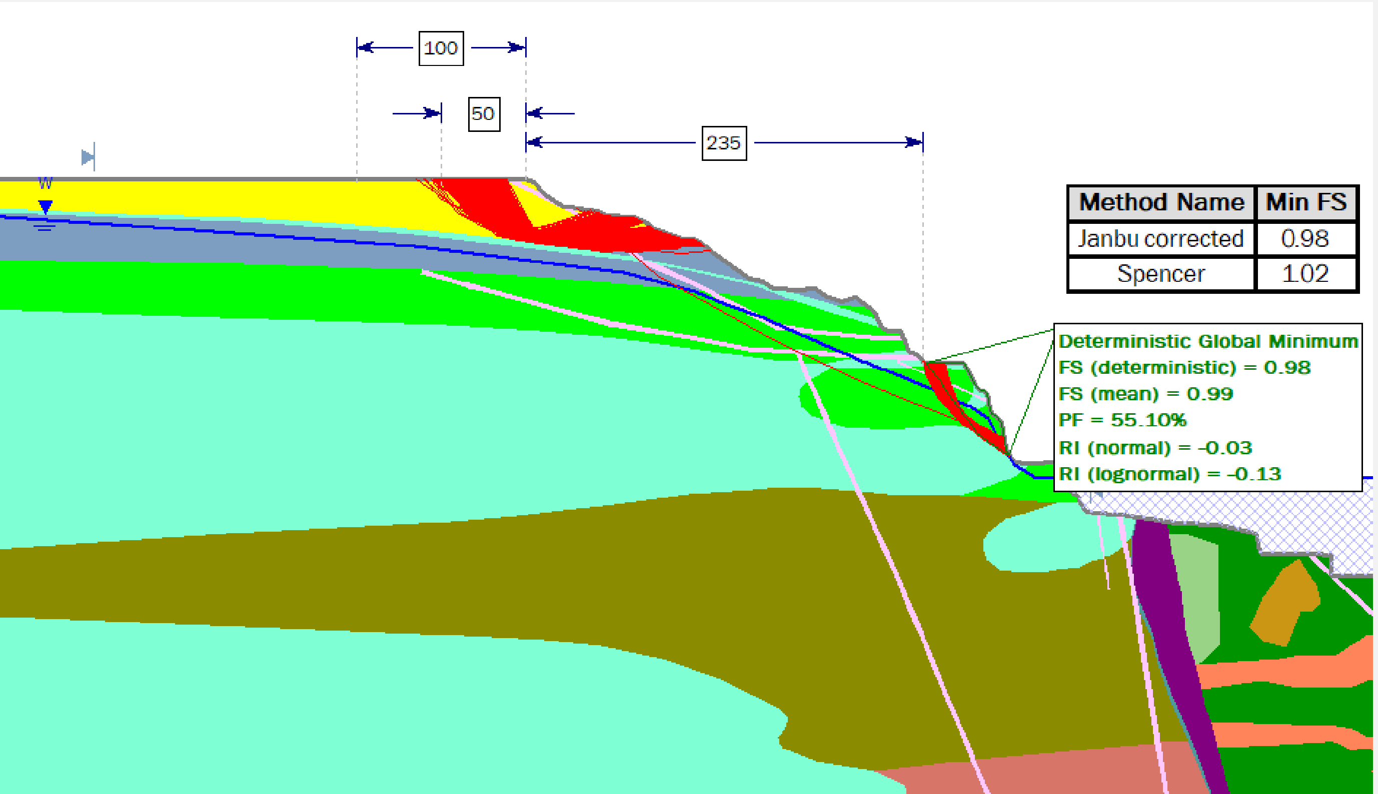
Probability FS ≤ 1.0 = 0.8%

Section S7_N11&N12



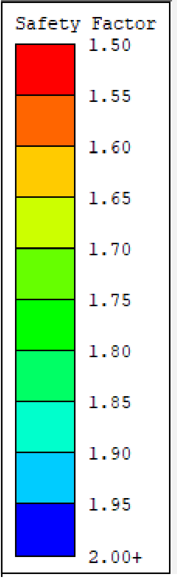
Section S7_N11&N12 – 10 years –break-back: -235m

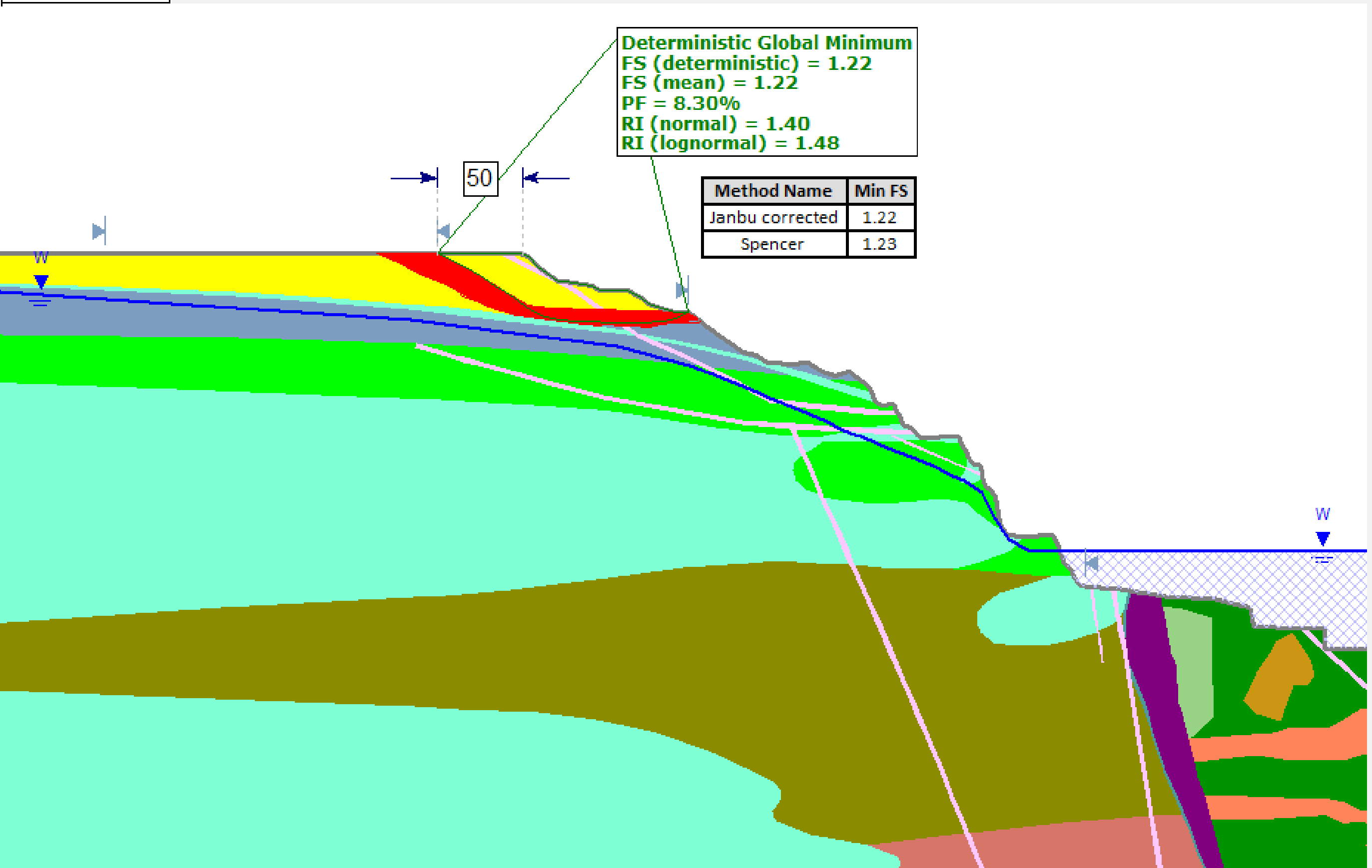


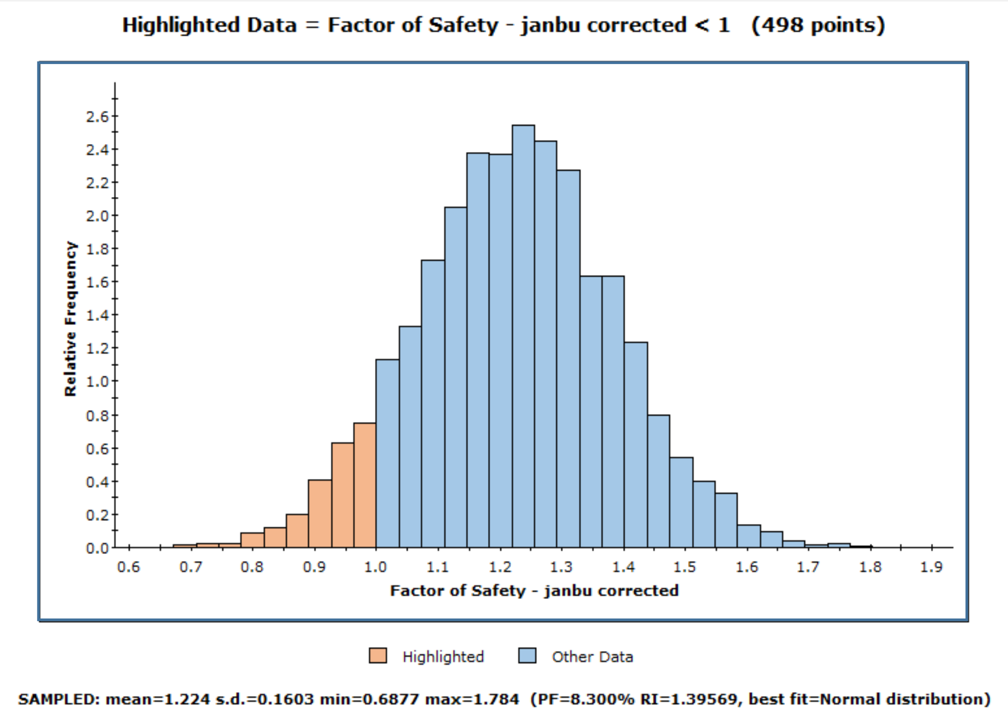


Probability FS ≤ 1 = 55.1%

Section S7_N11&N12 – 10 years –break-back: 50m

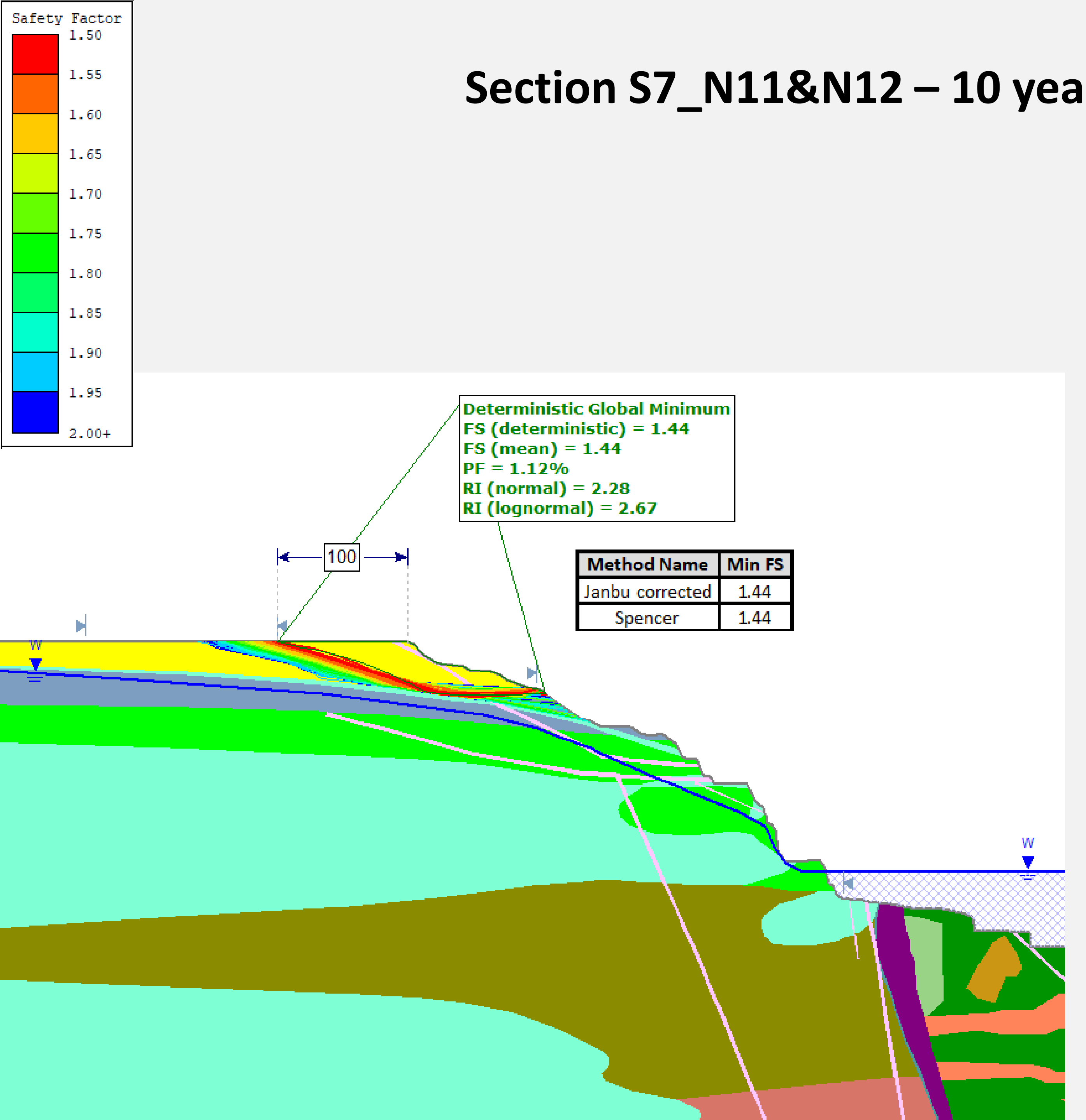




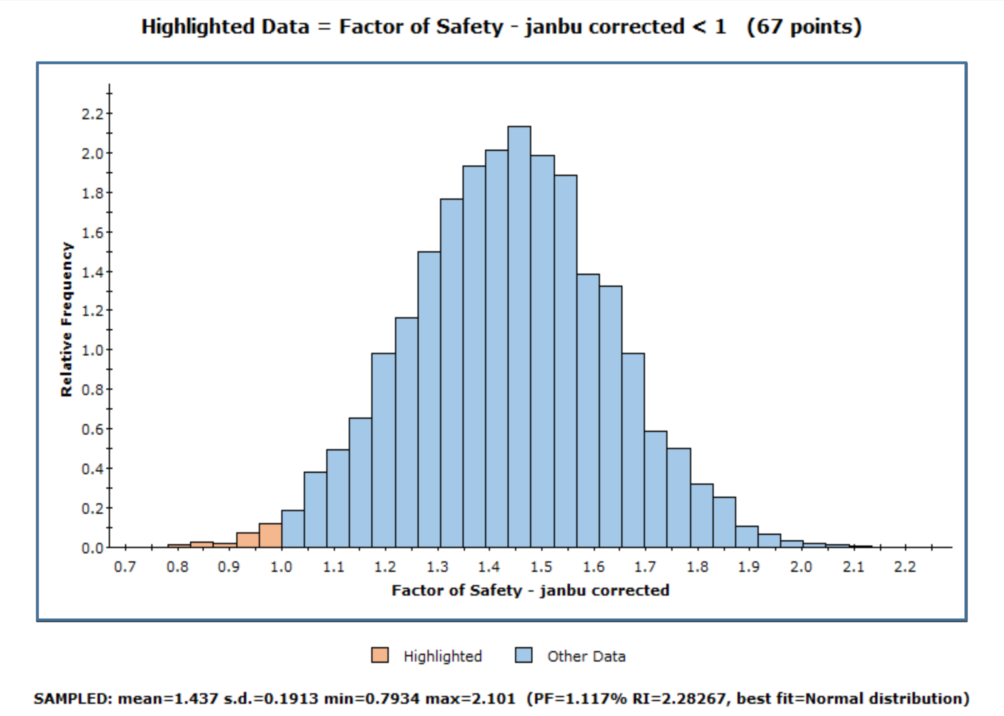


Probability FS ≤ 1 = 8.3%

Section S7_N11&N12 – 10 years –break-back: 100m

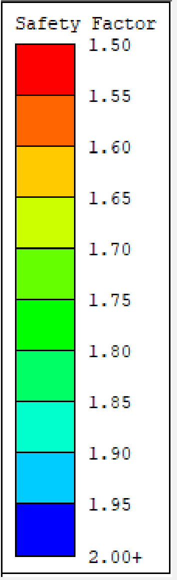


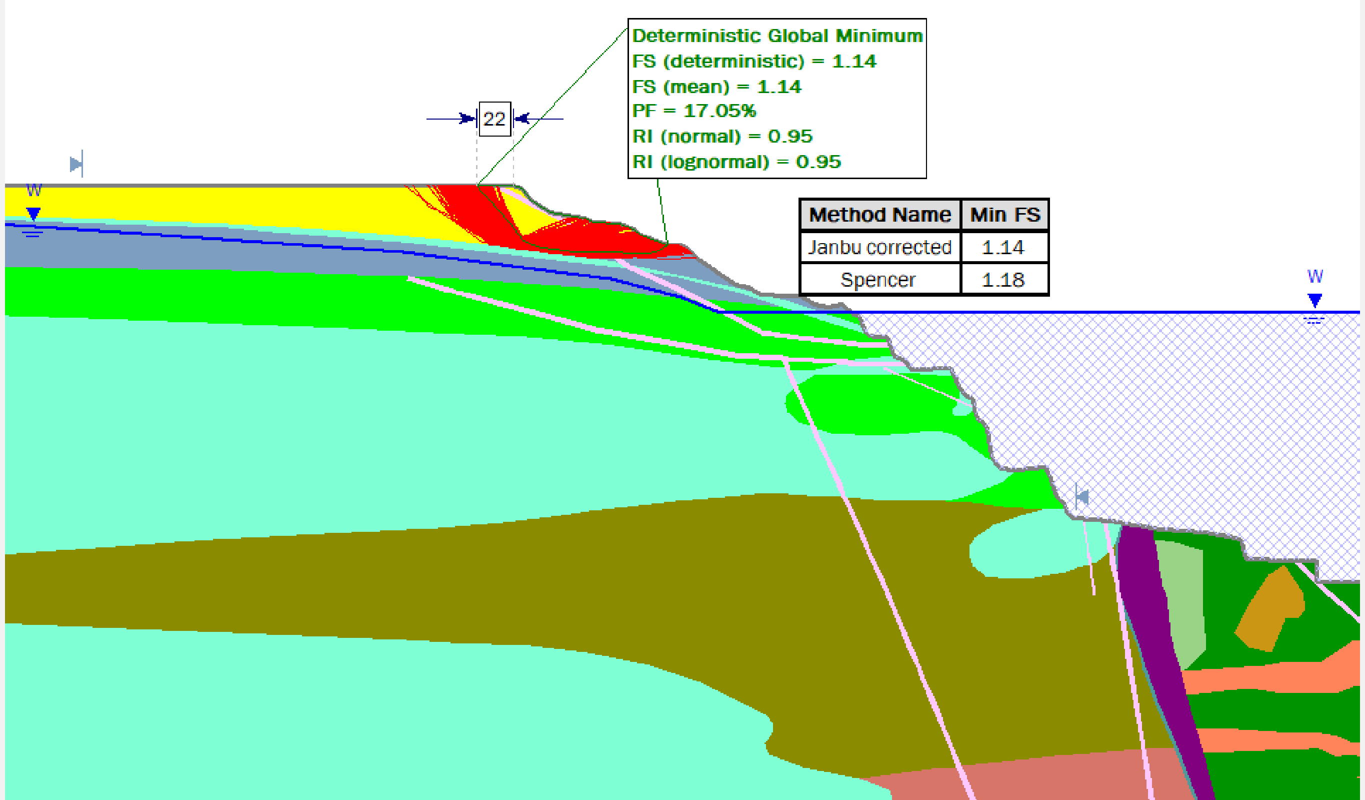
Method Name	Min FS
Janbu corrected	1.44
Spencer	1.44

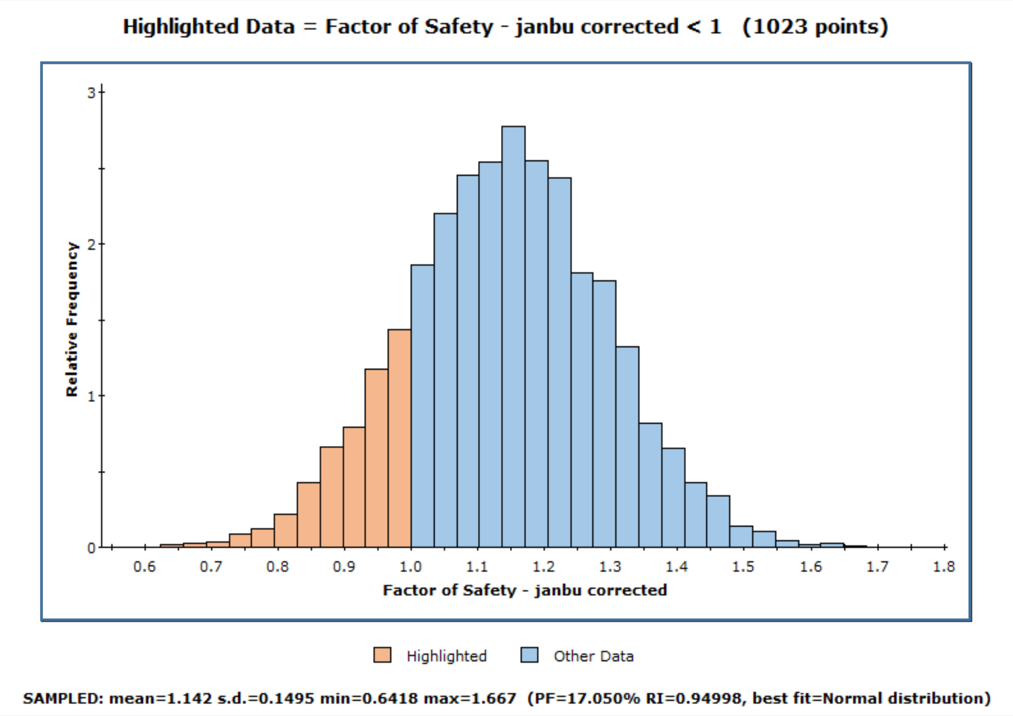


Probability FS ≤ 1 = 1.1%

Section S7_N11&N12 – 200 years –break-back: 22m



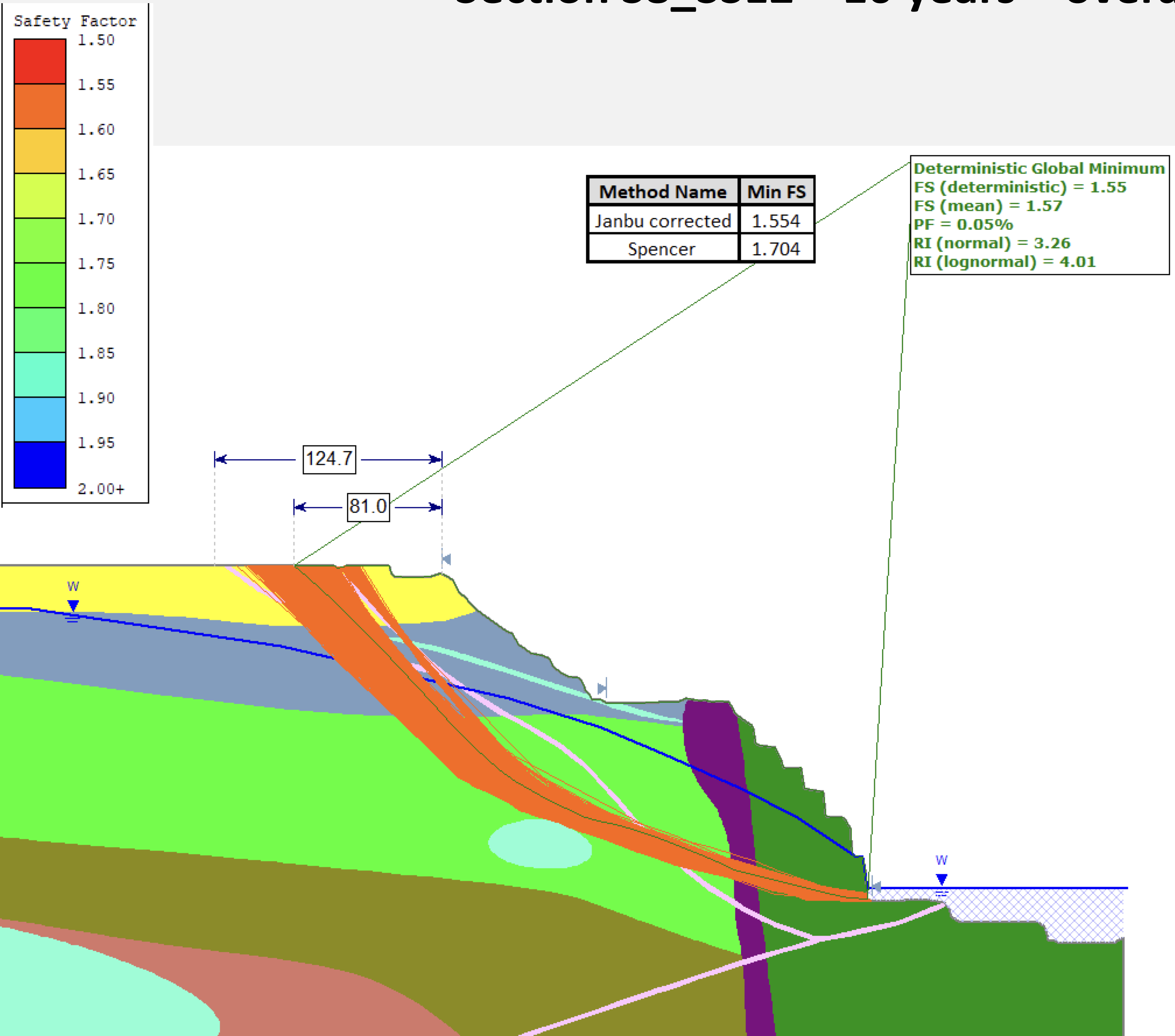




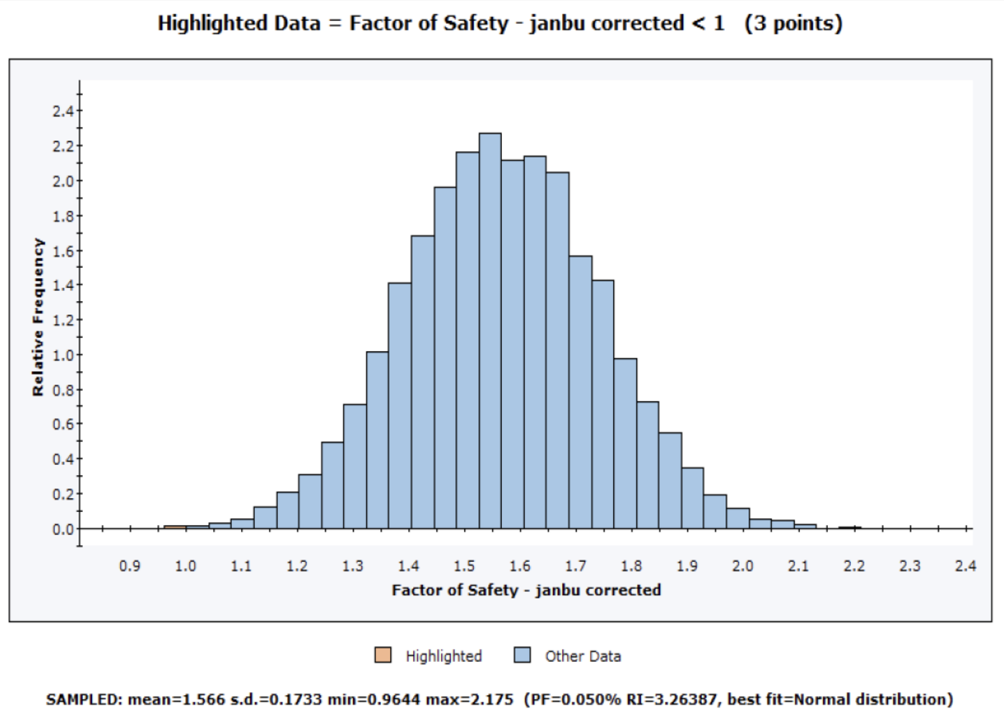
Probability FS ≤ 1 = 17.1%

Section S8_SSE2

Section S8_SSE2 – 10 years – overall break-back: 81m

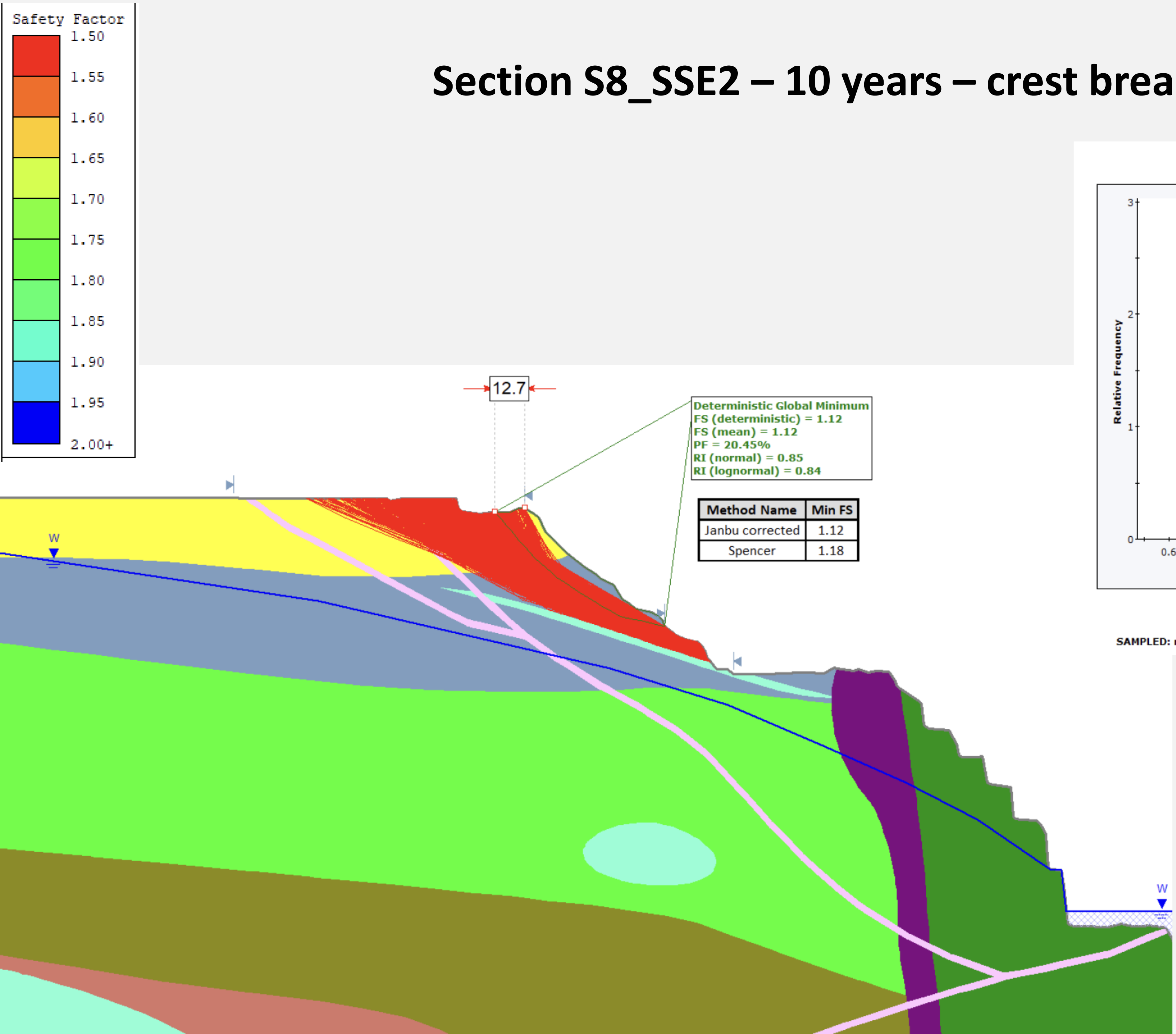


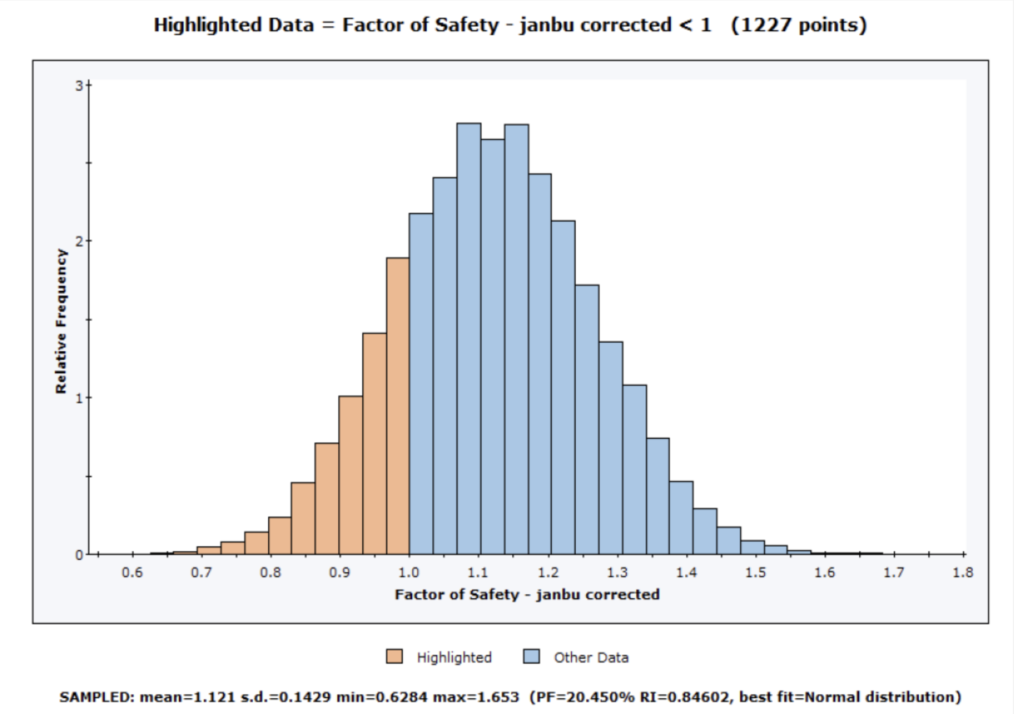
Method Name	Min FS
Janbu corrected	1.554
Spencer	1.704



Probability FS ≤ 1 = 0.1%

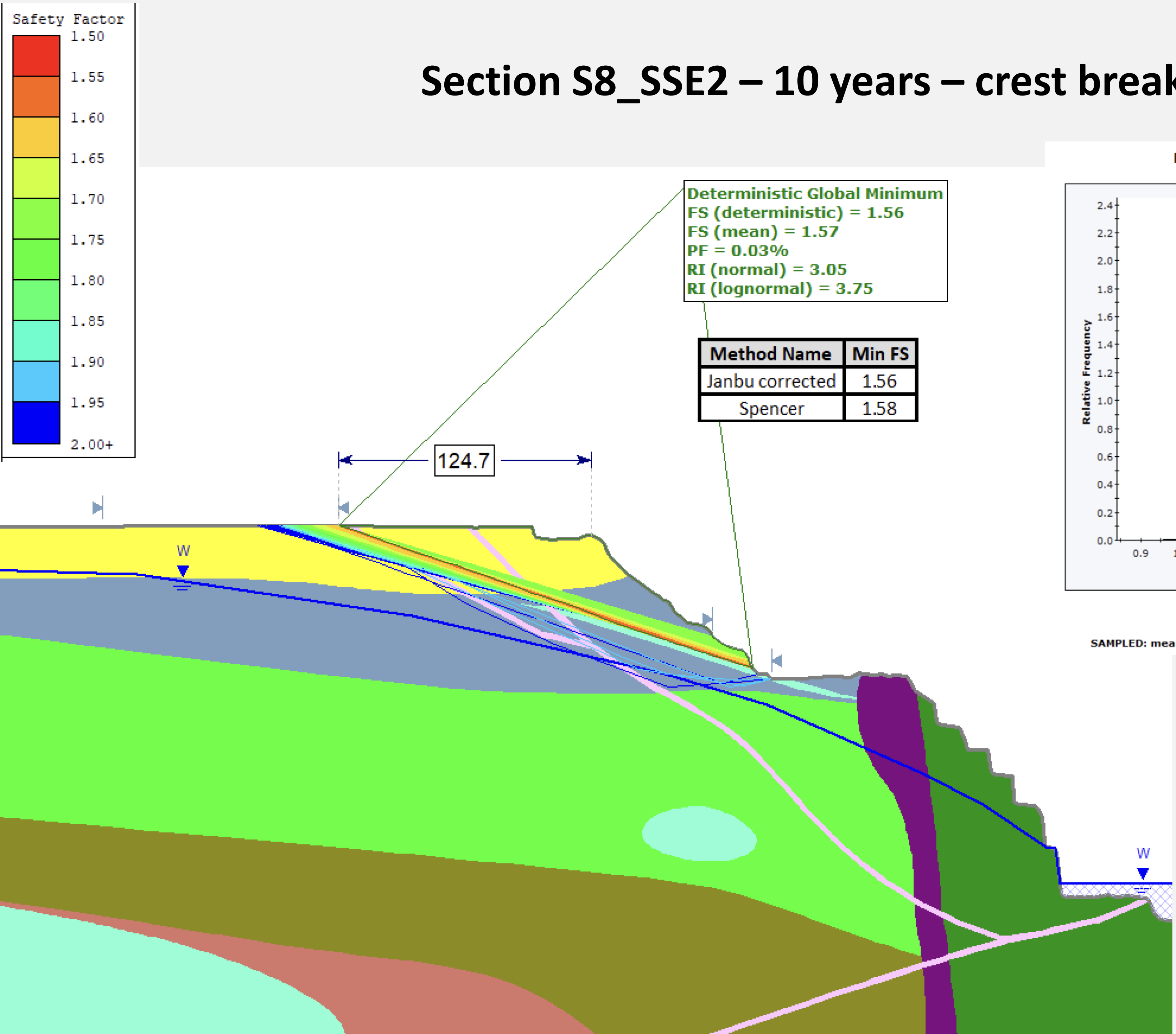
Section S8_SSE2 – 10 years – crest break-back: 13m



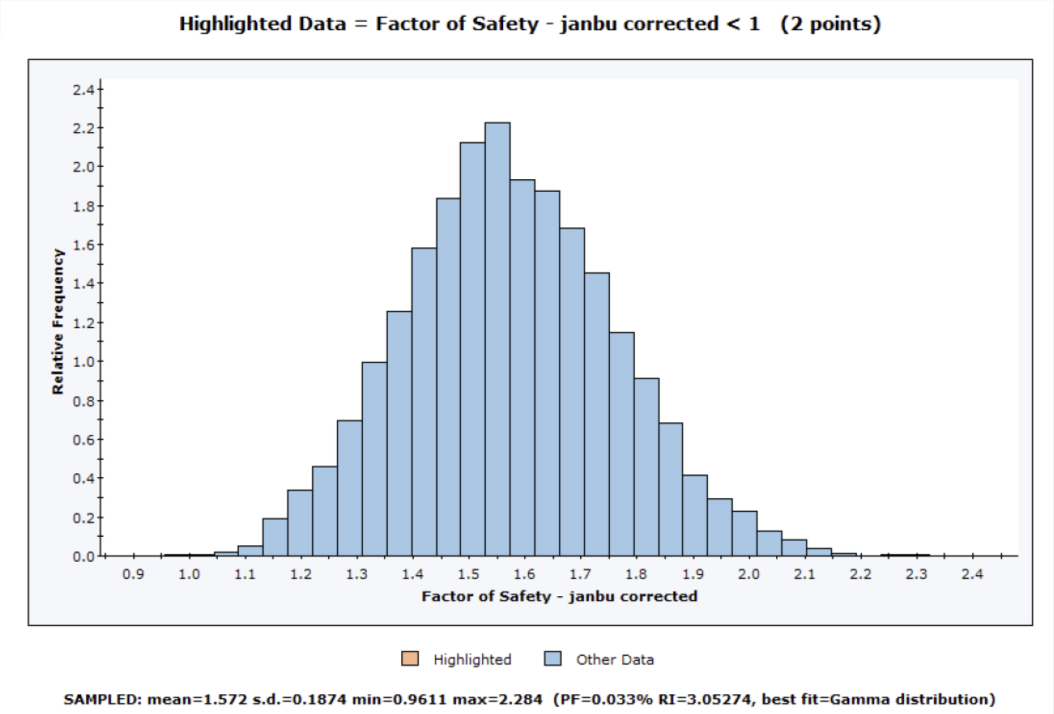


Probability FS ≤ 1 = 20.5%

Section S8_SSE2 – 10 years – crest break-back: 125m



Method Name	Min FS
Janbu corrected	1.56
Spencer	1.58

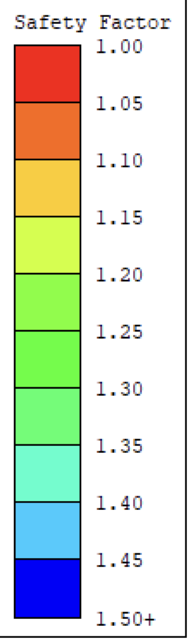


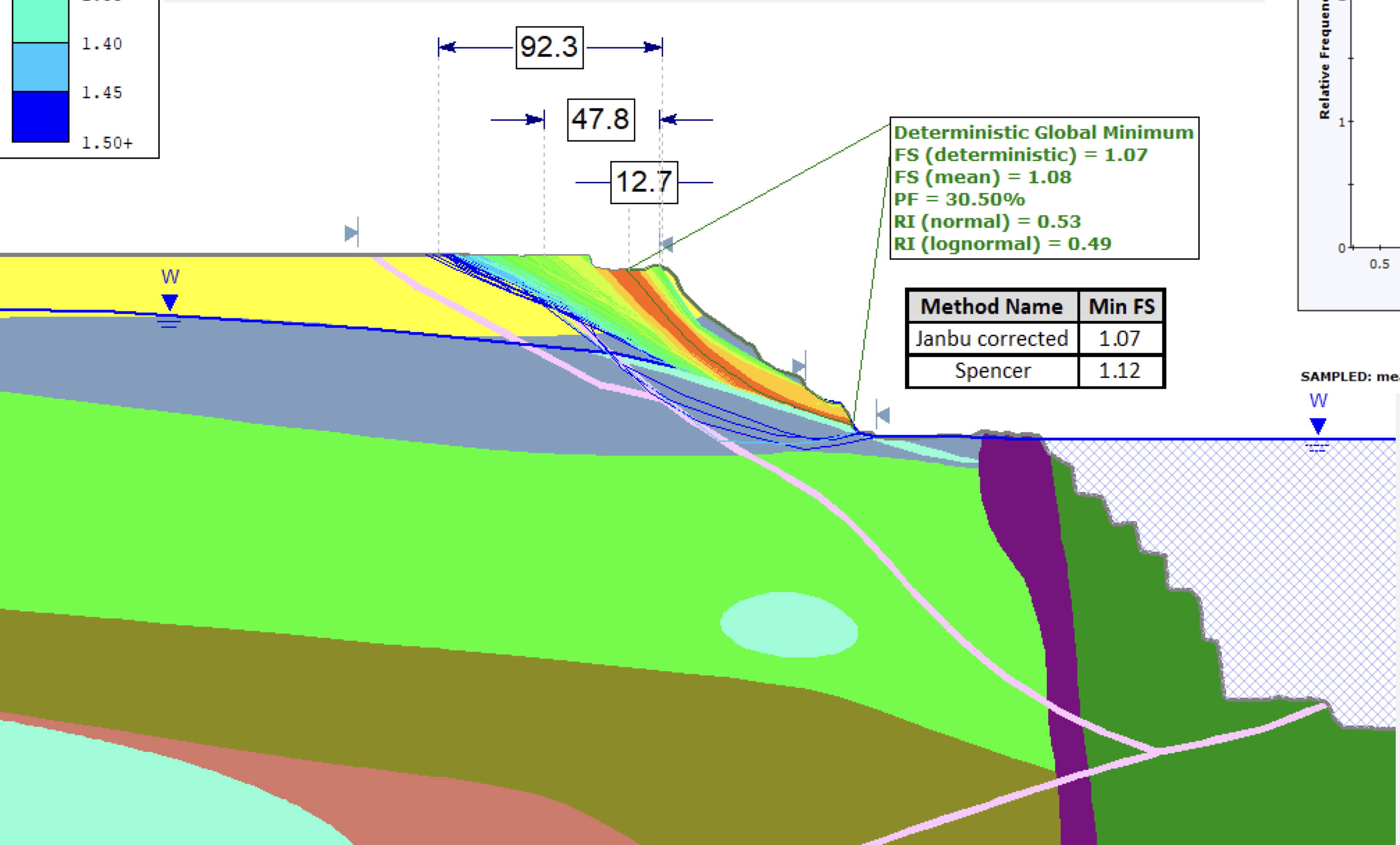
Probability FS ≤ 1 = 0.0%

APPENDIX D: SPECIAL CASES - SLOPE STABILITY ANALYSIS RESULTS

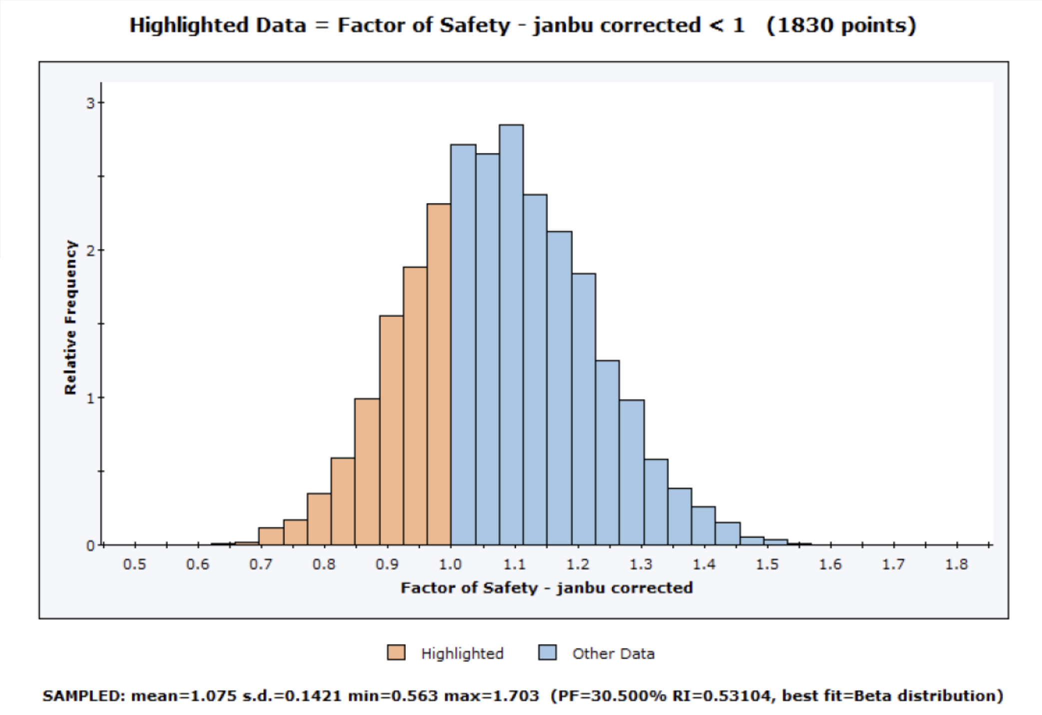
Section S8_SSE2
Special Conditions

Section S8_SSE2 – 200 years – base case revised wl - BB: 13m



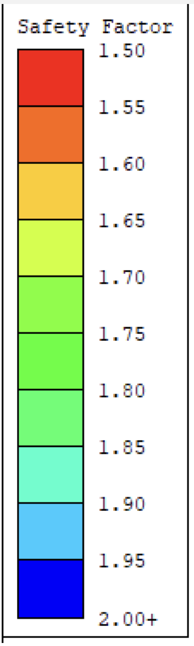


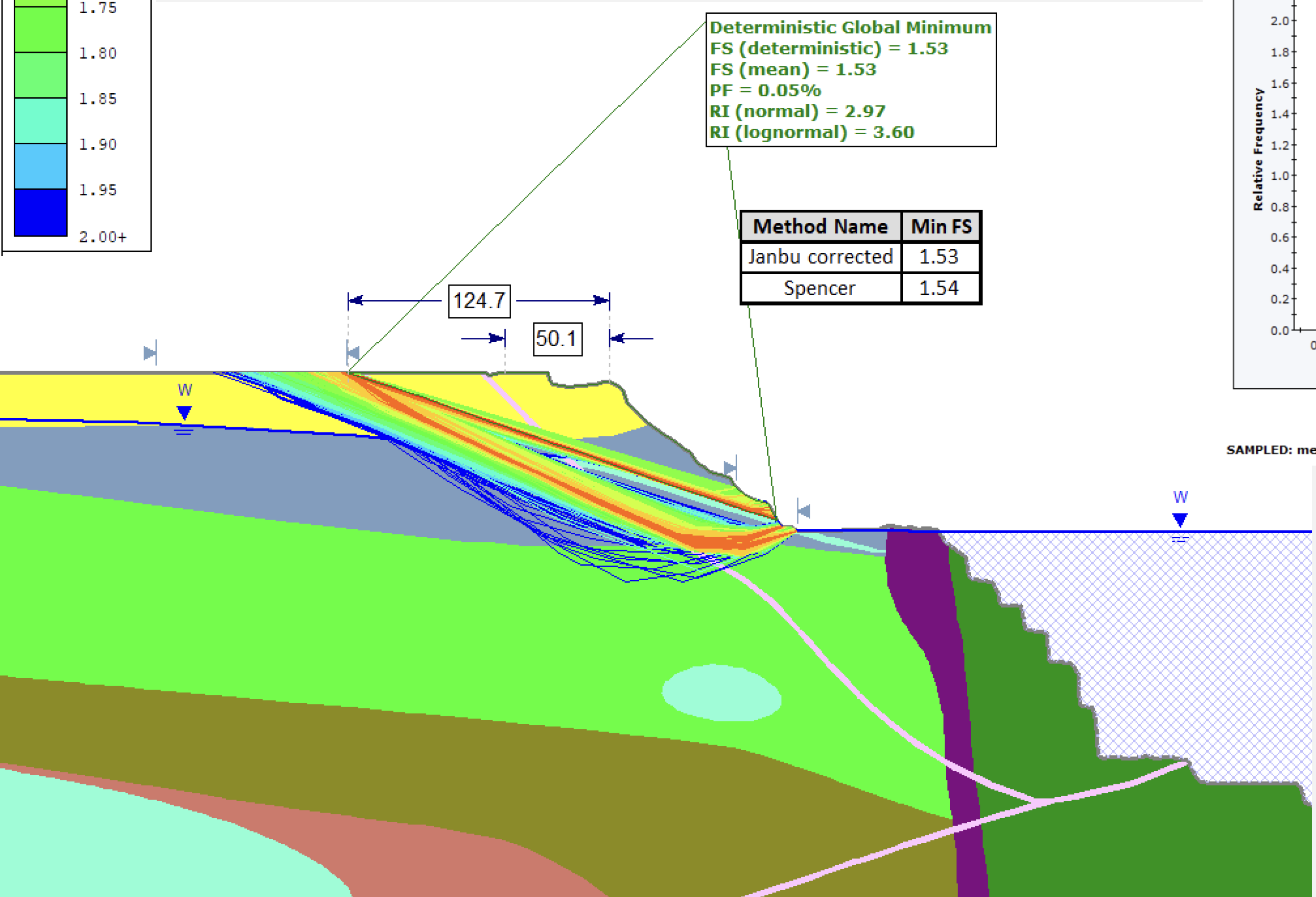
Method Name	Min FS
Janbu corrected	1.07
Spencer	1.12



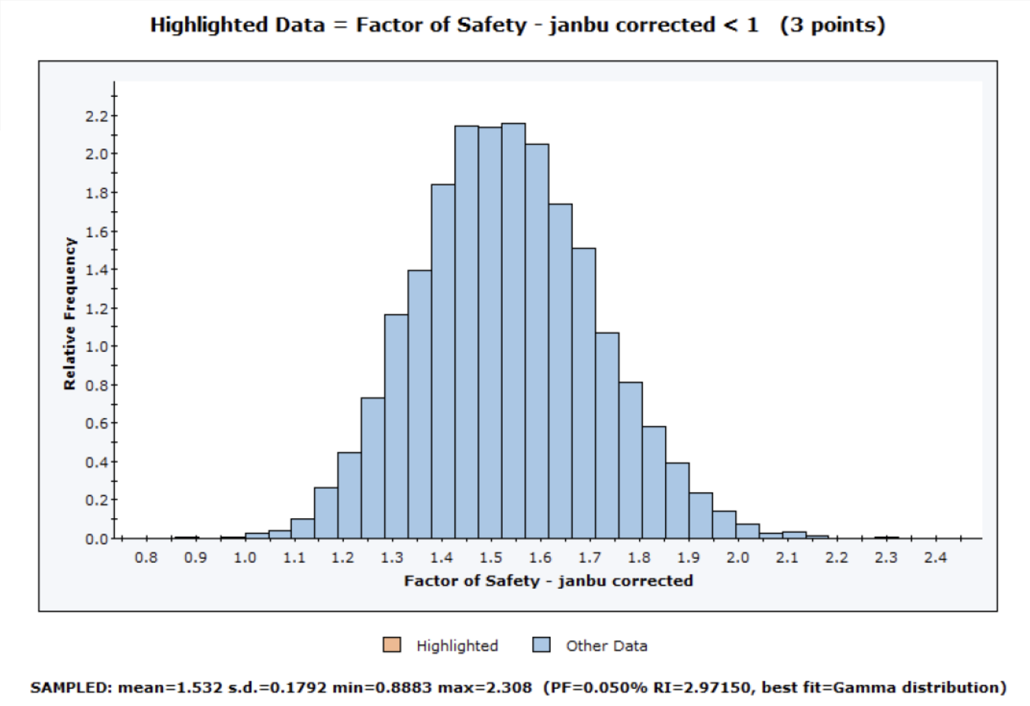
Probability FS ≤ 1 = 30.5%

Section S8_SSE2 – 200 years – base case at fence - BB: 125m



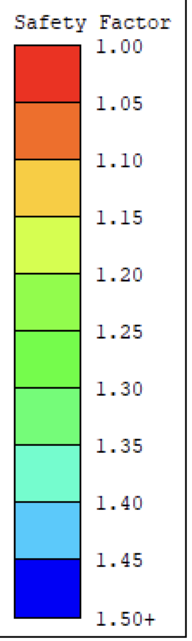


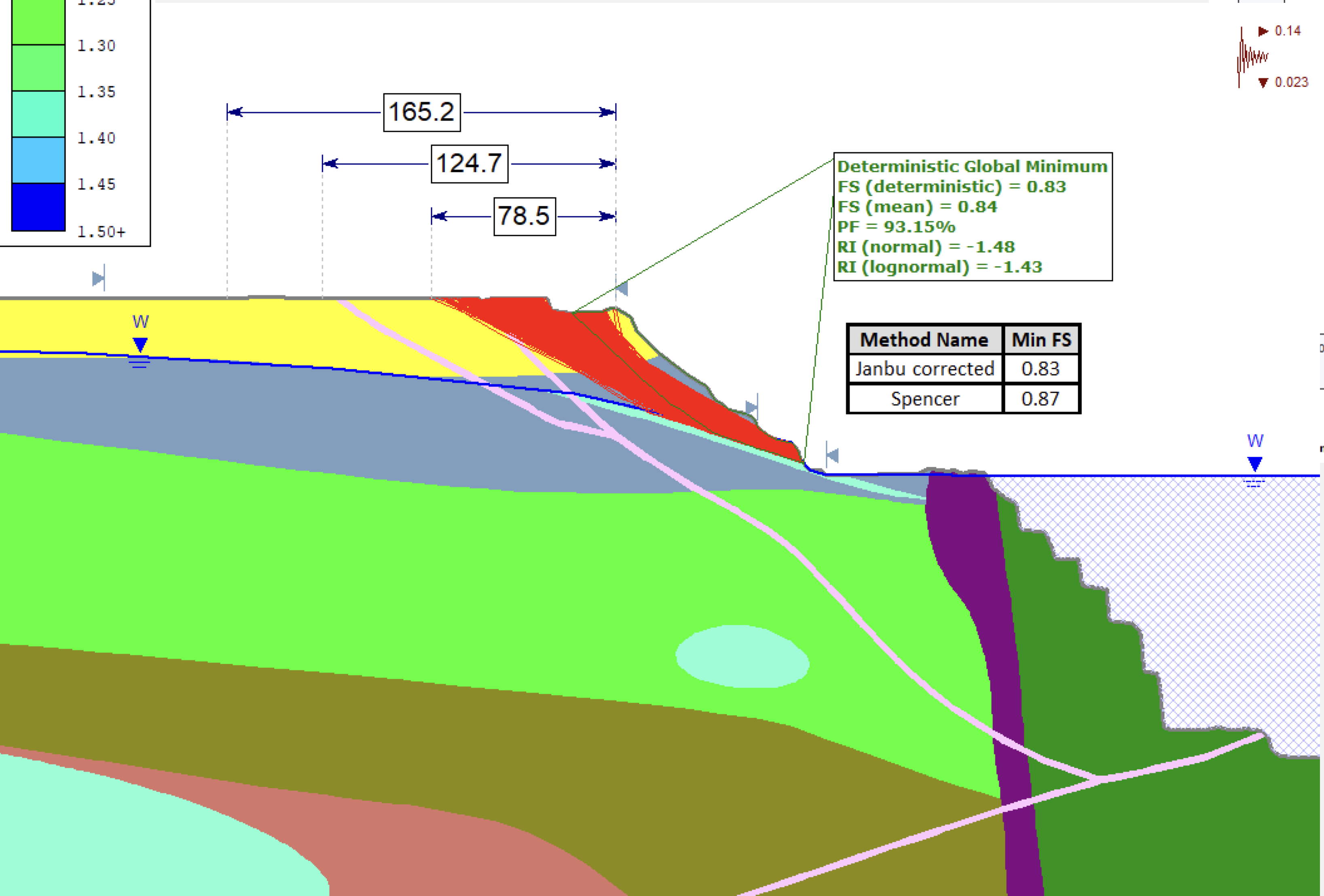
Method Name	Min FS
Janbu corrected	1.53
Spencer	1.54

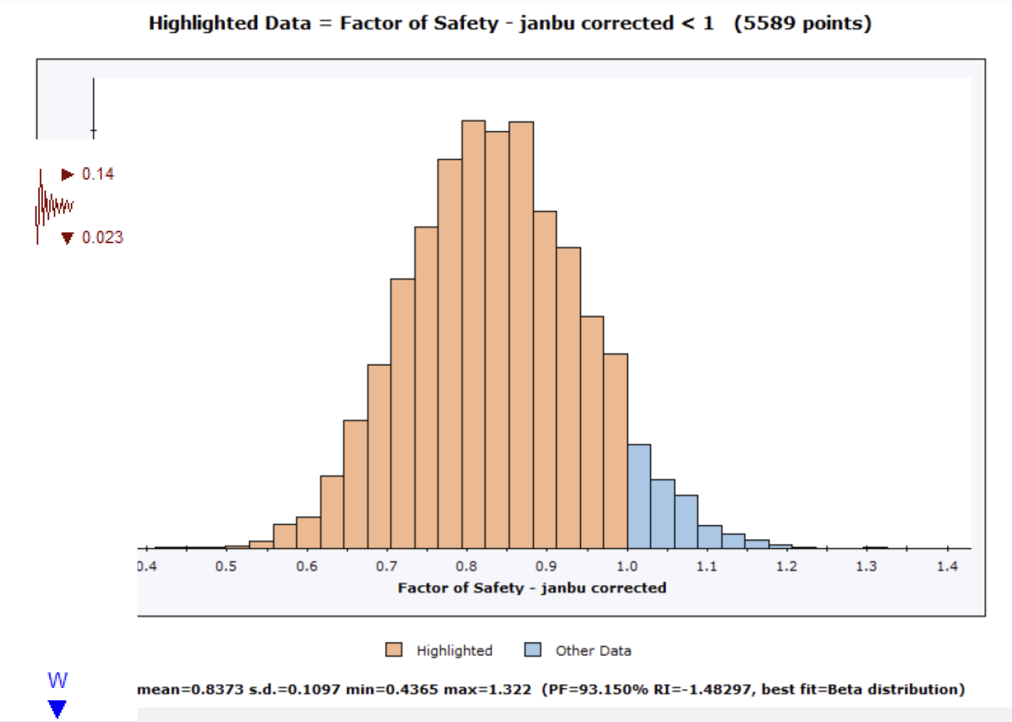


Probability FS ≤ 1 = 0.1%

Section S8_SSE2 – 200 years – seismic loading - BB: 13m

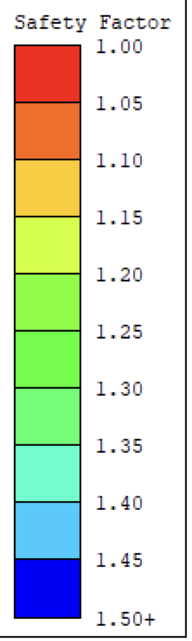


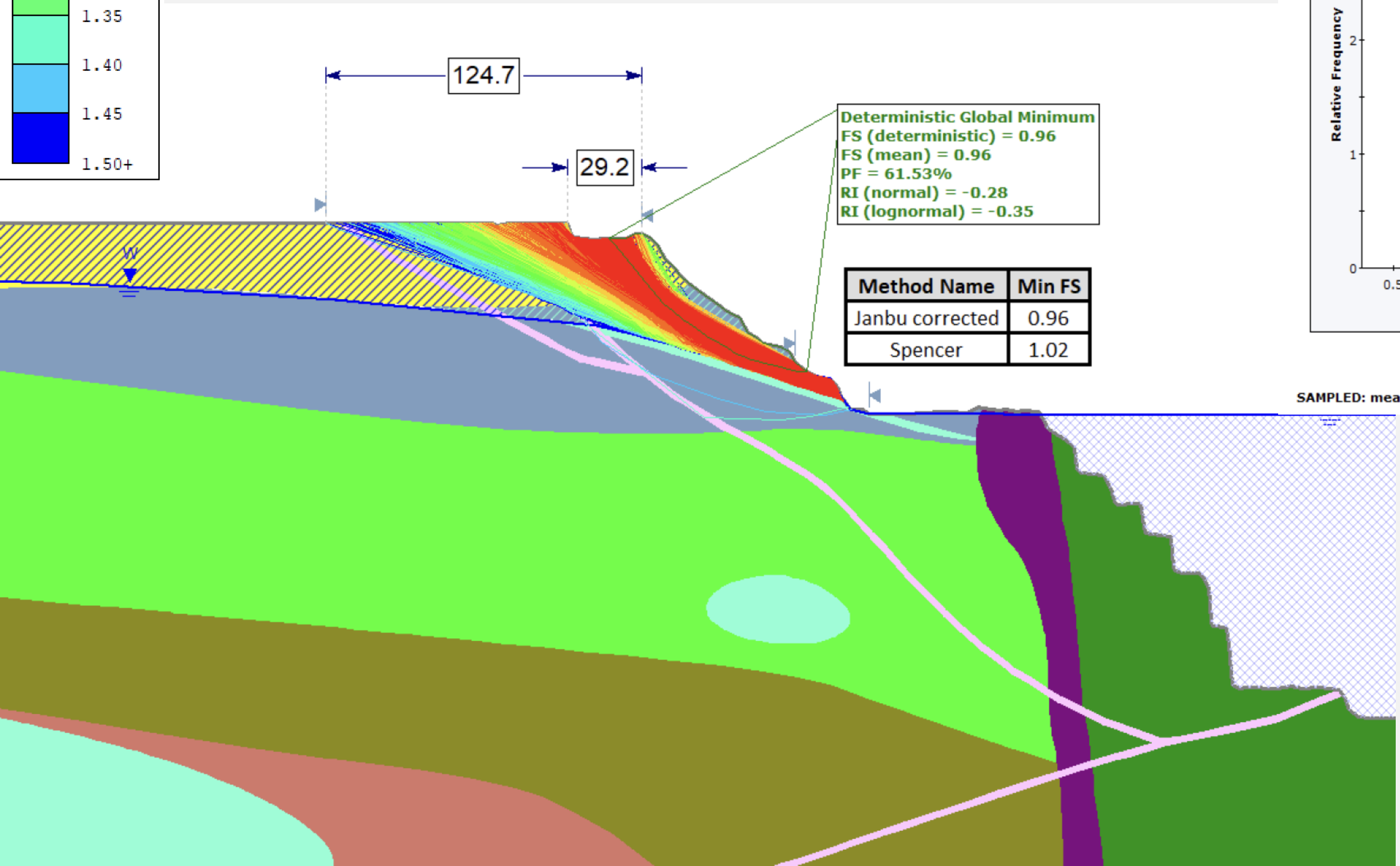




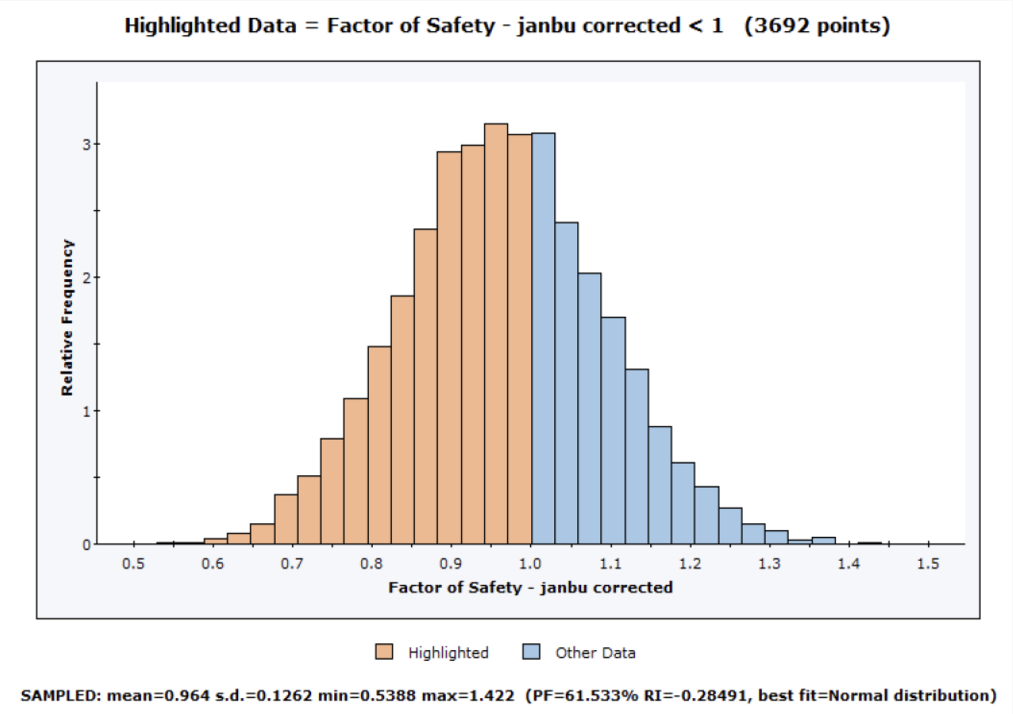
Probability FS ≤ 1 = 93.2%

Section S8_SSE2 – 200 years – transient pp Ru=0.15 - BB: 13m



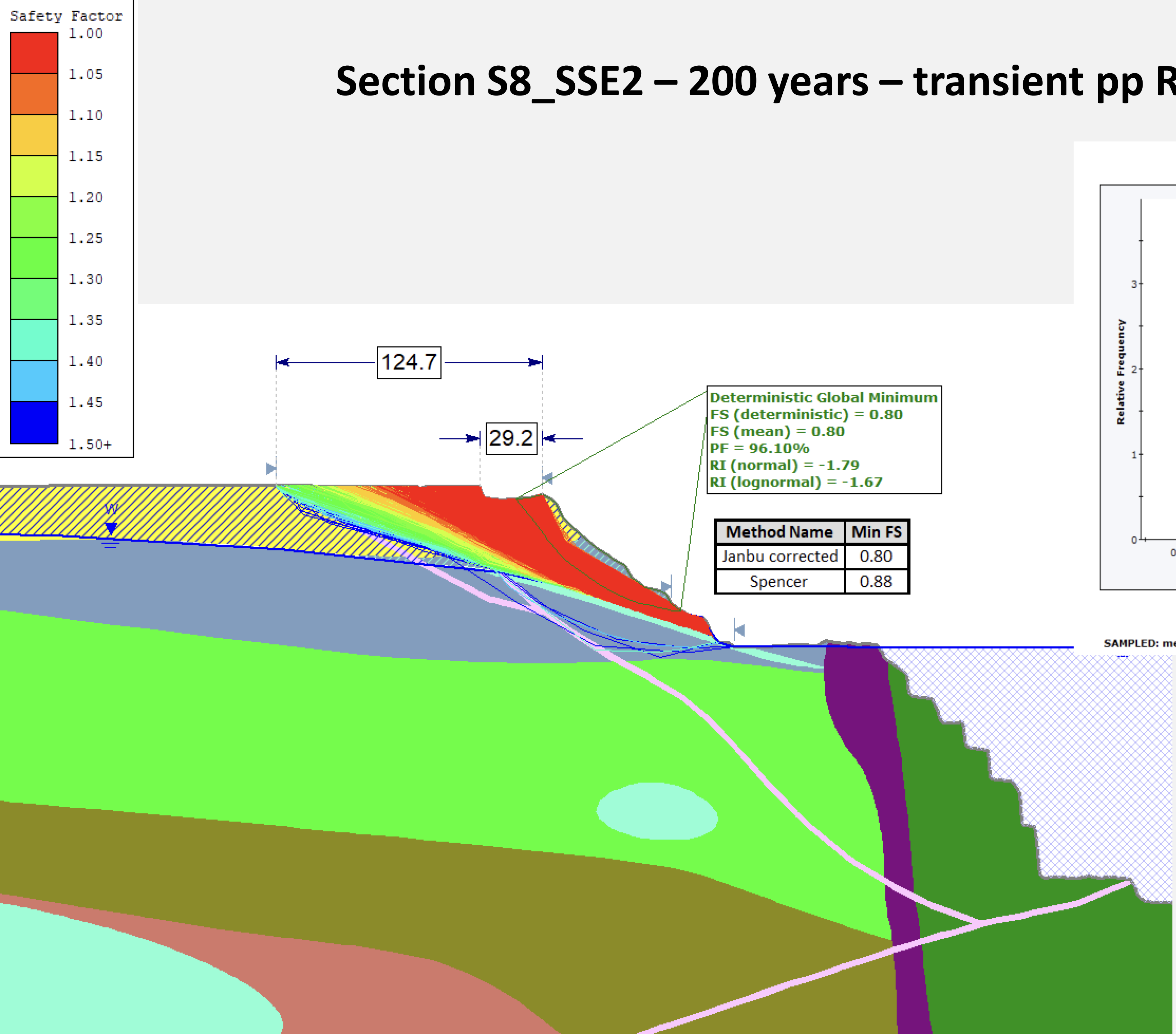


Method Name	Min FS
Janbu corrected	0.96
Spencer	1.02

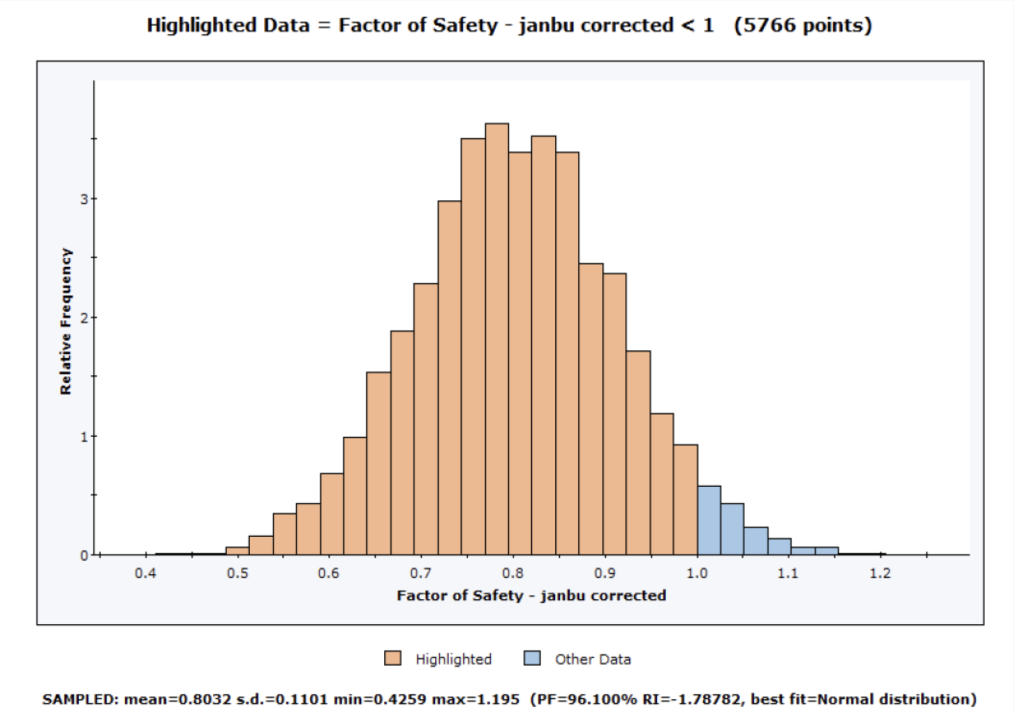


Probability FS ≤ 1 = 61.5%

Section S8_SSE2 – 200 years – transient pp Ru=0.30 - BB: 13m

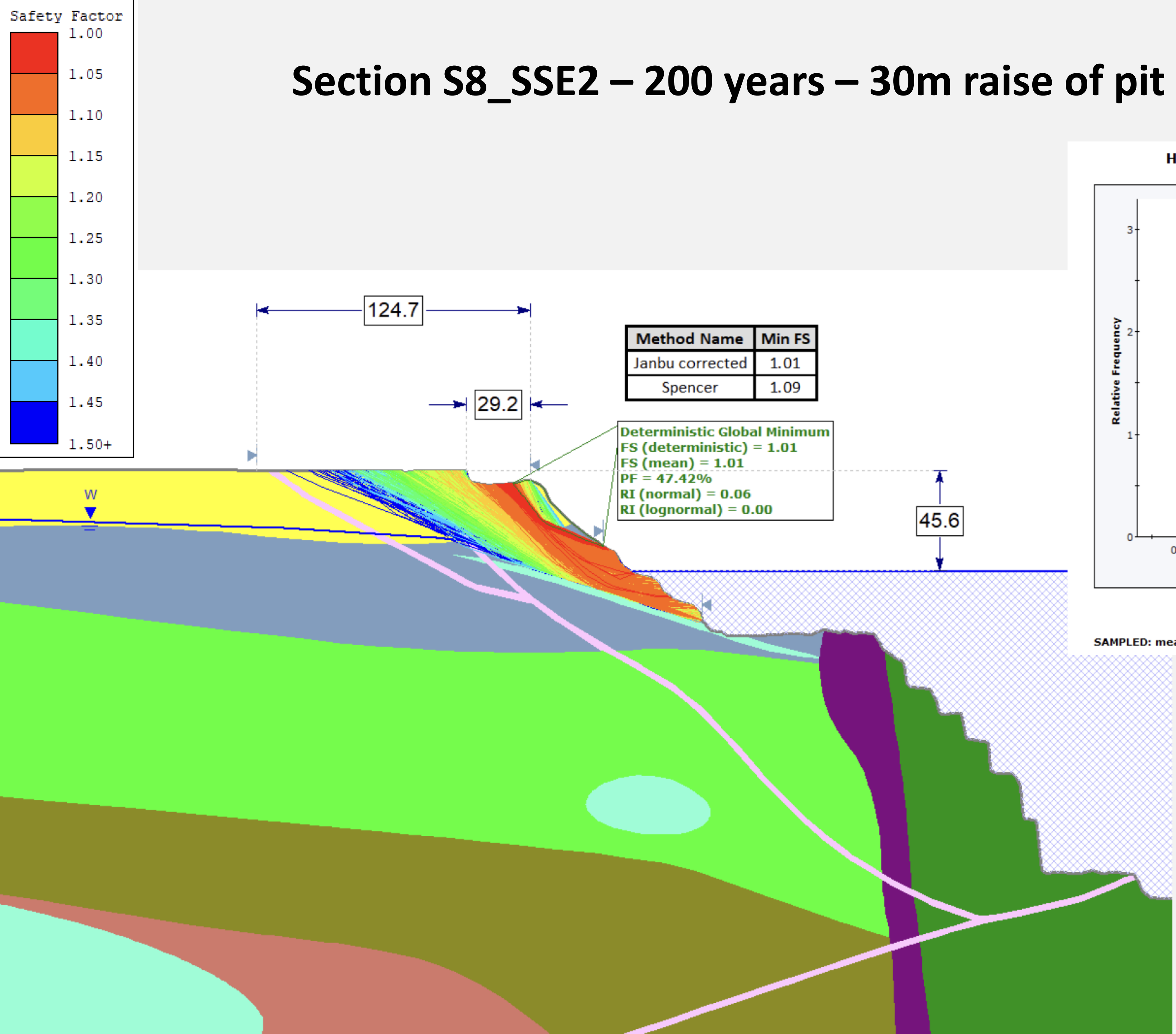


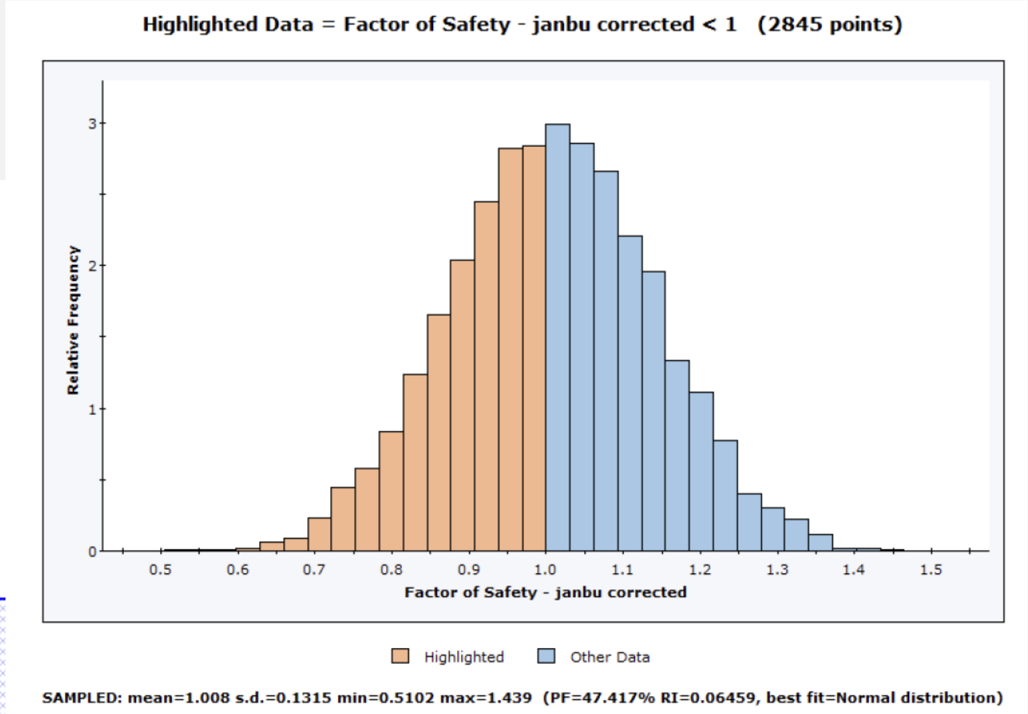
Method Name	Min FS
Janbu corrected	0.80
Spencer	0.88



Probability FS ≤ 1 = 96.1%

Section S8_SSE2 – 200 years – 30m raise of pit lake level - BB: 10m

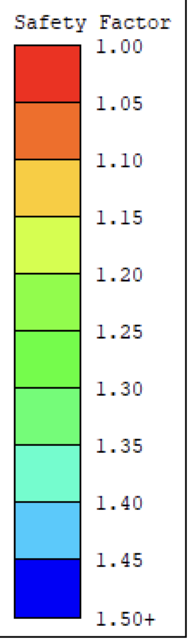


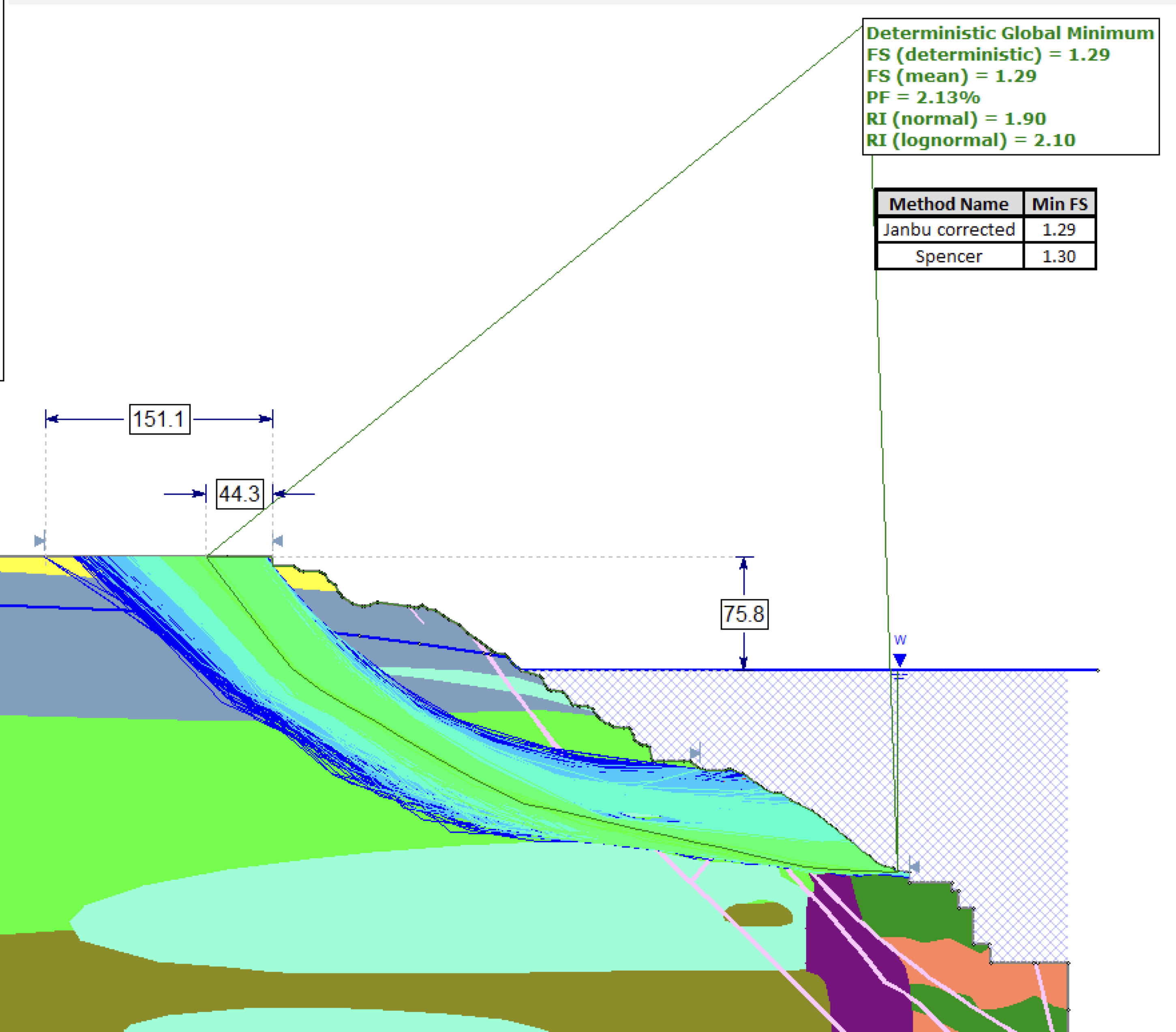


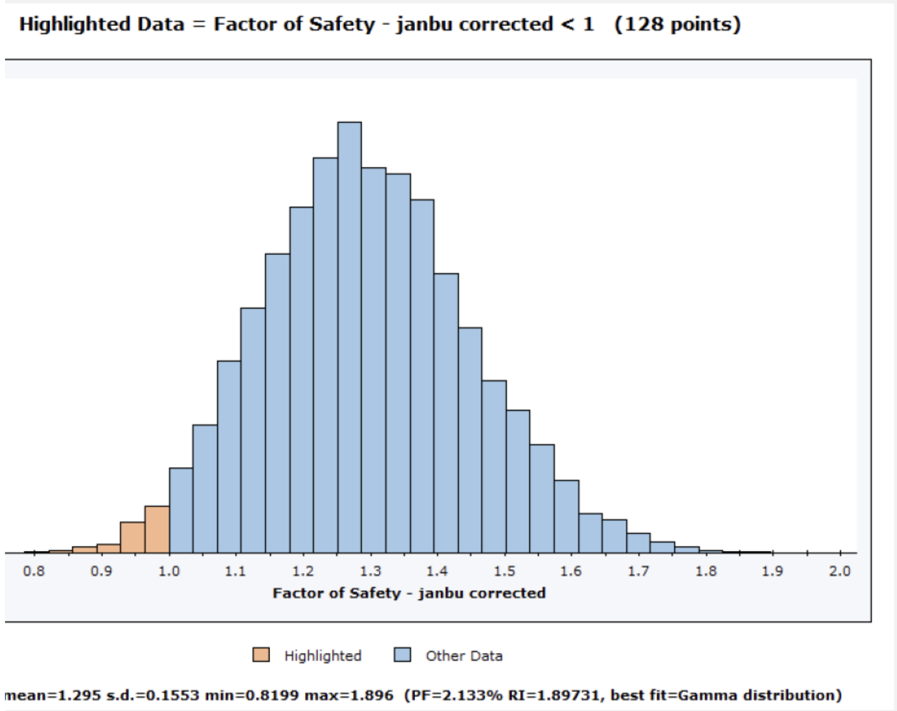
Probability FS ≤ 1 = 47.4%

Section S3_N28
Special Conditions

Section S3_N28 – 200 years – base case revised wl - BB: 44m

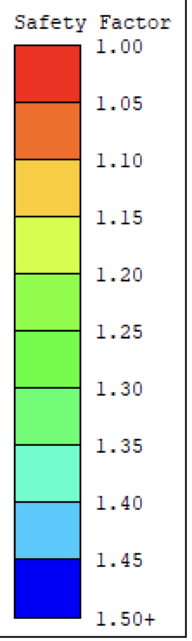


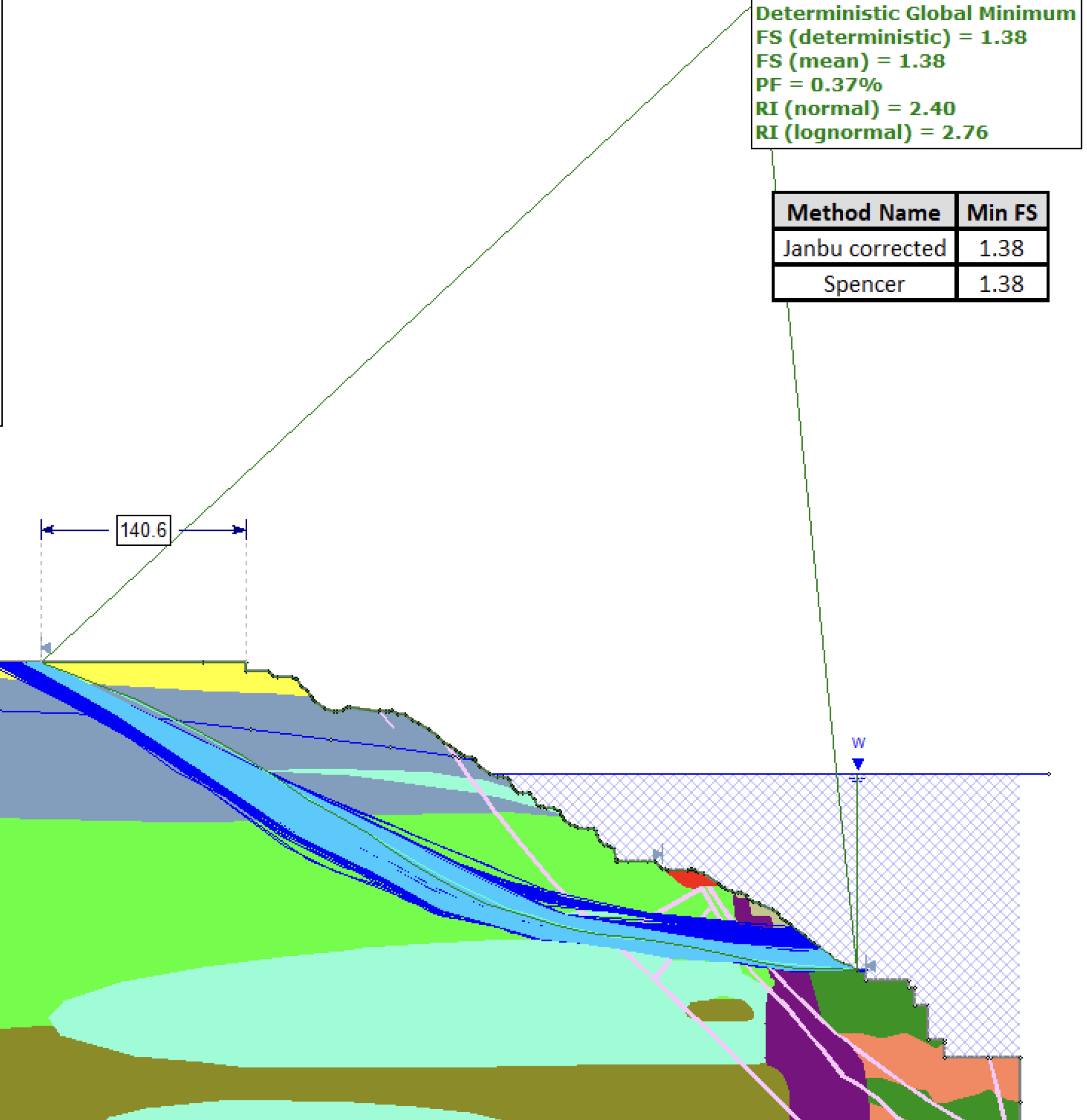


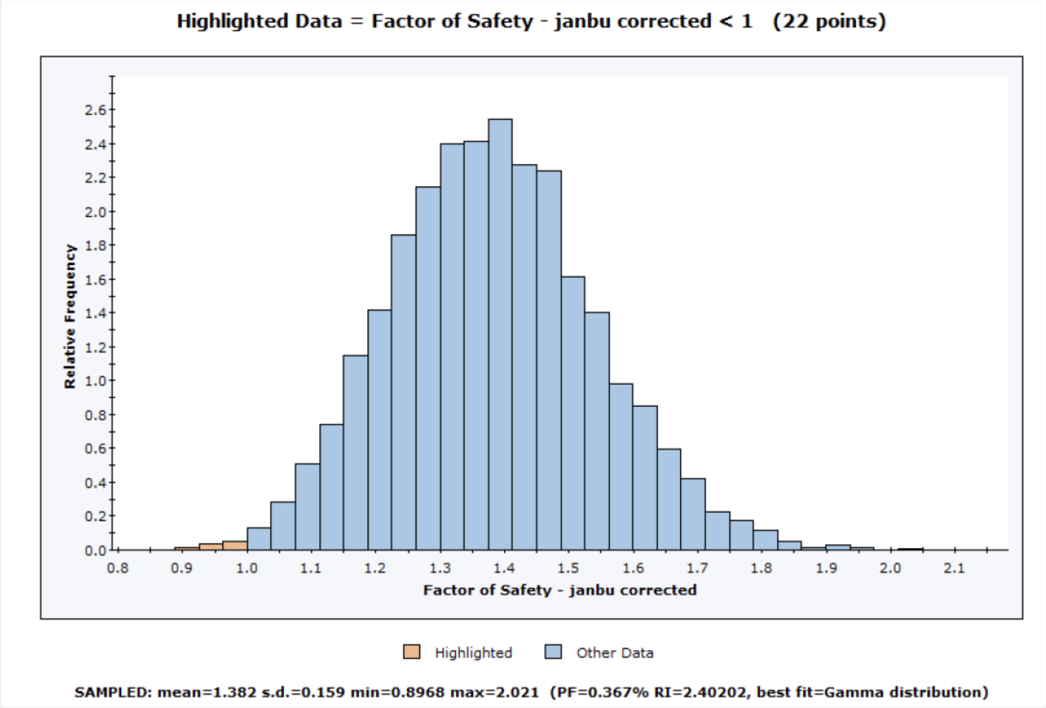


Probability FS ≤ 1 = 2.1%

Section S3_N28 – 200 years – base case at fence - BB: 140m

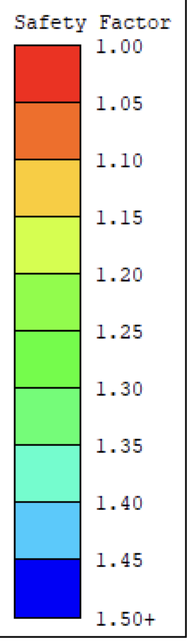


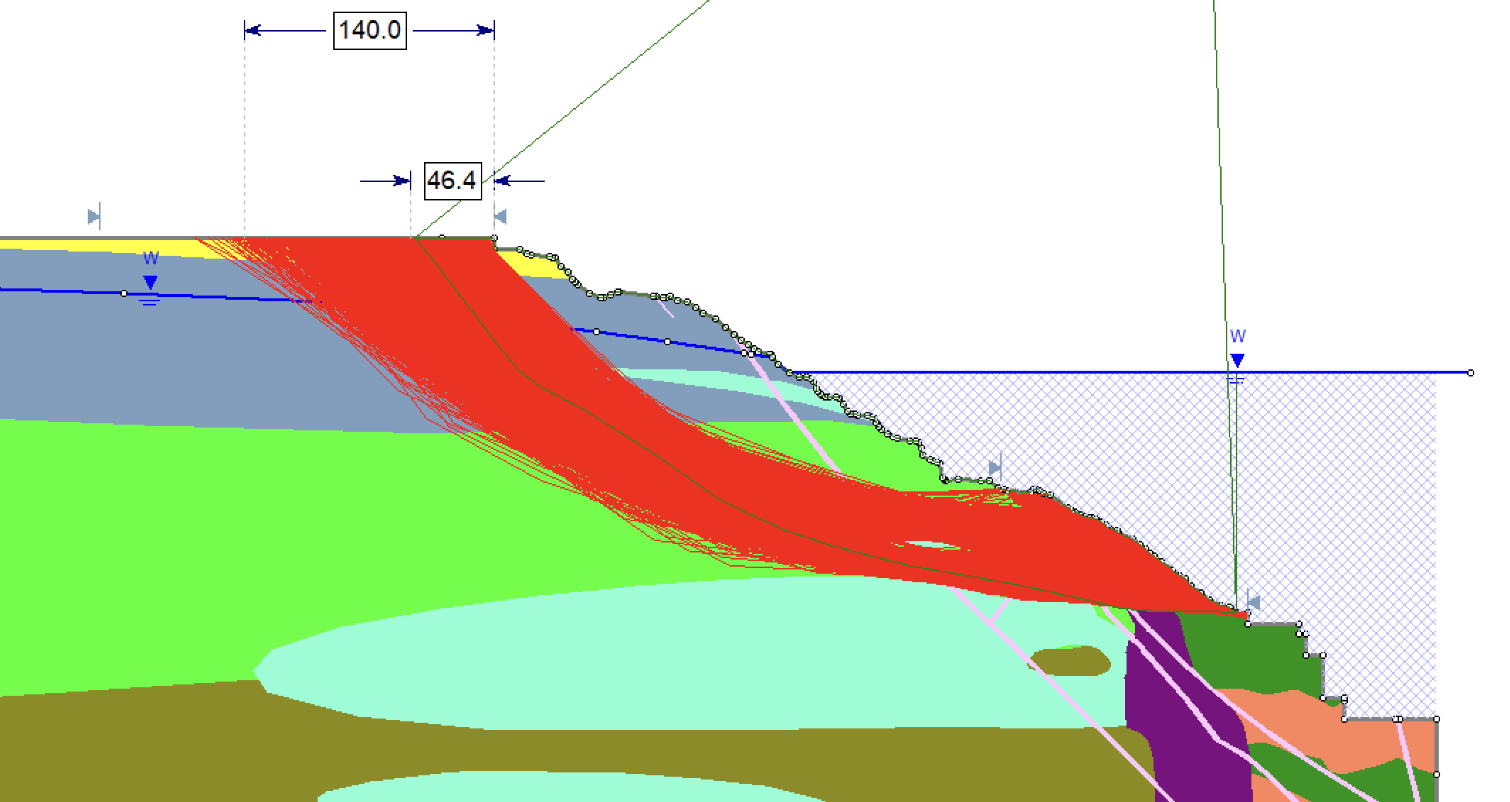




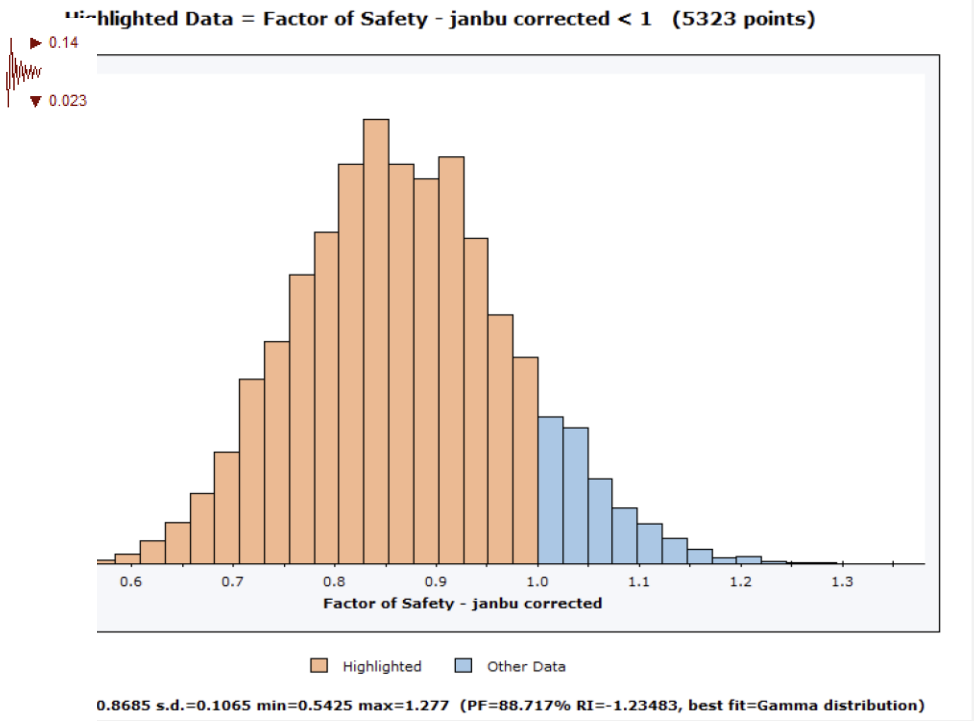
Probability FS ≤ 1 = 0.4%

Section S3_N28 – 200 years – seismic loading - BB: 46m



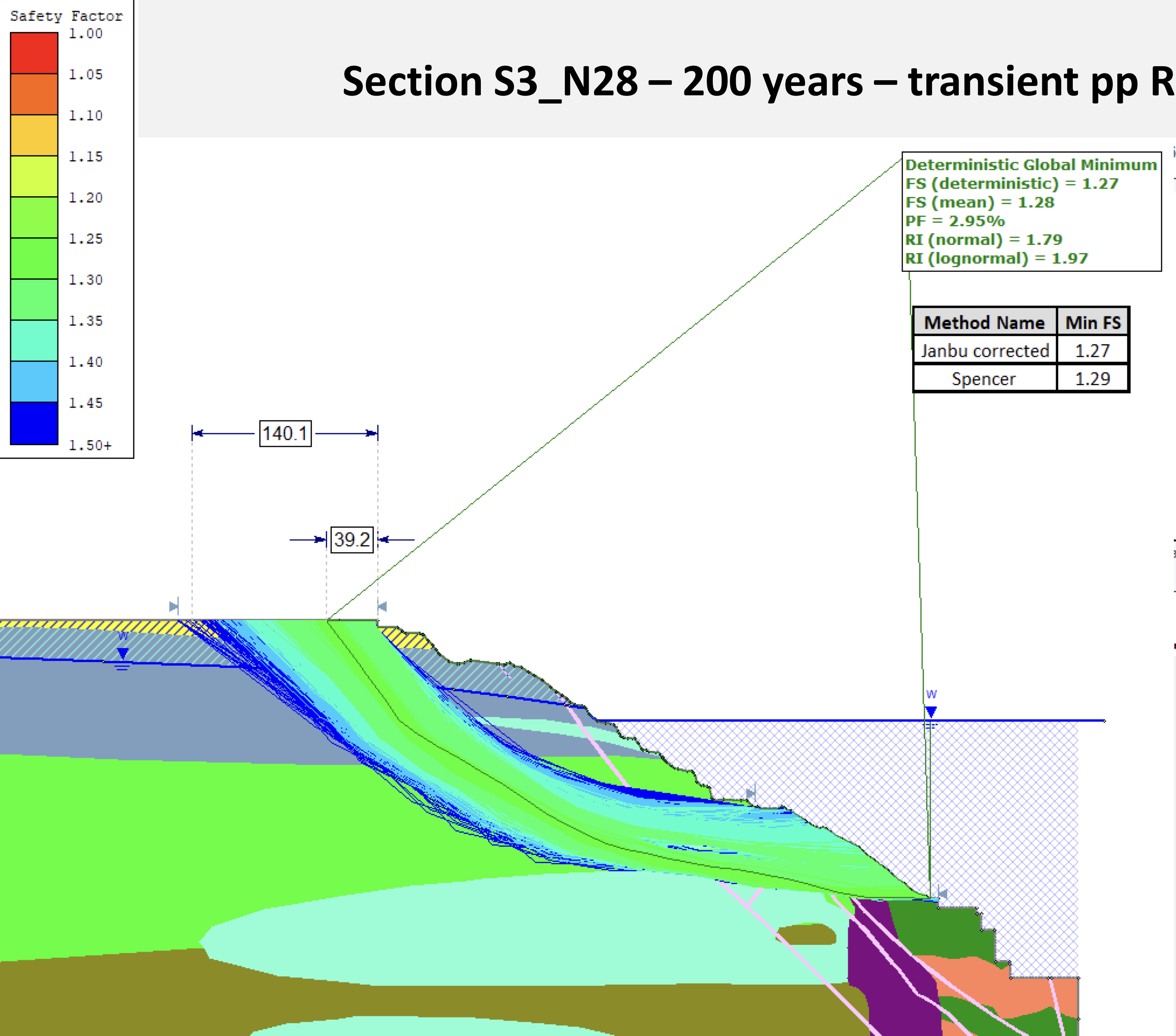


Method Name	Min FS
Janbu corrected	0.87
Spencer	0.89

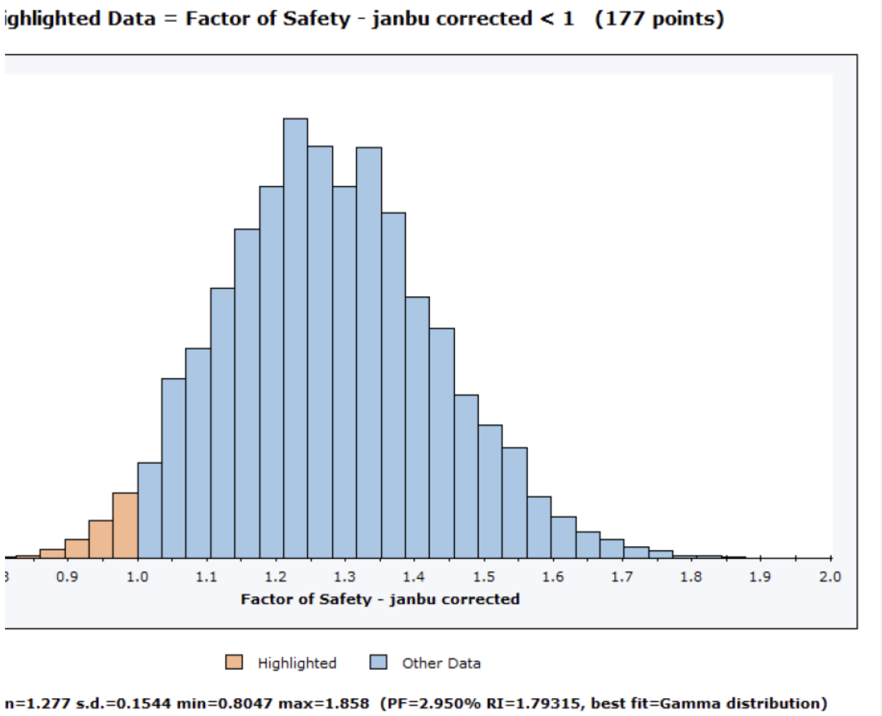


Probability FS ≤ 1 = 88.7%

Section S3_N28 – 200 years – transient pp Ru=0.15 - BB: 39m

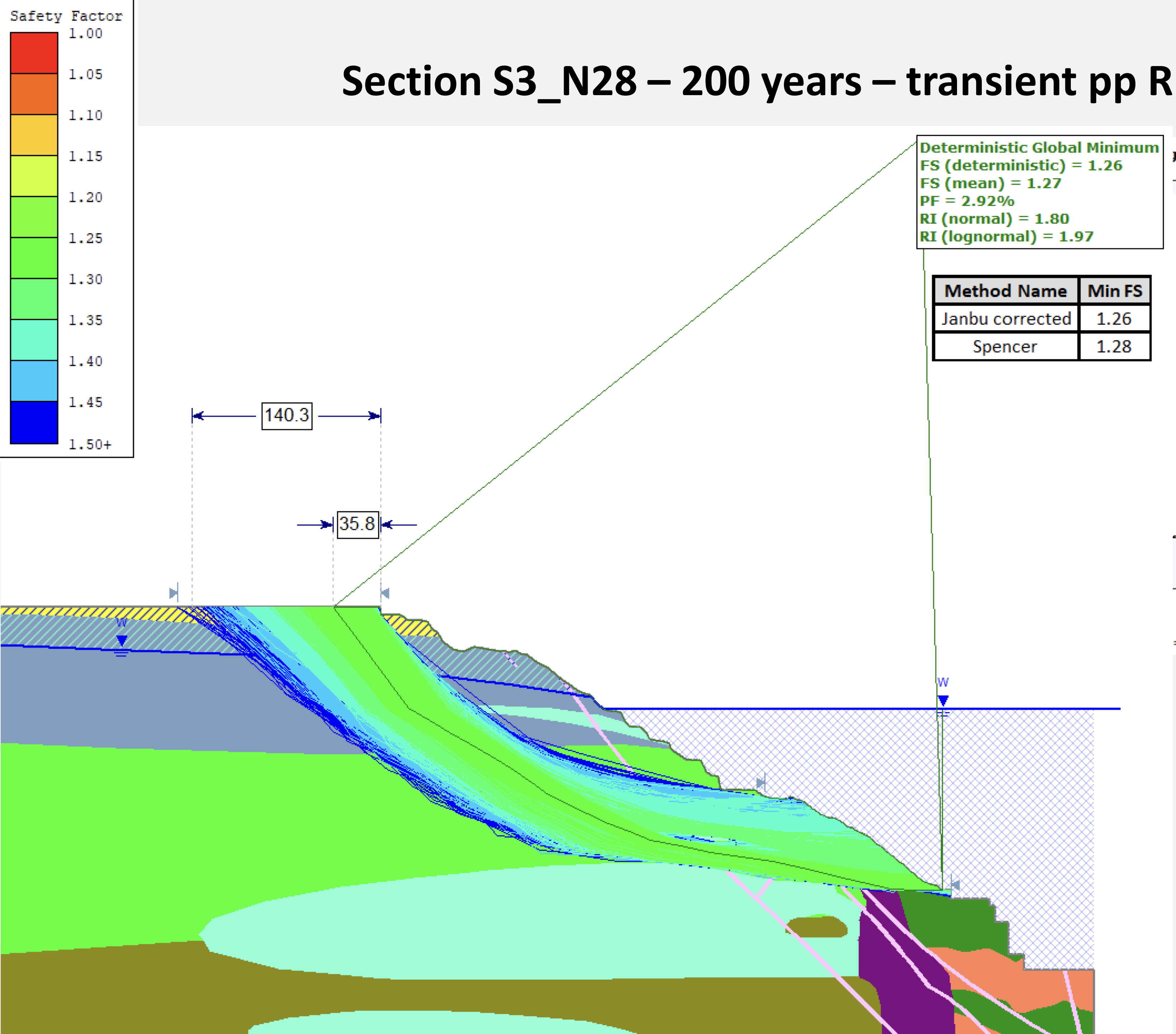


Method Name	Min FS
Janbu corrected	1.27
Spencer	1.29

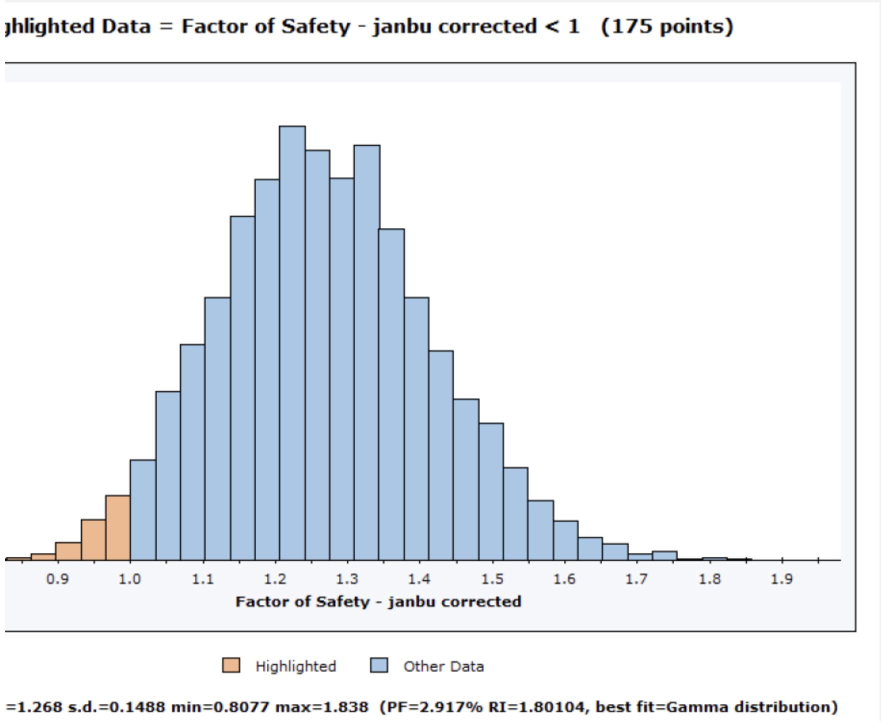


Probability FS ≤ 1 = 3.0%

Section S3_N28 – 200 years – transient pp Ru=0.30 - BB: 39m

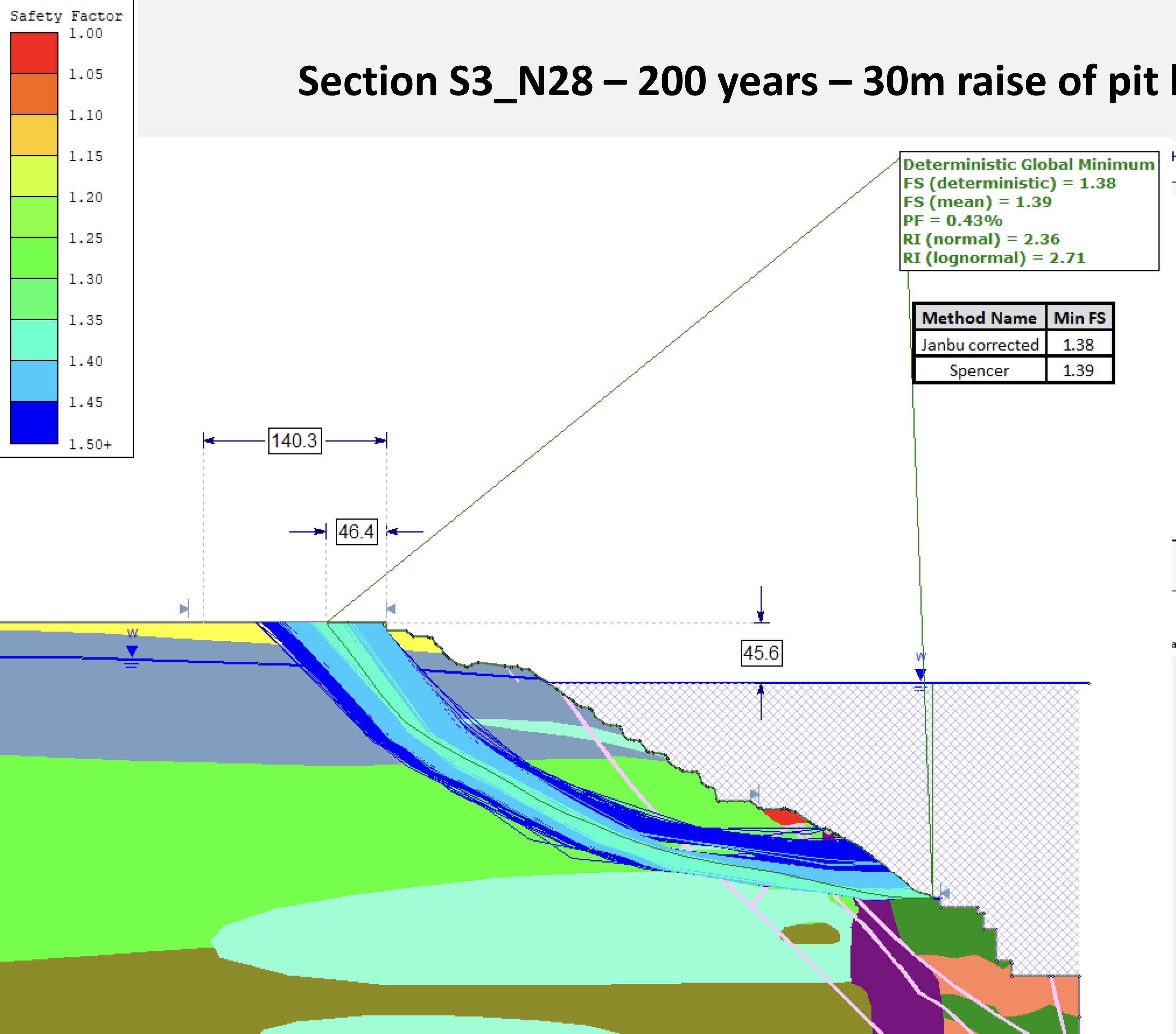


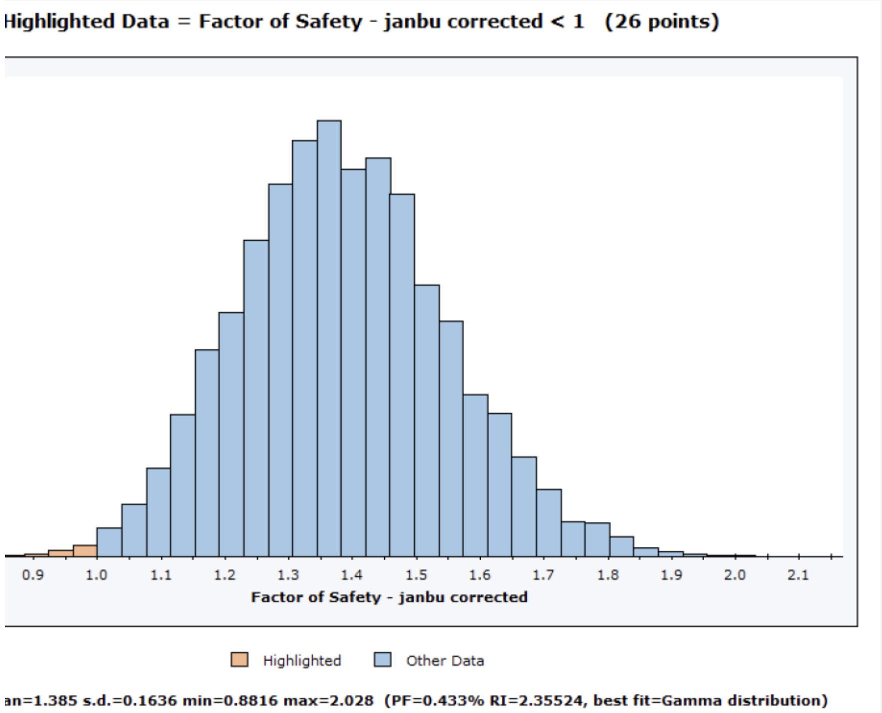
Method Name	Min FS
Janbu corrected	1.26
Spencer	1.28



Probability FS ≤ 1 = 3.0%

Section S3_N28 – 200 years – 30m raise of pit lake level - BB: 46m

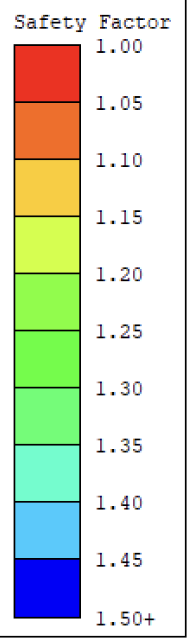


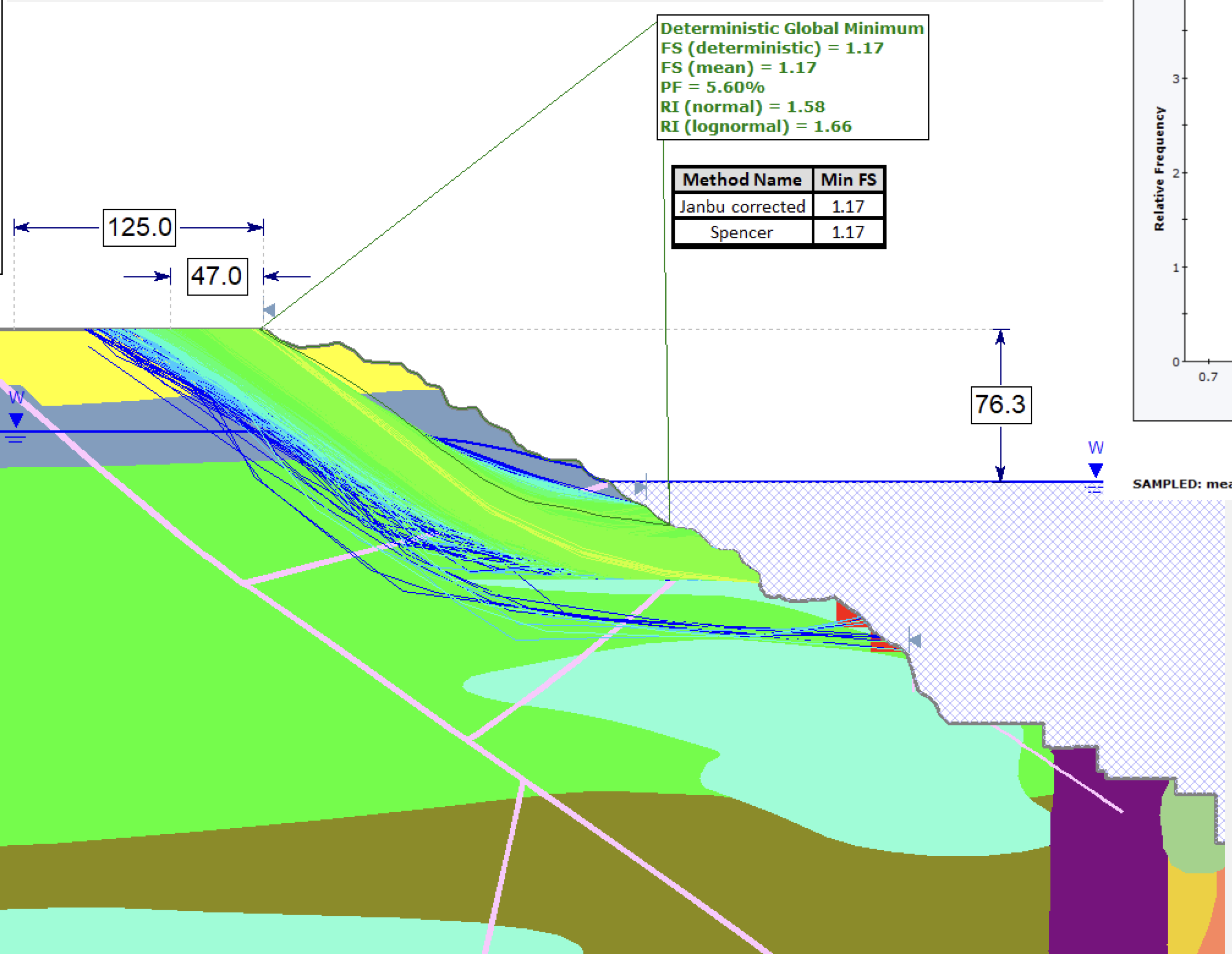


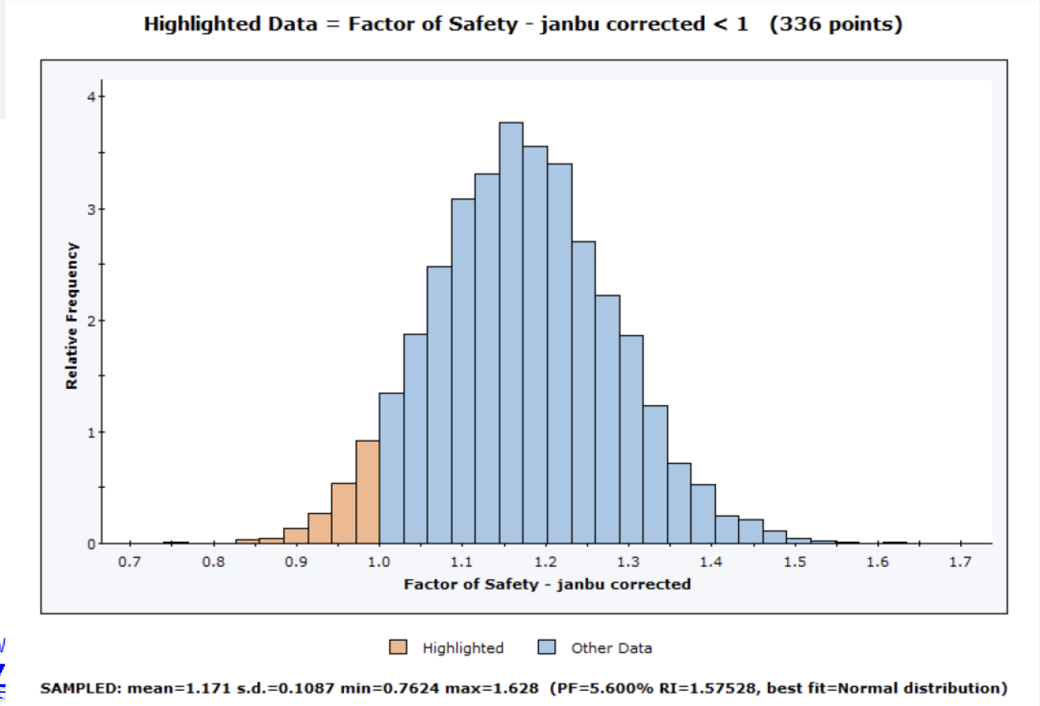
Probability FS ≤ 1 = 0.4%

Section S4_NE10&NE11
Special Conditions

Section S4_NE10&NE11 – 200 years – base case revised wl - BB: 0m

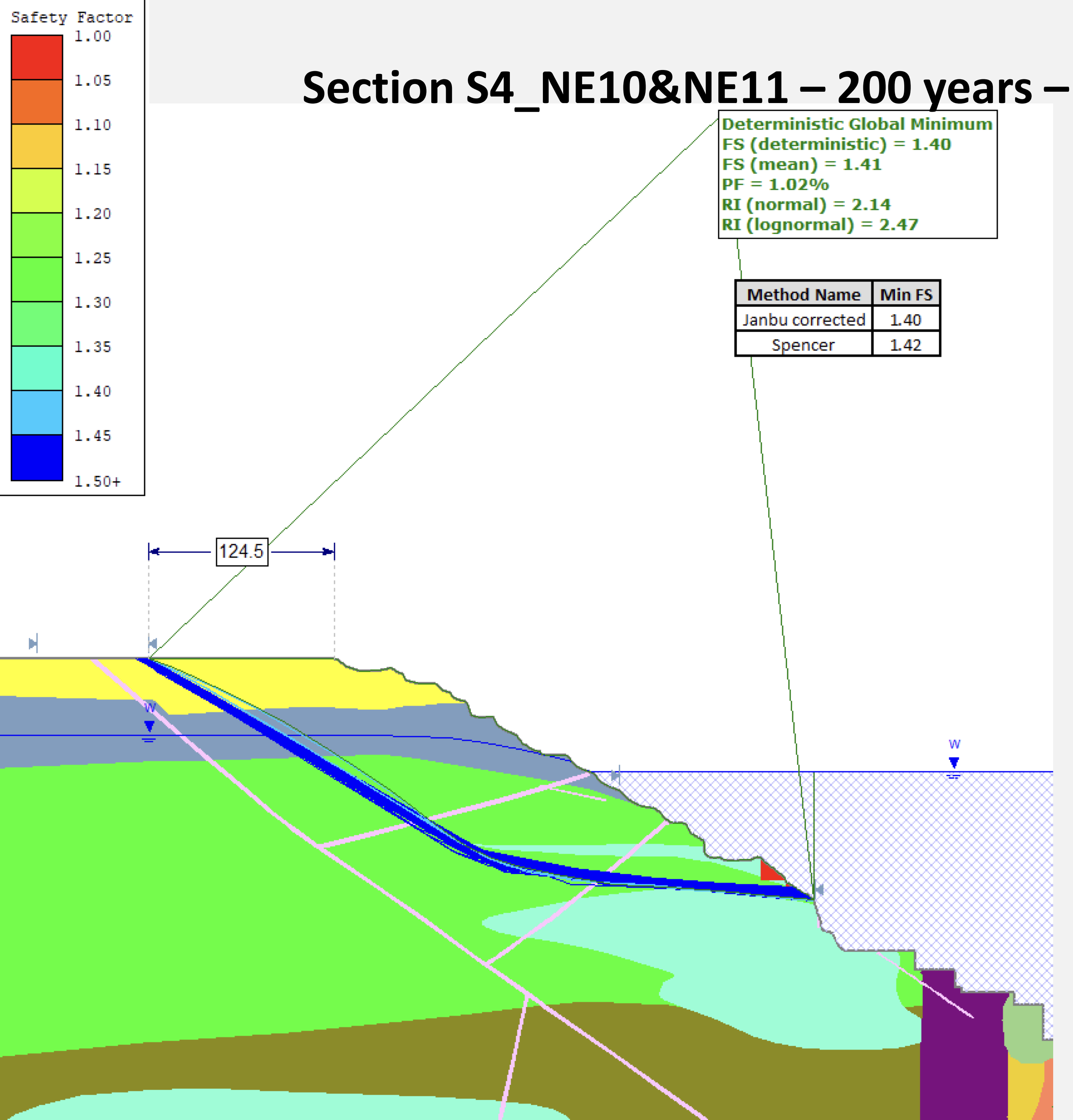




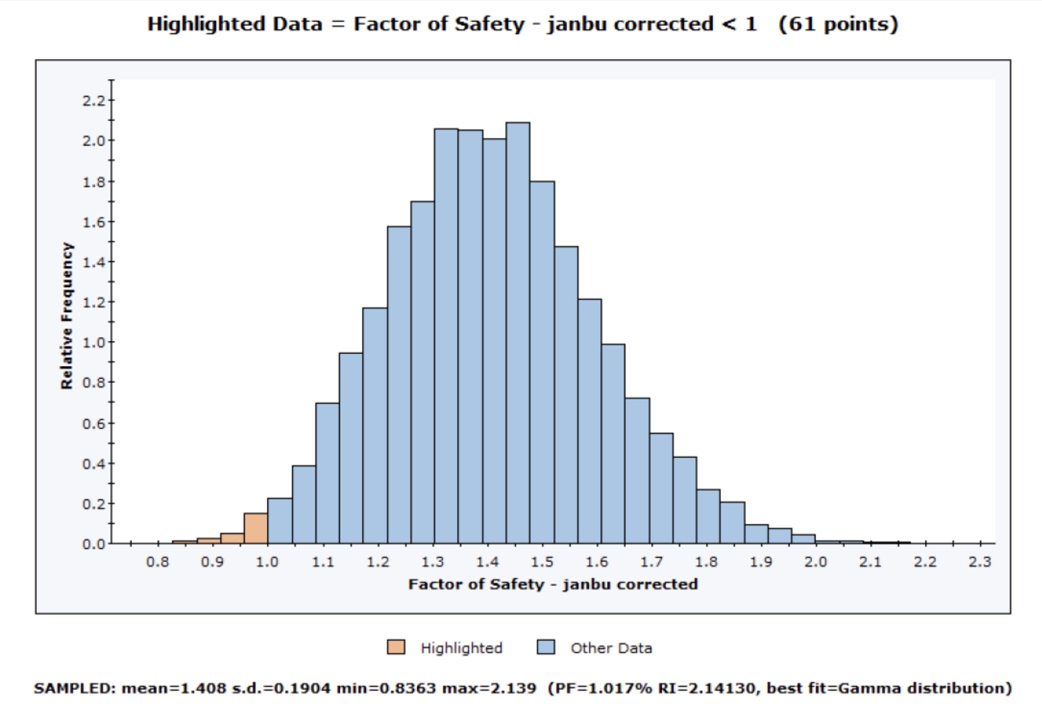


Probability FS ≤ 1 = 5.6%

Section S4_NE10&NE11 – 200 years – base case at fence - BB: 125m

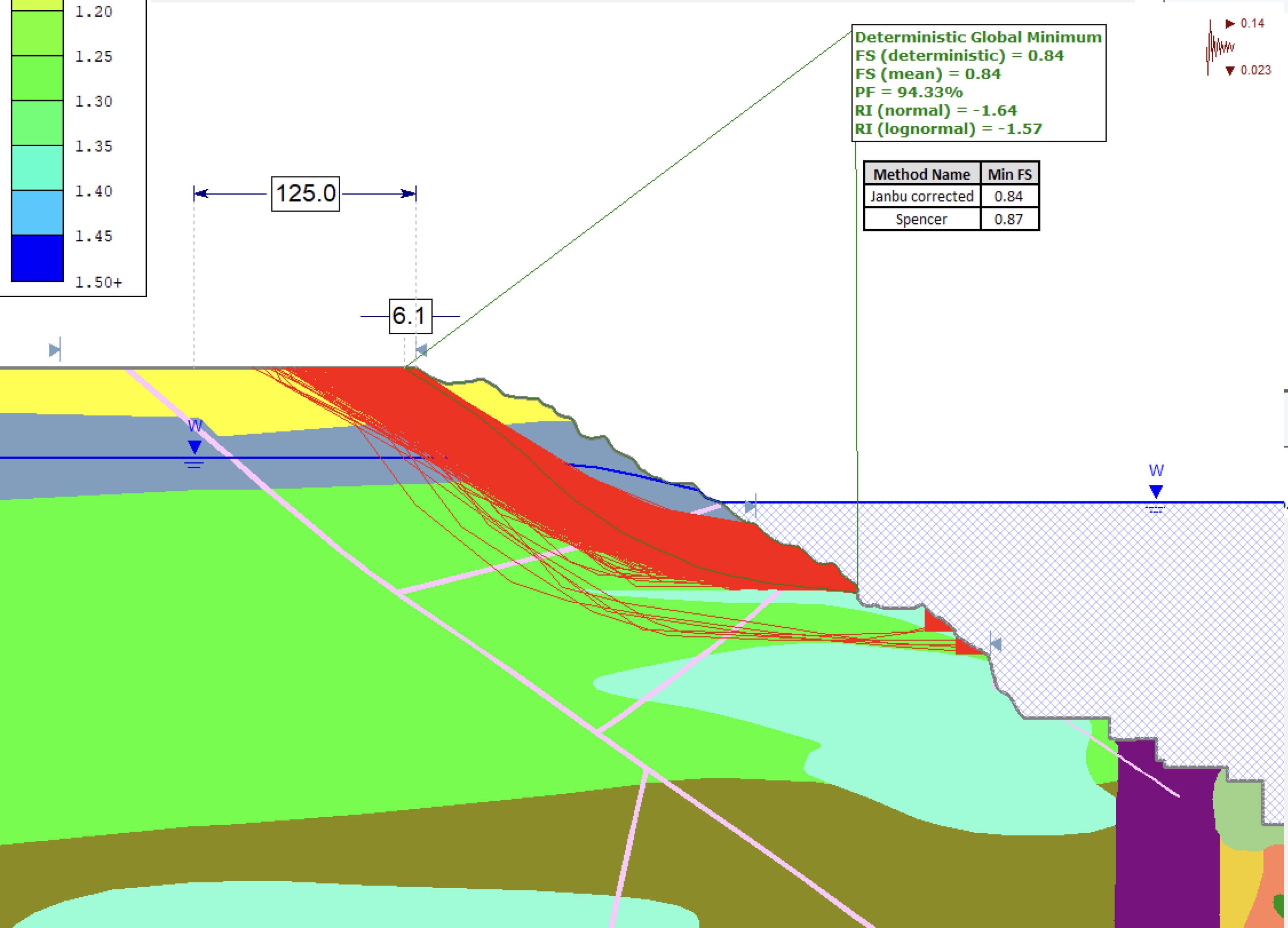


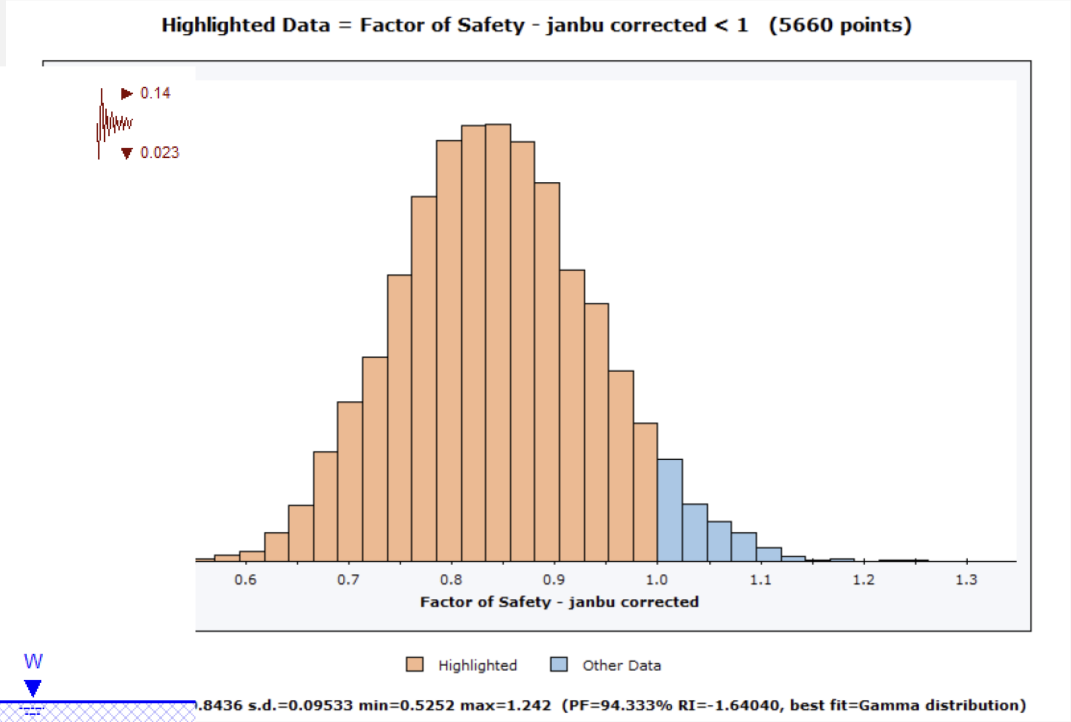
Method Name	Min FS
Janbu corrected	1.40
Spencer	1.42



Probability FS ≤ 1 = 1.0%

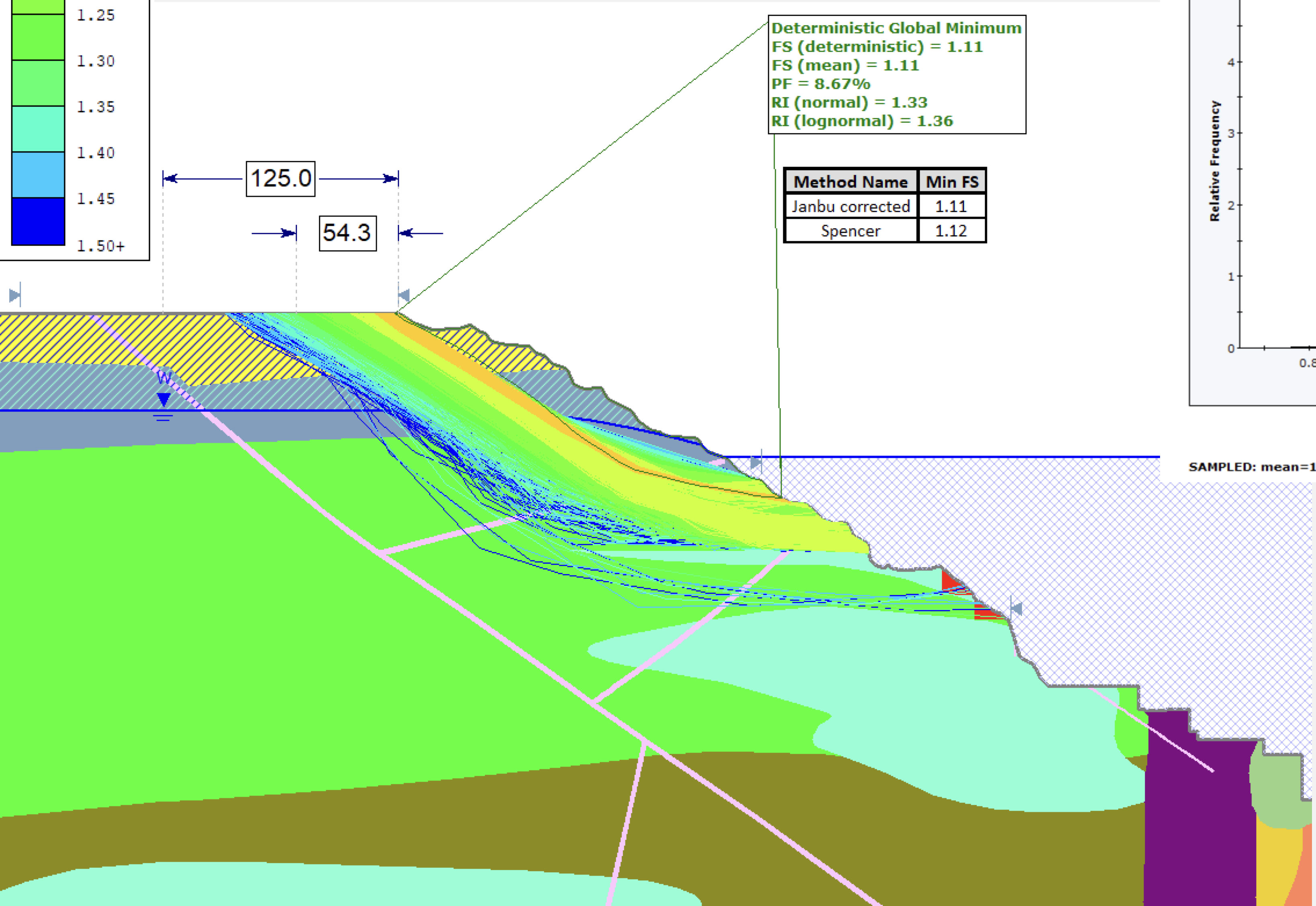
Section S4_NE10&NE11 – 200 years – seismic loading - BB: 6m



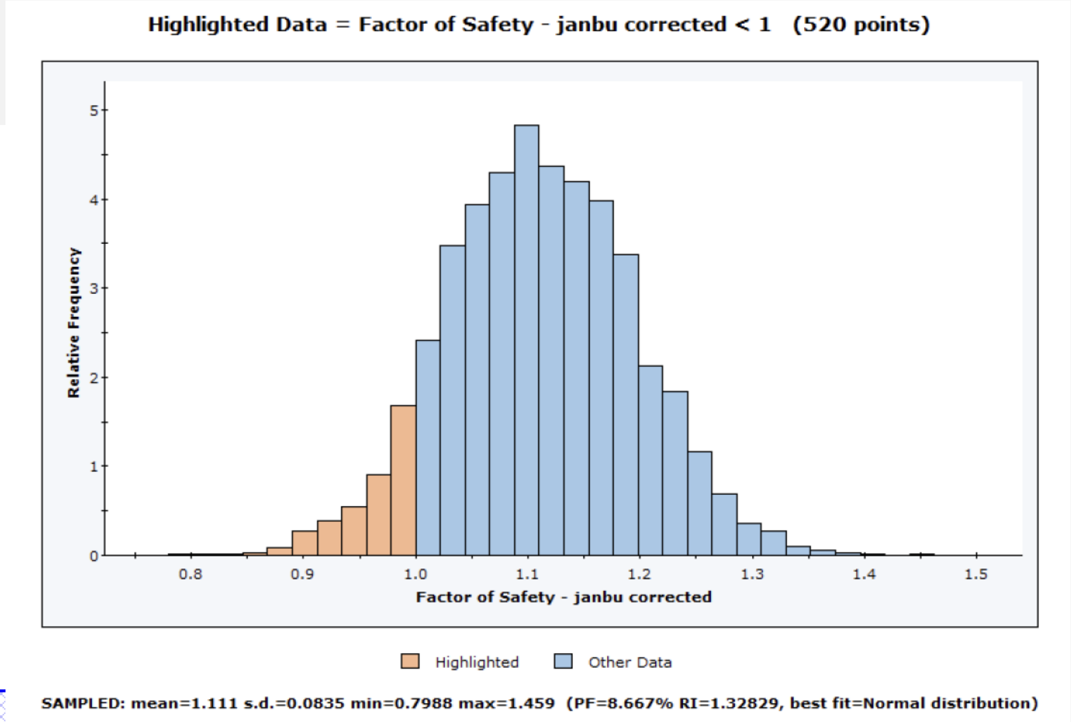


Probability FS ≤ 1 = 94.3%

Section S4_NE10&NE11 – 200 years – transient pp Ru=0.15 - BB: 0m

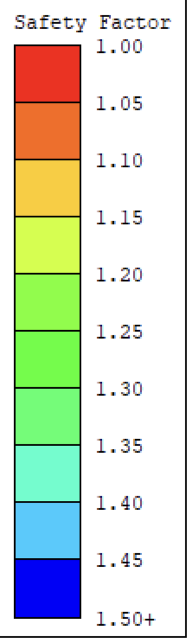


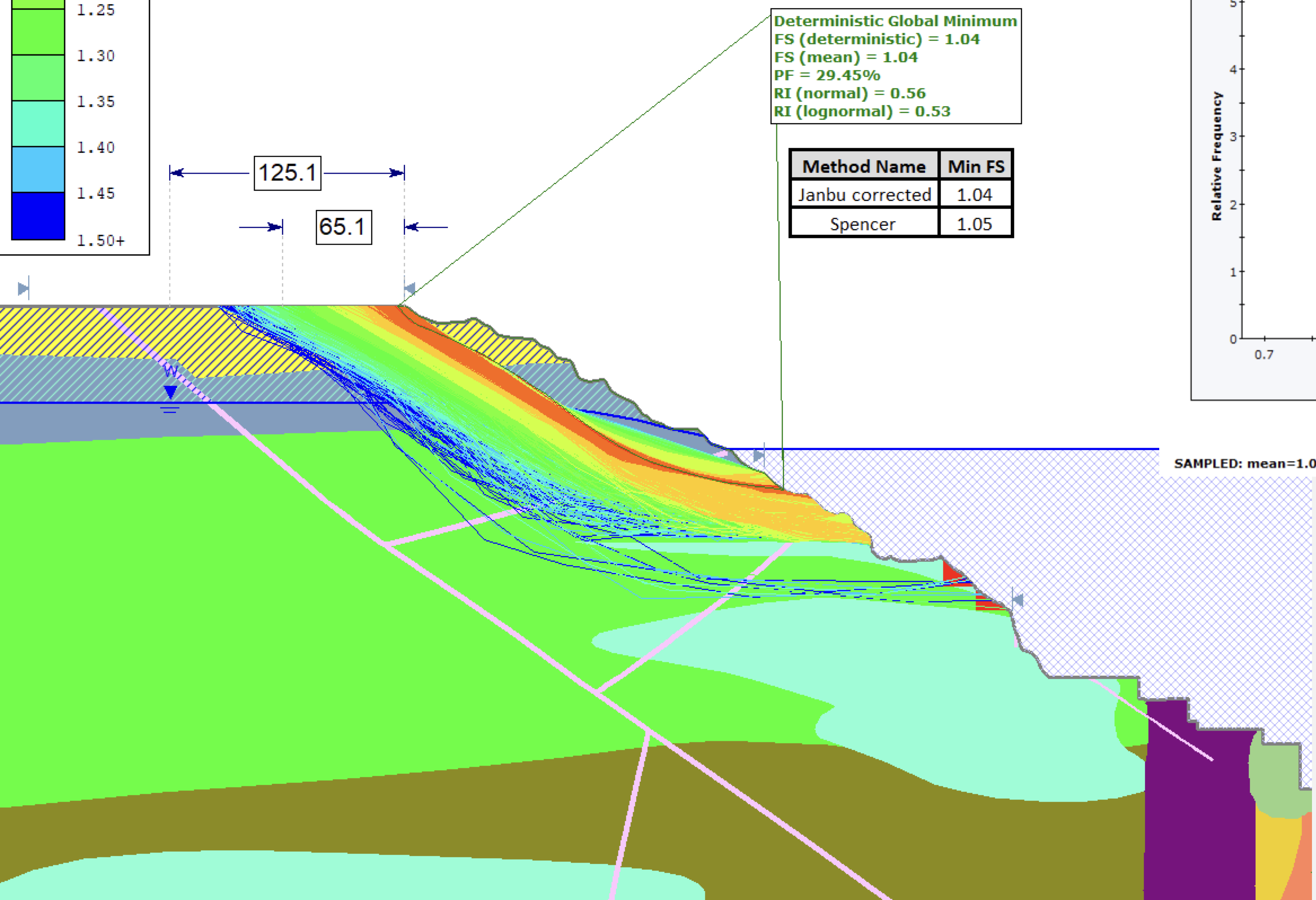
Method Name	Min FS
Janbu corrected	1.11
Spencer	1.12

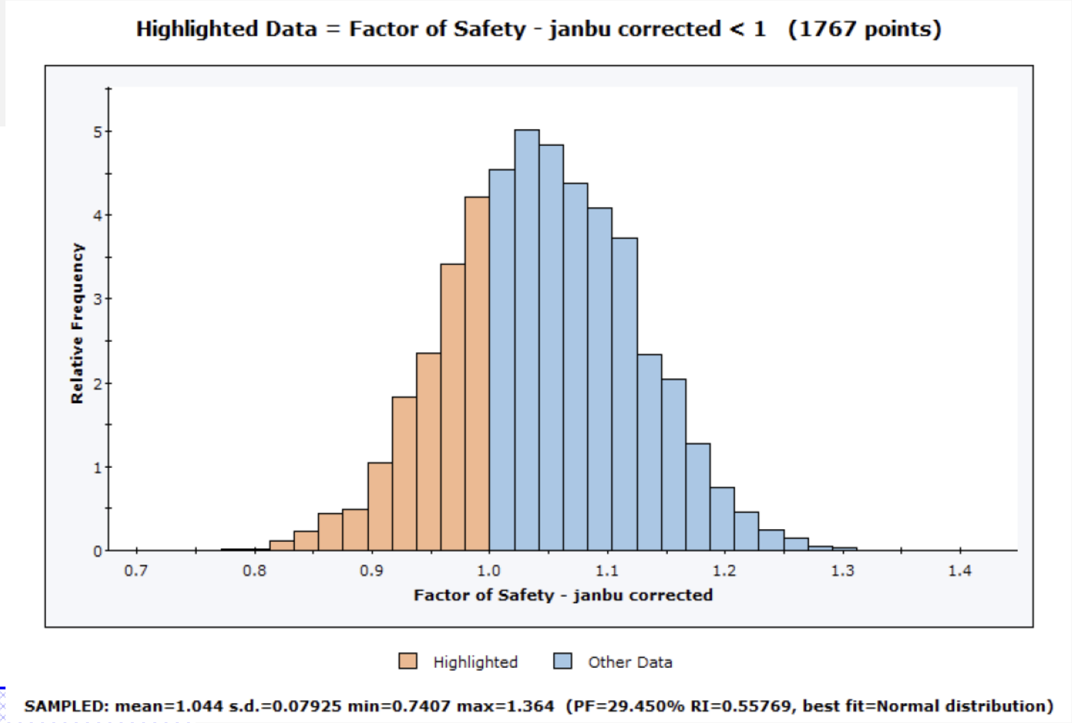


Probability FS ≤ 1 = 8.7%

Section S4_NE10&NE11 – 200 years – transient pp Ru=0.30 - BB: 0m

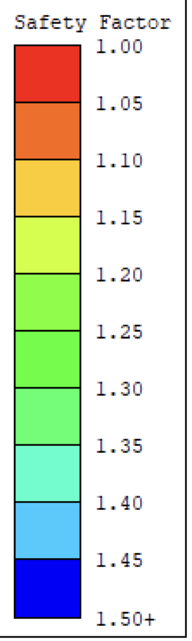


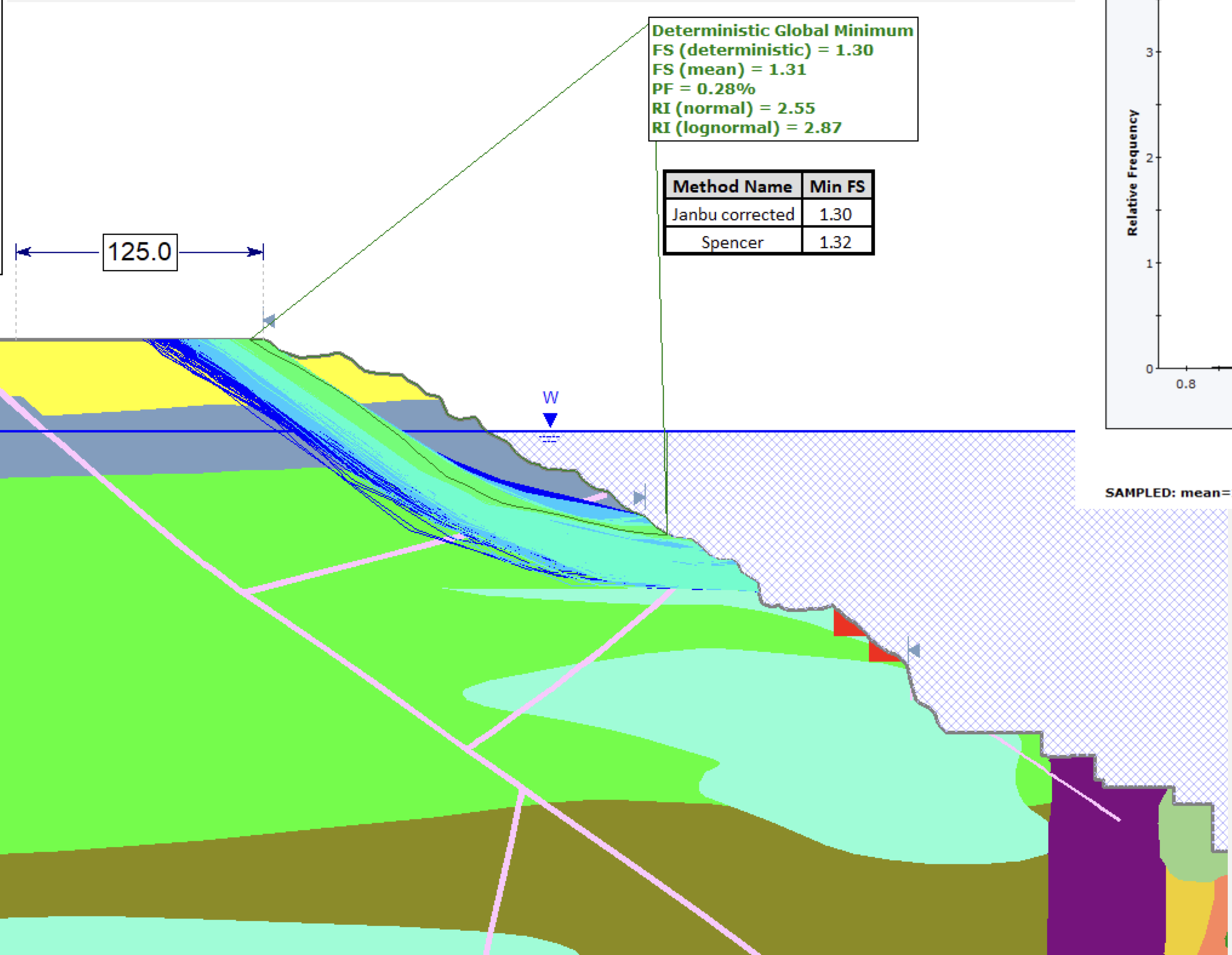


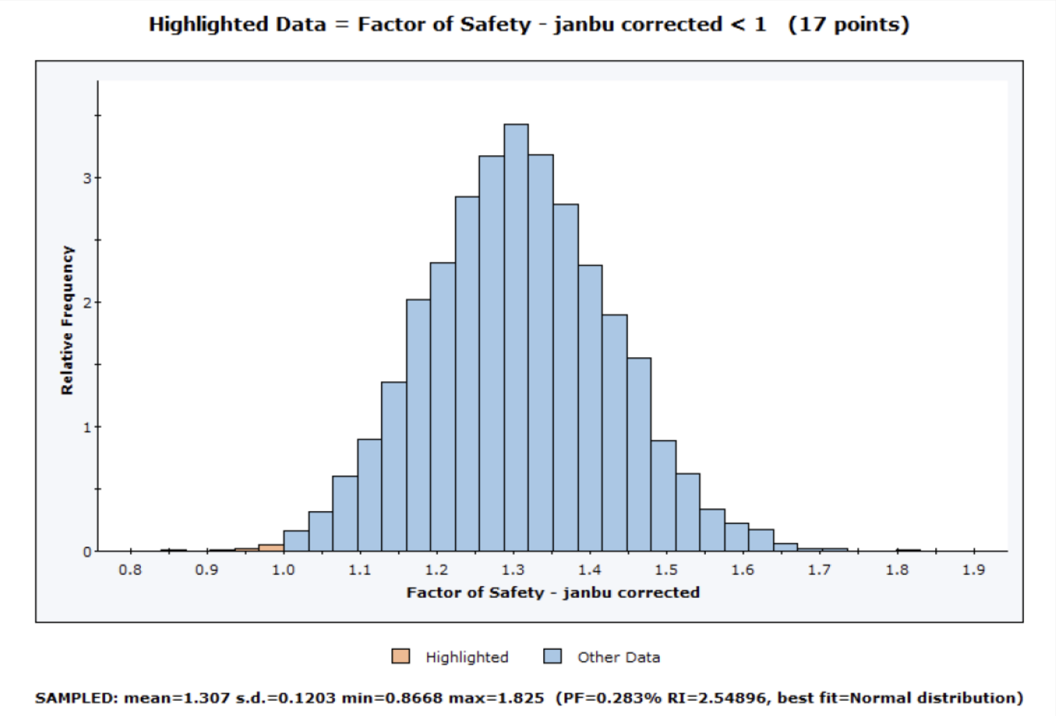


Probability FS ≤ 1 = 29.5%

Section S4_NE10&NE11 – 200 years – 30m raise of pit lake level - BB: 7m



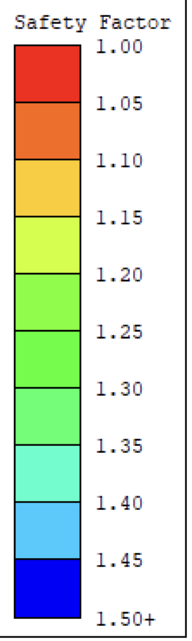


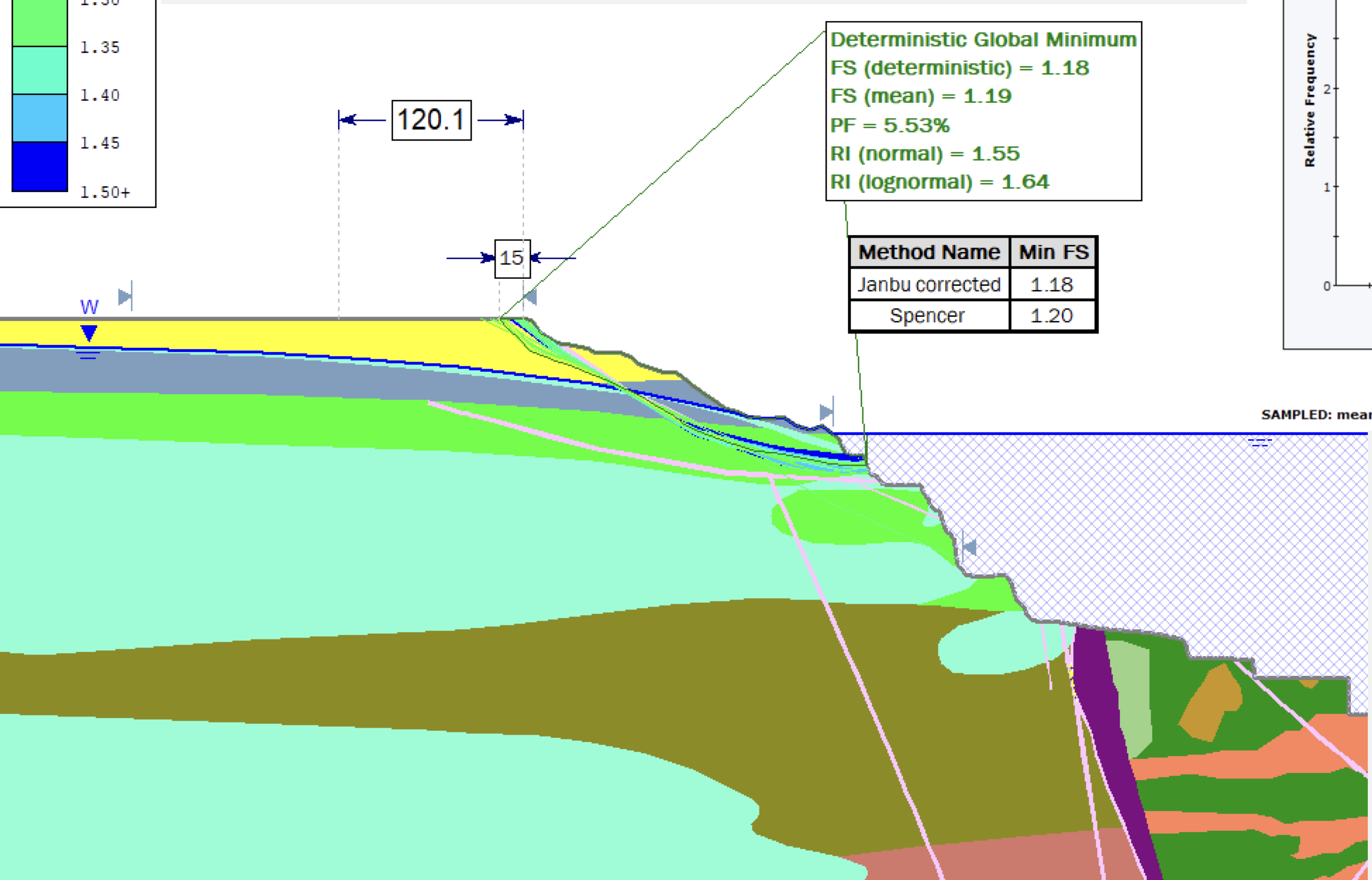


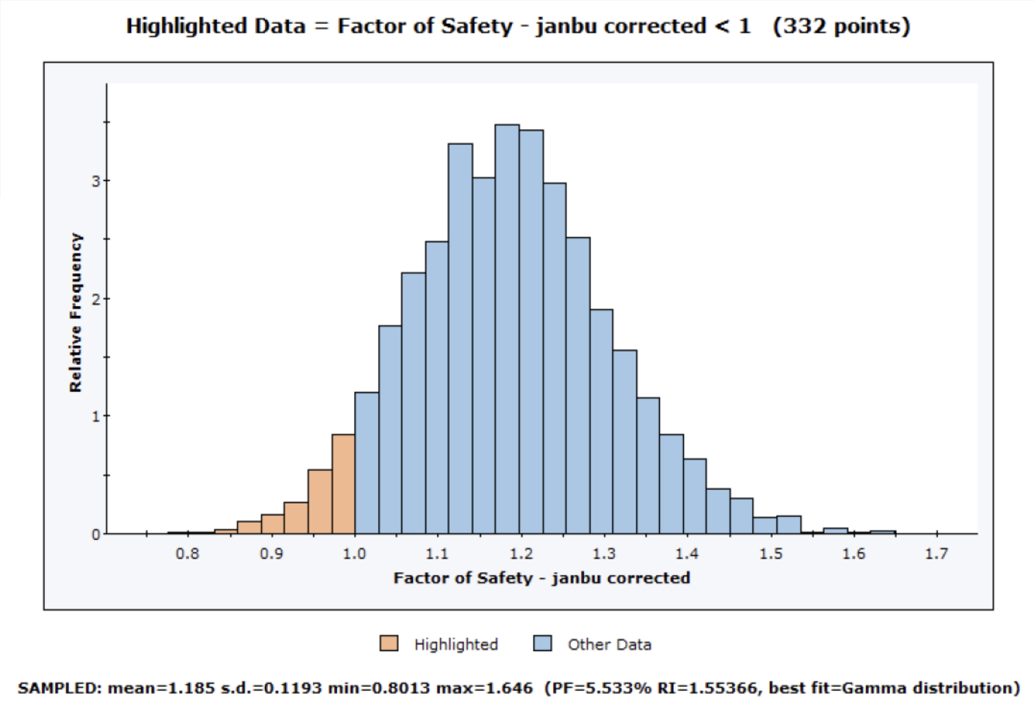
Probability FS ≤ 1 = 0.3%

Section S7_N11&N12
Special Conditions

Section S7_N11&N12 – 200 years – base case revised wl - BB: 15m

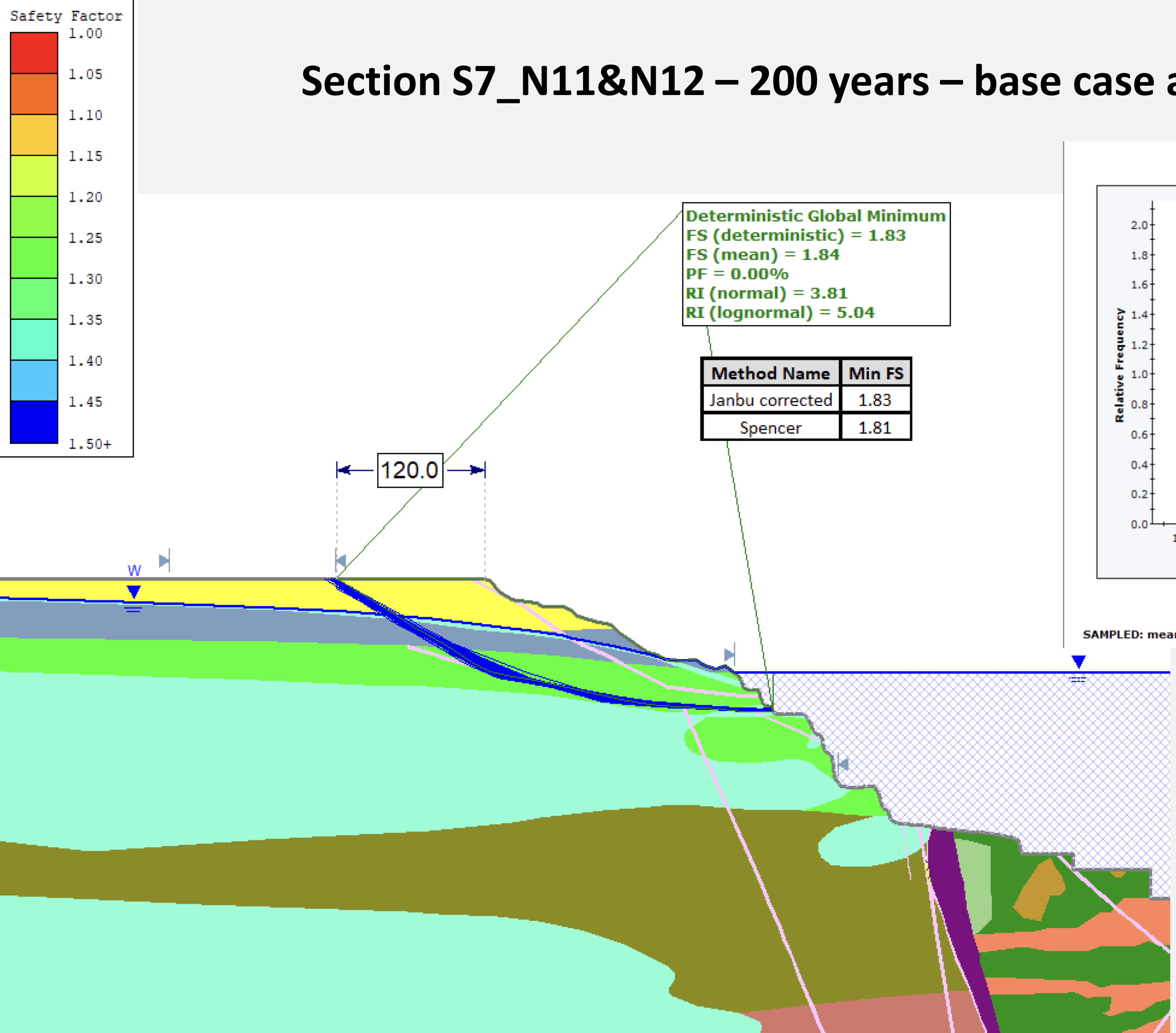




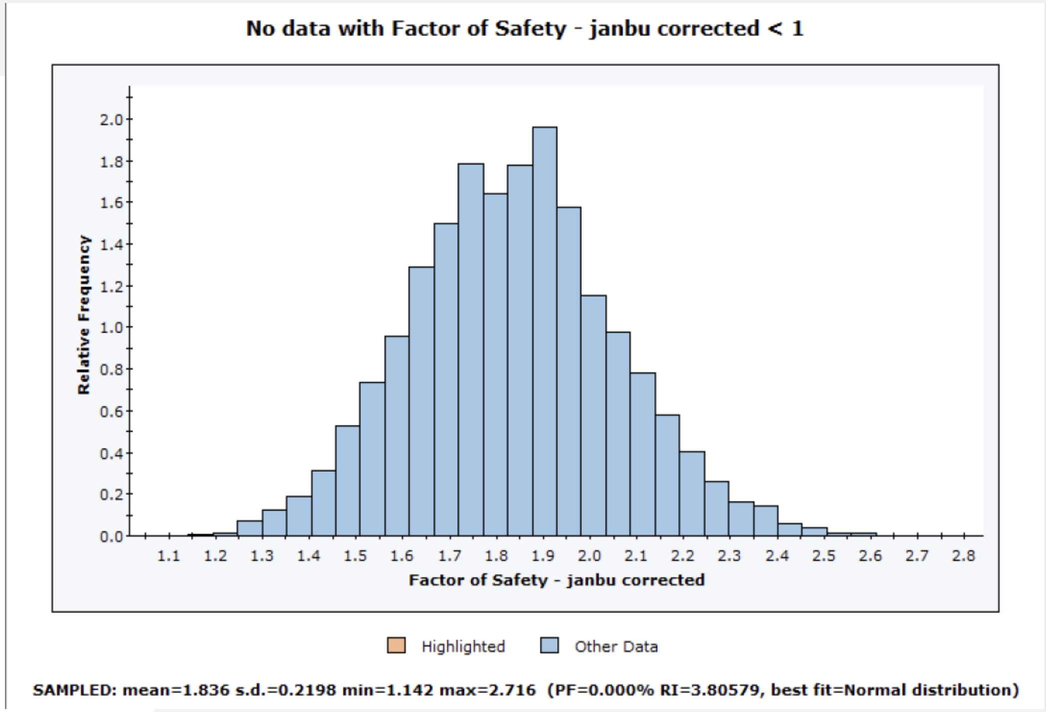


Probability FS ≤ 1 = 5.5%

Section S7_N11&N12 – 200 years – base case at fence - BB: 120m

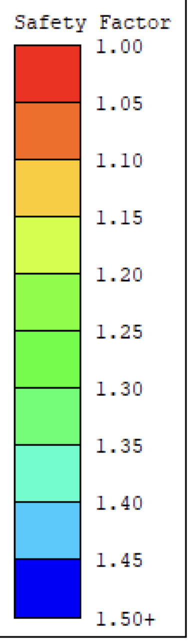


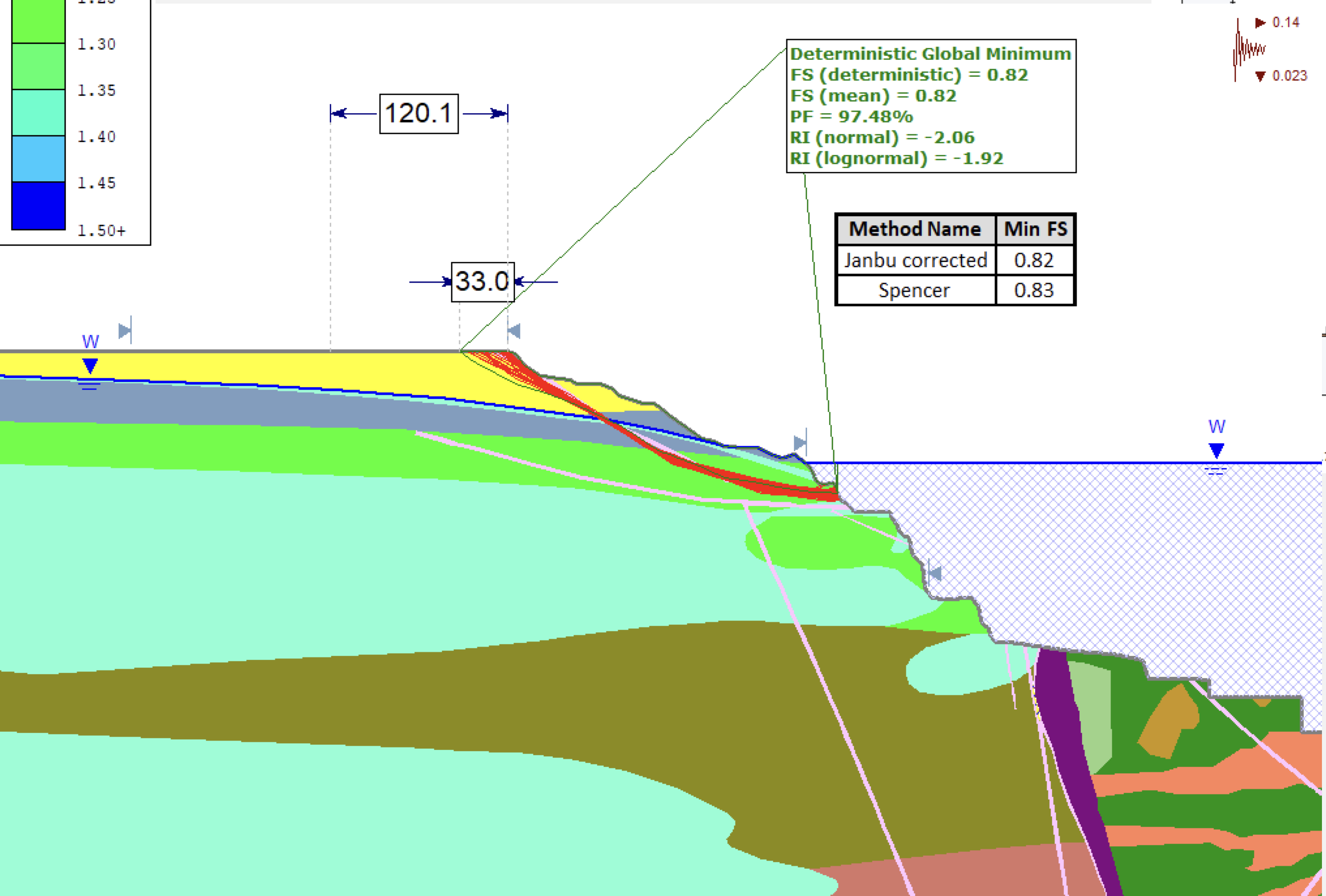
Method Name	Min FS
Janbu corrected	1.83
Spencer	1.81

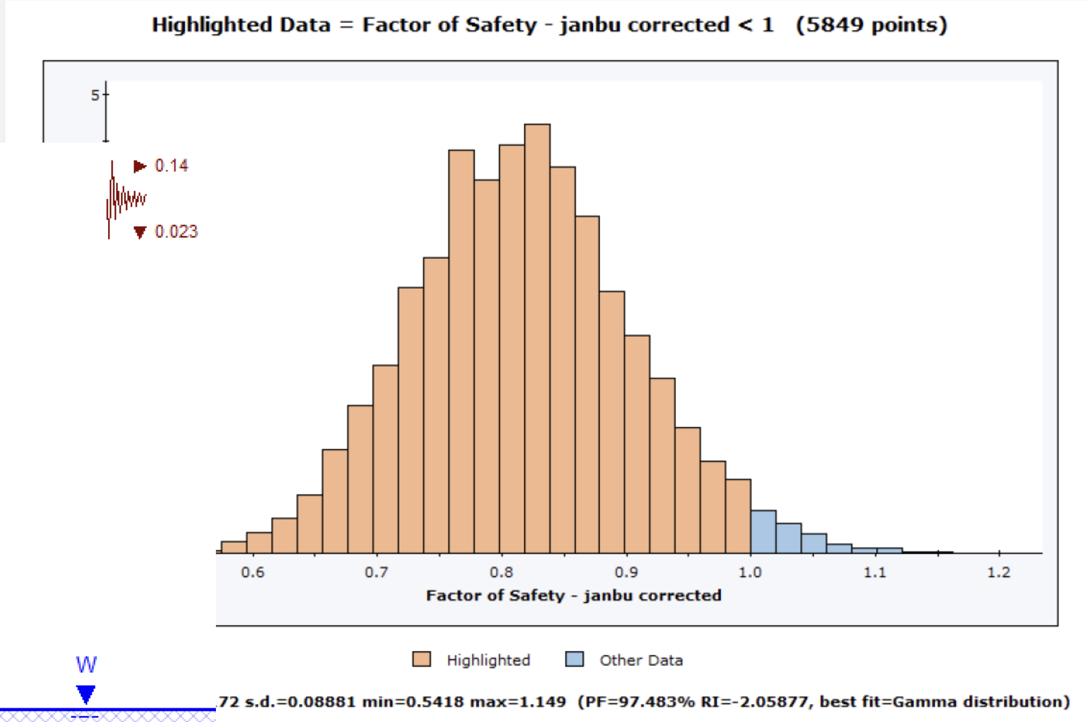


Probability FS ≤ 1 = 0.0%

Section S7_N11&N12 – 200 years – seismic loading - BB: 33m

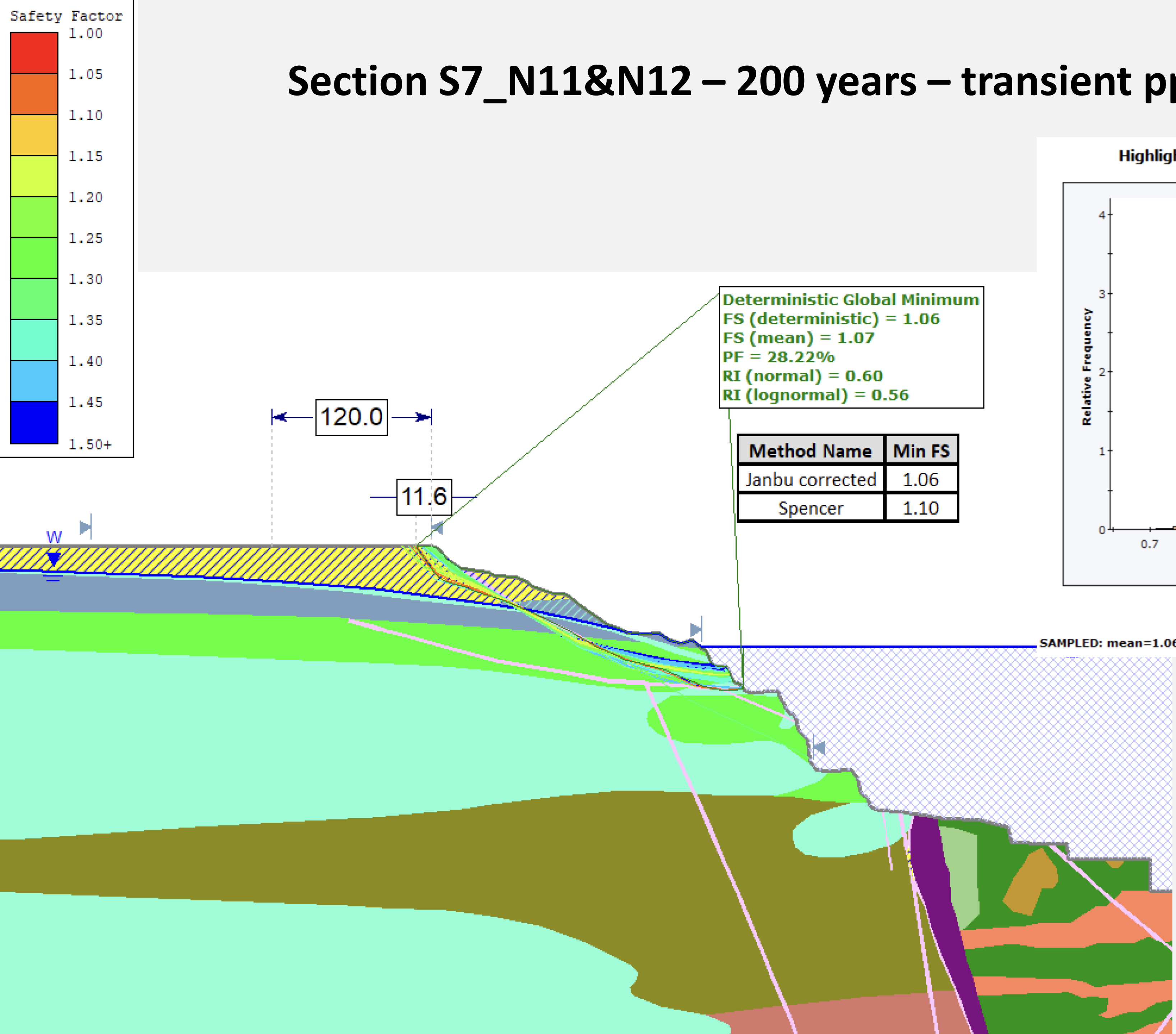




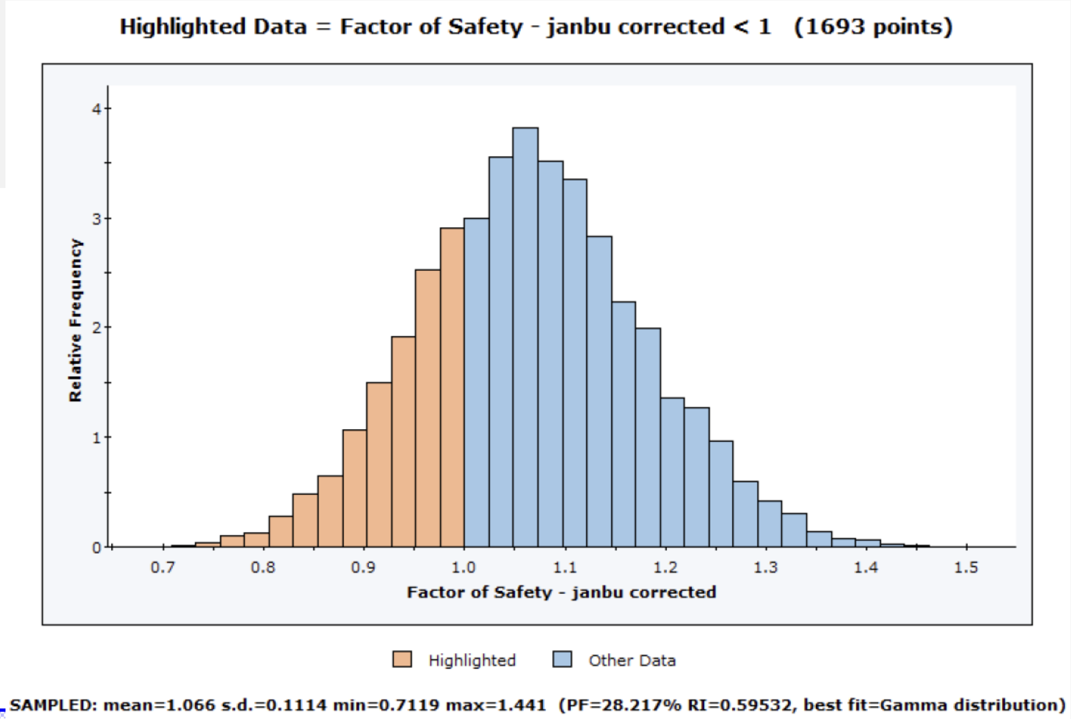


Probability FS ≤ 1 = 97.5%

Section S7_N11&N12 – 200 years – transient pp Ru=0.15 - BB: 12m

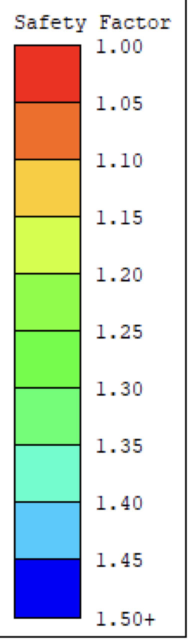


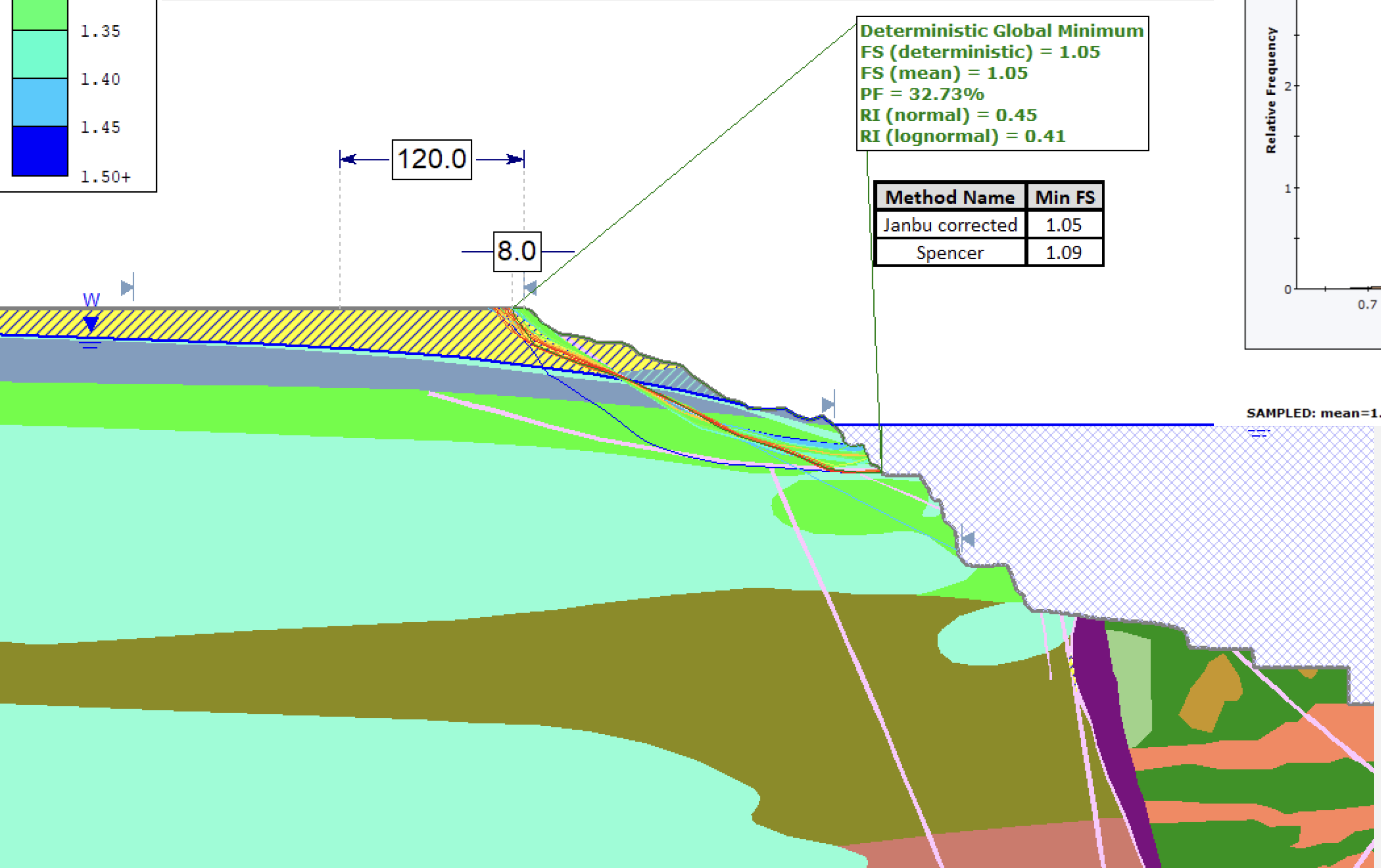
Method Name	Min FS
Janbu corrected	1.06
Spencer	1.10

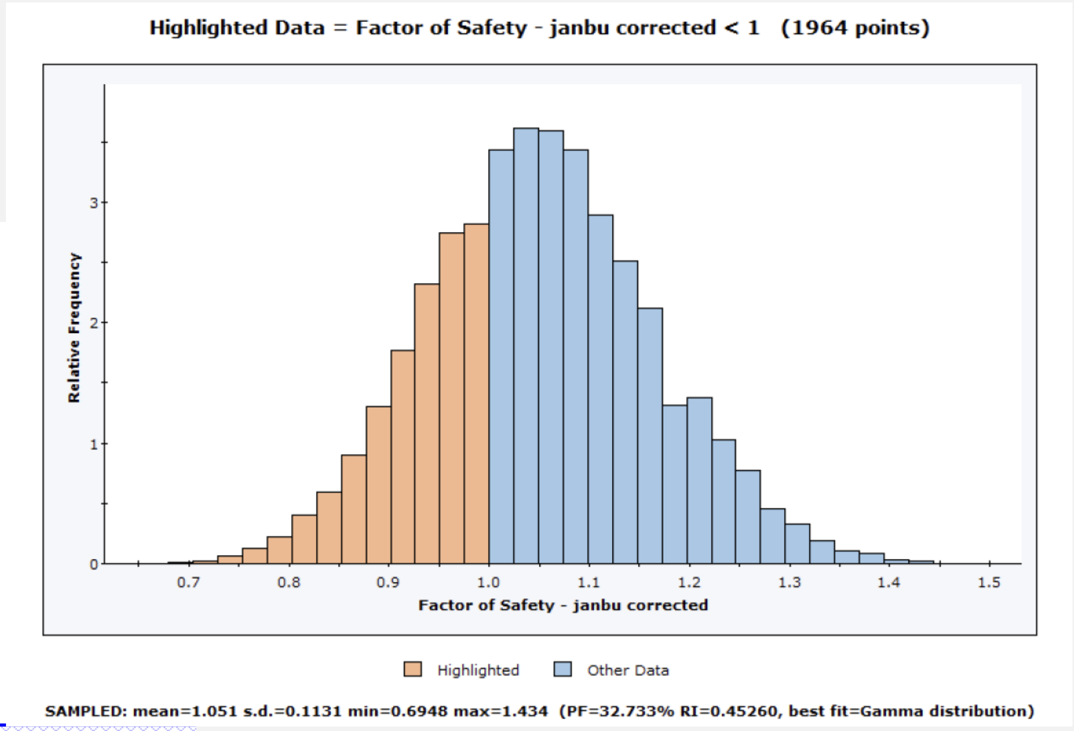


Probability FS ≤ 1 = 28.2%

Section S7_N11&N12 – 200 years – transient pp Ru=0.30 - BB: 12m

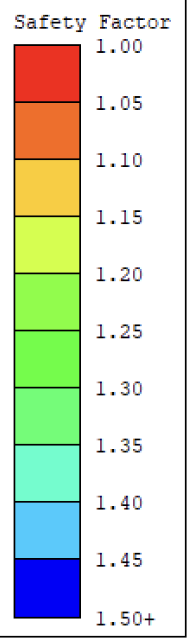


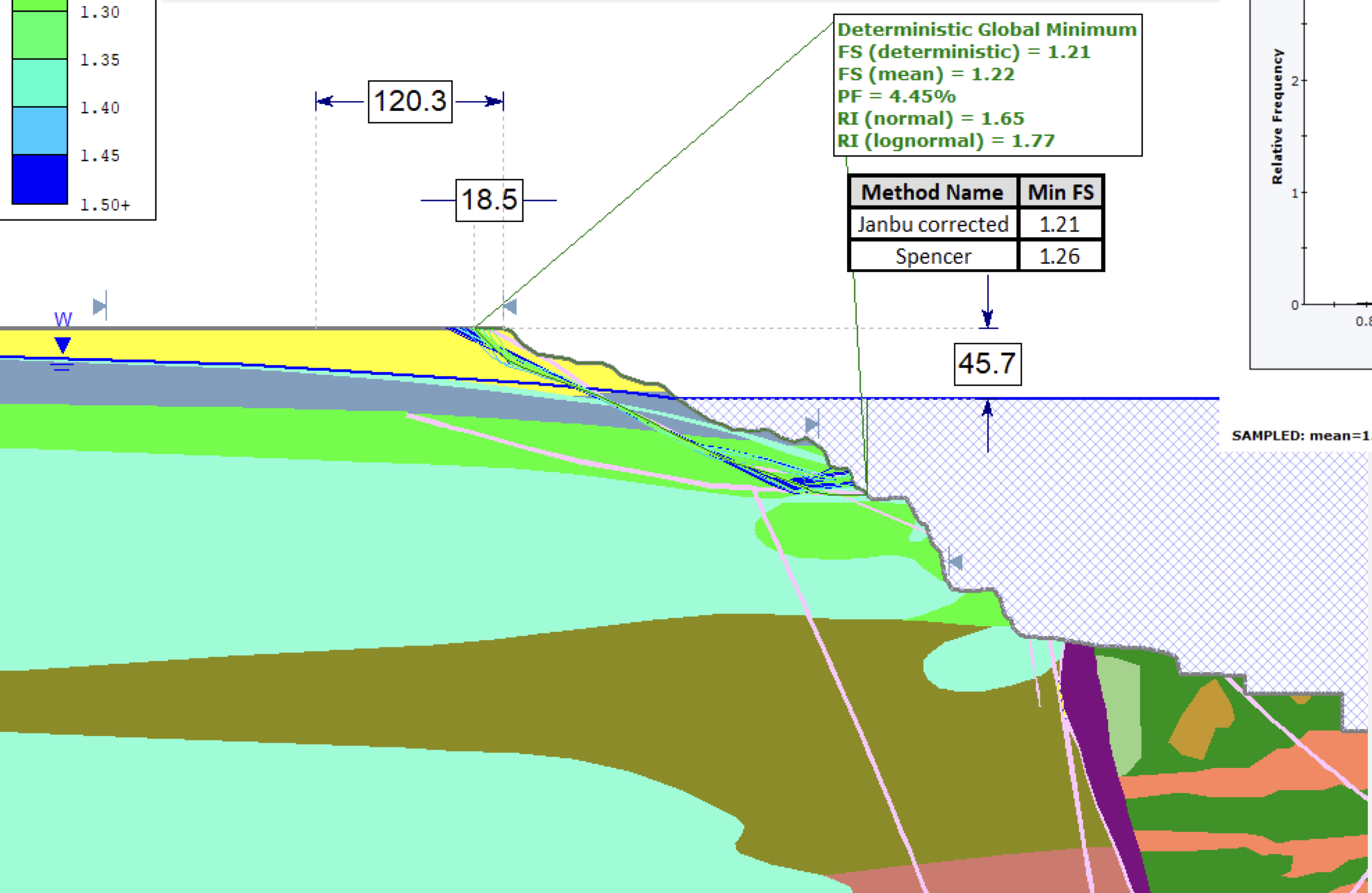




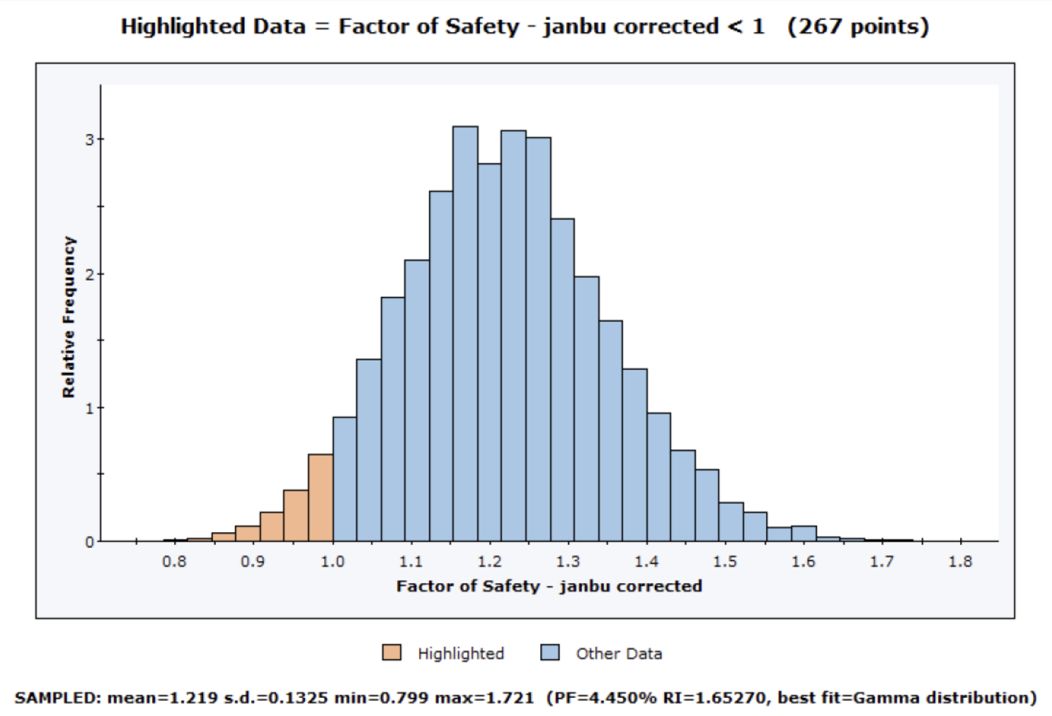
Probability FS ≤ 1 = 32.7%

Section S7_N11&N12 – 200 years – 30m raise of pit lake level - BB: 19m





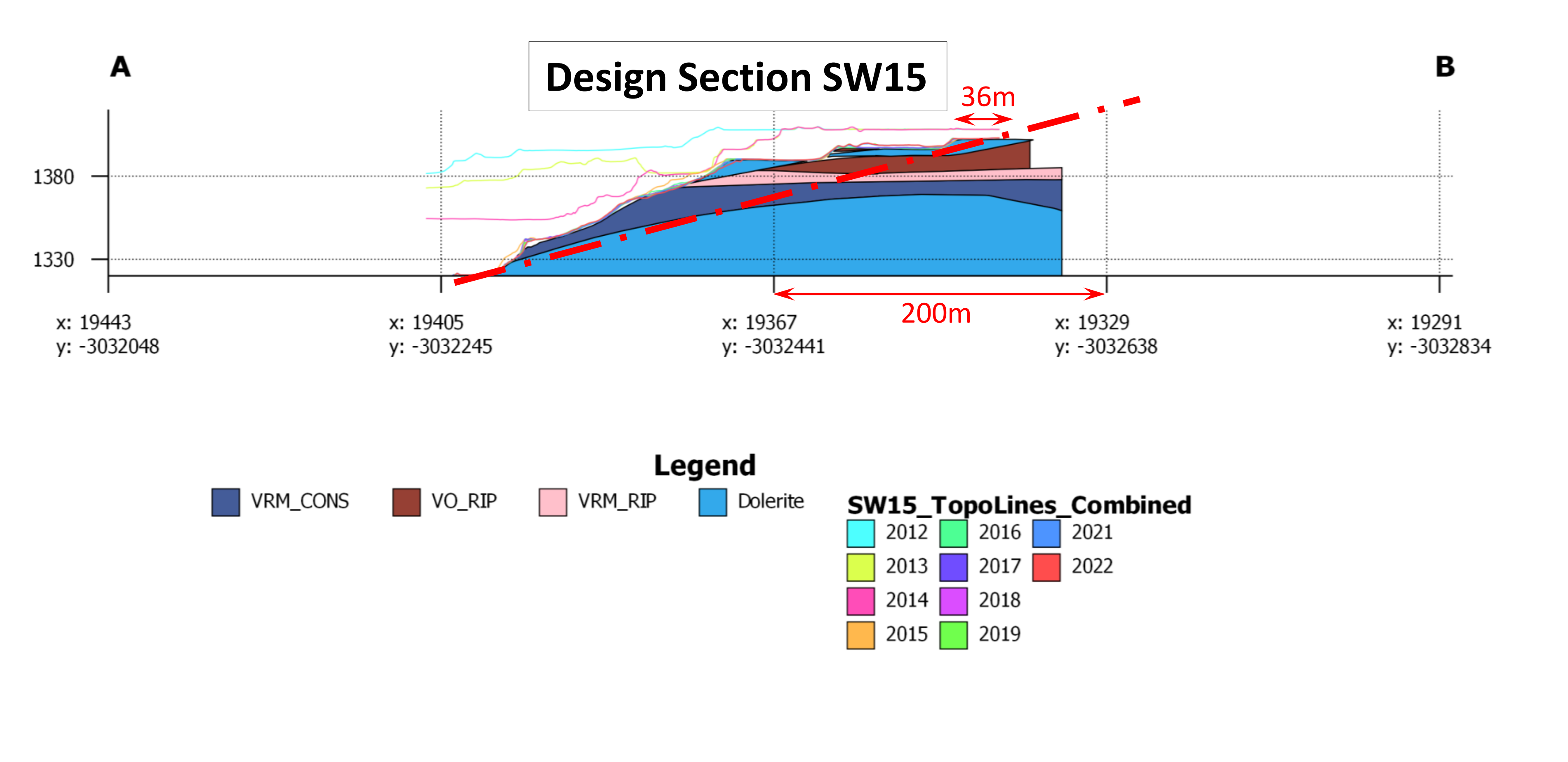
Method Name	Min FS
Janbu corrected	1.21
Spencer	1.26



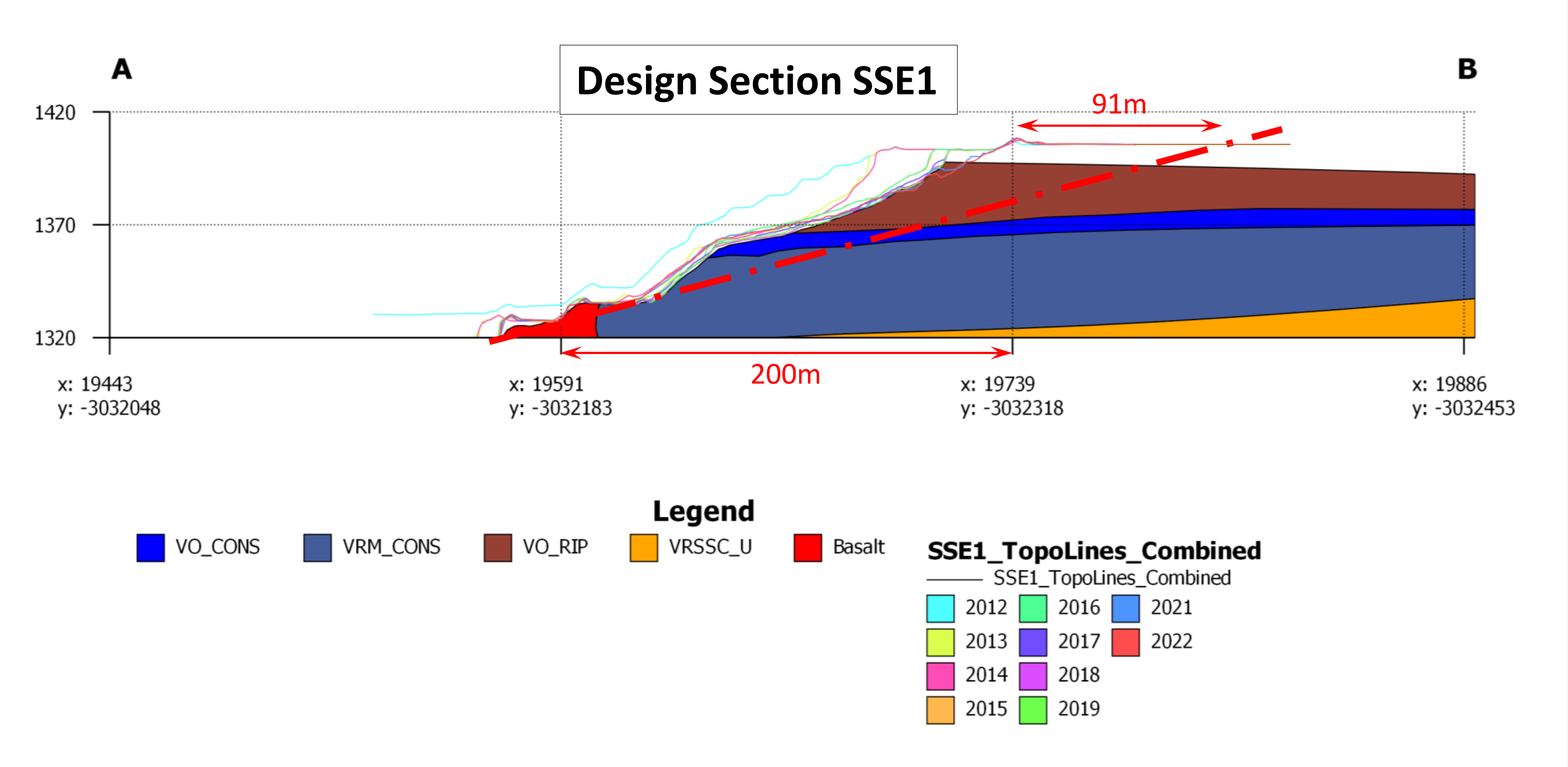
Probability FS ≤ 1 = 4.5%

APPENDIX E: HISTORICAL RECORD OF BREAK-BACK ON SELECT DESIGN SECTIONS

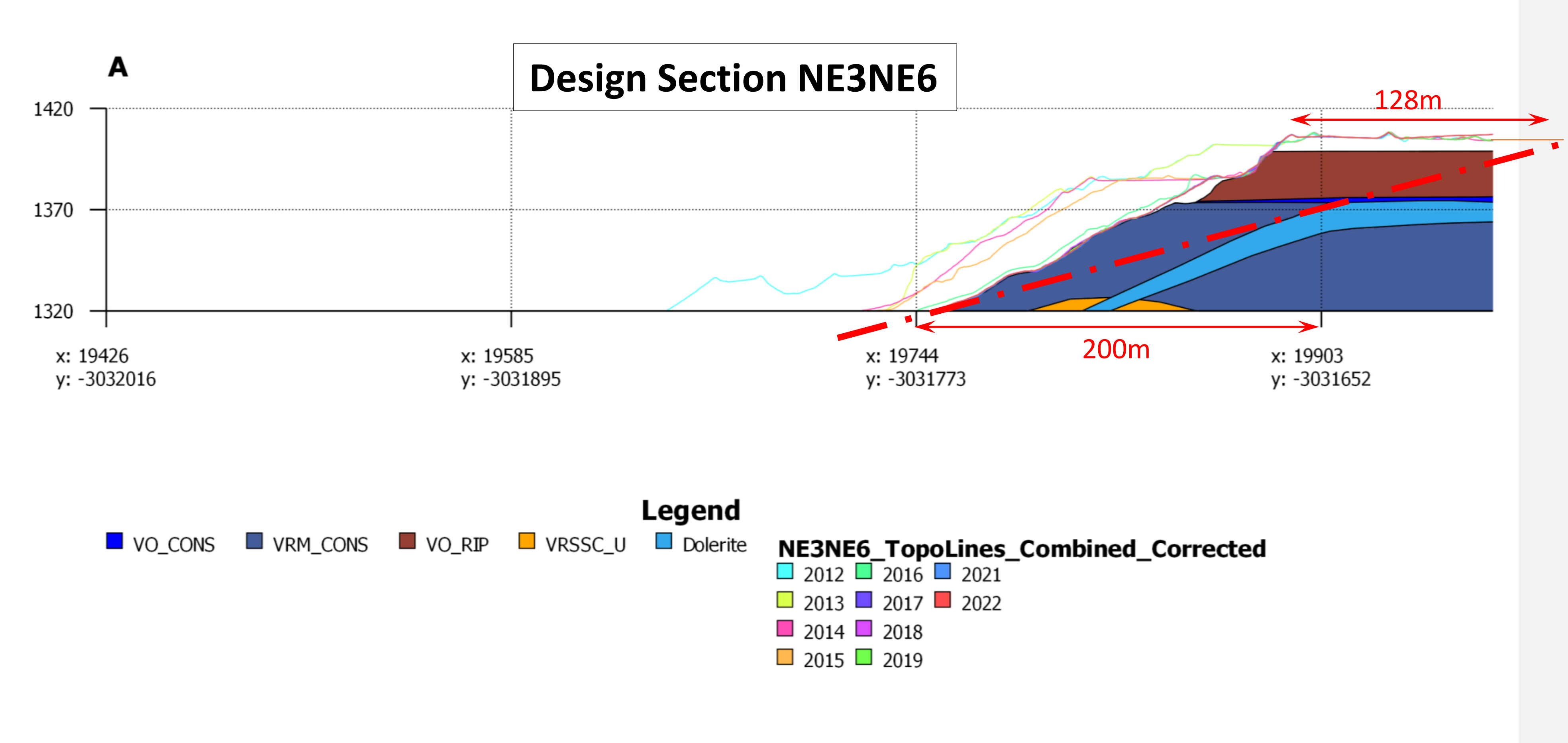
Annual Pit Slope Regression based on LiDAR Scans



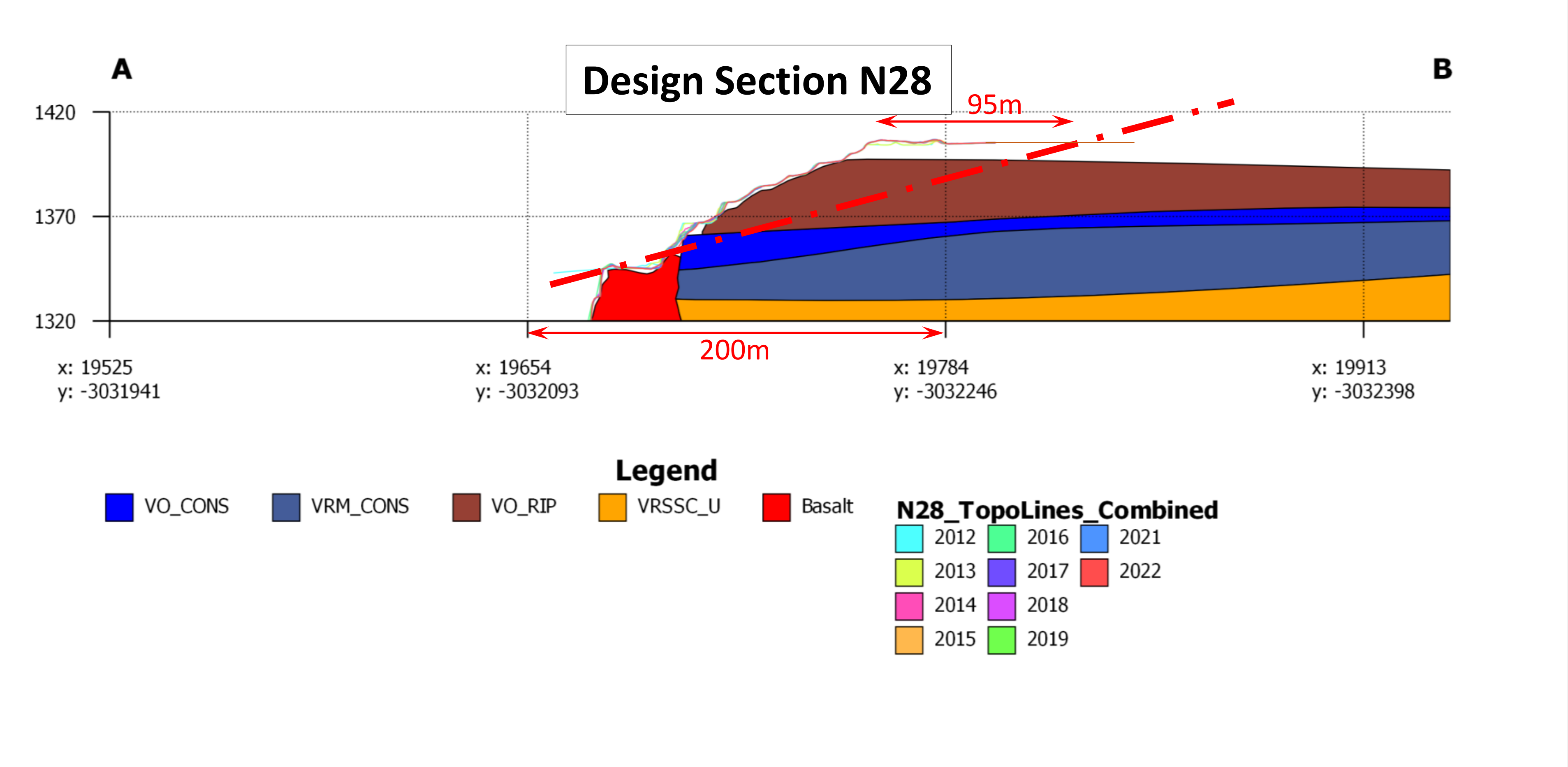
Annual Pit Slope Regression based on LiDAR Scans



Annual Pit Slope Regression based on LiDAR Scans



Annual Pit Slope Regression based on LiDAR Scans



Annual Pit Slope Regression based on LiDAR Scans