



الجمهورية الجزائرية الديمقراطية الشعبية
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قسم المناجم والجيوتكنولوجيا
Département Mines Et Géotechnologie



MEMOIRE

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Filière : Génie minier

Option : Géotechnique

Geotechnical characterization and heterogeneous slope stability modeling Affected by Diapiric formations

Par

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Chapitres réalisés	Signature de l'encadreur
Chap.01 : Slope stability analysis	
Chap.02: Geological and geomorphological settings	
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مؤسسة التعليم العالي : جامعة العربي التبسي – تبسة

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خاص بالالتزام بقواعد النزاهة العلمية لانجاز بحث

أنا الممضي أدناه،

السيد جابري أحمد ياسين الصفة : طالب

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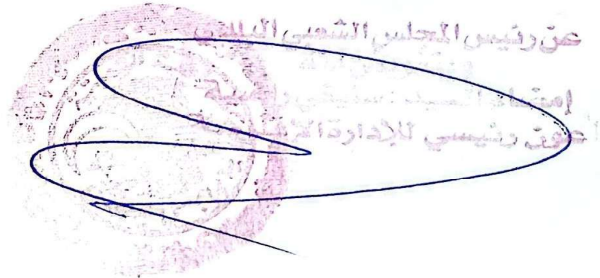
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و المكلف بانجاز أعمال بحث مذكرة التخرج عنوانها :

**Geotechnical Characterization and Heterogeneous Slope Stability Modeling
Affected by Diapiric Formations**

أصرح بشرفي أنني ألتزم بمراعاة المعايير العلمية و المنهجية و معايير الأخلاقيات المهنية و النزاهة
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جامعة العربي التبسي - تبسة



مقرر رقم: 019 مؤرخ في: 04 جوان 2026

يتضمن تعيين لجنة مناقشة مذكرة الماستر

إن مدير جامعة العربي التبسي بتبسة.

- بموجب القرار الوزاري رقم 318 و المؤرخ في 05 ماي 2021 المتضمن تعيين السيد "قواسمية عبد الكريم" مديرا لجامعة العربي التبسي - تبسة.

- و بمقتضى المرسوم التنفيذي رقم: 12-363 مؤرخ في 8 أكتوبر 2012، يعدل ويتم المرسوم التنفيذي رقم 09-08 المؤرخ في: 04 جانفي 2009 و المتضمن إنشاء جامعة العربي التبسي بتبسة.

- و بمقتضى المرسوم التنفيذي رقم 08-265 المؤرخ في 17 شعبان عام 1429 الموافق 19 غشت سنة 2008 الذي يحدد نظام الدراسات للحصول على شهادة الليسانس وشهادة الماستر وشهادة الدكتوراه، لاسيما المادة 9 منه.

- وبموجب القرار رقم 362 المؤرخ في 09 جوان 2014 الذي يحدد كفاءات إعداد ومناقشة مذكرة الماستر. لاسيما المادتان 10 و 11 منه.

- وبموجب القرار رقم 1380 المؤرخ في 09 أوت 2016 والمتضمن مواءمة التكوينات في الماستر بعنوان جامعة تبسة في ميدان "علوم وتكنولوجيا".

- وبموجب القرار رقم 921 المؤرخ في 25 أوت 2021 المعدل للحق القرار 1080 المؤرخ في 13 أكتوبر 2015 والمتضمن مواءمة التكوينات في الماستر بعنوان جامعة تبسة في ميدان "علوم وتكنولوجيا"، اختصاص جيوتقني

- وبعد الاطلاع على محضر المجلس العلمي لمعهد المناجم رقم 03 المؤرخ في 2026/05/24 .

يقرر ما يأتي:

المادة الأولى: تُعيّن بموجب هذا المقرر لجنة مناقشة مذكرة الماستر المحضرة من طرف الطالب : جابري أحمد ياسين.

المولود بتاريخ 2004/04/26 بتبسة .

والمسؤومة ب : Geotechnical characterization and heterogeneous slope stability modeling affected by Diapiric formations

والمسجل بمعهد المناجم

المادة 2: تتشكل اللجنة المشار إليها في المادة الأولى من الأعضاء الآتي ذكرهم:

رقم	الاسم واللقب	الرتبة	مؤسسة الانتماء	الصفة
1	فوزي مبروك	أستاذ مساعد - أ-	جامعة العربي التبسي - تبسة	رئيسا
2	ياسين براح	أستاذ محاضر - أ	جامعة العربي التبسي - تبسة	مشرفا
3	عادل جلال	أستاذ	جامعة العربي التبسي - تبسة	ممتحنا

المادة 3: يكلف رئيس قسم المناجم والجيوتكنولوجيا بتنفيذ هذا المقرر الذي يُسلم نسخة عنه إلى كل من الطالب المعني والمشرف على المذكرة وأعضاء لجنة المناقشة فور توقيعه.

المادة 4: تحفظ نسخة عن هذا المقرر في الملف البيداغوجي للطلاب المعني، وينشر في النشرة الرسمية لجامعة العربي التبسي.

عن المدير ويتفويض منه
مدير معهد المناجم



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وزارة التعليم العالي والبحث العلمي
جامعة العربي التبسي - تبسة



مقرر رقم: (٢) مؤرخ في: 04 جوان 2026

يتضمن الترخيص بمناقشة مذكرة الماستر

إن مدير جامعة العربي التبسي بتبسة،

- بموجب القرار الوزاري رقم 318 و المؤرخ في 05 ماي 2021 المتضمن تعيين السيد "قواسمية عبد الكريم" مديرا لجامعة العربي التبسي - تبسة،

- وبمقتضى المرسوم التنفيذي رقم: 12-363 مؤرخ في 8 أكتوبر 2012، يعدل ويتمم المرسوم التنفيذي رقم 09-08 المؤرخ في: 04 جانفي 2009 والمتضمن إنشاء جامعة العربي التبسي بتبسة،

- وبمقتضى المرسوم التنفيذي رقم 08-265 المؤرخ في 17 شعبان عام 1429 الموافق 19 غشت سنة 2008 الذي يحدد نظام الدراسات للحصول على شهادة الليسانس وشهادة الماستر وشهادة الدكتوراه، لاسيما المادة 9 منه،

- وبموجب القرار رقم 362 المؤرخ في 09 جوان 2014 الذي يحدد كفايات إعداد ومناقشة مذكرة الماستر، لاسيما المادة 7 منه،

- وبموجب القرار رقم 1080 المؤرخ في 13 أكتوبر 2015 والمتضمن تأهيل ماستر الفروع ذات تسجيل وطني بجامعة تبسة،

- وبموجب القرار رقم 375 المؤرخ في 15 جوان 2020 المعدل للمحق القرار 1080 المؤرخ في 13 أكتوبر 2015 والمتضمن تأهيل ماستر الفروع ذات تسجيل وطني بجامعة تبسة، اختصاص جيوتقني

- وبموجب المقرر رقم 14.2 المؤرخ في 04 جوان 2026 والمتضمن تعيين لجنة مناقشة مذكرة الماستر، وبعد الاطلاع على مقرر تعيين لجنة مناقشة مذكرة الماستر المؤرخ في 04 جوان 2026،

يقرر ما يأتي:

المادة الأولى: يُرخص للطالب جابري أحمد ياسين،

المولود بتاريخ 2004/04/26 بتبسة، بمناقشة مذكرة الماستر والموسومة بـ:

**Geotechnical characterization and heterogeneous slope stability modeling
affected by Diapiric formations**

المادة 2: يكلف رئيس قسم المناجم والجيوتكنولوجيا بتنفيذ هذا المقرر الذي يسلم نسخة عنه إلى الطالب المعني بالمناقشة وأعضاء لجنة المناقشة فور توقيعه، وبضمن نشره عبر فضاءات المؤسسة المادية والرقمية.

المادة 3: تُحفظ نسخة عن هذا المقرر ضمن الملف البيداغوجي للطالب المعني وينشر في النشرة الرسمية لجامعة العربي التبسي.

جُزّر ب تبسة، في:

عن المدير، ويتفويض منه
مدير معهد المناجم



Abstract

This work presents a stability analysis of a railway cut excavated through Triassic diapiric formations, where extreme lithological heterogeneity governs slope performance. Following initial instability, an integrated system of slope geometry reduction, shotcrete armoring, and nail reinforcement was evaluated using limit equilibrium and finite element methods (GEO5, PLAXIS 2D/3D), informed by geophysical surveys, borehole sampling, and laboratory testing. Under dry static conditions, all cross-sections satisfy stability criteria with an unsupported factor of safety (FoS) of 2.145, decreasing marginally to 2.107 post-reinforcement. The Turonian limestone is self-supporting with displacements $\leq 10^{-3} m$. Site characterization reveals severe hazards: Triassic gypsum-rich clays exhibit sulphate concentrations of 49,000 – 69,000 *mg/kg* (DS-5) and gypsum content up to 12.35%; weathered limestones contain karstic voids forcing a Class V (Very Poor) rating despite intact UCS of 19.9 – 140.5 *MPa*; Mio-Pliocene clays show plasticity indices up to 26.9%.

Critically, the trench is catastrophically vulnerable to rainfall-induced flow failure without waterproofing. Under transient hydraulic loading, vulnerable profiles reach failure within 0.0026 to 0.01645 days (3.744 min – 23.688 min), driven by excess pore pressures of 108.8 – 823.97 *kN/m²* (2D) and peak values of 4.31×10^4 *kN/m²* (3D). The integrated hydraulic control system—geomembrane, sulfate-resistant shotcrete, and drainage—maintains dry conditions, limiting displacements to 17.74 cm with pore pressure dissipation from 1400 *kN/m²* to 1060 *kN/m²* within one day. This regional displacement does not extend its effects to the structure's domain; the local displacements simulated within the structure's domain do not exceed, according to the model, a value of 10 mm, which falls within the safe design limits imposed by railway specifications. However, these results remain subject to the model's assumptions, necessitating field verification through an automated monitoring system. It was also shown that structural reinforcement alone is insufficient unless supported by effective hydraulic isolation.

Keywords: Slope stability; Triassic diapir; finite element method; rainfall-induced failure; gypsum dissolution; railway cut

الملخص

يقدم هذا العمل دراسة لاستقرار خندق سكة حديدية محفور عبر تكوينات تريبسية ديابيرية، حيث تتحكم اللاتجانسية الليثولوجية الشديدة بصورة أساسية في سلوك واستقرار المنحدرات. وبعد تسجيل حالات عدم استقرار أولية، تم تقييم نظام متكامل يضم تخفيض هندسة المنحدر، وتدعيم السطح بالخرسانة المرشوشة، والتسليح بالمسامير الأرضية، وذلك باستخدام طريقتي توازن الحدود والعناصر المحددة) برنامجا GEO5 و PLAXIS 2D/3D، بالاعتماد على نتائج المسوحات الجيوفيزيائية، وأعمال الحفر وأخذ العينات، والاختبارات المخبرية.

في الظروف الجافة والتحميلات الساكنة، استوفت جميع المقاطع المدروسة معايير الاستقرار، حيث بلغ معامل الأمان (FoS) في الحالة غير المدعمة 2.145، وانخفض بشكل طفيف إلى 2.107 بعد تنفيذ أعمال التدعيم. كما أظهرت طبقة الحجر الجيري التوروني قدرة ذاتية على الاستناد، مع إزاحات لا تتجاوز 10^{-3} م. وكشفت الدراسة الجيوتقنية للموقع عن وجود مخاطر كبيرة؛ إذ تحتوي الأطنان التريبسية الغنية بالجبس على تراكيز كبريتات تتراوح بين 49,000 و 69,000 ملغ/كغ) العينة (DS-5، مع نسبة جبس تصل إلى 12.35%. كما تحتوي الأحجار الجيرية المتجوعة على فراغات كارستية أدت إلى تصنيفها ضمن الفئة الخامسة (ضعيفة جداً)، رغم أن مقاومة الضغط أحادي المحور للصخر السليم تتراوح بين 19.9 و 140.5 ميغاباسكال. بالإضافة إلى ذلك، أظهرت أطنان الميو-بليوسين مؤشرات لدونة تصل إلى 26.9%.

وتُظهر النتائج بصورة حاسمة أن الخندق معرض بشكل بالغ للانهييار الانسيابي الناتج عن هطول الأمطار في حال غياب العزل المائي. فعند التعرض لأحمال هيدروليكية غير مستقرة، تصل المقاطع الأكثر حساسية إلى الإنهيار خلال فترة تتراوح بين 0.0026 يوم و 0.01645 يوم (أي ما يعادل 3.744 دقيقة إلى 23.688 دقيقة)، نتيجة تطور ضغوط مائية مسامية فائضة تتراوح بين 108.8 و 823.97 كيلو نيوتن/م² في النماذج ثنائية الأبعاد، وتبلغ قيمة قصوى تصل إلى 4.31×10^4 كيلو نيوتن/م² في النماذج ثلاثية الأبعاد. وقد أثبت نظام التحكم الهيدروليكي المتكامل — المكوّن من غشاء عازل للمياه، وخرسانة مرشوشة مقاومة للكبريتات، ونظام تصريف — فعاليته في الحفاظ على ظروف جافة، حيث حصر الإزاحات في حدود 17.74 سم، مع خفض ضغط الماء المسامي من 1400 كيلو نيوتن/م² إلى 1060 كيلو نيوتن/م² خلال يوم واحد. ولا تمتد تأثيرات هذه الإزاحة الإقليمية إلى نطاق المنشأة؛ فالإزاحات الموضوعية المحاكاة في نطاق المنشأة لا تتجاوز، وفق النموذج، قيمة 10 ملم، وهي تقع ضمن الحدود التصميمية الآمنة التي تفرضها مواصفات السكك الحديدية. غير أن هذه النتائج تظل محكومة بافتراضات النموذج، مما يستوجب التحقق الميداني عبر نظام مراقبة آلي. كما تبين أن التدعيم الإنشائي وحده غير كافٍ ما لم يُدعم بعزل هيدروليكي فعال.

وعليه، فإن التصميم الإلزامي المقترح يتكوّن من ثلاثة مستويات حماية متكاملة: غشاء عازل أولي فوق الأطنان التريبسية، وطبقة ثانوية من الخرسانة المرشوشة المقاومة للكبريتات، ونظام تصريف محيطي ثالثي.

الكلمات المفتاحية: استقرار المنحدرات؛ الديابير التريبسي؛ طريقة العناصر المحددة؛ الانهيار الناتج عن هطول الأمطار؛ ذوبان الجبس؛ خندق سكة حديدية.

Résumé

Ce travail présente une analyse de stabilité d'une tranchée ferroviaire excavée dans des formations diapiriques triasiques, où l'extrême hétérogénéité lithologique contrôle le comportement des talus. À la suite d'une instabilité initiale, un système intégré comprenant la réduction de la géométrie des pentes, le revêtement en béton projeté et le renforcement par clouage a été évalué à l'aide des méthodes d'équilibre limite et des éléments finis (GEO5, PLAXIS 2D/3D), en s'appuyant sur des prospections géophysiques, des forages de reconnaissance et des essais de laboratoire.

Dans des conditions statiques sèches, toutes les sections étudiées satisfont aux critères de stabilité, avec un coefficient de sécurité (Fos) de 2,145 à l'état non renforcé, diminuant légèrement à 2,107 après la mise en place des renforcements. Le calcaire turonien présente un comportement autoportant avec des déplacements $\leq 10^{-3} m$. La caractérisation du site met en évidence plusieurs aléas majeurs : les argiles triasiques riches en gypse présentent des concentrations en sulfates comprises entre 49 000 et 69 000 mg/kg (DS-5) ainsi qu'une teneur en gypse pouvant atteindre 12,35 % ; les calcaires altérés renferment des cavités karstiques conduisant à une classification de Classe V (Très mauvaise qualité) malgré une résistance à la compression simple (UCS) comprise entre 19,9 et 140,5 MPa ; les argiles mio-pliocènes affichent des indices de plasticité atteignant 26,9 %.

De manière critique, la tranchée est extrêmement vulnérable aux ruptures par écoulement induites par les précipitations en l'absence d'un système d'étanchéité. Sous chargement hydraulique transitoire, les profils les plus sensibles atteignent la rupture en seulement 0,0026 à 0,01645 jour (3.744 min – 23.688 min), sous l'effet de surpressions interstitielles comprises entre 108,8 et 823,97 kN/m² (modèles 2D) et pouvant atteindre des valeurs maximales de $4,31 \times 10^4$ kN/m² (modèles 3D). Le système intégré de contrôle hydraulique — constitué d'une géomembrane, d'un béton projeté résistant aux sulfates et d'un réseau de drainage — permet de maintenir des conditions sèches, en limitant les déplacements à 17,74 cm et en réduisant la pression interstitielle de 1400 kN/m² à 1060 kN/m² en une journée. Ce déplacement régional ne propage pas ses effets au domaine de la structure ; les déplacements locaux simulés dans le domaine de la structure ne dépassent pas, selon le modèle, une valeur de 10 mm, ce qui se situe dans les limites de conception sécuritaires imposées par les spécifications ferroviaires. Cependant, ces résultats restent soumis aux hypothèses du modèle, ce qui nécessite une vérification sur le terrain via un système de surveillance automatisé. Il a également été démontré que le renforcement structurel seul est insuffisant à moins d'être accompagné d'une isolation hydraulique efficace.

Mots-clés : stabilité des talus ; diapir triasique ; méthode des éléments finis ; rupture induite par les précipitations ; dissolution du gypse ; tranchée ferroviaire.

الشكر والإهداء

بسم الله الرحمن الرحيم

الحمد لله الذي بنعمته تتم الصالحات، وله الفضل والإحسان في كل خطوة أخطوها في طلب العلم. أحمده سبحانه وتعالى على ما أعانني وأمدني بالقوة والصبر لإتمام هذا العمل المتواضع، وأشكره على أن من عليّ بنعمة العقل والإرادة.

بعد ذلك، أتقدم بخالص الشكر والتقدير إلى أستاذي الفاضل والمشرف المتميز **الدكتور ياسين براح**، الذي لم ييخل عليّ بوقته ولا بعلمه، وكان دليلي وناصري الأمين في كل مرحلة من مراحل هذا البحث. ساهم توجيهه الدقيق، وملاحظاته الثرية، ودعمه المستمر في إخراج هذه الدراسة إلى حيز الوجود. له مني كل الامتنان والعرفان.

كما أتوجه بالشكر الجزيل إلى أعضاء لجنة المناقشة الكرام: **الدكتور فوزي مبروك** على قبوله رئاسة اللجنة، و**البروفيسور عادل جلاي** لتكريمه بتقييم هذا العمل. أسأل الله أن يجزيهم خير الجزاء على ما يبذلونه من جهد في سبيل خدمة العلم والطلبة.

شكري الخاص موجه إلى كل أساتذتي في **قسم المناجم والجيوتكنولوجيا** بجامعة الشهيد الشيخ العربي التبسي – تبسة، الذين كان لهم فضل كبير في تكويني طوال سنوات الدراسة. كما لا يفوتني أن أشكر كل الفنيين والإداريين الذين سهّلوا مهمتي، سواء في المختبرات أو في المصالح الإدارية.

أود أن أعبر عن امتناني العميق لشركة **كوسيدار** على توفير البيانات الجيوتقنية الأساسية، وعلى دعمهما اللوجستي الذي مكّني من إجراء التحليلات العددية والنمذجة اللازمة. جزاهم الله خيراً على تعاونهم.

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Abbreviations

Abbreviation	Full form
DH	Depth to hard stratum (Taylor's method).
EM	Engineer Manual (USACE).
FEM	Finite Element Method.
FOS / FS	Factor of Safety.
GPS	Generalized Procedure of Slices (Janbu).
GSI	Geological Strength Index.
HB	Hoek-Brown failure criterion.
HS	Hardening Soil model.
HS small	Hardening Soil model with small-strain stiffness.
JR	Jointed Rock model.
LE	Limit Equilibrium.
LEM	Limit Equilibrium Method.
LoT	Line of Thrust.
MC	Mohr-Coulomb model.
OCR	Over-Consolidation Ratio.
PLAXIS	Finite element software for geotechnical engineering.
SS	Soft Soil model.
SSC	Soft Soil Creep model.
SSR	Shear Strength Reduction technique.
USACE	United States Army Corps of Engineers.

Symbol	Description	Unit
A	Skempton's pore pressure parameter	-
a	Attraction term in modified Mohr-Coulomb criterion	kN/m^2
B	Skempton's pore pressure parameter	—
b_i	Width of slice i	m
C_1	Weight influence term in Corps of Engrs. method	kN
C_2	Water pressure influence term in Corps of Engrs. method	kN

C_3	External loads term in Corps of Engrs. method	kN
C_4	Cohesion and pore pressure term in Corps of Engrs. method	kN
C	Soil cohesion	kN/m^2
c'	Effective cohesion	kN/m^2
c_u	Undrained cohesion (undrained shear strength)	kN/m^2
c_{si}	Cohesion at interslice surface	kN/m^2
c'_i	Effective cohesion at base of slice ii	kN/m^2
D	Disturbance factor (Hoek-Brown)	—
D_d	Horizontal lever arm (distance from centre of rotation to weight resultant)	m
d'	Lever arm for submerged weight component	m
d_i	Length of interslice surface	m
d_t	Lever arm of water thrust resultant	m
Δu	Excess pore water pressure increment	kN/m^2
E	Young's modulus	kN/m^2
E_i	Interslice normal force	kN
E_{50}	Secant stiffness modulus at 50% failure stress	kN/m^2
E_{oed}	Oedometer modulus	kN/m^2
E_{ur}	Unloading/reloading stiffness modulus	kN/m^2
F	Factor of safety (general)	—
F_c	Factor of safety for cohesion component	—
F_f	Final factor of safety (Janbu)	—
F_j	Factor of safety from Janbu simplified method	—
F_L	Local factor of safety at interslice surface (Sarma)	—
F_m	Factor of safety from force equilibrium	—
F_S	Factor of safety from moment equilibrium	—
F_{trial}	Force-equilibrium factor of safety (Janbu)	—
f_o	Trial reduction factor in SSR technique	—
$f(x)$	Factor of safety (general)	—
G_0	Janbu correction factor	—
H	Prescribed function for interslice force ratio (Morgenstern-Price)	—
H_i	Small-strain shear modulus	kN/m^2
K	Slope height	m
k_c	Height of slice ii	m
L	Constant depending on soil type (Janbu correction)	—
l_j	Critical horizontal seismic acceleration coefficient (Sarma)	—
M	Length of slip surface	m
$m_{\alpha i}$	Length of slice base	m

N_i	Moment	$kN \cdot m$
N_s	Bishop's stability coefficient	—
N	Normal force at base of slice	kN
N	Taylor's stability number	—
O	Denominator term in Corps of Engrs. method	—
P	Number of nodes (FEM)	—
P_{wi}	Centre of rotation (circular slip surface)	—
P_j	External point load	kN
R	Water pressure force on interslice surface	kN
R_i	Resultant normal force at slice base (Janbu)	kN
S	Radius of circular slip surface	m
s_u	Moment arm for slice ii	m
T	Soil slip code (landslide type)	—
T_i	Undrained shear strength	kN/m^2
U_i	Interslice shear force	kN
u	Mobilised shear force at base of slice	kN
u_i	Water pressure force on slice side	kN
W	Pore water pressure	kN/m^2
W_i	Pore water pressure at base of slice ii	kN/m^2
W_t	Total weight of sliding mass	kN
X_i	Weight of slice ii	kN
x_j	Weight of sliding mass including tension crack	kN
y_c	Interslice shear force	kN
Z_i	Nodal coordinate	m
Z_{i+1}	Horizontal distance from centre of rotation to water thrust resultant	m
z_c	Lateral interslice force (Lowe-Karafiath / Corps of Engrs.)	kN
z_i	Lateral force on downslope side of slice	kN
a	Height of thrust line above base of slice	m
V_p	Primary wave velocity	m/s
V_s	Shear wave velocity	m/s
G_{max}	Small-strain shear modulus (Dynamic)	kN/m^2
E_{max}	Maximum dynamic Young's modulus	kN/m^2
ν_{dyn}	Dynamic Poisson's ratio	—
TCR	Total Core Recovery	%
RQD	Rock Quality Designation	%
$E.O.H$	End of Hole	m
LL	Liquid Limit	%
PL	Plastic Limit	%

PI	Plasticity Index	%
w	Natural water content	%
G_s	Specific Gravity of soil solids	—
ρ_h	Bulk density of soil/rock	g/cm^3
ρ_d	Dry density of soil/rock	g/cm^3
C_c	Compression Index	—
C_s	Swelling (Recompression) Index	—
σ'_v	Effective vertical stress	kN/m^2
σ'_p	Pre-consolidation pressure	kN/m^2
λ^*	Modified compression index (Soft Soil model)	—
κ^*	Modified swelling index (Soft Soil model)	—
E_{50}^{ref}	Reference secant stiffness modulus	kN/m^2
σ_{ci}	Unconfined compressive strength of intact rock	kN/m^2
σ_t	Tensile strength of rock	kN/m^2
m_i	Intact rock parameter (Hoek-Brown)	—
E_{rm}	Rock mass deformation modulus	kN/m^2

General Introduction

In mountainous terrains underlain by diapiric formations, the excavation of deep railway trenches or cuts frequently encounters severe slope stability problems, stemming from the extreme lithological heterogeneity and contrasting mechanical behaviors of interbedded materials. The presence of Triassic diapiric complexes—typically comprising ductile, soluble gypsum-clay—juxtaposed against competent but fractured limestones and highly plastic clayey sequences, creates a critical geological environment where conventional stability assumptions often fail. For linear infrastructure such as phosphate transport railways, the construction of a cut through such formations demands not only rigorous geotechnical characterization but also a sophisticated modeling approach capable of capturing the spatial variability, anisotropic strength, and time-dependent weathering that govern failure mechanisms.

This dissertation presents a comprehensive stability analysis of a railway cut excavated across a mountain slope characterized by heterogeneous alternations of limestone, clayey formations, and Triassic diapiric materials. The primary focus is on evaluating the influence of geological alternation—where abrupt transitions between stiff, brittle limestones and soft, ductile clay-gypsum units occur over short distances—as a critical parameter controlling slope performance. Following initial instability observations, the project was stabilized using an integrated engineering solution comprising slope geometry reduction, projected shotcrete armoring, and systematic nail reinforcement, specifically designed to mitigate the adverse effects of rainfall-induced pore pressure fluctuations and surficial erosion. To achieve these objectives, the dissertation is structured into four sequential chapters as follows:

Chapter 1 establishes the geological and geomorphological context, detailing the origin, spatial distribution, and tectonic history of the Triassic diapiric formations, the interbedded limestone competent layers, and the overlying clayey sequences, with emphasis on their weathering profiles and discontinuity networks.

Chapter 2 addresses slope stability analysis, examining failure causality, trigger mechanisms (particularly hydrological and excavation-induced), and the limitations of conventional limit equilibrium methods when applied to strongly heterogeneous media.

Chapter 3 presents the geotechnical investigation program, integrating geophysical surveys (electrical resistivity tomography and seismic refraction), targeted borehole sampling, and laboratory testing—including triaxial and direct shear tests on clay-gypsum mixtures and limestone joints—to derive representative strength and stiffness parameters for each lithofacies. Finally, Chapter 4 details the geotechnical numerical modeling conducted in both limit equilibrium and finite element method using Geo5 and PLAXIS 2D/3D, simulating staged excavation, nail reinforcement, shotcrete lining, and transient rainfall infiltration. The models explicitly incorporate the heterogeneous alternation of materials and evaluate the post-stabilization performance under operational (train-induced) and environmental loading. The outcomes provide validated design recommendations for cut slopes in diapir-affected terrains, demonstrating how targeted stabilization measures can reliably manage the risks posed by geological heterogeneity.

Chapter 01 : Slope stability analysis

I.1. Introduction

The instability and subsequent degradation of both natural and anthropogenic slopes—such as deep cut excavations and high embankments—represent complex geomechanical phenomena primarily driven by gravitational shear stresses exceeding the shear strength of the geomaterial (Wilun & Starzewski, 1972; Duncan et al., 2014). This fundamental mechanism is frequently exacerbated by dynamic environmental triggers, most notably seismic loading, which induces transient pore water pressure fluctuations and inertial forces within the slope mass (Terzaghi, 1950; Abramson et al., 2002).

The specific failure mechanism and kinematics of a slope are governed by a multifaceted interplay of intrinsic and extrinsic factors. These chiefly include the lithological and structural geology, groundwater hydrology, topographic geometry, and localized climatic regimes, alongside the progressive mechanical weathering of the constituent rock or soil matrix (Skempton, 1964; Wilun & Starzewski, 1972; Varnes, 1978). Consequently, these compounding variables give rise to a diverse taxonomy of mass movements, ranging from superficial raveling and translational slides to deep-seated rotational failures (Bishop, 1955; Hutchinson, 1988).

I.1.1. Types of slips

Falls are usually associated with short-term failure of steep slopes (in artificial excavations or river banks) usually in rock with vertical joints (Wilun & Starzewski, 1972).

Rotational slides: occur characteristically in slopes of fairly uniform clay (Wilun & Starzewski, 1972).

Translational slides: generally, result from the presence of a heterogeneity, in the form of a weak soil layer, located at shallow depth beneath the slope (Wilun & Starzewski, 1972).

Flows: are mass movements in which there are no well-defined failure planes; Skempton and Hutchinson (1969) differentiate between earth flows and mud flows; the latter are glacier-like in form whereas the former are considered to be transitional in character between the slides and mud flows.

For a more comprehensive and systematic classification of landslide types, the scheme proposed by Varnes (1978) and later modified by Cruden & Varnes (1996) is widely adopted in geotechnical databases. Table 1 below presents the landslide types used in such databases, including codes for selection.

Table 1 : Landslide Types Used in the Database (Classification after Varnes, 1978; Cruden & Varnes, 1996).

Landslide Type	Landslide Code	Description
Soil slip	S	Soil slip, soil slide
Shallow landslide	Sh	Shallow landslide (most commonly a soil slip or soil slide)
Debris flow	D	Debris flow
Lahar	L	Lahar, a debris flow in volcanic materials
Slide, Earth flow	Sl, E	Slide (rotational or translational), Earth flow
Shallow landslides and debris flows	Sh, D	A combination of shallow landslides and debris flows
Soil slip, debris flow, mud flow	S, D, M	A combination of soil slip, debris flow and mud flow
All	A	All types of landslides

NOTE: that some types in this classification (e.g., lahar) are specific to volcanic environments and may not be directly relevant to conventional slope stability analysis in non-volcanic soils. The present text focuses on falls, rotational and translational slides, and flows as described above.

From the slope stability point of view the most dangerous conditions are encountered in areas where in the past the soils have been deformed and folded by tectonic activities or ice advances or where previous mass movements have taken place.

In these circumstances pre-existing slip planes (slickensides) are usually present along which the shear strength is only a fraction of the strength of the intact soil (Wilun & Starzewski, 1972).

The typical features of landslide areas are: depressions and bulges on natural slopes; deformed trees; springs on slopes; slickensides and deformed layers of clays (Wilun & Starzewski, 1972).

I.1.2. Procedure for estimating stability

There are 3 steps in process of estimating stability (Taylor, 1948):

1. Estimate disturbing forces.
2. Determine shearing resistance of soil including a factor of safety, (FS).
3. Select appropriate analysis.

Taylor (1948) describes in detail how different (F) values for (F_c) and (F_ϕ) may be combined. The limit state equilibrium approach determines the overall stability of the sliding mass by analysing various potential failure surfaces to determine which has the lowest (F); a circular arc is chosen because it is the simplest to analyse and is usually sufficiently accurate (Taylor, 1948).

I.1.3. Fundamentals of Limit Equilibrium (LE) Methods

All limit equilibrium methods utilize the Mohr-Coulomb expression to determine the shear strength (τ_f) along the sliding surface. The shear stress at which a soil fails is defined as its shear strength.

According to Janbu (1973), a state of limit equilibrium exists when the mobilized shear stress (τ) is expressed as a fraction of the shear strength. As Nash (1987) states: “At the moment of failure, the shear strength is fully mobilized along the failure surface when the critical state conditions are reached.” The Factor of Safety (FS) is generally defined as the ratio of available shear strength to mobilized shear stress:

$$FS = \frac{\tau_f}{\tau} \quad \text{Eq.(1)}$$

The shear strength is usually expressed by the Mohr- Coulomb linear relationship, where the τ_f and τ are defined by:

Shear strength (available) :

$$\tau_f = c' + \sigma' \tan(\varphi') \text{ or } (a + \sigma') \tan(\varphi') \quad \text{Eq.(2)}$$

Shear stress (mobilized) :

$$\tau = \frac{\tau_f}{FS} = \frac{c' + \sigma' \tan(\varphi')}{FS} \quad \text{Eq.(3)}$$

However, Abramson et al. (2002) clarify that FOS can be defined in three distinct ways:



Figure 1 : Limit Equilibrium: Based on shear strength (Total or Effective stress approach).

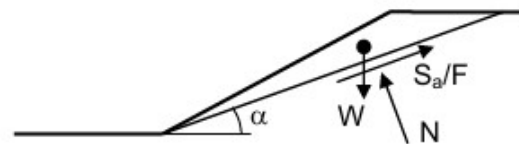


Figure 2 : Force Equilibrium: Ratio of resisting forces to driving forces.

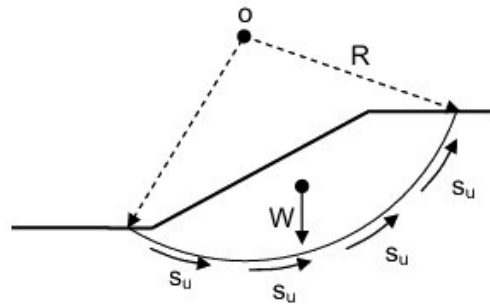


Figure 3 : Moment Equilibrium: Ratio of resisting moments to driving moments.

The choice between a total stress approach (s_u - analysis) or an effective stress approach ($c' - \phi'$ analysis) depends on the soil type, loading conditions, and the time elapsed after excavation (Janbu 1973; Grande 1977).

I.2. Total Stress Analysis (Stability of Cohesive Soils)

Undrained conditions apply (i.e., during or at end of construction).

Soil strength parameters are (c_u) and (ϕ_u) .

The excess pore water pressure due to loading has not had time to dissipate to any extent (Taylor, 1948).

I.2.1. Rotational analysis for undrained clay ($\phi_u = 0$)

For a circular slip surface, taking moments about the centre (O):

$$[FS = \frac{\text{Restraining moment}}{\text{Disturbing moment}} = \frac{c_u \times R^2 \times \theta_{\text{rad}}}{W \times d}] \quad \text{Eq.(4)}$$

where:

- $(R) = \text{radius}$
- $(\theta_{\text{rad}}) = \text{angle of arc in radians}$
- $(W) = \text{weight of sliding mass}$
- $(d) = \text{horizontal lever arm}$

I.2.2. Effect of Tension Crack

A tension crack always develops behind the crest of a slope as failure starts to occur in cohesive soil (Taylor, 1948). Cohesive soil can support a vertical face to a maximum height:

$$z_c = \frac{2 \times c_u}{\gamma} \quad \text{Eq.(5)}$$

If the tension crack fills with water, an extra thrust adds to the disturbing moment. The factor of safety becomes:

$$F = \frac{c_u \times R^2 \times \theta_c \left(\frac{\pi}{180} \right)}{W_t \times d_t + \frac{1}{2} \gamma_w \times z_c^2 \times y_c} \quad \text{Eq.(6)}$$

I.2.3. Partly Submerged Slopes

To allow for external water level, use submerged unit weight:

$$\gamma_{\text{sub}} = \gamma_{\text{sat}} - \gamma_w \quad \text{Eq.(7)}$$

The factor of safety is:

$$F = \frac{c_u R^2 \theta \times \left(\frac{\pi}{180} \right)}{(w \times d) + (w' \times d')} \quad \text{Eq.(8)}$$

I.2.4. Rapid Drawdown

If the retained water is quickly lowered, there is no water pressure on the face of the slope, and water levels in the slope immediately after drawdown remain constant. Thus (γ_{sat}) is used for area (A) moment, and the (F) value will be lowered (Taylor, 1948):

$$F = \frac{c_u \times R^2 \times \theta \times \left(\frac{\pi}{180}\right)}{(w \times d) + (w_{sat} \times d')} \quad \text{Eq.(9)}$$

I.2.5. Rapid Estimation of Stability (Taylor, 1948)

I.2.5.1. For Undrained Cohesive Soil ($\phi_u = 0^\circ$)

Taylor (1948) defined the **Stability Number**:

$$N_s = \frac{c_u}{\gamma \times H} \quad \text{Eq.(10)}$$

for ($F=1$).

For a safe design including a factor of safety:

$$N_s = \frac{c_u}{F \times \gamma \times H} \quad \text{Eq.(11)}$$

Taylor devised a chart that relates (N_s) to angle of slope, (β), for ($\phi_u = 0^\circ$) soils and ($\phi_u > 0^\circ$) soils (Taylor, 1948). Taylor extended his analysis to enable the influence of a hard stratum at depth (DH) to be found more accurately (Taylor, 1948).

Taylor's method is an approximate analysis intended for feasibility studies on temporary works; it does not provide information on the location of centre (O) or radius (R) (Taylor, 1948).

I.2.5.2. Frictional Resistance in Undrained Soil ($\phi_u > 0^\circ$)

Frictional resistance must be taken into account, but its value is not constant along the slip surface. A factor of safety must now apply to the frictional component also (Taylor, 1948):

$$\tau = \frac{c_u}{F_c} + \frac{\sigma_n \times \tan(\phi_u)}{F_\phi} \quad \text{Eq.(12)}$$

I.2.5.3. Layered Soil

For layered soil, use weighted average unit weight

$$N_s = \frac{c_u}{F \times \gamma_{av} \times H} \quad \text{Eq.(13)}$$

(Taylor, 1948).

I.2.5.4. Location of the Critical Circle

Taylor's curves give no indication of where the centre of rotation is located, although toe failures are likely in a slope where ($\beta > 54^\circ$) and ($\phi_u = 0^\circ$) (Taylor, 1948). (from Whitlow, 2001, p. 371) provides an indication of the most critical circle as a starting point for detailed analysis.

I.3. Effective Stress Analysis

I.3.1. Soil Strength Considerations

Any change in total stress ($\Delta\sigma$) in a saturated cohesive soil immediately alters pore water pressure by an amount (du) (excess pore water pressure). The (du) that is developed due to a change in total stress is given by Skempton's (1954) equation:

$$[du = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]] \quad \text{Eq.(14)}$$

where (A) and (B) are pore pressure parameters (Skempton, 1954).

For an excavated slope there are three stability conditions to evaluate (Skempton & Hutchinson, 1969):

- Short term (total stress analysis)
- Long term (effective stress analysis)
- Long term for heavily over-consolidated soils where weathering causes strength deterioration over 20–100 years

The concept of residual strength is discussed in Whitlow (2001, pp. 254–256).

I.3.2. Effective Strength Parameters

(c') and (ϕ') are obtained from either fully drained triaxial tests or consolidated undrained tests with pore water pressure measurement (Whitlow, 2001).

I.3.3. Total Stress Conditions (Short Term)

Applies during or immediately after construction. Conditions are similar to the undrained triaxial test (Whitlow, 2001).

If a clay soil is partially saturated, e.g., compacted clay, then ($\phi_u > 0^\circ$) and stability may be analysed to include a friction component (Whitlow, 2001).

I.4. Detailed Limit Equilibrium Methods (Slice Methods)

Several limit equilibrium (LE) methods have been developed for slope stability analyses.

All LE methods are based on certain assumptions for the interslice normal (E) and shear (T) forces, and the basic difference among the methods is how these forces are determined or assumed.

In addition to this, the shape of the assumed slip surface and the equilibrium conditions for calculation of the FOS are among the others.

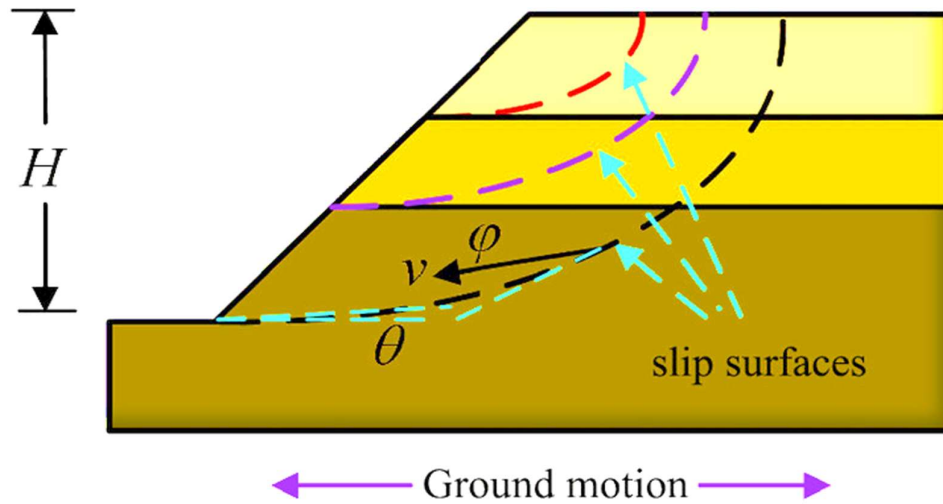


Figure 4 : Multiple Slip Surfaces Based.

I.4.1. Summary of LE Methods (after Abramson et al. 2002; Nash 1987)

Table 2 : Summary of LE Methods (after Abramson et al. 2002; Nash 1987).

Methods	Circular	Non-circular	$\sum_{=0}^M$	$\sum_{=0}^F$	Assumptions for T and E
Ordinary	√	–	√	–	Neglects both E and T
Bishop simplified	√	(*)	√	(**)	Considers E, but neglects T
Janbu simplified	(*)	√	–	√	Considers E, but neglects T
Janbu GPS	√	√	(***)	√	Considers both E and T, act at LoT
Lowe-Karafiath	–	√	–	√	Resultant inclines at, $\theta = \frac{1}{2}(\alpha + \beta)$
Corps of Engrs.	–	√	–	√	Resultant inclines at, $\theta = \frac{1}{2}(\alpha_1 + \alpha_2)$
Sarma	√	√	√	√	Interslice shear, $T = ch + E \times \tan(\varphi)$
Spencer	√	(*)	√	√	Constant inclination, $T = \tan(\theta) \cdot E$
Morgenst.-Price	√	√	√	√	Defined by $f(x)$, $T = f(x) \cdot \lambda \cdot E$

- (*): Can be used for both circular and non-circular failure surfaces.
- (**): satisfies vertical force equilibrium for base normal force.
- (***): satisfies moment equilibrium for intermediate thin slices (Janbu 1957, Grande 1997).

Free-body diagrams of slice methods:

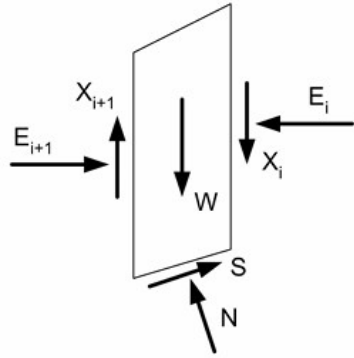


Figure 5 : free body diagram of slice method Lowe-Karafiath.

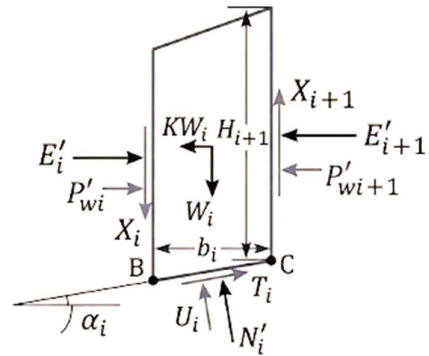


Figure 6 : free body diagram of slice method Sarma.

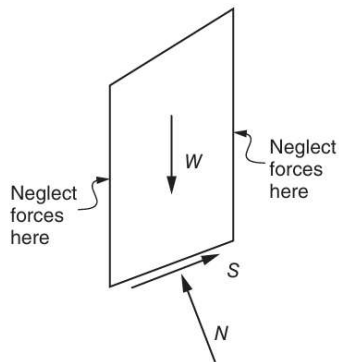


Figure 7 : free body diagram of slice method Ordinary.

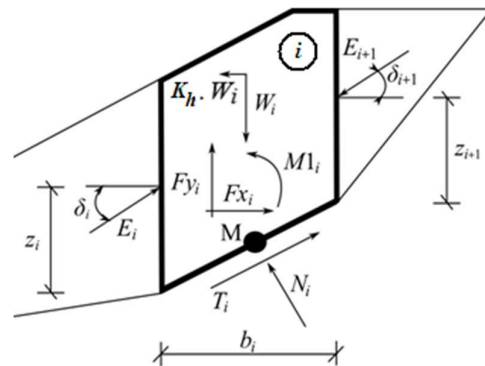


Figure 8 : free body diagram of slice method Morgenstern-Price.

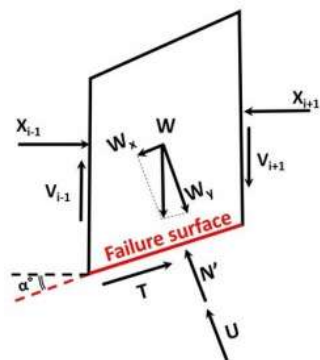


Figure 9 : free body diagram of slice method Bishop simplified.

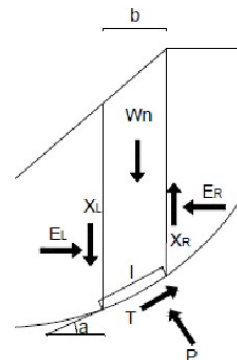


Figure 10 : free body diagram of slice method JANBU SIMPLIFIED.

I.4.2. Calculation of Factor of Safety for Main Methods

Table 3 : Limit Equilibrium Methods and Equations for Factor of Safety Calculation.

Methods	Calculation of the safety coefficient	
Ordinary	$FS = \frac{\sum [c' \times \Delta l + [W \times \cos(\alpha) - u \times \Delta l \times \cos(\alpha)^2] \times \tan(\phi')]}{\sum [W \times \sin(\alpha)]}$	Eq.(15)
Bishop simplified	$FS = \frac{\frac{1}{m_{\alpha_i}} \times \sum [(W_i - u_i b_i) \times \tan(\phi'_i) + c'_i b_i]}{\sum [W_i \times \sin(\alpha_i)]}$	Eq.(16)
	$F_j = \frac{\sum [\cos(\alpha_j) \cdot (c' \cdot l_j + (P_j - U_j) \cdot \tan(\phi'))]}{\sum [P_j \cdot \sin(\alpha_j)]}$	Eq.(17)
Janbu simplified	To account for the neglected interslice shear forces, Janbu introduced a correction factor (f_o), leading to the final Factor of Safety (F_f) (Janbu et al., 1956): $FS_f = f_o \cdot F_j$ The calculation of F requires an iterative approach since F appears on both sides of the equations (Duncan & Wright, 2005): 1. Assume an initial F (e.g., $F = 1.0$) to calculate P_j. 2. Use the calculated P_j to find a new F from the force equilibrium equation. 3. Repeat the process until the value of F converges ($F_k = F_{k-1}$). The final FS is reached when the boundary conditions are met ($\sum E_i = 0$). The rigorous Factor of Safety is often related to the force-equilibrium F_S via a correction factor (f_0) (Janbu, 1973): $FS_{rigorous} = f_0 \times FS_{force_only}$ The correction factor (f_0) is calculated based on the curvature of the slip surface using empirical data (Janbu, 1954): $f_0 = 1 + K \times \left[\frac{d}{L} - 1.4 \times \left(\frac{d}{L} \right)^2 \right]$ (Where K is a constant depending on soil type, d is the maximum depth, and L is the length of the slip surface).	
Janbu GPS		
Lowe-Karafiath	1. Assume a trial Factor of Safety (FS). 2. Calculate the interslice force (Z_{i+1}) for each slice, starting from the first slice where $Z_1 = 0$. 3. Continue to the last slice at the toe. If the final interslice force is not zero, adjust the FS and repeat the process. $Z_{i+1} = \frac{Z_i \cdot \cos(\alpha_i - \theta) + [W_i + (U_i - U_{i+1}) \cdot \sin(\theta)] \cdot \sin(\alpha_i) - \frac{1}{FS} [c'_i \cdot \Delta l_i + (N_i - u_i \cdot \Delta l_i) \cdot \tan(\phi'_i)]}{\cos(\alpha_i - \theta)}$	
Corps of Engrs.	• An initial value for FS is assumed. • Interslice forces (Z) are calculated slice-by-slice starting from the crest ($Z_1 = 0$). • The boundary condition must be satisfied at the toe ($Z_{last} = 0$). • The value of FS is adjusted until convergence is achieved. As a force equilibrium method, the lateral force at the downslope side of a slice (Z_{i+1}) is calculated as follows (Lowe & Karafiath, 1960; USACE, 2003): $Z_{i+1} = Z_i + \frac{C_1 + C_2 + C_3 - C_4}{n}$ $\theta_i = \frac{\phi_{surface} + \alpha_i}{2}$ • Weight Influence: $C_1 = W \times \left[\frac{\tan(\phi')}{FS \times \cos(\alpha)} - \sin(\alpha) \right]$ • Water Pressure Influence: $C_2 = (U_i - U_{i+1}) \times \left[\cos(\alpha) + \frac{\tan(\phi')}{FS} \times \sin(\alpha) \right]$ • External Loads: $C_3 = P \times \left[\sin(\alpha - \beta) - \frac{\tan(\phi')}{FS} \times \cos(\alpha - \beta) \right]$	

- **Cohesion and Pore Pressure:** $C_4 = \frac{c' \times \Delta A}{FS} + \frac{u \times \Delta A \times \tan(\phi')}{FS}$
- **Denominator (n):** $n = \cos(\alpha - \theta) + \frac{\tan(\phi')}{FS} \times \sin(\alpha - \theta)$

The Sarma method is a generalized limit equilibrium method that assumes the slip surface can be of any shape (circular or non-circular).

It also assumes that interslice forces satisfy a strength criterion similar to that of the base of slices, with a uniform factor of safety FS .

The method can additionally compute the critical horizontal seismic acceleration coefficient k_c (Sarma, 1973).

Basic Equations:

For each slice i , the equilibrium and failure conditions are:

1. Horizontal equilibrium:

$$\sum(E_i \times \cos(\alpha_i) - X_i \times \sin(\alpha_i)) = 0 \quad \text{Eq.(24)}$$

2. Vertical equilibrium:

$$\sum(E_i \times \sin(\alpha_i) + X_i \times \cos(\alpha_i)) = \sum W_i \quad \text{Eq.(25)}$$

3. Shear strength mobilization at the base of slice:

$$T_i = \frac{c'_i \times b_i \times \sec(\alpha_i) + (N_i - u_i \times b_i \times \sec(\alpha_i)) \times \tan(\phi'_i)}{FS} \quad \text{Eq.(26)}$$

4. Shear strength mobilization at the interslice surface:

$$X_i = \frac{c_{si} \times d_i + (E_i - P_{wi}) \times \tan(\phi_{si})}{FS} \quad \text{Eq.(27)}$$

$$FS_L = \frac{(E_i - P_{wi}) \times \tan(\phi'_i) + (c'_i \times H_i)}{X_i} \quad \text{Eq.(28)}$$

For a solution to be valid, it must satisfy:

- **No Tension Condition:** The thrust line must remain within the soil mass:
 $0 \leq z_i/H_i \leq 1$.
- **Internal Stability:** $FS_L \geq 1.0$.

The Spencer method assumes that all interslice forces (normal and shear) have a constant inclination angle (θ) throughout the sliding mass.

Force ratio assumption:

$$\frac{X_i}{E_i} = \tan(\theta) = \text{constant} \quad \text{Eq.(29)}$$

Factor of Safety from Moment Equilibrium (Fm):

$$Fm = \frac{\left[\sum \left(c'_i \times b_i \times \sec(\alpha_i) + (W_i \times \cos(\alpha_i) - u_i \times b_i \times \sec(\alpha_i)) \times \tan(\phi'_i) \right) \times \left(\frac{1}{m_{\alpha_i}} \right) \right]}{\sum (W_i \times \sin(\alpha_i))} \quad \text{Eq.(30)}$$

$$m_{\alpha_i} = \cos(\alpha_i) + \frac{\sin(\alpha_i) \times \tan(\phi'_i)}{Fm} \quad \text{Eq.(31)}$$

Factor of Safety from Force Equilibrium (Ff):

$$Ff = \frac{\left\{ \sum [(W_i \times \cos(\alpha_i) - u_i \times b_i \times \sec(\alpha_i)) \times \tan(\phi'_i) + c'_i \times b_i \times \sec(\alpha_i)] \times \frac{\cos(\alpha_i - \theta)}{\cos(\theta)} \right\}}{\left\{ \sum (W_i \times \sin(\alpha_i - \theta)) + \sum [(W_i \times \cos(\alpha_i) - u_i \times b_i \times \sec(\alpha_i)) \times \tan(\phi'_i) + c'_i \times b_i \times \sec(\alpha_i)] \times \frac{\sin(\alpha_i - \theta)}{\cos(\theta)} \right\}} \quad \text{Eq.(32)}$$

Solution Steps:

- Assume an initial value of θ .
- Compute Fm iteratively because it appears in m_{α_i} .
- Compute Ff using the assumed θ .
- Adjust θ until $Fm = Ff$.
- The final converged value is the factor of safety FS .

Note:

The simplified expressions in older references are local approximations and are insufficient for full limit equilibrium analysis in the Spencer method.

Morgenstern-Price The Morgenstern-Price method (Morgenstern & Price, 1965) assumes that the ratio of shear to normal interslice forces follows a prescribed function $f(x)$ scaled by a constant λ :

$$\frac{X}{E} = \lambda \times f(x) \quad \text{Eq.(33)}$$

The function $f(x)$ can be chosen arbitrarily (e.g., constant, half-sine, clipped sine) (Morgenstern & Price, 1965).

Normal force at the base of slice (N_i):

From Morgenstern and Price (1965), the normal force is derived as:

$$N_i = \frac{W_i - (X_{i+1} - X_i) + \frac{c'_i \times b_i \times \sec(\alpha_i) \times \sin(\alpha_i) - u_i \times b_i \times \sec(\alpha_i) \times \tan(\phi'_i) \times \sin(\alpha_i)}{F}}{\cos(\alpha_i) + \frac{\sin(\alpha_i) \times \tan(\phi'_i)}{F}} \quad \text{Eq.(34)}$$

Overall horizontal force equilibrium:

$$\sum \left[N_i \sin \alpha_i - \frac{c'_i \times b_i \times \sec(\alpha_i) \times \cos(\alpha_i) + (N_i - u_i \times b_i \times \sec(\alpha_i)) \times \tan(\phi'_i) \times \cos(\alpha_i)}{F} \right] = 0 \quad \text{Eq.(35)}$$

Overall moment equilibrium:

$$\sum \left[N_i \times \cos(\alpha_i) - \frac{c'_i \times b_i \times \sec(\alpha_i) \times \sin(\alpha_i) + (N_i - u_i \times b_i \times \sec(\alpha_i)) \times \tan(\phi'_i) \times \sin(\alpha_i)}{F} \right] \cdot R_i = 0 \quad \text{Eq.(36)}$$

Double iteration solution:

1. Assume initial values of λ and F (Morgenstern & Price, 1965).
2. Compute N_i from the first slice to the last.
3. Check the two global equilibrium equations.
4. Adjust λ and F using a double iteration loop until both equations are satisfied.

Note: The original file contained only the differential form $dS = \frac{c' \times dx \times \sec(\alpha) + (dN') \times \tan(\phi')}{F}$, which describes the local failure condition but not the global solution (Morgenstern & Price, 1965; Duncan & Wright, 2005).

1.5. Choice of Factor of Safety

Factors to be considered include consequences of failure and confidence in available information.

Recommended (FS) values (BS 6031, 1981, as cited in Whitlow, 2001) :

Table 4 : Recommended Factors of Safety (FS) for Different Slope Stability Conditions.

Condition	Recommended (FS)
Long term steady state seepage	1.25 – 1.5
After sudden drawdown	1.2
Slides with pre-existing surfaces	1.2
Natural slopes	1.10 – 1.20
Spoil tip	1.5
Problems involving buildings	2

1.6. Cuttings in Overconsolidated Clays

The strength of freshly exposed over-consolidated clays gradually decreases with time due to weathering (Skempton & Hutchinson, 1969). Deeper major failures can occur after longer periods (50–100 years). Time-dependent softening is not measured in a standard test (Whitlow, 2001, p. 213) shows peak and residual strengths from a ring shear test.

Residual strength (for (c') and (ϕ')) are the only available parameters to use for long-term stability (Skempton & Hutchinson, 1969). —

Morgenstern (cited in Whitlow, 2001) studied natural slopes in London Clay, finding that the majority of slopes are below (10°) .

Knowing the age of cuttings of various slope angles which have failed has enabled Morgenstern to determine the rate of strength reduction for London Clay (Whitlow, 2001).

I.7. Finite Element Method (FEM) in Slope Stability

The finite element method (FEM) is a numerical technique for finding approximate solutions to differential equations (Li, Qiao, & Tang, 2018). The basic idea of the finite element method is to obtain the solution of a complicated problem by replacing it with a simpler one (Bielak, 2024). Since the actual problem is replaced by a simpler model, the resulting solution is generally approximate rather than exact (Rao, 1982). For many practical engineering problems, existing mathematical techniques are insufficient to obtain exact solutions and may even fail to provide convenient approximate solutions (Potts & Zdravkovic, 1999). Therefore, in the absence of other efficient methods, the finite element method is widely adopted for engineering analysis (Smith, 1986).

I.7.1. Basic Element Shapes

The discretization of a domain (or solution region) into smaller subregions is the first step in the Finite Element Method (FEM). The shape, size, number, and arrangement of elements must be selected carefully so that the original geometry is represented as accurately as possible while avoiding unnecessary computational effort (Naylor & Pande, 1981).

For any physical body, engineering judgment is required to choose the most appropriate elements for discretization. In most applications, the choice of element type depends on the geometry of the structure and on the number of independent spatial coordinates required to describe the problem (Zienkiewicz, Taylor, & Nithiarasu, 2025).

The most commonly used finite elements are classified according to their dimensionality:

- **One-dimensional (1D) elements:** used for structural members whose length is much greater than their cross-sectional dimensions, such as bars, trusses, and beams (Ochsner & Merkel, 2018).
- **Two-dimensional (2D) elements:** Applicable to planar structures, plates, shells, and axisymmetric configurations.
- **Three-dimensional (3D) elements** – used for solid bodies exhibiting complex geometries and stress distributions (Huang, Ingrassia, & Kulhawy, 1983).

The following Figure illustrates examples of commonly used one-dimensional, two-dimensional, and three-dimensional elements

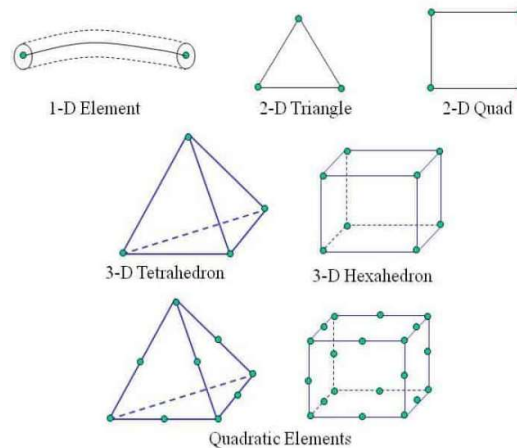


Figure 11: dimensional finite elements.

I.7.2. One-Dimensional Finite Element Method

The fundamental idea of the Finite Element Method (FEM) is to divide the problem domain (Ω) into a number of smaller subdomains called finite elements, where a typical element is denoted by (Ω^e). In a one-dimensional (1D) problem, these subdomains are simply intervals (Logg, Mardal, & Wells, 2012). The collection of all finite elements covering the domain is referred to as the finite element mesh.

The next step is to select a finite number of points within the mesh, preferably at the boundaries of the elements. These points are known as nodes or nodal points, and they are denoted by (x_j), where:

$$j = 1, 2, \dots, n$$

For each nodal point (x_j), a corresponding shape function ($\varphi_j(x)$) is defined over the domain:

$$\varphi_j : \Omega \rightarrow R$$

The shape functions must satisfy the following conditions (Zienkiewicz et al., 2025):

- **Interpolation Property:** $\varphi_j(x) = \begin{cases} 1, & i = j \\ 0, & i \neq j \end{cases}$

This condition ensures that each shape function takes the value 1 at its own node and 0 at all other nodes.

- **Prescribed Behavior within Each Element:** The shape functions must follow a specified interpolation order within each element, such as linear, quadratic, or higher-order polynomial interpolation, depending on the number of nodal points employed in the element (Bathe, 1996).
- **Continuity Condition:** The shape function must remain continuous over the entire domain: $\varphi_j(x)$

Using these shape functions, the approximate solution ($\bar{u}(x)$) can be constructed as:

$$\bar{u}(x) = \sum_{j=1}^{n+1} u_j \times \varphi_j(x)$$

Where :

- ($\bar{u}(x)$) is the finite element approximation of the exact solution ($u(x)$),
- (u_j) represents the value of the solution at node (x_j),
- ($\varphi_j(x)$) is the shape function associated with node (j).

Thus, the unknown continuous field variable is approximated by interpolating the nodal values using the selected shape functions (Zhao, 2009).

This approximation forms the basis of the finite element formulation and transforms the continuous problem into a solvable system of algebraic equations.

I.7.3. Shear Strength Reduction (SSR) Technique

According to Griffiths & Lane (1999), the Shear Strength Reduction (SSR) technique reduces the soil shear strength parameters — cohesion (c) and the tangent of the friction angle ($\tan(\varphi)$) — simultaneously by the same reduction factor (F_{trial}), as follows:

$$c_{reduced} = \frac{c}{F_{trial}} \quad \text{Eq.(37)}$$

$$\tan(\varphi_{reduced}) = \frac{\tan(\varphi)}{F_{trial}} \quad \text{Eq.(38)}$$

The reduction is applied incrementally until the slope fails. The Factor of Safety (FS) is then defined as the value of F_{trial} at which failure occurs (Griffiths & Lane, 1999).

I.7.3.1. Failure Criteria

Griffiths & Lane (1999) identified that, in finite element programs, failure is detected using one of the following criteria (commonly referred to as limit-state or failure indicators in numerical algorithms):

Non-convergence: The numerical solution fails to achieve equilibrium for the applied load after a certain reduction factor.

Plastic Zone Connection (Yield Zone Completion): Yield zones extend continuously from the toe to the crest of the slope, forming a continuous slip surface.

Displacement Jump (Displacement Run-off): A sharp and notable rise in nodal displacement occurs alongside only a trivial increase in F_{trial} , signaling a loss of resistance and impending failure.

I.8. Constitutive Models for Slope Stability Analysis

I.8.1. Basic Models

I.8.1.1. Linear Elastic (LE)

Based on Hooke's Law, it uses Young's modulus (E) and Poisson's ratio (ν). It is isotropic and assumes a linear stress-strain relationship. It is generally not suitable for soil, but effectively models stiff structural elements like concrete tunnels or massive, intact bedrock.

I.8.1.2. Mohr-Coulomb (MC)

A first-order approximation model. It assumes linear elasticity and perfectly plastic behavior. It uses five parameters: E , ν , cohesion (c), friction angle (ϕ), and dilatancy angle (ψ). It is the "industry standard" for initial scoping studies due to its computational efficiency.

I.8.2. Advanced Soil Models (Hardening Models)

I.8.2.1. Hardening Soil (HS)

Unlike the MC model, the HS model captures the stress-dependency of stiffness. It uses three different stiffness moduli: E_{50} (triaxial loading), E_{ur} (unloading/reloading), and E_{oed} (oedometer loading). It is highly recommended for most geotechnical projects involving excavation and foundation settlements.

I.8.2.2. HS with Small-Strain Stiffness (HSsmall)

A refinement of the HS model. Soils are significantly stiffer at very small strains (before reaching the elastic limit). This model captures this non-linear stiffness degradation using the small-strain shear modulus (G_0) and the reference strain ($\gamma_{0.7}$). It is essential for analyzing building settlements adjacent to deep excavations or dynamic loading.

I.8.3. Soft Soil & Creep Models

I.8.3.1. Soft Soil (SS) Model

Specifically designed for primary compression in normally consolidated clays. It is based on the Cam-Clay theory, which accounts for the logarithmic relationship between volumetric strain and mean effective stress.

I.8.3.2. Soft Soil Creep (SSC) Model

Soft soils (clays, peat, silts) exhibit viscous behavior. This model simulates secondary compression (creep) over time. It is vital for long-term settlement analysis of embankments and foundations on very soft layers.

I.8.4. Specialized Rock & Concrete Models

I.8.4.1. Hoek-Brown (HB)

Used for weathered rock mass. It defines a non-linear failure envelope that describes both shear and tensile failure as a function of the rock's Geological Strength Index (GSI) and disturbance factor (D).

I.8.4.2. Jointed Rock (JR)

An anisotropic model that simulates rock layers with clear stratification or fault planes. It allows for plasticity to occur along specific shear planes while the rock mass remains elastic.

I.8.4.3. Concrete Model

An advanced elasto-plastic model for shotcrete and structural concrete. It considers the time-dependent evolution of strength and stiffness, as well as hardening/softening behaviors in both compression and tension.

I.8.5. Advanced Dynamic & Cyclic Models

I.8.5.1. UBC3D-PLM

The primary model for liquefaction analysis. It uses two yield surfaces to simulate the densification and excess pore pressure generation in sands under seismic (cyclic) loading.

I.8.5.2. UDCAM-S

A simplified model for undrained cyclic loading in clays. It focuses on strength and stiffness degradation, making it ideal for offshore geotechnical engineering.

Table 5 : Summary Table of Model Selection.

Model	Best Application	Key Consideration
Linear Elastic (LE)	Rigid structural elements (concrete tunnels, massive bedrock)	Linear elastic behavior (Not suitable for soil)
Mohr-Coulomb (MC)	Preliminary analysis / Scoping studies	Simple, efficient, "industry standard"
Hardening Soil (HS)	General excavations and foundation settlements	Stress-dependent stiffness
HSsmall	Deep excavations, urban projects, dynamic loading	Small-strain stiffness degradation/recovery
Soft Soil (SS)	Primary consolidation in soft soils	Designed for primary settlement
Soft Soil Creep (SSC)	Soft soils with time-dependent settlements	Accounts for creep (viscosity)
UBC3D-PLM	Seismic analysis and liquefaction	Pore-pressure generation in saturated sands
UDCAM-S	Offshore geotechnical engineering	Strength and stiffness degradation under cyclic loading

I.8.6. Calibration: The Role of the SoilTest Facility

The SoilTest feature in PLAXIS is essential for model calibration. It allows you to simulate lab tests (Triaxial, Oedometer, Direct Simple Shear) numerically. By comparing the software's output to your actual laboratory results, you ensure that the chosen constitutive model parameters (like E_{50} or c) accurately reflect your site's physical soil behavior.

I.9. Applicability, Limitations, and Comparison

Table 6 : Comparison between Limit Equilibrium (LE) and Finite Element (FEM) Analysis Methods.

	Limit Equilibrium (LE)	Finite Element (FEM)
Advantages	Characterized by simplicity and ease of understanding for practitioners.	Robust and accurate method requiring fewer assumptions regarding failure mechanisms.
	Facilitates rapid sensitivity analysis of various input parameters.	Slip surfaces evolve naturally through zones of insufficient shear strength.
	Leverages extensive engineering experience in calculating the Factor of Safety (FOS).	Enables the simultaneous evaluation of slope stability and deformation behavior.
Limitations	Lacks an integrated stress-strain relationship.	Demands significantly higher computational resources and complex modeling compared to traditional methods.
	Requires a priori assumptions regarding the shape and location of the potential slip surface.	
	Sensitive to assumptions concerning interslice side forces.	
	Lacks appropriate procedures for rapid loading conditions.	

I.10. Conclusion

The preceding synthesis of classical and contemporary slope stability mechanics reveals a fundamental duality in geotechnical practice: the enduring utility of limit equilibrium (LE) methods for routine analysis, alongside the growing necessity of numerical techniques such as the finite element method (FEM) for capturing complex failure mechanisms.

Limit equilibrium methods: rooted in the Mohr-Coulomb criterion, the factor of safety concept, and slice-based formulations—provide a robust, transparent framework for assessing both total stress (undrained) and effective stress (drained) conditions. They successfully address rapid drawdown, tension crack hydraulics, layered profiles, and Taylor’s stability number for rapid estimation. However, LE methods assume simultaneous strength mobilization along a predefined slip surface, cannot simulate progressive failure or strain-softening, and exhibit subjectivity in interslice force assumptions (e.g., Ordinary vs. Bishop vs. Janbu). These limitations are particularly critical in overconsolidated clays, where long-term weathering reduces strength to residual values over decades—a phenomenon that LE treats only empirically.

The finite element method: overcomes these shortcomings by discretizing the slope, assigning shape functions, and solving governing differential equations approximately but systematically. FEM captures: (i) strain localization and shear band propagation without presupposing slip surface geometry, (ii) coupled hydro-mechanical behavior (transient pore pressures, drawdown), (iii) strain-softening and post-peak degradation, and (iv) complex stratigraphic irregularities. Nevertheless, FEM demands high-quality constitutive parameters, careful mesh design, numerical convergence, and significant expertise—making it less accessible for rapid or low-consequence assessments. The approximation inherent in FEM (replacing continuous fields with nodal interpolations) introduces discretization errors requiring mesh refinement studies.

Practical recommendations: Employ LE methods (e.g., Bishop Simplified or Morgenstern-Price) for routine screening, parametric studies, and code-based design (BS 6031), selecting factors of safety based on consequence class and data confidence. Adopt FEM for high-consequence slopes, complex geological settings (relict slip surfaces, tectonic deformation, previous landslides), time-dependent softening problems (cuttings in overconsolidated clays over 50–100-year service lives), and seismic loading scenarios. Ultimately, the goal remains consistent: ensuring disturbing forces remain reliably below available shear resistance. Future work should focus on bridging empirical LE insights with advanced numerical models through probabilistic frameworks, field monitoring, and back-analysis of documented failures, advancing slope stability practice toward risk-informed, performance-based geotechnical engineering.

Chapter 02: Geological and geomorphological settings

II.1.Geographic Location and Administrative Context

The study area is located in the municipality of Bekkaria, in the eastern part of the wilaya of Tebessa (northeastern Algeria), approximately 20 km southeast of the wilaya capital and in close proximity to the Algerian-Tunisian border (ONS, 2020). The geographic coordinates of the area are 35°22'20"N and 8°14'32"E.

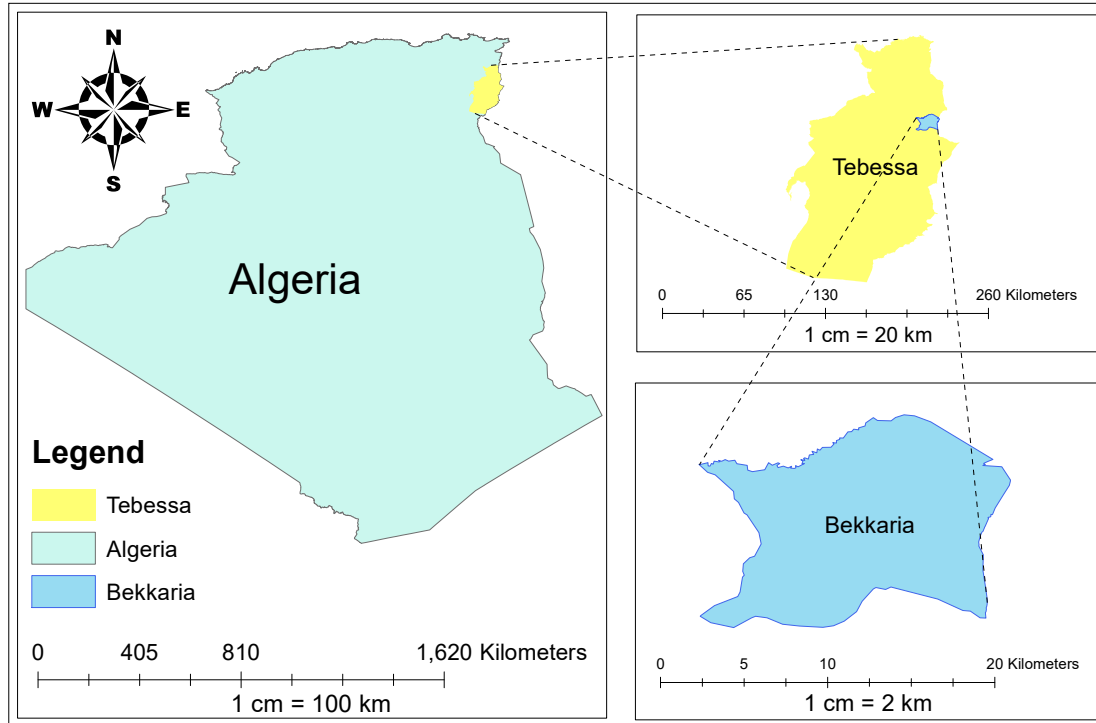


Figure 12 : Geographic Location of the Study Area in the Municipality of Bekkaria, Tebessa.

More specifically, the project concerns the section of the Tenoukla open-cut trench extending from KP 24+700 to KP 25+078 (Cosider, 2024).



Figure 13 : the railway embankment and the trench excavation site.

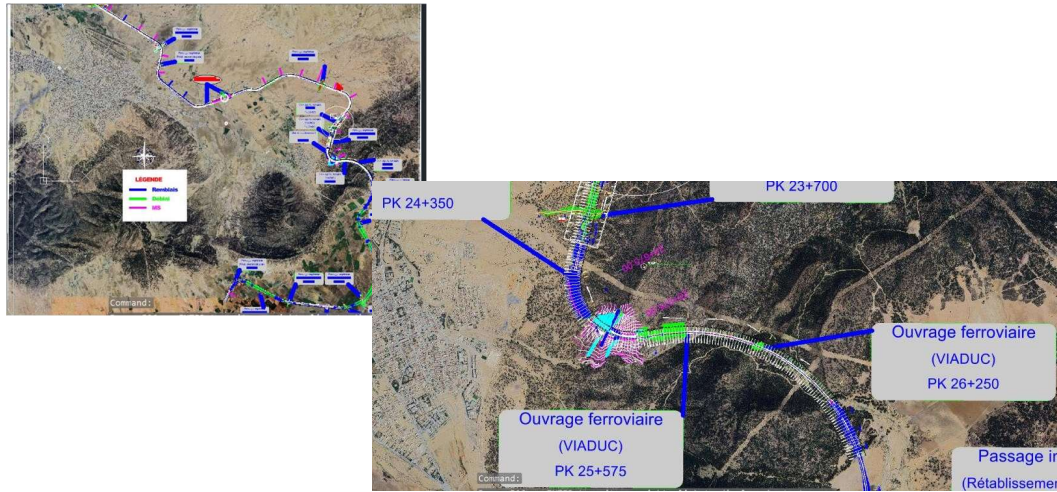


Figure 14 : Location of the railway crossing the study area and the project zone.

Geomorphologically, the site belongs to the Eastern High Plains unit, a transition zone between the Tellian Atlas and the Saharan Atlas (ANRH, 2018).

II.1.1. Topography

The regional topography is characterized by moderately rugged terrain, with elevations ranging from 800 m to 1,200 m (IGN, 2021). The railway route is situated midway up the slope on the right bank of Oued Djebissa, a major tributary of Oued Bekkaria (TE-ISL, 2024b).

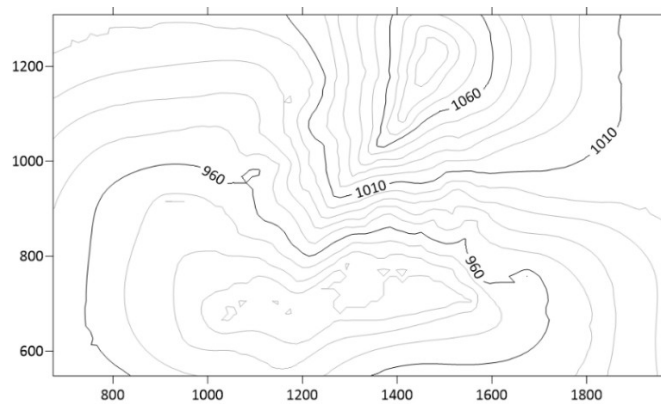


Figure 15 : Contour Map of Surface Topography and Elevation Gradients in Djebel Djebissa Region (35.380599°N, 8.264060°E) Generated via Golden Software Surfer.

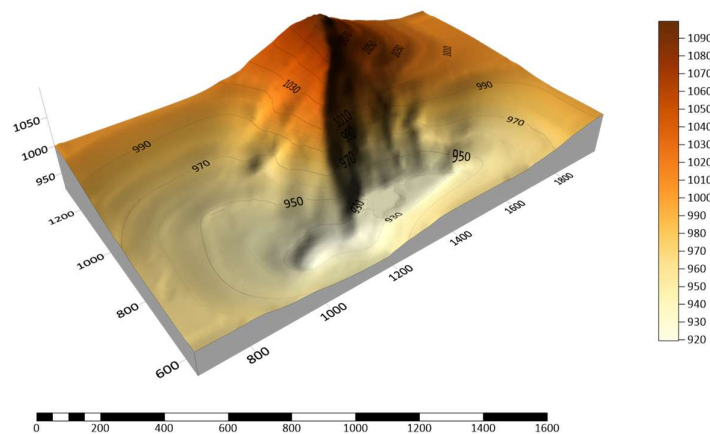


Figure 16 : 3D Topographic Surface Modeling of the Djebel Djebissa Region (35.380599°N, 8.264060°E) Generated via Golden Software Surfer.

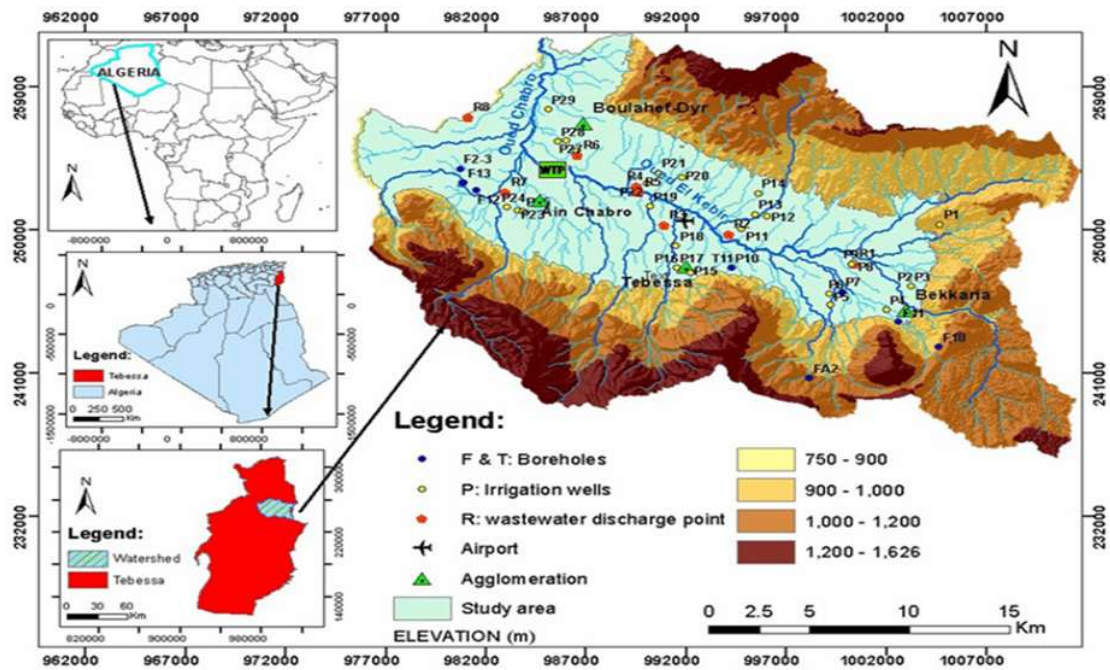


Figure 17 : Geographical location of the study area, localization of wastewater discharge points and water samples. by Houichi et al., 2024, *Revista Brasileira de Recursos Hídricos*, 29, e24.

II.1.2. Geological and Lithological Characteristics

The geomorphological setting is shaped by the interaction between tectonic activity and lithological variability. The site traverses three major geological formations (TE-ISL, 2024a; Hadji et al., 2017):

Miocene-Pliocene red clays: Predominant in the first section, exhibiting a stiff to very stiff consistency.



Figure 18 : Field View of the Miocene–Pliocene Red Clay Unit.

Compact Turonian limestones: Located in the center of the route, consisting of massive limestone banks with a 70° dip toward the southeast.



Figure 19 : Field View of the Compact Turonian limestones Unit.

Triassic formations: Located at the western end, composed of variegated gypsum-rich clays and marls.



Figure 20 : Field View of the Triassic formations Unit.

II.2. Structural Architecture and Gravimetric Analysis of the Tebessa Region (Algeria)

II.2.1. Regional Geological Context

Structural Architecture and Gravimetric Analysis of the Tebessa Region (Algeria): Insights from the Study of Defaflia et al. (2022)

The Tebessa region, situated in the northeastern sector of the Saharan Atlas, exhibits a complex geological and structural framework. According to Defaflia et al. (2022), the area is characterized by a succession of anticlines, synclines, diapiric structures, and grabens developed within a thick sedimentary sequence resting on a Paleozoic basement.

The development and present configuration of these large-scale geological structures are closely related to the tectonic evolution of the western Mediterranean domain. Tebessa forms part of the foreland graben system associated with the North African Alpine orogeny, reflecting the combined effects of compressional and extensional tectonic processes that have shaped the region throughout its geological history (Defaflia et al., 2022).

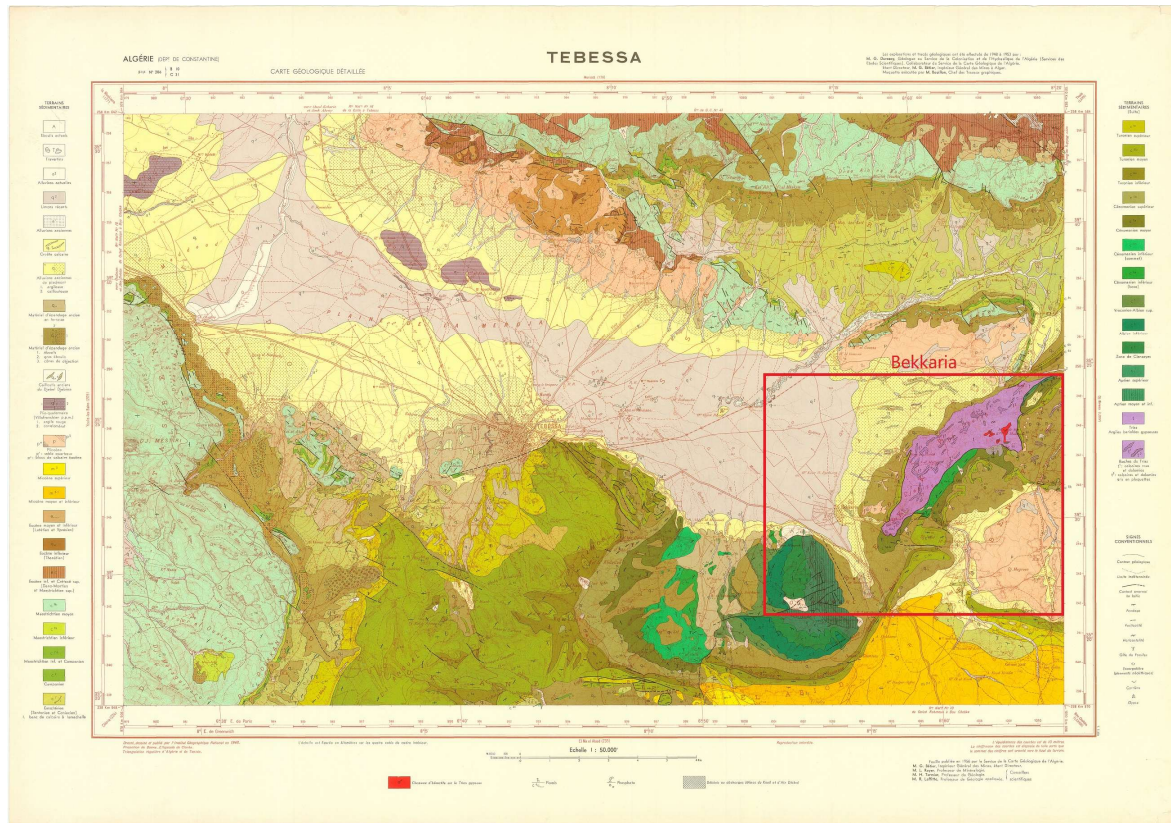


Figure 21 : Regional Geological Map of Tebessa (Service de la Carte Géologique de l'Algérie, 1956) alongside a detailed lithostratigraphic map of the Bekkaria study area.

II.2.2. Stratigraphic and Structural Summary of the Tebessa Region

The structural configuration of North Africa is a result of a succession of several extensional and contractional tectonic events occurring since the late Permian. These events resulted in the formation of three distinct structural domains: the Tell-Rif domain, the Atlas domain, and the Saharan Platform (Frizon de Lamotte et al., 2000, 2009). The origin of the tectonic activity responsible for the deformation observed in northern Africa varies significantly from one domain to another (Bracène & Frizon de Lamotte, 2002).

Located in the extreme northeastern part of the Saharan Atlas of Algeria, the Tebessa region is characterized by a complex structural architecture consisting of a succession of anticline and syncline structures, diapirs, and grabens developed within a thick sedimentary cover (Aboud et al., 2022). Geophysical and structural results indicate that the main tectonic trends within the study area follow NE-SW, E-W, ENE-WNW, and NW-SE directions. The stratigraphic column of the region (Fig. A) reveals a sedimentary cover thickness ranging from 2 to 8 km, underlain by the Paleozoic basement.

A key rheological feature of this sequence is the Triassic evaporite layer (approximately 700 m thick), which facilitated the opening of grabens and the upward migration of diapiric formations by providing a zone of mechanical weakness (Aboud et al., 2022).

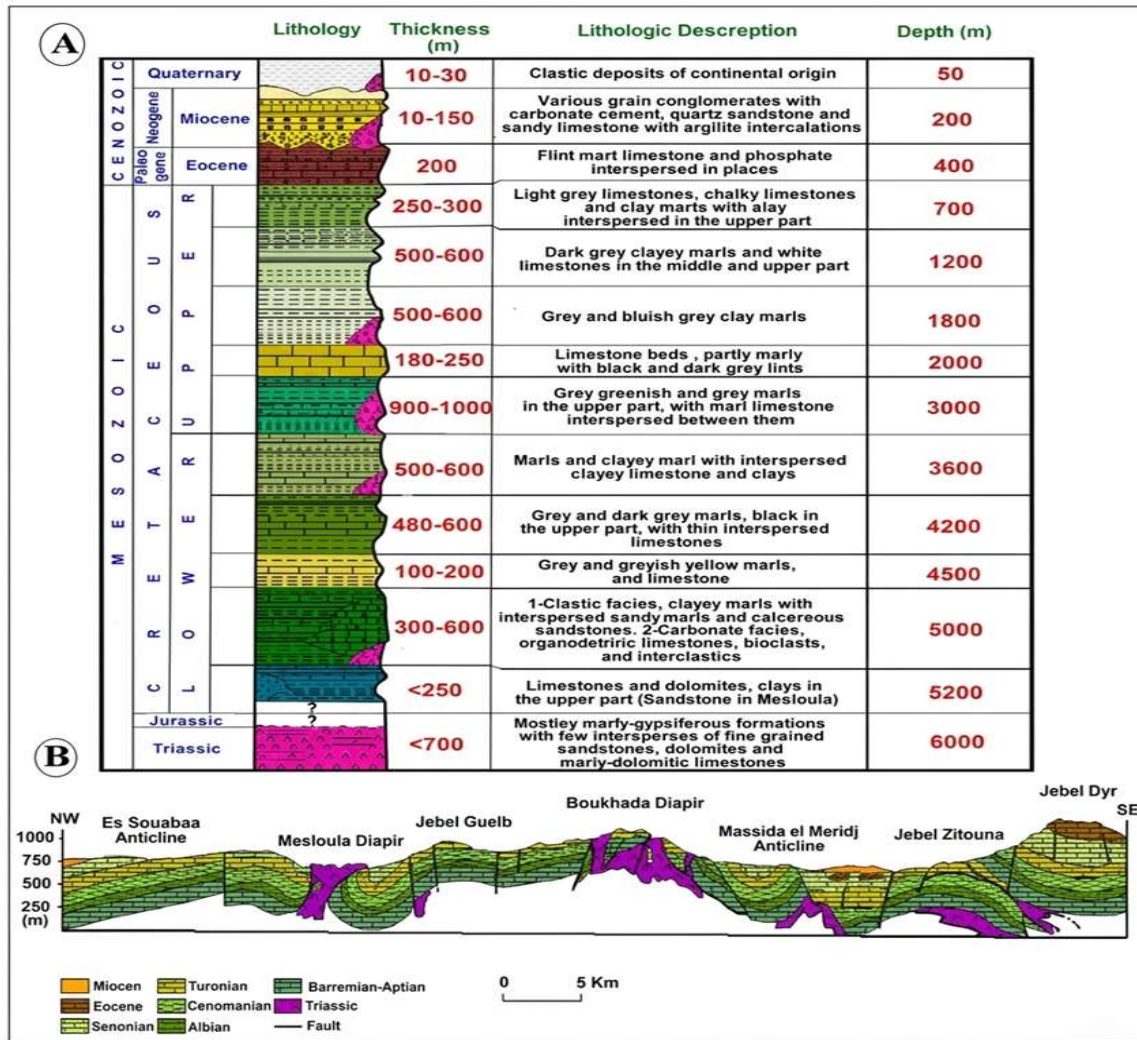


Figure 22 : Lithostratigraphic Column of the Tébessa Region (Triassic–Quaternary): Tectono-Sedimentary Evolution and Major Unconformities (after Durozoy, G., 1956).

The overlying Mesozoic sequence is dominated by thick marine deposits. As illustrated in the lithostratigraphic profile, the Lower Cretaceous is composed of marls, clayey limestones, and dolomites, while the Upper Cretaceous (Cenomanian to Senonian) consists of thick layers of bluish-grey clay marls and limestone beds. The Cenozoic era is represented by Eocene phosphate-rich flint limestones and Neogene conglomerates, topped by Quaternary clastic deposits of continental origin. Structurally, the eastern part of the Tébessa region is marked by a basement uplift, which contributes to the positive gravity anomalies associated with the diapiric intrusions, such as the Mesloulia and Boukhada diapirs (Aboud et al., 2022; Fig. B).

II.2.3. Climatological Setting: Classification, Thermal Regime, and Precipitation

The climate of the Tébessa region is a critical factor in slope stability analysis, as it governs the hydraulic boundary conditions and pore-water pressure fluctuations within the geological formations. According to the modified Köppen–Geiger classification system, the study area is primarily characterized as a BSk (Cold Semi-Arid) climate, transitioning in some sectors towards BSh (Hot Semi-Arid) during prolonged dry periods. This classification is defined by a potential evapotranspiration that exceeds annual precipitation and is accompanied by significant seasonal thermal variations.

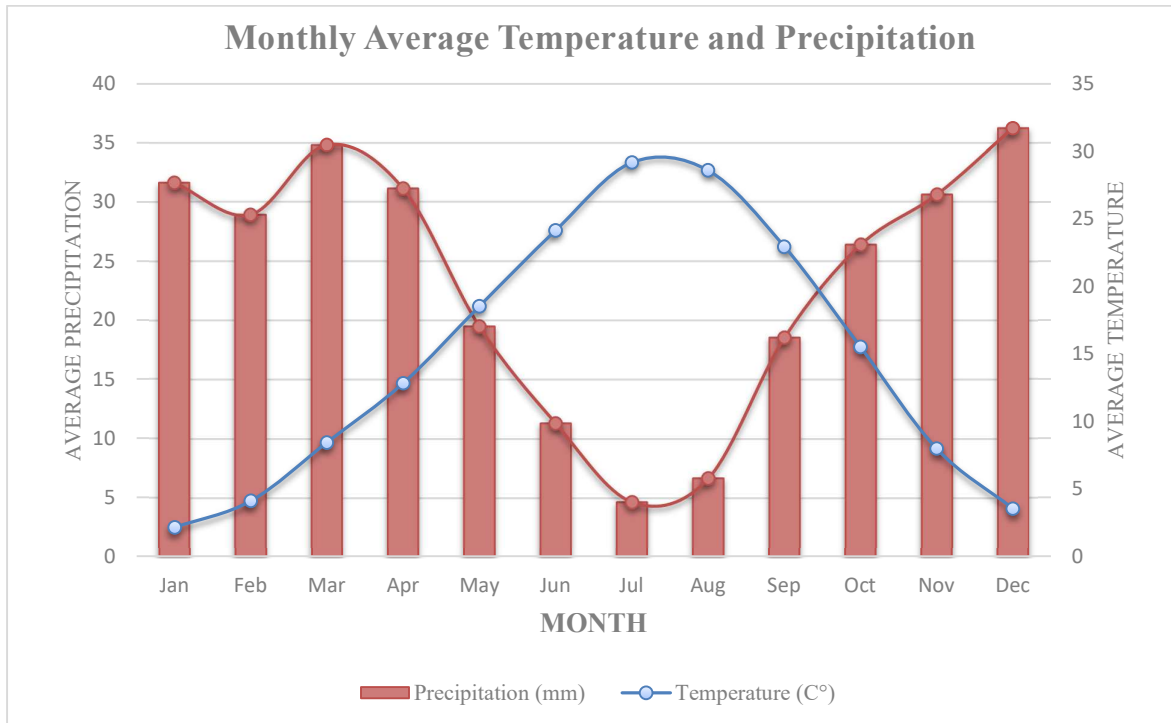


Figure 23 : Monthly Average Temperature and Precipitation (1991–2025): Tebessa (Climate Proxy for Bekkaria).

II.1.1.1. Thermal Regime

The region exhibits a wide thermal amplitude throughout the year. Mean monthly temperatures fluctuate from a minimum of approximately 5°C in January, where daily lows can approach 1°C, to a maximum of approximately 35°C in July (Aboud et al., 2022). During the peak summer months, extreme daily highs frequently exceed 33°C, leading to intense desiccation of the upper soil layers and the potential development of shrinkage cracks in clay-rich lithologies, such as the Cretaceous marls identified in the stratigraphic column.

II.1.1.2. Precipitation Patterns

The hydrological regime is characterized by an average annual rainfall ranging between 300 and 500 mm. The distribution is markedly seasonal, with approximately 70% of the total precipitation occurring between October and March. This period represents the most critical window for landslide triggering, as sustained rainfall increases the degree of saturation in the heterogeneous slope materials.

Furthermore, the region is subject to high interannual variability driven by orographic effects from the Atlas Mountains, which can result in localized high-intensity storm events that further compromise the factor of safety (F_s) of the regional slopes.

II.2.4. Piezometry and Groundwater Levels

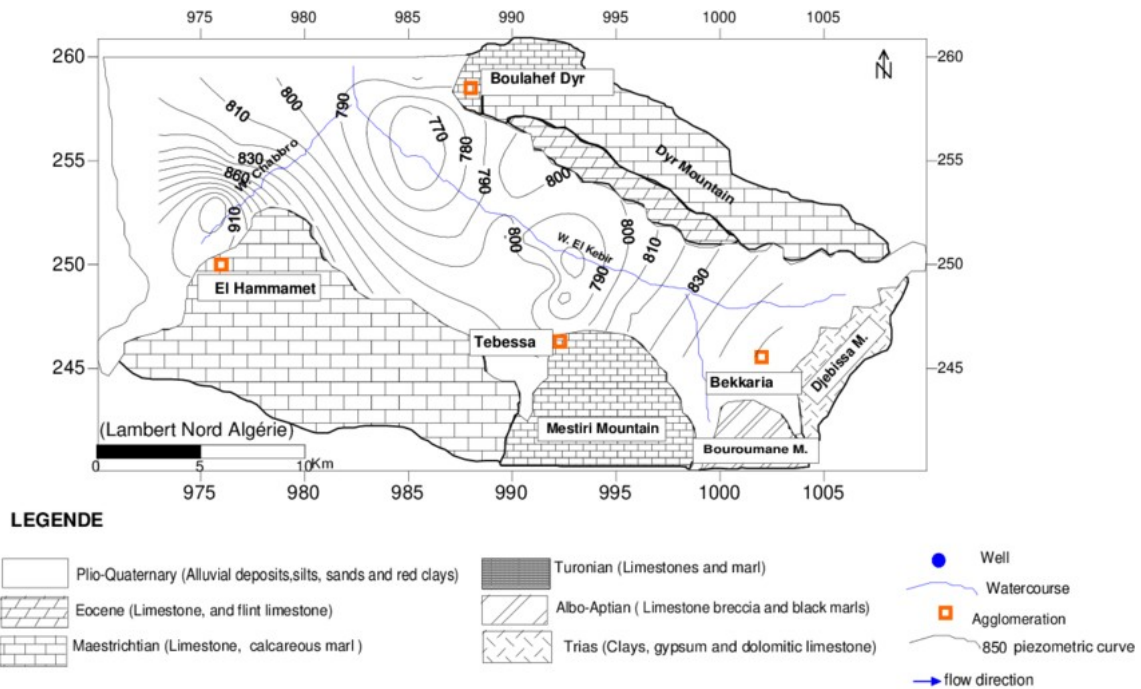


Figure 24 : Carte piézométrique de la nappe de la Merdja, juin 2015.

The P1 aquifer in the Bekkaria area is unconfined and closely follows seasonal precipitation, showing wet-season rises up to 4 m in piezometric head (Ghrieb, 2010).

Two deeper, confined horizons (P2 and P3) lie beneath impermeable marl and limestone; over the last decade their heads have declined by 1–2 m due to abstraction and reduced recharge (Drias et al., 2022). Transient perched water tables often form within clayey layers above P1 during heavy rains, elevating pore pressures and weakening trench sidewalls (Mrad et al., 2018).

II.2.5. Specific Geological Framework: Salt Tectonics and Flow Mechanisms

Beyond the surface topography, the subsurface stability of the Tebessa -Bekkaria region is significantly influenced by halokinetic processes. The presence of salt structures and 'Diapirs' has a profound impact on the mechanical behavior of the overburden and the regional stress field. Therefore, understanding the mechanisms of salt flow and the evolution of these diapirs is crucial for a reliable geotechnical interpretation. The following section details the fundamental principles of salt tectonics, flow mechanisms, and the various stages of diapirism that characterize the study area:

II.3. Salt Diapirs and Evaporite Tectonic Structures

II.3.1. Global Distribution and Salt Structures

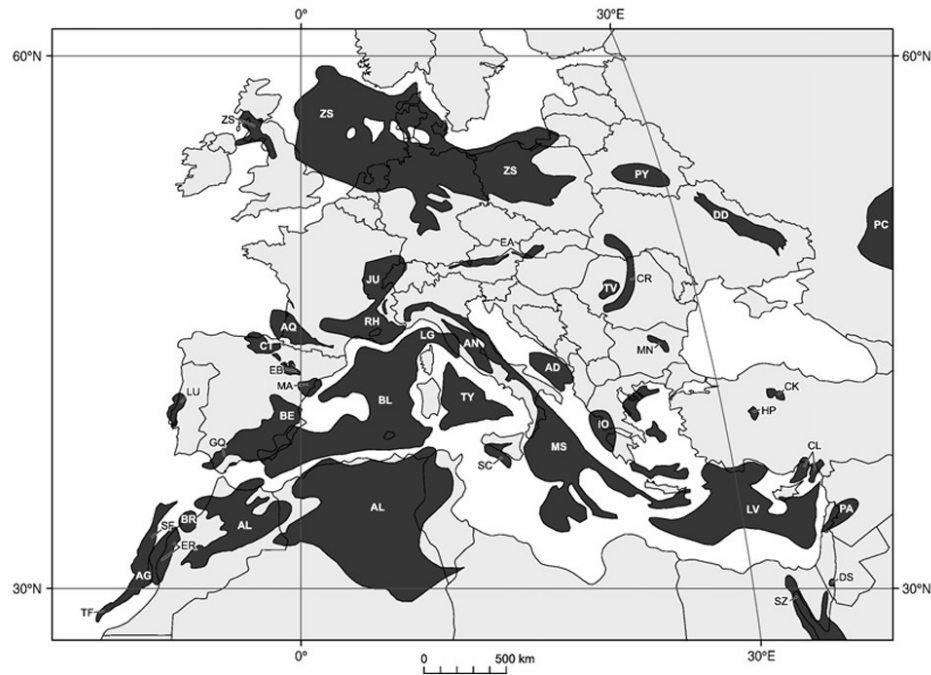


Figure 25: Global distribution map of major collision salt basins.

II.3.2. Schematic shapes of salt structures

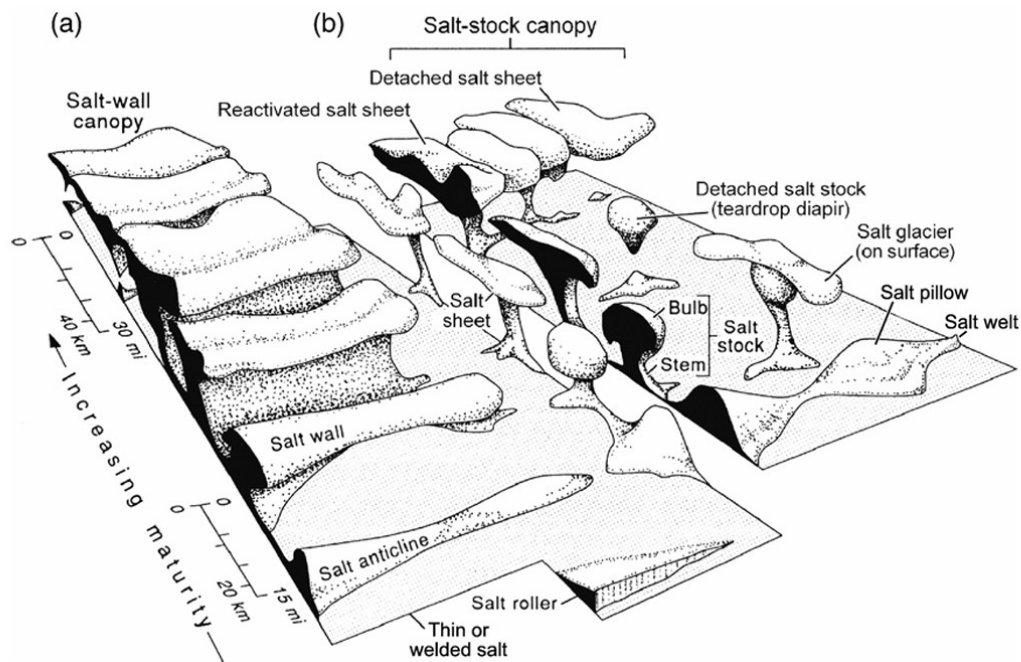


Figure 26 : Block diagram showing schematic shapes of salt structures Simplified from Jackson and Talbot (1991).

Structural maturity and size increase toward the composite, coalesced structures in the background:

Elongated structures rising from line sources.

Structures rising from point sources.

Schematic shapes of salt structures:

II.3.2.1. Mechanisms Driving Salt Flow

II.3.2.1.1. Total hydraulic head (head)

The movement of salt is not merely a matter of density; it is governed by the total hydraulic potential (head) at any given point:

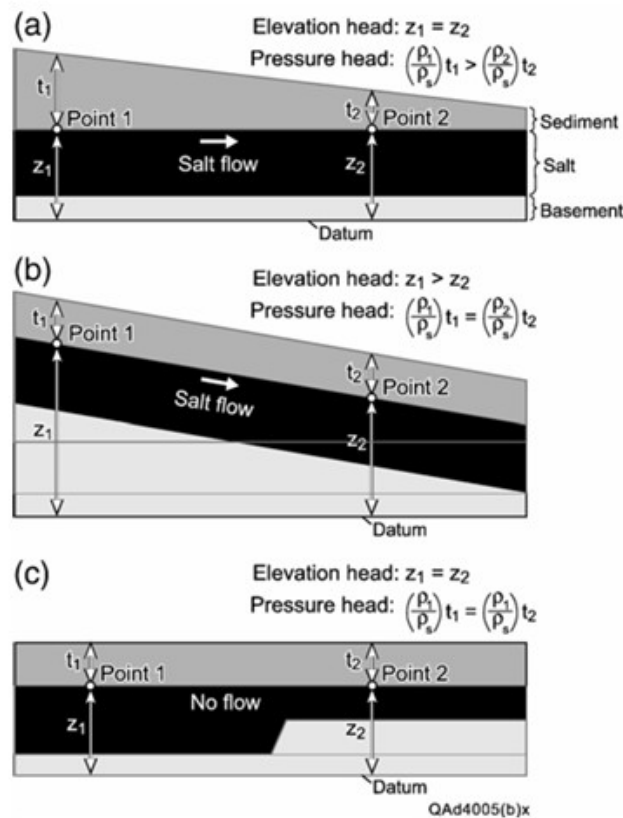


Figure 27 : Controls of Elevation and Pressure Heads on Salt Flow Dynamics.

II.3.2.1.2. Differential Pressure Head

Even if the salt layer is perfectly horizontal ($z_1 = z_2$), flow will occur if there is a difference in the thickness or density of the overburden ($t_1 > t_2$).

This creates a pressure gradient that drives salt toward areas of lower confinement.

II.3.2.1.3. Differential Elevation Head

Salt can also flow due to structural tilt ($z_1 > z_2$).

Even if the overburden pressure is equal, the gravitational potential energy at the higher elevation drives the salt "down-dip."

II.3.2.1.4. Static Equilibrium (No Flow)

When both the pressure head and elevation head are balanced, the salt remains stagnant, regardless of its inherent buoyancy.

Note: The Role of Buoyancy Today:

While buoyancy was once the "initiator," it is now understood that roof strength can keep salt trapped for millions of years.

Buoyancy is no longer seen as the primary factor in initiating diapirism; instead, it is a secondary force that becomes significant only after the overburden has been weakened or thinned by tectonic extension or erosion.

II.3.2.1.5. Principal Types of Loading

Salt flow can be driven by three principal types of loading:

- A. Gravitational loading.
- B. Displacement loading.
- C. Thermal loading.

The relative importance of each mechanism depends on several factors, including the depth of salt burial, the geometry of the salt body, the geological setting, and the thermal conditions of the salt.

The movement of salt from regions subjected to higher loads toward areas of lower loads is commonly referred to as salt withdrawal. However, the more accurate term is salt expulsion, because the term withdrawal incorrectly suggests that a diapir actively draws salt from its surroundings. In reality, the salt is forced out of the source layer and into the diapir due to differential loading.

From a mechanical perspective, the interpretation of salt tectonics generally involves two main steps:

- Determining the direction of salt flow.
- Identifying the source of differential loading responsible for driving this flow.

II.3.2.1.6. Representative Geochemical Composition of Diapiric Formations

The following data illustrate the typical chemical and mineral composition of these salt formations, which play a key role in determining their mechanical response to different types of loading:

Table 7 : Representative Geochemical Composition of Diapiric Formations Adapted from generic geochemical data for evaporite diapirs (e.g., Warren, 2016; Schreiber et al., 2007).

Constituent Category	Element / Parameter	Unit	Average Content (Range)	Dominant Mineral Phases
Major Oxides	(SiO_2) Silica	%	5.2 – 0.5	Silicates ,Quartz
	Alumina (Al_2O_3)	%	2.8 – 0.1	Minerals (Illite) Clay
	Iron Oxide (Fe_2O_3)	%	1.5 – 0.05	Goethite ,Hematite
Ions Evaporite	(Na^+) Sodium	%	38.5 – 25.0	($NaCl$) Halite
	Chloride (Cl^-)	%	60.2 – 45.0	($NaCl$) Halite
	Sulfate (SO_4^{2-})	%	15.0 – 1.5	Gypsum ,Anhydrite
	Calcium (Ca^{2+})	%	12.0 – 0.5	Anhydrite ,Calcite
	Magnesium (Mg^{2+})	%	3.5 – 0.1	Carnallite ,Dolomite
Elements Trace	(Sr) Strontium	ppm	1500 – 100	Celestine
	Bromine (Br)	ppm	300 – 20	in Halite Substitution
Physical-Chemical	Dissolved Solids Total	mg/L	Variable	Concentration Brine
	Insoluble Residue	%	15.0 – 2.0	Carbonates ,Silicates

II.3.2.1.7. Gravitational Loading

Gravitational loading in geological settings results primarily from the weight of the overlying rock strata and the body forces acting within ductile materials such as salt formations. Over geological time scales, salt exhibits a viscous behavior that allows it to deform and flow in a manner comparable to highly viscous fluids. Due to this rheological behavior, the mechanical response of salt can be conceptually interpreted using fluid statics principles, particularly the notion of hydraulic head (Kehle, 1988). In this framework, deformation and flow occur in response to gradients in hydraulic head, with material movement directed from regions of higher head toward regions of lower head. Consequently, the direction of salt flow under gravitational loading can be predicted by estimating the hydraulic head gradient. When hydraulic head is spatially uniform, no net flow occurs and the system remains in equilibrium. The total hydraulic head (h) is conceptually composed of two main contributions: an elevation component (z), representing the vertical position relative to a reference datum, and a pressure-related component associated with the overburden load. This conceptual representation provides a qualitative description of pressure distribution within salt bodies and their tendency to deform under differential loading conditions.

II.3.2.1.8. Displacement Loading

Displacement loading occurs when one boundary of a rock body is forced to move relative to another (Suppe, 1985). In salt tectonics, this mechanism is observed when the flanks of a salt body move toward or away from each other due to regional tectonic shortening or extension. This process is particularly common in sedimentary basins containing pre-existing salt structures, as these structures tend to concentrate regional strain because salt behaves mechanically weaker than surrounding rocks.

II.3.2.2. Thermal Loading

In salt systems, thermal loading stems from volumetric changes driven by temperature fluctuations. When salt is heated, it expands thermally, which lowers its density and fosters the development of buoyancy forces. Under favorable conditions, these density contrasts can initiate convective circulation within the salt layer. The occurrence of thermal convection in evaporitic deposits has been proposed for several geological settings, including the Danakil Depression in Eritrea and the Boulby Potash Mine in England. These natural and industrial analogues indicate that heat-induced flow can happen under specific thermal and rheological conditions. The magnitude of thermal convection is governed by multiple critical factors, such as elevated temperature gradients, greater thermal expansivity, increased salt layer thickness, alongside reduced thermal diffusivity and kinematic viscosity. Taken together, these factors raise the likelihood of instability and convective motion inside salt bodies. Although the significance of thermal convection in salt diapirs contained within sedimentary sequences has not been firmly proven, it remains a reasonable mechanism that might assist the upward movement of deep, heated salt toward cooler and shallower structural levels.

Supporting this viewpoint, the vortex-like and coiled shapes observed in certain diapirs of the Great Kavir region in Iran are hard to interpret exclusively through mechanical processes, and could hint at the role of thermal convection during their formation (Jackson et al., 1990).

II.3.3. Mechanics of salt flow

II.3.3.1. Salt Tectonics Evolution: Buoyancy to Differential Loading

Historically, for much of the past 70 years, salt tectonics was dominated by the buoyancy-driven model. In this view, sediments were treated as a dense fluid with negligible yield strength, envisioned to sink into a less-dense salt layer and displace it upward to form diapirs—defined by Jackson and Talbot (1991) as masses that have flowed ductilely to pierce the overburden. However, since the late 1980s, the "fluid overburden" concept has been replaced by the recognition of roof strength and the physics of Differential Loading, as illustrated in the provided diagrams:

II.3.3.1.1. Factors Limiting Salt Movement

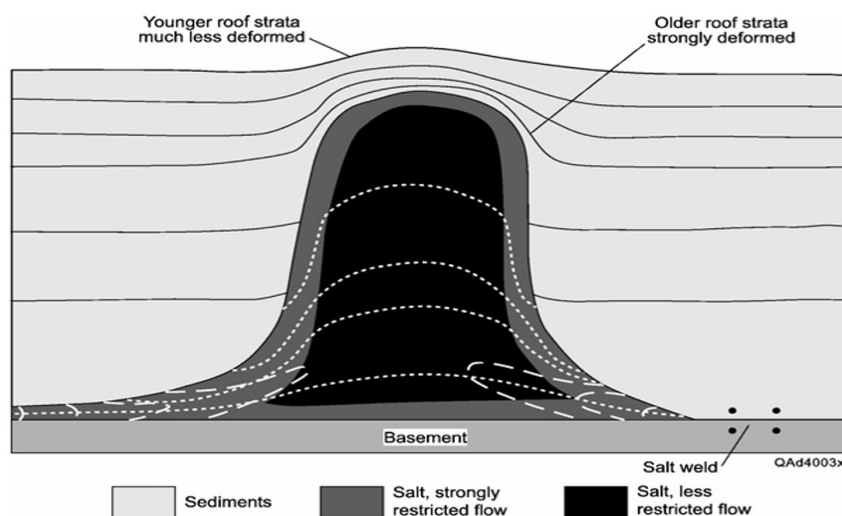


Figure 28 : Structural Characteristics and Deformation Patterns in Salt Diapirs.

While nature provides numerous ways to generate differential loads—suggesting that almost all salt bodies are under some driving force—not all of them actually move. Two primary factors act as a "brake" on salt deformation:

Roof Strength: The sedimentary rocks overlying the salt increase in shear and frictional strength as they are buried deeper (up to 8 km). Therefore, thick sedimentary roofs are much harder to deform. Without help from regional tectonic stretching or squeezing, a roof thicker than a few hundred meters is unlikely to be moved by salt alone.

Boundary Drag: As seen in the diagram, the ability of salt to flow is restricted by friction along its top and bottom contacts. This creates a "Strongly Restricted Flow" zone at the periphery, while the center of the salt body experiences "Less Restricted Flow."

II.3.3.1.2. Mechanisms of Salt Piercement

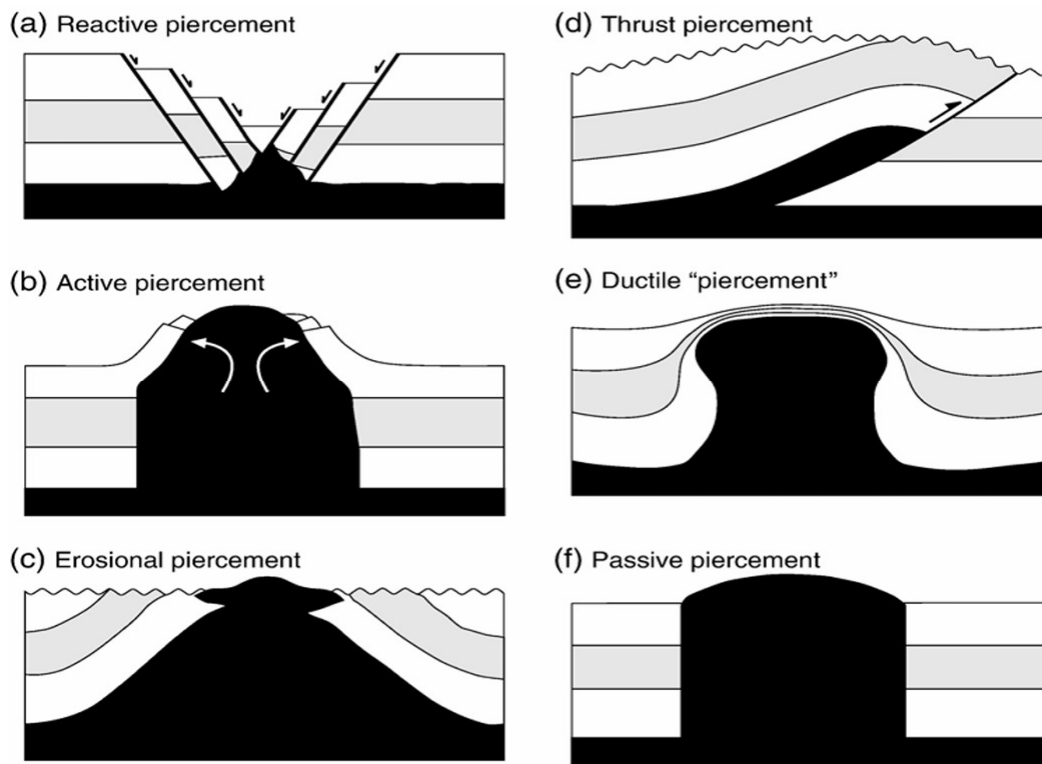


Figure 29 : Conceptual models of salt piercement mechanisms.

Table 8 : Description of piercement mechanisms.

Reactive Piercement:	Occurs as a response to regional extension (rifting) that thins and weakens the roof.
Active Piercement:	The salt has enough buoyancy or pressure to actively lift and deform the overlying sediments.
Erosional Piercement:	The roof is removed by surface erosion, allowing the salt to reach the surface.
Thrust:	Driven by compression or the folding of layers, squeezing the salt upward.
Ductile Piercement:	Driven by compression or the folding of layers, squeezing the salt upward.
Passive Piercement:	The salt remains at or near the surface while sediments continue to bury the surrounding areas.

II.3.3.1.3. Evolution of Salt Diapirism

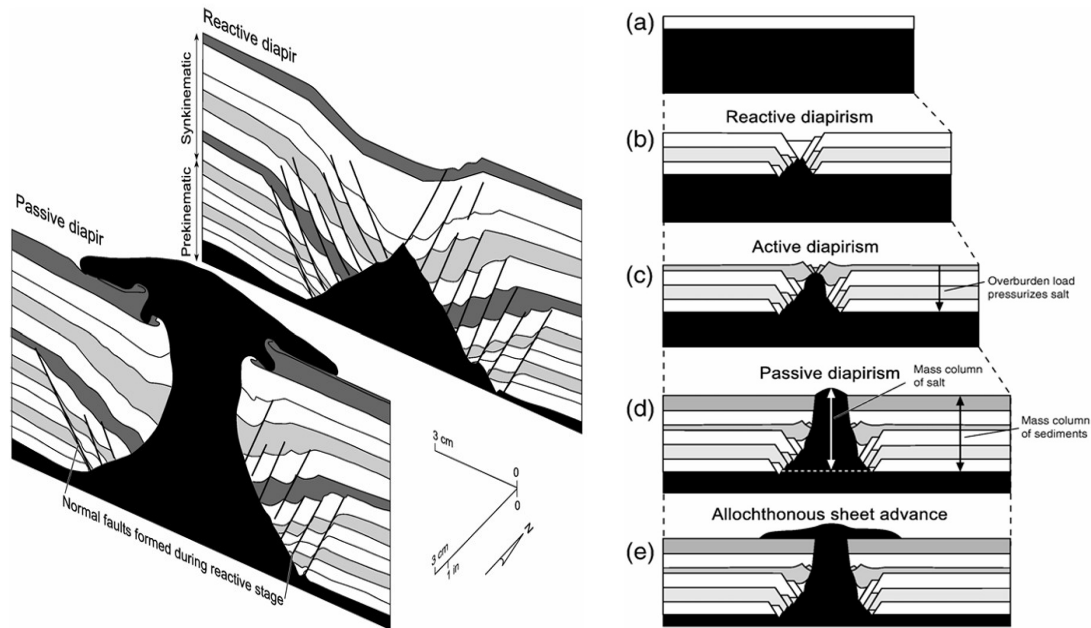


Figure 30 : Diapir piercement during regional extension Modified from Vendeville and Jackson (1992a).

The following table illustrates the sequential mechanical and temporal stages of salt diapir growth, ranging from the initial faulting phase to the lateral spreading of salt sheets.

Table 9 : Stages of Geological Development of Salt Diapirism.

1. Reactive stage:	Initial thinning of the roof due to faulting.
2. Active Stage:	The overburden load pressurizes the salt, forcing it to "punch" through the weakened roof.
3. Passive Stage:	A balance is reached where the salt column grows upward at the same rate as surrounding sediment deposition.
4. Allochthonous Sheet Advance:	In the final stage, the salt "overflows" its original position and spreads out laterally as a horizontal sheet over younger sediments.

The progression of a diapir through its evolutionary stages is not guaranteed; its maturity depends on three critical variables:

- **Salt Availability:** The source volume required to sustain growth.
- **Tectonic Extension:** The total amount of regional stretching.
- **Rate Interaction:** The competition between extension and sedimentation rates.

Salt cannot rise unless the overlying rock is displaced or removed. This occurs through :

- Regional extension, which creates spaces between rock masses.
- Erosion, which removes portions of the overlying rock.
- Active thrust of salt, which fractures the overlying rock and displaces it.
- The presence of thrust faults, which allow salt to intrude into their walls.

II.3.3.2. Factors Limiting Salt Deformation and Flow

While salt is known for its ability to flow under geological stress, this mobility is not absolute. According to Hudec and Jackson (2007), two primary mechanical factors limit the ability of salt bodies to move: the strength of the overburden and the dynamics of boundary drag.

II.3.3.2.1. Overburden Strength and Brittle-Ductile Constraints

The first limiting factor is the geomechanical strength of the overlying sediment. Within the crust—at least down to the brittle–ductile transition at approximately 8 km depth—sedimentary rocks typically exhibit an increase in both shear and frictional strength as burial depth and confining pressure rise. Consequently, thick sedimentary "roofs" are significantly more difficult to deform than thinner ones. Research suggests that roofs exceeding several hundred meters in thickness are unlikely to be displaced by salt with modest structural relief unless aided by regional tectonic extension or shortening.

II.3.3.2.2. Boundary Drag and Rheological Mechanisms

The second limitation involves boundary drag along the upper and lower contacts of the salt layer. The periphery of most salt bodies functions as a zone of restricted flow where the salt is sheared against stationary country rocks.

The characteristics of this shear zone depend heavily on the salt's flow law. As noted by **Van Keken (1993)** and **Spiers and Carter (1998)**, the flow law is governed by grain size and differential stress:

- **Newtonian Viscous Flow:** Results in a wider, more distributed boundary shear zone.
- **Power-Law Viscous Flow:** Results in a narrower, more intense shear zone where apparent viscosity decreases as the shear rate increases near the fluid boundary.

In thick salt layers, viscous shear forces are negligible as they affect only a minor portion of thickness. In thin layers, however, viscous resistance can immobilize the salt.

II.3.3.2.3. Thickness Thresholds and the Formation of Salt Welds

The relationship between layer thickness and mobility is non-linear. Following the equations provided by **Nelson (2001)**, the volumetric flux of laminar flow within a buried salt layer is proportional to the third power of the layer thickness (*Thickness*³). For example, halving the thickness of a salt layer reduces its flow rate by a factor of eight.

As salt expulsion continues, the layer may reach a threshold thickness (often a few tens of meters) where it becomes nearly immobile, regardless of the differential load applied. Ultimately, if the salt supply is exhausted through a combination of creep flow and dissolution, the top and bottom contacts merge to form a salt weld, effectively ending the growth of the associated dome.

II.3.3.3. Evolutionary Models of Diapirism (Figure Synthesis)

The morphology and maturity of salt structures are further dictated by the interplay between salt rise and external tectonic forces:

II.3.3.3.1. Passive Diapirism

According to Giles and Lawton (2002), diapir cross-sections are governed by the balance between net rise rate and sediment aggradation. Diapirs widen upward when rise outpaces aggradation, stay vertical

when rates are equal, and narrow or become buried when sedimentation overtakes rise (as shown by evolutionary stages in Previous Figures).

II.3.3.3.2. Reactive Piercement

During regional extension, diapirs undergo a multi-stage evolution. Physical models by Vendeville and Jackson (1992a) demonstrate that diapirism often begins reactively in the footwalls of normal faults. As the graben floor thins and faulting dissects the overburden, the salt eventually forces its way to the surface, transitioning from a reactive to a passive state (see Figure 3).

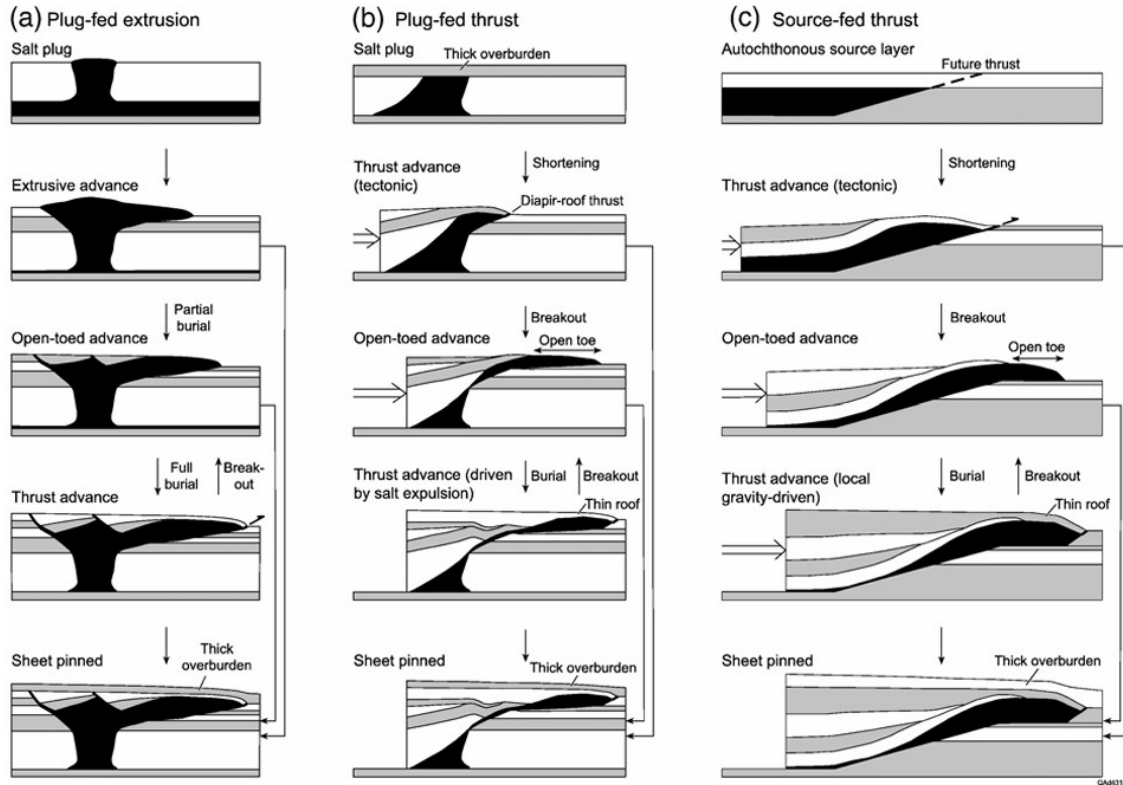


Figure 31 : Evolutionary lineages of the three primary salt-sheet types Modified from Hudec and Jackson, 2006.

Note: The diagram illustrates three distinct genetic lineages:

- A. Plug-fed extrusion.
- B. Plug-fed thrust.
- C. Source-fed thrust.

Vertical arrows between rows indicate observed growth sequences. The progression of these sheets may terminate at any stage within the lineage, governed by the interplay of salt supply, sedimentation rates, and regional tectonics.

II.4. Geotechnical and Structural Analysis of Diapiric Formations in Bekkaria (Tebessa)

The Tebessa region, specifically Bekkaria, is situated within the Saharan Atlas, a zone significantly influenced by the ascent of Triassic salts. This diapiric activity induces substantial deformation in the overlying sedimentary strata (limestone and marl), creating a highly heterogeneous geotechnical environment. This study links salt flow mechanics to field data and laboratory results from the ANESRIF railway project.

II.4.1. Structural Anatomy of the Diapir

Diapirs are complex tectonic systems rather than simple ascending salt masses. They consist of three primary mechanical zones:

The Stem: The ascending core composed of Triassic salts (halite, gypsum, anhydrite) mixed with "Variegated Clays" (Argiles Bariolées). It exhibits high plastic behavior, flowing under pressure.

Cap Rock: A rigid layer formed atop the salt dome due to the dissolution of soluble salts, leaving behind less soluble minerals like gypsum and calcite. In Bekkaria, this is observed as gypsiferous marl or weathered limestone.

Crushed/Shear Zone: The boundary between the diapir and the surrounding sedimentary rock. Intense friction during ascent creates a tectonic breccia (observed in borehole SCD 08 between 1.90m and 6.20m), representing a major geotechnical weakness.

II.4.2. Halokinesis: Mechanics of Salt Flow

Mechanics of Salt Flow Triassic salt behaves as a highly viscous fluid over geological time. Its ascent is driven by three primary triggers:

Differential Loading: Variations in the thickness of the overburden sedimentary layers drive salt from high-pressure zones to lower-confinement areas.

Tectonic Squeezing: Compressive forces during the formation of the Saharan Atlas "squeezed" the salt upward through structural faults.

Buoyancy: Salt is less dense than sedimentary rocks (like limestone), causing it to rise upward, similar to a bubble of oil in water.

II.4.3. Numerical Modeling and Stability Analysis

The Hoek-Brown criterion was used to model rock behavior. The selection of the Geological Strength Index (GSI) proved critical. A GSI of 40-55 was applied to the limestone, while the diapiric zones required significantly lower values (10-20).

II.4.4. Geotechnical Risks and Engineering Challenges

Located within the Tellian Atlas domain, the study area is strongly influenced by compressional tectonics, which has generated a dense fault network controlling both the regional relief and the structural organization of the geological formations (Bougherira et al., 2021). The proximity of the Djebel Doukan massif, with elevations exceeding 1,600 m, significantly affects runoff concentration, erosion processes, and slope instability mechanisms (IGN, 2021). In this context, the presence of Triassic diapiric formations introduces additional geotechnical complexity due to the combined effects of karstification, dissolution, mineralogical heterogeneity, and time-dependent deformation.

One of the principal engineering challenges concerns the chemical aggressiveness and mechanical instability of gypsum- and salt-rich formations. These materials are highly sensitive to water infiltration and dissolution processes, which may generate underground cavities, progressive settlement, and localized collapses. Hadji et al. (2017) emphasize that the karstification of limestone massifs and evaporitic formations represents a major hazard for linear infrastructure and underground works. Consequently, several protective measures have been implemented, including shotcrete applications for

slope stabilization and waterproofing geomembranes in Triassic sectors to reduce infiltration and limit dissolution phenomena (ANESRIF, 2022; TE-ISL, 2024a).

Unlike conventional brittle geomaterials, diapiric salt exhibits ductile rheological behavior characterized by steady-state creep under deviatoric stress. According to Jeremic (1994), this continuous deformation may progressively induce the closure of underground openings, excessive displacement around excavations, and instability of support systems. In infrastructure projects, long-term creep can also affect tunnel linings, retaining structures, and foundation performance, particularly where salt bodies are located at shallow depth.

Another major hazard is related to dissolution and karstification processes occurring when evaporitic formations interact with freshwater aquifers. Gutiérrez et al. (2008) demonstrated that salt dissolution sinkholes constitute a critical risk for roads, railways, and underground structures because the progressive removal of soluble material may trigger sudden surface subsidence. In diapiric terrains, these processes are often accelerated by the intense fracturing associated with tectonic deformation.

Slope stability is also strongly influenced by the presence of evaporitic formations. The inclusion of salt layers generally reduces the overall shear resistance of the rock mass and promotes plastic deformation along mechanically weak interfaces. In such conditions, numerical stability analyses require advanced constitutive approaches capable of representing both brittle and ductile behaviors simultaneously. As indicated in the technical documentation of Plaxis B.V. (2024), the shear strength reduction (SSR) method must be carefully adapted in diapiric environments because classical assumptions may underestimate the influence of salt flow and progressive failure mechanisms.

A further challenge arises from the pronounced structural and mineralogical heterogeneity within diapiric bodies. Diapirs rarely consist of pure halite; instead, they commonly form highly chaotic megabreccias composed of carbonates, clastics, gypsum, anhydrite, and fragmented evaporitic blocks (Jackson & Hudec, 2017). This heterogeneity generates significant spatial variability in stiffness, strength, and permeability parameters, making standard in-situ correlations such as CPT or SPT interpretations less reliable. As a result, boreholes located only a short distance apart may encounter radically different lithological and geomechanical conditions.

The cap rock overlying diapirs constitutes another critical zone of instability. Casas-Sainz et al. (2022) describe this layer as a highly fractured and heterogeneous boundary formed mainly by residual gypsum and anhydrite after salt dissolution. These zones frequently contain pressurized brine pockets capable of causing sudden borehole collapse, drilling instability, and environmental contamination if intersected during excavation or investigation works. Because of these exotic blocks, discontinuous lithological distributions, and variable mineralogical compositions, deterministic geomechanical models often fail to reproduce the stochastic nature of diapiric formations accurately (Li et al., 2020). For this reason, recent research increasingly recommends probabilistic numerical approaches capable of integrating uncertainties related to material distribution and mechanical variability.

From a geotechnical modeling perspective, it is essential to distinguish between the brittle mechanical response of the cap rock and the ductile flow behavior of the underlying salt core. The interface between these two contrasting mechanical domains frequen

Chapter 03: Geotechnical Investigations and data interpretations

III.1. Site Investigation – Geophysical Testing, Diamond Drilling, and Laboratory Characterization:

This chapter presents the comprehensive site investigation program carried out for the Tnoukla project in Tebessa, Algeria. The primary objective of this investigation is to thoroughly characterize the subsurface conditions along the proposed alignment, with a particular focus on identifying geological features—such as karstic voids, dissolution zones, and tectonic discontinuities—that could adversely affect slope stability.

The investigation integrates three complementary approaches, as detailed in the following sections: Geophysical seismic methods—comprising seismic refraction (conducted in accordance with AFNOR, 2012, NF P 94-424) and borehole techniques (crosshole and downhole tests)—were employed to delineate subsurface stratigraphy and to estimate key dynamic properties (e.g., shear wave velocity, maximum shear modulus, and Poisson's ratio) for both soil and rock units. Diamond core drilling was also conducted, involving eight boreholes (SCD-01 to SCD-08), with continuous core logging, determination of Total Core Recovery (TCR) and Rock Quality Designation (RQD), and systematic identification of tectonic or karstic features. Laboratory testing on representative undisturbed samples covered physical properties (density, water content, particle size distribution, Atterberg limits), chemical analyses (gypsum, sulfate, chloride, and carbonate contents), and mechanical parameters, including unconfined compressive strength (UCS), tensile strength, direct shear strength parameters, and standard penetration test (SPT) results. The results presented in this chapter provide the essential input parameters for the subsequent slope stability analyses.

III.1.1. Geophysical Investigation:

This section outlines the basic principles and equations utilized in seismic investigation (seismic refraction and borehole seismic methods), as well as the calculation of primary and secondary wave velocities and the dynamic property parameters of soil and rock (Gadallah & Fisher, 2009; Richart, Hall & Woods, 1970; Kramer, 1996).

Table 10 : Data Interpretation and Analytical Models.

Seismic Investigation Techniques	Seismic refraction	Primary Waves (P-waves)	$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}}$	Eq.(39)
		Secondary Waves (S-waves)	$V_s = \sqrt{\frac{G}{\rho}}$	Eq.(40)
		The depth (H) to the refracting interface	$H = \frac{x_c}{2} \cdot \sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$	Eq.(41)
Borehole Seismic Testing	Cross hole tests	Compression Wave Velocity (V_p)	$V_p = \frac{D}{t_p}$	Eq.(42)
		Shear Wave Velocity (V_s)	$V_s = \frac{D}{t_s}$	Eq.(43)
		Dynamic Shear Modulus (G_{max})	$G_{max} = \rho \cdot (V_s)^2$	Eq.(44)
		Dynamic Young's Modulus (E_{max})	$E_{max} = 2 \cdot G_{max} \cdot (1 + \nu)$	Eq.(45)
		Poisson's Ratio (ν)	$\nu = \frac{0.5 \cdot (V_p/V_s)^2 - 1}{(V_p/V_s)^2 - 1}$	Eq.(46)
		vertical travel time (t_v)	$t_v = t_m \cdot \cos\theta = t_m \cdot \frac{V_s}{V_p}$	Eq.(47)
	Downhole seismic tests	The Shear Modulus (G)	$G = \rho \cdot V_s^2 = \frac{\gamma}{g} \cdot V_s^2$	Eq.(48)

Table 11 : Typical Soil and Rock Parameters Based on Seismic Velocity Measurements (Keçeli, 2012).

Soil and Rock type	V_p (m/s)	V_s (m/s)	p (g/cm)	q_a (kg/cm ²)
Gabro	4500-6000	2700-4000	3.2-3.5	51.4-93
Granite	3300-5640	2000-3760	2.9-3.4	36-86
Schist	3200-5200	1454-3500	2.7-3.4	18-67
Limestone	1200-6190	600-3350	2.2-3.33	65-60
Mudstone	600-1900	300-700	1.8-2.26	2.8-5.8
Dilluvial gravel	900-2200	250-600	1.75-2.2	1.2-3.6
Gravel, dry sand	500-1000	200-300	1.7-1.8	1.3-1.6
Loose sand	600-1800	150-500	1.5-2	0.6-2.9
Aluvial gravel	400-1900	100-430	1.4-2	0.4-1.9
Dilluvial clay	500-1800	100-350	1.4-1.9	0.3-1.3
Alluvial clay	210-600	70-150	1.3-1.5	0.3-0.6

Table 12 : Stratigraphic velocity distribution and Stratum descriptions for the investigated profiles.

Feature	Profile ps1 & ps2 (clay-dominant)	Profile ps3 (rock-dominant)
Surface Stratum	$v_p \approx 300$ m/s (loose soil/unconsolidated material) (Alluvial clay).	High velocity $v_p = 3530$ m/s at the start of the profile (Schist or Granite)
Intermediate Stratum	v_p between 800 and 1319 m/s (transition to dense clay) (Dilluvial clay)	Values align with ps1/ps2 for the remainder of the profile
Deep Stratum	v_p between 1640 and 1770 m/s (coherent/compact materials) (Dilluvial clay)	Consistent with deeper consolidated formations

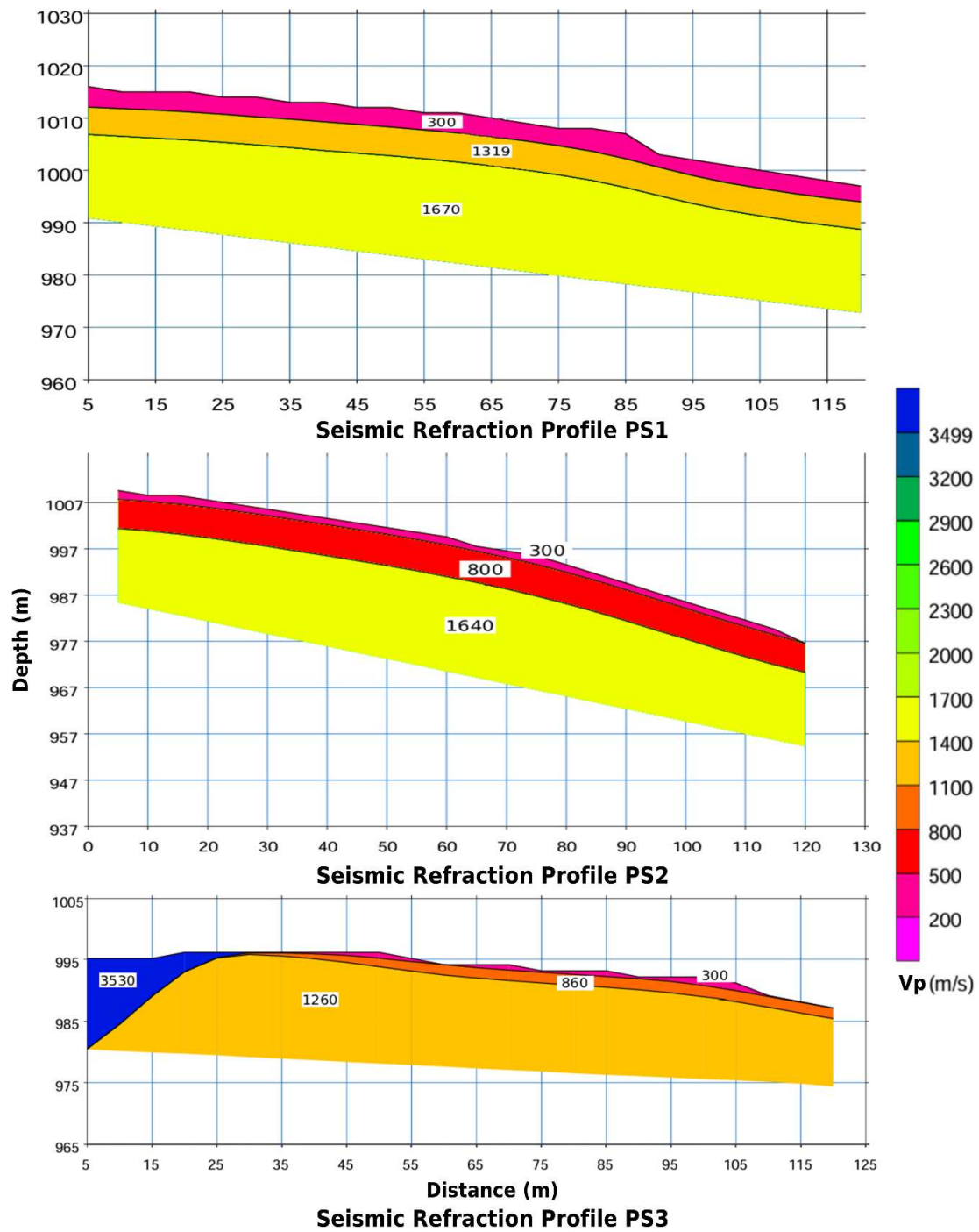


Figure 32 : Geophysical Interpretations of Subsurface Profiles (PS1–PS3).

III.1.2. Lithology description from diamond core drilling (dd)

The procedures and classification criteria for diamond core drilling, including the determination of Total Core Recovery (TCR) and Rock Quality Designation (RQD), follow the recommendations established by Deere and Deere (1988) and ASTM D6032/D6032M (2017). RQD values are calculated as the percentage of intact core pieces longer than 100 mm in the total length of each core run using the following equation:

$$RQD = \frac{\sum \text{Lengths of core pieces} > 10\text{cm}}{\text{Length of the interval}} \times 100 \quad \text{Eq.(49)}$$

III.1.2.1. Core drillings recovery of Tnoukla project Tebessa

All planned probing work has been completed according to the following data:

Table 13 : Progress Status of Boreholes.

N°	Designation	Inclination	Length (m)	< colspan="3" > Coordinates			Status
				X	Y	Z	
1	SCD-01	Incliné 30°	51	432766.75	3914747.392	1013.373	Completed
2	SCD-02	Vertical	50	433009.583	3914688.252		Completed
3	SCD-03	Vertical	60	432711.892	3914782.07	1003.98	Completed
4	SCD-04	Vertical	60	432769.69	3914720.018	1015.95	Completed
5	SCD-05	Vertical	60	432841.851	3914730.537	1009.857	Completed
6	SCD-06	Vertical	40	432741.7	3914836.9		Completed
7	SCD-07	Incliné 30°	53	432754.3684	3914789.0402	1013.29	Completed
8	SCD-08	Vertical	20	432989.5	3914691.95		Completed

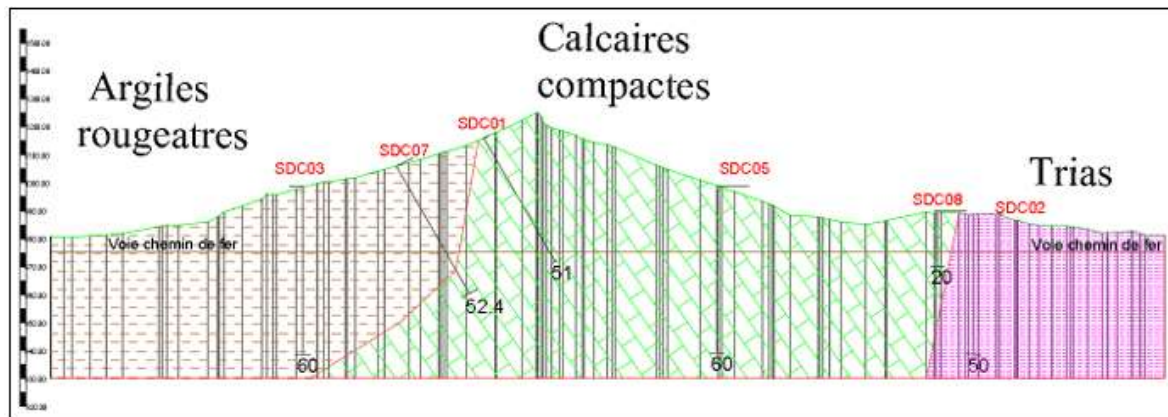


Figure 33 : Coupe géologique préliminaire suivant l'axe de la voie.

Note: Both SCD-07 and SCD-01 were executed with a 30° inclination to precisely determine the interface between the clay Stratum and the limestone substratum (Bautista & Aguilar, 2023; Kumar & Anbazhagan, 2025).

Observed formations :

- The sections reveal Strata of reddish clay (Argiles rougeâtres).
- compact limestone (Calcaires compactes).
- Triassic formations (Trias).

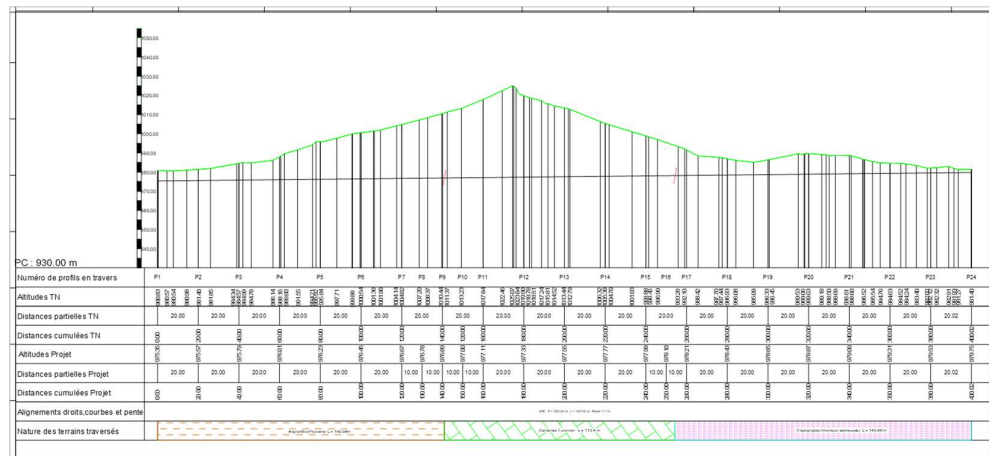


Figure 34 : Profils en long.

The project's longitudinal path has been divided into 24 cross sections, each with precise coordinates.

The distribution takes into account the geological variations of the site as follows:

- **Sections (01-09):** Cover the range of clay formations.
- **Sections (10-17):** Cover the range of compact limestone.
- **Sections (18-24):** Cover the range of Triassic formations.

The detailed lithological profiles for all exploratory boreholes are provided in the attached to the (ANNEX 01).

III.1.3. Laboratory Geotechnical Test Results

Annex 02 documents the detailed physical, chemical, and mechanical parameters obtained from the available tests on borehole samples SCD-02 to SCD-05.

III.2. Extraction of Input Parameters for Numerical Analysis

The numerical simulation phase requires transitioning from discrete laboratory and field data to continuous, representative profiles and values for all geomechanical formations and geological strata present at the study site. The methodology employed to characterize these diverse formations relies on deriving empirical regression equations, formulated through mathematical relationships to directly correlate the physical and mechanical properties of the materials with depth. As a general evaluation criterion, the coefficient of determination (R^2) was used to assess the quality of these regression models. The R^2 value, which ranges from 0 to 1, indicates the proportion of the variance in the dependent variable that is explained by the model. For evaluating the fit of regression models in engineering applications, an R^2 value of 0.70 or greater is generally considered suitable. The obtained R^2 values for the derived depth-dependent relationships consistently meet this threshold, confirming the reliability and predictive capability of the proposed equations.

III.2.1. Parameter Estimation for Soil Formations: Miocene-Pliocene Red Clays

The Hardening Soil (HS) model was selected to simulate the geotechnical behavior of the Miocene-Pliocene red clays due to its strong alignment with the laboratory results extracted from borehole SCD-03, based on the following justifications:

Stress-Dependent Stiffness: This model links soil stiffness to the confining pressure level, perfectly mirroring the borehole findings. The analysis revealed a substantial increase in the reference oedometer stiffness (E_{oed}) reaching up to 25,158 kPa in the deeper Strata as the confining pressure increased.

Plastic Strain & Hardening Representation: The model accurately captures the high plasticity of the on-site red clays, where the plasticity index (PI) ranged between 17.3 % and 26.9 %. This characterization is further validated by pre-consolidation pressure (σ'_p) values reaching up to 1,550 kPa and a compression index (C_c) of 0.176.

Excavation Simulation and Distinction Between Loading and Unloading (Loading vs. Unloading): The model stands out for its capacity to precisely differentiate between soil stiffness during primary loading and during unloading phases (e.g., excavation works). Oedometer data confirms this behavior, demonstrating that the swelling index (C_c) is remarkably small compared to the compression index. This results in highly elevated unloading/reloading stiffness (E_{ur}) values at depth, reaching up to 189,931 kPa, which is critical for reliably predicting slope stability and preventing overestimated excavation heave.

III.2.1.1. Stiffness Formulation and Constitutive Relationships

To precisely model the deformation of the Miocene-Pliocene red clays under the Hardening Soil framework, the formulation requires three distinct reference stiffness parameters evaluated at a reference pressure (p^{ref}): Secant Stiffness (E_{50}^{ref}) for triaxial loading, Oedometer Stiffness (E_{oed}^{ref}) for primary one-dimensional compression, and Unloading/Reloading Stiffness (E_{ur}^{ref}). The stress-dependency of these stiffness parameters is mathematically governed by the following power law:

$$E = E^{ref} \left(\frac{ccos\phi - \sigma'_3 \sin\phi}{ccos\phi + p^{ref} \sin\phi} \right)^m \quad \text{Eq.(50)}$$

$$E_{oed}^{ref} = \frac{2.303 \cdot p^{ref} \cdot (1 + e)}{C_c} \quad \text{Eq.(51)}$$

$$E_{ur}^{ref} = K_0^{nc} \cdot \left[\frac{2.303 \cdot p^{ref} \cdot (1 + e)}{C_s} \right] \cdot \left[\frac{(1 + \nu_{ur})(1 - 2\nu_{ur})}{(1 - \nu_{ur})} \right] \quad \text{Eq.(52)}$$

Where (m) is the amount of stress dependency, typically taken as 0.5 for sands and 1.0 for clays.

III.2.1.2. Depth-Dependent Regression Models

To transition from discrete laboratory measurements to continuous profiles across the eight designated strata, empirical regression models were fitted against depth (Depth). The derived mathematical relationships for the physical, strength, and reference pressure parameters are expressed as follows:

III.2.1.2.1. Compression Index (C_c) and Swelling Index (C_s)

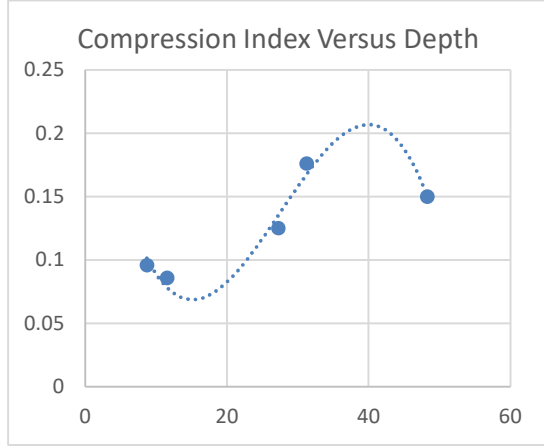


Figure 35 : Curve of Compression Index vs. Depth for Borehole SCD 03.

$$C_c \approx -0.000018 \times \text{Depth}^3 + 0.001488 \times \text{Depth}^2 - 0.03266 \times \text{Depth} + 0.28473$$

The cubic regression for the compression index (C_c) gives $R^2 = 0.9527$, indicating an excellent fit to the measured data.

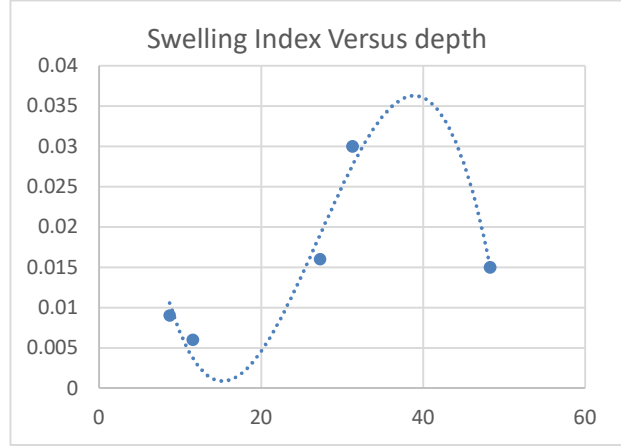


Figure 36 : Curve of Swelling Index vs. Depth for Borehole SCD 03.

$$C_s \approx -(5.3716E-06) \times \text{Depth}^3 + 0.000436 \times \text{Depth}^2 - 0.009556 \times \text{Depth} + 0.06421$$

The cubic regression for the swelling index (C_s) gives $R^2 = 0.9351$, reflecting a very strong correlation with depth.

Table 14 : Consolidation Characterization (C_c and C_s Profiles) of Soil Strata for Borehole SCD 03.

D (m)	C_c	C_c (Moy)	C_s	C_s (Moy)
0	0.28	0.264739	0.064	0.058365
1.3	0.24		0.053	
1.3	0.24	0.219801	0.053	0.045219
3.2	0.19		0.038	
3.2	0.19	0.18369	0.038	0.033378
31.9	0.17		0.029	
31.9	0.17	0.180196	0.029	0.030747
34.2	0.19		0.033	
34.2	0.19	0.192355	0.033	0.033696
35.9	0.20		0.035	
35.9	0.20	0.200786	0.035	0.035462
38.2	0.20		0.036	
38.2	0.20	0.20379	0.036	0.035014
42.3	0.20		0.034	
42.3	0.20	0.201425	0.034	0.033918
60	0.20		0.034	

III.2.1.2.2. Effective Cohesion (c') Effective Friction Angle (ϕ')

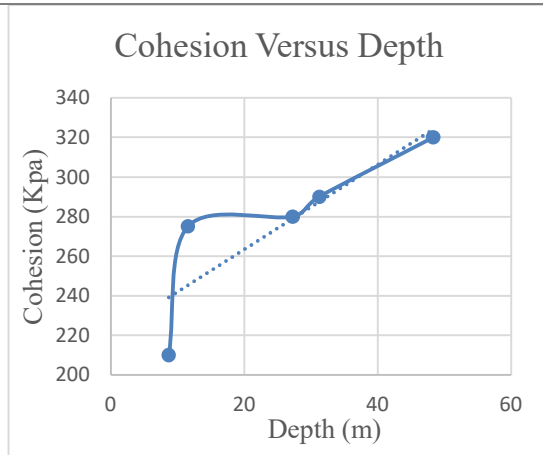


Figure 37 : Curve of Cohesion vs. Depth for Borehole SCD 03.

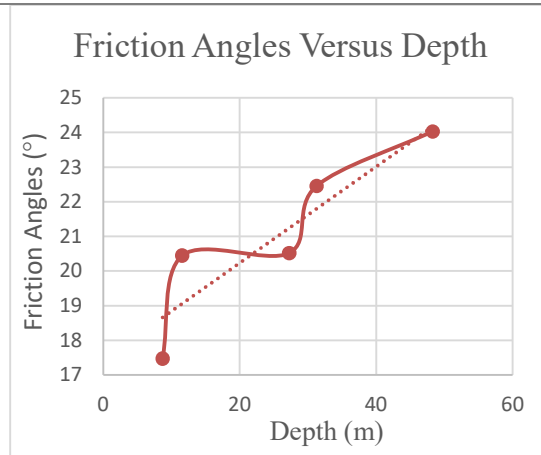


Figure 38 : Curve of Friction Angles vs. Depth for Borehole SCD 03.

$$C = f(\text{Depth}) = 2.1442 \times (\text{Depth}) + 220.5$$

The linear regression for effective cohesion (c') yields $R^2 = 0.73$, which is considered acceptable given the natural scatter of shear strength data in over consolidated clays.

$$\phi^\circ = f(\text{Depth}) = 0.1391 \times (\text{Depth}) + 17.449$$

The linear regression for the effective friction angle (ϕ') gives $R^2 = 0.8223$, indicating a good correlation with depth.

Table 15 : Shear Strength Parameters (Cohesion and Friction Angle) with Depth for Borehole SCD 03.

Depth (m)	Cohesion - C (Kpa)	Cohesion Moy - C (Kpa)	Friction Angle - ϕ (°)	Friction Angle Moy (°)
0	220.50		17.45	
1.3	223.29	221.89	17.63	17.54
1.3	223.29		17.63	
3.2	227.36	225.32	17.89	17.76
3.2	227.36		17.89	
31.9	288.90	258.13	21.89	19.89
31.9	288.90		21.89	
34.2	293.83	291.37	22.21	22.05
34.2	293.83		22.21	
35.9	297.48	295.65	22.44	22.32
35.9	297.48		22.44	
38.2	302.41	299.94	22.76	22.60
38.2	302.41		22.76	
42.3	311.20	306.80	23.33	23.05
42.3	311.20		23.33	
60	349.15	330.18	25.80	24.56

III.2.1.2.3. Reference Pressure (p^{ref})

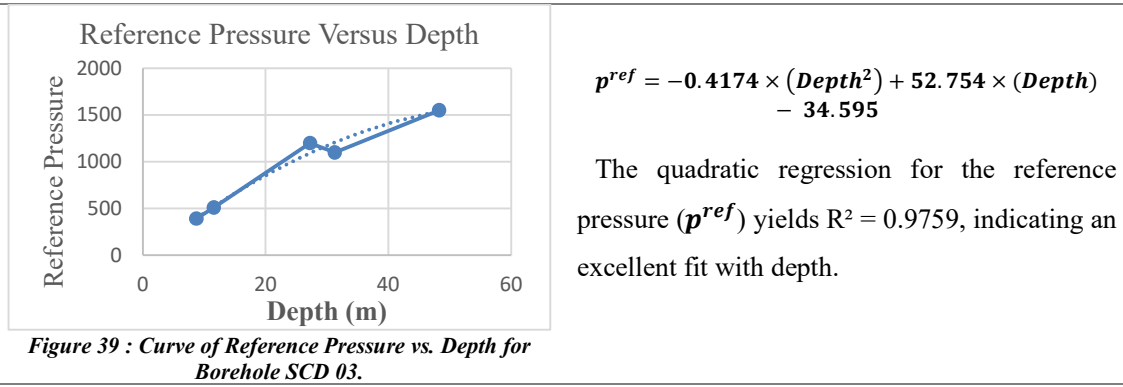


Table 16 : Subsurface Reference Pressure Profiles for Geotechnical Numerical Modeling for Borehole SCD 03.

Depth (m)	Reference Pressure (p^{ref})	Reference Pressure Moy (p^{ref})
0	0.00	
1.3	33.28	16.64
1.3	33.28	
3.2	129.94	81.61
3.2	129.94	
31.9	1223.51	676.73
31.9	1223.51	
34.2	1281.38	1252.45
34.2	1281.38	
35.9	1321.32	1301.35
35.9	1321.32	
38.2	1371.52	1346.42
38.2	1371.52	
42.3	1450.05	1410.79
42.3	1450.05	
60	1628.01	1539.03

III.2.1.2.4. Initial Void Ratio (e)

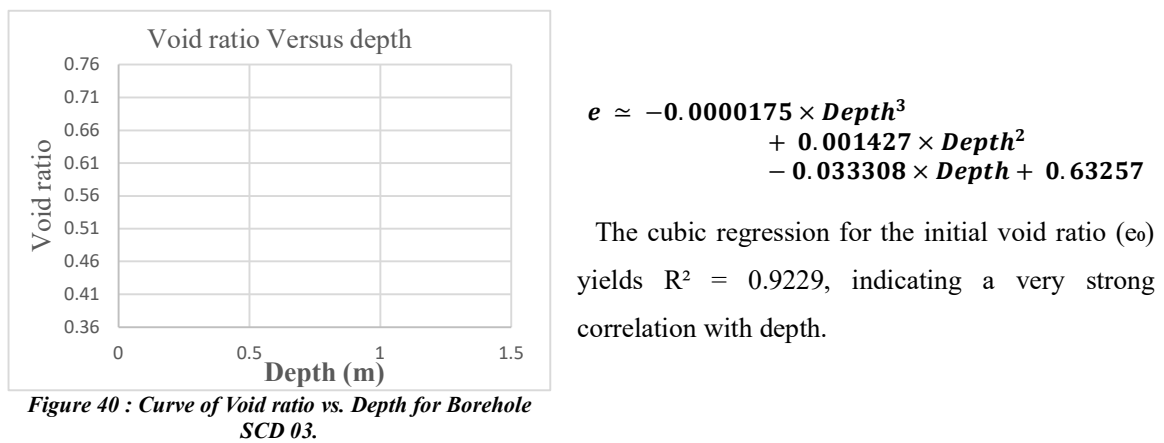


Table 17 : Initial Void Ratio (e_0) and Its Mean Values with Depth for Borehole SCD 03.

Depth (m)	initial void ratio (e_0)	initial void ratio Moy (e_0)
0	0.63	0.61
1.3	0.59	
1.3	0.59	
3.2	0.54	0.57
3.2	0.54	
31.9	0.45	
31.9	0.45	0.50
34.2	0.46	
34.2	0.46	
35.9	0.47	0.46
35.9	0.47	
38.2	0.47	
38.2	0.47	0.47
42.3	0.45	
42.3	0.45	
60	0.40	0.43

III.2.1.2.5. Unsaturated Unit Weight (γ_{unsat})

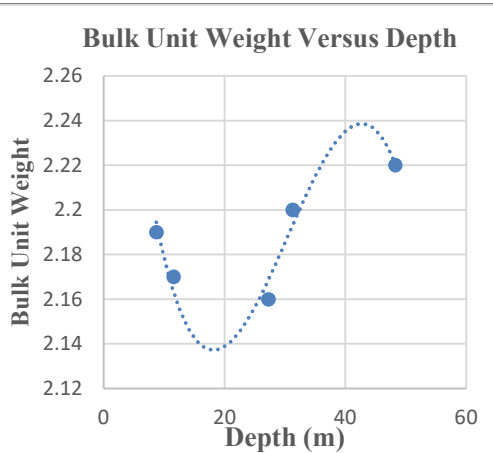


Figure 41 : Curve of Bulk Unit Weight vs. Depth for Borehole SCD 03.

$$\gamma_{unsat} \approx -0.0000138 \times Depth^3 + 0.00126 \times Depth^2 - 0.03211 \times Depth + 2.38759$$

The cubic regression for the unsaturated unit weight (γ_{unsat}) yields $R^2 = 0.9184$, indicating a very strong correlation with depth.

Table 18 : Physical Properties (Density and Unit Weight) for Geotechnical Characterization for Borehole SCD 03.

Depth (m)	γ_{unsat} (t/m^3)	γ_{unsat} Moy (t/m^3)	γ_{unsat} Moy (kN/m^3)
0	2.39	23.42	23.23
1.3	2.35	23.03	
1.3	2.35	23.03	
3.2	2.3	22.54	22.78
3.2	2.3	22.54	
31.9	2.2	21.55	
31.9	2.2	21.55	22.04
34.2	2.21	21.69	
34.2	2.21	21.69	
35.9	2.22	21.77	21.73
35.9	2.22	21.77	
38.2	2.23	21.87	
38.2	2.23	21.87	21.82
42.3	2.24	21.96	
42.3	2.24	21.96	21.92
60	2.2	21.58	

III.2.1.2.6. Saturated Unit Weight (γ_{sat})

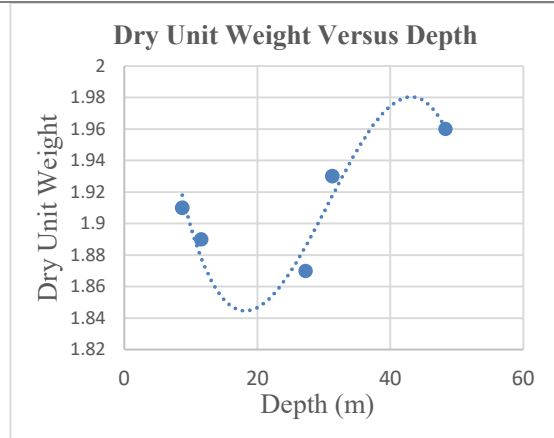


Figure 42 : Curve of Dry Unit Weight vs. Depth for Borehole SCD 03.

$$\gamma_{dry} \approx -0.0000175 \times Depth^3 + 0.001607 \times Depth^2 - 0.04108 \times Depth + 2.1652$$

The cubic regression for the dry unit weight (γ_{dry}) yields $R^2 = 0.8771$, indicating a good fit with depth.

To calculate the saturated unit weight (γ_{sat}) of the clayey formations under fully saturated conditions, the fundamental phase relationship linking the total unit weight to the void ratio and the specific gravity of solids was utilized according to the following equation: $\gamma_{sat} = \frac{G_s + e}{1 + e} \cdot \gamma_w$

Table 19 : Table for calculating the saturated unit weight (γ_{sat}) for Borehole SCD 03.

Depth (m)	ρ_{dry} (t/m ³)	γ_{dry} (kN/m ³)	$\gamma_{dry} Moy$ (t/m ³)	e	G_s	γ_{sat} (kN/m ³)
0	2.17	21.24				
1.3	2.11	20.74	20.9928	0.61	3.72	24.71
1.3	2.11	20.74				
3.2	2.05	20.11	20.42589	0.57	3.56	23.99
3.2	2.05	20.11				
31.9	1.92	18.86	19.4849	0.5	3.27	22.75
31.9	1.92	18.86				
34.2	1.94	19.04	18.95037	0.46	3.09	22.04
34.2	1.94	19.04				
35.9	1.95	19.16	19.0982	0.46	3.09	22.19
35.9	1.95	19.16				
38.2	1.97	19.29	19.22516	0.47	3.14	22.36
38.2	1.97	19.29				
42.3	1.98	19.42	19.35792	0.46	3.09	22.45
42.3	1.98	19.42				
60	2.2	21.58	20.50258	0.43	2.95	23.45

III.2.1.3. Poisson's Ratio (ν_{ur})

This value is consistent with the typical ranges provided in literature for the soil types encountered in the project area, as shown in Table 35.

Table 20 : Typical values of Poisson's ratio (ν_u) for various soil types (Adapted from Bowles, 1996).

Type of Soil	Poisson's ratio, ν
Clay, saturated	0.50 – 0.40
Clay, unsaturated	0.30 – 0.10
Sandy clay	0.30 – 0.20
Silt	0.35 – 0.30
Sand, gravelly sand	0.40 – 0.30
Rock	0.40 – 0.10
Loess	0.30 – 0.10

Table 21 : Subsurface Lithological Profile and Poisson's Ratio Calibration for Numerical Modeling for Borehole SCD 03.

Depth (m)	Lithological Description	ν : Value	Technical Justification
0.00 – 1.30	Alluvium: Gravel, pebbles in clayey-silty-sandy matrix.	0.35	Typical for mixed deposits (Gravelly sand range: 0.30–0.40).
1.30 – 3.20	Plastic Clay: Greenish-brown with gypsum and oxidation.	0.42	High plasticity/weathering suggests higher moisture (Saturated Clay range).
3.20 – 31.90	Compact Clay: Brown; sensitive to water content changes.	0.3	Unsaturated compact state (Unsaturated Clay range: 0.10–0.30).
31.90 – 34.20	Weathered Compact Silty Clay: With localized marl.	0.32	Transition zone; slightly higher due to silt/marl content.
34.20 – 35.90	Compact Clay: Similar to the upper compact Stratum.	0.3	Consistent with the unsaturated compact clay classification.
35.90 – 38.20	Firm Silty-Sandy Clay: Dark brown.	0.28	Within the Sandy Clay range (0.20–0.30) for firm consistency.
38.20 – 42.30	Marly Clay: Slightly gravelly, firm.	0.33	Characteristic of Marly/Silty behavior in a firm state.
42.30 – 60.00	Compact Clay: Consistent clayey substratum.	0.3	Deep compact clay in a stable, unsaturated state.

III.2.2. Parameter Estimation for Triassic Formations

For the complex and highly heterogeneous Triassic formations, given that the available laboratory data (specifically from borehole SCD-02 samples) is limited and lacks advanced mechanical testing, a geotechnical benchmarking approach was adopted. This methodology relies on implementing well-documented, validated parameters from established literature and international databases for geological materials that share analogous lithological descriptions, physical properties, depositional origins, and geological ages.

III.2.2.1. Selection of constitutive models for Triassic formations

Soft Soil Model: This model was applied to the upper stiff clay Stratum (from 0.0 to 14.4 meters) to accurately capture the initial hardening behavior and volumetric yielding. Laboratory tests conducted on borehole SCD-02 at shallow depths (from 6.45 to 7.00 meters) classified the material as low-plasticity clay (CL) with a high percentage of fine particles (88.92% smaller than 80 micrometers) and a plasticity index (PI) of 16.35%. Given the low over consolidation ratio ($OCR = 1.3$) and the presence of gypsum traces, the Soft Soil model effectively captures its highly compressible nature, ensuring realistic settlement predictions during initial loading stages rather than overestimating the shear hardening component.

HS small model: For medium and deep formations – including compacted marly clay, varied clayey marl, and deep clayey marl extending to a depth of 50.0 meters – the HS small model was selected to simulate their rock-like stiff response. The physical index parameters from SCD-02 indicate a gradual increase in density and cohesion with depth, where dry density (ρ_d) reaches 2.1 t/m³ and sand content rises to 48%. These cohesive clay Strata exhibit nonlinear elastic behavior and high sensitivity to stress amplitude during loading-unloading cycles (e.g., excavation works). The HSsmall model dynamically captures this behavior through a stress-dependent reference stiffness that increases with depth, and

accounts for stiffness degradation at very small strains by incorporating the initial small-strain shear modulus, thereby preventing serious underestimation of ground movements around deep open trenches.

Mohr-Coulomb Model: The elastic-perfectly plastic Mohr-Coulomb model was isolated and assigned exclusively to the highly weathered clayey passage located between the deep marly zones. Geologically, this narrow zone represents a local weakness area and a brittle gap within the surrounding stiff marly mass, as its structure has undergone intense weathering and disturbances leading to significant deterioration of its shear strength properties.

Since this structurally damaged zone undergoes direct shear failure and exhibits full plasticity immediately upon reaching the strength limit, rather than displaying gradual and continuous hardening behavior, the Mohr-Coulomb model provides a realistic and straightforward failure envelope that fully matches this mechanism. Furthermore, restricting the use of advanced constitutive models to the surrounding cohesive Strata while employing this conventional model in this narrow and critical inter Stratum zone is a necessary engineering step to ensure numerical stability and computational convergence within the finite element program.

III.2.2.2. Derivation of Geotechnical Parameters

III.2.2.2.1. Unit Weight Profiles

Table 22 : Average unit weight profile with depth – unsaturated (γ_{unsat}) and saturated (γ_{sat}) condition.

Depth (m)	Depth Moy (m)	γ_{unsat} Moy (kN/m^3)	γ_{sat} Moy (kN/m^3)
0	7.2	18	20
14.4			
14.4	17.7	19	21
21			
21	31.95	19.5	21.5
42.9			
42.9	43.5	17	19
44.1			
44.1	47.05	20	22
50			

III.2.2.2.2. Shear Strength Parameters

Table 23 : Average shear strength parameters – cohesion (c') and friction angle (ϕ') with depth.

Depth (m)	Depth Moy (m)	Cohesion Moy - C (Kpa)	Friction Angle Moy (°)
0	7.2	35	22
14.4			
14.4	17.7	70	28
21			
21	31.95	120	31
42.9			
42.9	43.5	18	17
44.1			
44.1	47.05	160	34
50			

III.2.2.2.3. Poisson's Ratio (ν_{ur})

This value is consistent with the typical ranges provided in literature for the soil types encountered in the project area, as shown in Table 35.

Table 24 : Unloading/reloading Poisson's ratio (ν_{ur}) per stratum with technical justification.

Depth (m)	Lithological Description	ν_{ur}	Technical Justification
0.00 – 14.40	Firm Clay Rich in Gypsum	0.15	Within unsaturated clay range (0.10–0.30); low value due to low overconsolidation (OCR=1.3) and near-elastic unload/reload behavior.
14.40 – 21.00	Compact Marly Clay	0.25	Within sandy clay range (0.20–0.30); moderate value reflects dense gypsum-rich marl with small-strain response under cyclic loading.
21.00 – 42.90	Variegated Argillaceous Marl	0.22	Within rock (0.10–0.40) or sandy clay range; relatively low for quasi-rock, high-stiffness formation.
42.90 – 44.10	Highly Weathered Clayey Passage	0.33	Within silt (0.30–0.35) and saturated clay range; moderately high for a localized weak zone with reduced stiffness.
44.10 – 50.00	Deep Argillaceous Marl	0.2	Within rock range (0.10–0.40); low value representing highly compact, low-permeability substratum with elastic unload/reload behavior.
0.00 – 14.40	Firm Clay Rich in Gypsum	0.15	Within unsaturated clay range (0.10–0.30); low value due to low overconsolidation (OCR=1.3) and near-elastic unload/reload behavior.
14.40 – 21.00	Compact Marly Clay	0.25	Within sandy clay range (0.20–0.30); moderate value reflects dense gypsum-rich marl with small-strain response under cyclic loading.
21.00 – 42.90	Variegated Argillaceous Marl	0.22	Within rock (0.10–0.40) or sandy clay range; relatively low for quasi-rock, high-stiffness formation.

III.2.2.2.4. Stratum 1 (Soft Soil model)

For the Soft Soil model used in the uppermost Stratum (0.0–14.4 m), the following parameters are adopted:

Table 25 : Modified compression and swelling indices (λ and κ) for the Soft Soil model.

Modified compression index	Modified swelling index
λ^*	κ^*
0.0067	0.0047

III.2.2.2.5. Typical Ranges of Reference Secant Stiffness (E_{50}^{ref})

Table 26 : Typical Ranges of E_{50}^{ref} for Different Soil Types (Compiled from Published Geotechnical Literature).

Soil Type	E_{50}^{ref}
Soft Clay	3.5 – 30
Medium Clay	20 – 50
Hard Clay	40 – 80
Sandy Clay	70 – 180
Stiff Marl	~90 – 120
Coarse Sand	500 – 800

III.2.2.3. Stiffness Formulation and Constitutive Relationships

In the Hardening Soil (HS) and Hardening Soil with Small-Strain Stiffness (HSsmall) models, several stiffness parameters are interrelated to provide a realistic representation of soil deformation behavior. When direct laboratory or in-situ measurements are unavailable, empirical relationships commonly adopted in geotechnical practice may be used for preliminary parameter estimation.

III.2.2.3.1. Unloading/Reloading Stiffness

The unloading–reloading stiffness is generally assumed to be greater than the secant stiffness used for primary loading. A commonly adopted approximation in the Hardening Soil model is:

$$E_{ur}^{ref} \approx 3 \times E_{50}^{ref} \quad \text{Eq.(53)}$$

where:

- E_{50}^{ref} = reference secant stiffness in standard drained triaxial loading.
- E_{ur}^{ref} = reference unloading/reloading stiffness.

This ratio reflects the fact that soil exhibits a significantly stiffer response during unloading and reloading than during initial loading.

III.2.2.3.2. Estimation of the Small-Strain Shear Modulus

When small-strain measurements are not available, the initial shear modulus may be estimated using the elastic relationship:

$$G_0^{ref} \approx \frac{E_{ur}^{ref}}{2(1 + \nu_{ur})} \quad \text{Eq.(54)}$$

where: G_0^{ref} = small – strain shear modulus

his equation is derived from the theory of isotropic linear elasticity and is typically used only as a preliminary estimate. More reliable values should preferably be obtained from laboratory testing or geophysical measurements such as shear-wave velocity (V_s) determination.

III.2.3. Parameter Estimation for Rock Formations: Compact Turonian Limestones

The Hoek-Brown criterion was used to estimate the mechanical properties of the Turonian compact limestone in boreholes SCD-04 and SCD-05, based on laboratory results and lithological descriptions. The choice of this criterion stems from three main reasons. First, the criterion directly relies on the unconfined compressive strength (σ_{ci}) obtained from laboratory tests. The data show clear variations in this strength depending on depth and rock type, ranging between 57.3 and 108.7 MPa in SCD-05, and between 19.9 and 140.5 MPa in SCD-04, especially in karst-affected or fractured zones. Second, the criterion allows representing the non-linear behavior of rock under confining stresses. This is necessary due to the presence of low-quality rock intervals, such as an RQD value of 53% in SCD-05 and open karstic voids in SCD-04. The estimated GSI values range between 20 and 75, and mi coefficients between 8 and 10, which are suitable inputs for the Hoek-Brown criterion. Third, the criterion distinguishes between intact rock properties and rock mass behavior through the elastic modulus (E'_{rm} and Poisson's ratio ν), estimated for each lithological interval separately, such as the value of 647 MPa in the fractured clay matrix zone, up to 31,480 MPa in the hard intact limestone. This differentiation prevents overestimating deformations when simulating deep excavations.

In summary, the Hoek-Brown criterion was selected because it can accommodate the actual variability in geotechnical data (UCS, GSI, E'_{rm}) without requiring arbitrary adjustments suited for soil models.

III.2.3.1. Depth-Dependent Regression Models

III.2.3.1.1. Unconfined Compressive Strength (σ_{ci})

The average UCS value for each Stratum was used. $\sigma_{ci} \approx \text{Average UCS (MPa)} \times 1000 = [kN/m^2]$

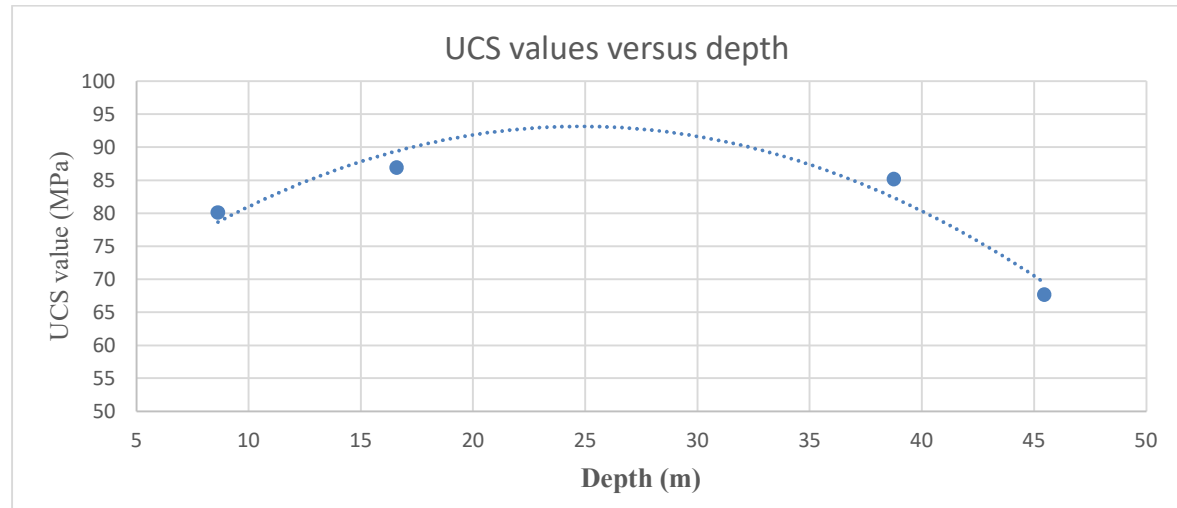


Figure 43 : Mathematical modeling of UCS values versus depth for Borehole SCD 05.

The quadratic regression for UCS in borehole SCD-05 yields $R^2 = 0.9128$, indicating a very strong correlation with depth.

$$\text{UCS (MPa)} = -0.0555 \times \text{Depth}^2 + 2.7505 \times \text{Depth} + 59.039$$

Table 27 : Summary of Depth-Dependent Unconfined Compressive Strength (σ_{ci}) for Borehole SCD 05.

Stratum	Depth (m)	UCS value	Unconfined Compressive Strength (σ_{ci})	UCS Moy value	Unconfined Compressive Strength Moy (σ_{ci})
1	0	59.04	59039.00	67.11	67107.54
	6.8	75.18	75176.08		
2	6.8	75.18	75176.08	76.61	76610.42
	8.3	78.04	78044.76		
3	8.3	78.04	78044.76	82.82	82817.22
	14.8	87.59	87589.68		
4	14.8	87.59	87589.68	90.34	90336.34
	24	93.08	93083.00		
5	24	93.08	93083.00	91.65	91653.00
	32	90.22	90223.00		
6	32	90.22	90223.00	91.65	91653.00
	24	93.08	93083.00		
7	24	93.08	93083.00	88.25	88249.50
	38	83.42	83416.00		
8	38	83.42	83416.00	73.30	73303.50
	48	63.19	63191.00		
9	48	63.19	63191.00	43.73	43730.00
	60	24.27	24269.00		

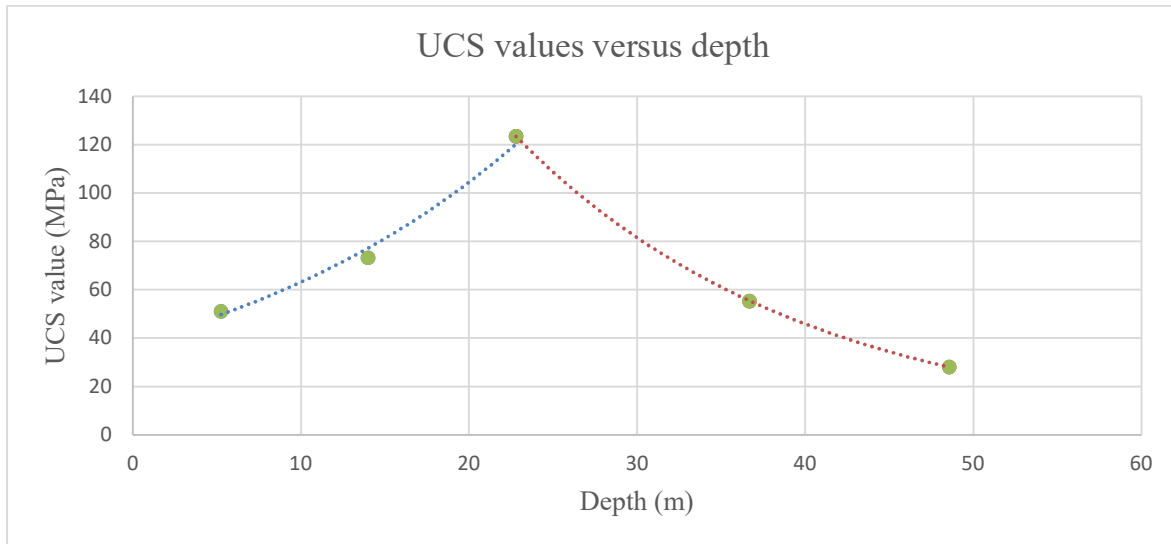


Figure 44 : Mathematical modeling of UCS values versus depth for Borehole SCD 04.

- $0 \text{ m} \leq \text{Depth} \leq 22.8 \text{ m}$:

The exponential regression for UCS in borehole SCD-04 ($0 \leq \text{Depth} \leq 22.8 \text{ m}$) gives $R^2 = 0.9911$, indicating an excellent fit.

$$\text{UCS (MPa)} = 38.202 \times e^{0.0503 \times \text{Depth}}$$

- $22.8 \text{ m} \leq \text{Depth} \leq 60 \text{ m}$:

The exponential regression for UCS in borehole SCD-04 ($22.8 \leq \text{Depth} \leq 60 \text{ m}$) yields $R^2 = 1.00$, reflecting a nearly perfect fit to the measured data.

$$\text{UCS (MPa)} = 458.59 \times e^{-0.058 \times \text{Depth}}$$

Table 28 : Summary of Depth-Dependent Unconfined Compressive Strength (σ_{ci}) for Borehole SCD 04.

Stratum	Depth (m)	UCS value	Unconfined Compressive Strength (σ_{ci})	UCS Moy value	Unconfined Compressive Strength Moy (σ_{ci})
1	0	38.20	38202.00	44.42	44416.60
	5.6	50.63	50631.19		
2	5.6	50.63	50631.19	50.63	50631.19
	6.4	50.63	50631.19		
3	6.4	52.71	52710.14	52.71	52710.14
	6.6	52.71	52710.14		
4	6.6	53.24	53243.08	53.24	53243.08
	8	53.24	53243.08		
5	8	57.13	57127.63	57.13	57127.63
	8.9	57.13	57127.63		
6	8.9	59.77	59773.23	59.77	59773.23
	15.5	59.77	59773.23		
7	15.5	83.31	83307.44	83.31	83307.44
	16.4	83.31	83307.44		
8	16.4	87.17	87165.44	87.17	87165.44
	27.7	87.17	87165.44		
9	27.7	153.88	153883.82	153.88	153883.82
	33.7	153.88	153883.82		
10	33.7	64.95	64946.03	64.95	64946.03
	52.1	64.95	64946.03		
11	52.1	22.34	22339.50	22.34	22339.50
	58.4	22.34	22339.50		
12	58.4	15.50	15501.81	14.81	14814.89
	60	14.13	14127.97		

III.2.3.1.2. Tensile Strength (σ_t)

Calculated in the report using the splitting formula:

$$\sigma_t = f_{ct} = \frac{2 \times F}{\pi \times L \times d} \quad \text{Eq.(55)}$$

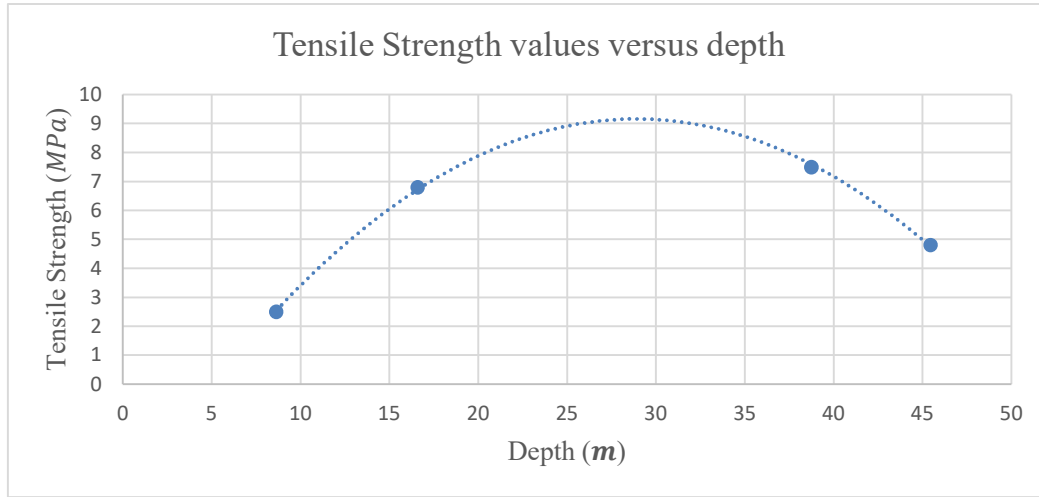


Figure 45 : A Study of the Correlation Between Depth and the Tensile Strength of the SCD 05 borehole.

$$\sigma_t \text{ (MPa)} = -0.0161 \times \text{Depth}^2 + 0.9307 \times \text{Depth} - 4.2892$$

The quadratic regression for tensile strength in borehole SCD-05 gives $R^2 = 0.9986$, indicating an almost perfect fit with depth.

Table 29 : Summary of Depth-Dependent Tensile Strength (σ_t) for Borehole SCD 05.

Stratum	Depth (m)	(σ_t) value (MPa)	(σ_t) Moy value (MPa)	Tensile Strength Moy σ_t (kN/m ²)
1	0	0.00	0.65	647.55
	6.8	1.30		
2	6.8	1.30	1.81	1810.79
	8.3	2.33		
3	8.3	2.33	4.14	4142.55
	14.8	5.96		
4	14.8	5.96	7.37	7366.31
	24	8.77		
5	24	8.77	8.89	8890.40
	32	9.01		
6	32	9.01	8.89	8890.40
	24	8.77		
7	24	8.77	8.30	8301.50
	38	7.83		
8	38	7.83	5.56	5559.50
	48	3.29		
9	48	3.29	1.65	1645.00
	60	0.00		

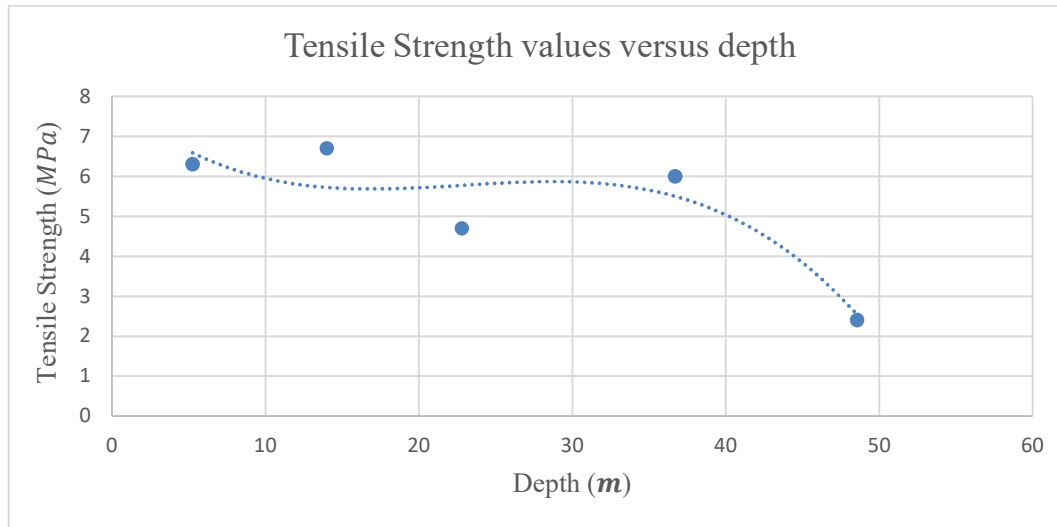


Figure 46 : A Study of the Correlation Between Depth and the Tensile Strength of the SCD 04 borehole.

The cubic regression for tensile strength in borehole SCD-04 yields $R^2 = 0.7982$, indicating a reasonable fit given the natural scatter caused by karstic and tectonic disturbances.

$$\sigma_t \text{ (MPa)} = -0.0002 \times \text{Depth}^3 + 0.0155 \times \text{Depth}^2 - 0.3293 \times \text{Depth} + 7.9189$$

Table 30 : Summary of Depth-Dependent Tensile Strength (σ_t) for Borehole SCD 04.

Stratum	Depth (m)	(σ_t) value (MPa)	(σ_t) Moy value (MPa)	Tensile Strength Moy σ_t (kN/m ²)
1	0	0.00	0.21	208.91
	5.6	0.42		
2	5.6	0.42	0.71	712.82
	6.4	1.01		
3	6.4	1.01	1.08	1079.96
	6.6	1.15		
4	6.6	1.15	1.64	1639.05
	8	2.13		
5	8	2.13	2.42	2422.37
	8.9	2.72		
6	8.9	2.72	4.49	4493.69
	15.5	6.27		
7	15.5	6.27	6.46	6456.32
	16.4	6.64		
8	16.4	6.64	7.89	7890.92
	27.7	9.14		
9	27.7	9.14	8.96	8964.30
	33.7	8.79		
10	33.7	8.79	5.91	5905.51
	52.1	3.02		
11	52.1	3.02	2.50	2500.26
	58.4	1.98		
12	58.4	1.98	1.88	1879.63
	60	1.78		

III.2.3.1.3. Rock Mass Stiffness (E'_{rm})

Since the report did not provide a direct Young's Modulus for the rock mass, the Hoek & Diederichs (2006) empirical equation was applied. This is the standard approach for PLAXIS Hoek-Brown inputs when field seismic data is unavailable.

$$E_{rm} = E_i \left(0.02 + \frac{1 - D/2}{1 + e^{((60+15D-GSI)/11)}} \right) \quad \text{Eq.(56)}$$

E_i (Intact Rock Modulus):

Estimated as:

$$E_i = MR \times \sigma_{ci} \quad \text{Eq.(57)}$$

Table 31 : Suggestions of MR values by Deere (1968) and Palmstrom and Singh (2001).

Rock Material	MR Value Range
Sandstone	200–350
Siltstone	350–400
Conglomerate	300–400
Crystalline Limestone	400–600
Sparitic Limestone	600–800
Micritic Limestone	800–1000
Peridotite	250–300

Table 32 : Proposed MR zones and geological justifications based on the lithological descriptions and depth of the SCD 04 borehole.

Depth (m)	Lithological Description	Suggested MR Range	Justification / Reasoning
0.00 – 5.60	Hard Limestone	600 – 800	Described as "Hard" but with "weathered surface" and clay traces. Matches Sparitic Limestone due to surface degradation.
5.60 – 8.90	Altered Limestone / Karstic Voids	400 – 600	Presence of "critical hazards" like open voids and karstic infilling. Significant loss of stiffness matches Crystalline Limestone range.
8.90 – 15.50	Hard Limestone (Sandy)	600 – 800	"Recovery of consistency" with centimetric calcite fissures. Sandy aspect suggests a slightly less stiff matrix than micritic, aligning with Sparitic.
15.50 – 16.40	Karstified Level	400 – 600	Highly porous and dissolved zone with a drop in RQD to 44%. Matches Crystalline Limestone due to high secondary porosity.
16.40 – 27.70	Hard Limestone	800 – 1000	Stable rock mass with high RQD (up to 88%). High-quality limestone matrix matches Micritic Limestone standards.
27.70 – 33.70	Deformation Zone (Fault)	300 – 400	Brecciated and highly weathered/fractured rock within a shear zone. Integrity drops to Conglomerate/Siltstone levels.
33.70 – 52.10	Stiff Limestone	600 – 800	High-quality rock mass with RQD up to 99%. Described as "Stiff," making it a perfect fit for Sparitic Limestone.
52.10 – 58.40	Deep Weak Zone (Slip)	250 – 400	Highly weathered brecciated limestone with clay cement and very low RQD (23%). Integrity is equivalent to Peridotite/Siltstone.
58.40 – 60.00	Hard Limestone (Base)	800 – 1000	Base of borehole with excellent rock quality and 94% RQD. Matches the stiffest Micritic Limestone classification.

Table 33 : Proposed MR zones and geological justifications based on the lithological descriptions and depth of the SCD 05 borehole.

Depth (m)	Lithological Description	Suggested MR Range	Justification / Reasoning
0.00 – 6.80	Hard Limestone	800 – 1000	Described as "Hard" with "Excellent rock mass quality" and high RQD (96%). Matches Micritic Limestone.
6.80 – 8.30	Dissolution Zone	400 – 600	Presence of "small cavities" and "karstic activity" significantly reduces stiffness. Matches Crystalline Limestone range due to secondary porosity.
8.30 – 14.80	Hard Limestone	800 – 1000	"Solid rock mass" with very high RQD (96-98%) and 100% recovery. Strongest carbonate classification (Micritic).
14.80 – 24.00	Fractured Limestone	600 – 800	"Severely weathered" and "highly rich in calcite." Calcite filling improves it over the dissolution zone but it's weaker than solid rock. Matches Sparitic.
24.00 – 32.00	Hard Limestone	800 – 1000	Returns to "Hard Limestone" description despite a micro-cavity at 26.40m; high RQD (up to 98%) suggests a very stiff matrix.
32.00 – 34.00	Highly Fractured Level	300 – 400	"Geotechnical weak zone" with a "sudden drop in RQD" (53%) and "clayey matrix." Strength drops to Conglomerate/Siltstone equivalent levels.
34.00 – 38.00	Hard Limestone	800 – 1000	"Recovery of rock mass stiffness" and 98% RQD. Re-entry into high-quality Micritic Limestone.
38.00 – 48.00	Stiff Limestone	600 – 800	Described as "Stiff" rather than "Hard." Consistent formation but likely slightly more yielding than the "Hard" Strata. Matches Sparitic.
48.00 – 60.00	Stiff Brecciated Limestone	400 – 600	"Moderately fractured" and "Brecciated" with localized weathering. The mechanical integrity is lower than solid stiff limestone. Matches Crystalline.

III.2.3.1.4. D (Disturbance Factor)

Assumed to be **0.0** for high-quality borehole sampling.

Table 34 : Guidelines for the estimation of Disturbance Factor (D) by Hoek, Carranza-Torres, and Corkum (2002).

Excavation Case Description	Suggested D Value
Excellent quality-controlled blasting.	0
Mechanical excavation (manual or machine-drilled).	0
Small-scale blasting (in small tunnels or mines).	0.5
Very poor blasting (random blasting causing deep fractures).	0.8
Production blasting in large-scale open-pit mining operations.	1

III.2.3.1.5. Hoek-Brown material constant m_i

Based on the lithological characterization of limestone samples collected from the study area, the material constant (m_i) for intact rock was determined according to the standard classification table proposed by Hoek (2007), which classifies rocks based on their origin and formation.

Table 35 : Intact rock m_i constants by group (Hoek, 2007).

Rock ripe	Class	Group	Texture			
			Coarse	Medium	I Fine	I Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones (7 ± 2) Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
			Non- Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)
	Evaporites			Gypsum (8 ± 2)	Anhydrite (12 ± 2)	
	Organic					Chalk (7 ± 2)
METAMORPHIC	Non-Foliated		Marble (9 ± 3)	Homfels (19 ± 4)	Quartzites (20 ± 3)	
			Metasandstone (19 ± 3)			
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites (26 ± 6)		
	Foliated**		Gneiss (28 ± 5)	Schists (12 ± 3)	Phyllites (7 ± 3)	Slates (7 ± 4)
IGNEOUS	Plutonic	Light	Granite (32 ± 3)	Diorite (25 ± 5)		
			Granodiorite (29 ± 3)			
		Dark	Gabbro (27 ± 3)	Dolerite (16 ± 5)		
			Norite (20 ± 5)			
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava	Rhyolite (25 ± 5)		Dacite (25 ± 3)	Obsidian (19 ± 3)
			Andesite (25 ± 5)		Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

Note: Parentheses indicate estimated values.

Basic composition :

- For crystalline or hard limestone (Hard/Sparitic): $m_i = 12 \pm 3$.
- For fine-grained limestone (Micritic): $m_i = 9 \pm 2$.

Table 36 : Recommended m_i values for Borehole SCD 05 based on the Hoek-Brown Criterion (Hoek, 2007).

Stratum	Depth (m)	Lithological Description	Recommended m_i	Technical Justification
1	0.00 – 6.80	Hard Limestone: Pinkish-grey, slightly fractured.	10	Hard limestone with calcite veins (Standard range).
2	6.80 – 8.30	Dissolution Zone: Level affected by cavities.	8	Reduced value due to karstic dissolution fabric.
3	8.30 – 14.80	Hard Limestone: Solid rock mass.	11	High-quality sparitic limestone (Upper range).
4	14.80 – 24.00	Fractured Limestone: Weathered and micritic.	9	Micritic limestone with significant weathering.
5	24.00 – 32.00	Hard Limestone: Reddish, slightly fractured.	10	Compact limestone structure.
6	32.00 – 34.00	Highly Fractured: Fragments in clayey matrix.	7	Corresponds to silty/clayey limestone estimates.
7	34.00 – 48.00	Hard Limestone: Closed oblique fissures.	10	Rebound in rock mass competence and stiffness.
8	48.00 – 60.00	Stiff Brecciated Limestone: Deformation zone.	8	Tectonic breccia associated with diapiric activity.

Table 37 : Recommended m_i values for Borehole SCD 04 based on the Hoek-Brown Criterion (Hoek, 2007).

Stratum	Depth (m)	Lithological Description	Recommended m_i	Technical Justification
1	0.00 – 6.40	Hard Limestone: Fractured surface Stratum.	10	Competent surface limestone.
2	6.40 – 8.90	Karstic Void / Altered Limestone.	7	Highly altered rock surrounding karstic voids.
3	8.90 – 15.50	Hard Limestone: Sandy/Silty texture.	9	Sandy limestone (Lower range for carbonates).
4	16.40 – 27.70	Hard Limestone: Fissured but solid.	11	Dense carbonate matrix with closed fissures.
5	27.70 – 33.70	Deformation Zone: Brecciated structure.	8	Structural disturbance due to proximity to salt diapir.
6	33.70 – 52.10	Stiff Limestone: Sparitic and very hard.	12	Peak rock strength; intact sparitic limestone.
7	52.10 – 60.00	Deep Weak Zone: Highly brecciated/Weak.	7	Severe tectonic crushing in the deep weak zone.

III.2.3.1.6. Unit Weights (γ_{unsat} and γ_{sat})

Table 38 : SCD 04 - Estimated Bulk, Saturated, and Dry Unit Weights for Numerical Modeling.

Stratum	Depth (m)	Bulk Density (g/cm^3)	γ_{unsat} (kN/m^3)	γ_{unsat} Moy (kN/m^3)	γ_{sat} (kN/m^3)	γ_{sat} Moy (kN/m^3)	γ_{dry} (kN/m^3)
1	0.00	2.54	24.92	23.94	25.15	24.20	24.50
	5.60	2.34	22.96		23.25		22.40
2	5.60	2.34	22.96	23.06	23.25	23.33	22.40
	6.40	2.36	23.15		23.40		22.60
3	6.40	2.36	23.15	23.15	23.40	23.40	22.60
	6.60	2.36	23.15		23.40		22.60
4	6.60	2.36	23.15	23.35	23.40	23.63	22.60
	8.00	2.40	23.54		23.85		23.00
5	8.00	2.40	23.54	23.64	23.85	23.98	23.00
	8.90	2.42	23.74		24.10		23.20
6	8.90	2.42	23.74	24.43	24.10	24.75	23.20
	15.50	2.56	25.11		25.40		24.60
7	15.50	2.56	25.11	25.11	25.40	25.40	24.60
	16.40	2.56	25.11		25.40		24.60
8	16.40	2.56	25.11	25.41	25.40	25.70	24.60
	27.70	2.62	25.70		26.00		25.10
9	27.70	2.62	25.70	25.95	26.00	26.25	25.10
	33.70	2.67	26.19		26.50		25.60
10	33.70	2.67	26.19	24.43	26.50	24.80	25.60
	52.10	2.31	22.66		23.10		22.10
11	52.10	2.31	22.66	21.98	23.10	22.43	22.10
	58.40	2.17	21.29		21.75		20.70
12	58.40	2.17	21.29	21.10	21.75	21.58	20.70
	60.00	2.13	20.90		21.40		20.30

Table 39 : SCD 05 - Estimated Bulk, Saturated, and Dry Unit Weights for Numerical Modeling.

Stratum	Depth (m)	Bulk Density (g/cm^3)	γ_{unsat} (kN/m^3)	γ_{unsat} Moy (kN/m^3)	γ_{sat} (kN/m^3)	γ_{sat} Moy (kN/m^3)	γ_{dry} (kN/m^3)
1	0	2.89	28.33	27.33	28.83	27.83	27.9
	6.8	2.68	26.33		26.83		25.8
2	6.8	2.68	26.33	26.02	26.83	26.52	25.8
	8.3	2.62	25.70		26.20		25.2
3	8.3	2.62	25.70	26.02	26.20	26.52	25.2
	14.8	2.68	26.33		26.83		25.8
4	14.8	2.68	26.33	25.95	26.83	26.45	25.8
	24	2.61	25.56		26.06		25.1
5	24	2.61	25.56	25.86	26.06	26.36	25.1
	32	2.67	26.16		26.66		25.7
6	32	2.67	26.16	25.48	26.66	25.98	25.7
	24*	2.53	24.81		25.31		24.3
7	24*	2.53	24.81	25.99	25.31	26.49	24.3
	38	2.77	27.17		27.67		26.7
8	38	2.77	27.17	27.68	27.67	28.18	26.7
	48	2.87	28.18		28.68		27.7
9	48	2.87	28.18	28.32	28.68	28.82	27.7
	60	2.9	28.45		28.95		28

The report provided the Bulk Density (ρ_h) in t/m^3 .

Saturated Unit Weight (γ_{sat})

$$\gamma_{sat} \approx (\gamma_{bulk} \times g) + 0.5 \text{ kN/m}^3 \quad \text{Eq.(58)}$$

Dry Unit Weight (γ_{dry})

$$\gamma_{dry} \approx \frac{\gamma_{sat}}{1 + e \cdot w} \text{ (estimated based on Limestone porosity of 5-10\%)} \quad \text{Eq.(59)}$$

III.2.3.1.7. Poisson's Ratio (ν)

Since no triaxial strain data was provided, ν was assigned based on the rock mass quality described in the technical observations:

- **Stiff/Hard Rock** : $\nu = 0.20 - 0.25$
- **Fractured/Weathered Rock** : $\nu = 0.30 - 0.35$

III.2.3.1.8. Geological Strength Index (GSI)

There are numerous computational approaches used to estimate the Geological Strength Index (GSI) based on the Rock Quality Designation (RQD), in order to ensure accuracy in numerical modeling in the absence of direct measurements. The most important of these methodologies are as follows:

III.2.3.1.8.1. Methodology of Hoek et al. (2013)

This relationship is one of the most recent and widely accepted approaches in the scientific community for directly linking the GSI value to stimulation survey data, as it aims to overcome the subjectivity associated with traditional descriptive tables. The relationship is mathematically expressed as follows:

$$GSI = 1.5 \times JCond_{89} + \frac{RQD}{2} \quad \text{Eq.(60)}$$

Where the variable $JCond_{89}$ represents the condition of rock joints according to Bieniawski (1989); values ranging from 20 to 25 are adopted for rock masses in good condition.

III.2.3.1.8.2. Zhang's (2016) Estimation Model

In cases where there is uncertainty regarding the detailed description of joint surfaces, Zhang (2016) proposed a simplified correlation that relies primarily on RQD for rock formations with typical tectonic characteristics, given by the equation:

$$GSI = 0.5 \times RQD + 15 \quad \text{Eq.(61)}$$

It should be noted that this equation provides a preliminary estimate consistent with rock masses with moderate to good interjoint conditions.

III.2.3.1.8.3. Traditional Approach of Hoek & Brown (1997)

Based on the fundamental principles of rock mass classification, a correlation has been established between GSI and the RMR (Rock Mass Rating) classification system. Given that RMR is structurally based on the RQD value as a key factor, the following relationship can be derived:

$$GSI = RMR_{89} - 5 \quad \text{Eq.(62)}$$

Table 40 : Comparison of Methodologies for Estimating GSI Based on RQD and Rock Quality.

RQD Range	Recommended Method	Source Reference	Justification & Applicability
RQD > 70%	Hoek et al. (2013)	Hoek, Carter, & Diederichs	Best for high-quality rock masses (like your Hard Limestone). It yields precise results when joint conditions are known.
30% < RQD < 70%	Hoek & Brown (1997)	Hoek & Brown	Suitable for "Fair" rock quality where RMR is a better representative of the overall mass behavior than RQD alone.
Any (when data is limited)	Zhang (2016)	Zhang	Used for rapid estimation or preliminary design when detailed joint surface data (J_{Cond}) is missing.
RQD < 30%	Direct Geological Observation	Marinos & Hoek (2000)	Equations become less reliable for very poor/crushed rock; qualitative GSI charts are preferred for "Poor" to "Very Poor" rock.

Table 41 : Calculated General Erosion Index (GSI) values and methodologies applied based on the lithological description and depth of the SCD 04 borehole.

(m) Depth	Description Lithological	RQD (%)	Applied Methodology	GSI Value
5.60 – 0.00	Limestone (Weathered) Hard	84.50	(2013) .Hoek et al	75.3
6.40 – 5.60	Limestone (Karstic) Altered	61.00	(2016) Zhang	45.5
6.60 – 6.40	(Vide) Karstic Void	--	--	0
8.00 – 6.60	Limestone Altered/Fractured	61.00	(2016) Zhang	45.5
8.90 – 8.00	(Vide) Karstic Void	--	--	0
15.50 – 8.90	Limestone (Sandy) Hard	65.00	(2016) Zhang	47.5
16.40 – 15.50	Level Karstified	44.00	(2016) Zhang	37
27.70 – 16.40	Limestone (Fissured) Hard	74.00	(2013) .Hoek et al	70
33.70 – 27.70	Zone (Brecciated) Deformation	66.50	(2016) Zhang	48.3
52.10 – 33.70	Limestone Stiff	82.00	(2013) .Hoek et al	74
58.40 – 52.10	Zone (Weak) Deformation	27.00	(2016) Zhang	28.5
60.00 – 58.40	Limestone Hard	94.00	(2013) .Hoek et al	80

Table 42 : Calculated General Erosion Index (GSI) values and methodologies applied based on the lithological description and depth of the SCD 05 borehole.

Depth (m)	Lithological Description	RQD (%)	Methodology Applied	Calculated GSI Value
0.00 – 6.80	Hard Limestone	96	Hoek et al. (2013)	81
6.80 – 8.30	Dissolution Zone	95	Hoek et al. (2013)	80.5
8.30 – 14.80	Hard Limestone	97	Hoek et al. (2013)	81.5
14.80 – 24.00	Fractured Limestone	92.5	Hoek et al. (2013)	79.2
24.00 – 32.00	Hard Limestone	85.5	Hoek et al. (2013)	75.7
32.00 – 34.00	Highly Fractured Level	53	Zhang (2016)	41.5
34.00 – 38.00	Hard Limestone	98	Hoek et al. (2013)	82
38.00 – 48.00	Stiff Limestone	95.5	Hoek et al. (2013)	80.7
48.00 – 60.00	Stiff Brecciated Limestone	87.5	Hoek et al. (2013)	76.7

Note: The value $J_{Cond_{89}} = 22$ from the methodology of Hoek et al. (2013) was adopted as the average for good rock condition.

III.3. Geotechnical Classification

III.3.1. Soil classification

This study establishes a methodology for the geotechnical evaluation of soil strata encountered in the study area, focusing on Miocene-Pliocene red clays and Triassic diapiric formations. The methodology is based on internationally recognised classification systems (USCS, AASHTO) and standard geotechnical literature (Bowles, 1996; BS 5930).

III.3.1.1. Plasticity chart (casagrande)

The plasticity chart is used to classify fine-grained soils ($\geq 50\%$ passing the No. 200 sieve) based on their Liquid Limit (LL) and Plasticity Index (PI).

The A-line is defined by: $PI = 0.73 \times (LL - 20)$

Table 43 : Classification of Fine-Grained Soils (USCS).

Region	USCS Group	Description
Above A-line, $LL < 50$	CL	Low plasticity clay
Above A-line, $LL \geq 50$	CH	High plasticity clay
Below A-line, $LL < 50$	ML	Low plasticity silt
Below A-line, $LL \geq 50$	MH	High plasticity silt
PI between 4 and 7 below A-line	CL-ML	Silty clay

Application to Atterberg Limits Data (SCD-03)

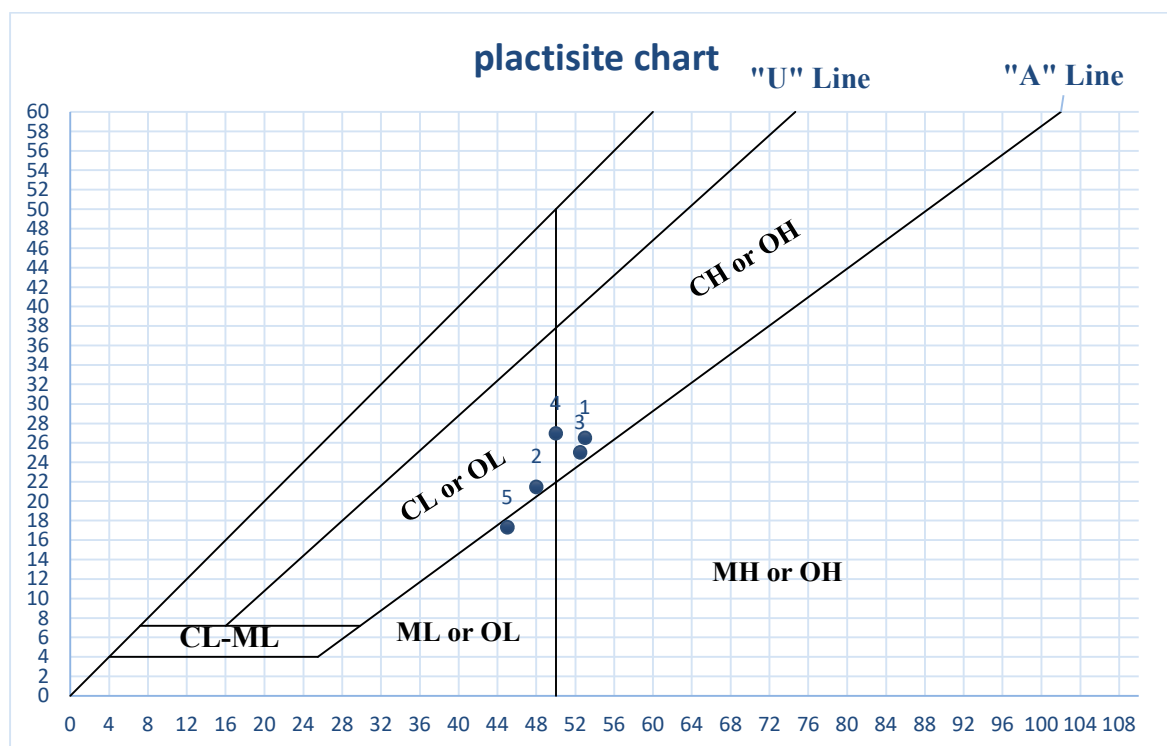


Figure 47 : Plasticity Chart for Miocene-Pliocene red clays (Borehole SCD-03).

Table 44 : Atterberg Limits Test Results and Soil Classification (USCS).

Depth (m)	Points	LL (%)	PL (%)	PI (%)	PI vs. A-line	USCS Group
8.40 – 9.00	1	53	26.49	26.51	PI > 24.09	CH
11.35 – 11.80	2	48	26.53	21.47	PI > 20.44	CL
27.00 – 27.50	3	52.5	27.48	25.02	PI > 23.73	CH
31.00 – 31.50	4	50	23.04	26.96	PI > 21.90	CH
48.00 – 48.60	5	45	27.67	17.33	PI < 18.25	ML / CL-ML

The upper part of the profile (down to approximately 32 m) consists mainly of high-plasticity clays (CH) with occasional low-plasticity clay (CL). At greater depth (approximately 48 m), the soil transitions toward low-plasticity silt or silty clay (ML / CL-ML).

III.3.1.2. Physical and mechanical parameters per stratum

The following eight strata were defined based on depth intervals and consistent physical and mechanical properties derived from laboratory data and regression analysis.

Table 45 : Geotechnical Properties and Classification Parameters of Soil Strata.

Stratum	Depth (m)	γ_{unsat} (kN/m ³)	γ_{sat} (kN/m ³)	c' (kPa)	ϕ' (°)	C_c	C_s	e_0	USCS
1	0.0 – 1.3	23.23	24.71	222	17.5	0.265	0.058	0.61	CH
2	1.3 – 3.2	22.78	23.99	225	17.8	0.22	0.045	0.57	CL
3	3.2 – 31.9	22.04	22.75	258	19.9	0.184	0.033	0.5	CH
4	31.9 – 34.2	21.62	22.04	291	22.1	0.18	0.031	0.46	CH
5	34.2 – 35.9	21.73	22.19	296	22.3	0.192	0.034	0.46	CL
6	35.9 – 38.2	21.82	22.36	300	22.6	0.201	0.035	0.47	CL
7	38.2 – 42.3	21.92	22.45	307	23.1	0.204	0.035	0.46	CL
8	42.3 – 60.0	21.77	23.45	330	24.6	0.201	0.034	0.43	ML / CL-ML

Notes :

- c' and ϕ' are effective shear strength parameters.
- C_c represents the compression index obtained from oedometer tests—
- e_0 represents the initial void ratio.

III.3.1.3. Qualitative assessment of soil quality

III.3.1.3.1. Criteria Considered

The following parameters are considered collectively, without any numerical scoring system:

- USCS classification.
- Plasticity Index (PI).
- Effective cohesion (c').
- Compression index (C_c).
- Unit weight and depth conditions.

III.3.1.3.2. Quality Classes

Table 46 : Classification of Soil Quality Based on Engineering Properties.

Class	Soil Quality	General Description
I	Very Good	High strength, low compressibility, negligible swelling
II	Good	Good strength with moderate compressibility
III	Fair	Moderate strength with notable swelling or settlement
IV	Poor	Weak or highly compressible soil
V	Very Poor	Extremely weak or unstable soil

III.3.1.3.3. Assigned Classes and Engineering Recommendations

Table 47 : Summary of Soil Strata Quality Classification and Engineering Interpretation.

Stratum	Quality Class	Main Interpretation
1	Fair	Expansive surface clay with relatively high compressibility
2	Good	Moderate plasticity and acceptable strength
3	Good	Thick competent clay Stratum with good bearing capacity
4	Good	Strong overconsolidated clay
5	Good	Dense and competent clay
6	Good to Very Good	Good bearing conditions with lower swelling risk
7	Very Good	Excellent bearing stratum
8	Very Good	Best deep foundation horizon

III.3.1.4. Discussion of specific formations

III.3.1.4.1. Miocene-Pliocene Red Clays

- Predominantly high-plasticity clays (CH) in shallow to intermediate depths.
- Very high apparent cohesion due to overconsolidation effects.
- Moderate to high swelling potential in upper strata.
- Presence of shrinkage fissures during dry seasons.
- Medium compressibility in shallow horizons.

III.3.1.4.1.1. Recommended Additional Testing

- Swelling pressure tests.
- Oedometer inundation tests.
- Additional triaxial CU testing.

III.3.1.4.2. Triassic Diapiric Formations

III.3.1.4.2.1. Chemical Exploration of the Diapiric Formation

Chemical analysis of soil and groundwater is an essential tool for characterising Triassic diapiric formations. These formations are dominated by evaporite minerals (halite, gypsum, anhydrite) interbedded with claystones, marls, and tectonic breccias.

The chemical composition is used to determine:

- The presence of soluble minerals such as gypsum, anhydrite, and halite.
- The risk of sulphate attack on concrete foundations.
- The possibility of gypsum dissolution, which may create cavities and cause settlement.
- The aggressiveness of groundwater due to chlorides, sulphates, pH, and CO₂ levels.

The following table presents the typical geochemical composition of evaporite domes:

Table 48 : Representative Geochemical Composition of Evaporite Diapirs.

Constituent Category	Parameter / Element	Unit	Typical Range	Dominant Mineral Phases
Major Oxides	SiO_2	%	0.5 – 5.2	Quartz, silicates
	Al_2O_3	%	0.1 – 2.8	Clay minerals (illite)
	Fe_2O_3	%	0.05 – 1.5	Hematite, goethite
Evaporite Ions	Na^+	%	25.0 – 38.5	Halite (NaCl)
	Cl^-	%	45.0 – 60.2	Halite
	SO_4^{2-}	%	1.5 – 15.0	Anhydrite, gypsum
	Ca^{2+}	%	0.5 – 12.0	Calcite, anhydrite
	Mg^{2+}	%	0.1 – 3.5	Dolomite, carnallite
	Sr	ppm	100 – 1500	Celestine
Trace Elements	Br	ppm	20 – 300	Substitution in halite
	Total Dissolved Solids (TDS)	mg/L	50,000 – 400,000 (typical brine)	Brine concentration
Physical-Chemical	Insoluble residue	%	2.0 – 15.0	Silicates, carbonates

Chemical analyses were performed on samples from borehole SCD-02 located within the diapiric formation. The results are summarized in the following table.

Table 49 : Chemical Analysis Results – Borehole SCD-02.

Borehole (Sondage)	Depth (m)	Gypsum Content (%)	SO ₄ (%)	SO ₄ (mg/kg)	Chlorides (%)	CaCO ₃ (%)	CO ₂ (%)
SCD-02	7.00 - 6.45	Traces	Traces	Traces	0.57	5.75	2.51
	18.35 - 18.00	12.35	6.89	68,919.04	0.71	5.83	2.54
	21.35 - 21.00	8.91	4.97	49,773.58	0.53	5.37	2.34

III.3.1.4.2.2. Interpretation in the Context of Diapiric Formations

Comparison of the measured chemical data with the typical ranges presented in Table 1 leads to the following interpretations:

Sulphate Content (SO_4^{2-}): Typical range in diapiric evaporitic formations: 1.5 – 15.0%.

Measured values:

- 6.89% at 18 m depth.
- 4.97% at 21 m depth.

These values fall within the typical range of sulphate-rich diapiric formations and indicate a significant presence of gypsum and anhydrite.

According to BRE Special Digest 1, sulphate concentrations exceeding 10,000 mg/kg are classified as DS-5 (very severe sulphate aggressiveness). The measured concentrations in this study (approximately 49,000–69,000 mg/kg) therefore indicate a highly aggressive environment for concrete structures.

Gypsum Content: Measured gypsum contents:

- 12.35% at 18 m.
- 8.91% at 21 m.

These values confirm the presence of considerable gypsum within the formation.

Since gypsum is highly soluble in water, its dissolution may generate karstic voids and internal cavities.

In the presence of groundwater circulation, this process can lead to sudden settlement, ground instability, or localized collapse.

Chloride Content (Cl^-): Measured chloride values: 0.53 – 0.71%.

Pure halite formations generally contain chloride concentrations between 45% and 60%. The relatively low chloride values measured here indicate that the material is not composed of halite rock, but rather consists mainly of gypsiferous clay or marl containing minor halite cement or dissolved chlorides in pore water.

From a durability perspective, chloride contents below 0.4% are generally preferred for reinforced concrete. The measured values are moderate and may require:

- Increased concrete cover.
- Use of protective coatings or sulphate-resistant materials.
- Additional corrosion protection measures for reinforcement.

Calcium Carbonate Content ($CaCO_3$): Measured values: 5.37–5.83%.

These relatively low carbonate contents are typical of evaporite-dominated formations, indicating that calcite is not the dominant mineral phase.

Carbon Dioxide Content (CO_2): Measured values: 2.34–2.54%.

The CO_2 content is considered moderate. Although it may contribute to carbonation processes over time, it is not considered chemically aggressive by itself under normal geotechnical conditions.

Table 50 : Evaluation of Sulphate Attacks and Required Cementitious Mitigation Measures according to BRE Specifications.

Depth	SO ₄ (mg/kg)	ACEC Class (BRE)	Required Cement Type
6.45–7.00	Traces (<1000)	DS-1 (low)	Ordinary Portland cement (OPC)
18.00–18.35	68,919	DS-5 (very high)	Sulphate-resisting Portland cement (SRPC) + protective coating
21.00–21.35	49,774	DS-5 (very high)	Same

III.3.1.4.3. Application to Atterberg Limits Data (SCD-02)

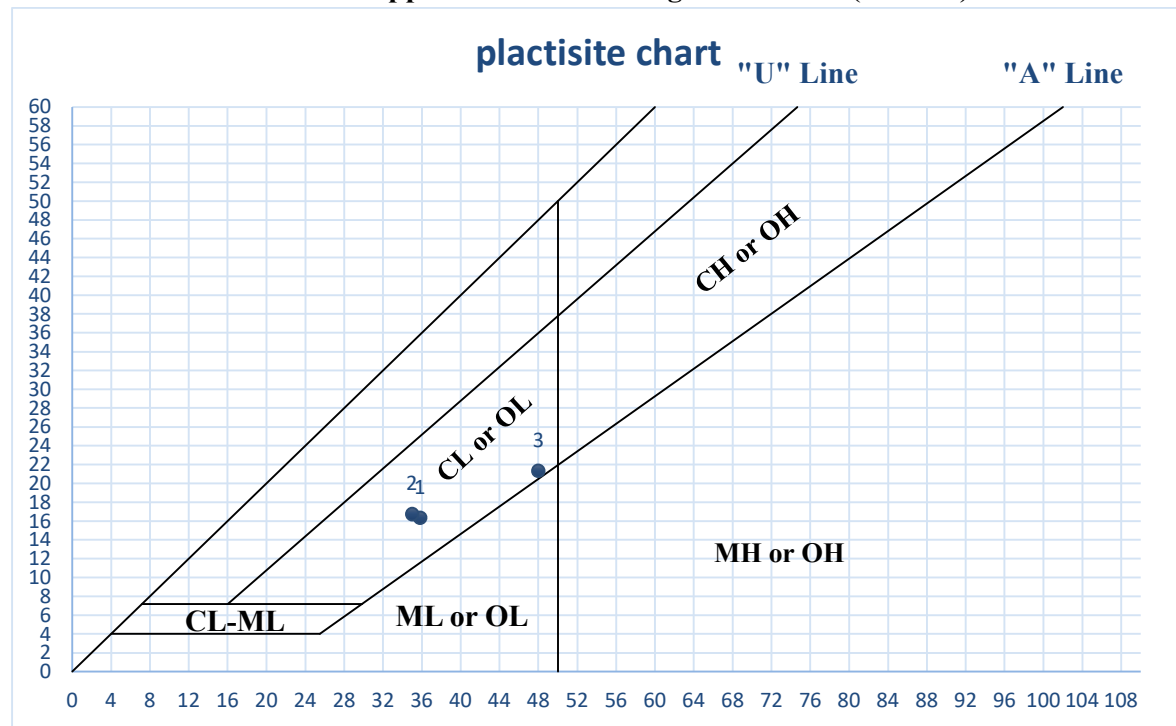


Figure 48 : Plasticity Chart for Triassic diapiric formations (Borehole SCD-02).

Table 51 : Plasticity Characteristics and USCS Classification of Soil Samples.

Depth (m)	LL (%)	Points	PL (%)	PI (%)	A-line at given LL	PI vs. A-line	USCS Group
6.45 – 7.00	35.8	1	19.45	16.35	11.53	>	CL
18.00 – 18.35	35	2	18.26	16.74	10.95	>	CL
21.00 – 21.35	48	3	26.66	21.34	20.44	>	CL

III.3.1.5. Conclusions and general recommendations

Table 52 : Classification of Triassic Diapiric Soils.

Stratum	1 (Firm gypsum-rich clay)	2 (Compact marly clay)	3 (Variegated argillaceous marl)	4 (Highly weathered clayey passage)	5 (Deep argillaceous marl)
Depth (m)	0.0 – 14.4	14.4 – 21.0	21.0 – 42.9	42.9 – 44.1	44.1 – 50.0
γ_{unsat} (kN/m³)	18	19	19.5	17	20
γ_{sat} (kN/m³)	20	21	21.5	19	22
c' (kPa)	35	70	120	18	160
ϕ' (°)	22	28	31	17	34
v_{ur}	0.15	0.25	0.22	0.33	0.2
E50_{ref} (MPa)	(Soft Soil)	45	90	6	120
Eur_{ref} (MPa)	—	135	270	—	360
G0_{ref} (MPa)	—	220	450	—	600
OCR	1.3	1.5	1.8	—	2
K0_{nc}	0.63	0.53	0.48	0.71	0.44
USCS	CL	CL	CH	(CL/ML)	(CL/ML)
Geotech Class	Fair	Good	Good to Very Good	Poor	Very Good
Chemical Class (SO₄)	DS-1 (low)	DS-5 (very high)	DS-5	unknown	likely low
Integrated Class	III	III (chemical downgrade)	II/III	IV	I
Key Measures	Remove topsoil, OPC ok	SRC, protective coating, gypsum dissolution assessment	SRC, swelling test, pile to deeper Stratum if possible	Penetrate, grout, SRC	Ideal pile bearing; SRC optional

III.3.1.5.1. Conclusions for Miocene-Pliocene Red Clays

- The surface Stratum (0–1.3 m) is Fair and requires removal or replacement.
- Strata 2 to 5 are Good and suitable for shallow foundations with moisture control.
- Strata 7 and 8 are Very Good and ideal for deep foundations and pile bearing.

III.3.1.5.2. Conclusions for Triassic Diapiric Formations

- Gypsum-rich Strata (Strata 2 and 3) have very high sulphate content (DS-5) requiring sulphate-resisting cement.
- The weathered passage (Stratum 4) is Poor and must be avoided or penetrated by piles.
- The deep marl (Stratum 5) is Very Good and recommended as pile bearing stratum.

III.3.1.6. Conclusions and general recommendations

- The surface Stratum (0–1.3 m) is classified as Fair and should preferably be removed or improved before construction.
- Intermediate Strata generally exhibit good engineering behaviour suitable for shallow foundations with moisture control measures.
- Deep Strata below approximately 38 m provide Very Good bearing conditions suitable for pile foundations.

- d) Additional investigations are recommended, including:
- SPT values.
 - Undrained shear strength measurements.
 - Swelling pressure determination.
 - Groundwater monitoring.
- e) Triassic diapiric formations require detailed separate evaluation due to heterogeneity and dissolution risks.

III.3.2. Rock Mass Classification

The rock mass quality of the Compact Turonian limestones encountered in boreholes SCD 04 and SCD 05 has been evaluated using the Rock Mass Rating (RMR) system proposed by Bieniawski (1989). This system is one of the most widely used empirical classification methods in rock engineering, providing a quantitative basis for estimating stand-up time, support requirements, and deformation behaviour of excavated rock masses.

The RMR value is calculated as the sum of five basic parameters, each assigned a rating according to predefined ranges:

$$RMR = (A_1 + A_2 + A_3 + A_4 + A_5) + B \quad \text{Eq.(63)}$$

- a) **Uniaxial compressive strength (UCS)** of intact rock material.
- b) **Rock Quality Designation (RQD)** as a measure of core integrity.
- c) **Spacing of discontinuities** (joints, fractures, bedding planes).
- d) **Discontinuity condition** is broken into five sub-parameters: persistence, aperture, roughness, infilling, and weathering. These five sub-ratings sum to the total condition rating.
- e) **Groundwater conditions** (inflow, joint water pressure, or general state).

An additional orientation adjustment is then applied, which depends on the strike and dip of discontinuities relative to the Tinoukla Trench excavation. For this study, the orientation adjustment assumes an alignment specific to the Tinoukla Trench geometry; the applied correction is indicated for each stratum.

The tables below summarise the complete rating system, including all parameter ranges and corresponding point values, as published by Bieniawski (1989). These tables serve as the reference key for the detailed stratum-by-stratum evaluation of the Tinoukla Trench presented in the following sections.

Table 53 : CLASSIFICATION PARAMETERS (BIENIAWSKI, 1989).

1	Strength of Rock Matrix	Point Load Strength Index (MPa)	>10	10-4	4-2	2-1	Uniaxial Compression		
		Uniaxial Compressive Strength (MPa)	>250	250-100	100-50	50-25	25-5	5-1	<1
		Rating	15	12	7	4	2	1	0
2	Rock Quality Designation (RQD) (%)		100-90%	90-75%	75-50%	50-25%	<25%		
		Rating	20	17	13	8	3		
3	Discontinuity Spacing		> 2m	2-0.6m.	0.6-0.2m.	0.2-0.06m.	< 0.06m.		
		Rating	20	15	10	8	5		
4	Discontinuity Condition	Persistence (Length)	< 1m	1-3m	3-10m	10-20m	>20m		
		Rating	6	4	2	1	0		
		Aperture (Opening)	Closed	<0.1mm	0.1-1mm	1-5mm	>5mm		
		Rating	6	4	2	1	0		
		Roughness	Very Rough	Rough	Slightly Rough	Wavy/ Undulating	Smooth		
		Rating	6	4	2	1	0		
		Infilling (Gouge)	None	Hard <5mm	Hard >5mm	Soft <5mm	Soft >5mm		
		Rating	6	4	2	1	0		
		Alteration/ Weathering	Unaltered	Slightly Altered	Moderately Altered	Highly Altered	Decomposed		
		Rating	6	4	2	1	0		
5	Groundwater	Inflow per 10m of tunnel	None	<10 L/min	10-25 L/min	25-125 L/min	>125 L/min		
		Joint Pressure Ratio	0	0-0.1	0.1-0.2	0.2-0.5	>0.5		
		General Condition	Dry	Slightly Damp	Damp	Dripping	Flowing Water		
		Rating	15	10	7	4	0		

Table 54 : DISCONTINUITY ORIENTATION IN THE TUNNEL (BIENIAWSKI, 1989).

Direction Perpendicular to the tunnel axis				Direction Parallel to the tunnel axis		Any Direction, Dip 0°- 20°
Excavation with the Dip		Excavation against the dip				
Dip 45°-90°		Dip 20°-45°				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very Favorable	Favorable	Medium	Unfavorable	Very unfavorable	Medium	Unfavorable

Table 55 : DISCONTINUITY ORIENTATION CORRECTION (BIENIAWSKI, 1989).

Direction and Dip		Very Favorable	Favorable	Medium	Unfavorable	Very Unfavorable
Rating	Tunnel	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Here is a standard RMR rock mass classification table based on Bieniawski (1989), relating RMR value to class, quality, cohesion, friction angle, and stand-up time.

Table 56 : ROCK MASS CLASSIFICATION ACCORDING TO RMR (BIENIAWSKI, 1989).

RMR	100-81	80-61	60-41	40-21	<20
Class	I	II	III	IV	V
Quality	Very Good	Good	Medium	Poor	Very Poor
Cohesion (kPa)	>400	300–400	200–300	100–200	<100
Angle of Friction	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°
Stand-up time for unsupported span length	10 years, 15m.	6 months, 8m.	1 week, 5m.	10 hours, 2.5m.	30 minutes, 1m.

The ratings for each stratum in boreholes SCD-04 and SCD-05 were assigned using:

- Laboratory UCS results (converted to MPa).
- RQD values from borehole logs.
- Discontinuity spacing estimated from RQD and core descriptions.
- Discontinuity condition interpreted from qualitative core observations (persistence, aperture, roughness, infilling, weathering).
- Dry groundwater condition (maximum rating of 15 points) due to absence of recorded inflow during drilling.

Orientation adjustments were applied based on structural observations and assumed tunnel direction, with penalties ranging from 0 to –12 points as indicated in the orientation sub-table.

The resulting RMR values are then classified into five classes (I to V), ranging from Very Good to Very Poor, as shown in Table 4 (Bieniawski, 1989). For each class, typical cohesion and friction angle ranges, as well as indicative stand-up times, are provided for engineering use.

Methodology: For each stratum, the five joint condition parameters (persistence, aperture, roughness, infilling, weathering) were estimated from the original borehole descriptions.

Table 57 : Detailed Parameter-Based RMR89 Assessment for Borehole SCD-04 (Strata 1 to 6).

Strata	S1	S2	S3	S4	S5	S6
Depth (m)	0.00–5.60	5.60–6.40	6.40–6.60	6.60–8.00	8.00–8.90	8.90–15.50
UCS (MPa)	44.4	50.6	52.7	53.2	57.1	59.8
UCS Pts	4	7	7	7	7	7
RQD (%)	84.5	61	0	61	0	65
RQD Pts	17	13	3	13	3	13
Spacing	200-600 mm	60-200 mm	disintegrated	60-200 mm	disintegrated	200-600 mm
Spacing Pts	15	10	0	10	0	15
Joint Condition Details	P:1-3m (4); A:0.1-1mm (2); R: Rough (4); I: Hard<5mm (4); W: Mod (2)	Karstic void - P:>20m (0); A:>5mm (0); R: Smooth (0); I: Soft>5mm (0); W: Decomp (0)	P:1-3m (4); A:1-5mm (1); R: Rough (4); I: Hard<5mm (4); W: Mod (2)	P:1-3m (4); A:1-5mm (1); R: Rough (4); I: Hard<5mm (4); W: Mod (2)	P:<1m (6); A: Closed (6); R: Rough (4); I: None (6); W: Slight (4)	P:1-3m (4); A:0.1-1mm (2); R: Rough (4); I: Hard<5mm (4); W: Mod (2)
Joint Sum	26	16	0	15	0	26
Water	15	15	15	15	15	15
Base Sum	77	61	25	60	25	76
Orient. Adj.	-2 (Fav.)	-8 (Unfav.)	0	-8 (Unfav.)	0	-5 (Fair)
Final RMR	75	53	25	52	25	71
Class	Good	Fair	Poor	Fair	Poor	Good

Table 58 : Detailed Parameter-Based RMR89 Assessment for Borehole SCD-04 (Strata 7 to 12).

Strata	S7	S8	S9	S10	S11	S12
Depth (m)	15.50–16.40	16.40–27.70	27.70–33.70	33.70–52.10	52.10–58.40	58.40–60.00
UCS (MPa)	83.3	87.2	153.9	65	22.3	14.8
UCS Pts	7	7	12	7	2	1
RQD (%)	44	74	66.5	82	27	94
RQD Pts	8	13	13	17	8	20
Spacing	60-200 mm	200-600 mm	60-200 mm	200-600 mm	20-60 mm	>600 mm
Spacing Pts	10	15	10	15	8	20
Joint Condition Details	P:3-10m (2); A:0.1-1mm (2); R: Rough (4); I: Hard<5mm (4); W: Mod (2)	P:1-3m (4); A: Closed (6); R: Rough (4); I: None (6); W: Slight (4)	Brecciated - P:10-20m (1); A:1-5mm (1); R: Rough (4); I: Soft<5mm (2); W: High Alt (1)	P:<1m (6); A: Closed (6); R: Rough (4); I: None (6); W: Slight (4)	Very open, clay infill - P:>20m (0); A:>5mm (0); R: Smooth (0); I: Soft>5mm (0); W: Decomp (0)	P:<1m (6); A: Closed (6); R: Rough (4); I: None (6); W: Slight (4)
Joint Sum	14	24	9	26	0	26
Water	15	15	15	15	15	15
Base Sum	54	74	59	80	33	82
Orient. Adj.	-8 (Unfav.)	-5 (Fair)	-12 (Very Unfav.)	-5 (Fair)	-12 (Very Unfav.)	-5 (Fair)
Final RMR	46	69	47	75	21	77
Class	Fair	Good	Fair	Good	Poor	Good

Note: The abbreviations in the table represent the rock mass discontinuity parameters as follows:

P = Persistence/A = Aperture/R = Roughness/ I = Infilling/W = Weathering.

Summary for SCD-04:

The rock mass quality ranges from Class II (Good) to Class IV (Poor), with karstic voids (Strata 3 and 5) requiring special treatment. Engineering judgement overrides calculated RMR for these voids.

Improved Notes for SCD-04 Detailed Evaluation:

Note: all joint condition parameters for SCD-04 were inferred from qualitative core descriptions. For karstic voids (Strata 3 and 5), engineering judgement overrides the calculated RMR.

Strata 3 and 5 (karstic voids): The calculated RMR is approximately 25 (Class IV – Poor), whereas the original classification assigned Class V (Very Poor) with RMR 18. The difference arises because the UCS rating still contributes 7 points even in voids where no intact rock mass exists. In reality, karstic voids have negligible rock mass quality; therefore, an RMR value below 20 is geotechnically justified. It is recommended to override the calculated rating and assign Class V (Very Poor) manually for these strata.

Stratum 9: Very high UCS (153.9 MPa) gives 12 points (maximum). However, intense brecciation and unfavourable joint orientation (adjustment –12) reduce the final RMR to 47 (Class III – Fair). This highlights that high intact strength alone does not guarantee good rock mass quality.

Stratum 11: Very weak rock (UCS = 22.3 MPa → 2 points; RQD = 27% → 8 points) with clay infill in discontinuities (joint condition sum = 0/30). Final RMR = 21 (Class IV – Poor). This stratum requires careful support design.

Stratum 12: Low UCS (14.8 MPa → 1 point) but excellent RQD (94% → 20 points) and wide spacing (>600 mm → 20 points). Final RMR = 77 (Class II – Good). This case illustrates a known limitation of the RMR89 system: it tends to overrate massive but weak rock masses. Engineering judgement should be exercised when using this rating for support design.

Table 59 : Detailed Parameter-Based RMR89 Assessment for Borehole SCD-05 (Strata 1 to 5).

Strata	S1	S2	S3	S4	S5
Depth (m)	0.00–6.80	6.80–8.30	8.30–14.80	14.80–24.00	24.00–32.00
UCS (MPa)	67.1	76.6	82.8	90.3	91.7
UCS Pts	7	7	7	7	7
RQD (%)	96	95	97	92.5	85.5
RQD Pts	20	20	20	20	17
Spacing	>600 mm	200-600 mm	>600 mm	60-200 mm	200-600 mm
Spacing Pts	20	15	20	10	15
Persistence	<1m (6)	1-3m (4)	<1m (6)	3-10m (2)	<1m (6)
Aperture	Closed (6)	0.1-1mm (2)	Closed (6)	1-5mm (1)	Closed (6)
Roughness	Rough (4)	Slightly rough (2)	Rough (4)	Rough (4)	Rough (4)
Infilling	None (6)	Hard <5mm (4)	None (6)	None (6)	None (6)
Weathering	Unaltered (6)	Slightly altered (4)	Unaltered (6)	Moderately altered (2)	Unaltered (6)
Joint Sum (max30)	28	16	28	15	28
Water	15	15	15	15	15
Base Sum	90	73	90	67	82
Orient. Adj.	-2 (Fav.)	-5 (Fair)	-5 (Fair)	-8 (Unfav.)	-5 (Fair)
Final RMR	88	68	85	59	77
Class	Very Good	Good	Very Good	Fair	Good

Table 60 : Detailed Parameter-Based RMR89 Assessment for Borehole SCD-05 (Strata 6 to 9).

Strata	S6	S7	S8	S9
Depth (m)	32.00–34.00	34.00–38.00	38.00–48.00	48.00–60.00
UCS (MPa)	91.7	88.3	73.3	43.7
UCS Pts	7	7	7	4
RQD (%)	53	98	95.5	87.5
RQD Pts	13	20	20	17
Spacing	20-60 mm	>600 mm	200-600 mm	60-200 mm
Spacing Pts	8	20	15	10
Persistence	>20m (0)	<1m (6)	1-3m (4)	3-10m (2)
Aperture	>5mm (0)	Closed (6)	Closed (6)	1-5mm (1)
Roughness	Smooth (0)	Very rough (6)	Rough (4)	Rough (4)
Infilling	Soft >5mm (0)	None (6)	None (6)	Hard <5mm (4)
Weathering	Highly altered (1)	Unaltered (6)	Slightly altered (4)	Moderately altered (2)
Joint Sum (max30)	1	30	24	13
Water	15	15	15	15
Base Sum	44	92	81	59
Orient. Adj.	-12 (Very Unfav.)	-5 (Fair)	-5 (Fair)	-8 (Unfav.)
Final RMR	32	87	76	51
Class	Poor	Very Good	Good	Fair

Summary for SCD-05: Most strata are Class I or II (Very Good to Good), except Stratum 6 (Poor, RMR=32) due to clay-filled fractures, and Stratum 9 (Fair, RMR=51) due to brecciation.

Improved Notes for SCD-05 Detailed Evaluation:

Note: All joint condition parameters (persistence, aperture, roughness, infilling, weathering) were inferred from qualitative descriptions in the borehole logs, as no direct measurements were available.

Stratum 2: Description "slightly open" → aperture inferred as 0.1–1 mm (2 pts); "calcite filling" → infilling interpreted as hard <5 mm (4 pts). Roughness not explicitly mentioned → estimated as "slightly rough" (2 pts).

Stratum 4: Description "moderate weathering, 1–5 mm aperture" → aperture set to 1–5 mm (1 pt); weathering rated as moderately altered (2 pts). Persistence not given → estimated as 3–10 m (2 pts) based on fractured appearance.

Stratum 6: Highly fractured with clayey matrix → all five joint condition parameters assigned the lowest ratings: persistence >20 m (0 pts), aperture >5 mm (0 pts), roughness smooth (0 pts), infilling soft >5 mm (0 pts), weathering highly altered (1 pt). Total joint condition sum = 1/30.

Stratum 7: Description "very rough" → roughness rated 6 pts (maximum).

Stratum 9: Description "broken structure, calcite filling, moderate weathering" → persistence estimated 3–10 m (2 pts), aperture 1–5 mm (1 pt), roughness rough (4 pts), infilling hard <5 mm (4 pts), weathering moderate (2 pts).

III.4. Conclusion

This site investigation for the Tnoukla project successfully characterized three distinct geological formations—Mio-Pliocene clays, Turonian limestones, and Triassic diapiric materials—using integrated seismic geophysics, core drilling, and laboratory testing.

The investigation revealed critical hazards: karstic voids within weathered limestone requiring classification as Class V (Very Poor) despite high intact strength, severe sulphate concentrations (49,000–69,000 mg/kg; DS-5 aggressivity) demanding sulphate-resisting cement, and a weathered Triassic passage necessitating foundation bypass.

Laboratory data were transformed into continuous profiles using regression models ($R^2 \geq 0.70$ –0.99), justifying the Hardening Soil model for clays, the Hoek-Brown criterion for limestones (UCS: 19.9–140.5 MPa; GSI: 20–75), and a benchmarking approach (Soft Soil, HSsmall, Mohr-Coulomb) for Triassic formations.

RMR classifications ranged from Class I (Very Good) to Class V (Very Poor), with engineering judgement applied to override ratings where karstic voids were present. Plasticity indices up to 26.9% indicate moderate to high swelling potential in the clays.

Recommendations include shallow foundations with moisture control for clays, deep pile bearing below 38 m, DS-5 concrete protection for Triassic gypsum-rich layers, and grouting of karstic zones. The integrated methodology provides a transferable template for site characterization in evaporite-karstic terrains, and the parameter sets are now available for numerical slope stability analyses and detailed engineering design.

Chapter 04: Geotechnical Numerical Modeling of the Tnoukla Railway Trench

IV.1. Introduction

The stability assessment of excavated slopes in heterogeneous geological media represents a fundamental challenge in geotechnical engineering, particularly when the stratigraphic sequence comprises materials with contrasting mechanical behaviors and chemical susceptibilities. The Tnoukla Railway Trench project, situated within a complex geological setting characterized by Mio-Pliocene clays, Turonian limestones, and Trias argillaceous clays (the latter exhibiting elevated gypsum and sulphate concentrations), necessitates a rigorous analytical framework capable of capturing both the static and transient hydraulic responses of the excavated system. To address this requirement, the present chapter employs a dual numerical methodology that integrates two complementary approaches: Limit Equilibrium Method (LEM) analyses conducted using GEO5 software, and Finite Element Method (FEM) simulations performed via PLAXIS 2D and 3D. This integrated framework is deliberately designed to leverage the respective advantages of each platform—GEO5 providing validated factor-of-safety calculations for circular and polygonal failure surfaces across multiple cross-sections, while PLAXIS enables advanced simulation of soil-structure interaction, nonlinear constitutive behavior, transient seepage, and coupled hydro-mechanical responses within the heterogeneous stratigraphic sequence. The selection of appropriate constitutive models reflects the distinct mechanical signatures of each geological formation: the Hardening Soil (HS) model for the Mio-Pliocene clays to capture stress-dependent stiffness, the converted Hoek-Brown model for the Turonian limestones to represent massive rock mass strength, and a depth-dependent combination of Soft Soil, HS-small, and Mohr-Coulomb models for the Trias argillaceous clays to account for stiffness degradation and gypsum-rich claystone behavior. The numerical models incorporate the full suite of support elements specified in the detailed design, including sulfate-resistant shotcrete with welded wire mesh (simulated as PLATE elements) and grouted anchor bars (simulated as Anchor elements), as well as the multi-layered railway track structure subjected to an equivalent dynamic loading of 80 kPa representing heavy phosphate train passages. A critical hydrological consideration underlying the modeling framework is the absence of groundwater encountered across all eight boreholes during the site investigation—attributed to prolonged regional drought and the low recharge potential of the geological strata—which justifies the exclusion of initial phreatic surfaces from the baseline models. Furthermore, the project's location within a seismically dormant zone permits the deliberate omission of seismic loading from all stability calculations. The two-dimensional modeling encompasses five representative cross-sections selected to capture the transition between successive geological formations along the trench alignment, while the three-dimensional modeling extends this analysis through volumetric terrain reconstruction—generated within Autodesk 3ds Max and exported to PLAXIS 3D via OBJ format—to capture spatial variability, excavation sequencing, and true three-dimensional displacement fields. Particular attention is devoted to the transient hydraulic response of the system, with rainfall infiltration applied as a boundary condition to evaluate the development of excess pore water pressures, the potential for flow failure, and the efficacy of proposed waterproofing and drainage countermeasures including geomembranes, gutters, and weep holes. Through this comprehensive multi-platform, multi-dimensional numerical framework, the present chapter seeks to evaluate the global and local stability of the Tnoukla Railway Trench under

static operational loading, to assess the contribution of the proposed reinforcement system to deformation control and long-term integrity, to identify the hydrological conditions under which flow failure may initiate, and to establish the hydraulic protection measures necessary to ensure durable performance of the excavated slope system.

IV.2. Geotechnical Properties and Constitutive Models Used in Numerical Modeling (PLAXIS 2D/3D)

(These values are derived from the investigation and analysis presented in the previous chapter - Site Investigation).

IV.2.1. Material properties

In this study, the physical and mechanical properties of the three main identified formations were input according to the values obtained from the previous chapter, while adopting the appropriate constitutive models previously determined for each geological formation, as follows:

- Clay formations (**Mio-Pliocene Clays**): Hardening Soil (HS) model.
- Limestone formations (**Turonian Limestones**): Converted Hoek-Brown model.
- Triassic formations (**Trias Argillaceous Clays**): Combined models including Soft Soil, HS small, and Mohr-Coulomb depending on depth.

The following tables summarize these properties:

IV.2.1.1. Clay formations (Hardening Soil Model)

Table 61 : Parameters and Geotechnical Properties for Clay formations.

Stratum	1	2	3	4	5	6	7	8
γ_{unsat} (kN/m^3)	23.23	22.78	22.04	21.62	21.73	21.82	21.92	21.77
γ_{sat} (kN/m^3)	24.71	23.99	22.75	22.04	22.19	22.36	22.45	23.45
c'_{moy} (kPa)	221.89	225.32	258.13	291.37	295.65	299.94	306.8	330.18
ϕ' moy (°)	17.54	17.76	19.89	22.05	22.32	22.6	23.05	24.56
ν	0.35	0.42	0.3	0.32	0.3	0.28	0.33	0.3
C_c	0.265	0.22	0.184	0.18	0.192	0.201	0.204	0.201
C_s	0.058	0.045	0.033	0.031	0.034	0.035	0.035	0.034
e	0.61	0.57	0.5	0.46	0.46	0.47	0.46	0.43
P_{ref}	16.64	81.61	676.73	1252.45	1301.35	1346.42	1410.79	1539.03
E_{oed}	233.01	1342.24	12724.39	23365.97	22743.6	22697.61	23272.77	25158.54
E_{ur}	942.62	3677.5	78843.38	153215.5	155504.6	163270.62	150248.04	189931.85
E_{50}	233.01	1342.24	12724.39	23365.97	22743.6	22697.61	23272.77	25158.54

IV.2.1.2. Limestone formations (Hoek-Brown Model)

Table 62 : Summary of Calculated Hoek-Brown Parameters and Geotechnical Properties for Borehole SCD 05.

Stratum	1	2	3	4	5	6	7	8	9
σ_{ci} (kN/m ³)	67107.5	76610.4	82817.2	90336.3	91653	91653	88249.5	73303.5	43730
σ_t (kN/m ³)	647.55	1810.79	4142.55	7366.31	8890.4	8890.4	8301.5	5559.5	1645
Proposed MR Range	800 – 1000	400 – 600	800 – 1000	600 – 800	800 – 1000	300 – 400	800 – 1000	600 – 800	400 – 600
MR Average	900	500	900	700	900	350	900	700	500
GSI (Estimated)	81	80.5	81.5	79.2	75.7	41.5	82	80.7	76.7
m_i	10	8	11	9	10	7	10	10	8
E_i	60396.79	38305.21	74535.50	63235.44	82487.70	32078.55	79424.55	51312.45	21865.00
E'_{rm} (MN/m ²)	53808.52	33927.70	66779.44	55102.01	68174.11	5673.25	71545.40	45556.21	18372.52
γ_{unsat} (kN/m ³)	27.33	26.02	26.02	25.95	25.86	25.48	25.99	27.68	28.32
γ_{sat} (kN/m ³)	27.83	26.52	26.52	26.45	26.36	25.98	26.49	28.18	28.82
ν_{nu}	0.2	0.25	0.2	0.3	0.22	0.35	0.2	0.25	0.3

Table 63 : Summary of Calculated Hoek-Brown Parameters and Geotechnical Properties for Borehole SCD 04 part 1.

Stratum	1	2	3	4	5	6
σ_{ci} (kN/m ²)	44416.6	50631.19	52710.14	53243.08	57127.63	59773.23
σ_t (kN/m ²)	208.91	712.82	1079.96	1639.05	2422.37	4493.69
Proposed MR Range	600 – 800	400 – 600	400 – 600	400 – 600	400 – 600	600 – 800
MR average	700	500	500	500	500	700
GSI (Estimated)	75.3	45.5	0	45.5	0	47.5
m_i	10	10	7	7	7	9
E_i	31091.62	25315.60	26355.07	26621.54	28563.82	41841.26
E'_{rm} (MN/m ²)	25518.05	5850.96	639.34	6152.80	692.92	11003.78
γ_{unsat} (kN/ m ³)	23.94	23.06	23.15	23.35	23.64	24.43
γ_{sat} (kN/m ³)	24.44	23.56	23.65	23.85	24.14	24.93
ν_{nu}	0.25	0.3	0.35	0.3	0.35	0.28

Table 64 : Summary of Calculated Hoek-Brown Parameters and Geotechnical Properties for Borehole SCD 04 part 2.

Stratum	7	8	9	10	11	12
σ_{ci} (kN/m^2)	83307.44	87165.44	153883.8	64946.03	22339.5	14814.89
σ_t (kN/m^2)	6456.32	7890.92	8964.3	5905.51	2500.26	1879.63
Proposed MR Range	400 – 600	800 – 1000	300 – 400	600 – 800	250 – 400	800 – 1000
MR average	500	900	350	700	325	900
GSI (Estimated)	37	70	48.3	74	28.5	80
m_i	8	11	8	12	7	7
E_i	41653.72	78448.90	53859.34	45462.22	7260.34	13333.40
E'_{rm} (MN/m^2)	5414.30	57488.46	14898.31	36424.75	537.12	11738.03
γ_{unsat} (kN/m^3)	25.11	25.41	25.95	24.43	21.98	21.1
γ_{sat} (kN/m^3)	25.61	25.91	26.45	24.93	22.48	21.6
ν_{nu}	0.32	0.22	0.3	0.2	0.35	0.2

IV.2.1.3. Triassic formations (Combined Models)

Table 65 : Parameters and Geotechnical Properties for Triassic formations.

Stratum	1	2	3	4	5
Model	Soft Soil	Hssmall	Hssmall	MC	Hssmall
Depth range (m)	0.0 – 14.4	14.4 – 21.0	21.0 – 42.9	42.9 – 44.1	44.1 – 50.0
γ_{unsat} (kN/m^3)	18	19	19.5	17	20
γ_{sat} (kN/m^3)	20	21	21.5	19	22
c' (kPa)	35	70	120	18	160
ϕ' ($^\circ$)	22	28	31	17	34
E_{50}^{ref} (MPa)	—	45	90	6 (E^{ref})	120
E_{ur}^{ref} (MPa)	—	135	270	—	360
G_0^{ref} (MPa)	—	220	450	—	600
$\gamma_{0.7}$ ($\times 10^{-4}$)	—	1.5	1	—	0.8
M	—	0.65	0.6	—	0.55
OCR	1.3	1.5	1.8	—	2
K_0^{nc}	0.63	0.53	0.48	0.71	0.44

In this study, a set of support elements was used in the modeling. They were distributed according to the design plan and defined according to the following standard engineering and mechanical properties for each element.

IV.2.2.Reinforcement elements used in the modeling

IV.2.2.1. Shotcrete with Welded Wire Mesh

In this study, sulfate-resistant shotcrete (CRS) was used as a surface support element to protect the slopes in clayey areas (Mio-Pliocene and Trias) against surface erosion, in accordance with the project's detailed design (thickness 10 cm, two layers of welded wire mesh).

This shotcrete was modeled in PLAXIS 2D/3D using the PLATE element, which simulates the structural behavior of thin elements under bending, tension, and compression loads. The equivalent mechanical properties of the shotcrete with wire mesh were input according to the values listed in the table below, derived from design specifications and engineering calculations.

Table 66 : Properties of the PLATE Element for Shotcrete.

Property	Symbol	Unit	Value	Remarks
Axial stiffness (direction 1)	EA ₁	Kn/m	2.500×10^6	Reflects tensile/compressive resistance
Axial stiffness (direction 2)	EA ₂	Kn/m	2.500×10^6	Same value for perpendicular direction
Elastic modulus (direction 1)	E ₁	Kn/m ²	25.00×10^6	25 gpa (reinforced concrete)
Elastic modulus (direction 2)	E ₂	Kn/m ²	25.00×10^6	Equal to E ₁ (isotropic material)
Bending stiffness	EI	Kn·m ² /m	2083	Calculated based on thickness d=0.10 m
Poisson's ratio	N	-	0.25	Typical value for concrete
Equivalent thickness	D	M	0.1	Layer thickness (10 cm)
Unit weight	Γ	Kn/m ³	24	Unit weight of reinforced concrete

IV.2.2.2. Grouted Anchor Bars

These bars were modeled as Anchor elements, which simulate the interaction between the bar and the surrounding soil through a grout-soil interface.

Anchor Properties :

- **Steel grade:** Fe E500 (B500SD) – high adherence (HA).
- **Nominal diameter :** 25 mm.
- **Yield strength (f_y):** 500 mpa.
- **Elastic modulus (E):** 210,000 mpa (2.1×10^8 kn/m²).

Table 67 : Anchor Properties Table.

Property	Symbol	Unit	Value
Cross-sectional area	A	m ²	4.91×10^{-4}
Axial stiffness	EA	kN	≈ 98,200
Elastic modulus	E	kN/m ²	2.1×10^8
Yield strength	f_y	kN/m ²	5.0×10^5
Unit weight	γ	kN/m ³	78.5
Bar diameter	D	m	0.025
Effective contact area	A _{eff}	m ² /m	0.0785

IV.2.3.Railway structure

The railway track structure was introduced as a multi-layer system, in ascending order, consisting of: Sub-Ballast, Subgrade, Technical Layer, and Ballast. These layers were modelled as a continuous medium using the elasto-plastic Mohr-Coulomb constitutive model, a model that is perfectly suited for granular geomaterials.

The following table presents the input properties of these layers:

Table 68 : properties of Railway structure layers.

Parameter	Ballast	Sub-Ballast	Subgrade	Technical Layer
Material Model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Drainage Type	Non-porous	Non-porous	Drained	Drained
Unsaturated Unit Weight γ_{unsat} (KN/m ³)	18	19.5	20	19
Saturated Unit Weight γ_{sat} (KN/m ³)	20	21.5	22	21
Young's Modulus E (MPa)	350	300	250	200
Poisson's Ratio ν	0.4	0.4	0.35	0.33
Cohesion c (KPa)	1	1	10	20
Friction Angle ϕ°	45	38	35	30
Dilatancy Angle ψ°	12	8	5	0
Permeability (Horizontal) k_x (m/s)	1×10^{-3}	5×10^{-5}	1×10^{-7}	1×10^{-6}
Permeability (Vertical) k_y (m/s)	1×10^{-3}	5×10^{-5}	1×10^{-7}	1×10^{-6}
Reference Pressure p_{ref} (KPa)	100	100	100	100
Interface Reduction Factor R_{inter}	0.9	0.85	0.75	0.8

Note: To realistically represent the effect of heavy phosphate train passages in the various models of the study area, an equivalent load of 80 kPa (KN/m²) was applied over a length of 1.9 meters for each track of the double railway line. This load reflects the equivalent dynamic loading resulting from the axle load of loaded wagons (approximately 25 tonnes per axle), taking into account an appropriate dynamic amplification factor for operating speeds (60–80 Km/h).

This load was introduced into the numerical models as follows:

- In 2D modeling, it was represented as a line load along the track.
- In 3D modeling, it was represented as a distributed surface load on the designated area.

IV.3. 2D Numerical Modeling

Modeling was performed through a set of steps:

IV.3.1. Cross-section Selection

Five cross-sections were selected along the trench to represent the project: one per formation, plus one between consecutive pairs:

PROFILE 05:

Numerical modeling using PLAXIS 2D has been conducted to analyze Section 05, which spans the Miocene-Pliocene red clays formation.

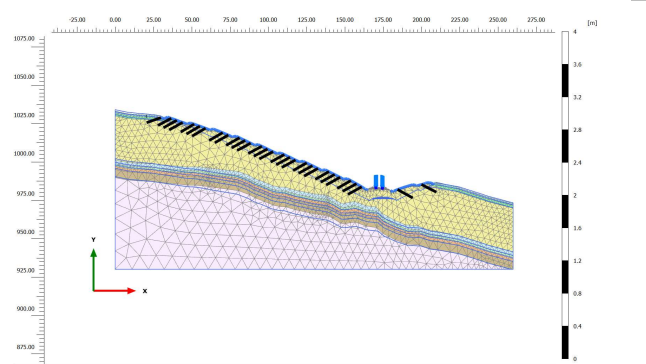


Figure 49 : Numerical modeling of Section 05 using PLAXIS 2D

PROFILE 09:

Numerical modeling using PLAXIS 2D has been conducted to analyze Section 09, spanning the contact zone between the Miocene-Pliocene red clays and the Turonian limestone formations.

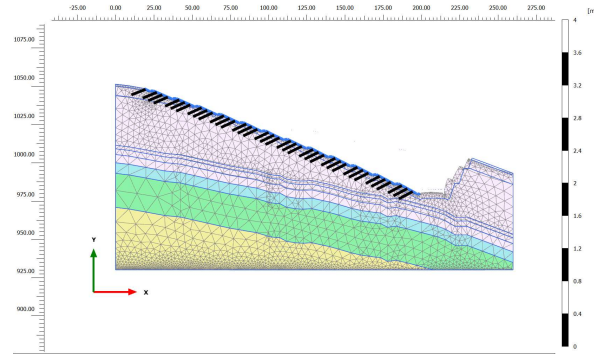


Figure 50 : Numerical modeling of Section 09 using PLAXIS 2D.

PROFILE 15:

Numerical modeling using PLAXIS 2D has been conducted to analyze Section 15, which spans the Turonian limestone formation.

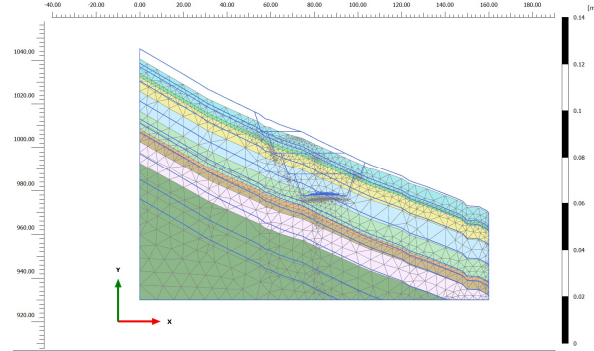


Figure 51 : Numerical modeling of Section 15 using PLAXIS 2D.

PROFILE 20:

Numerical modeling using PLAXIS 2D has been conducted to analyze Section 20, spanning the contact zone between the Turonian limestone and the Triassic formations.

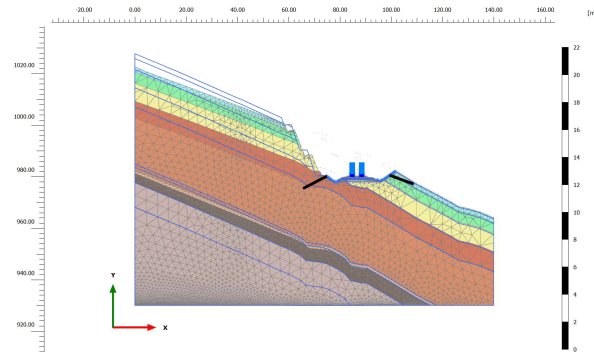


Figure 52 : Numerical modeling of Section 20 using PLAXIS 2D.

PROFILE 22:

Numerical modeling using PLAXIS 2D has been conducted to analyze Section 22, which spans the Triassic formations.

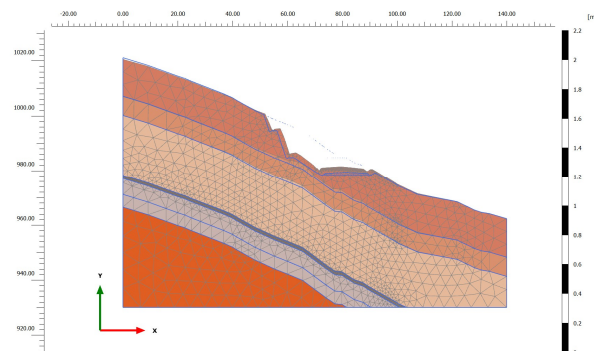


Figure 53 : Numerical modeling of Section 22 using PLAXIS 2D.

IV.3.2. Model Geometry Construction

In this study, the geometric shape of each cross-section (Profiles 05, 09, 20, 22) was designed using autocad software, based on the longitudinal and design sections derived from the geotechnical study report. The boundaries of the geological layers (Miocene clays, Turonian limestones, Triassic clays) were drawn, along with the support elements (shotcrete, anchors) and the track structure layers (ballast, sub-ballast, subgrade, technical layer).

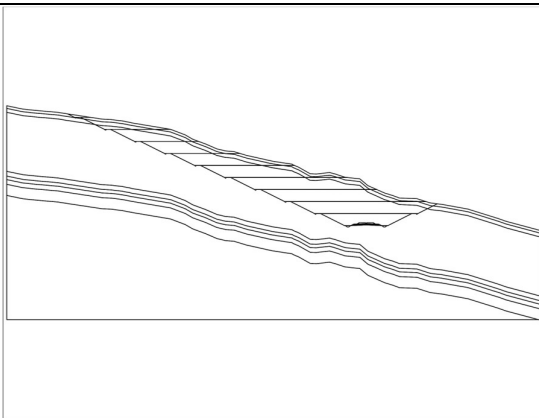


Figure 54 : Cross-Section 05: Geotechnical Model of Soil Strata.

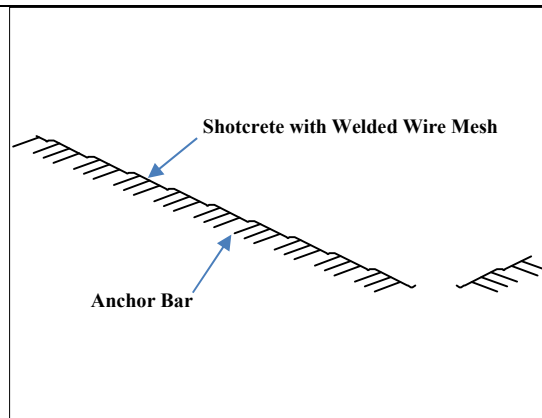


Figure 55 : Support System Diagram for Cross-Section 05.

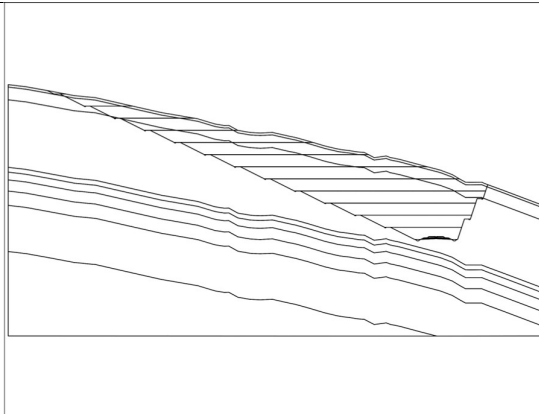


Figure 56 : Cross-Section 09: Geotechnical Model of Soil Strata.

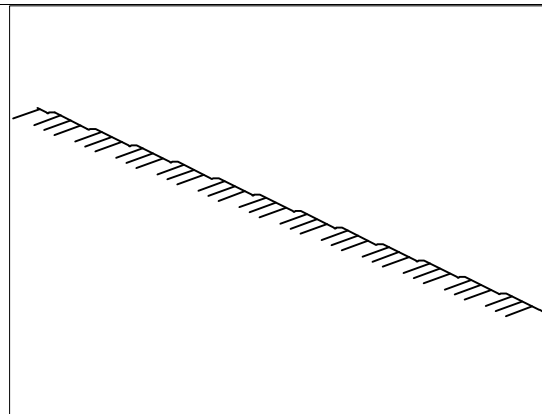


Figure 57 : Support System Diagram for Cross-Section 09.

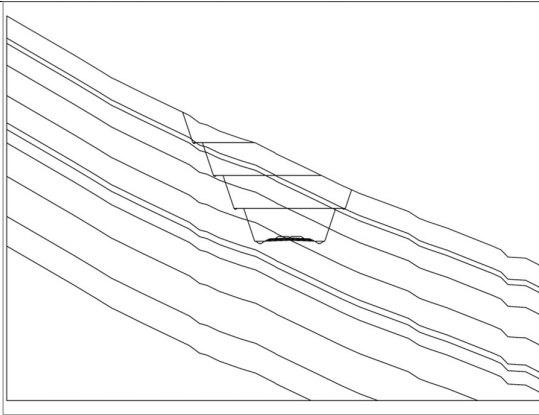
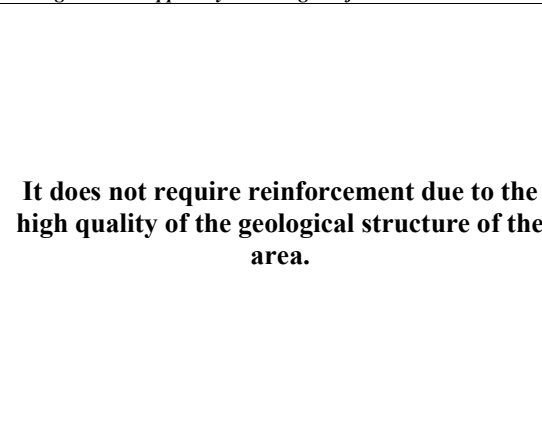


Figure 58 : Cross-Section 15: Geotechnical Model of Soil Strata.



It does not require reinforcement due to the high quality of the geological structure of the area.

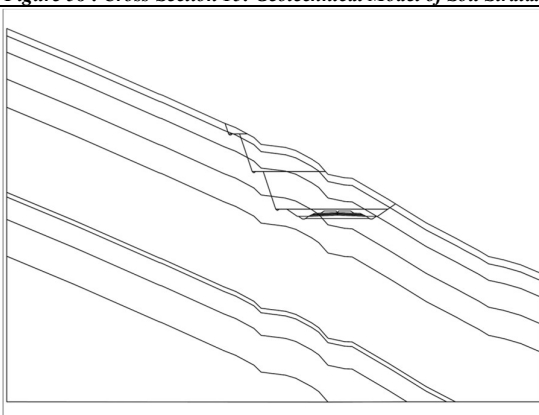


Figure 59 : Cross-Section 20: Geotechnical Model of Soil Strata.

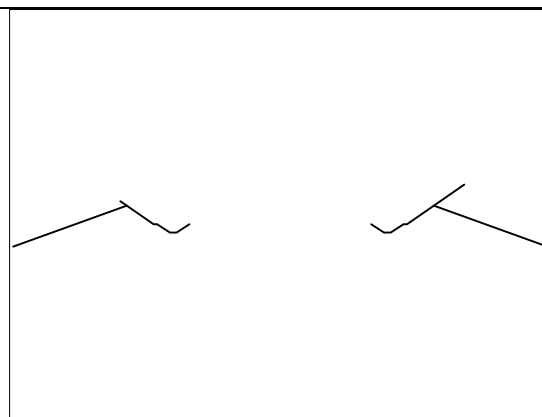


Figure 60 : Support System Diagram for Cross-Section 20.

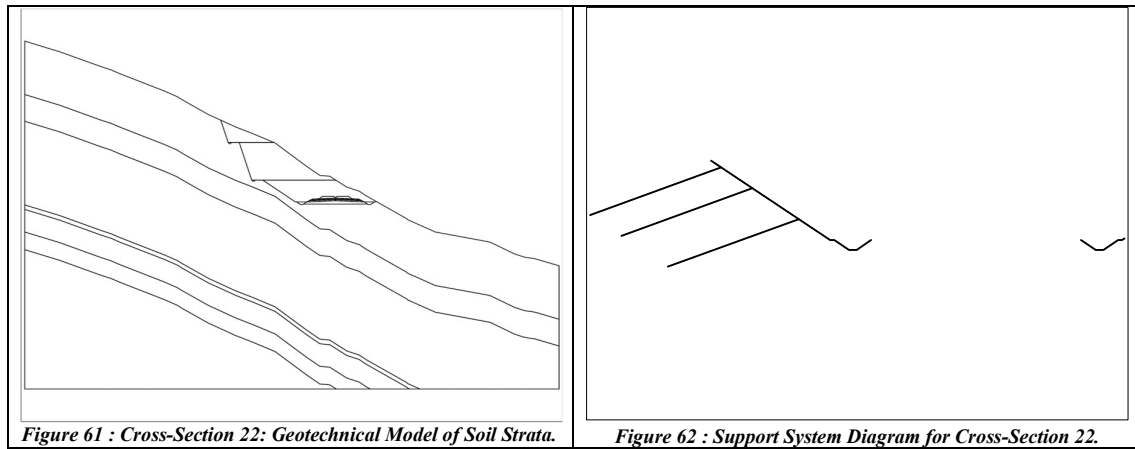


Figure 61 : Cross-Section 22: Geotechnical Model of Soil Strata.

Figure 62 : Support System Diagram for Cross-Section 22.

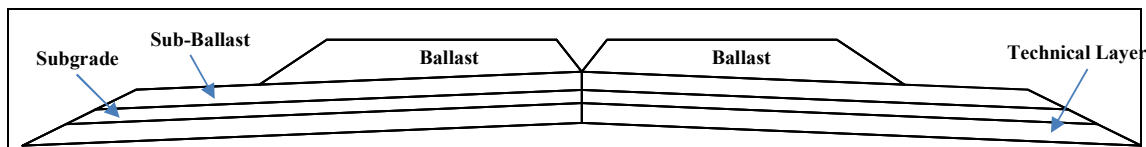


Figure 63 : Typical Cross-Section of Railway Formation Layers.

After completing the drawing, the geometric shape was exported in a file format compatible with PLAXIS 2D software (e.g., DXF file), and then directly imported into the PLAXIS 2D Input environment. There, the materials, boundaries, and loading conditions were redefined for each drawn region.

IV.3.3. Input of Material Properties

The material properties in each model were defined according to its specific lithological composition, based on the location of the cross-section, using the property tables previously presented.

- Triassic formations Stratum 01 - Triassic formations Stratum 02 - Triassic formations Stratum 03 - Triassic formations Stratum 04 - Triassic formations Stratum 05	- reddish clay Stratum 01 - reddish clay Stratum 02 - reddish clay Stratum 03 - reddish clay Stratum 04 - reddish clay Stratum 05 - reddish clay Stratum 06 - reddish clay Stratum 07 - reddish clay Stratum 08	- compact limestone Stratum 01 - compact limestone Stratum 02 - compact limestone Stratum 03 - compact limestone Stratum 04 - compact limestone Stratum 05 - compact limestone Stratum 06 - compact limestone Stratum 07 - compact limestone Stratum 08
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Figure 64 : Material Legend for the 2D Numerical Mode.

Note: The material layers extracted from borehole SCD-04 were not used in the two-dimensional (2D) modeling, because the project layout avoided passing through its zone. Consequently, these layers will not fall within the affected range of the analyzed cross-sections. Therefore, the data from this borehole were only used in the three-dimensional (3D) modeling, which requires a wider spatial coverage.

IV.3.4. Mesh Generation

The numerical analysis begins with mesh generation, a fundamental step that subdivides the geometric model into interconnected triangular and quadrilateral elements joined at nodes. To ensure both accuracy and computational efficiency, a medium to fine mesh density was adopted, with local refinement applied in critical zones such as slope surfaces, contacts between geological layers, and around structural elements (e.g., shotcrete and anchors). This approach guarantees precise stress-strain calculations while maintaining reasonable computation time.

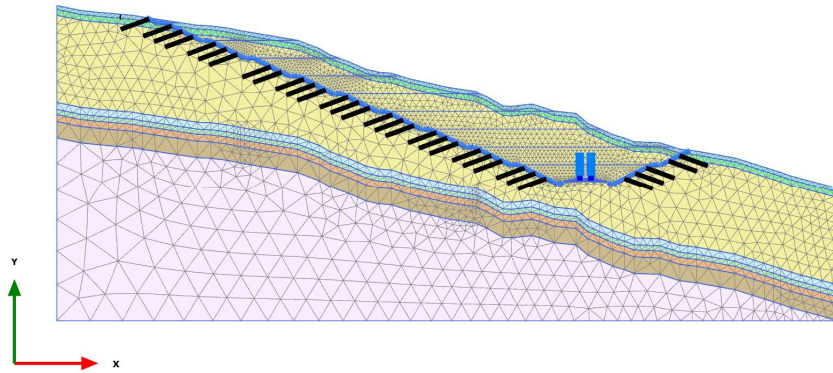


Figure 65 : Finite Element Mesh of the cross-section 05.

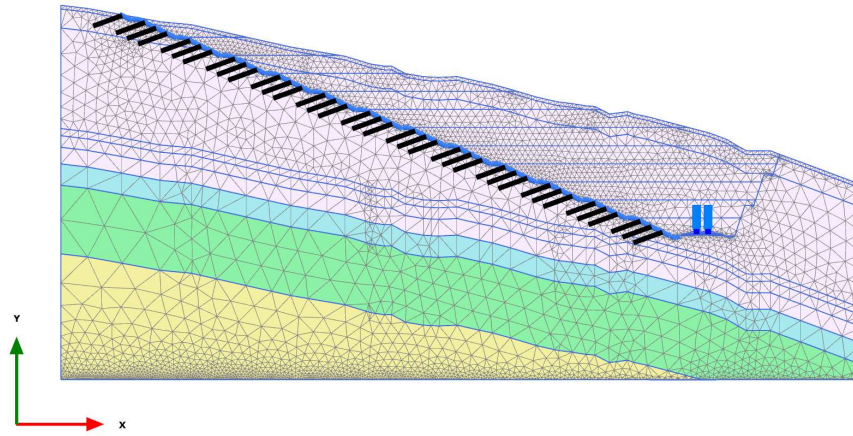


Figure 66 : Finite Element Mesh of the cross-section 09.

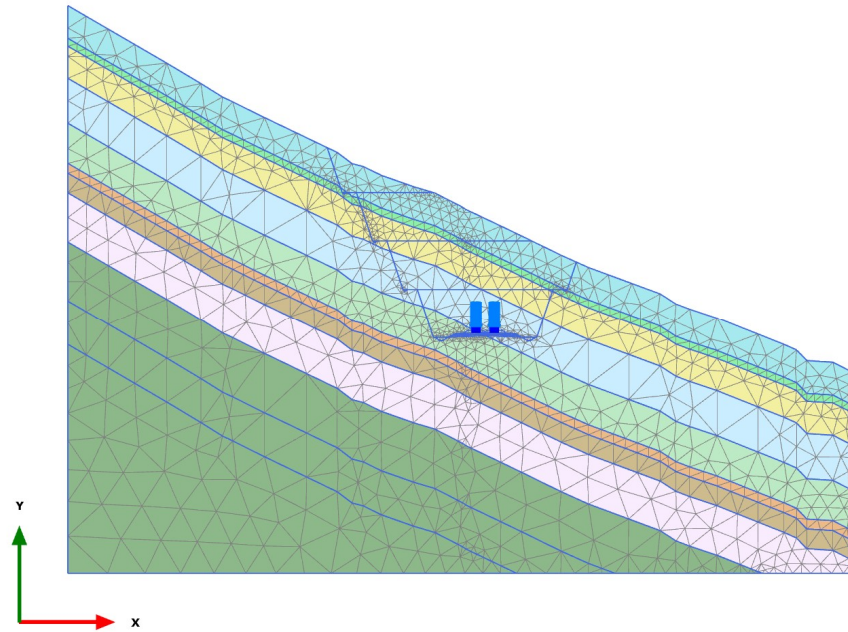


Figure 67 : Finite Element Mesh of the cross-section 15.

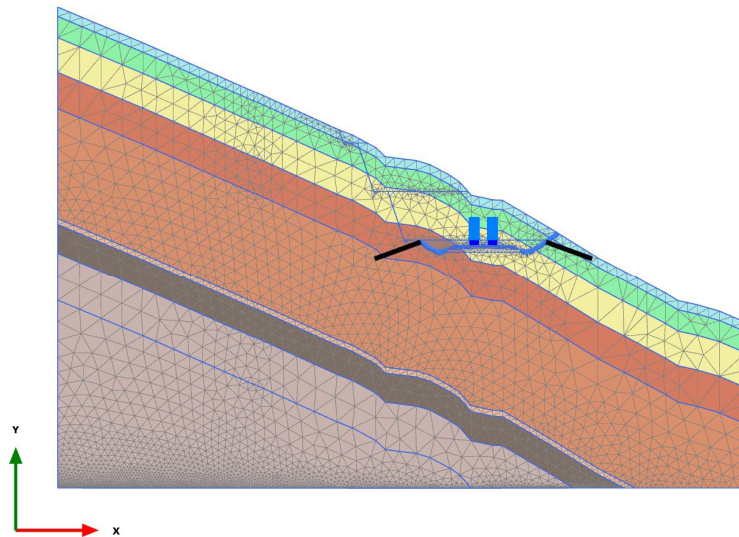


Figure 68 : Finite Element Mesh of the cross-section 20.

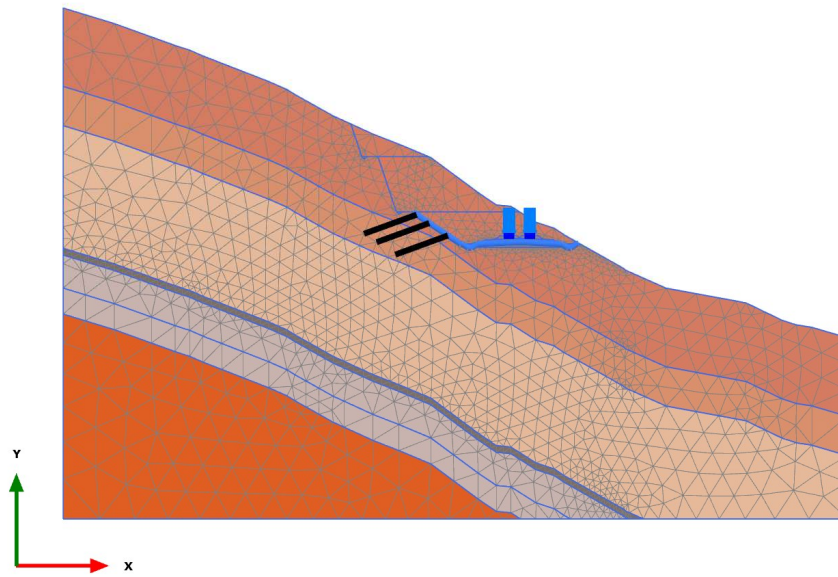


Figure 69 : Finite Element Mesh of the cross-section 22.

IV.3.5. Analysis Results of Selected Cross-Sections

IV.3.5.1. PROFILE 05

IV.3.5.1.1. Numerical Simulation Stages

Tableau 1 : Summary of calculation phases and sequential construction steps for Section 05.

PHASE	DESCRIPTION
INITIAL PHASE	Initial balancing using Gravity Loading.
PHASE 1-10	Excavation processes (sequential removal of soil clusters).
PHASE 11	Installation of support/lining systems.
PHASE 12	Activation of roadbed layers (Ballast/Sub-ballast).
PHASE 13	Application of distributed surface load (80 KN/m^2).
PHASE 14	Safety Analysis (φ/c reduction).

IV.3.5.1.2. Evaluation of Soil Displacement Response

U_x Deformations:

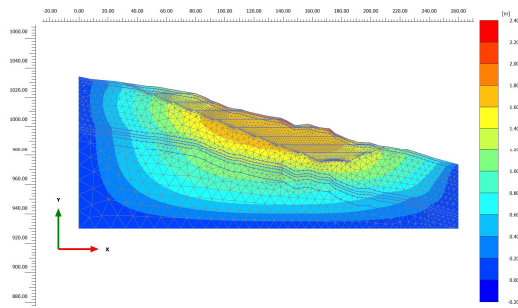


Figure 70 : Horizontal Displacement (U_x) distribution at the Initial Construction Phase.

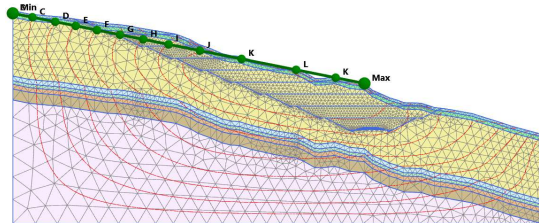


Figure 71 : Cross-section showing U_x displacement contours at the initial construction phase.

Maximum value = 2.231 m (Element 1730 at Node 32111)
Minimum value = 0.000 m (Element 14 at Node 114)

U_y Deformations:

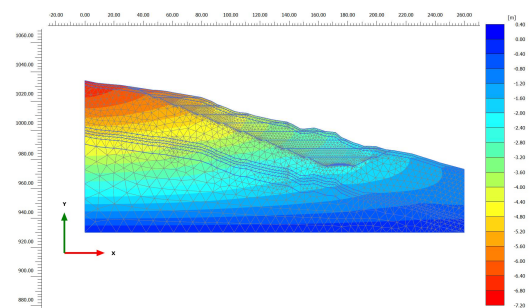


Figure 72 : Vertical Displacement (U_y) distribution at the Initial Construction Phase.

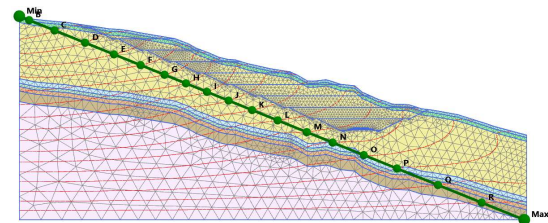


Figure 73 : Cross-section showing U_y displacement contours at the initial construction phase.

Maximum value = 0.000 m (Element 5890 at Node 48512)
Minimum value = -7.000 m (Element 20 at Node 52)

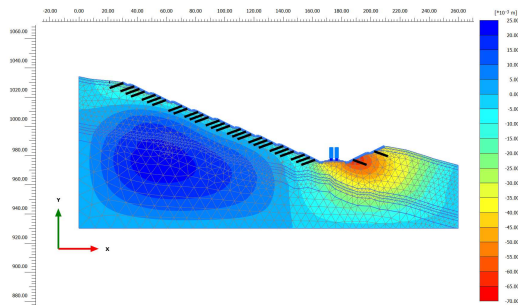


Figure 74 : Vertical Displacement (U_x) distribution at Phase 14.

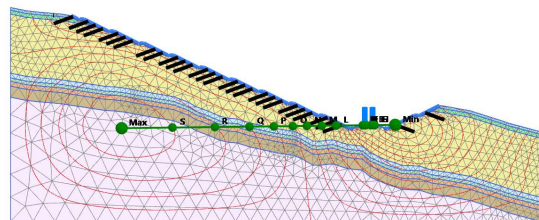


Figure 75 : Cross-section showing U_x displacement contours at Phase 14.

Maximum value = 0.02253 m (Element 6204 at Node 22759)
Minimum value = -0.06702 m (Element 3053 at Node 45994)

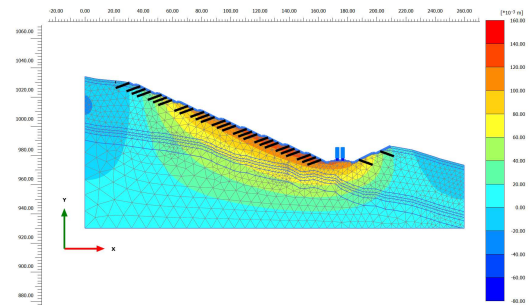


Figure 76 : Vertical Displacement (U_y) distribution at Phase 14.

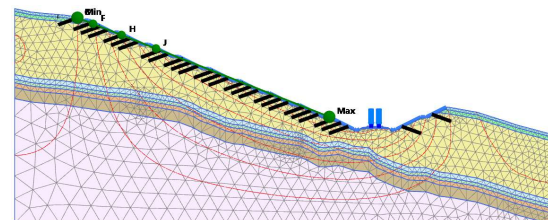


Figure 77 : Cross-section showing U_y displacement contours at Phase 14.

Maximum value = 0.1495 m (Element 3320 at Node 34728)
Minimum value = -0.06214 m (Element 44 at Node 622)

Table 69 : Summary of Nodal Displacement Extremes (U_x And U_y) per Calculation Phase for Section 05.

Phase N :	Maximum value		Minimum value	
	u_x	u_y	u_x	u_y
Initial Phase	2.231	0.000	0.000	-7.000
1	3.373×10^{-3}	0.01610	-6.915×10^{-3}	-1.785×10^{-3}
2	3.523×10^{-3}	0.04404	-0.01507	-2.125×10^{-3}
3	6.138×10^{-3}	0.08169	-0.02376	-2.709×10^{-3}
4	7.711×10^{-3}	0.1076	-0.02906	-3.009×10^{-3}
5	0.01030	0.1263	-0.03250	-2.288×10^{-3}
6	0.01341	0.1351	-0.03440	-5.743×10^{-3}
7	0.01626	0.1435	-0.03841	-9.175×10^{-3}
8	0.01866	0.1525	-0.04134	-0.01208
9	0.02078	0.1579	-0.05002	-0.01466
10	0.02223	0.1604	-0.06667	-0.01652
11	0.02255	0.1512	-0.06654	-0.03500
12	0.02253	0.1501	-0.06679	-0.03620
13	0.02253	0.1495	-0.06698	-0.03638
14	0.02253	0.1495	-0.06702	-0.06214

IV.3.5.1.3. Definition of Monitoring Nodes

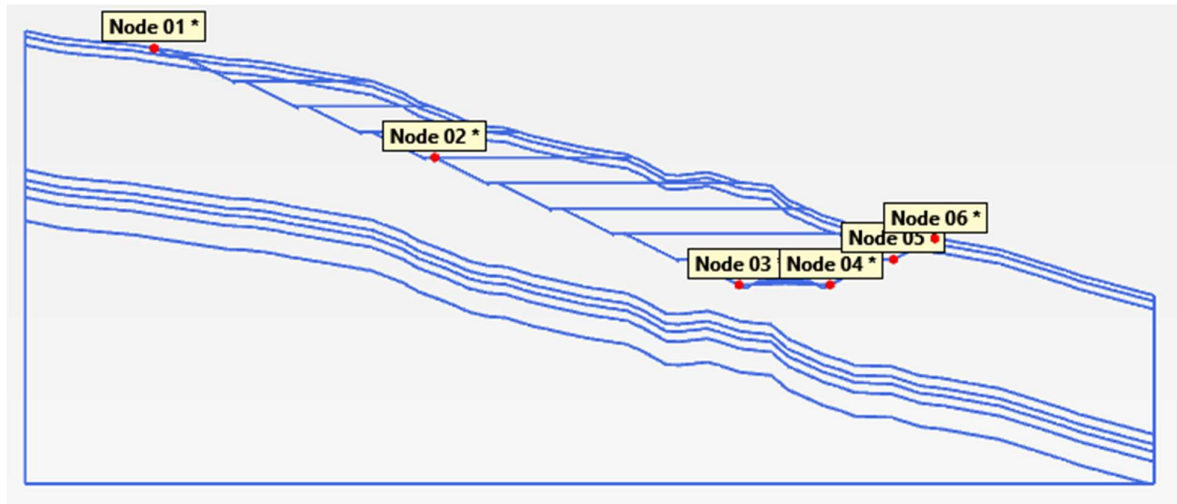


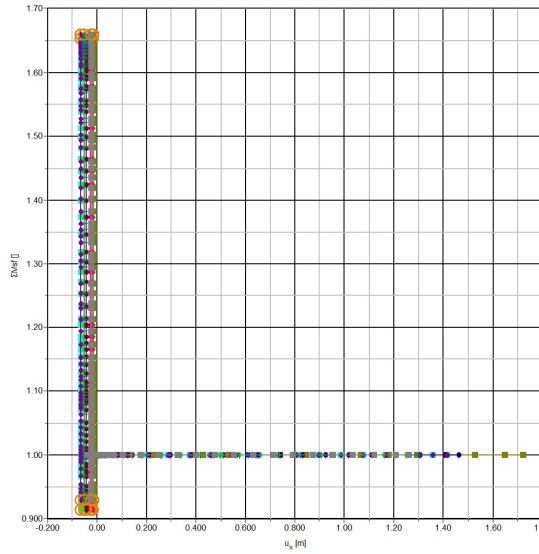
Figure 78 : Monitoring nodes distribution in Cross Section 05.

Table 70 : Coordinates and functional description of monitoring nodes in Cross Section 05.

Node	Side	X-coordinate	Y-coordinate	Geotechnical Location	Functional Description
Node 01	Left	29.66	1030	Upper Crest (Left)	Monitoring settlement at the left upper crest.
Node 02	Left	94.31	1005	Slope Face (Left)	Measuring lateral deformation of the left slope face.
Node 03	Left	164.4	975.7	Slope Toe (Left)	Monitoring foundation contact near road base on left side.
Node 04	Right	184.4	975.7	Slope Toe (Right)	Monitoring toe displacement on the right side.
Node 05	Right	200.0	981.6	Slope Face (Right)	Observing deformation of the right slope face.
Node 06	Right	209.5	986.3	Upper Crest (Right)	Monitoring crest settlement on the right side.

IV.3.5.1.4. Evaluation of Stability and Deformation Curves

U_x Deformations:



U_y Deformations:

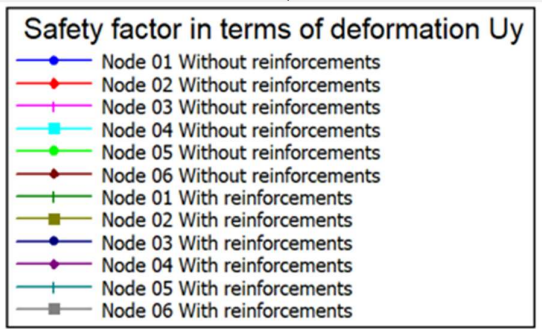
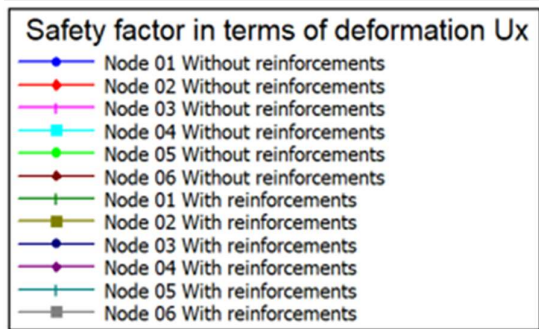
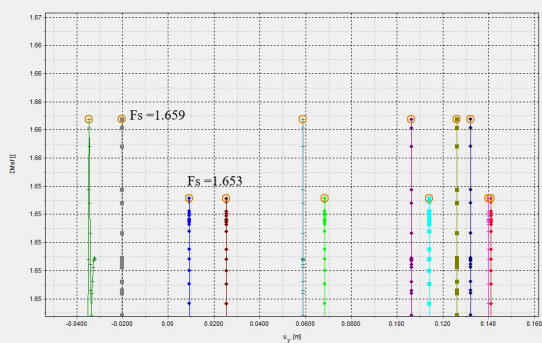
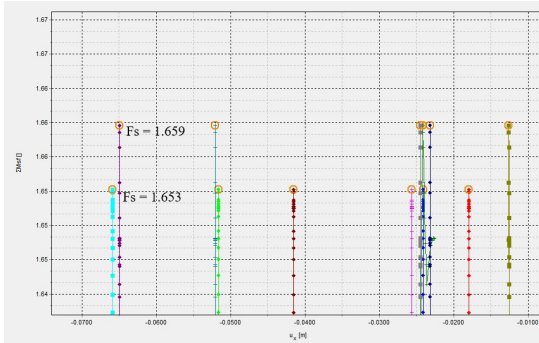
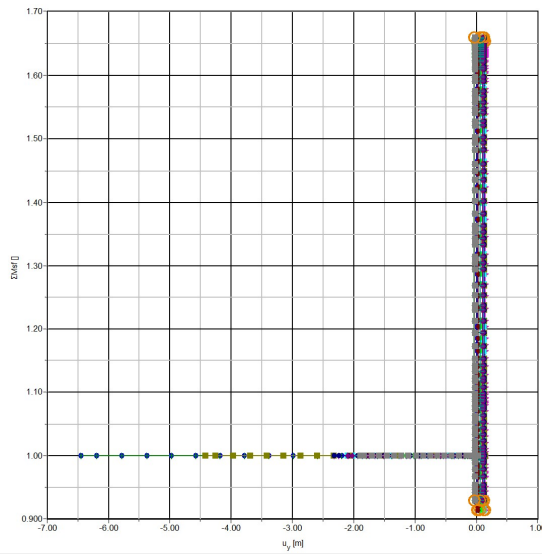


Figure 79 : Safety Factor vs. Deformations (U_x & U_y) for Section 05 (Reinforced and Unreinforced States).

The stability analysis demonstrates that the Factor of Safety (FoS) increased from 1.653 in the unsupported state to 1.659 following the implementation of reinforcements. Both values strictly satisfy the design requirements ($FoS \geq 1.5$), confirming the overall stability of the excavated section under the applied design loads.

The marginal increase in FoS indicates that the excavated section exhibits high inherent stability and structural adequacy even before intervention. In this context, the reinforcement system acts as a proactive, precautionary measure designed to mitigate potential localized deformations and ensure long-term structural integrity, rather than serving as a critical support for global stability.

IV.3.5.1.5. Dynamic deformation results

A. U_x Deformations:

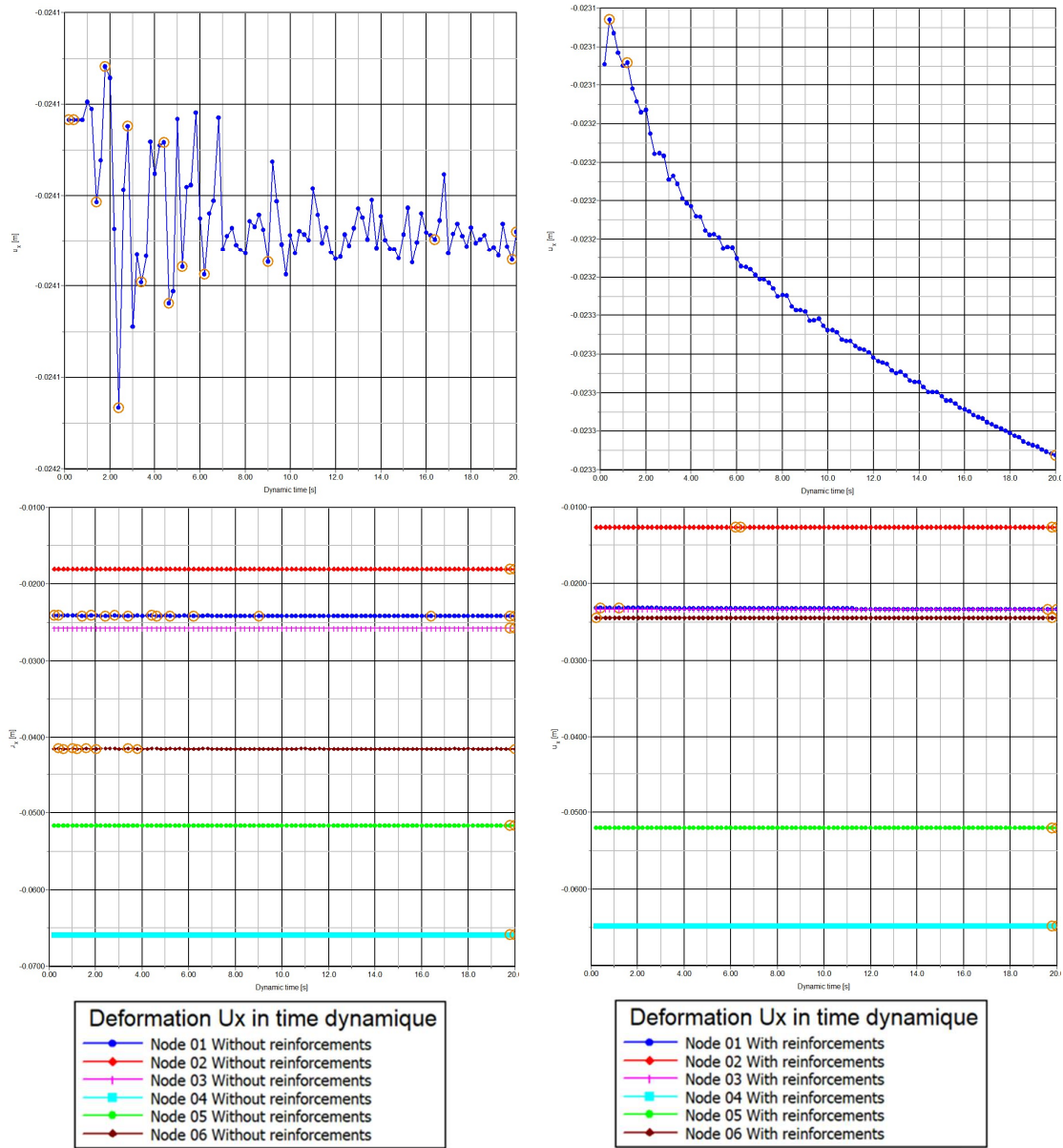


Figure 80 : Deformation U_x vs. Dynamic Time for Section 05 (With and Without Reinforcements).

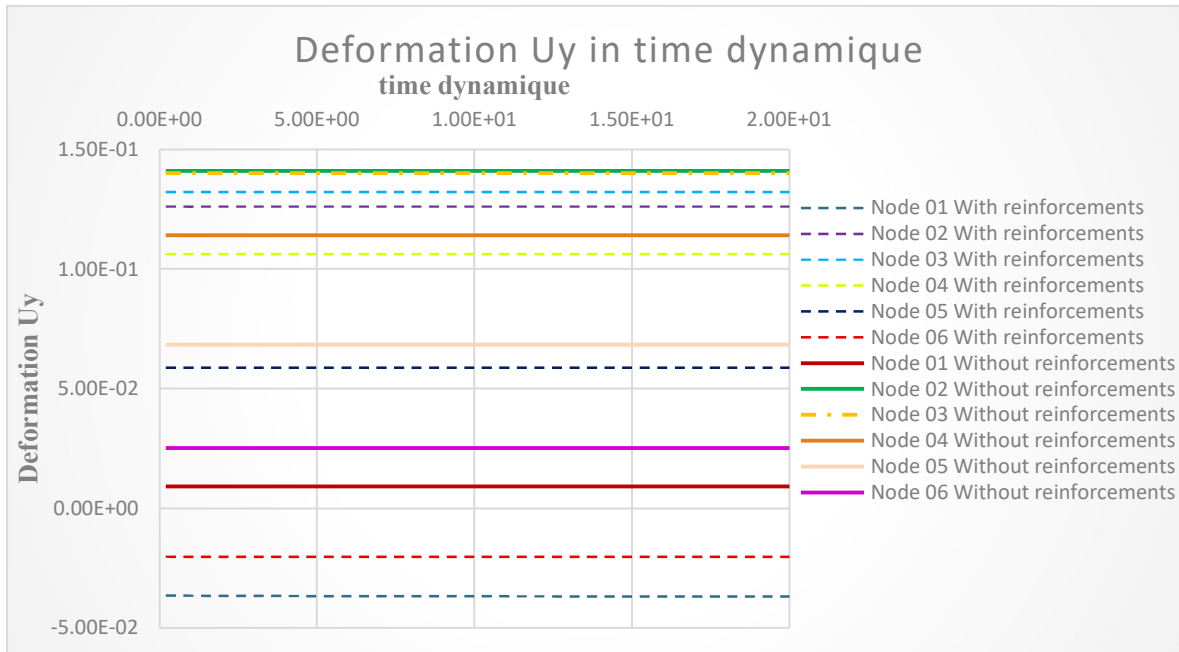


Figure 83 : Vertical deformation (U_y) evolution curve vs. dynamic time for various comparison nodes for Section 05.

The evolution of horizontal displacements (U_x) and vertical displacements (U_y) over time (from 0.2 to 20 seconds) was analyzed for six monitoring nodes (Nodes 01–06) distributed along cross-section 05 (Mio-Pliocene clay formation). The comparison was carried out between two cases (Without reinforcements / With reinforcements).

Tableau 2 : Comparison of Horizontal deformation (U_x) and Improvement Percentages for Section 05.

Node	U_x Without reinforcement (m)	U_x With reinforcement (m)	Improvement (%)
Node 01	-0.0241	-0.0233	3.30%
Node 02	-0.018	-0.013	30.00%
Node 03	-0.0257	-0.0234	8.90%
Node 04	-0.0659	-0.0649	1.50%
Node 05	-0.0516	-0.052	-0.80%
Node 06	-0.0415	-0.024	41.20%

Tableau 3 : Comparison of Vertical deformation (U_y) and Improvement Percentages for Section 05.

Node	U_y Without reinforcement (m)	U_y With reinforcement (m)	Improvement (%)
Node 01	+0.00915	-0.0368	Full reversal
Node 02	0.141	0.126	10.60%
Node 03	0.14	0.132	5.70%
Node 04	0.114	0.106	7.00%
Node 05	0.0684	0.0588	14.00%
Node 06	0.0252	-0.0203	Full reversal

Note :

- The positive sign (+) means heave.
- The negative sign (-) means settlement.
- The reinforcement improved the local behavior, particularly at nodes 2 and 6, making it a precautionary measure for long-term safety against cracking and under exceptional conditions.

IV.3.5.1.6. Flow Failure Analysis

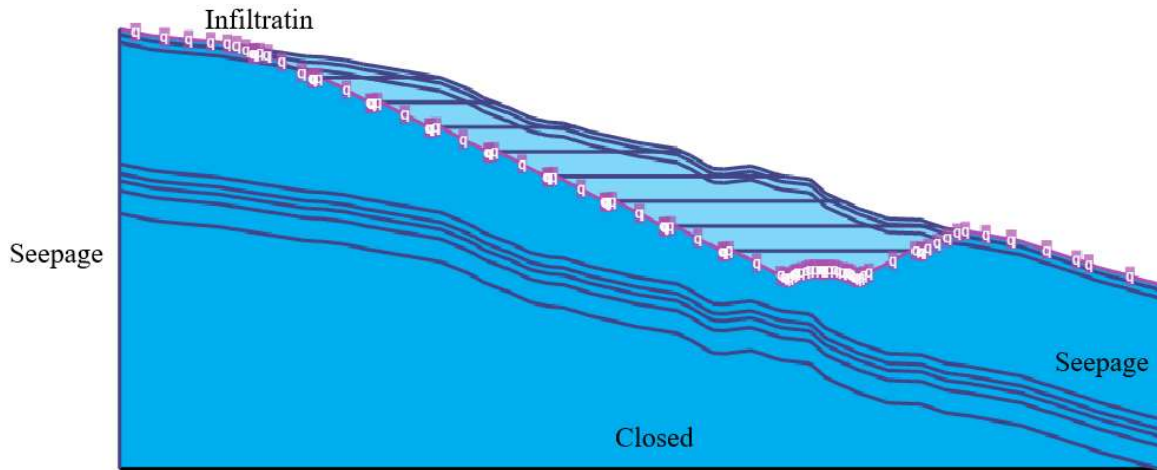


Figure 84 : Distribution of Seepage and Flow Boundary Conditions for Cross-Section 05.

The two-dimensional analysis indicated that the model reached a state of flow failure at time step 61 ($t \approx 0.01296$ days), where the maximum excess pore water pressure (p_{max}) attained a value of approximately 157.7 kN/m^2 . This failure is manifested by the model's inability to achieve equilibrium (non-convergence) and the development of a critical slip surface extending through the railway excavations. The progressively divergent displacement values (not shown in the table, but inferred from the pore pressure peak) indicate that the generated excess pore water pressure has led to a reduction in the soil's shear strength, resulting in a loss of the slope's dynamic stability. Subsequently, the pore pressure dissipated gradually, dropping to zero after step 1060 ($t \approx 0.02558$ days), indicating post-failure consolidation. This indicates the need to protect the area from water seepage by using impermeable materials, such as geomembranes, and by diverting water away from the site through appropriate drainage systems.

IV.3.5.2. PROFILE 09

IV.3.5.2.1. Numerical Simulation Stages

Tableau 4 : Summary of calculation phases and sequential construction steps for Section 09.

PHASE	DESCRIPTION
INITIAL PHASE	Initial balancing using Gravity Loading.
PHASE 1-13	Excavation processes (sequential removal of soil clusters).
PHASE 14	Installation of support/lining systems.
PHASE 15	Activation of roadbed layers (Ballast/Sub-ballast).
PHASE 16	Application of distributed surface load (80 kN/m^2).
PHASE 17	Safety Analysis (ϕ/c reduction).

IV.3.5.2.2. Evaluation of Soil Displacement Response

U_x Deformations:

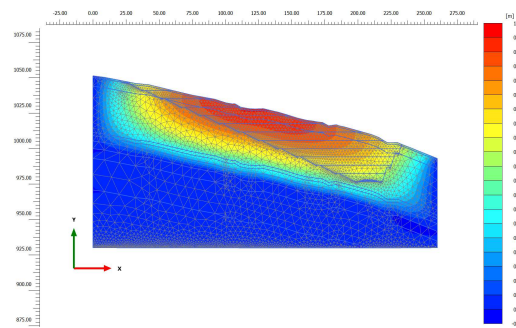


Figure 85 : Horizontal Displacement (U_x) distribution at the Initial Construction Phase.

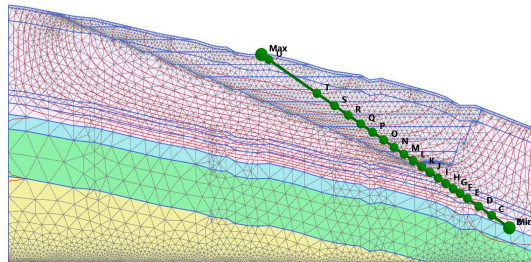


Figure 86 : Cross-section showing U_x displacement contours at the initial construction phase.

Maximum value = 0.9579 m (Element 1119 at Node 45259)
Minimum value = -0.07244×10^{-3} m (Element 6235 at Node 79883)

U_y Deformations:

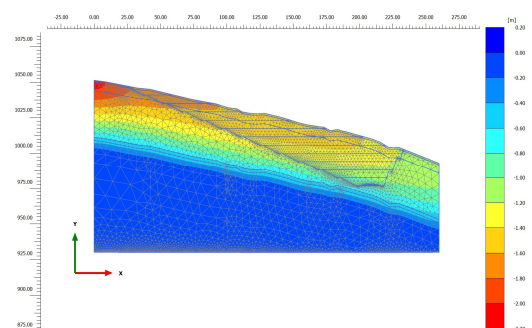


Figure 87 : Vertical Displacement (U_y) distribution at the Initial Construction Phase.

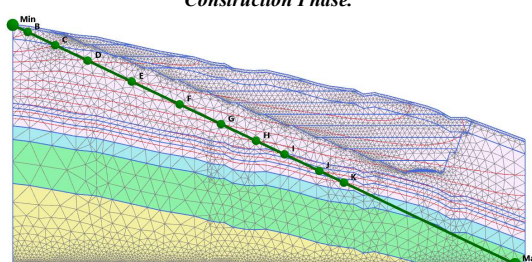


Figure 88 : Cross-section showing U_y displacement contours at the initial construction phase.

Maximum value = 0.000 m (Element 6960 at Node 74171)
Minimum value = -2.057 m (Element 13 at Node 3091)

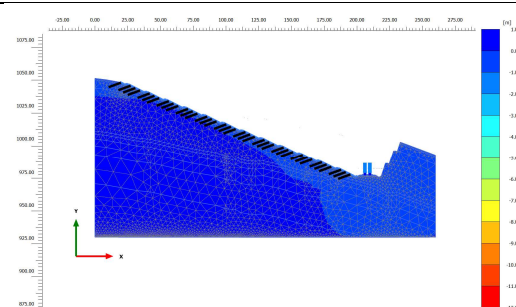


Figure 89 : Vertical Displacement (U_y) distribution at Phase 17.

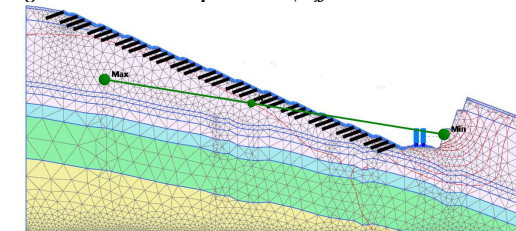


Figure 90 : Cross-section showing U_x displacement contours at Phase 17.

Maximum value = 8.483×10^{-3} m (Element 3224 at Node 11320)
Minimum value = -11.15 m (Element 4324 at Node 88211)

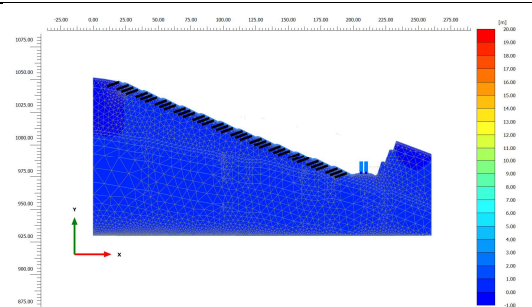


Figure 91 : Vertical Displacement (U_y) distribution at Phase 17.

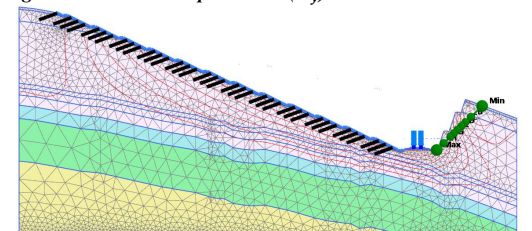


Figure 92 : Cross-section showing U_y displacement contours at Phase 17.

Maximum value = 19.78 m (Element 4356 at Node 89099)
Minimum value = -0.01613 m (Element 4783 at Node 89623)

Table 71 : Summary of Nodal Displacement Extremes (U_x And U_y) per Calculation Phase for Section 09.

Phase N :	Maximum value		Minimum value	
	u_x	u_y	u_x	u_y
Initial Phase	0.9579	0.000	-0.07244×10^{-3}	-2.057
1	1.658×10^{-3}	2.278×10^{-3}	-0.5242×10^{-3}	-1.170×10^{-3}
2	4.224×10^{-3}	0.01764	-3.876×10^{-3}	-1.618×10^{-3}
3	3.477×10^{-3}	0.03335	-7.294×10^{-3}	-1.826×10^{-3}
4	4.825×10^{-3}	0.04605	-9.714×10^{-3}	-2.382×10^{-3}
5	5.568×10^{-3}	0.05748	-0.01317	-3.096×10^{-3}
6	6.019×10^{-3}	0.06651	-0.01759	-3.563×10^{-3}
7	6.398×10^{-3}	0.07495	-0.02185	-3.976×10^{-3}
8	6.734×10^{-3}	0.08148	-0.02627	-4.351×10^{-3}
9	7.014×10^{-3}	0.08492	-0.03047	-4.669×10^{-3}
10	7.220×10^{-3}	0.08709	-0.03382	-4.911×10^{-3}
11	7.353×10^{-3}	0.08707	-0.04574	-5.072×10^{-3}
12	7.435×10^{-3}	0.08470	-0.06267	-5.181×10^{-3}
13	7.479×10^{-3}	0.08402	-0.09451	-0.01644
14	8.351×10^{-3}	0.07980	-0.09482	-0.01655
15	8.456×10^{-3}	0.07980	-0.09500	-0.01664
16	8.482×10^{-3}	0.07979	-0.09508	-0.01667
17	8.483×10^{-3}	19.78	-11.15	-0.01613

IV.3.5.2.3. Definition of Monitoring Nodes

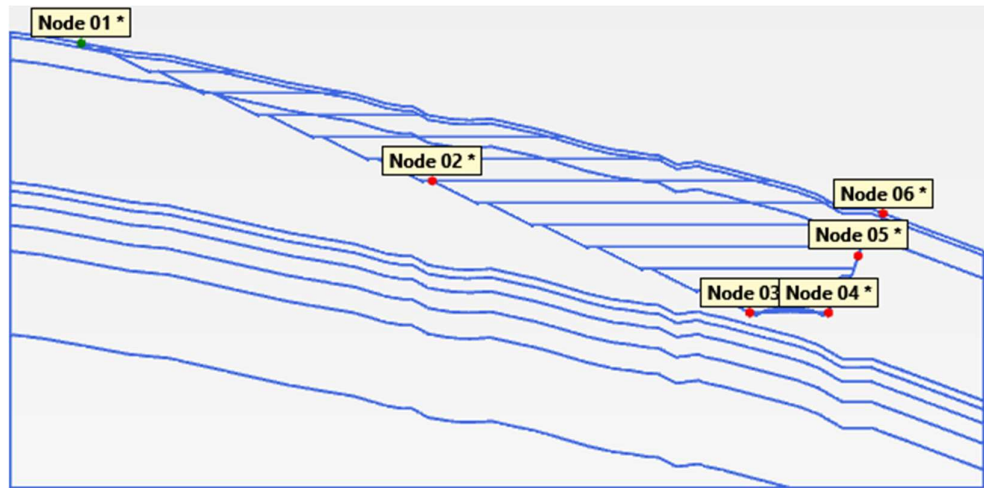


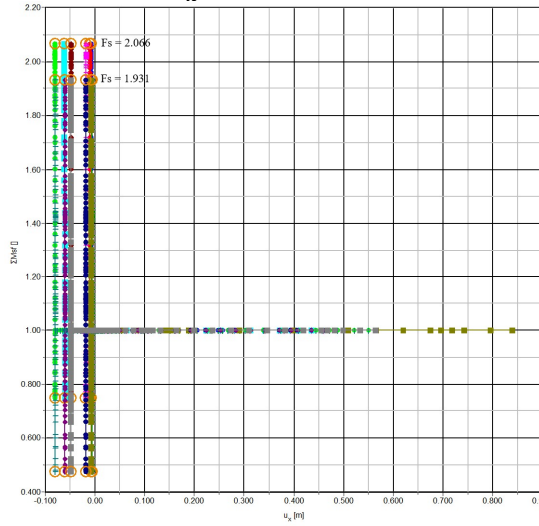
Figure 93 : Monitoring nodes distribution in Cross Section 09.

Table 72 : Localization and coordinates determination of the nodes for the cross-section 09.

Node	Side	X-coordinate	Y-coordinate	Geotechnical Location	Functional Description
Node 01	Left	19.06	1048	Upper Crest (Left)	Monitoring settlement at the highest left crest above road base.
Node 02	Left	112.8	1012	Slope Face (Left)	Measuring lateral deformation of the left slope face.
Node 03	Left	197.5	976.6	Slope Toe (Left)	Monitoring foundation contact near road base on left side.
Node 04	Right	218.5	976.6	Slope Toe (Right)	Monitoring toe displacement on the right side adjacent to road base.
Node 05	Right	226.4	991.7	Slope Face (Right)	Observing deformation of the right slope face.
Node 06	Right	233.1	1003	Upper Crest (Right)	Monitoring crest settlement on the right upper slope.

IV.3.5.2.4. Evaluation of Stability and Deformation Curves

U_x Deformations :



U_y Deformations :

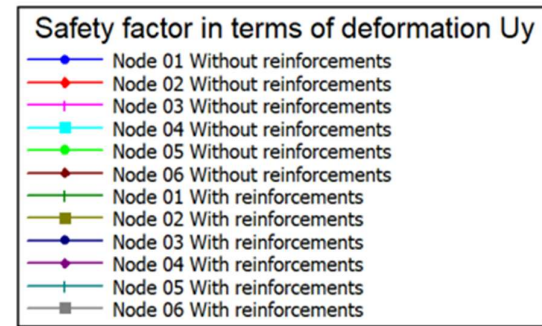
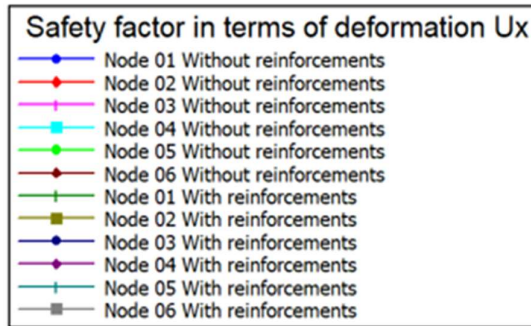
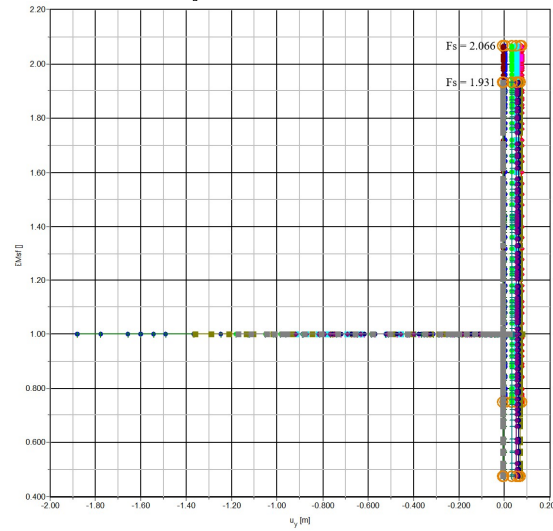


Figure 94 : Safety Factor vs. Deformations (U_x & U_y) for Section 09 (Reinforced and Unreinforced States)

The displacement-time dynamic analysis indicates that the Factor of Safety (FoS) decreased from 2.066 in the unsupported condition to 1.931 after the application of reinforcements. This reduction in the global safety factor is attributed to changes in the soil-structure interaction model, where the introduction of reinforcements induces additional stresses and alters the stress redistribution paths within the excavated mass. Despite this numerical decrease, the structural system maintains a high level of resilience under the applied loads. The observed convergence in the deformation patterns of the monitored nodes suggests that the reinforcement system effectively limits excessive soil displacement, contributing to a controlled geotechnical response. In this context, the reinforcements enhance local performance and manage time-dependent deformations, thereby ensuring the long-term structural integrity and stability of the excavated section.

IV.3.5.2.5. Dynamic deformation results

A. U_x Deformations:

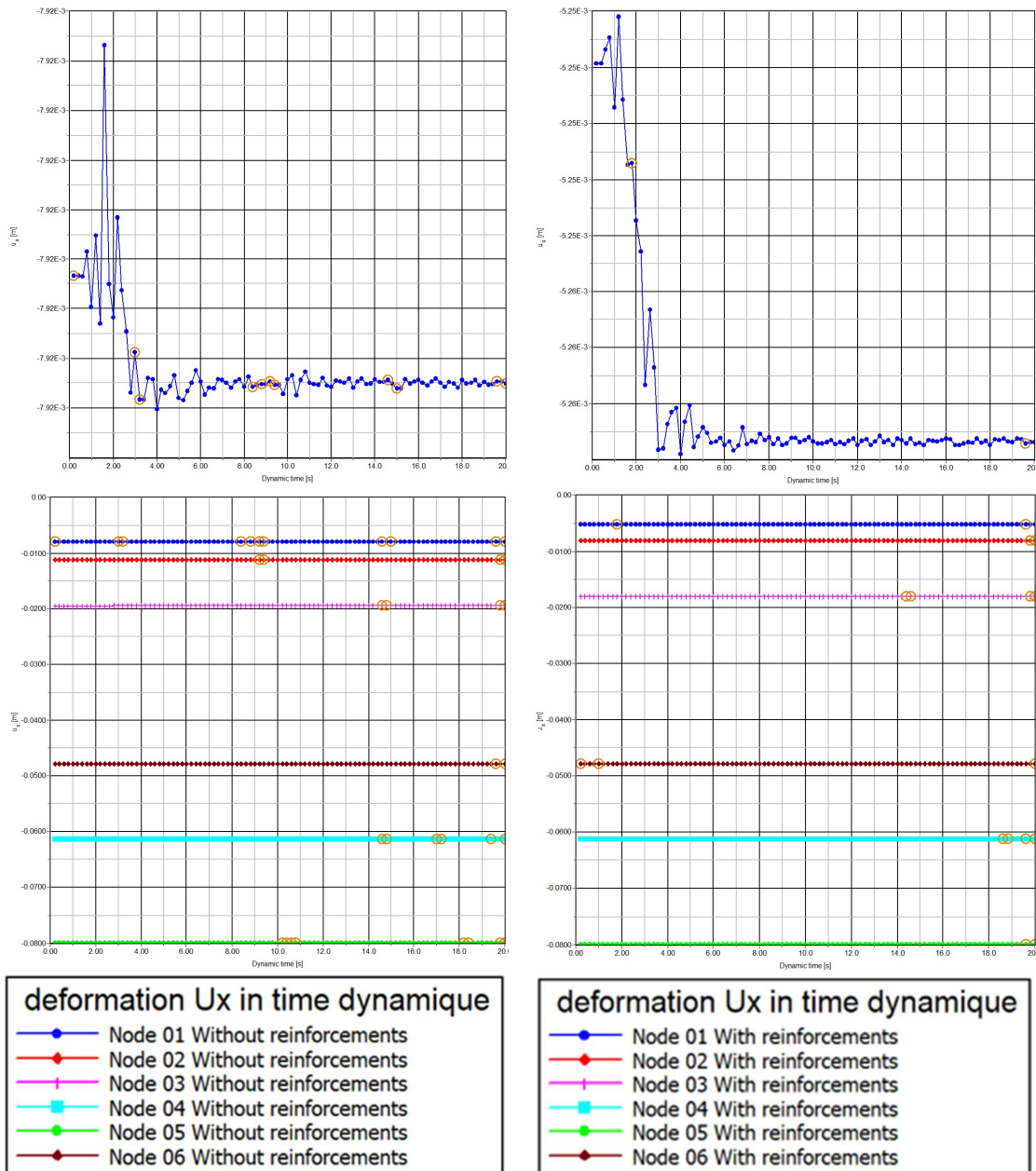


Figure 95 : Deformation (U_x) vs. Dynamic Time for Section 09 (With and Without Reinforcements).

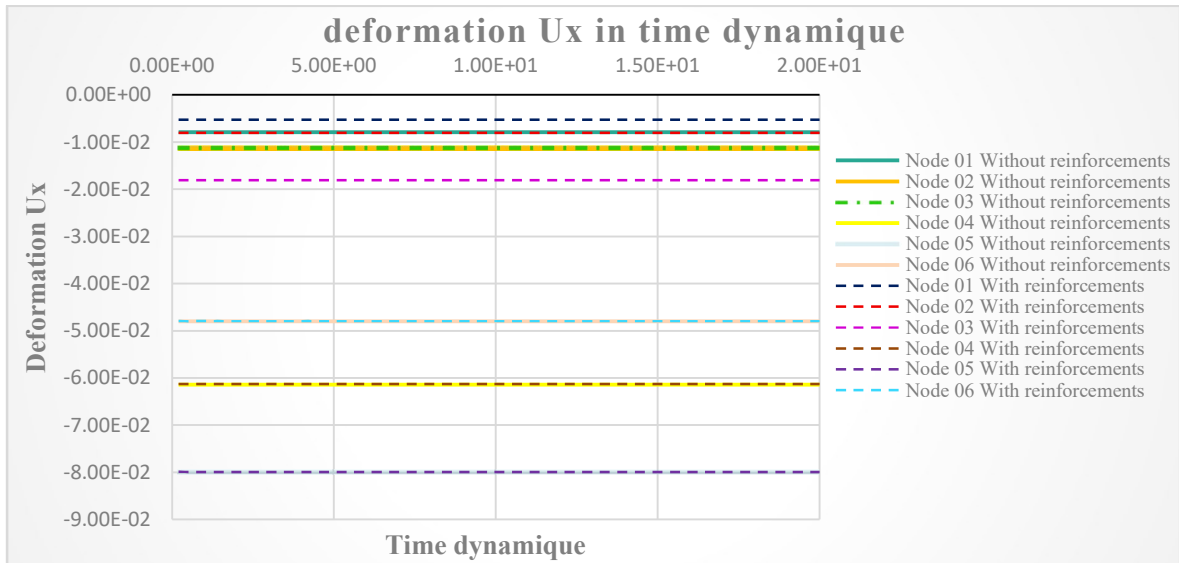


Figure 96 : Horizontal deformation (U_x) evolution curve vs. dynamic time for various comparison nodes for Section 09.

B. U_y Deformations:

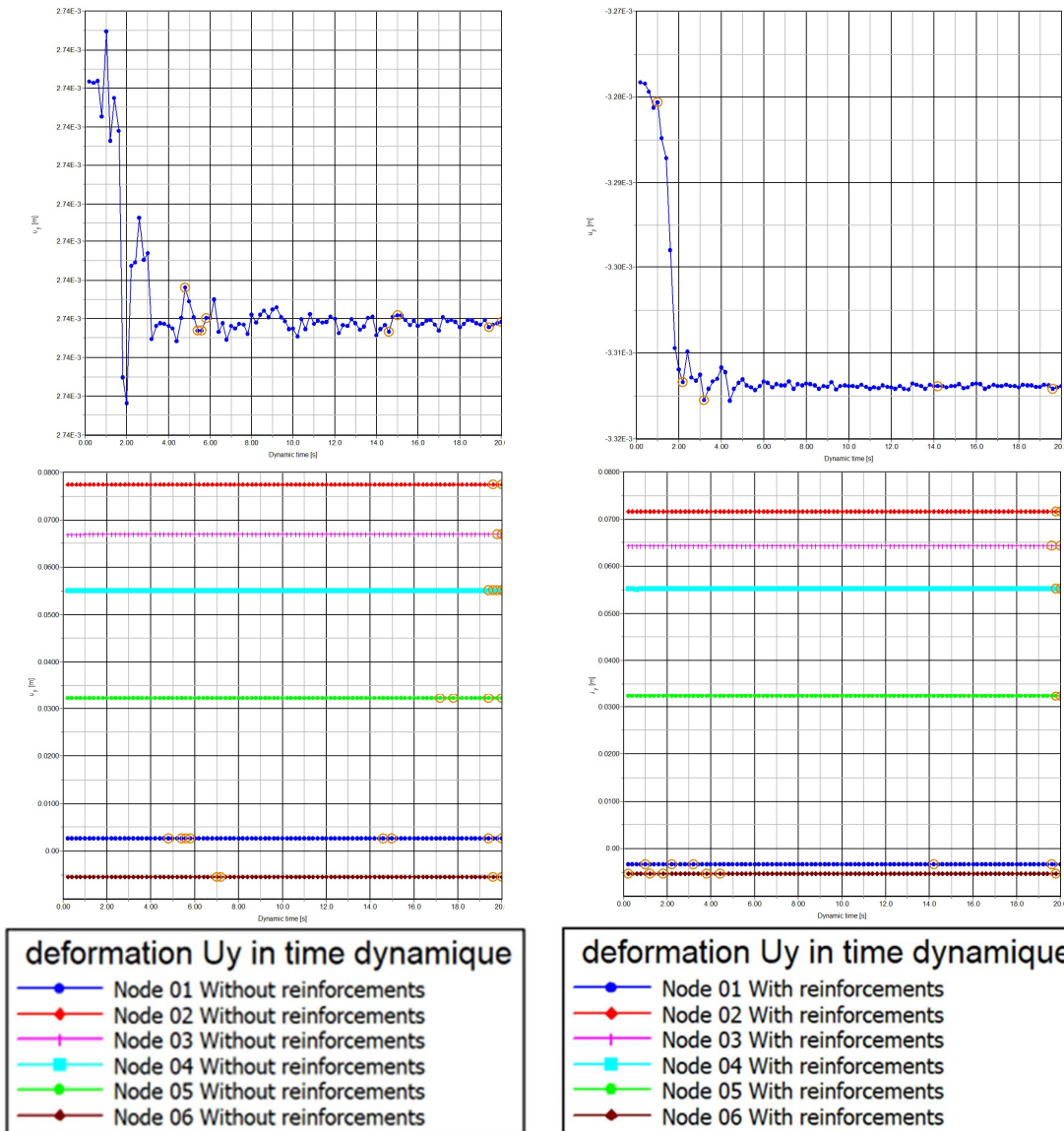


Figure 97 : Deformation (U_y) vs. Dynamic Time for Section 09 (With and Without Reinforcements).

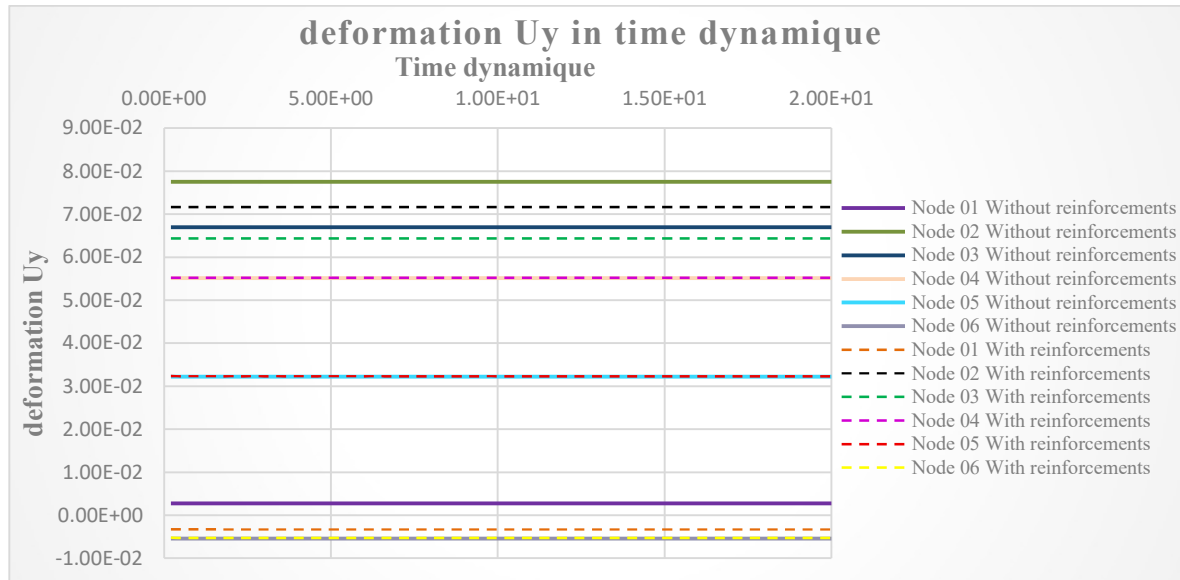


Figure 98 : Vertical deformation (U_y) evolution curve vs. dynamic time for various comparison nodes for Section 09.

The evolution of horizontal displacements (U_x) and vertical displacements (U_y) over time (from 0.2 to 20 seconds) was analyzed for six monitoring nodes (Nodes 01–06) distributed along Cross-Section 05 within the Mio-Pliocene clay formation. The comparison was performed between two configurations (Without reinforcement / With reinforcement):

Table 73 : Comparison of Horizontal deformation (U_x) and Improvement Percentages for Section 09.

Node	U_x Without reinforcement (m)	U_x With reinforcement (m)	Improvement (%)
Node 01	-0.00792	-0.00525	33.71%
Node 02	-0.01120	-0.00806	28.04%
Node 03	-0.01120	-0.01810	-61.61%
Node 04	-0.06140	-0.06130	0.16%
Node 05	-0.07990	-0.07980	0.13%
Node 06	-0.04800	-0.04790	0.21%

Table 74 : Comparison of Vertical deformation (U_y) and Improvement Percentages for Section 09.

Node	U_y Without reinforcement (m)	U_y With reinforcement (m)	Improvement (%)
Node 01	0.00274	-0.00331	Full reversal
Node 02	0.07750	0.07160	7.61%
Node 03	0.06690	0.06430	3.89%
Node 04	0.05510	0.05520	-0.18%
Node 05	0.03220	0.03230	-0.31%
Node 06	-0.00544	-0.00528	2.94%

Note :

- The positive sign (+) indicates heave.
- The negative sign (–) indicates settlement.

The reinforcement significantly improved the local behavior, particularly at nodes 1 and 6 for vertical displacements (full reversal from heave to settlement), and at nodes 1, 2 and 6 for horizontal displacements (improvements of 28–34%), despite a localized worsening at node 3. This makes it a precautionary measure for long-term safety against cracking and under exceptional conditions.

IV.3.5.2.6. Flow Failure Analysis

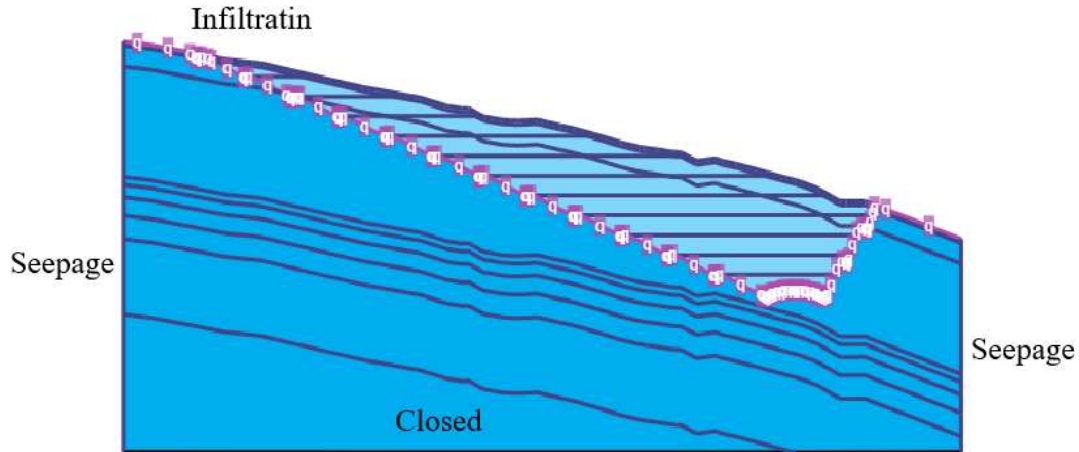


Figure 99 : Distribution of Seepage and Flow Boundary Conditions for Cross-Section 09.

Infiltration conditions used :

$$q: 0.1200 \text{ m/day} \quad \psi_{\text{Min}}: -1.000 \text{ m} \quad \psi_{\text{Max}}: 0.1000 \text{ m}$$

The two-dimensional analysis indicated that the model reached a state of flow failure at time $t = 0.01645$ days, where the maximum excess pore water pressure (p_{max}) attained a value of approximately 823.971 kN/m^2 . This failure is manifested by the model's inability to achieve equilibrium (non-convergence) and the development of a critical slip surface extending through the railway excavations. The progressively divergent displacement values (not shown in the table, but inferred from the pore pressure peak) indicate that the generated excess pore water pressure has led to a reduction in the soil's shear strength, resulting in a loss of the slope's dynamic stability. This indicates the need to protect the area from water seepage by using impermeable materials, such as geomembranes, and by diverting water away from the site through appropriate drainage systems.

IV.3.5.3. PROFILE 15

IV.3.5.3.1. Numerical Simulation Stages

Tableau 5 : Summary of calculation phases and sequential construction steps for Section 15.

PHASE	DESCRIPTION
INITIAL PHASE	Initial balancing using Gravity Loading.
PHASE 1-4	Excavation processes (sequential removal of soil clusters).
PHASE 05	Activation of roadbed layers (Ballast/Sub-ballast).
PHASE 06	Application of distributed surface load (80 kN/m^2).
PHASE 07	Safety Analysis (ϕ/c reduction).

IV.3.5.3.2. Definition of Monitoring Nodes

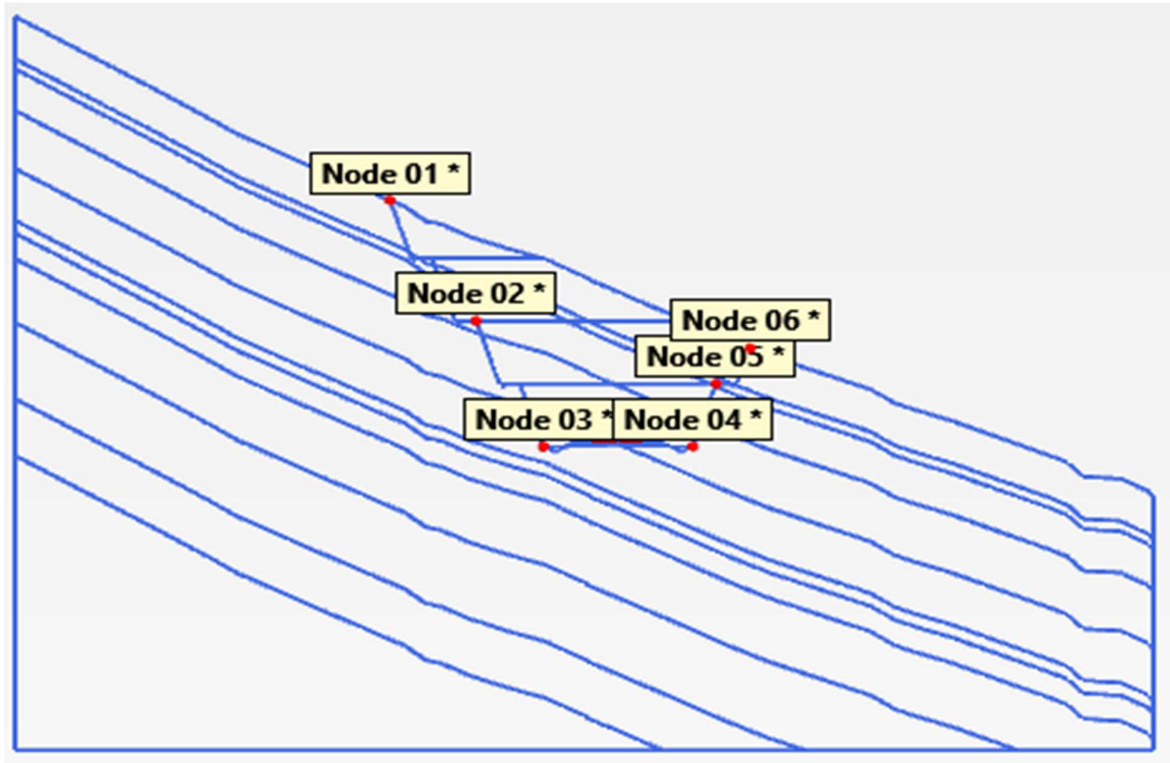


Figure 100 : Monitoring nodes distribution in Cross Section 15.

Table 75 : Localization and coordinates determination of the nodes for the cross-section 15.

Node	Side	X-coordinate	Y-coordinate	Geotechnical Location	Functional Description
Node 01	Left	52.67	1016	Upper Crest (Left)	Monitoring settlement at the left upper crest above road base.
Node 02	Left	64.77	997.4	Slope Face (Left)	Measuring lateral deformation of the left slope face.
Node 03	Left	74.23	977.7	Slope Toe (Left)	Monitoring foundation contact near road base on left side.
Node 04	Right	95.23	977.7	Slope Toe (Right)	Monitoring toe displacement on the right side adjacent to road base.
Node 05	Right	98.51	987.5	Slope Face (Right)	Observing deformation of the right slope face.
Node 06	Right	103.3	993.1	Upper Crest (Right)	Monitoring crest settlement on the right upper slope.

IV.3.5.3.3. Evaluation of Soil Displacement Response

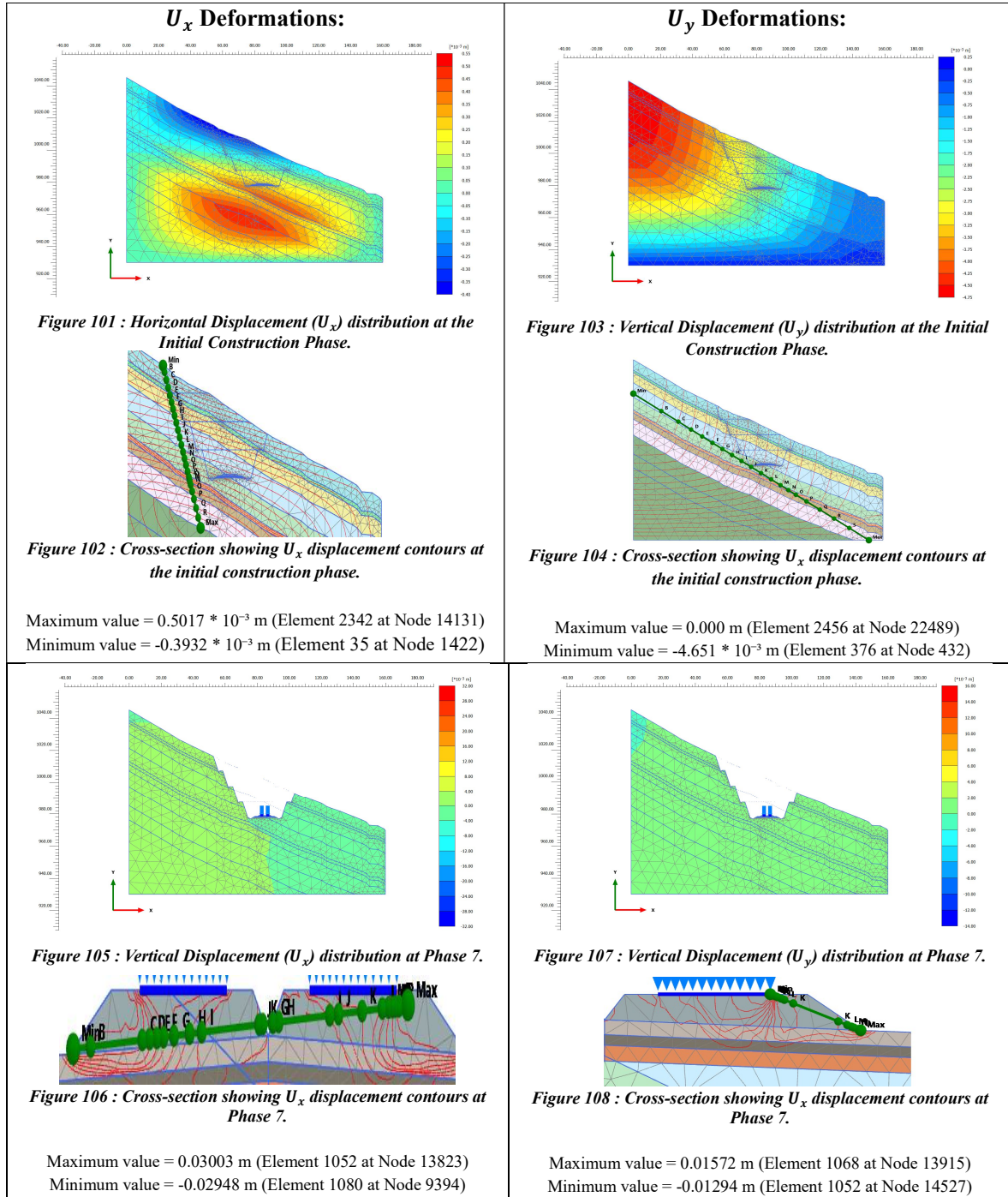


Table 76 : Summary of Nodal Displacement Extremes (U_x And U_y) per Calculation Phase for Section 15.

Phase N :	Maximum value		Minimum value	
	u_x	u_y	u_x	u_y
Initial Phase	0.5017×10^{-3}	0.000	-0.3932×10^{-3}	-4.651×10^{-3}
1	0.1172×10^{-3}	0.08108×10^{-3}	-0.1231×10^{-3}	-0.08187×10^{-3}
2	0.1584×10^{-3}	0.2521×10^{-3}	-0.1522×10^{-3}	-0.01281×10^{-3}
3	0.1897×10^{-3}	0.4657×10^{-3}	-0.1726×10^{-3}	0.000
4	0.2227×10^{-3}	0.6297×10^{-3}	-0.1870×10^{-3}	-2.873×10^{-6}
5	0.2234×10^{-3}	0.6182×10^{-3}	-0.1929×10^{-3}	-3.930×10^{-6}
6	0.2241×10^{-3}	0.6032×10^{-3}	-0.2558×10^{-3}	-5.158×10^{-6}
7	0.03003	0.01572	-0.02948	-0.01294

IV.3.5.3.4. Evaluation of Stability and Deformation Curves

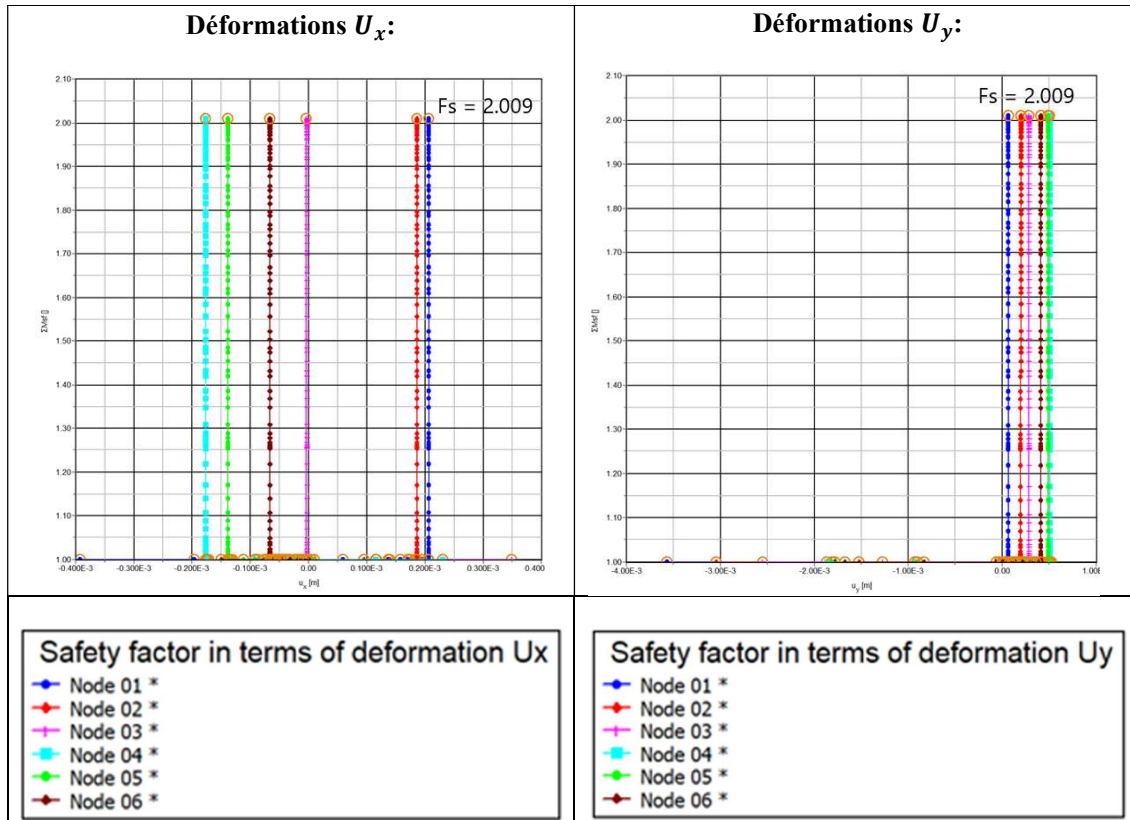


Figure 109 : Safety Factor vs. Deformations (U_x & U_y) for Section 15 (Reinforced and Unreinforced States).

The structural stability analysis indicates that the Factor of Safety (FoS) computed for the section in its unsupported condition is 2.009. This high value suggests that the geotechnical system inherently possesses a high level of resilience and resistance under self-weight and applied loads, without the immediate need for external structural interventions.

IV.3.5.3.5. Dynamic deformation results

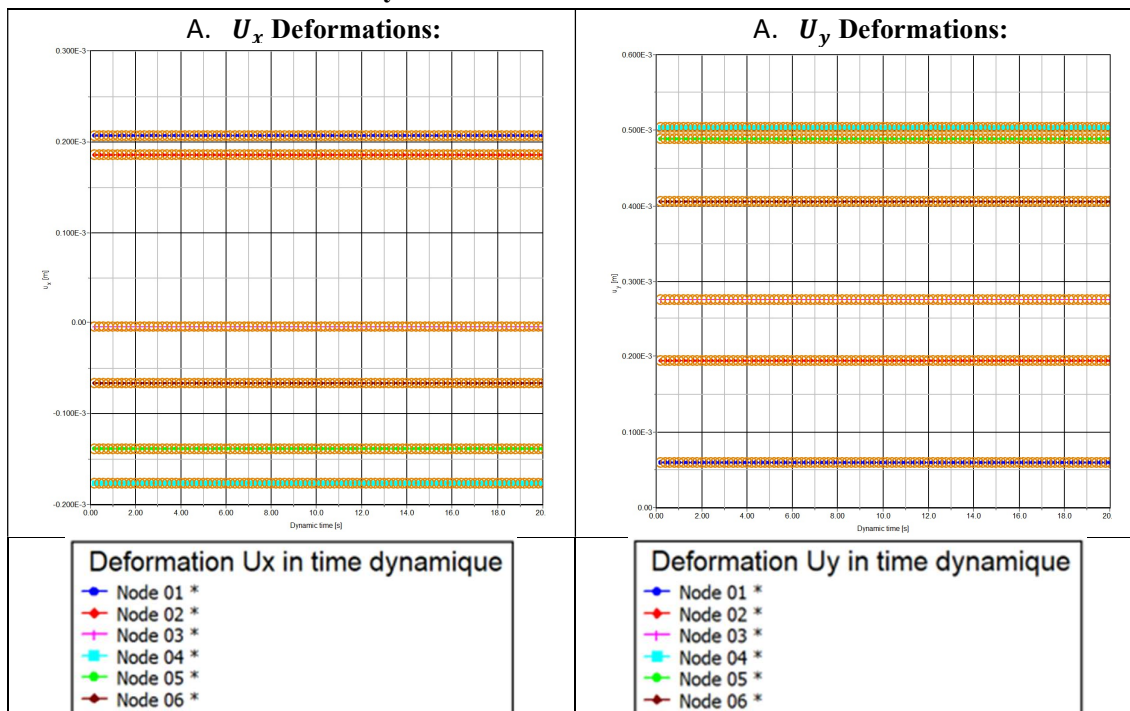


Figure 110 : Deformation (U_x) vs. Dynamic Time for Section 15 (With and Without Reinforcements).

The evolution of horizontal displacements (U_x) and vertical displacements (U_y) over time (from 0.2 to 20 seconds) was analyzed for six monitoring nodes (Nodes 01–06) distributed along cross-section 05 (Clay formation). The comparison was carried out between two cases (Without reinforcements / With reinforcements).

Table 77 : Comparison of Horizontal deformation (U_x) and Improvement Percentages for Section 15.

Node	U_x Without reinforcement (m)
Node 01	-0.0079
Node 02	-0.0112
Node 03	-0.0112
Node 04	-0.0614
Node 05	-0.0799
Node 06	-0.048

Table 78 : Comparison of Vertical deformation (U_y) and Improvement Percentages for Section 15.

Node	U_y Without reinforcement (m)
Node 01	0.00274
Node 02	0.0775
Node 03	0.0669
Node 04	0.0551
Node 05	0.0322
Node 06	-0.00544

Note

- The positive sign (+) indicates heave.
- The negative sign (–) indicates settlement.

IV.3.5.3.6. Flow Failure Analysis

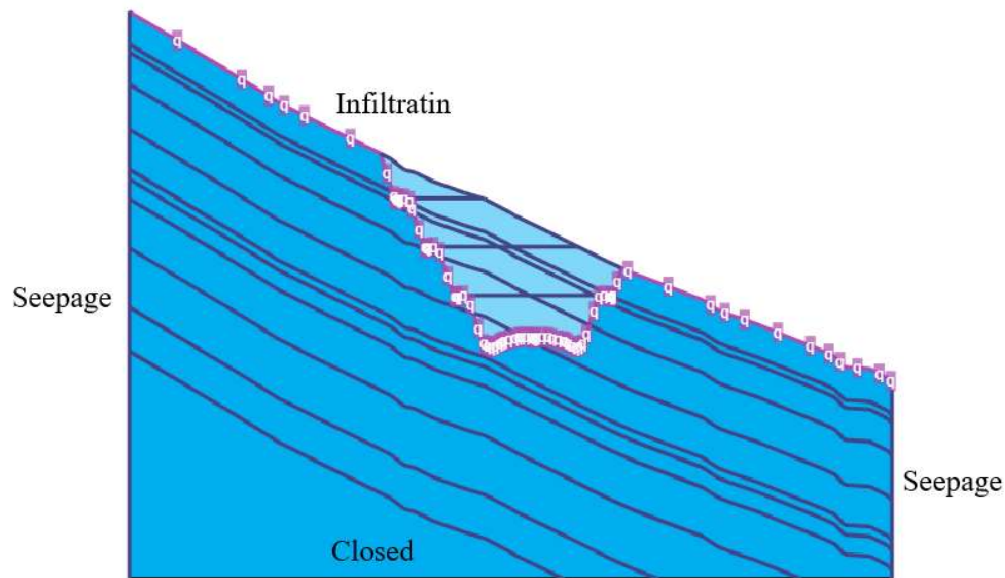


Figure 111 : Distribution of Seepage and Flow Boundary Conditions for Cross-Section 15.

Infiltration conditions used :

$$q: 0.1200 \text{ m/day}$$

$$\psi_{\text{Min}}: -1.000 \text{ m}$$

$$\psi_{\text{Max}}: 0.1000 \text{ m}$$

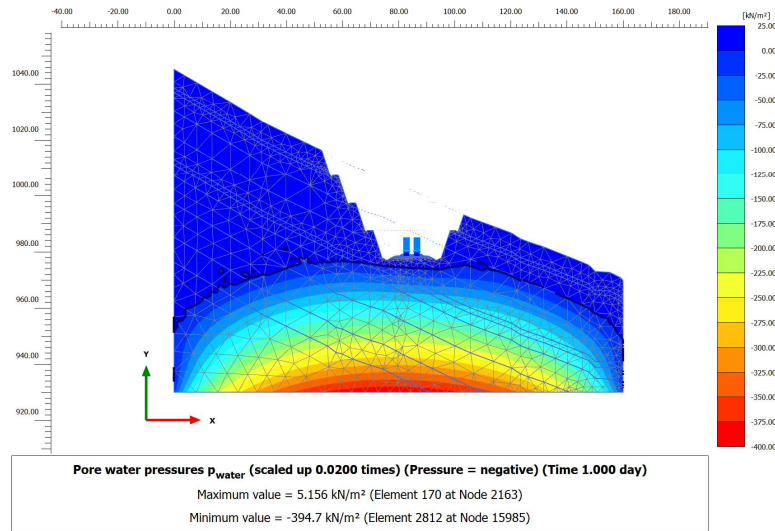


Figure 112 : Pore water pressure (p_{water}) distribution contour for Section 15.

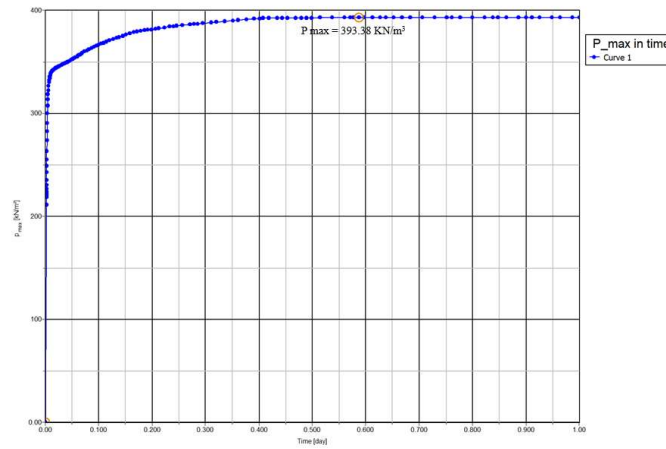


Figure 113 : Curve of maximum pore water pressure (P_{max}) versus time.

A. Hydro-Geotechnical Analysis and Pore Pressure Distribution

The model boundary conditions apply a rainfall infiltration mechanism on the sloping surface, with active seepage boundaries on the sides and a fully closed base. Based on this time-dependent hydraulic improvement, the results at the end of the first day indicate a precise formation of the phreatic level, where a negative suction pressure of up to -394.7 kN/m^2 is recorded in the unsaturated zone, while positive pressures stabilize in the saturated zone. The curve of maximum pressure evolution (P_{max}) over time shows a rapid dynamic response during the early hours, followed by a steady-state seepage condition reaching an absolute value of 393.38 kN/m^2 After approximately 14 hours, confirming excellent numerical stability of the model. The model boundary conditions simulate rainfall infiltration along the sloping surface, with active seepage boundaries applied at the lateral sides and a fully impermeable base. Based on this transient hydraulic process, the results obtained at the end of the first day indicate the accurate development of the phreatic level. Negative pore-water pressures reaching up to -394.7 kN/m^2 are observed within the unsaturated zone, whereas positive pore pressures stabilize in the saturated region. Furthermore, the evolution curve of the maximum pore

pressure (P_{max}) over time demonstrates a rapid transient response during the initial hours, followed by a steady-state seepage regime reaching an absolute value of 393.38 kN/m^2 after approximately 14 hours. These results confirm the excellent numerical stability and hydraulic convergence of the model.

B. Failure Mechanism and Identification of the Critical Structure

From a mechanical perspective, the vertical displacement (U_y) contour clearly reveals the development of a deep slip surface extending beneath the applied geotechnical structure toward the slope toe. The concentration of maximum deformations along this failure path allows it to be identified as the critical structure of the model, representing the governing mechanism controlling the overall stability of the soil mass. In contrast, the remaining portions of the geological formation located outside this critical zone inherently exhibit higher safety factors.

C. Stability Assessment and Overall Safety Factor

The safety calculation curves as a function of displacement (both horizontal (u_x) and total displacement ($|u|$)) confirm that the overall safety factor of the section in its unsupported condition is equal to 2.009. The full numerical convergence observed at all monitored points, together with their vertical stability at the peak of the calculation process and the very small virtual displacements recorded (less than $(1.00 \text{ times } 10^{-3} \text{ M})$), demonstrate that the geotechnical system possesses a high degree of inherent stability and resilience.

These results indicate a controlled and well-regulated geotechnical behavior, ensuring satisfactory structural safety and long-term mechanical stability of the section, even under seepage conditions and applied loads, without requiring immediate external reinforcement.

IV.3.5.4. PROFILE 20

IV.3.5.4.1. Numerical Simulation Stages

Tableau 6 : Summary of calculation phases and sequential construction steps for Section 20.

PHASE	DESCRIPTION
INITIAL PHASE	Initial balancing using Gravity Loading.
PHASE 1-4	Excavation processes (sequential removal of soil clusters).
PHASE 5	Installation of support/lining systems.
PHASE 6	Activation of roadbed layers (Ballast/Sub-ballast).
PHASE 7	Application of distributed surface load (80 kN/m^2).
PHASE 8	Safety Analysis (φ/c reduction).

IV.3.5.4.2. Definition of Monitoring Nodes

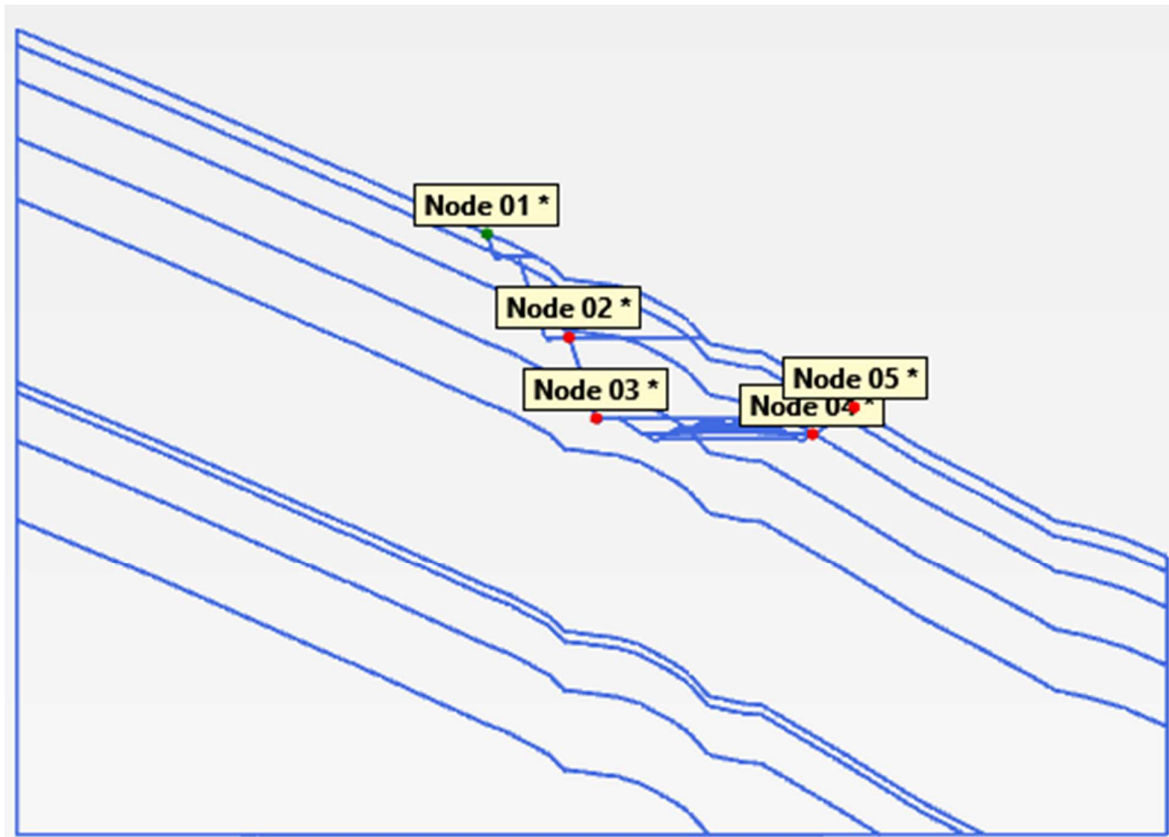


Figure 114 : Monitoring nodes distribution in Cross Section 20.

Table 79 : Localization and coordinates determination of the nodes for the cross-section 20.

Node	Side	X-coordinate	Y-coordinate	Geotechnical Location	Functional Description
Node 01	Left	57.22	1003	Upper Crest	Monitoring displacement at the highest point.
Node 02	Left	67.2	990.3	Slope Face	Monitoring deformation of the left slope face.
Node 03	Left	70.48	980.5	Slope Bench	Monitoring stress concentration at the shoulder.
Node 04	Right	96.83	978.6	Slope Toe	Monitoring foundation contact and toe displacement.
Node 05	Right	101.9	981.8	Upper Crest	Monitoring the crest of the right slope.

IV.3.5.4.3. Evaluation of Soil Displacement Response

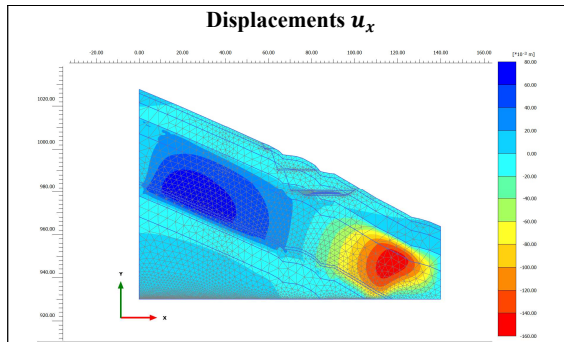


Figure 115 : Horizontal Displacement (U_x) distribution at the Initial Construction Phase.

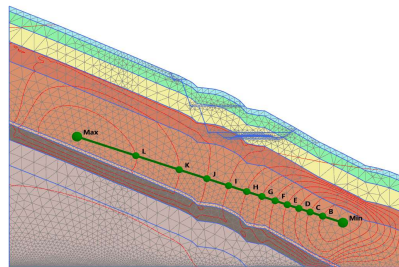


Figure 116 : Cross-section showing U_x displacement contours at the initial construction phase.

Maximum value = 0.07104 m (Element 2396 at Node 25687)
Minimum value = -0.1544 m (Element 1759 at Node 66668)

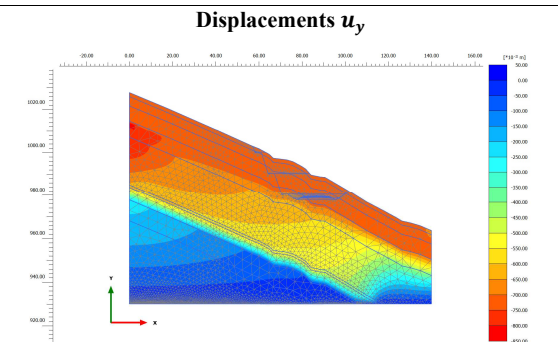


Figure 117 : Vertical Displacement (U_y) distribution at the Initial Construction Phase.

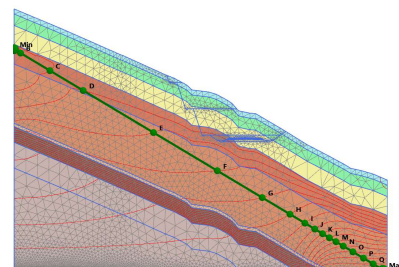


Figure 118 : Cross-section showing U_y displacement contours at the initial construction phase.

Maximum value = 0.000 m (Element 2508 at Node 67858)
Minimum value = -0.8100 m (Element 1010 at Node 20356)

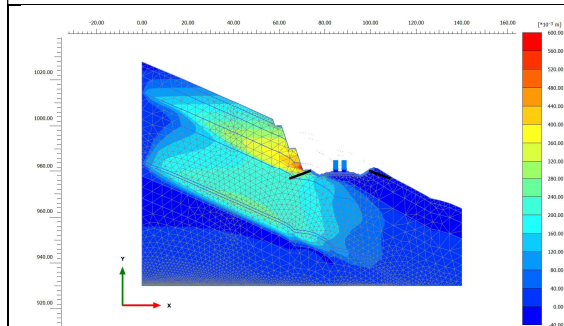


Figure 119 : Vertical Displacement (U_x) distribution at Phase 8.

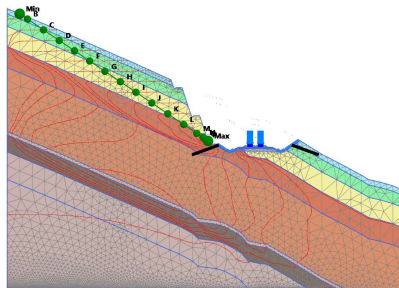


Figure 120 : Cross-section showing U_x displacement contours at Phase 8.

Maximum value = 0.5797 m (Element 1237 at Node 47750)
Minimum value = -0.01433 m (Element 5 at Node 20415)

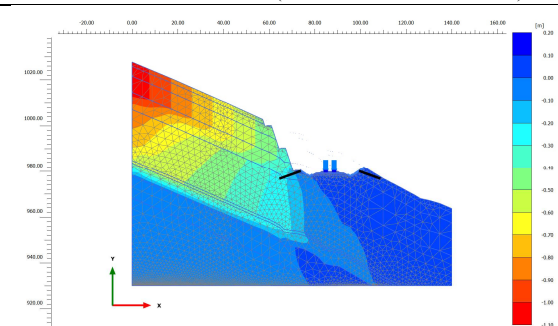


Figure 121 : Vertical Displacement (U_y) distribution at Phase 8.

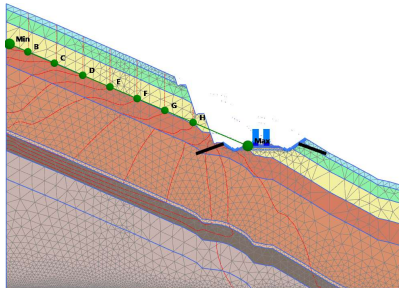


Figure 122 : Cross-section showing U_y displacement contours Phase 8.

Maximum value = 0.1183 m (Element 1267 at Node 49849)
Minimum value = -1.061 m (Element 213 at Node 20361)

Table 80 : Summary of Nodal Displacement Extremes (U_x and U_y) per Calculation Phase for Section 20.

Phase N :	Maximum value		Minimum value	
	u_x	u_y	u_x	u_y
Initial Phase	0.07104	0.000	-0.1544	-0.8100
1	0.07104	-0.07977	-0.1544	0.9307×10^{-3}
2	0.02191	0.01078	-2.579×10^{-3}	-0.08818
3	0.02524	0.07741	-0.02350	-0.6788
4	0.3011	0.08032	-0.01165	-0.7032
5	0.3263	0.08012	-9.713×10^{-3}	-0.7038
6	0.3263	0.07995	-0.01006	-0.7041
7	0.3262	0.07971	-0.01087	-0.7045
8	0.5797	0.1183	-0.01433	-1.061

IV.3.5.4.4. Evaluation of Stability and Deformation Curves

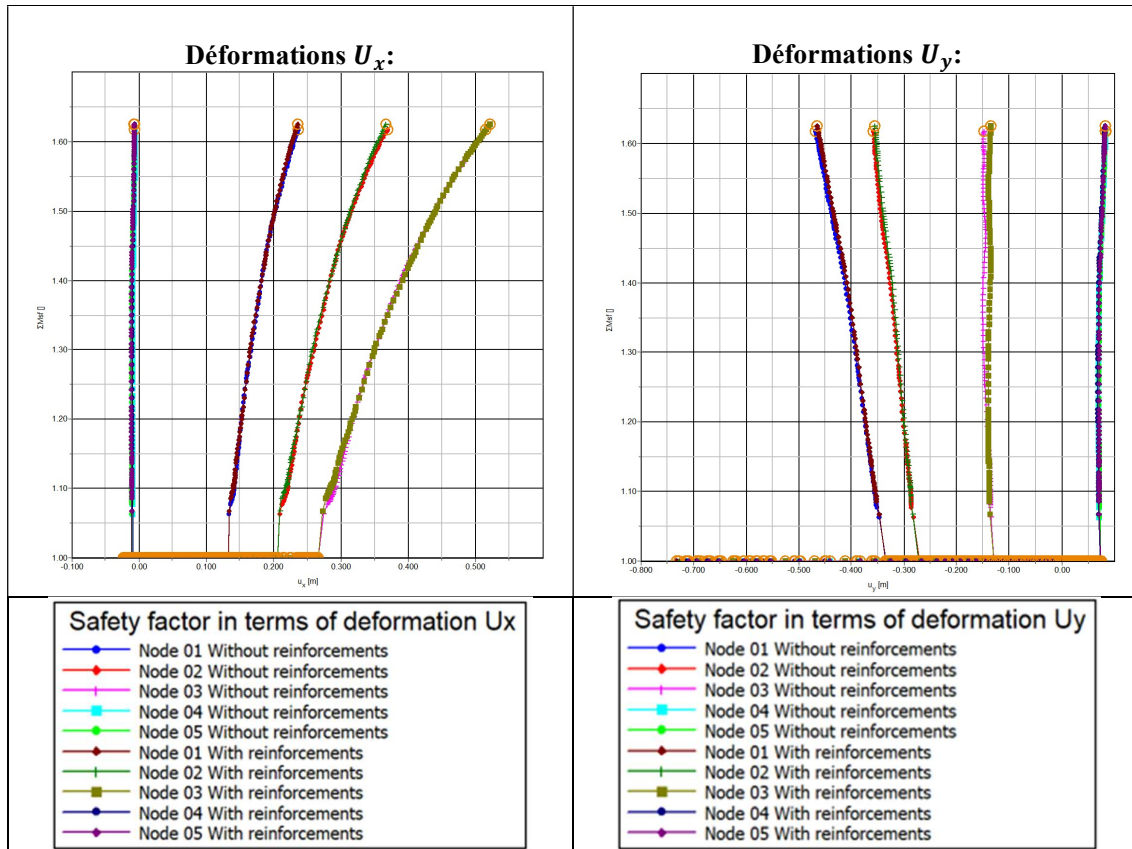


Figure 123 : Safety Factor vs. Deformations (U_x & U_y) for Section 20 (Reinforced and Unreinforced States).

The stability analysis demonstrates that the Factor of Safety (FoS) increased from 1.618 in the unsupported state to 1.625 following the implementation of reinforcements. Both values satisfy the design requirements ($FoS \geq 1.5$), confirming the overall stability of the railway cutting (trench) under the applied load of 80 kn/m^2 .

The marginal increase in FoS indicates that the excavated section exhibits inherent stability and structural adequacy. In this context, the reinforcement system acts as a proactive measure designed to mitigate localized deformations and ensure long-term structural integrity, rather than serving as a critical support for global stability.

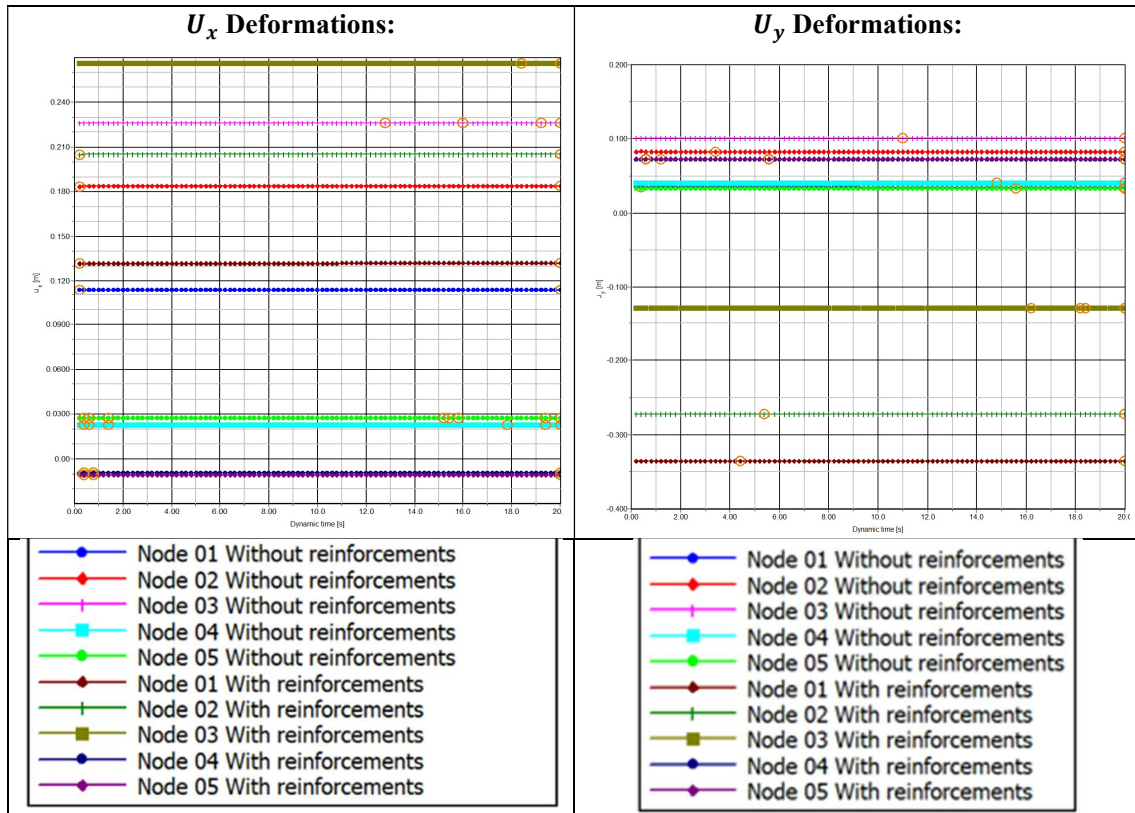


Figure 124 : Deformation (U_x) vs. Dynamic Time for Section 20 (With and Without Reinforcements).

The attached analytical chart illustrates the response of a structural system consisting of five nodes over time, comparing two scenarios: "without reinforcement" and "with reinforcement." The chart clearly demonstrates the behavior of the structural system under the applied loads, as all data paths exhibit a stable horizontal trend throughout the 20-second simulation period. This indicates that the system has reached a state of complete static equilibrium (steady state), with no dynamic oscillations observed during this period.

The results indicate that the introduction of reinforcement elements had a tangible and direct impact on the vertical displacement values (u_y) for the monitored nodes. A variation in node behavior was observed based on their position within the system. For nodes with positive displacement (ranging between 0.03 m and 0.10 m), the reinforcement contributed to a reduction in the overall displacement values, reflecting an increase in structural stiffness. Conversely, for nodes experiencing significant settlement (negative displacement reaching -0.34 m in the initial state), the reinforcement facilitated stress redistribution and reduced the settlement magnitude, leading to the stabilization of these nodes at new and improved displacement levels.

IV.3.5.4.5. Flow Failure Analysis

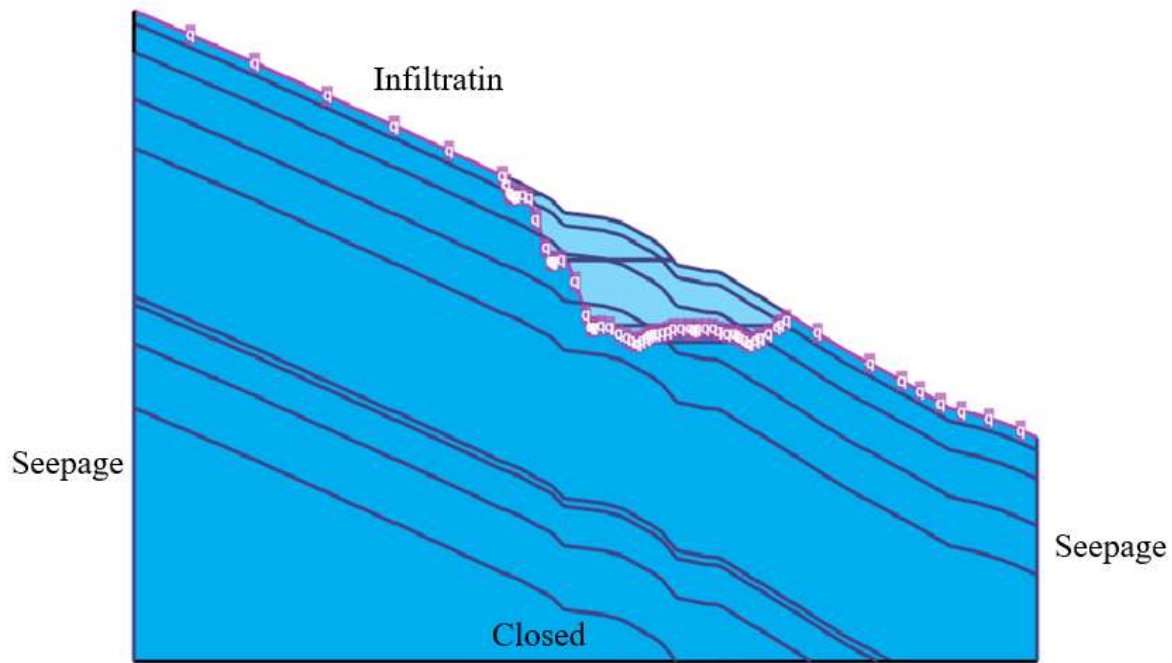


Figure 125 : Distribution of Seepage and Flow Boundary Conditions for Cross-Section 20.

Infiltration conditions used:

$$q: 0.1200 \text{ m/day} \quad \psi_{\text{Min}}: -1.000 \text{ m} \quad \psi_{\text{Max}}: 0.1000 \text{ m}$$

The two-dimensional analysis indicated that the model reached a state of flow failure. At This failure is manifested by the model's inability to achieve equilibrium (non-convergence) and the development of a critical slip surface extending through the railway excavations. The progressively divergent displacement values indicate that the generated excess pore water pressure has led to a reduction in the soil's shear strength, resulting in a loss of the slope's dynamic stability.

IV.3.5.5. PROFILE 22

IV.3.5.5.1. Numerical Simulation Stages

Tableau 7 : Summary of calculation phases and sequential construction steps for Section 22.

PHASE	DESCRIPTION
INITIAL PHASE	Initial balancing using Gravity Loading.
PHASE 1-3	Excavation processes (sequential removal of soil clusters).
PHASE 04	Installation of support/lining systems.
PHASE 05	Activation of roadbed layers (Ballast/Sub-ballast).
PHASE 06	Application of distributed surface load (80 KN/m^2).
PHASE 07	Safety Analysis (φ/c reduction).

IV.3.5.5.2. Definition of Monitoring Nodes

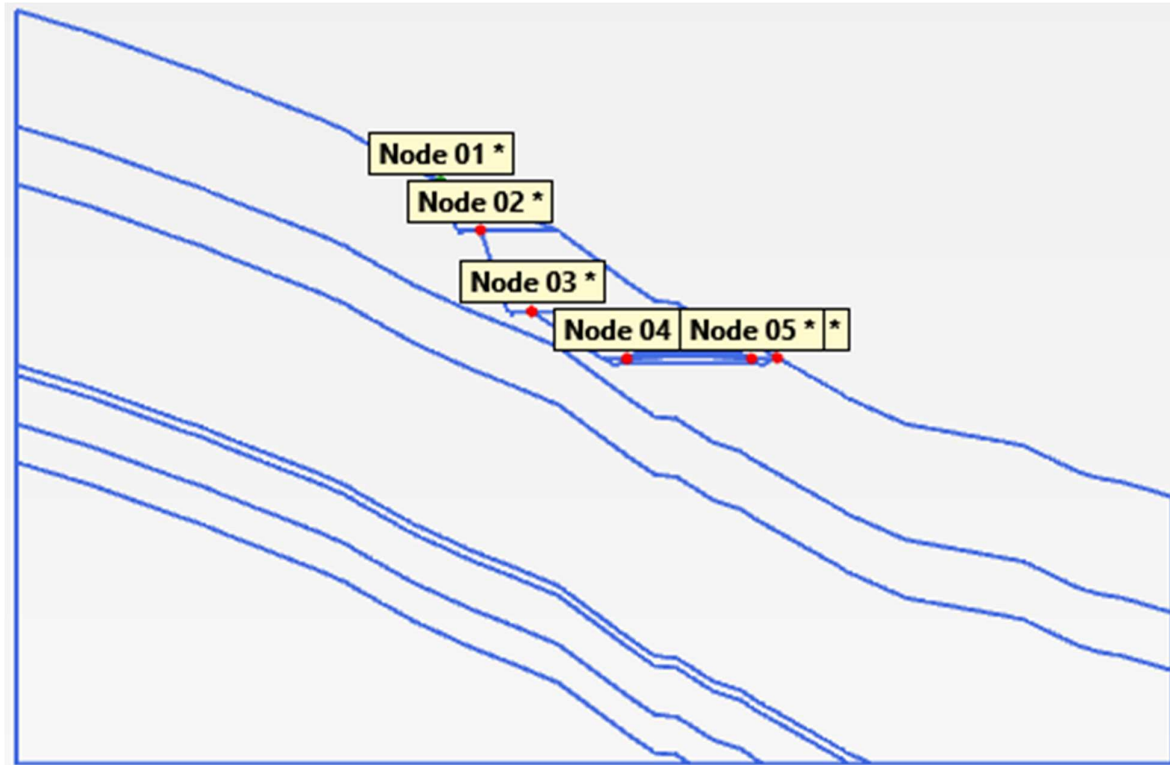


Figure 126 : Monitoring nodes distribution in Cross Section 22.

Table 81 : Localization and coordinates determination of the nodes for the cross-section 22.

Node	Side	X-coordinate	Y-coordinate	Geotechnical Location	Functional Description
Node 01	Left	51.35	1000	Upper Crest (Left)	Monitoring settlement at the left upper crest above road base.
Node 02	Left	56.2	994.6	Slope Face (Left)	Measuring lateral deformation of the left slope face.
Node 03	Left	62.38	984.7	Slope Toe (Left)	Monitoring foundation contact near road base on left side.
Node 04	Right	72.89	979	Slope Toe (Right)	Monitoring toe displacement on the right side adjacent to road base.
Node 05	Right	88.97	979	Slope Face (Right)	Observing deformation of the right slope face.

IV.3.5.5.3. Evaluation of Soil Displacement Response

U_x Deformations :

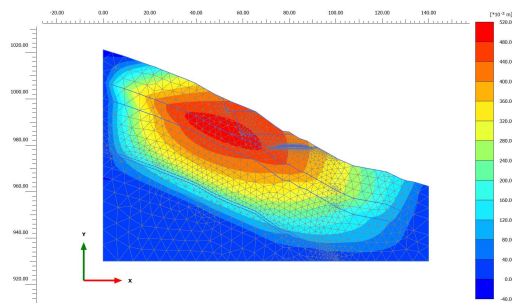


Figure 127 : Horizontal Displacement (U_x) distribution at the Initial Construction Phase.

Maximum value = 0.5048 m (Element 262 at Node 6527)
Minimum value = -0.01656 m (Element 144 at Node 22)

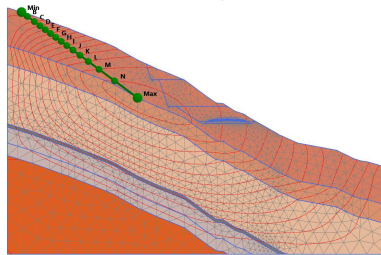


Figure 128 : Cross-section showing U_x displacement contours at the initial construction phase.

Maximum value = 0.5017×10^{-3} m (Element 2342 at Node 14131)
Minimum value = -0.3932×10^{-3} m (Element 35 at Node 1422)

U_y Deformations :

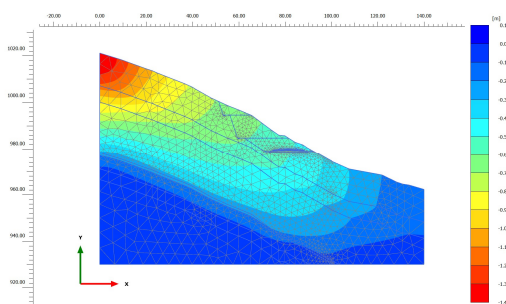


Figure 129 : Vertical Displacement (U_y) distribution at the Initial Construction Phase.

Maximum value = 0.000 m (Element 1710 at Node 25143)
Minimum value = -1.370 m (Element 164 at Node 4)

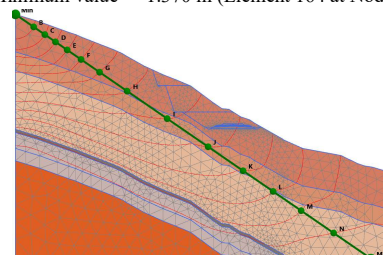


Figure 130 : Cross-section showing U_y displacement contours at the initial construction phase.

Maximum value = 0.000 m (Element 2456 at Node 22489)
Minimum value = -4.651×10^{-3} m (Element 376 at Node 432)

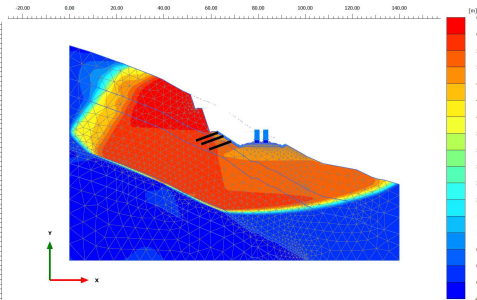


Figure 131 : Horizontal Displacement (U_x) distribution at Phase 7.

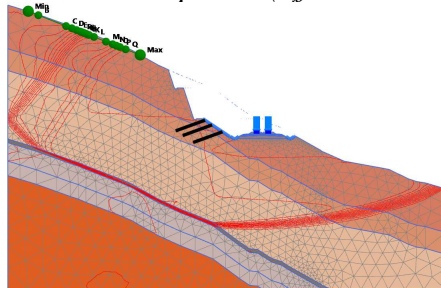


Figure 132 : Cross-section showing U_x displacement contours at Phase 7.

Maximum value = 6.268 m (Element 330 at Node 2586)
Minimum value = -0.08425 m (Element 144 at Node 21)

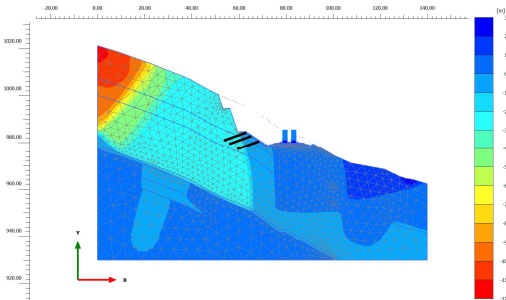


Figure 133 : Vertical Displacement (U_y) distribution at Phase 7.

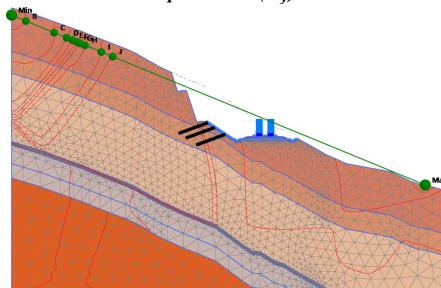


Figure 134 : Cross-section showing U_x displacement contours at Phase 7.

Maximum value = 2.617 m (Element 185 at Node 24569)
Minimum value = -11.08 m (Element 164 at Node 6)

Table 82 : Summary of Nodal Displacement Extremes (U_x And U_y) per Calculation Phase for Section 22.

Phase N :	Maximum value		Minimum value	
	u_x	u_y	u_x	u_y
Initial Phase	0.5048	0.000	-0.01656	-1.370
1	1.437×10^{-3}	0.01100	-2.087×10^{-3}	-0.01598×10^{-3}
2	0.02078	0.03338	-0.02022	-3.138×10^{-3}
3	0.03117	0.04269	-0.03365	-0.01170
4	0.03159	0.04208	-0.03385	-0.01205
5	0.03195	0.04051	-0.03381	-0.01243
6	0.03273	0.03802	-0.03366	-0.01364
7	6.268	2.617	-0.08425	-11.08

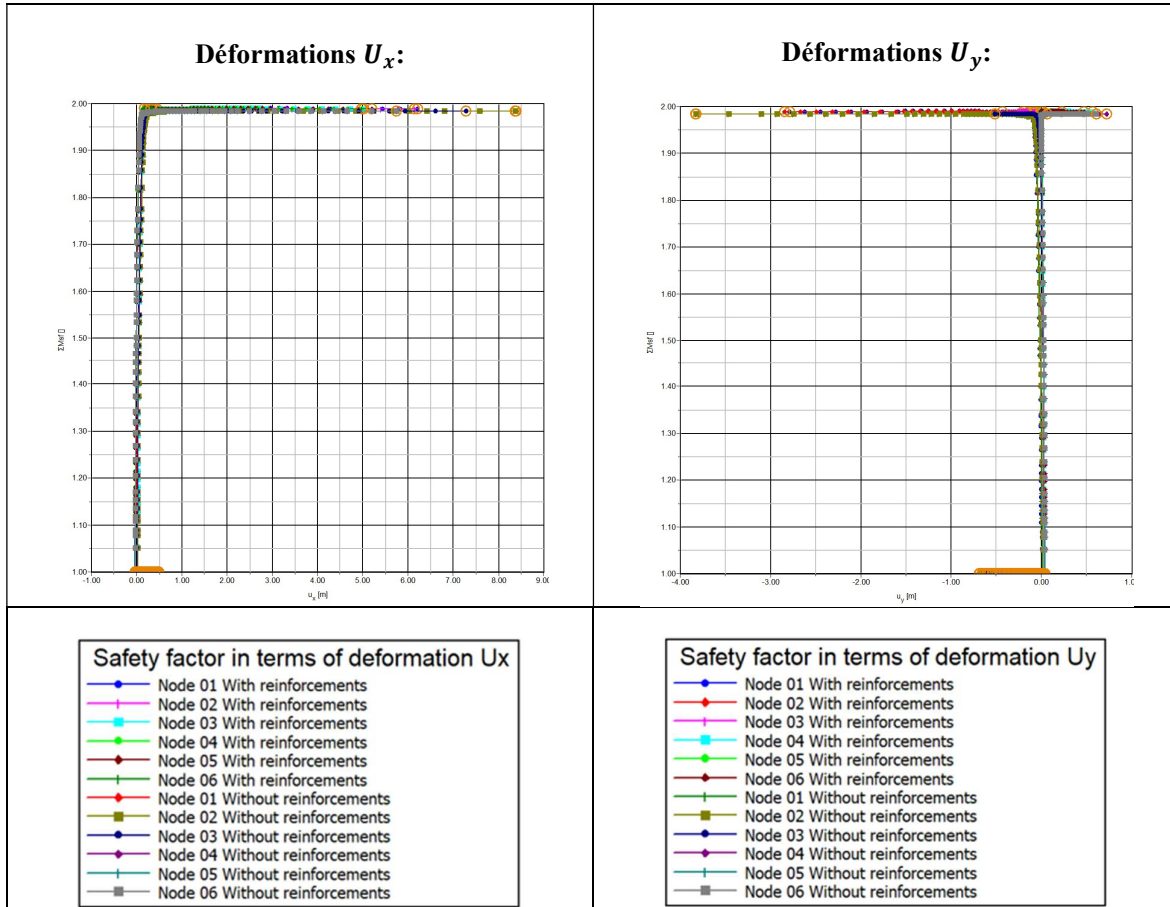


Figure 135 : Safety Factor vs. Deformations (U_x & U_y) for Section 22 (Reinforced and Unreinforced States).

The displacement-based stability analysis indicates that the Factor of Safety (FoS) slightly increased from 1.984 in the unsupported condition to 1.990 following the application of reinforcements. This marginal improvement in the global safety factor reflects the positive contribution of the reinforcement system to the soil-structure interaction, effectively redistributing stresses within the excavated mass. Despite the modest numerical increase, the structural system exhibits high resilience under the applied loads. The convergence observed in the deformation patterns of the monitored nodes suggests that the reinforcements successfully restrain excessive soil displacement, facilitating a controlled geotechnical response. Consequently, these reinforcements enhance local performance and mitigate time-dependent deformations, thereby ensuring the long-term structural integrity and stability of the excavated section.

IV.3.5.5.4. Dynamic deformation results

A. U_x Deformations:

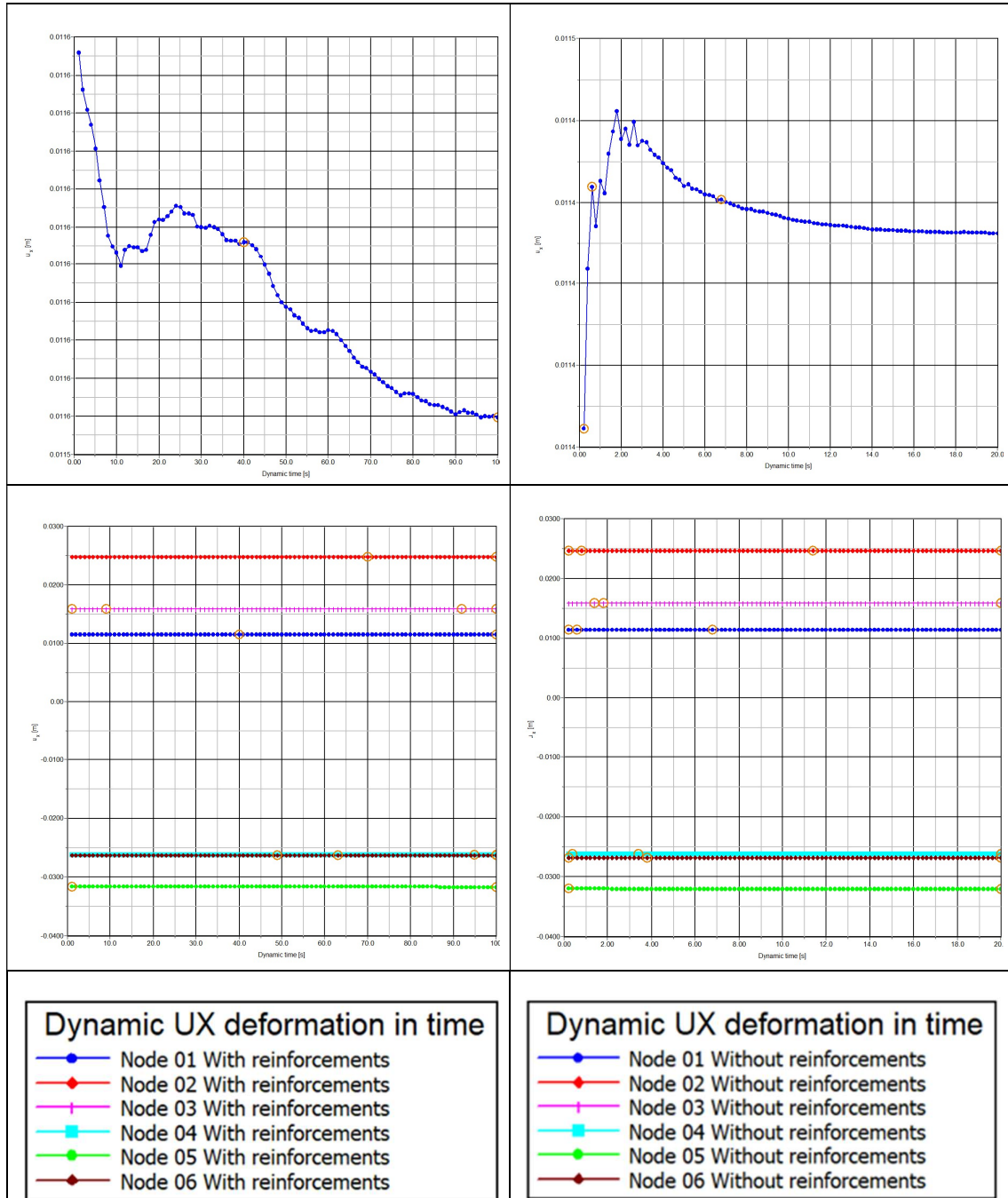


Figure 136 : Deformation (U_x) vs. Dynamic Time for Section 22 (With and Without Reinforcements).

B. U_y Deformations:

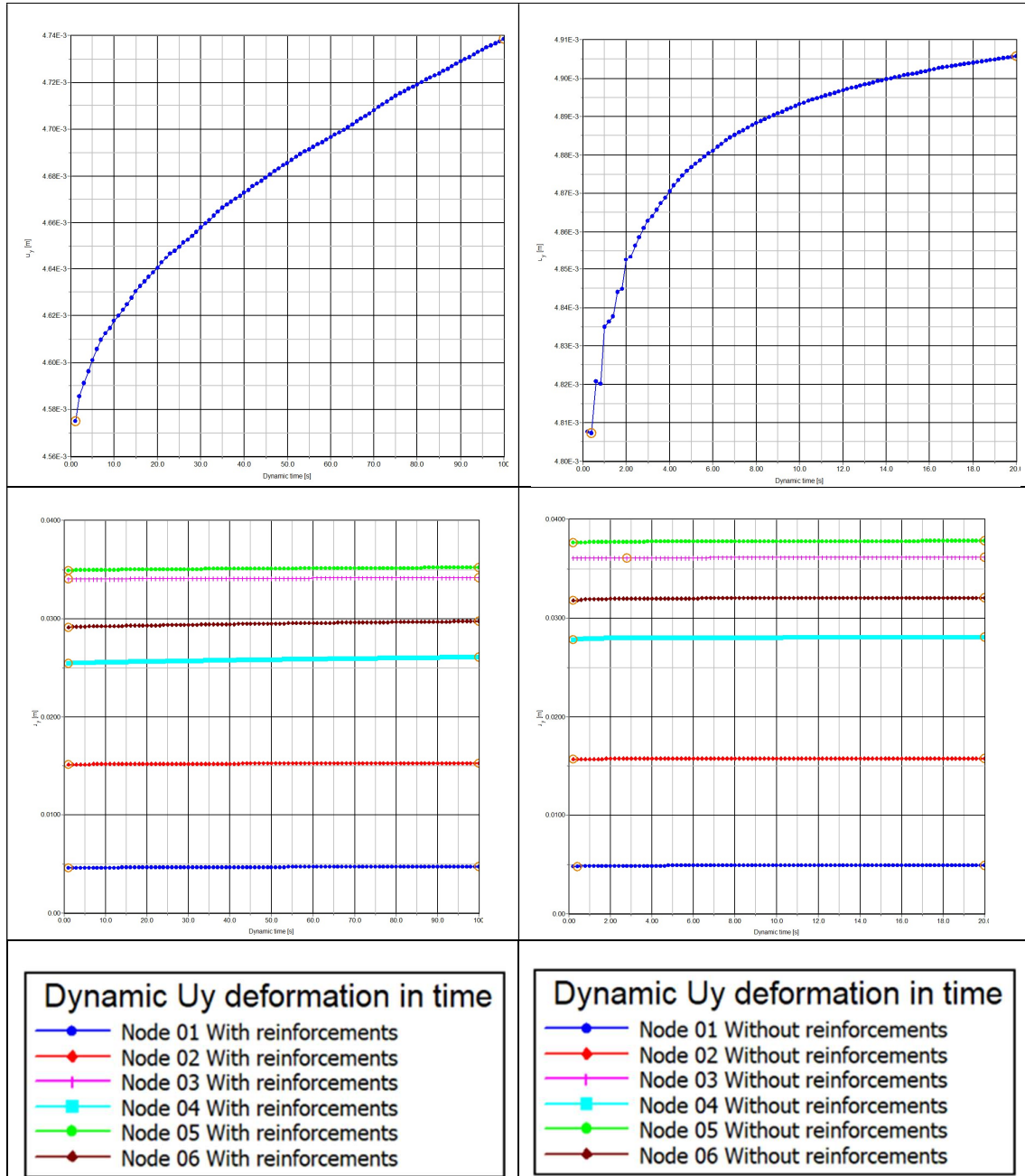


Figure 137 : Deformation (U_y) vs. Dynamic Time for Section 22 (With and Without Reinforcements).

Simulation data indicates that the structural system reached a state of static equilibrium throughout the monitoring period, characterized by stable displacement trends and no dynamic oscillations. The introduction of reinforcement elements altered nodal displacement values, reflecting a change in system stiffness. For nodes subjected to vertical settlement (negative displacement), reinforcement facilitated stress redistribution, reducing the magnitude of settlement compared to the unreinforced state. Numerically, horizontal displacement (u_x) at Node 01 stabilized at approximately 0.0116 m with reinforcement versus 0.0114 m without. Reinforcement reduced the magnitude of negative vertical displacements (settlement) across affected nodes, stabilizing them at lower absolute values relative to the initial state. In conclusion, the reinforcement elements functioned to redistribute stresses and adjust displacement values at the monitored nodes, with the data showing a clear correlation between the application of reinforcement and changes in the structural deformation profile.

IV.3.5.5.5. Flow Failure Analysis

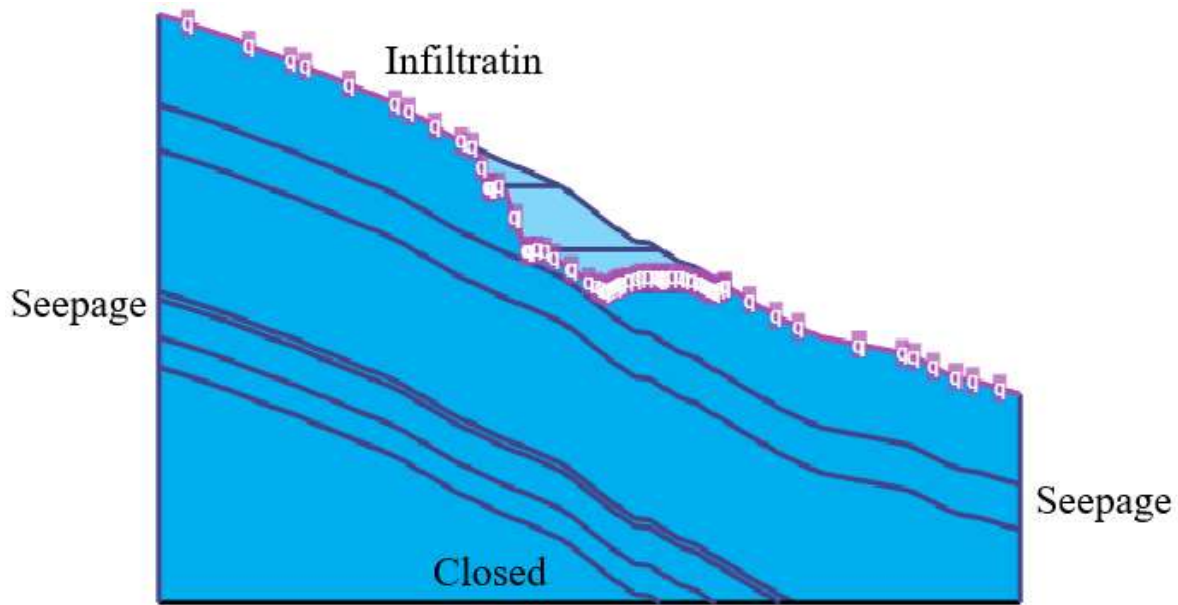


Figure 138 : Distribution of Seepage and Flow Boundary Conditions for Cross-Section 22.

Infiltration conditions used :

$$q: 0.1200 \text{ m/day} \quad \psi_{\text{Min}}: -1.000 \text{ m} \quad \psi_{\text{Max}}: 0.1000 \text{ m}$$

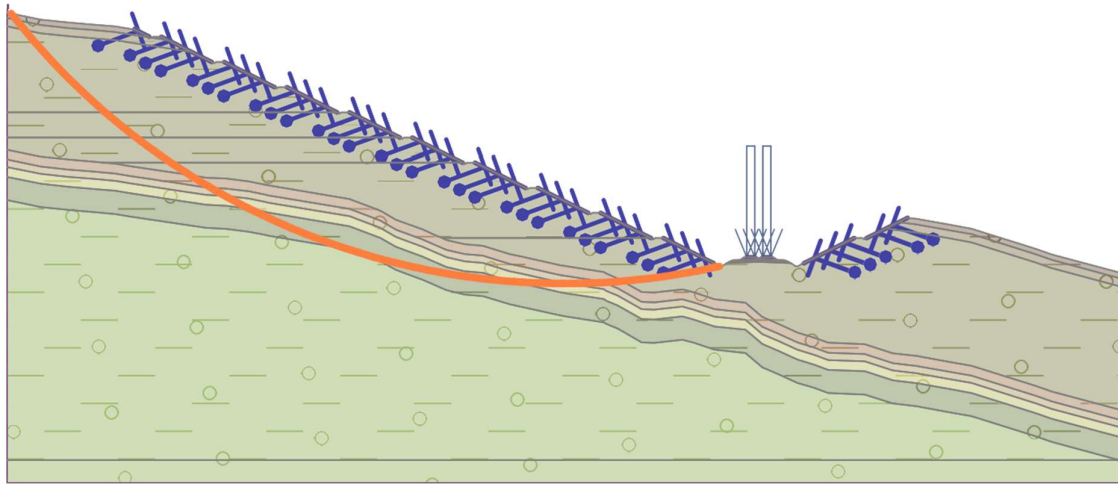
The two-dimensional analysis indicated that the model reached a state of flow failure at time step 61 ($t \approx 0.0026$ days), where the maximum excess pore water pressure (p_{max}) attained a value of approximately 108.8 kN/m^2 . This failure is manifested by the model's inability to achieve equilibrium (non-convergence) and the development of a critical slip surface extending through the railway excavations. The progressively divergent displacement values (not shown in the table, but inferred from the pore pressure peak) indicate that the generated excess pore water pressure has led to a reduction in the soil's shear strength, resulting in a loss of the slope's dynamic stability.

- This indicates the need to protect the area from water seepage by using impermeable materials, such as geomembranes, and by diverting water away from the site through appropriate drainage systems.

IV.4. Numerical Modeling using GEO5

In this stage, we will perform an integrated digital modeling process for the Tenoukla trench. To do so, we rely on GEO5 – a specialized geotechnical software that operates on the Limit Equilibrium (LE) approach for slope stability analysis and for evaluating both circular and polygonal failure surfaces using appropriate material models. This process is applied to the selected cross-sections (05, 09, 15, 20, 22), following the same stages previously outlined for each section

The results are as follows:



Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop: $FS = 3.35 > 1.50$ **ACCEPTABLE**

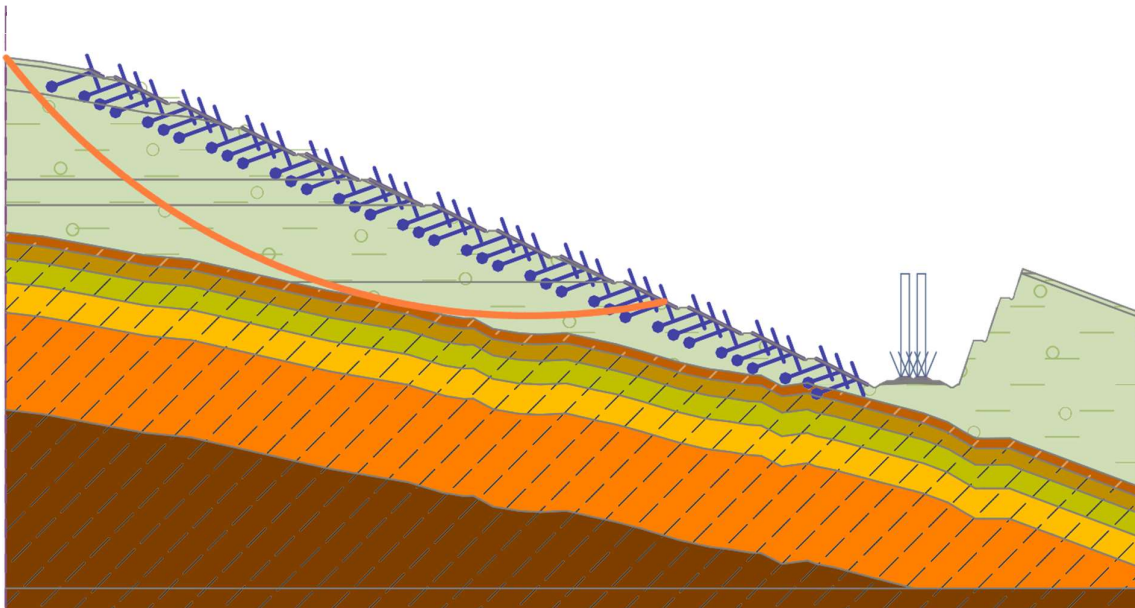
Fellenius / Petterson: $FS = 3.27 > 1.50$ **ACCEPTABLE**

Spencer: $FS = 3.34 > 1.50$ **ACCEPTABLE**

Jambu: $FS = 3.34 > 1.50$ **ACCEPTABLE**

Morgenstern-Price: $FS = 3.34 > 1.50$ **ACCEPTABLE**

Figure 139 : Geometric model and failure surface for Profile 05 using GEO5 (limit equilibrium analysis).



Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop: $FS = 4.13 > 1.50$ **ACCEPTABLE**

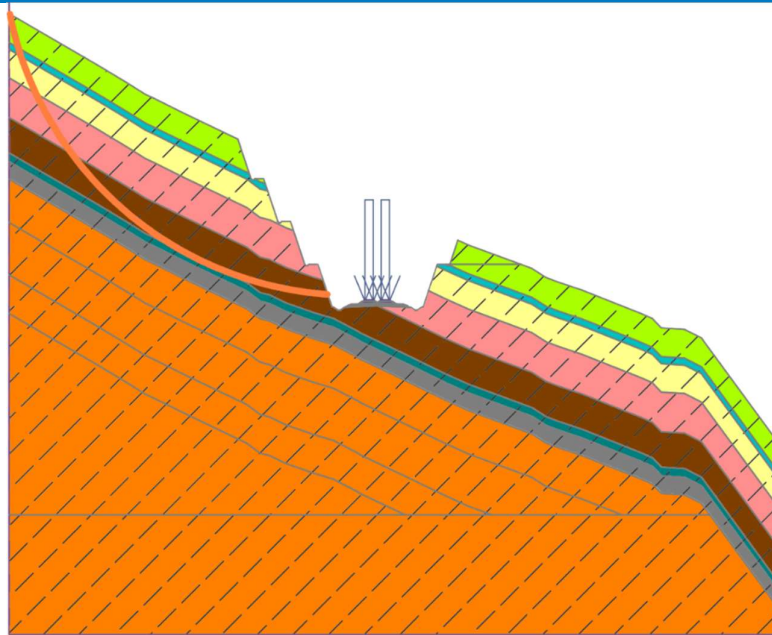
Fellenius / Petterson: $FS = 4.04 > 1.50$ **ACCEPTABLE**

Spencer: $FS = 4.12 > 1.50$ **ACCEPTABLE**

Jambu: $FS = 4.13 > 1.50$ **ACCEPTABLE**

Morgenstern-Price: $FS = 4.13 > 1.50$ **ACCEPTABLE**

Figure 140 : Geometric model and failure surface for Profile 09 using GEO5 (limit equilibrium analysis).



Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop: FS = 8.77 > 1.50 **ACCEPTABLE**

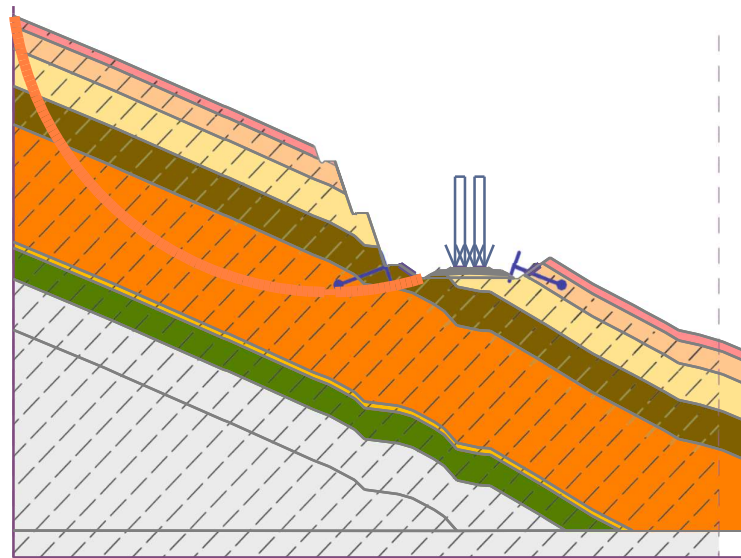
Fellenius / Petterson: FS = 9.77 > 1.50 **ACCEPTABLE**

Spencer: FS = 9.56 > 1.50 **ACCEPTABLE**

Jambu: FS = 9.72 > 1.50 **ACCEPTABLE**

Morgenstern-Price: FS = 9.72 > 1.50 **ACCEPTABLE**

Figure 141 : Geometric model and failure surface for Profile 15 using GEO5 (limit equilibrium analysis).



Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop: FS = 3.07 > 1.50 **ACCEPTABLE**

Fellenius / Petterson: FS = 5.41 > 1.50 **ACCEPTABLE**

Spencer: **The solution has not been found.**

Jambu: FS = 5.36 > 1.50 **ACCEPTABLE**

Morgenstern-Price: FS = 5.36 > 1.50 **ACCEPTABLE**

Figure 142 : Geometric model and failure surface for Profile 20 using GEO5 (limit equilibrium analysis).

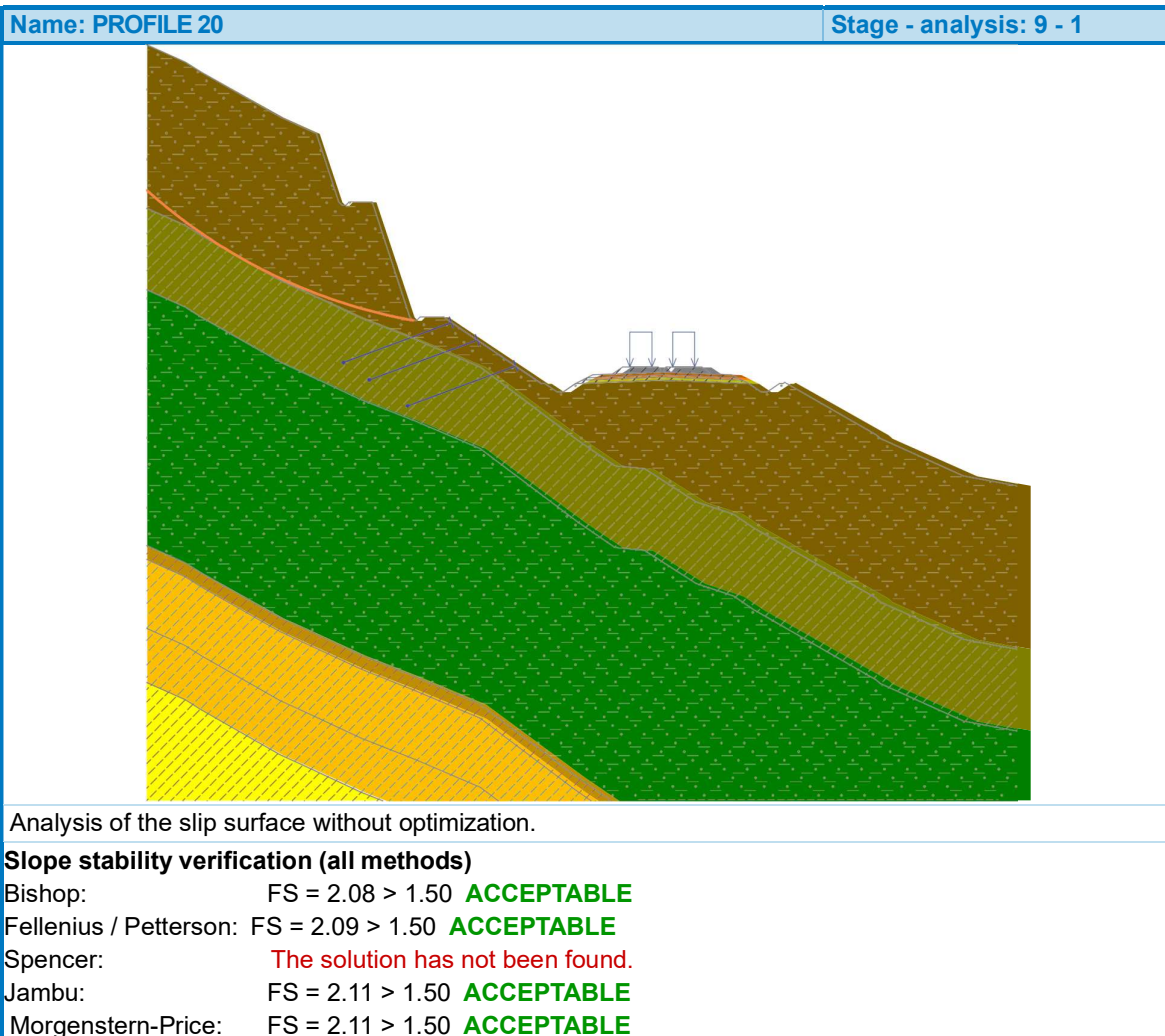


Figure 143 : Geometric model and failure surface for Profile 22 using GEO5 (limit equilibrium analysis).

IV.5. Concluding Remarks on the 2D Stability Assessment

The safety factor for all studied sections exceeds 1.5, confirming the overall stability of the project under static design conditions. The previously observed failure of sections 5, 9, 20, and 22 under rainfall, as well as the relative stability of section 15, can be clearly explained by the geotechnical and hydrological data presented in the Tenoukla report.

Deformation Response to Sequential Construction Stages:

- The Initial Phase represents a purely geostatic equilibrium condition under the self-weight of the soil, with neither excavation nor external loading applied. The displacement values obtained at this stage—particularly the significant negative vertical settlements—serve as a reference baseline. These initial values are subsequently subtracted from the results of the following phases, either automatically by the numerical software (through displacement resetting) or manually, in order to determine the net additional deformations induced exclusively by the construction activities.
- Across all profiles, particularly Profiles 05 and 09, a steady and progressive increase in both horizontal and vertical displacement magnitudes is observed as the excavation stages advance.

- Following the installation of the support and lining systems and the activation of the railway track layers (ballast and sub-ballast), a clear stabilization of the displacement values becomes evident, with certain cases even exhibiting a slight reduction. For example, in Profile 05, the maximum vertical displacement ($u_y (max)$) decreases from 0.1604 m to 0.1512 m.
- The application of the operational surface load induces only a marginal increase in displacements compared with the preceding stage. This negligible increment indicates that the adopted design is capable of accommodating the service loads while maintaining the system response within the elastic or near-elastic range.
- In all profiles, the substantial increase in displacement values observed during the Safety Analysis phase represents an artificial and intentionally induced response inherent to this stage. Its underlying mechanism is based on the progressive reduction of the shear strength parameters, namely the cohesion (c) and the internal friction angle (φ). This systematic strength reduction is deliberately implemented to drive the numerical model toward failure, thereby enabling the determination of the critical safety factor corresponding to the onset of instability and eventual collapse.

Geotechnical Facies and Recommended Protection Systems:

The excavation and investigation campaign (8 boreholes, seismic refraction profiles, and laboratory tests) revealed the presence of three distinct geotechnical facies along the open trench:

- Mio-Pliocene clays – high cohesion (up to 275 KPa) and friction angle exceeding 20° in some locations (e.g., borehole SDC-03).
- Turonian limestone – hard and massive.
- Trias argillaceous clays – rich in gypsum (up to 12.35% at depths >18 m), with a high sulphate concentration (SO₄ up to 6.89%), making them aggressive and susceptible to dissolution upon contact with water.

Consequently, the site needs effective drainage (gutters/weep holes – caniveaux/barbacanes) and robust waterproofing, specifically: geomembrane over the Trias clays, and shotcrete on slopes to prevent erosion. Without these, even rare heavy rainfall would raise pore pressures and dissolve gypsum in the Trias clays, causing internal erosion and dual cohesion loss (via ingress and dissolution). This justifies the adopted reinforcement (anchors and welded mesh), which redirects flow and limits pressure but does not fully stop infiltration. However, it remains insufficient without prior drainage and waterproofing.

IV.6. 3D Numerical Modeling

Modeling was performed through a set of steps:

IV.6.1. Model Geometry Construction

The generation of the site's topography represents a critical step in ensuring the numerical model's accuracy. This process, conducted within the Autodesk 3ds Max environment, focuses on the precise geometric definition of the slope and the sequential excavation plan. By creating a complex 3D terrain mesh and refining the excavation geometry, the model provides an accurate representation of the site's final configuration and intended construction volumes. These finalized 3D geometries are subsequently exported in OBJ format to serve as the geometric foundation for the stability and displacement analyses within PLAXIS 3D, ensuring seamless interoperability between modeling and analysis platforms.

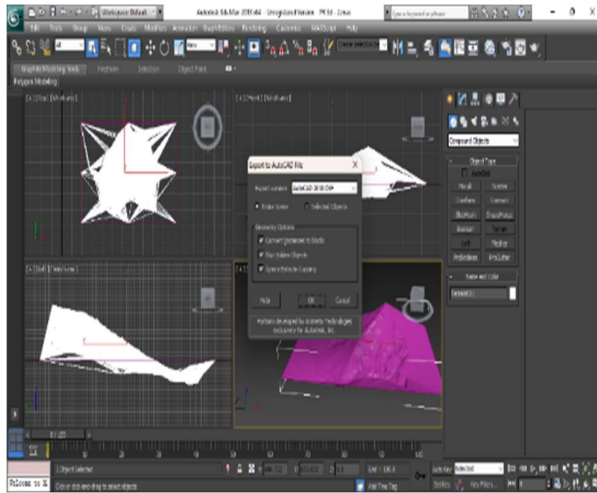


Figure 144 : Creation and Export of the 3D Terrain Model from Autodesk 3ds Max.

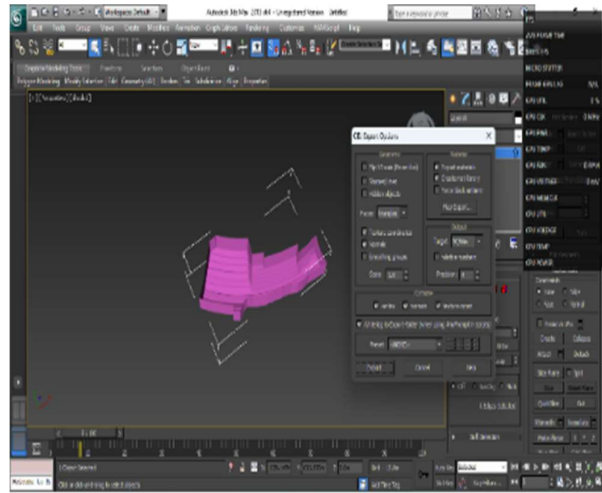


Figure 145 : Configuration of obj export parameters for the 3D slope and excavation model.

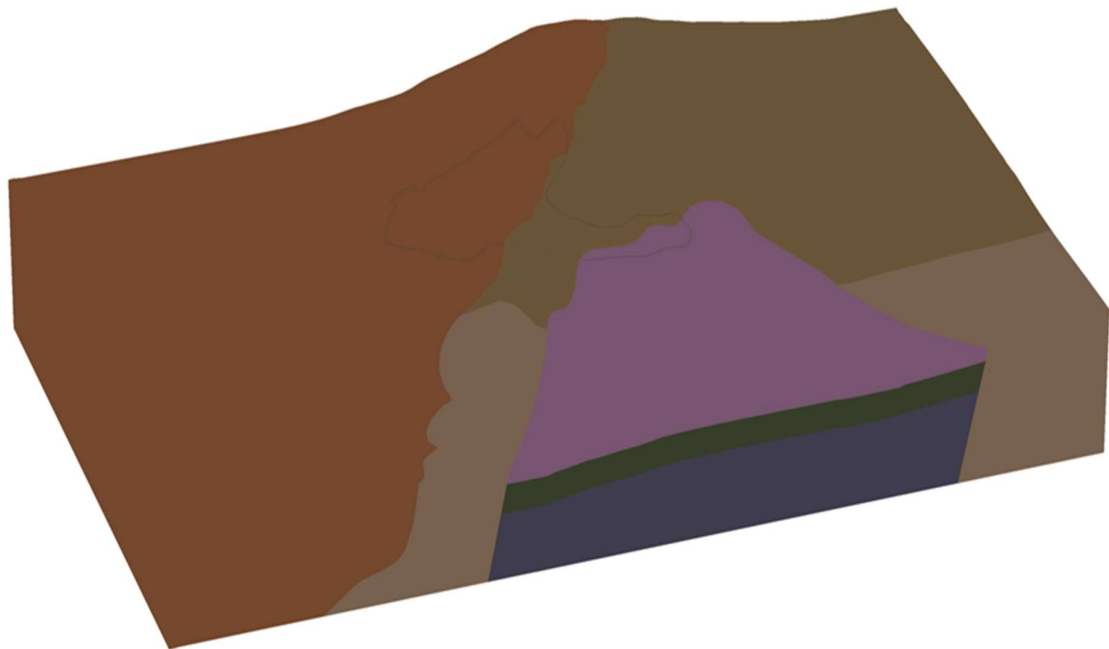


Figure 146 : The full volumetric model and stratification of the heterogeneous geotechnical layers.

IV.6.2. Input of Material Properties

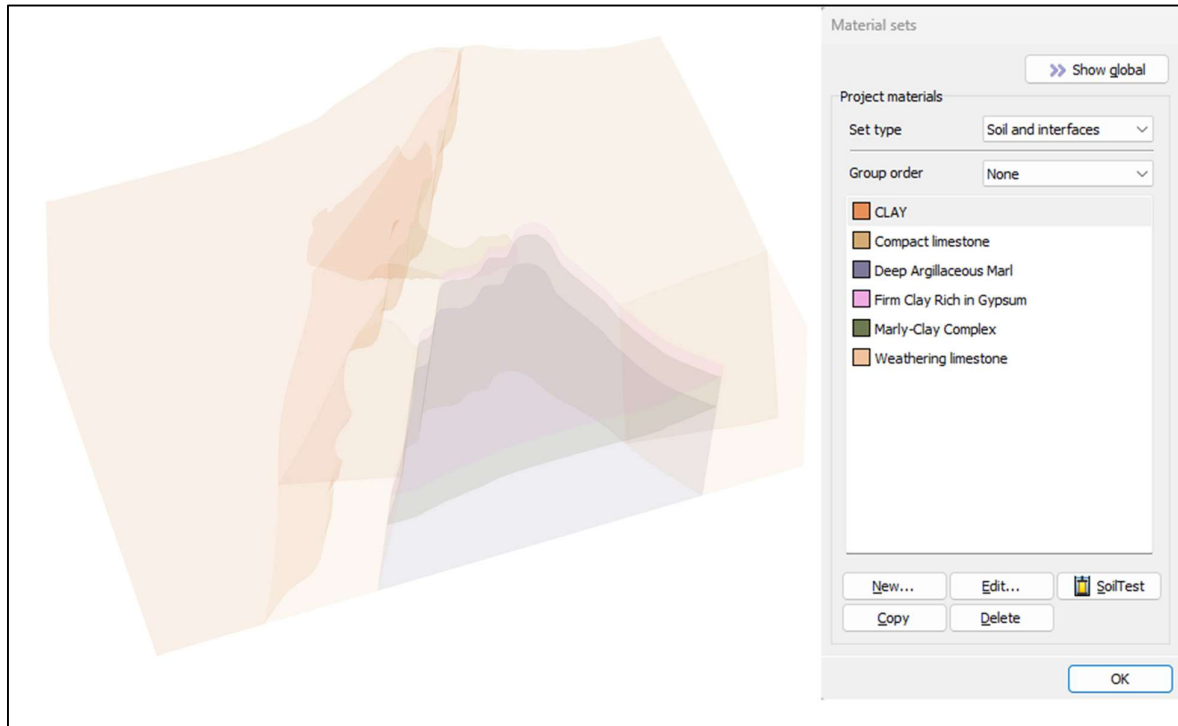


Figure 147 : geotechnical material sets definition.

IV.6.3. Mesh Generation

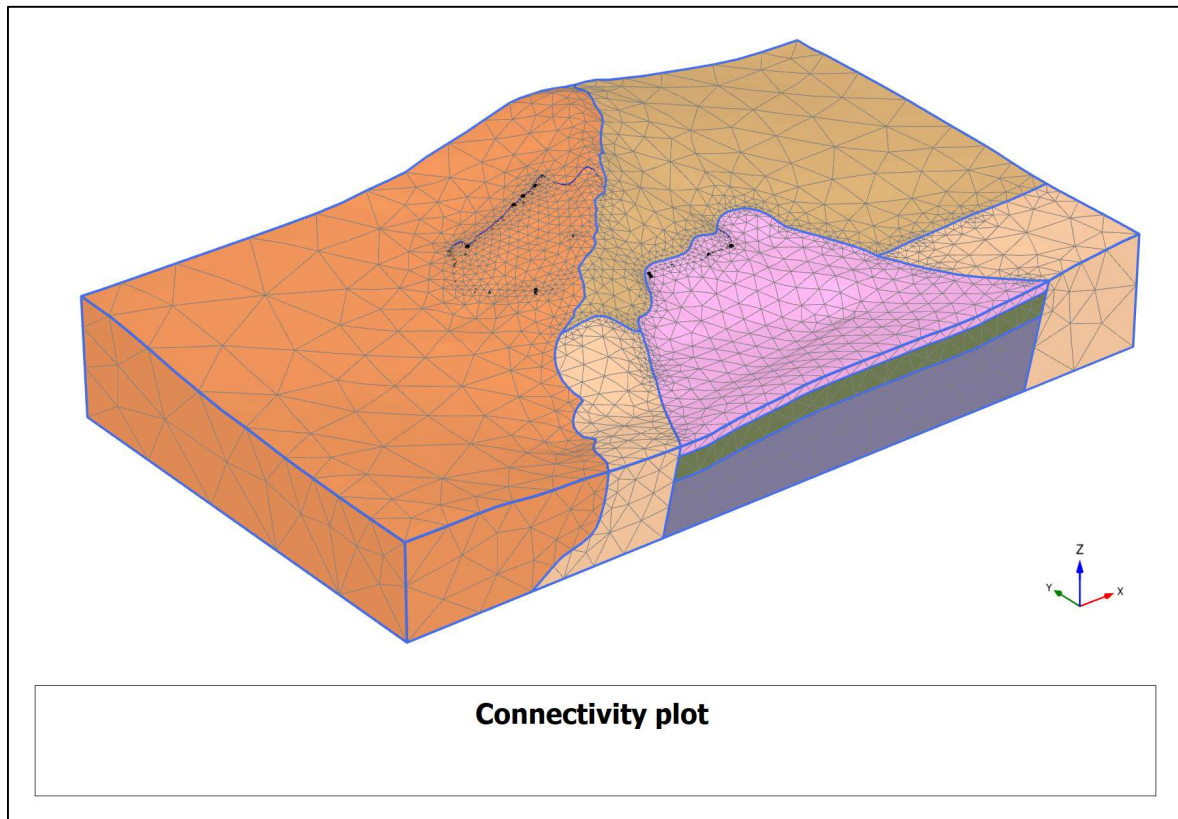


Figure 148 : Finite element mesh generation and connectivity plot of the 3D slope model.

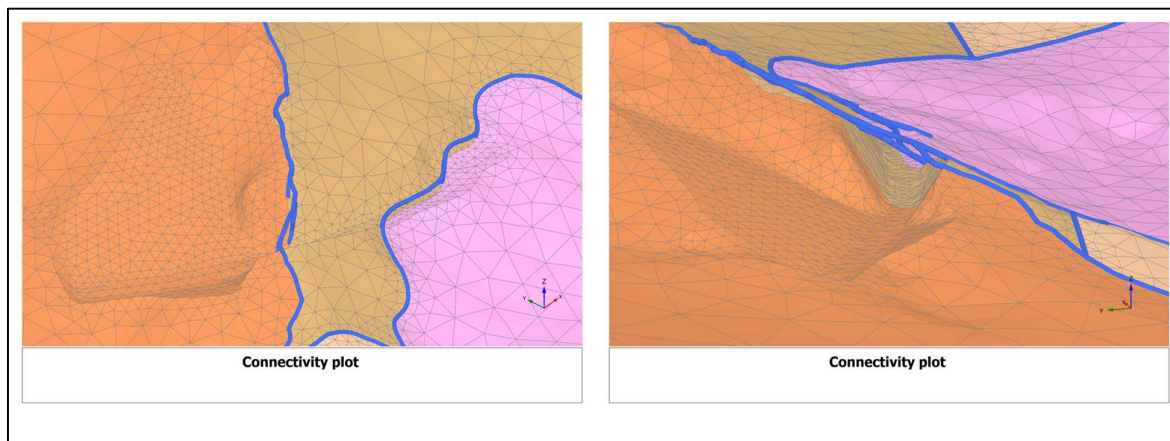


Figure 149 : Detailed view of the connectivity plot for the slope faces.

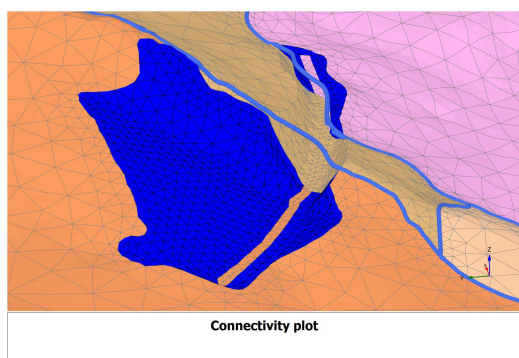


Figure 150 : Connectivity plot of the shotcrete plate support and surrounding slope interfaces.

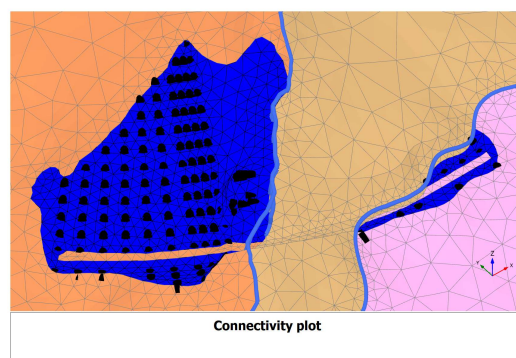


Figure 151 : 3D finite element mesh details of the structural support elements (Plates and Anchors) on the slope faces.

IV.6.4. Definition of Monitoring Nodes

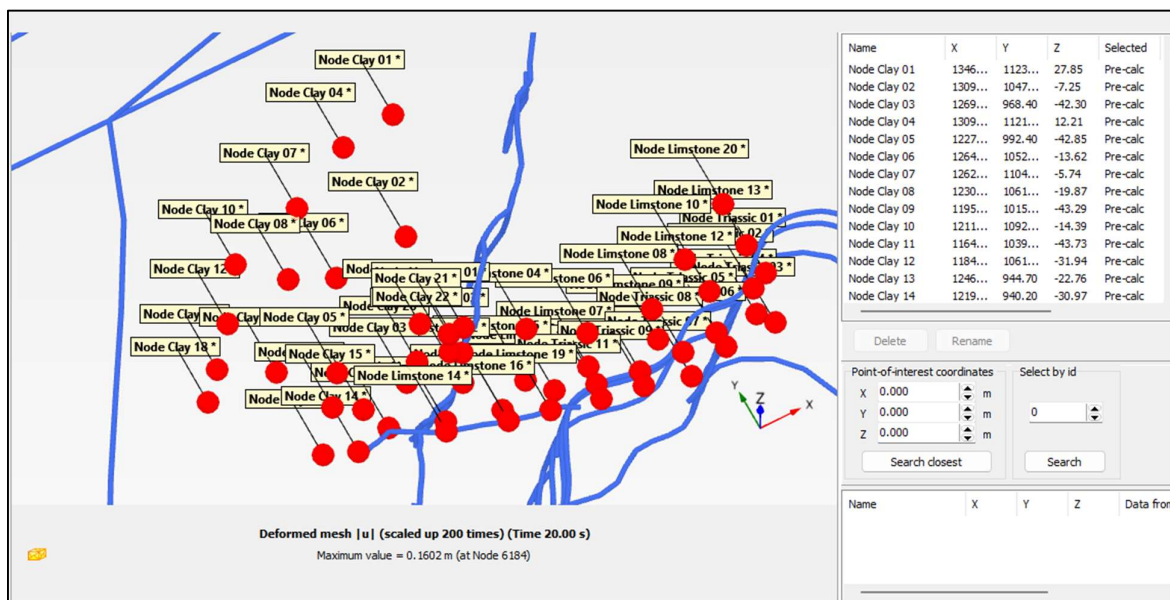


Figure 152 : spatial distribution of monitored nodes within the 3D finite element model.

Table 83 : Classification of Monitoring Nodes by Lithological Unit.

LITHOLOGICAL UNIT	NODE IDENTIFICATION
CLAY	Node Clay
LIMESTONE	Node Limestone
TRIASSIC	Node Triassic

IV.6.5.Numerical Simulation Stages

Table 84 : Summary of 3D model calculation phases and construction sequence.

PHASE	DESCRIPTION
INITIAL PHASE	Initial balancing using Gravity Loading.
PHASE 1	Baseline Deformation Phase
PHASE 2	Excavation processes (sequential removal of soil clusters).
PHASE 3	Installation of support/lining systems.
PHASE 4	Application of distributed surface load (80 KN/m^2).
PHASE 5	Safety Analysis (ϕ/c reduction).

IV.6.6.Evaluation of Soil Displacement Response

Table 85 : The horizontal displacement contour maps (u_x) scaled up 50.0 times.

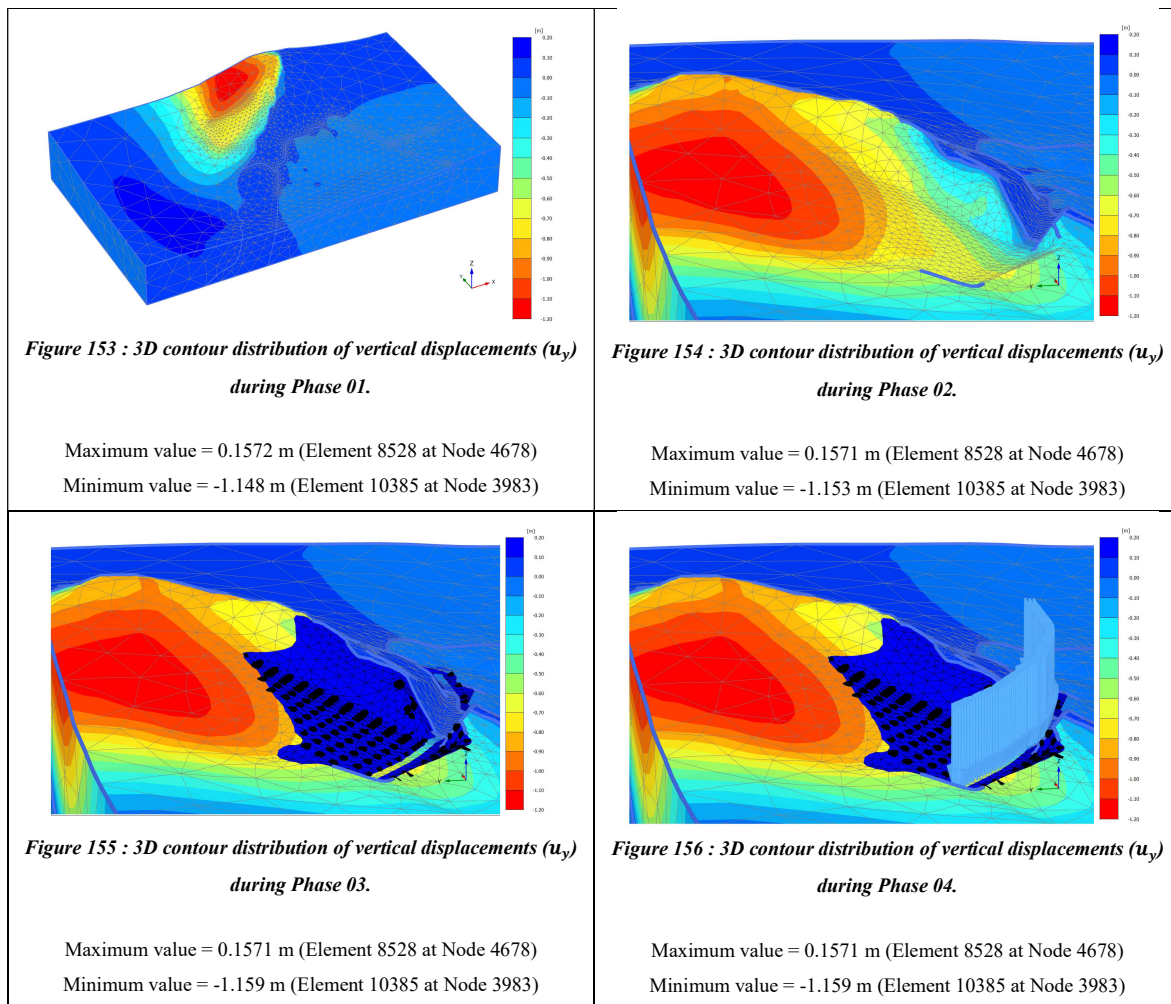


Table 86 : Lateral displacement contour maps (u_y) scaled up 50.0 times.

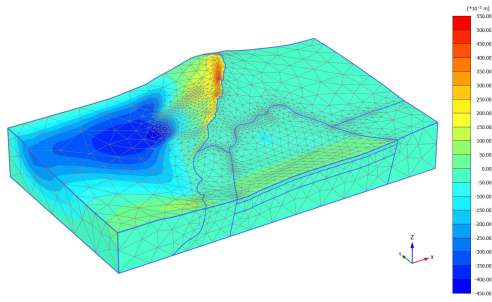


Figure 157 : 3D contour distribution of vertical displacements (u_y) during Phase 01.

Maximum value = 0.5356 m (Element 11509 at Node 4127)
Minimum value = -0.4170 m (Element 9819 at Node 4378)

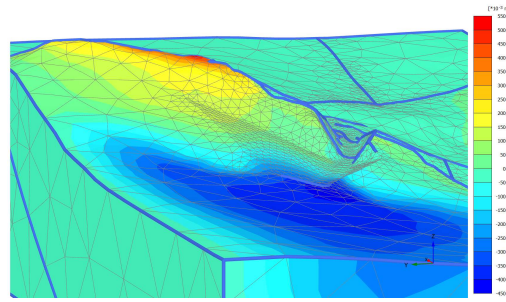


Figure 158 : 3D contour distribution of vertical displacements (u_y) during Phase 02.

Maximum value = 0.5453 m (Element 11509 at Node 4127)
Minimum value = -0.4226 m (Element 9819 at Node 4378)

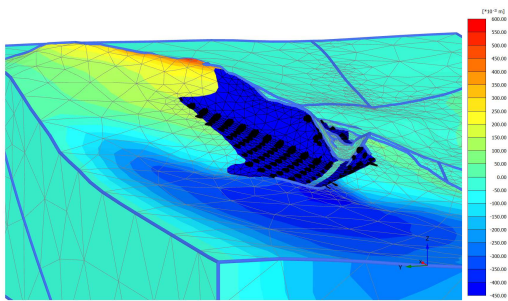


Figure 159 : 3D contour distribution of vertical displacements (u_y) during Phase 03.

Maximum value = 0.5533 m (Element 11509 at Node 4127)
Minimum value = -0.4228 m (Element 9819 at Node 4378)

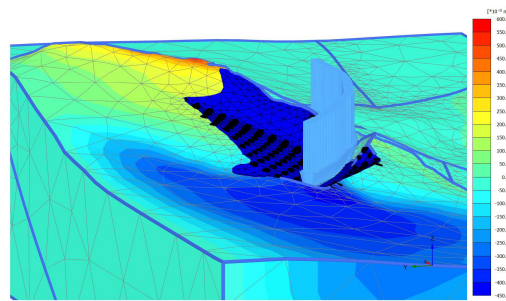
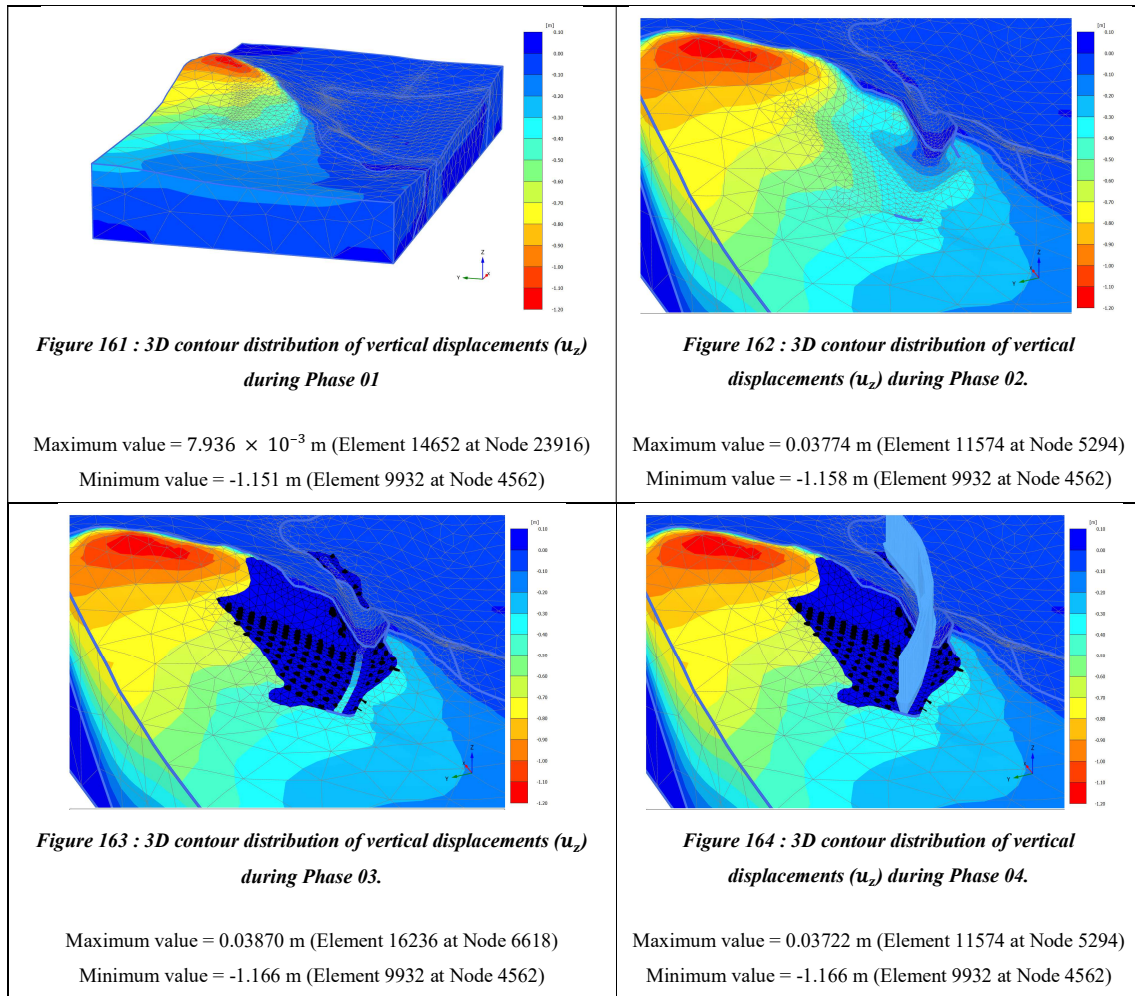


Figure 160 : 3D contour distribution of vertical displacements (u_y) during Phase 04.

Maximum value = 0.5533 m (Element 11509 at Node 4127)
Minimum value = -0.4228 m (Element 9819 at Node 4378)

Table 87 : Vertical displacement contour maps (u_z) scaled up 50.0 times.



Soil removal led to the release of confining stresses, causing an immediate activation of slope movement and an increase in lateral shear deformations (u_y) around the excavation edges.

The installation of piles and anchors successfully reduced the deformations locally at the surface.

Upon applying the additional surface load (80 KN/m^2), the results showed complete stability in deformation values, which engineering-wise proves the stability and coherence of the model.

IV.6.7. Volumetric deformations

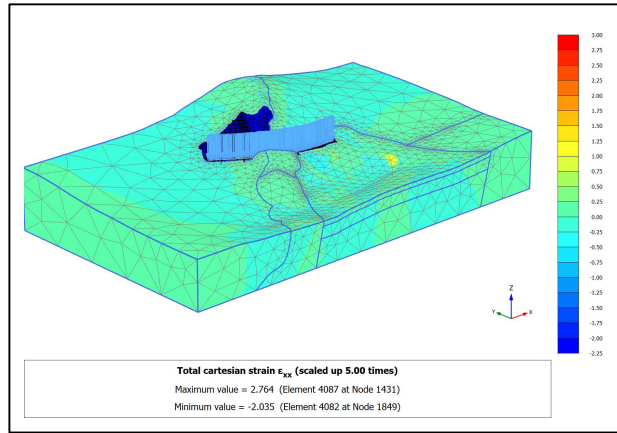


Figure 165 : 3D contour distribution of total cartesian strain (ϵ_{xx}).

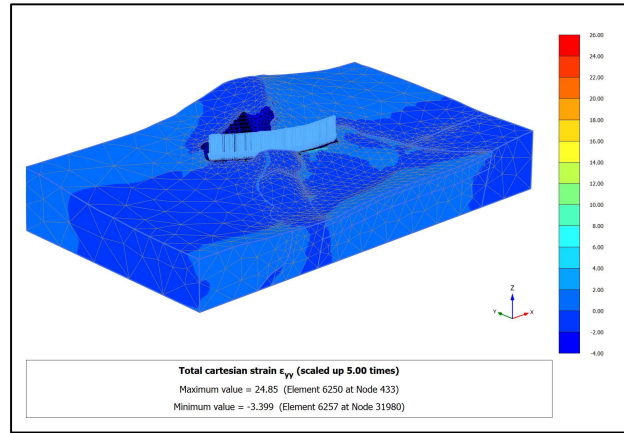


Figure 166 : 3D contour distribution of total cartesian strain (ϵ_{yy}).

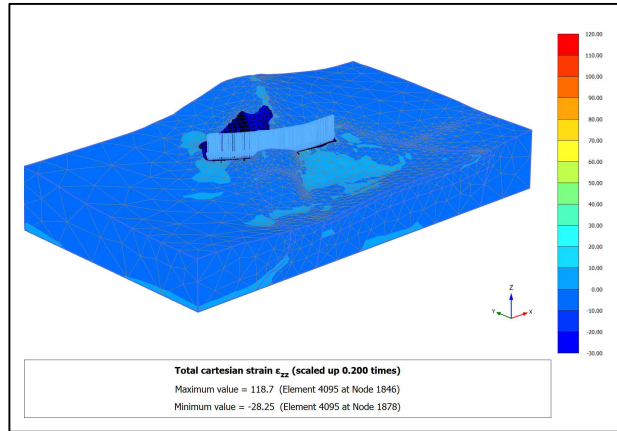


Figure 167 : 3D contour distribution of total cartesian strain (ϵ_{zz}).

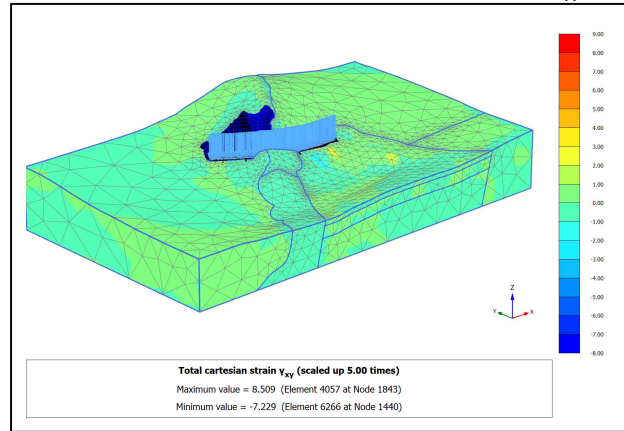


Figure 168 : 3D contour distribution of total cartesian strain (γ_{xy}).

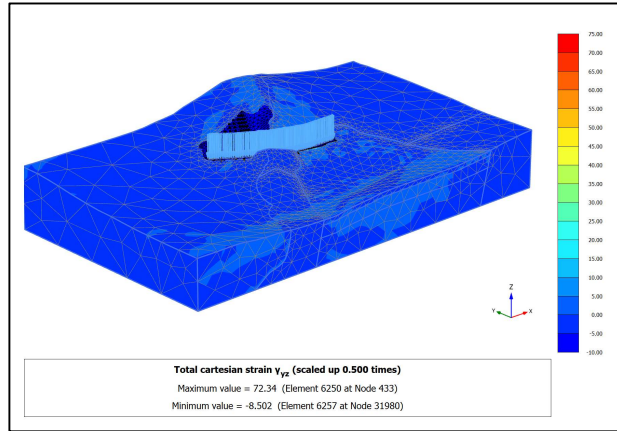


Figure 169 : 3D contour distribution of total cartesian strain (γ_{yz}).

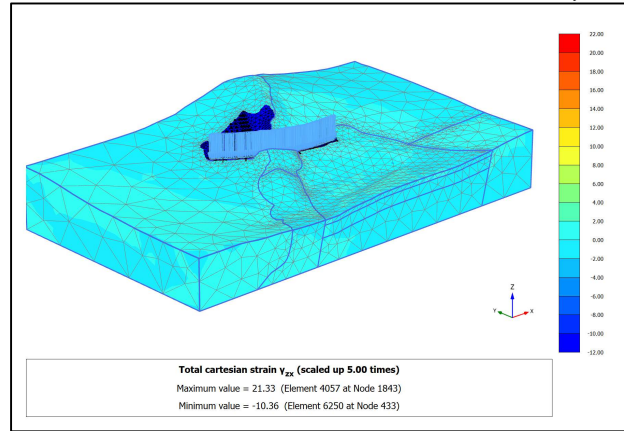


Figure 170 : 3D contour distribution of total cartesian strain (γ_{zx}).

IV.6.8.Evaluation of Stability and Deformation Curves

IV.6.8.1. Clay formations

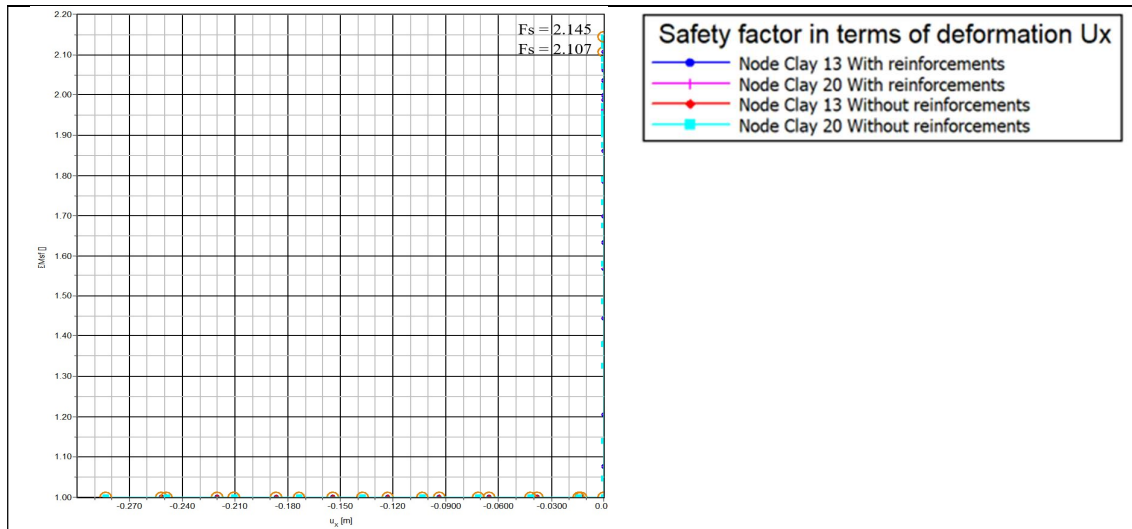


Figure 171 : Safety Factor vs. Deformations (U_x) for Clay formations (Reinforced and Unreinforced States).

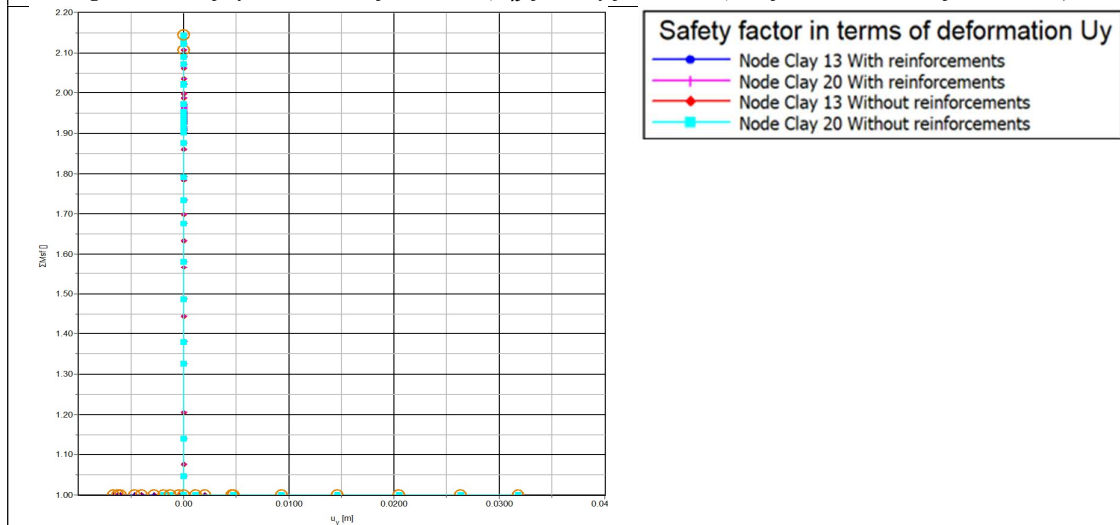


Figure 172 : Safety Factor vs. Deformations (U_y) for Clay formations (Reinforced and Unreinforced States).

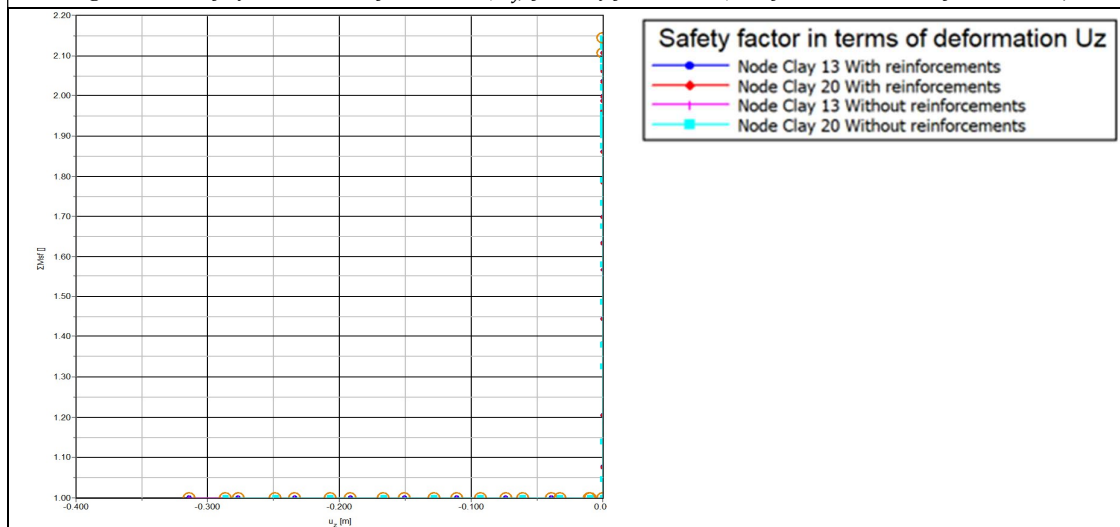


Figure 173 : Safety Factor vs. Deformations (U_z) for Clay formations (Reinforced and Unreinforced States).

IV.6.8.2. Limestone formations

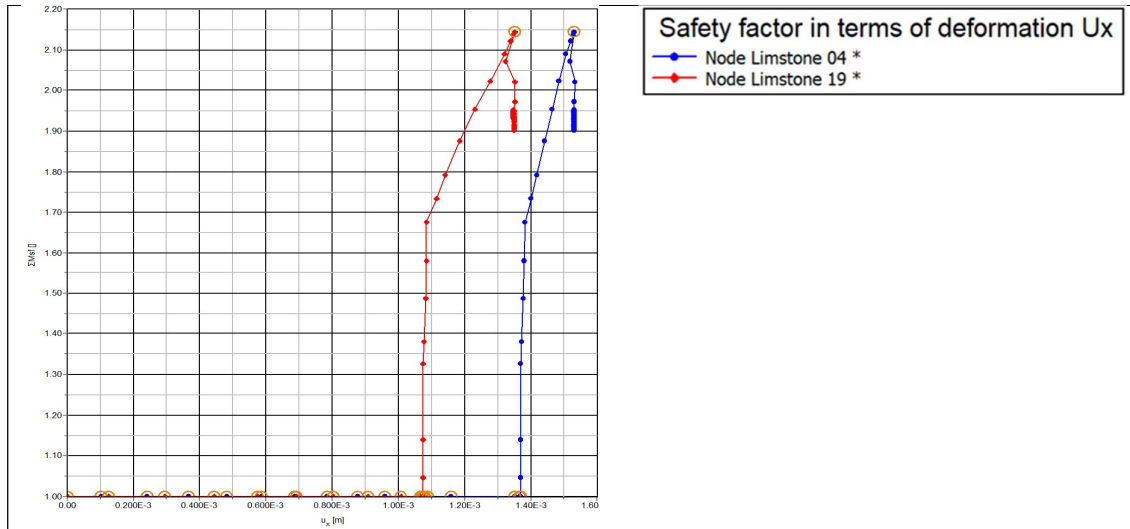


Figure 174 : Safety Factor vs. Deformations (U_x) for Limestone formations

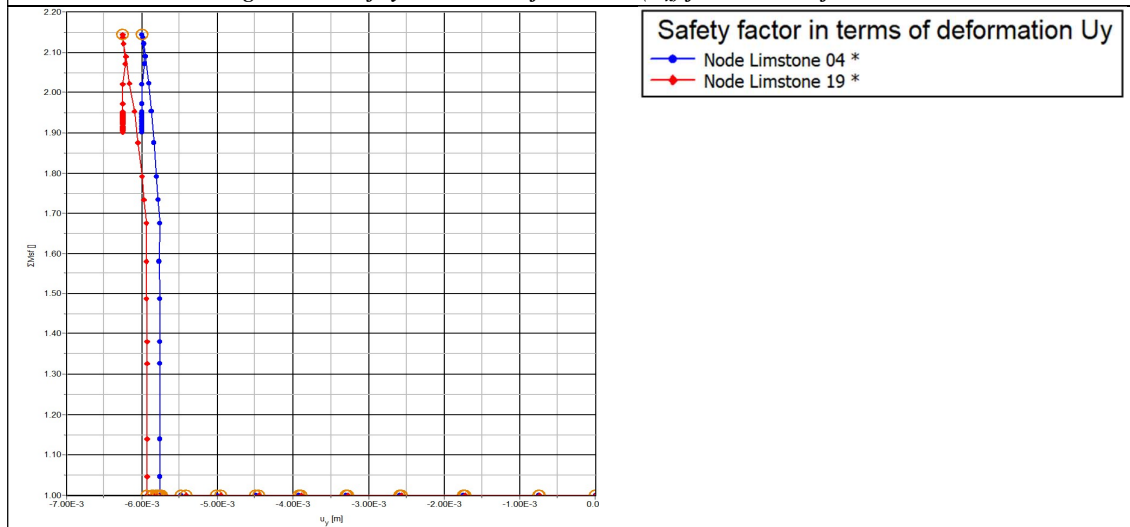


Figure 175 : Safety Factor vs. Deformations (U_y) for Limestone formations

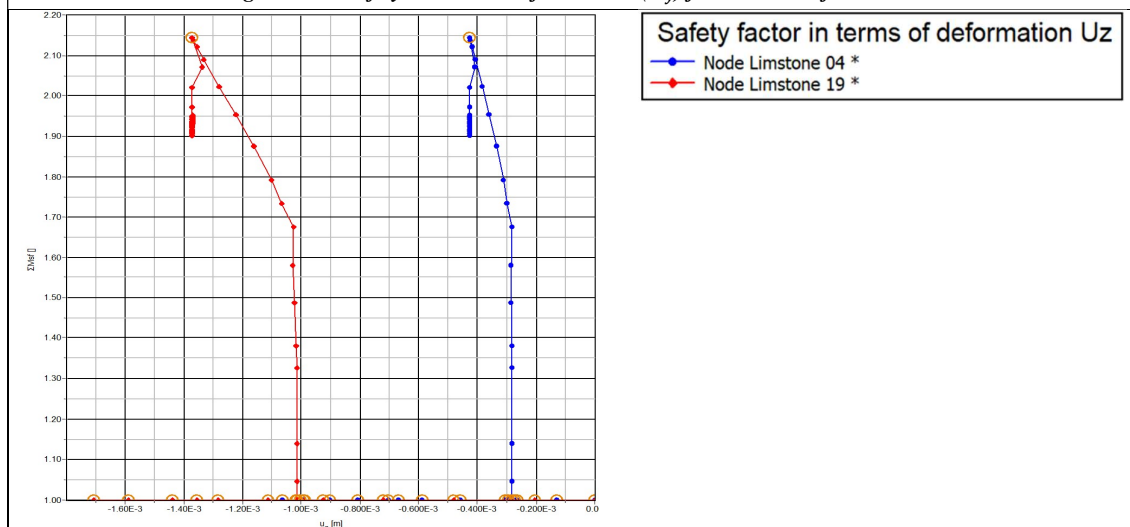


Figure 176 : Safety Factor vs. Deformations (U_z) for Limestone formations

IV.6.8.3. Triassic formations

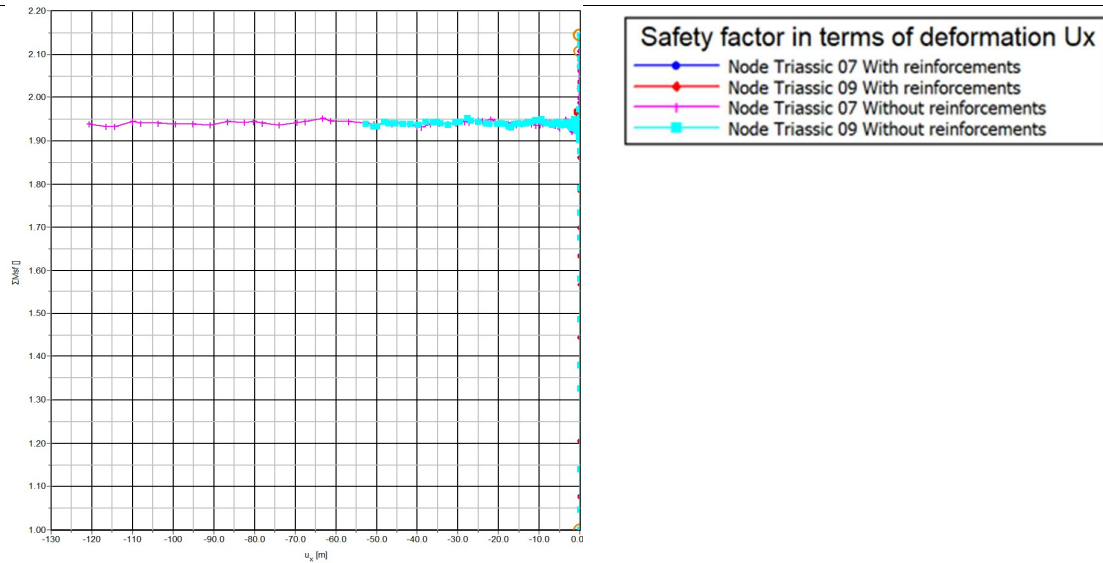


Figure 177 : Safety Factor vs. Deformations (U_x) for Triassic formations (Reinforced and Unreinforced States).

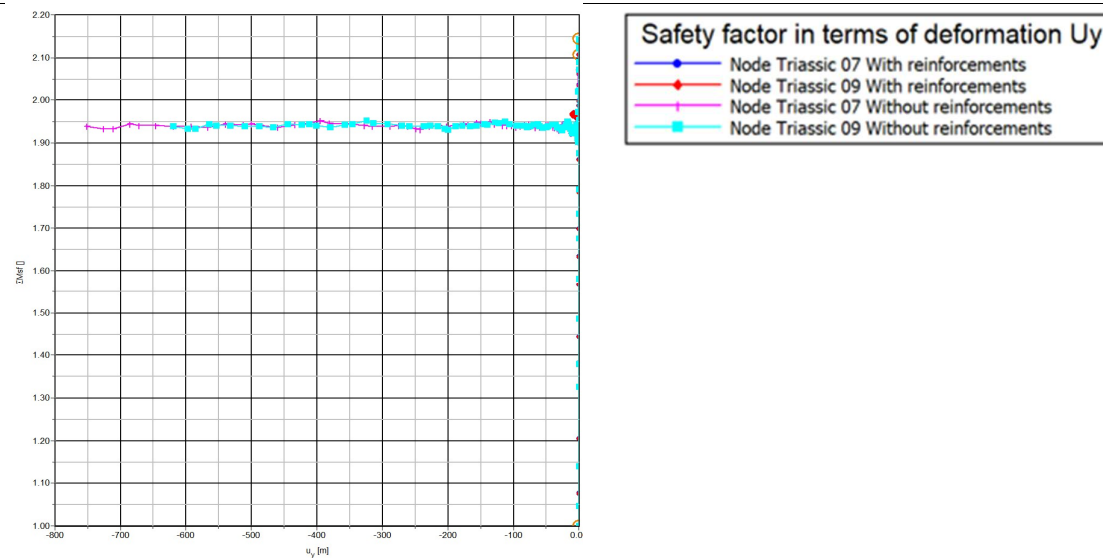


Figure 178 : Safety Factor vs. Deformations (U_y) for Triassic formations (Reinforced and Unreinforced States).

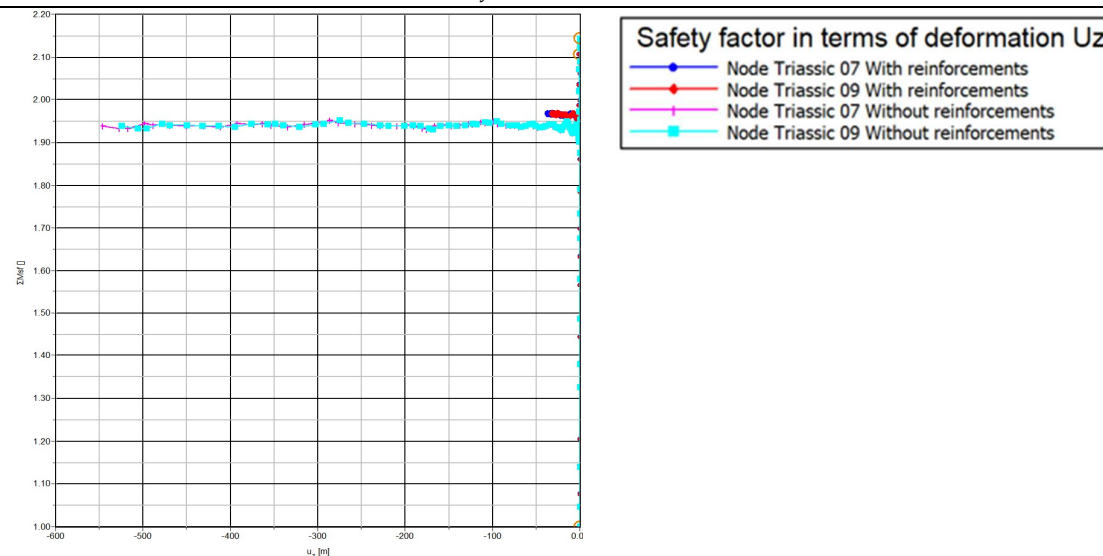


Figure 179 : Safety Factor vs. Deformations (U_z) for Triassic formations (Reinforced and Unreinforced States).

A. Clay and Triassic Formation (Nodes 13 and 20 / Nodes 07 and 09):

The stability analysis of the clay layer and Triassic formations reveals that the Factor of Safety (FoS) is 2.107 in the supported condition (after installation of the reinforcement system) and 2.145 in the unsupported condition. Both values comfortably satisfy the design criterion ($\text{FoS} \geq 1.5$), confirming the strong overall stability of this geotechnical section.

This slight decrease in the Factor of Safety after reinforcement arises from four confirmed causes, all related to the nature of the materials and their behavior during installation of the reinforcement system:

- **Overestimation of the initial resistance in the unsupported condition** due to simplified homogeneous modeling of the heterogeneous clay and Triassic formations, which made the reference value (2.145) higher than it actually is.
- **Local disturbance or remolding of the sensitive clay** during installation of the reinforcement system, resulting in a temporary or local reduction in resistance that is reflected in the calculated Factor of Safety after reinforcement.
- **The brittle behavior of the Triassic materials**, which causes a slight loss of resistance upon any minor deformation, even if that deformation results from the installation works of the reinforcement itself.
- **The inherent anisotropy of these formations**, where the orientation of the installed reinforcement elements does not perfectly align with the principal stress planes within the soil mass, creating a slight discrepancy in the efficiency of the reinforcement system.

Despite this decrease, the Factor of Safety in the supported condition (2.107) remains very high and clearly exceeds the minimum standard. Consequently, the analysis confirms that the reinforcement system acts primarily as a preventive measure aimed at limiting localized deformations and controlling long-term creep effects, rather than as an essential component for global slope stabilization.

B. Limestone Formation (Nodes 04 and 19):

For the limestone section, the analysis was performed under an unsupported configuration. The resulting curves indicate a progressive and well-controlled transition within the elastic-plastic domain. Owing to the high intrinsic stiffness and superior unconfined compressive strength of the limestone rock mass, the induced displacements (U_x , U_y , and U_z) remain extremely small, stabilizing within the millimeter scale (10^{-3} m). This behavior confirms that the excavated limestone section exhibits a fully self-supporting response and maintains excellent mechanical stability without the need for additional reinforcement.

IV.6.9. Dynamic deformation results

IV.6.9.1. Case 1 : without reinforcements

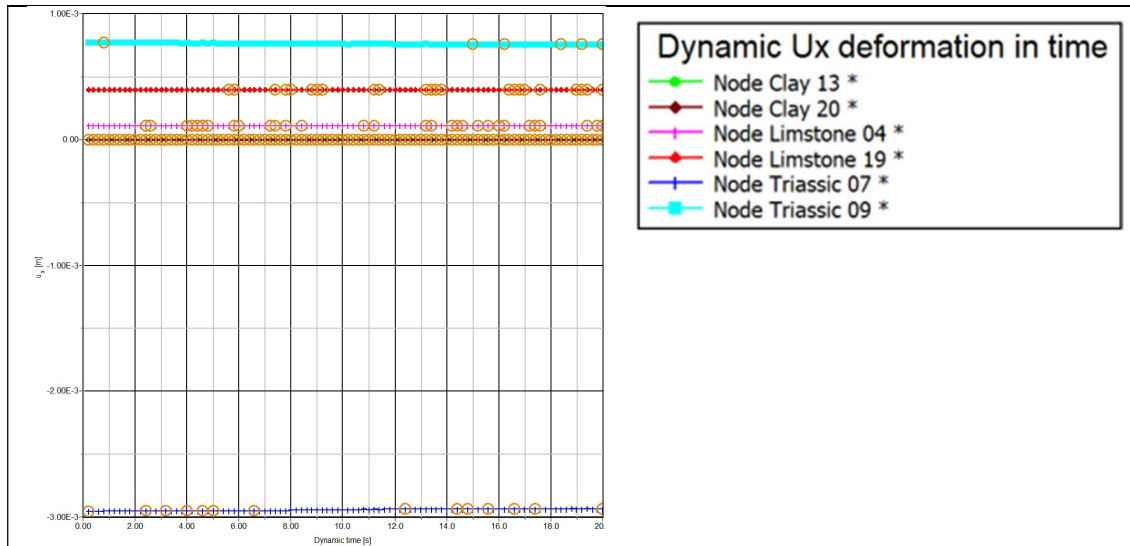


Figure 180 : 3D Horizontal Deformation (U_x) vs. Dynamic Time (Without Reinforcements).

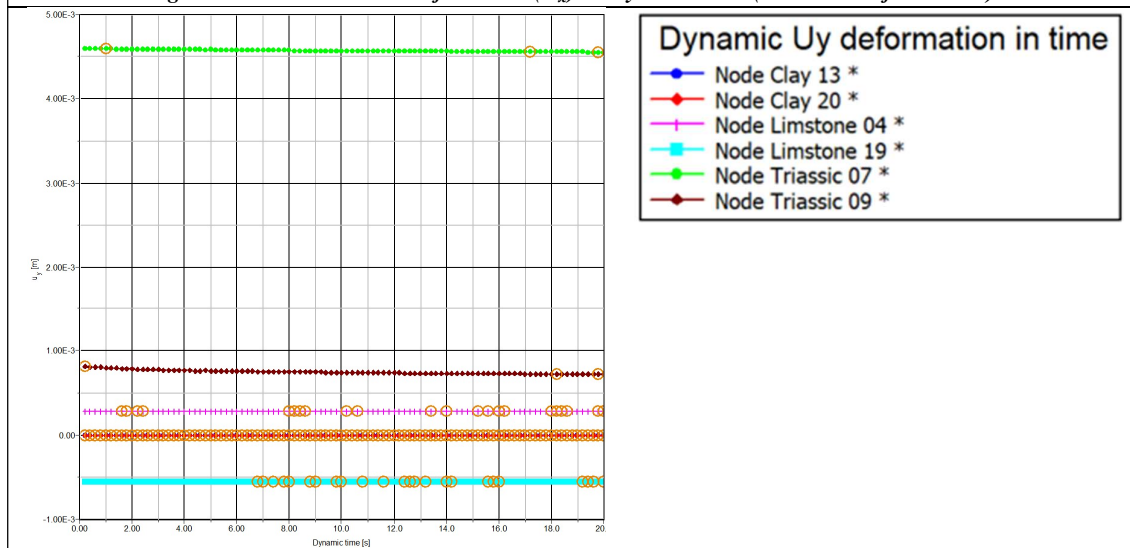


Figure 181 : 3D Horizontal Deformation (U_y) vs. Dynamic Time (Without Reinforcements).

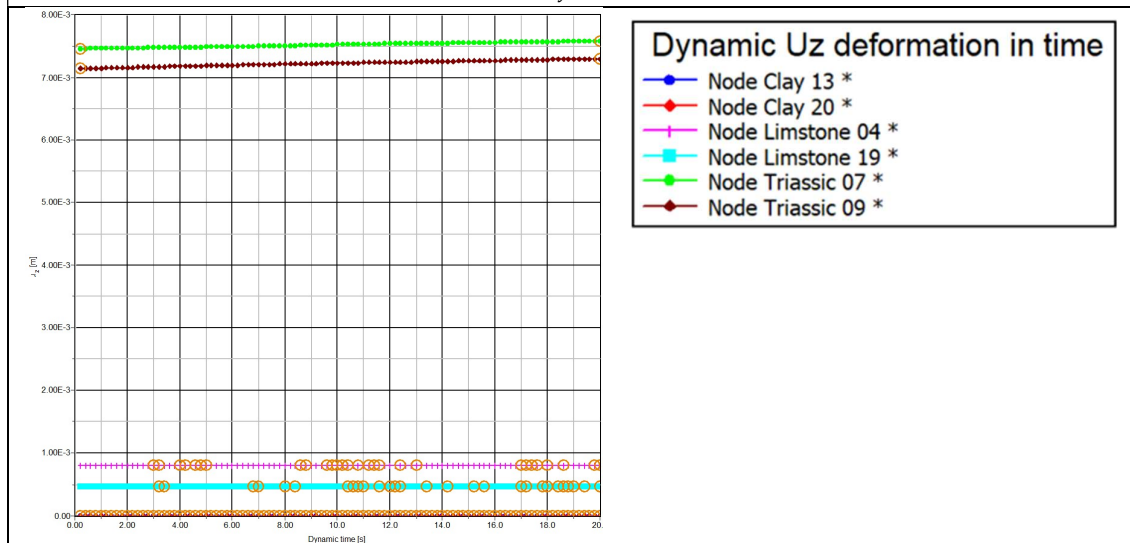


Figure 182 : 3D Vertical Deformation (U_z) vs. Dynamic Time (Without Reinforcements).

Table 88 : 3D Nodal Displacements (U_x , U_y , U_z) per Lithology (Without Reinforced).

Formation	Node	Displacement (u_x) [mm]	Displacement (u_y) [mm]	Displacement (u_z) [mm]
Triassic	7	2.935	4.5562	7.578936
	9	0.7559	0.72329	7.296
Limestone	4	0.11481	0.2864	0.806276
	19	0.40038	-0.55503	0.4665368

IV.6.9.2. Case 1 : With reinforcements

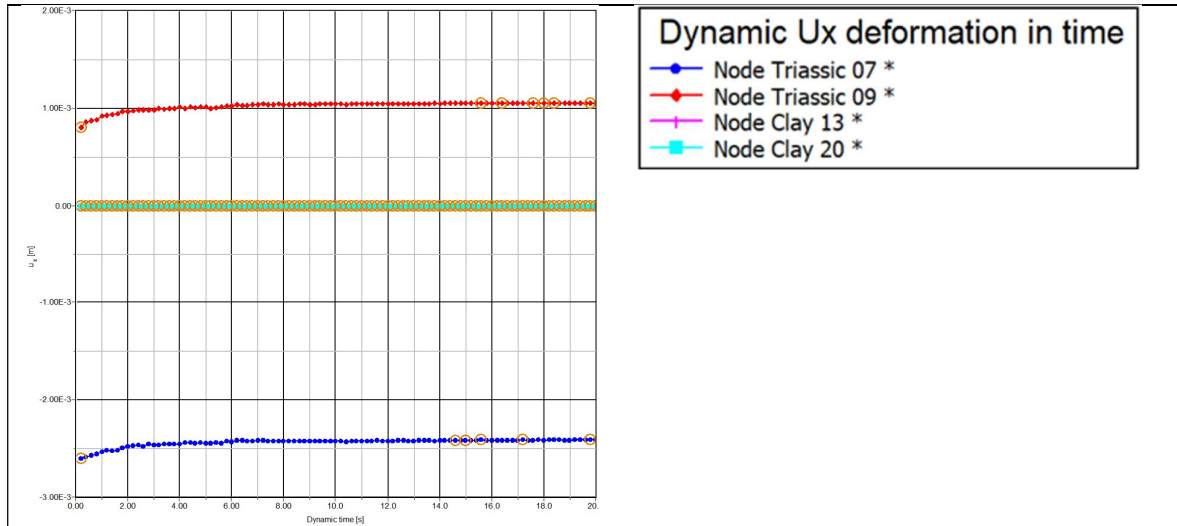


Figure 183 : 3D Horizontal Deformation (U_x) vs. Dynamic Time (With Reinforcements).

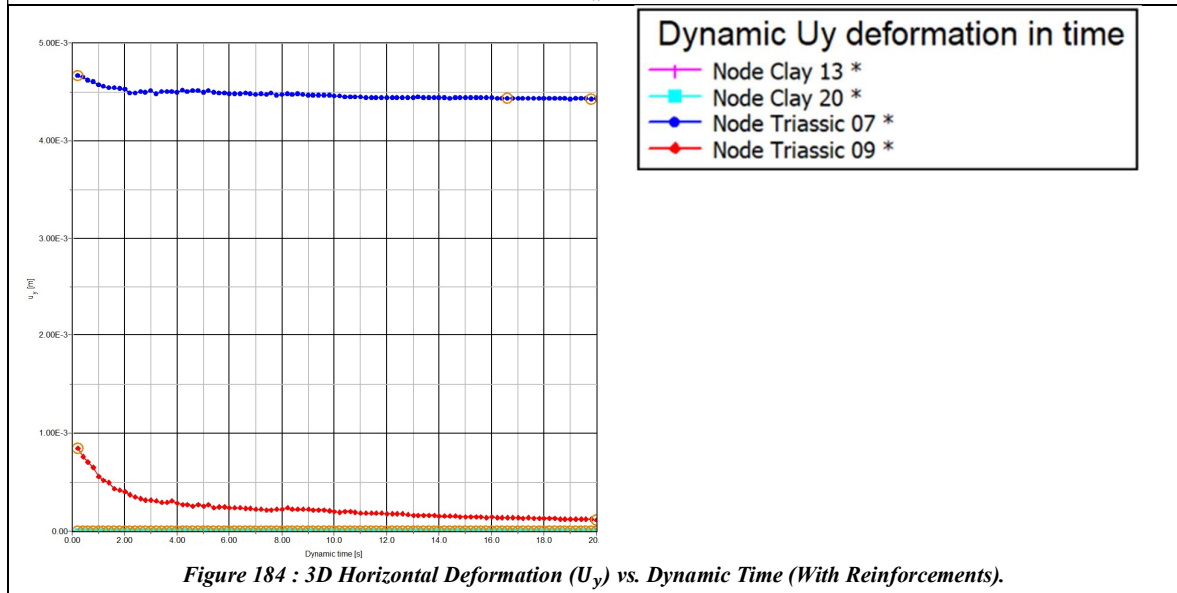


Figure 184 : 3D Horizontal Deformation (U_y) vs. Dynamic Time (With Reinforcements).

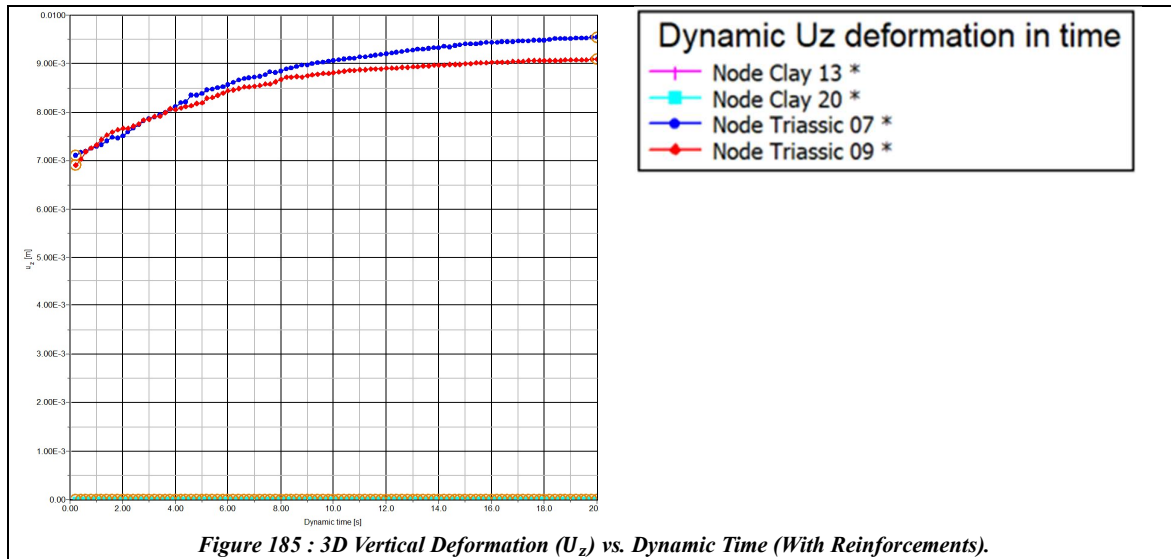


Table 89 : 3D Nodal Displacements (U_x , U_y , U_z) per Lithology (With Reinforced).

Formation	Node	Displacement (u_x) [mm]	Displacement (u_y) [mm]	Displacement (u_z) [mm]
Triassic	7	1.053	4.439	9.555
	9	-2.412	0.119	9.097

By comparing the mechanical behavior of soil layers under dynamic loads in two cases:

- With reinforcements.
- Without reinforcements.

It was observed that the deformation values at the Clay nodes are zero. This is mostly due to the fact that stresses and loads are transferred through the path of highest stiffness. The clay layer typically has a much lower Young's modulus compared to the Triassic or Limestone layers. This contrast results in the adjacent, stiffer rock layers generally bearing the greater share of dynamic deformations, while the clay layer remains relatively inert within the overall dynamic response. The results showed that reinforcements contribute to stress redistribution. The reinforcements successfully adjusted the horizontal displacement of the Triassic region significantly compared to the unreinforced case, thereby reducing the likelihood of horizontal shear. A slight increase in vertical displacement was also observed, which is explained by the shift of stresses from the horizontal to the vertical direction due to the restraints imposed by the reinforcements. When calibrating the Limestone layer, which acts as a self-stable solid mass, small and regular displacements were recorded, reflecting the material's cohesion in the absence of external reinforcements. By analyzing these data and readings, the following accident scenarios can be extrapolated should the current situation continue: In the absence of reinforcement, the layers exhibited much larger lateral displacements, increasing the risks of landslides or deformations in the rock mass or in the underlying railway structure. Moreover, the continuous accumulation of deformations over the long term may lead to landslides or deformations that cause damage to the railway.

IV.6.10. Flow Failure Analysis

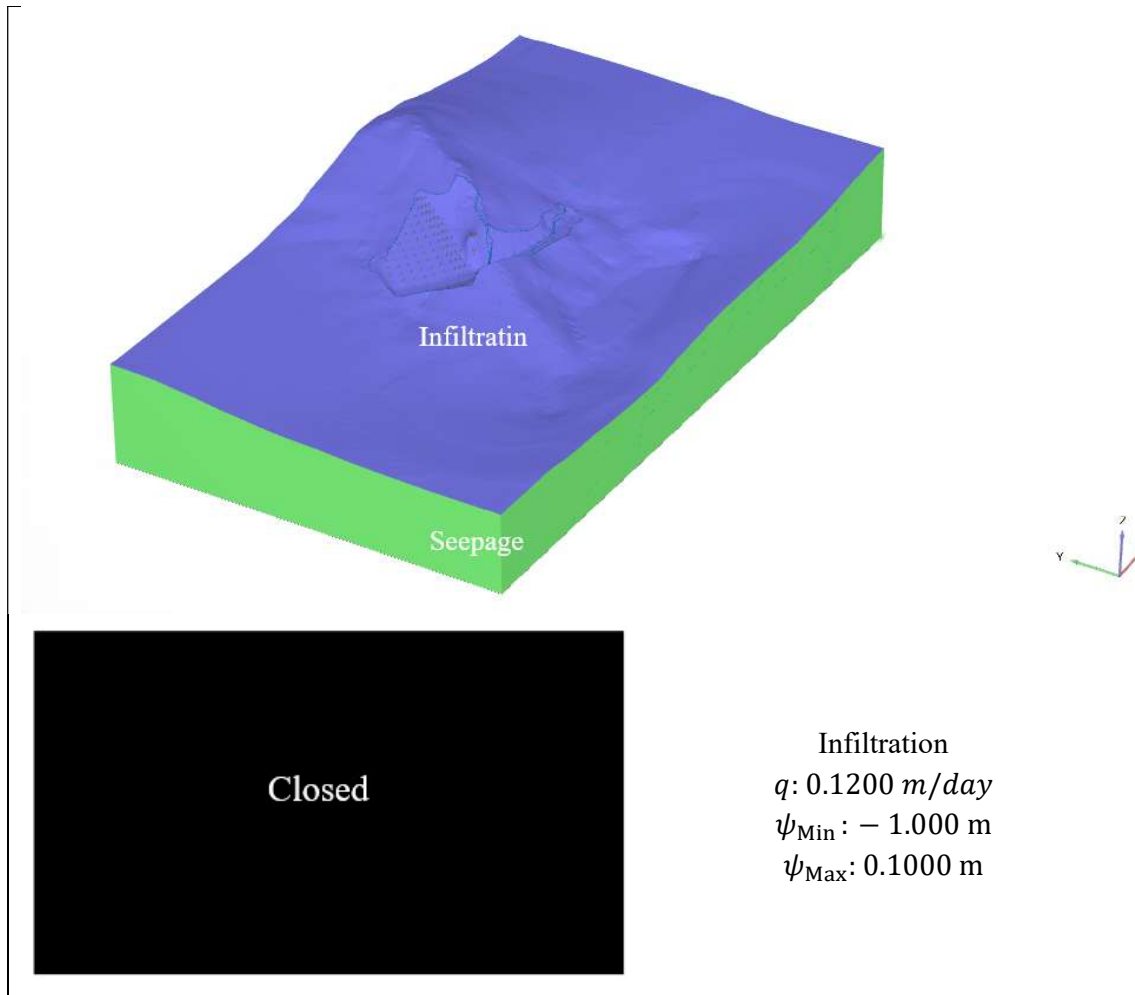


Figure 186 : Hydraulic boundary conditions for the natural state of the site (Without drainage or waterproofing systems).

The three-dimensional analysis indicated that the model reached a state of flow failure at a dynamic time of 1.00 day, with a critical peak excess pore water pressure (P_{max}) reaching up to $4.31 \times 10^4 \text{ Kn/M}^2$ (at time step $9.40 \times 10^{-2} \text{ days}$) just prior to total non-convergence. This failure is manifested by the model's inability to achieve equilibrium (non-convergence) and the development of a critical slip surface extending through the railway excavations. The progressively divergent displacement values indicate that the generated excess pore water pressure has led to a reduction in the soil's shear strength, resulting in a loss of the slope's dynamic stability.

IV.6.10.1. Performance study of drainage and waterproofing systems

the site requires to maintain its stability effective drainage systems (gutters and weep holes – caniveaux and barbacanes) and robust waterproofing, specifically two types of closed surfaces: a geomembrane, and shotcrete with welded wire mesh, which also acts as a rainwater barrier on slope surfaces and collects water into the drains, while the other peripheral systems transform the model to keep the Clay and Triassic formations dry.

Which can be represented in the model as follows:

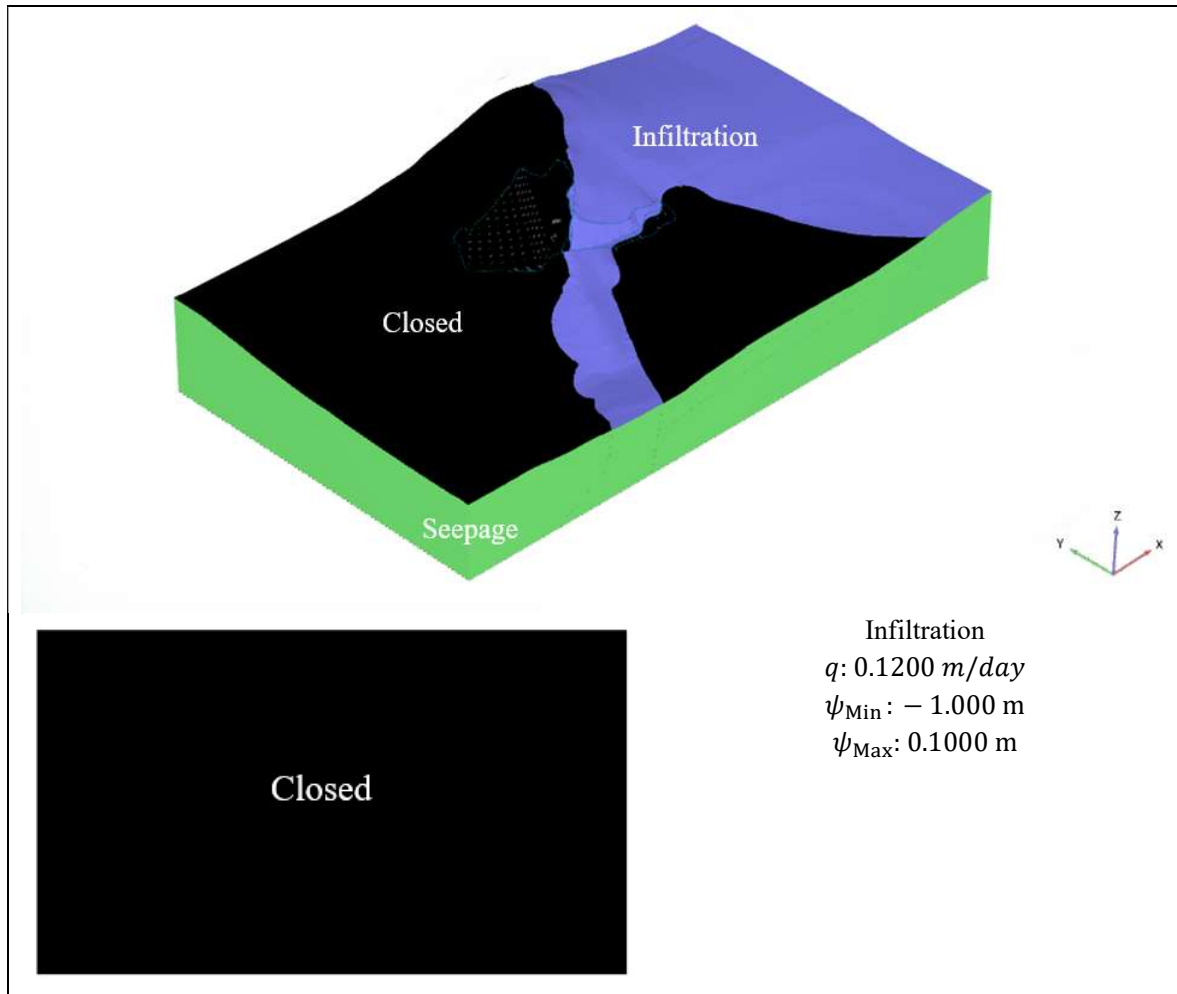


Figure 187 : Flow boundary conditions representing the geomembrane and drainage layers.

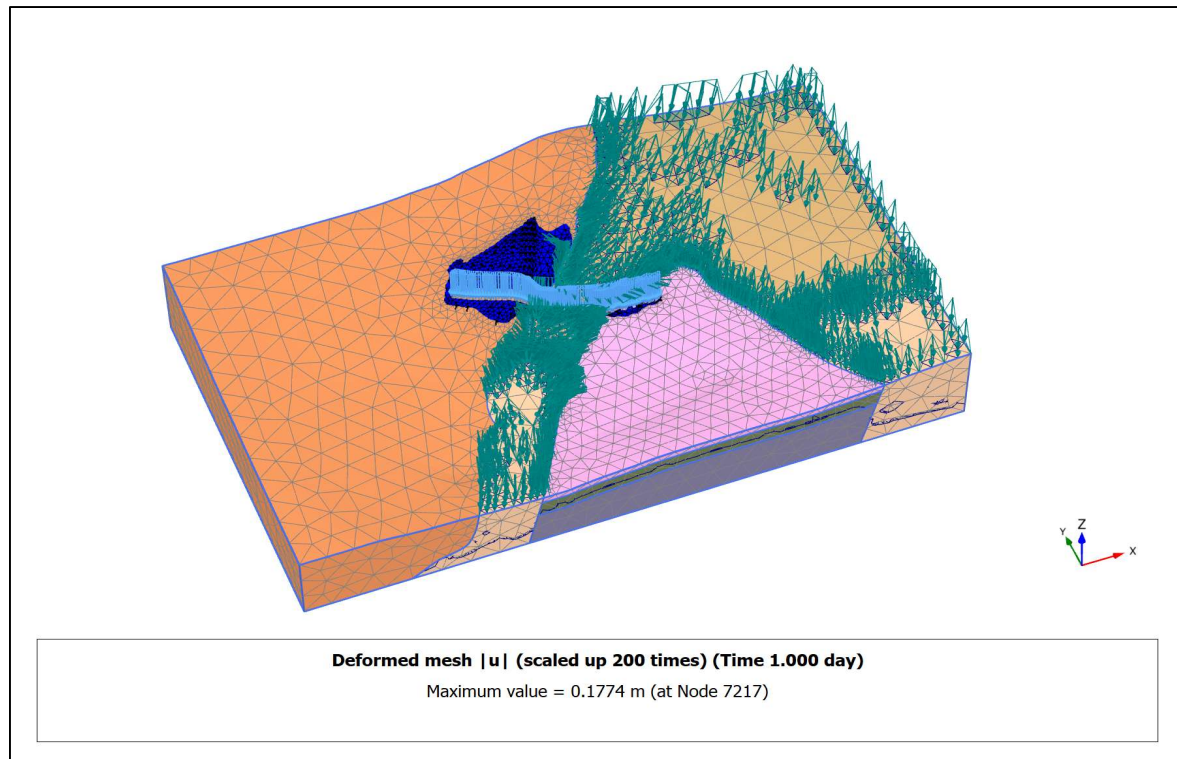


Figure 188 : 3D deformed finite element mesh with total displacement magnitude $|u|$ at $t = 1.000$ day (scaled up 200 times).

The numerical analysis reveals a stable coupled hydro-mechanical response, as evidenced by the total displacement contour

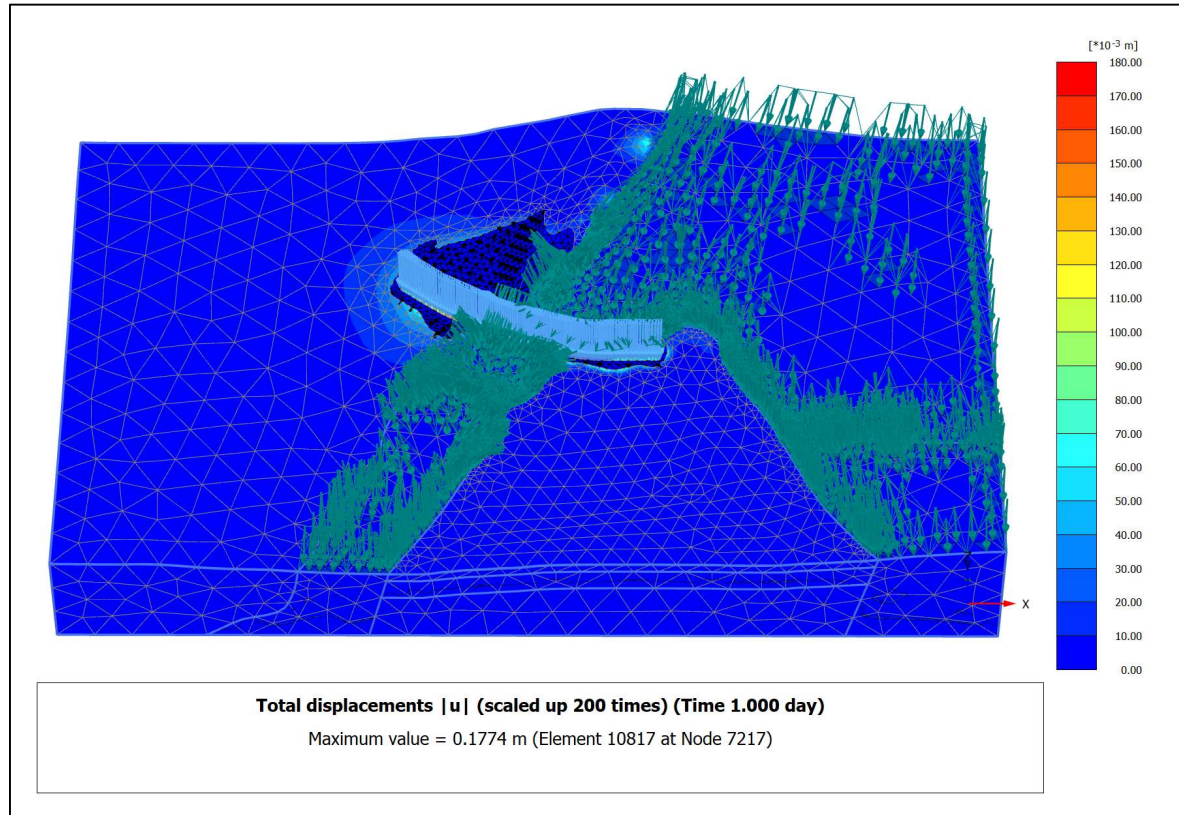


Figure 189 : 3D Numerical Modeling of Total Displacements Induced by Rainfall Infiltration.

These results show that structural movements remain largely localized and confined to the immediate vicinity of the newly constructed retaining system and excavation boundaries, while the surrounding soil mass remains stable (blue zone ranging from 0.00 to 30.00×10^{-3} m). A maximum total displacement of 0.1774 m (17.74 cm) is observed at Node 7217 (Element 10817). The $200\times$ scaled deformed mesh clearly illustrates the progressive lateral deformation mechanism, as the slope gradually moves toward the sequential excavation void.

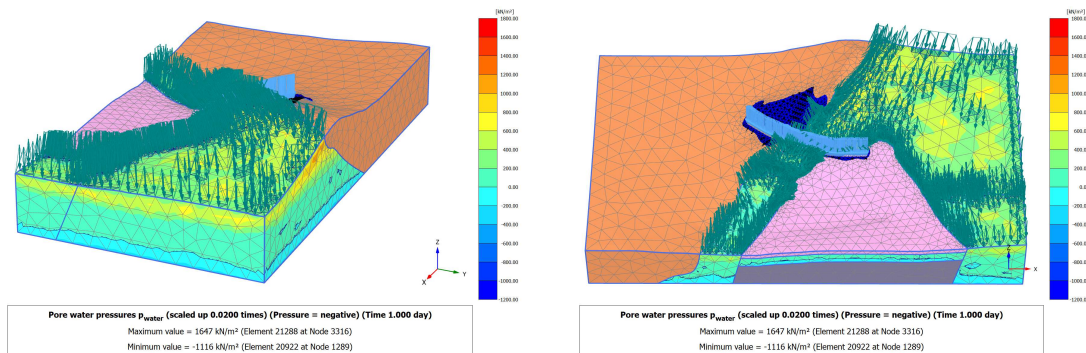


Figure 190 : 3D Numerical Simulation of Transient Pore Water Pressure.

Regarding pore water pressure distribution, the model exhibits a realistic hydrostatic gradient across the lithological cross-sections, with pore pressures increasing with depth up to a maximum of 1647 kN/m^2 in the deeper foundation layers (Element 21288 at Node 3316). In contrast, negative pore pressures (matric suction) reaching -1116 kN/m^2 develop in the shallow unsaturated zones and along the excavation faces due to stress relief induced by unloading. The downward-directed hydraulic vectors indicate surface boundary conditions and infiltration loading acting on the slope, contributing to the development of lateral driving forces toward the retaining structure.

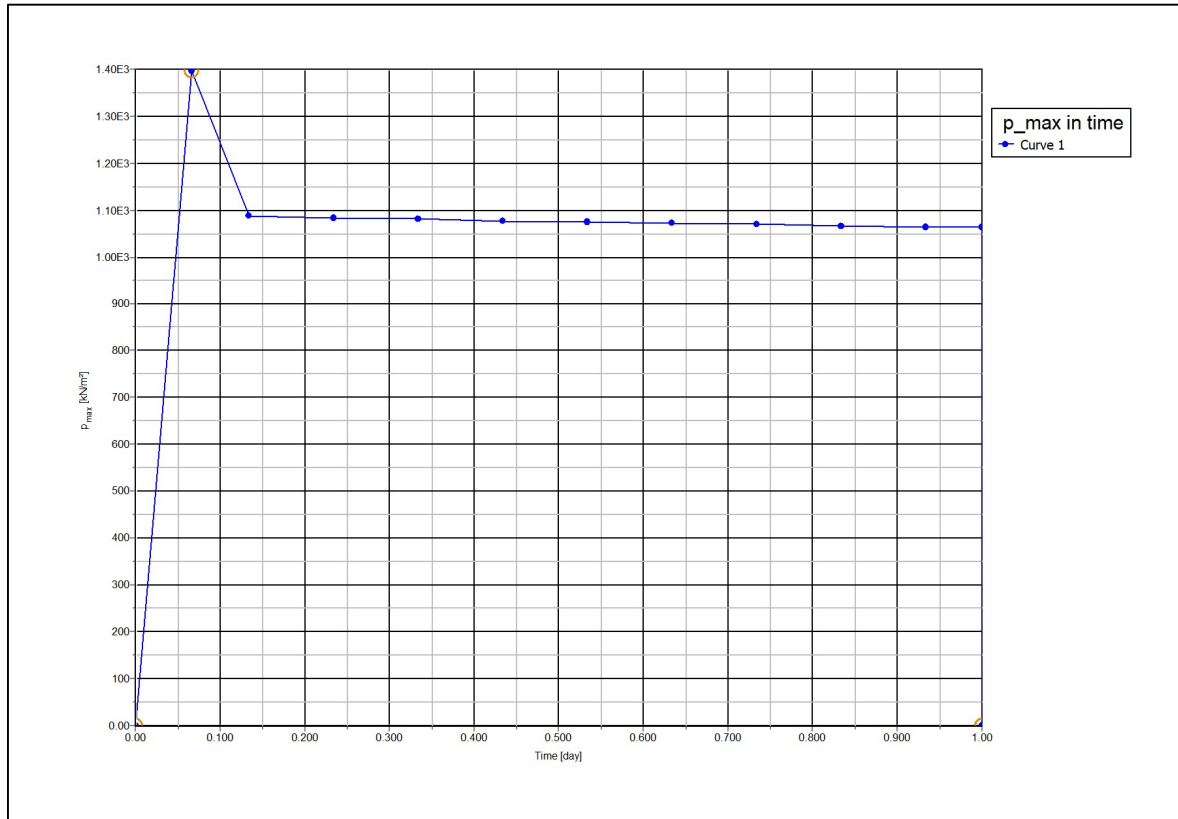


Figure 191 : Evolution of Maximum Pore Water Pressure Over Time Induced by Rainfall.

The transient analysis reveals that during the initial loading stage, at approximately $t = 0.07$ days, the pore water pressure peaks at roughly 1400 kN/m^2 ($1.40 \times 10^3 \text{ kN/m}^2$). This peak reflects an undrained transient response of the soil matrix to the rapid loading induced by excavation activities and subsequent structural adaptation. Following this peak, a pronounced downward trend signals the onset of excess pore water pressure dissipation. The curve then gradually stabilizes toward a quasi-steady value of approximately 1060 kN/m^2 , confirming that the model attains a stable drainage equilibrium within the one-day simulation period.

In summary, while deep-seated pore water pressures and surface water infiltration generate a maximum localized displacement of 17.74 cm, the displacements predicted within the facility footprint—according to the model—do not exceed 10 mm. This value remains well within the safe design limits prescribed by railway specifications. Consequently, the effective dissipation of excess pore water pressures, coupled with the subsequent stabilization, demonstrates that the implemented hydraulic system efficiently constrains deformation and ensures the overall stability of the modeled domain.

- These extracted results have shown a clear success for the applied systems (drains, waterproofing, and the shotcrete layer reinforced with a metal mesh) in achieving hydraulic stability for the slope and the clay and Triassic layers.

IV.8. CONCLUSION

The comprehensive geotechnical numerical modeling undertaken for the Tnoukla Railway Trench, integrating two-dimensional Finite Element Method (PLAXIS 2D), three-dimensional Finite Element Method (PLAXIS 3D), and Limit Equilibrium analyses (GEO5), has provided unprecedented insights into the stability and performance of the proposed excavation slopes under static, dynamic, and transient hydraulic conditions. The incorporation of high-fidelity 3D terrain modeling—generated within Autodesk 3ds Max and exported to PLAXIS 3D via OBJ format—has enabled a volumetric representation of the heterogeneous stratigraphy (Mio-Pliocene clays, Turonian limestones, and Trias argillaceous clays) that captures spatial variability and excavation sequencing with exceptional accuracy. The following definitive conclusions are drawn:

1. Global Stability Under Static Conditions – 2D and 3D Concordance

Both two-dimensional and three-dimensional analyses confirm that the Factor of Safety (FoS) for all cross-sections under static, unreinforced conditions consistently exceeds the minimum design threshold of $\text{FoS} \geq 1.5$. In the 3D model, the clay and Trias formations exhibited an FoS of 2.145 in the unsupported state and 2.107 following reinforcement installation—values substantially above the design requirement. The marginal decrease observed post-reinforcement (from 2.145 to 2.107) is attributed to four well-defined mechanisms: (i) overestimation of initial resistance due to simplified homogeneous modeling; (ii) local remolding of sensitive clays during anchor installation; (iii) brittle post-peak behavior of Trias materials; and (iv) anisotropy-induced misalignment between reinforcement orientation and principal stress planes. Critically, the 2.107 FoS remains exceptionally high, confirming that the reinforcement system serves primarily as a deformation-control and creep-mitigation measure rather than a global stability intervention. The Turonian limestone formation demonstrated fully self-supporting behavior with displacements stabilized at the millimeter scale (10^{-3} m), requiring no reinforcement.

2. Three-Dimensional Displacement Fields and Stress Redistribution

The 3D volumetric analysis revealed that soil removal induces release of confining stresses, triggering immediate slope movement and elevated lateral shear deformations (U_y) along excavation edges. Upon installation of the pile-anchor support system and application of the 80 KPa railway live load, deformation values converged to stable magnitudes, engineering-wise proving model coherence. Notably, clay layer nodes exhibited near-zero dynamic deformation due to load transfer along stiffness-preferential paths—the higher Young's modulus of Trias and limestone formations absorbing the majority of dynamic strains while the comparatively compliant clay remained inert. Reinforcement installation successfully redistributed stresses, significantly reducing horizontal displacements in the Trias region (thereby mitigating horizontal shear risk) while inducing a controlled increase in vertical displacement as stresses shifted from horizontal to vertical orientations due to reinforcement-imposed constraints.

3. Hydrologically Induced Failure Mechanism – A Consistent Finding Across Dimensions

Both 2D and 3D flow failure analyses converge on a singular, unambiguous conclusion: the Tnoukla trench is critically vulnerable to rainfall-induced failure absent robust waterproofing and drainage. In 2D simulations, Profiles 05, 09, 20, and 22 reached flow failure within 0.0026 to 0.01645 days (3.744 min – 23.688 min), with excess pore pressures ranging from 108.8 kN/m^2 to 823.97 kN/m^2 . The 3D analysis extended this understanding, revealing a peak excess pore pressure of $4.31 \times 10^4 \text{ kN/m}^2$ at time step 9.40×10^{-2} days, culminating in global non-convergence and development of a critical slip surface propagating through the railway excavation. The progressively divergent displacement values confirm that excess pore pressure generation reduces effective stress and shear strength, triggering catastrophic loss of slope stability. Profile 15 remained stable under identical hydraulic loading, achieving steady-state pore pressures of 393.38 kN/m^2 after 14 hours—a distinction attributable to local stratigraphic variations and inherent drainage characteristics.

4. Efficacy of Integrated Waterproofing and Drainage Systems

The performance assessment of the proposed hydraulic control systems—comprising geomembranes, shotcrete with welded wire mesh (acting as both surface protection and rainwater barrier), gutters, and weep holes demonstrated unequivocal success. The coupled hydro-mechanical 3D analysis revealed:

- Total displacements remained localized to the immediate vicinity of the retaining system, with the surrounding soil mass exhibiting stable behavior (blue zone: 0.00 to $30.00 \times 10^{-3} \text{ m}$). Maximum total displacement of 0.1774 m (17.74 cm) occurred at Node 7217, with the $200\times$ scaled deformed mesh illustrating progressive lateral deformation toward the excavation void.
- Pore water pressure distribution followed a realistic hydrostatic gradient, reaching 1647 kN/m^2 at depth, while negative pore pressures (matric suction) of -1116 kN/m^2 developed in shallow unsaturated zones and along excavation faces due to unloading-induced stress relief.
- Transient analysis captured a sharp pore pressure peak ($\approx 1400 \text{ kN/m}^2$) at $t \approx 0.07$ days, reflecting undrained response to rapid excavation and structural adjustment. Subsequent dissipation and stabilization toward a quasi-steady value of approximately 1060 kN/m^2 within the one-day simulation period confirmed successful drainage equilibrium.

5. Geochemical Vulnerability of Trias Argillaceous Clays – A Dual Mechanism

The elevated gypsum content (up to 12.35%) and sulphate concentration (SO_4 up to 6.89%) within the Trias clays render this formation susceptible to both mechanical degradation (via pore pressure rise) and chemical softening (via gypsum dissolution upon water ingress). This dual mechanism exacerbates internal erosion potential and compromises long-term slope integrity. The numerical results unequivocally demonstrate that the reinforcement system alone—anchors and shotcrete—is insufficient without prior hydraulic isolation. The waterproofing systems evaluated (geomembrane + shotcrete + drainage) successfully maintained the clay and Trias formations in a dry state throughout the simulated hydrological loading, as evidenced by the stable displacement contours and controlled pore pressure dissipation.

6. Extrapolated Failure Scenarios and Long-Term Prognosis

Comparative analysis of reinforced versus unreinforced configurations under dynamic loading permits extrapolation of two accident scenarios should current stabilization measures be inadequate or omitted:

- **Scenario A (Absence of reinforcement):** Layers exhibit substantially larger lateral displacements, increasing risks of progressive landslides, deformation of the rock mass, and damage to the underlying railway structure. Continuous long-term deformation accumulation may lead to catastrophic failure.
- **Scenario B (Reinforcement without drainage/waterproofing):** The reinforcement system redirects flow paths and limits pressure buildup but cannot prevent the chemical dissolution of gypsum within Trias clays. Internal erosion and dual loss of cohesion (mechanical + chemical) will inevitably compromise structural integrity.

7. Final Synthesis and Design Mandate

The Tnoukla Railway Trench, as designed, satisfies static stability criteria under dry conditions across both 2D and 3D numerical frameworks. However, the integrated numerical evidence—spanning FEM (PLAXIS 2D/3D) and Limit Equilibrium (GEO5) analyses—unequivocally demonstrates that the system is critically vulnerable to rainfall-induced flow failure unless robust, multi-layered hydraulic control measures are implemented. The 3D analysis confirms that the proposed integrated system (geomembrane + shotcrete with welded wire mesh + gutters + weep holes) achieves hydraulic stability, maintains the clay and Trias formations in a dry state, and limits total displacements to acceptable magnitudes ($\leq 17.74\text{ cm}$) with effective pore pressure dissipation. This regional displacement does not extend its effects to the structure's domain; the local displacements simulated within the structure's domain do not exceed, according to the model, a value of 10 mm, which falls within the safe design limits imposed by railway specifications.

Therefore, the project's long-term success hinges upon the rigorous, mandatory implementation of:

1. Primary waterproofing: Geomembrane placement directly over Trias clay surfaces to prevent infiltration and gypsum dissolution;
2. Secondary surface protection: Sulfate-resistant shotcrete (CRS) with welded wire mesh to inhibit surface erosion and act as a rainwater barrier;
3. Tertiary drainage: Perimeter drainage systems to collect intercepted water and maintain dry conditions within clay and Trias formations.

The reinforcement system, while beneficial for deformation control, stress redistribution, and long-term creep mitigation, does not substitute for hydraulic isolation. Only through the synergistic integration of structural reinforcement, chemical-resistant waterproofing, and active drainage can the Tnoukla Railway Trench be transformed from a potentially unstable hydraulic regime into a durable, resilient, and serviceable geotechnical infrastructure asset capable of withstanding both static operational loads and extreme rainfall events.

General conclusion

The integrated geotechnical site investigation and numerical modeling for the Tnoukla Railway Trench have produced a set of definitive, quantifiable results that define the project's stability, critical vulnerabilities, and mandatory design requirements.

Results of Site Characterization (Geological & Laboratory Testing) The investigation characterized three distinct formations: Mio-Pliocene clays, Turonian limestones, and Triassic diapiric materials. The principal result is the identification of severe geochemical hazards within the Triassic gypsum-rich clays, which contain sulphate concentrations between 49,000 and 69,000 mg/kg (classified as DS-5 aggressivity), and gypsum content up to 12.35%. These values mandate sulphate-resisting cement for all concrete elements. Karstic voids within weathered Turonian limestones force a classification of Class V (Very Poor) despite high intact strength (UCS ranging from 19.9 to 140.5 MPa; GSI from 20 to 75), requiring grouting of all karstic zones. The Mio-Pliocene clays exhibit plasticity indices up to 26.9%, indicating moderate to high swelling potential, necessitating shallow foundations with strict moisture control.

Results of Stability Analysis Under Dry (Static) Conditions Under unreinforced, dry conditions, the Factor of Safety (FoS) for all cross-sections consistently exceeds the minimum design threshold of $FoS \geq 1.5$. The 3D finite element model (PLAXIS 3D) yields an unsupported FoS of 2.145 for the clay and Trias formations. Following installation of the pile-anchor support system, the FoS decreases marginally to 2.107—a reduction attributed to local remolding of sensitive clays and post-peak brittle behavior of Trias materials, but still exceptionally above the design requirement. The Turonian limestone formation is fully self-supporting, with displacements stabilized at the millimeter scale ($10^{-3} m$), requiring no reinforcement.

Critical Vulnerability Result: Rainfall-Induced Flow Failure; the single most critical result is that the trench is catastrophically vulnerable to rainfall-induced flow failure in the absence of waterproofing and drainage. Under transient hydraulic loading, 2D simulations show that Profiles 05, 09, 20, and 22 reach flow failure within 0.0026 to 0.01645 days (3.744 min – 23.688 min), driven by excess pore pressures ranging from 108.8 to 823.97 kN/m^2 . The 3D analysis confirms this, revealing a peak excess pore pressure of $4.31 \times 10^4 kN/m^2$ at time step 9.40×10^{-2} days, leading directly to global numerical non-convergence and a critical slip surface propagating through the railway excavation. In contrast, Profile 15 remains stable under identical hydraulic loading, achieving steady-state pore pressures of 393.38 kN/m^2 after 14 hours—a result directly attributable to local stratigraphic variations and inherent drainage characteristics.

Results of the Integrated Waterproofing and Drainage System: Performance assessment of the proposed hydraulic control system (geomembrane + sulfate-resistant shotcrete with welded wire mesh + gutters + drainage openings) demonstrates unequivocally positive results. The system successfully maintains the clay and Triassic formations in a dry state throughout the simulated hydrological loading period. Total displacements remain localized, reaching a maximum value of 0.1774 m (17.74 cm) at node 7217, while the surrounding soil mass remains stable (blue zone: 0.00 to $30.00 \times 10^{-3} m$). This regional displacement does not extend its effects to the facility footprint; according to the model, the simulated

localized displacements within the facility area do not exceed 10 mm, which falls well within the safe design limits imposed by railway specifications. Furthermore, the transient analysis reveals a sharp pore pressure peak of approximately 1400 kN/m² at $t \approx 0.07$ days (undrained response to rapid excavation), followed by successful dissipation and stabilization toward a quasi-steady value of approximately 1060 kN/m² over the one-day simulation period, confirming drainage equilibrium. However, these results remain contingent on the model assumptions, necessitating field validation through an automated monitoring system. It is also demonstrated that structural support alone is insufficient unless complemented by effective hydraulic isolation. Two extrapolated accident scenarios are quantified:

- **Scenario A (without reinforcement):** Layers exhibit substantially larger lateral displacements, with progressive landslide development, rock mass deformation, and eventual catastrophic failure.
- **Scenario B (Reinforcement without drainage/waterproofing):** The reinforcement redirects flow paths but cannot prevent chemical dissolution of gypsum (up to 12.35%) within Trias clays. Internal erosion and dual loss of cohesion (mechanical + chemical) will inevitably compromise structural integrity.

Final Design Mandate Based on Results; the project's long-term success hinges upon the mandatory, sequential implementation of:

- (1) primary geomembrane waterproofing directly over Trias clay surfaces;
- (2) secondary sulfate-resistant shotcrete (CRS) with welded wire mesh;
- (3) tertiary perimeter drainage systems. Only through this synergistic integration can the Tnoukla Railway Trench achieve durable, resilient serviceability under both static operational loads and extreme rainfall events.

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ANNEX 01: Geotechnical Logging and Lithological Description

A.1.1 Borehole SCD-01

Table 90 : Geotechnical Logging and Lithological Description of Borehole SCD-01.

Depth (m)	Lithological Description	Recovery (%)	RQD (%)	Technical Observations
0.00 – 4.10	Hard Limestone: Grey, highly fractured and oxidized (surface formation).	33% - 52%	10% - 33%	Poor rock quality due to weathering.
4.10 – 8.00	Totally Destructured Formation: Composed of limestone fragments within a clayey matrix.	52% - 92%	10% - 44%	Highly fractured level; improving recovery.
8.00 – 25.50	Hard Limestone: Grey, moderately to highly fractured, with millimetric closed fissures filled with calcite.	92% - 98%	43% - 90%	Significant improvement in rock mass quality.
25.50 – 26.20	Highly Fractured Zone: Open fissures completely filled with clay.	96%	30%	Sudden drop in RQD due to clay infilling.
26.20 – 27.00	Hard Limestone: Grey, moderately to highly fractured, millimetric closed fissures (calcite).	98%	82%	Recovery of rock mass stability.
27.00 – 32.80	Hard Limestone: Grey-pinkish, slightly fractured, rich in calcite, with centimetric oblique fissures.	98% - 99%	82% - 94%	Very good rock quality; some sandy-clay infill.
32.80 – 33.10	Dissolution Zone: Area affected by localized dissolution (karstic features).	98%	94%	Potential for localized voids.
33.10 – 51.00	Hard Limestone: Grey-pinkish, slightly fractured, centimetric closed/open fissures (calcite/sandy-clay).	92% - 100%	66% - 94%	Consistent rock mass until E.O.H (51.0m).

A.1.2 Borehole SCD-02

Table 91 : Geotechnical Logging and Lithological Description of Borehole SCD-02.

Depth (m)	Lithological Description	Recovery (%)	Technical Observations
0.00 – 14.40	Firm Clay: Reddish, rich in gypsum, containing decimetric limestone rocky passages.	60% - 100%	Heterogeneous Stratum with alternating firm clay and rock fragments.
14.40 – 21.00	Compact Marly Clay: Reddish to greyish, rich in gypsum.	100%	High consistency with significant gypsum content.
21.00 – 42.90	Argillaceous Marl (Variegated): Slightly weathered, very compact, very rich in gypsum. Grey color with white gypsum spots.	100%	Very stable, high-recovery formation; variegated (bariolé) texture.
42.90 – 44.10	Highly Weathered Clayey Passage: Compact, very rich in gypsum, reddish color.	100%	Localized highly weathered zone; potential weak point.
44.10 – 50.00	Argillaceous Marl: Slightly weathered, very compact, very rich in gypsum. Grey color with white gypsum spots.	100%	Consistent compact marly substratum until E.O.H (50.0m).

A.1.3 Borehole SCD-03

Table 92 : Geotechnical Logging and Lithological Description of Borehole SCD-03.

Depth (m)	Lithological Description	Recovery (%)	Technical Observations
0.00 – 1.30	Alluvium: Composed of gravel and pebbles embedded in a clayey-silty-sandy matrix.	95%	Surficial transport deposits.
1.30 – 3.20	Plastic Clay: Greenish-brown, containing some pebbles/gravel with traces of oxidation and gypsum.	88%	Presence of gypsum indicates specific chemical weathering.
3.20 – 31.90	Compact Clay: Brown; becomes plastic upon contact with water.	92% - 100%	Significant thick Stratum; sensitive to water content changes.
31.90 – 34.20	Weathered Compact Silty Clay: Brown with grey marly spots.	100%	Transition zone with localized marl content.
34.20 – 35.90	Compact Clay: Brown; becomes plastic upon contact with water.	100%	Similar properties to the upper compact clay Stratum.
35.90 – 38.20	Firm Silty-Sandy Clay: Dark brown.	100%	Slightly higher sand/silt content; firm consistency.
38.20 – 42.30	Marly Clay: Slightly gravelly, firm, greenish-brown.	50% - 100%	Notable drop in recovery (50%) between 38m and 41m.
42.30 – 60.00	Compact Clay: Brown; becomes plastic upon contact with water.	100%	Base of the borehole; consistent clayey substratum.

A.1.4 Borehole SCD-04

Table 93 : Geotechnical Logging and Lithological Description of Borehole SCD-04.

Depth (m)	Lithological Description	Recovery (%)	RQD (%)	Technical Observations
0.00 – 5.60	Hard Limestone: Pinkish-grey, slightly fractured, oxidized with clay traces and roots.	94% - 100%	79% - 90%	Weathered surface rock mass.
5.60 – 6.40	Altered Limestone: Affected by dissolution (karsts) with calcitic or clayey infilling.	71%	61%	Initiation of karstic features.
6.40 – 6.60	Karstic Void (Vide)	--	--	Critical Hazard: Open void detected.
6.60 – 8.00	Altered/Fractured Limestone: Dissolution features with infilling.	71%	61%	Weathered rock between voids.
8.00 – 8.90	Karstic Void (Vide)	--	--	Critical Hazard: Second open void.
8.90 – 15.50	Hard Limestone: Sandy aspect (fine calcite crystals), centimetric fissures filled with calcite.	64% - 99%	54% - 76%	Recovery of rock mass consistency.
15.50 – 16.40	Karstified Level	86%	44%	Highly porous/dissolved zone.
16.40 – 27.70	Hard Limestone: Pinkish-grey to grey, centimetric fissures (closed or open with sandy-clay infill).	80% - 99%	60% - 88%	Stable rock mass with localized infilling.
27.70 – 33.70	Deformation Zone: Brecciated limestone, highly weathered/fractured, cemented by calcite and clays.	86% - 98%	53% - 80%	Geological shear/fault zone.
33.70 – 52.10	Stiff Limestone: Grey to pinkish-grey, slightly fractured with fine/long calcite-filled fissures.	95% - 100%	65% - 99%	High-quality stiff rock mass.
52.10 – 58.40	Deformation Zone: Brecciated limestone, highly weathered, clay/calcite cement with black stains.	77% - 95%	23% - 31%	Deep weak zone; potential slip surface.
58.40 – 60.00	Hard Limestone: Grey, slightly fractured.	100%	94%	Base of borehole; excellent rock quality.

A.1.5 Borehole SCD-05

Table 94 : Geotechnical Logging and Lithological Description of Borehole SCD-05.

Depth (m)	Lithological Description	Recovery (%)	RQD (%)	Technical Observations
0.00 – 6.80	Hard Limestone: Pinkish-grey, slightly fractured, minor oxidation; fine fissures filled with calcite.	100%	96%	Excellent rock mass quality at the surface.
6.80 – 8.30	Dissolution Zone: Level affected by dissolution with the appearance of small cavities.	97%	95%	Initial karstic activity; high recovery despite cavities.
8.30 – 14.80	Hard Limestone: Pinkish-grey, slightly fractured, oxidized fissures filled with calcite.	99% - 100%	96% - 98%	Solid rock mass with calcite veins.
14.80 – 24.00	Fractured Limestone: Highly rich in calcite; fissures are severely weathered and affected by dissolution.	100%	92% - 93%	Increased weathering; calcite-filled fissure at 23.5m.
24.00 – 32.00	Hard Limestone: Reddish, slightly fractured, calcite-filled fissures.	97% - 100%	73% - 98%	Micro-cavity (dissolution zone) at 26.40m.
32.00 – 34.00	Highly Fractured Level : Mixture of limestone fragments embedded in a clayey matrix.	98%	53%	Geotechnical weak zone ; sudden drop in RQD.
34.00 – 38.00	Hard Limestone : Pinkish-grey, slightly fractured, closed oblique fissures filled with calcite.	100%	98%	Recovery of rock mass stiffness.
38.00 – 48.00	Stiff Limestone : Reddish, slightly fractured, fine calcite-filled fissures.	100%	94% - 97%	Consistent stiff formation.
48.00 – 60.00	Stiff Brecciated Limestone : Moderately fractured, localized weathering ; centimetric fissures fully filled with calcite.	97% - 100%	77% - 98%	Altered/Oxidized level (52.8-54.6m); visible dissolution at 57.5m and 58.7m.

A.1.6 Borehole SCD-06

Table 95 : Geotechnical Logging and Lithological Description of Borehole SCD-06.

Depth (m)	Lithological Description	Recovery (%)	Technical Observations
0.00 – 0.40	Clay and Alluvium	100%	Surficial Stratum.
0.40 – 6.70	Compact Brown Clay: Becomes plastic upon contact with water.	100%	Standard Penetration Tests (SPT) show resistance >50 blows.
6.70 – 7.00	Clay and Alluvium	100%	Thin alluvial lens within the clay.
7.00 – 40.00	Compact Brown Clay: Becomes plastic upon contact with water; contains grey marly spots from 20m depth to the bottom.	100%	Highly consistent recovery; consistently high SPT resistance (>50 blows).

A.1.7 Borehole SCD-07

Table 96 : Geotechnical Logging and Lithological Description of Borehole SCD-07.

Depth (m)	Lithological Description	Recovery (%)	Technical Observations
0.00 – 5.00	Loose Surface Formation: Blocks and gravel within a clay-sandy matrix with root debris.	86% - 94%	Unconsolidated topsoil/alluvium.
5.00 – 9.70	Compact Brown Clay: Becomes plastic upon contact with water.	100%	Potential slip surface if saturated.
9.70 – 16.40	Heterogeneous Mixture: Limestone fragments (blocks, gravel) weakly cemented by a clay-sandy matrix.	96% - 100%	Transition zone with variable mechanical strength.
16.40 – 20.00	Compact Brown Clay: Becomes plastic upon contact with water.	100%	Secondary cohesive clay Stratum.
20.00 – 24.60	Compact Brown Clay: Associated with clay-sandy passages containing limestone gravel and pebbles.	100%	Increased coarse content within the clay matrix.
24.60 – 25.40	Hard Limestone Block: Grey color.	100%	Isolated high-stiffness inclusion.
25.40 – 27.00	Sandy Clay: Brown color.	100%	Higher sand fraction compared to upper Strata.
27.00 – 38.55	Compact Brown Clay: Becomes plastic upon contact with water; contains grey marly spots.	100%	Deep marly clay horizon.
38.55 – 40.00	Decomposed Level: Composed of sandy clay and gravel.	100%	Completely weathered/destructured zone.
40.00 – 44.00	Compact Brown Clay: Interbedded with clay-sandy passages containing gravel and limestone pebbles.	100%	Deep cohesive Stratum with inclusions.
44.00 – 52.40	Limestone/Clay Mixture: High abundance of hard grey limestone fragments within a soft brown clay matrix.	100%	Bottom of borehole; highly heterogeneous rock/soil mix.

A.1.8 Borehole SCD-02

Table 97 : Geotechnical Logging and Lithological Description of Borehole SCD-08.

Depth (m)	Lithological Description	Recovery (%)	RQD (%)	Technical Observations
0.00 – 1.90	Hard Limestone: Pinkish-grey, highly fractured, weathered and oxidized. Contains both open and calcite-filled fissures.	63%	11%	Very poor rock quality (surface weathering).
1.90 – 6.20	Crushed Zone (Zone Broyée): Mixture of variegated clay (bariolée) and weathered limestone blocks with gypsum traces.	63% - 94%	11% - 37%	Poor quality; tectonic/diapiric influence likely.
6.20 – 13.30	Hard Limestone: Pinkish-grey, slightly fractured, with decimetric crushed/oxidized passages; rich in recrystallized calcite.	94% - 100%	37% - 91%	Quality improves significantly with depth.
13.30 – 20.00	Hard Limestone: Grey to pinkish-grey, slightly fractured, rich in microfossils.	100%	88% - 91%	Good to excellent rock mass quality.

ANNEX 02: Laboratory Test Results

B.2.1 Borehole SCD-02

B.2.1.1 Physical Properties and Water Content

Table 98 : Physical Properties and Density Parameters of Soil Samples for Borehole SCD-02(NA 5210: 1992; NF P 94-050: 1995).

Borehole (Sondage)	Depth (m)	Water Content (%)	Wet Density ρ_h (t/m ³)	Dry Density ρ_s (t/m ³)
SCD-02	7.00 - 6.45	12.21	2.14	1.91
	18.35 - 18.00	6.02	2.23	2.1
	21.35 - 21.00	10.58	2.16	1.96

B.2.1.2 Particle Size Distribution

Table 99 : Grain Size Distribution and Fine Fraction (< 80 μ m) for Borehole SCD-02 (NA 5232: 2007; NA 5251: 2009).

Borehole (Sondage)	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	80 μ m
SCD-02	7.00 - 6.45	4	10	40	46	88.92
	18.35 - 18.00	—	48	23	29	53.68
	21.35 - 21.00	11	41	11	37	48.56

B.2.1.3 Atterberg Limits

Table 100 : Atterberg limits (LL, PL) and plasticity index (IP) test results for borehole SCD-02 (NA 16213).

DEPTH (M)		6.45 - 7						
THE LIQUIDITY LIMIT								
MEASURE NO.		1	2	3	4	LL (%)		
PENETRATION (MM)		13.95	16.92	17.25	19.8			
MOISTURE CONTENT		33.18	34.67	35.26	39.85		35.76	
THE PLASTIC LIMIT								
MEASURE NO.		1	2	3	4	5	PL (%)	
MOISTURE CONTENT		19.25	19.65	19.74				19.54667
IP		16.22						
DEPTH (M)		18 - 18.35						
THE LIQUIDITY LIMIT								
MEASURE NO.		1	2	3	4	LL (%)		
PENETRATION (MM)		13.57	16.48	18.84	21.75			
MOISTURE CONTENT		32.79	34.56	35.85	38.24		34.93	
THE PLASTIC LIMIT								
MEASURE NO.		1	2	3	4	5	PL (%)	
MOISTURE CONTENT		18.21	18.32					18.265
IP		16.66						
DEPTH (M)		21 - 21.35						
THE LIQUIDITY LIMIT								
MEASURE NO.		1	2	3	4	LL (%)		
PENETRATION (MM)		13.51	15.99	18.26	21.81			
MOISTURE CONTENT		42.29	46.56	50.58	56.19		48.25	
THE PLASTIC LIMIT								
MEASURE NO.		1	2	3	4	5	PL (%)	
MOISTURE CONTENT		26.4	26.91					26.655
IP		21.59						

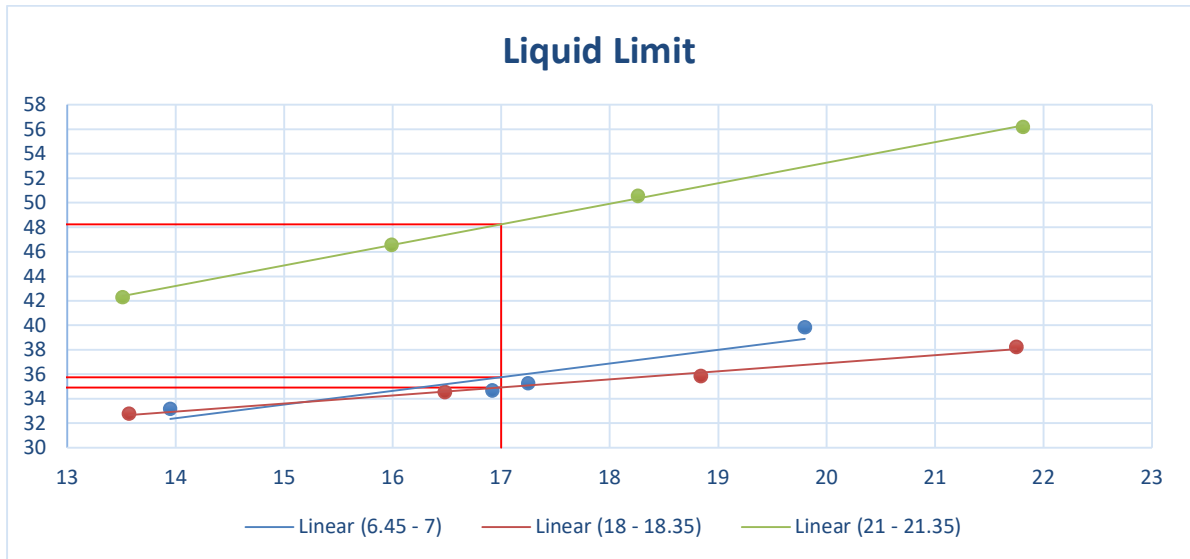


Figure 192 : Liquid Limit Curves for Soil Samples at Different Depths – Borehole SCD-02.

Table 101 : Summary of Atterberg Limits and Soil Classification for borehole SCD-02.

Borehole (Sondage)	Depth (m)	Liquid Limit (WL)	Plastic Limit (Wp)	Plasticity Index (PI)
SCD-02	7.00 - 6.45	35.8	19.45	16.35
	18.35 - 18.00	35	18.26	16.74
	21.35 - 21.00	48	26.66	21.34

Table 102 : General Statistics for borehole SCD-02 Properties.

Parameter	Water Content (%)	Bulk Density ρ_h (t/m ³)	Dry Density ρ_s (t/m ³)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit WL (%)	Plasticity Index PI
Maximum	12.21	2.23	2.1	48	40	46	48	21.34
Minimum	6.02	2.14	1.91	10	11	29	35	16.35
Average (Mean)	9.6	2.18	1.99	33	24.67	37.33	39.6	18.14

B.2.1.4 Chemical Analysis

Table 103 : Chemical Analysis Results of Soil Samples from Borehole SCD-02 (NF P 94-048).

Borehole (Sondage)	Depth (m)	Gypsum Content (%)	SO ₄ (%)	SO ₄ (mg/kg)	Chlorides (%)	CaCO ₃ (%)	CO ₂ (%)
SCD-02	7.00 - 6.45	Traces	Traces	Traces	0.57	5.75	2.51
	18.35 - 18.00	12.35	6.89	68,919.04	0.71	5.83	2.54
	21.35 - 21.00	8.91	4.97	49,773.58	0.53	5.37	2.34

B.2.2 Borehole SCD-03

B.2.2.1 Physical Properties and Water Content

Table 104 : Physical properties (Water content and densities) for borehole SCD-03 samples at different depths (NA 5210: 1992; NF P 94-050: 1995).

Borehole (Sondage)	Depth (m)	Water Content (%)	Wet Density ρ_h (t/m ³)	Dry Density ρ_s (t/m ³)
SCD-03	8.40 - 9.00	14.69	2.19	1.91
	11.35 - 11.80	15.02	2.17	1.89
	27.00 - 27.50	15.57	2.16	1.87
	31.00 - 31.50	14.11	2.2	1.93
	48.00 - 48.60	13.37	2.22	1.96

B.2.2.2 Particle Size Distribution

Table 105 : Grain Size Distribution and Fine Fraction (< 80 µm) for Borehole SCD-03 (NA 5232: 2007; NA 5251: 2009).

Borehole (Sondage)	Depth (m)	Sand (%)	Silt (%)	Clay (%)	< 80µm
SCD-03	8.40 - 9.00	8	36	56	96.27
	11.35 - 11.80	8	36	56	96.05
	27.00 - 27.50	4	32	64	98.59
	31.00 - 31.50	4	38	58	97.38
	48.00 - 48.60	10	40	50	92.38

B.2.2.3 Atterberg Limits

Table 106 : Atterberg limits (LL, PL) and plasticity index (IP) test results for borehole SCD-03 (NA 16213).

DEPTH (M)		8.4 - 9				
THE LIQUIDITY LIMIT						
MEASURE NO.	1	2	3	4	LL (%)	
PENETRATION (MM)	12	14.4	18.15	22.1		
MOISTURE CONTENT	45.58	48.77	56.56	59.68	53.14	
THE PLASTIC LIMIT						
MEASURE NO.	1	2	3	4	5	PL (%)
MOISTURE CONTENT	26.47	26.5				26.485
IP	26.66					
DEPTH (M)		11.35 - 11.8				
THE LIQUIDITY LIMIT						
MEASURE NO.	1	2	3	4	LL (%)	
PENETRATION (MM)	12.37	15.76	19.12	22.45		
MOISTURE CONTENT	42.8	45.58	49	57.08	48.03	
THE PLASTIC LIMIT						
MEASURE NO.	1	2	3	4	5	PL (%)
MOISTURE CONTENT	26.45	26.6				26.525
IP	21.51					
DEPTH (M)		27 - 27.5				
THE LIQUIDITY LIMIT						
MEASURE NO.	1	2	3	4	LL (%)	
PENETRATION (MM)	12.9	16.45	20	22.65		
MOISTURE CONTENT	46.72	53.84	55.93	57.58	52.45	
THE PLASTIC LIMIT						
MEASURE NO.	1	2	3	4	5	PL (%)
MOISTURE CONTENT	27.77	27.19				27.48
IP	24.97					
DEPTH (M)		31 - 31.5				
THE LIQUIDITY LIMIT						
MEASURE NO.	1	2	3	4	LL (%)	
PENETRATION (MM)	12.1	14.2	18.5	22		
MOISTURE CONTENT	43.17	45.41	51.86	57.58	49.95	
THE PLASTIC LIMIT						
MEASURE NO.	1	2	3	4	5	PL (%)
MOISTURE CONTENT	23.26	22.82				23.04
IP	26.91					
DEPTH (M)		48 – 48.6				
THE LIQUIDITY LIMIT						
MEASURE NO.	1	2	3	4	LL (%)	
PENETRATION (MM)	12.35	15.3	19.5	22.4		
MOISTURE CONTENT	40.98	43.34	47.2	50.69	45.18	
THE PLASTIC LIMIT						
MEASURE NO.	1	2	3	4	5	PL (%)
MOISTURE CONTENT	27.66	27.69				27.675
IP	17.51					

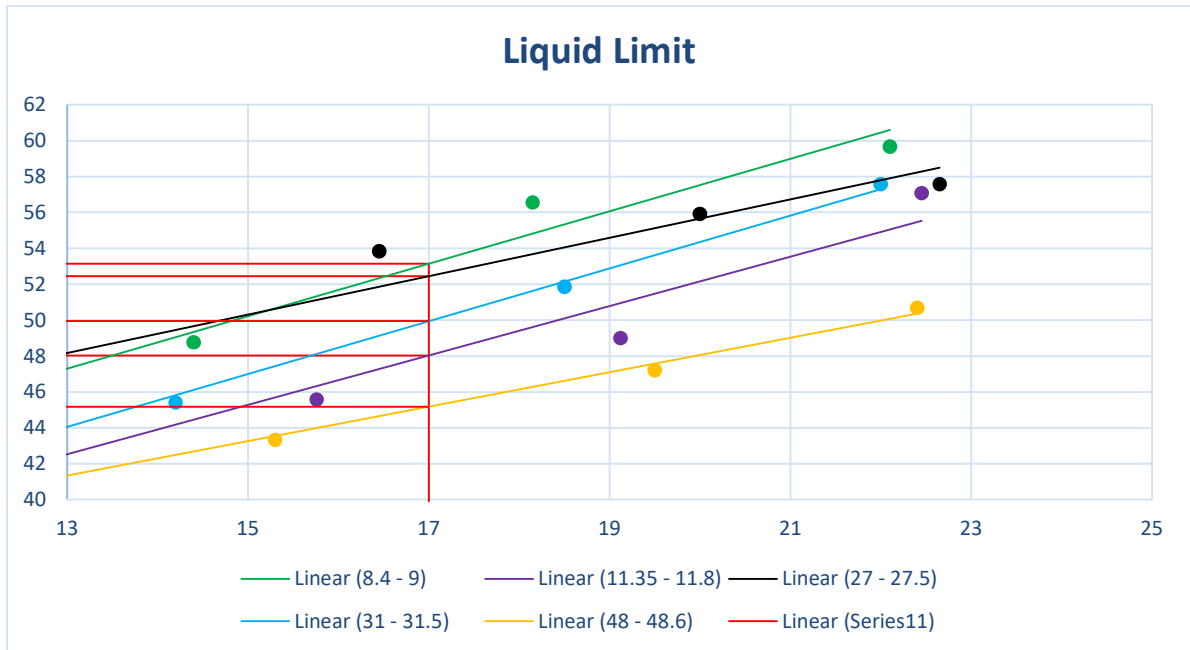


Figure 193 : Liquid Limit Curves for Soil Samples at Different Depths – Borehole SCD-03.

Table 107 : Summary of Atterberg Limits and Soil Classification.

Borehole (Sondage)	Depth (m)	Liquid Limit (WL)	Plastic Limit (Wp)	Plasticity Index (PI)
SCD-03	8.40 - 9.00	53	26.49	26.51
	11.35 - 11.80	48	26.53	21.47
	27.00 - 27.50	52.5	27.48	25.02
	31.00 - 31.50	50	23.04	26.96
	48.00 - 48.60	45	27.67	17.33

B.2.2.4 General Statistics for SCD-03 Properties

Table 108 : Statistical Summary of Physical and Plasticity Properties of Tested Soil.

Parameter	Water Content (%)	Bulk Density ρ_h (t/m ³)	Dry Density ρ_s (t/m ³)	Clay (%)	Liquid Limit WL (%)	Plasticity Index PI
Maximum	15.57	2.22	1.96	64	53	26.96
Minimum	13.37	2.16	1.87	50	45	17.33
Average (Mean)	14.55	2.19	1.91	56.8	49.7	23.46

B.2.2.5 Shear Strength Parameters (Cohesion and Friction Angle) for SCD-03

Table 109 : Shear Strength Parameters (c , ϕ) with Depth for Borehole SCD-03 (NA 16218: 2006).

Borehole	Depth (m)	Cohesion - C (Kpa)	Friction Angle - ϕ (°)
SCD-03	8.40 - 9.00	210	17.47
	11.35 - 11.80	275	20.45
	27.00 - 27.50	280	20.52
	31.00 - 31.50	290	22.45
	48.00 - 48.60	320	24.03

Statistics of shear parameters:

Table 110 : Statistical Summary of Shear Strength Parameters (C and ϕ) for the Tested Soil.

Metric	Cohesion - C (Kpa)	Friction Angle - ϕ (°)
Maximum	320	24.03
Minimum	210	17.47
Average (Mean)	275	20.98

B.2.2.6 Oedometer Test Results for Borehole SCD-03

Table 111 : Results of oedometer tests for borehole SCD-03 (XP P94-090-1).

Sample Code	2024-SS-134	2024-SS-136	2024-SS-142	2024-SS-144	2024-SS-150
Borehole	SCD 03	SCD 03	SCD 03	SCD 03	SCD 03
Depth (m)	8.40 – 9.00	11.30 – 11.80	27.00 – 27.50	31.00 – 31.50	48.00 – 48.50
e _f (final void ratio)	0.4436	0.424	0.4463	0.5051	0.4036
e _o (initial void ratio)	0.434	0.419	0.42	0.459	0.379
σ' _{vo} (kPa)	196.8	256.27	593.86	693.71	781.09
σ' _p (kPa)	390	510	1200	1100	1550
C _c	0.096	0.086	0.125	0.176	0.15
C _s	0.009	0.006	0.016	0.03	0.015

B.2.3 Borehole SCD-04

B.2.3.1 UCS (Unconfined Compressive Strength)

Table 112 : Summary of UCS test results for borehole SCD-04 (NF 1926: 2007).

Code	SC04-2024_SS_155 (04,90-05,60 m)		SC04-2024_SS_156 (13,78-14,21 m)		SC04-2024_SS_158 (22,55-23,05 m)		SC04-2024_SS_161 (36,50-36,90 m)		SC04-2024_SS_163 (48,35-48,80 m)		
No.	1		2		3		4		5		
Height (mm)	87.6	86.9	82.3	80.7	89.3	82.6	81.5	78	69.1	77.1	74.4
Diameter (mm)	92.8	92.8	79.2	79.2	79.2	79.2	78.2	78.2	77.7	76.5	77.8
h/d Ratio	0.94	0.94	1.04	1.02	1.13	1.04	1.04	1	0.89	1.01	0.96
Weight (kg)	1.503	1.376	1.028	1.021	1.113	1.049	1.053	1.005	0.766	0.883	0.822
Density (g/cm ³)	2.54	2.34	2.54	2.57	2.53	2.58	2.69	2.68	2.34	2.49	2.32
Failure Load (kN)	446.6	243.9	377.4	344.1	692.4	524.9	234.8	300.2	94.5	163.9	101.1
UCS (MPa)	66	36.1	76.6	69.8	140.5	106.5	48.5	62	19.9	34.6	21.3
Average UCS (MPa)	51.1		72.3		123.5		55.3		28		
Std . Deviation (S) (MPa)	21.1		4.8		24		9.5		9.4		
Coeff. of Variation (V) (%)	41.4		6.6		19.5		17.3		33.6		

B.2.3.2 Tensile Strength Test

Table 113 : Physical dimensions and splitting tensile strength test results for borehole SCD-04 (NF P 94-422).

Sample Codes	Mean Length L (mm)	Mean Diameter d (mm)	Failure Load F (N)	Splitting Tensile Strength f_{ct} (MPa)	Average Strength (MPa)
SC04-2024_SS_155 (04.90-05.60 m)	84.7	92.8	92.6	7.5	6.3
	93.3	92.8	69.3	5.1	
SC04-2024_SS_156 (13.78-14.21 m)	77.1	79.2	76.8	8	6.7
	80.4	79.2	52.7	5.3	
SC04-2024_SS_158 (22.55-23.05 m)	83.9	79.2	52.5	5	4.7
	84.8	79.2	46.7	4.4	
SC04-2024_SS_161 (36.50-36.90 m)	80.8	78.5	51.7	5.2	6
	78.9	78.5	64.7	6.7	
SC04-2024_SS_163 (48.35-48.80 m)	73.8	77.7	22.4	2.5	2.4
	69.4	77.8	18.4	2.2	

Table 114 : Summary of Mechanical Test Statistics for SCD-04.

Metric	Bulk Density ρ_h (t/m ³)	UCS (MPa)	Average UCS (MPa)	Tensile Strength (MPa)	Average Tensile Strength (MPa)
Maximum	2.69	140.5	123.5	8	6.7
Minimum	2.32	19.9	28	2.2	2.4
Average (Mean)	2.51	61.98	66.22	5.19	5.22

B.2.4 Borehole SCD-05

B.2.4.1 UCS (Unconfined Compressive Strength) Test

Table 115 : Summary of UCS test results for borehole SCD-05 (NF 1926: 2007).

Code	SC05 2024_SS_168 08,30-08,97 m		SC05 2024_SS_170 16,23-16,95 m		SC05 2024-SS-172 26,20-26,90 m		SC05 2024_SS_174 38,50-39,00 m		SC05 2024_SS_176 45,07-45,87 m	
	1		2		3		4		5	
No.										
Height (mm)	95,0	92,8	78,7	78,6	79,1	77,7	79,9	82,8	77,5	
Diameter (mm)	92,8	92,8	78,5	78,5	78,5	78,5	78,5	78,5	78,5	
h/d Ratio	1,02	0,99	1,00	1,00	1,01	0,99	1,02	1,05	0,99	
Weight (kg)	1,696	1,633	1,030	1,024	0,987	1,047	1,047	0,944	0,937	
Density (g/cm ³)	2,64	2,62	2,70	2,69	2,58	2,78	2,71	2,36	2,50	
Failure Load (kN)	514,0	570,1	526,2	315,3	277,2	499,9	412,5	323,1	331,5	
UCS (MPa)	76,0	84,3	108,7	65,1	57,3	103,3	85,2	66,8	68,5	
Average UCS (MPa)	80,1		86,9		/		85,2		67,7	
Std .Deviation (S) (MPa)	5,9		30,8		/		12,8		1,8	
Coeff. of Variation (V) (%)	7,3		35,4		/		15,0		1,2	

B.2.4.2 Tensile Strength Test

Table 116 : Physical dimensions and splitting tensile strength test results for borehole SCD-05 (NF P 94-422).

Specimen Codes	Average Length L (mm)	Average Diameter d (mm)	Failure Load F (kN)	Splitting Tensile Strength (f_t) (MPa)	Average (MPa)
SCO5 2024_SS_168 08,30–08,97 m	91,7	92,8	33,2	2,5	2,5
	94,7	92,8	32,4	2.4	
SCO5 2024_SS_170 16,23–16,95 m	79,2	78,5	80.8	8,3	6,8
	79,6	78,5	50,7	5,2	
SCO5 2024_SS_172 26,20–26,90 m	79.6	78,5	67,1	6,8	6,8
SCO5 2024_SS_174 38,50–36,00 m	77,7	78,5	77.8	7,9	7.5
	79.9	78.5	69.1	7	
SCO5 2024_SS_176 45,07–45,87 m	82,8	78.5	49.5	4.8	4,8
	77.5	78,5	49.8	4.8	

Table 117 : Summary of Mechanical Test Statistics for SCD-05 (NF P 94-422).

Metric	Bulk Density γ_h (t/m ³)	UCS (MPa)	Average UCS (MPa)	Tensile Strength (MPa)	Average Tensile Strength (MPa)
Maximum	2.78	108.7	86.9	8.3	7.5
Minimum	2.36	57.3	67.7	2.4	2.5
Average (Mean)	2.62	79.47	79.98	5.52	5.4