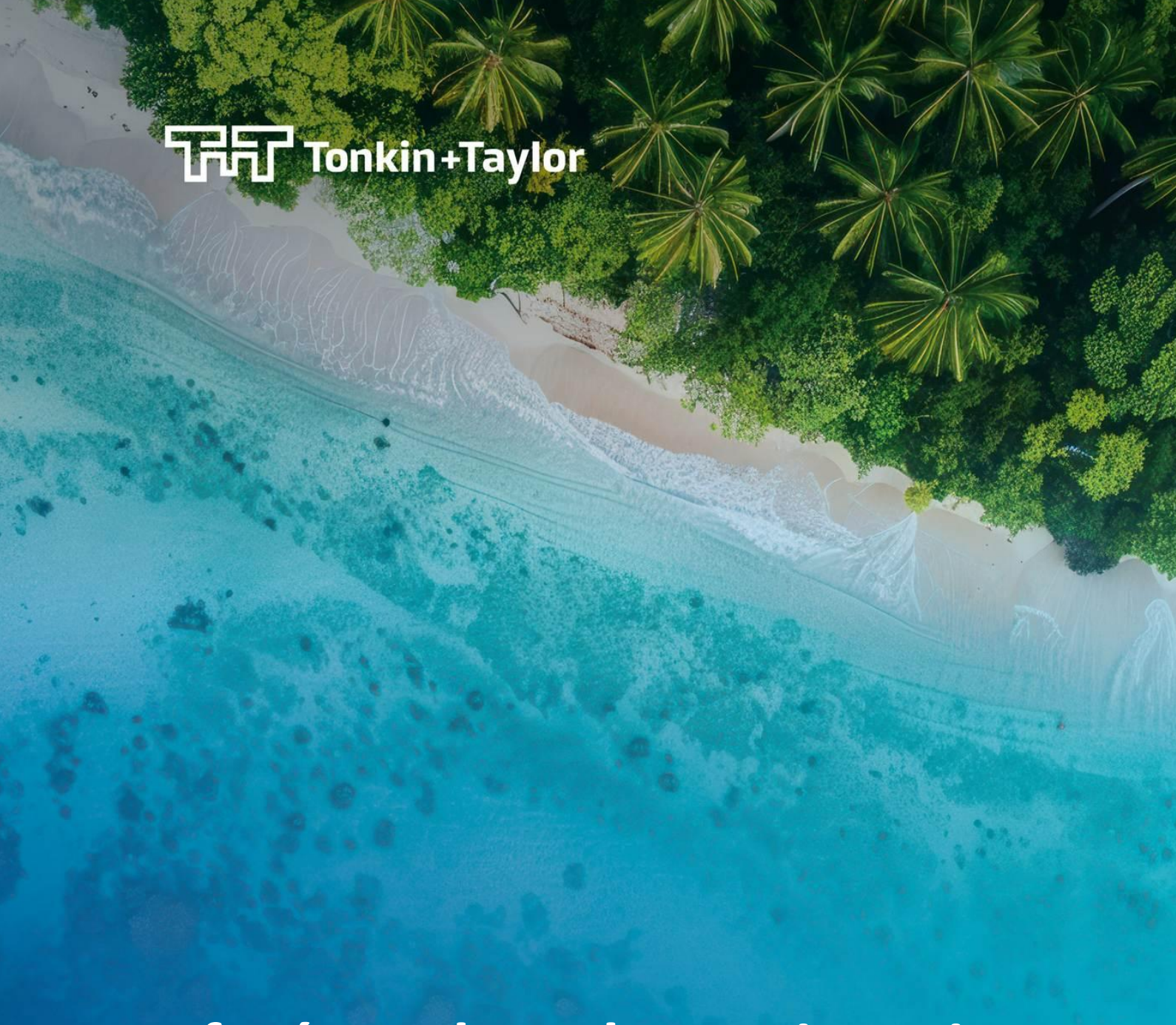

Efaté Earthquake Engineering Technical Assistance

Port Vila CBD Geotechnical Hazard Study: Geotechnical Assessment

Prepared for: Egis NZ Limited
Prepared by: Tonkin & Taylor International Ltd
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Executive Summary

The 17 December 2024 earthquake (magnitude Mw 7.3), centred about 25 km from Port Vila, caused loss of life and widespread damage to buildings and infrastructure across the city. Tonkin & Taylor International Ltd (T+TI) was engaged by Egis NZ Limited, under the New Zealand Ministry of Foreign Affairs and Trade (MFAT) Efaté Earthquake Engineering Technical Assistance (EEETA) programme, to assess the ground-related (geotechnical) earthquake hazards across the Port Vila central business district (CBD) to support the Port Vila City Council (PVCC) and Government of Vanuatu (GoV).

This report outlines where each geohazard should be considered within the Study Area. It forms the technical basis for the Geotechnical Guidance Document, which sets out in practical terms the investigation, foundation and design measures appropriate to manage these hazards for future development across the city.

The Study Area has been delineated by development type and density of geotechnical investigation, as follows:

- A Detailed Study Area covering the downtown CBD, where high-density, multi-storey development is present. This was the primary focus of geotechnical investigations; and
- A Broader Study Area of lower-density development, generally below three storeys, assessed primarily from existing data and geomorphic mapping.

This report brings together an enhanced understanding of the ground beneath the Port Vila CBD, based on 11 new boreholes, 7 km of geophysical surveys along the city streets, groundwater monitoring, laboratory testing, post-earthquake aerial (LiDAR) surveys, in addition to approximately 90 historic investigations.

From this information a three-dimensional geological model of the CBD has been developed, and the Broader Study Area has been divided into four ground zones (Zones A to D). The four ground zones are shown in the figures below.

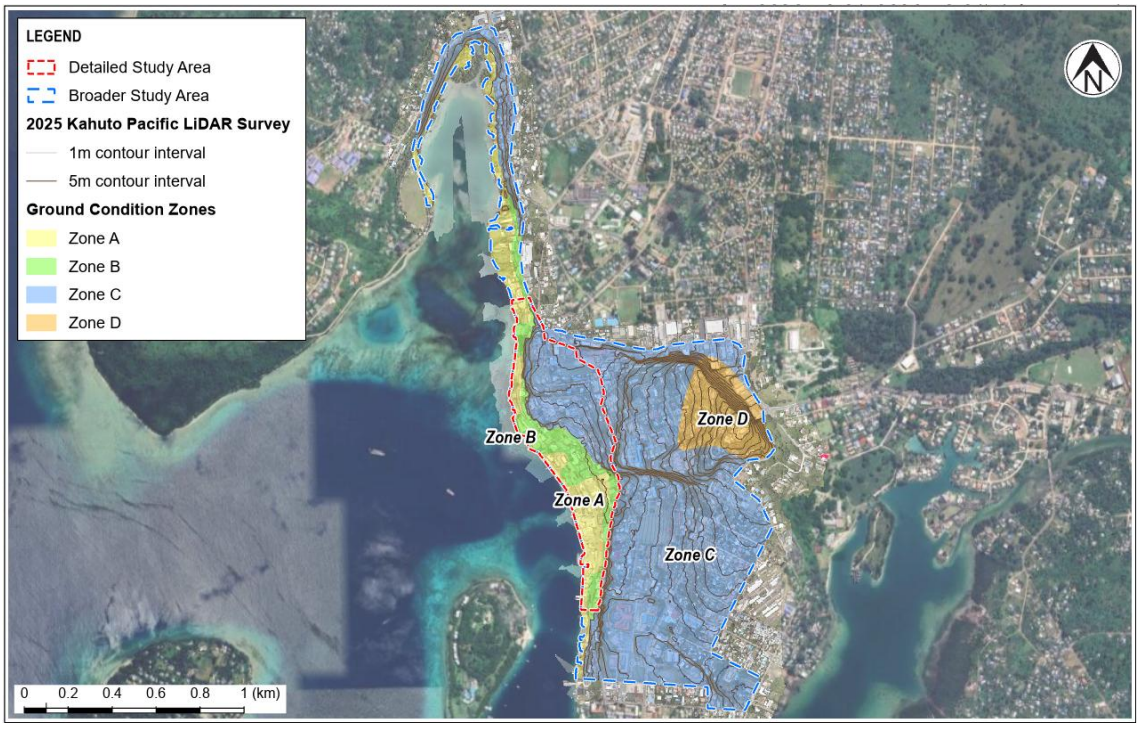


Figure ES.1: Ground condition zoning of the Study Area, Zones A to D.

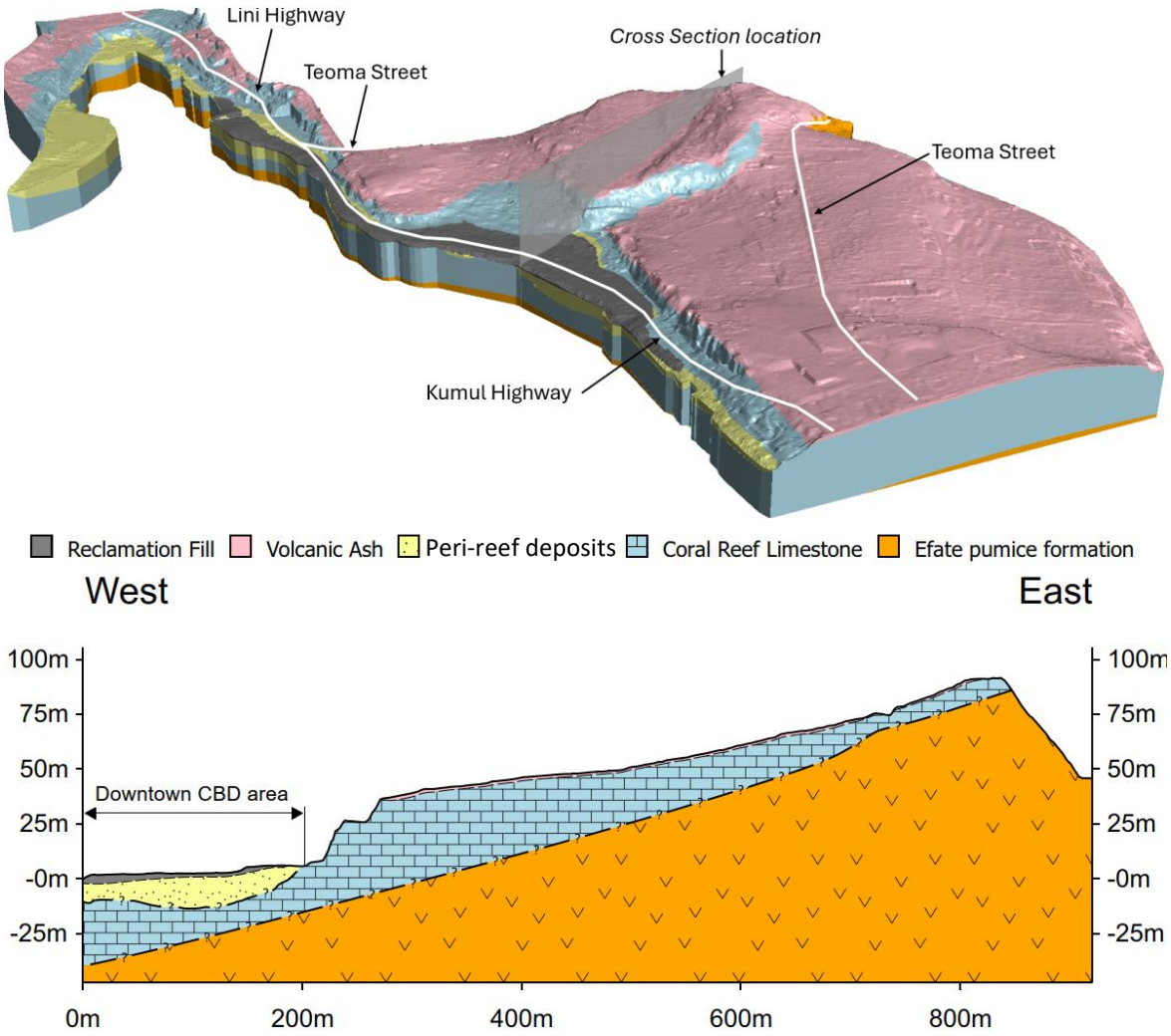


Figure ES.2: Three-dimensional geological model and schematic west-east section across the CBD, showing the reclaimed waterfront ground (Zones A/B) over peri-reef deposits and limestone, rising to the limestone terraces inland (Zones C/D).

Zones A and B occupy the low-lying, reclaimed coastal margin, where up to about 6 m of loose Reclamation Fill overlies variable peri-reef (coral sand and gravel) deposits and then competent Reef Limestone Formation, with a shallow, tidally-influenced groundwater table about 1 to 5 m below the surface. Zone A is characterised by thicker Reclamation Fill and extensive peri-reef deposits, whereas Zone B generally comprises shallower Reclamation Fill with possible localised peri-reef deposits (refer to Section 3.4 and Table 3.1 for further detail). Zones C and D occupy the higher terraces inland, where competent Reef Limestone Formation lies at or close to the surface beneath only a thin veneer of fill and volcanic ash, and groundwater is deep. The loose, saturated deposits found in Zone A, and parts of Zone B, are more vulnerable to earthquake-related ground damage than the competent limestone underlying the elevated parts of the city in Zones C and D. This contrast is consistent with much of the damage observed along the waterfront in December 2024.

Four earthquake-related hazards pertinent to the ground conditions encountered are ground shaking, liquefaction, slope instability, and fault surface rupture. The hazards are described and their implications on the rebuild are summarised in the table below.

Summary of assessed earthquake-related geotechnical hazards

Geohazard	Assessed areas	Description of the hazard
Ground shaking *(refer Section 4 for further details, and Appendix A for zones)*	Full Study Area	The ground shaking hazard has been estimated based on seismic hazard assessment specific to the Study Area (provided in Appendix D). Port Vila has a high ground shaking hazard in a global context, predominantly driven by large magnitude earthquakes along the Vanuatu Subduction Zone and within the subducting plate. Peak ground acceleration (PGA) and earthquake shaking for short and/or stiff structures have relatively small variability over the Study Area. However, shaking for tall and/or flexible structures is more influenced by underlying ground conditions. Areas underlain by softer soils (such as Zone A) are likely to further amplify earthquake shaking acting on tall and/or flexible structures.
Liquefaction *(refer Section 5 for further details, and Appendix A for zones)*	Zones A and B	Saturated, cohesionless soils are susceptible to liquefaction hazard. Reclamation Fill and Intra-reef deposits (subunits of peri-reef deposits) underlying Zone A and in localised areas in Zone B are assessed to be susceptible to liquefaction. Loose to medium dense soils are likely to liquefy at PGA of 0.2 to 0.6g, equivalent to 1 in 25 to 1 in 100 AEP levels of earthquake shaking. Consequences of liquefaction and lateral spread include vertical settlement and lateral ground deformation. Areas near the coast within Zones A and B are also susceptible to lateral spread. Vulnerability of structures to ground deformation will vary based on the form and weight of the structure and location of the structure.
Slope instability *(refer Section 6 for further details, and Appendix B for zones)*	Slope susceptibility zones	Steep slopes (generally greater than 45°) and man-made slopes (e.g. cutting into the hillside for development area) are likely to be susceptible to slope instability due to earthquake shaking, high groundwater pressure within rock defects following significant rain events and/or due to natural erosion. Based on observed performance following the Port Vila earthquake, slope instability within susceptible areas are typically limited to rockfalls or topples. Figures provided in Appendix B present slope runout and regression envelopes based on assessed slope instability for slopes greater than 4 m in height and 10 m in width. The slope runout zones vary between 10 m and 40 m over the Study Area, while the slope regression was generally limited to 5 m to 15 m in width at the crest of the slope.
Fault surface rupture *(refer Section 7 for further details, and Appendix C for overlays)*	Fault deformation overlay	Several east–west trending faults cross the Study Area and are assessed as possibly active. Ground deformation of approximately 100 to 400 mm occurred along sections of two of these faults (the Tank Hill and Palace Faults) during the December 2024 earthquake, interpreted as triggered slip rather than primary fault rupture. Indicative single-event displacements in future events on faults within the Study Area are expected to be in the order of 180 to 340 mm. A fault deformation overlay is recommended around the mapped fault traces (the greater of 20 m either side of the trace, or extension to the crest of a visible escarpment). Development within the overlay is not automatically precluded. The overlay should trigger site-specific assessment and design for ground deformation (e.g. well-tied, robust foundations). Critical infrastructure and post-disaster facilities should be located outside the overlay where practicable.

The geotechnical hazards assessed and presented in this report can be managed and mitigated through planning, design and construction to achieve resilient rebuilding of the Port Vila CBD by applying these three measures:

1 Investigating the ground before building, so that designs match the actual site conditions.

2 Pragmatic engineering, such as strong, well-tied foundations that can tolerate some ground movement.

3 Risk-based planning, applying greater scrutiny to large, important or critical facilities.

The figures within the appendices of this report show where each hazard needs to be considered and what level of assessment is appropriate.

The zoning maps and data are available digitally to support planning and consenting decisions[1]. This report is an area-wide study and does not remove the need for site-specific investigation, but it provides a consistent framework to support a more resilient rebuild of Port Vila.

[1] Port Vila Geohazard Viewer, https://portvilageohazardviewer.com/

1 Introduction

Tonkin & Taylor International Ltd (T+TI) has been engaged by Egis NZ Limited (Egis) to provide specialist geotechnical engineering services for the New Zealand Ministry of Foreign Affairs and Trade (MFAT) Efaté Earthquake Engineering Technical Assistance (EEETA) programme. This programme supports the Government of Vanuatu (GoV) in its recovery efforts following the 17 December 2024 earthquake, which caused significant damage to buildings and infrastructure in Port Vila.

As part of the programme, a geotechnical hazard study (the Study) was commissioned to help increase the long-term resilience of Port Vila's built environment. The Study Area is shown on Figure 1.1 and summarised in Section 2.5.

The Study has been delivered in the following stages:

1 Data Compilation and stakeholder collaboration
2 Geotechnical investigation
3 Ground zoning
4 Geotechnical Guidance Document
5 Digital deployment to a GIS portal

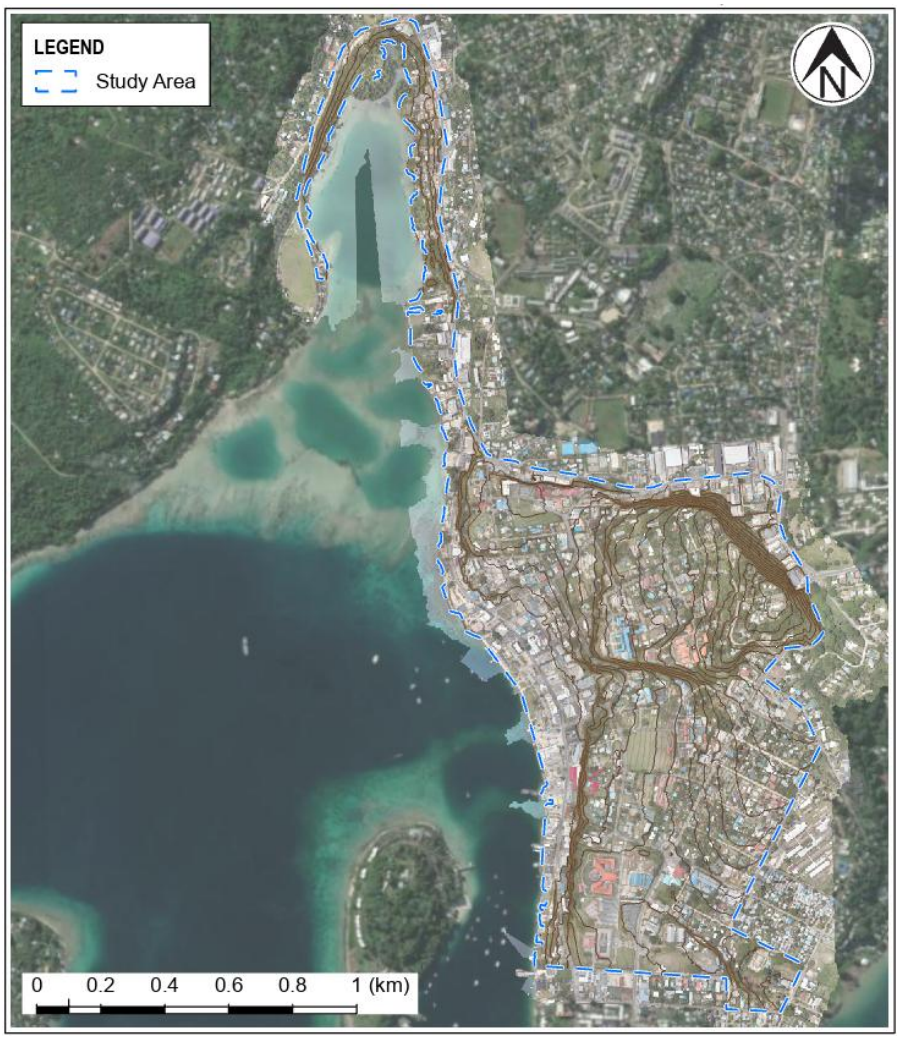


Figure 1.1: Study Area (outlined blue).

2 Context

2.1 Port Vila context

Port Vila is Vanuatu's capital and largest city, located on the island of Efaté, with a population of approximately 49,000 (2020 Vanuatu census). It is the economic and cultural hub of Vanuatu. Its CBD is the nation's principal centre of commerce, government, tourism, and trade, including the country's primary harbour and wharves. The CBD comprises a dense mix of government offices, businesses, banks, hotels, and a thriving local market.

Port Vila faces significant and long-standing urban management challenges. Sustained in-migration from the outer islands has driven urbanisation that has outpaced the provision of infrastructure and services. Unplanned and unregulated development has resulted in urban sprawl and informal settlements in hazard-prone areas (Port Vila CBD Vision and Concept Plan, July 2025).

However, as outlined in the Port Vila CBD Vision and Concept Plan: Earthquake Recovery and Rebuilding – 2035 and Beyond, the recovery efforts from the devastating Port Vila 2024 earthquake present a unique opportunity to '*build back better'.* The document presents a vision to transform the CBD into a *dynamic, pedestrian-friendly hub that celebrates its cultural diversity, fosters economic growth, and prioritises sustainability as illustrated on* Figure 2.1.



Figure 2.1: Recovery and renewal concept plan showing proposed strategic projects across CBD area (Port Vila CBD Vision and Concept Plan, July 2025).

Vanuatu and Port Vila rely significantly on donor funding for infrastructure development, often with overseas engineering input particularly for significant infrastructure projects. However, for the normal, day-to-day projects, significant challenges to the recovery efforts include variability in local experience and expertise, and lack of clear guidance to design and construct resilient structures. Vanuatu and Port Vila are vulnerable to significant natural hazards such as earthquakes, tsunami, tropical cyclones. A clear understanding of these hazards, and a consistent approach to designing and constructing for them, is therefore vital to '*build back better*'.

The Vanuatu Building Code Edition 2025 has been recently updated to support the recovery works and to promote consistency in design.

This geotechnical assessment report presents an assessment of geotechnical hazards for Port Vila CBD and will be used to support geotechnical guidelines for the recovery efforts.

2.2 Background to this study

The post-earthquake recovery in Port Vila presents a critical opportunity to guide the rebuilding of the CBD to be more resilient to future natural hazards. This opportunity was identified during initial observations from post-earthquake reconnaissance undertaken by T+TI in March and April 2025 (independent of the MFAT-funded EEETA programme), and by GNS Science (the New Zealand geoscience research institute), which recommended similar actions in their May 2025 report (Massey et al., 2025a).

These initial observations revealed marked variability in ground conditions in specific areas of the CBD, evidenced by distinct patterns of land deformation. There has been substantial uncertainty regarding these conditions due to a lack of consolidated geotechnical data and testing. Characterising these ground conditions was therefore recognised as essential to understanding the geohazards and to guide the adoption of appropriate structural designs for future development. This reduces uncertainty and enables geohazards pertinent to the conditions to be appropriately addressed through design and construction.

As part of the EEETA inception visit to Port Vila from 18 to 23 May 2025, T+TI – alongside MFAT – met with key stakeholders including the Port Vila City Council (PVCC), the Ministry of Infrastructure and Public Utilities (MIPU) and the Department of Urban Affairs and Planning (DUAP) to confirm the need for geotechnical hazard guidance to support the resilient development of buildings within the Port Vila CBD. The EEETA programme aims to provide the engineering guidance necessary to support this resilient long-term redevelopment. Combining visual evidence of ground deformation with subsurface data provides an opportunity to more reliably calibrate earthquake design guidelines for use in future development of the CBD and surrounding areas.

2.3 Key terms and definitions

Key terms and their definition applied in this report are presented in Table 2.1 for clarity.

Table 2.1: Definition of terms

Term	Definition in Hazard Risk Management
Hazard	A naturally occurring or human-induced condition that has the potential to affect to life, property or the environment.
Susceptibility	The relative potential for a hazard to occur at a location, based on the ground conditions present, e.g. the ground is susceptible to liquefaction if it comprises saturated, cohesionless material, and an area is susceptible to rockfall if it is near steep fractured rock slopes.
Triggering	An event or condition (such as earthquake shaking or intense rainfall) that initiates the hazard at a susceptible location.
Consequence	The impacts on life, property or the environment if the hazard occurs.
Likelihood	The chance of a hazard occurring in a given period, usually expressed as an annual probability of occurrence or an average return period. These are averages, not fixed intervals. E.g. a "100-year event" has a 1% chance of occurring in any year and could happen tomorrow even if one occurred recently.
Risk	The combination of the consequence of a hazard and the likelihood of it occurring.

2.4 Intended purpose and scope of works

The purpose of this report is to provide an assessment of the geotechnical hazards triggered by earthquake shaking and fault rupture, to support the development of geotechnical guidelines for the earthquake recovery efforts.

This geotechnical assessment is an area-wide evaluation using limited subsurface investigation data, existing information, and our professional judgement and experience. While the assessment provides an overview of geotechnical conditions and hazards across the Study Area, uncertainties remain at individual site locations. Therefore, site-specific geotechnical investigations are recommended to confirm and refine the geohazards identified in this report at individual sites, and to assess more “localised” geohazards that were not considered in this report (e.g. uncontrolled fill, consolidation settlement, high groundwater, shrink/swell).

The hazards assessed within this assessment can be managed and mitigated through careful design and detailing to achieve resilient rebuilding of the Port Vila CBD. Broad-level mitigation options are discussed in this report.

Furthermore, the findings of this report has been used as the technical basis for the Geotechnical Guidance Document, which sets out in practical terms what investigations, foundations and design measures are appropriate for different types of buildings in each ground zone. The zoning maps and data will also be made available digitally to support planning and consenting decisions. This report does not remove the need for site-specific investigation, but it provides a consistent framework to support a more resilient rebuild of Port Vila.

This report describes the assessment methodology, summarises the findings, and provides recommendations for further investigations and geotechnical assessments within the Study Area.

The report is laid out as follows:

- Section 2.5 and 2.6 present the extent of the Study Area and reports that have been prepared as part of the broader geotechnical services for the Study Area. These reports should be read in conjunction with this report.
- Section 3 sets out the regional and local geological setting based on published information, the 3D geological model developed for the Study Area from geotechnical and geophysical investigations. The section also provides the interpreted stratigraphy of Port Vila and the ground conditions within each assessed ground zone.
- Section 4 sets out the assessment of seismic shaking hazard.
- Section 5 sets out the assessment of liquefaction hazard.
- Section 6 sets out the assessment of slope hazard, including landslide mechanisms, susceptibility mapping, and conclusions and recommendations from the assessment.
- Section 7 sets out the assessment of fault surface rupture hazard.
- Section 8 presents a summary of the geotechnical hazard assessment and opportunities to support planning, design and construction of resilient structures.

2.5 Study extents

This Study defines two distinct areas, that aligns with the types of development observed within each area and the density of pre-existing investigation undertaken as part of this study. These areas were defined based on availability of pre-existing geotechnical data, the distribution of observed damage following the 17 December 2024 earthquake, and the likely type of development. These factors informed the proposed level of investigation for each of the following areas:

- **Broader (high-level) Study Area (Blue outline in Figure 2.2):** This area defines the full spatial extent of the Study, encompassing the wider CBD area provided by the PVCC (M. Fatdal, personal communication, May 21, 2025). Spatially, this area includes the Detailed Study Area, however the term is used herein to denote the broader regional context. Development within this zone is characterised by lower density and likely to generally comprise structures less than three storeys in height. Assessment in this zone relies primarily on compiled historical data and geomorphic mapping, with limited targeted geotechnical investigations undertaken as part of this study.
- **Detailed Study Area (Red outline in Figure 2.2):** This area encompasses the Port Vila Central Business District (CBD) downtown area. Future development in this area is expected to be high-density, with numerous multi-storey buildings, containing the highest economic investment and critical infrastructure, where the overall consequences of ground damage are likely to be more significant to Port Vila. Consequently, this area was the primary focus of the geotechnical investigations, allowing for a better understanding of the ground to support the resilience of existing and future key assets.



Figure 2.2: Detailed Study Area (outlined red) and broader Study Area (outlined blue).

2.6 Supplementary information

Supplementary reports have been prepared as part of this geotechnical hazard study and should be read in conjunction with this report. These reports are summarised in Table 2.2 below. Existing geotechnical information (prior to this study) has been consolidated into the Reconnaissance Report.

Table 2.2: Summary of supplementary reports

Document	Date	Summary
Investigation Implementation Plan	3 November 2025	Sets out the proposed scope, methodology and locations for the geotechnical investigations, providing the planning framework to undertake the boreholes, groundwater monitoring, laboratory testing and geophysical survey.
Reconnaissance Report	18 December 2025	Documents the outcomes of the Request for Information (RFI) workshops and consolidates pre-existing geotechnical, geological and geospatial data from stakeholders and the T+TI database, forming the existing-information baseline that the GFR and this geotechnical assessment builds upon.
Preliminary Ground Zoning Report	9 February 2026	Presents an initial geomorphic and ground-condition zoning of the Study Area developed from existing data and early field observations. That preliminary zoning has been refined and validated in this report.
Investigation progress update and proposed changes	23 February 2026	An interim update on investigation progress and proposed scope adjustments (e.g. deferral of BH01), explaining the changes from the original Implementation Plan that are reflected in the completed scope summarised in the GFR.
Geotechnical Factual Report	8 May 2026	Documents the factual results of the Stage 2 geotechnical investigation undertaken between November 2025 and March 2026, including a LiDAR and aerial photogrammetry survey, machine boreholes, groundwater monitoring, laboratory testing and surface and downhole geophysical surveys. This factual dataset underpins the ground zoning and geohazard assessments presented in this report.

3 Geological model and site setting

3.1 Regional geology and stratigraphy

The island of Efaté is predominantly volcanic in origin and lies within the Tertiary to Recent volcanic island arc system that forms Vanuatu. The volcanic sequence is dominated by the Efaté Pumice Formation which consists of a succession of non-welded pumice breccia and beds of sand and silt deposited during a major eruptive event approximately 1 Ma (million years ago).

The upper section of the Efaté Pumice Formation consists of the Rentabau Tuffs, a sequence of well sorted sand to silt beds. These beds were deposited within a submarine environment, likely as density currents (Raos & McPhie, 2003). These materials are extremely weak to weak in strength.

Upon the cessation of volcanic activity in the Pleistocene (up to 2.5 Ma), abundant fringing coral reef growth occurred around Efaté, overlying the older volcanic deposits. These reef deposits form the Reef Limestone Formation (Raos & McPhie, 2003). Periods of uplift and fluctuating sea levels during glacial and interglacial periods of the Quaternary (2.58 Ma to present) have resulted in the formation of several coral terraces up to 400 m above sea level in Efaté (Lecolle & Bernat, 1985). Large areas of Efaté Island's coastal edge have a capping of coral reefs. The published geological map presented in Figure 3.1 indicates that the Study Area is underlain by recent raised reef, older raised reef and Efaté Pumice Formation.

The broad tectonic setting involves uplift of the island, which sits on the upper (overriding) plate of the Vanuatu Subduction Zone (VSZ). Intra-plate deformation includes extensional normal faulting as evidenced by the normal-faulting source mechanism reported for the December 2024 earthquake, and normal fault grabens that are observed across the island.

The geological fault interpretation of the published geological map (Ash et al., 1978), shown in Figure 3.1, is based on aerial photograph lineament mapping and no indication of relative fault activity has been made. Specifically, as far as we are aware, no intrusive investigations have been undertaken to confirm the details of the geological faults shown on Figure 3.1.

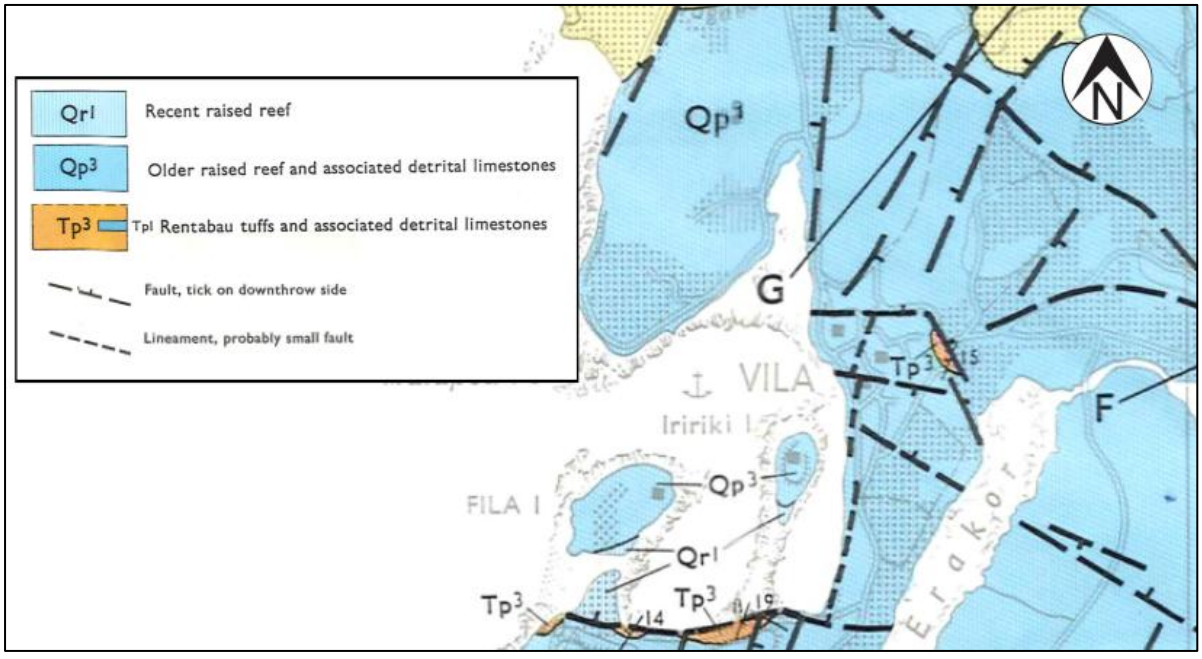


Figure 3.1: Geological setting of Port Vila (Geology of Efaté 1:100,000 New Hebrides Geological Survey Sheet 9).

3.2 Local geomorphology and setting

Based on an interpretation of the 2012 Efaté Light Detection and Ranging (LiDAR) and bathymetric Digital Elevation Model (DEM) (Geoscience Australia, 2012-2013), the broader Study Area occupies a series of five distinct reef terraces, located at elevations of approximately -1 to 5 m, 10 to 20 m, 20 to 35 m, 35 to 60 m, and 80 to 100 m Reduced Level (RL – approximate height in metres relative to mean sea level). These terraces are bounded by steep to subvertical (30 to 90°) N-S trending sea cliffs, approximately 10 to 20 m high, and are dissected by inferred E-W to NE-SW trending fault scarps that appear to vertically offset the terrace morphology by tens of metres.

The CBD downtown area is primarily located on the western fringe of Port Vila on the lowermost terrace (approx. 1 to 5 m RL) bordering the harbour. Significant reclamation, occurring during the colonial era and continuing since the 1970s to support the development of Port Vila, has obscured the natural surface morphology of the downtown area with approximately 1 to 4 metres of fill. However, the underlying morphology can be inferred from the adjacent sub-tidal terrain visible in the 2012 LiDAR and bathymetric datasets (Geoscience Australia, 2012-2013). This imagery reveals a carbonate platform punctuated by significant depressions ranging from 200 to 500 m in width and 5 to 20 m in depth. The intervening reef forms an interconnected framework across the harbour, though it is variable in width, narrowing to as little as 30 m in places.

The internal depressions and margins of the carbonate platform are expected to primarily contain two soil deposits, referred to collectively herein as ‘peri-reef’ deposits. These deposits are distinguishable by their composition, grain size and spatial extent, and include:

- **Intra-reef deposits**: These deposits fill the internal depressions of the carbonate platform, which act as sediment traps, collecting coral gravels and cobbles along with volcanic sediment washed in during storms. The deposits are expected to be interbedded and of mixed composition, ranging from coarse coral gravels and cobbles to finer carbonate sands, and are confined within the depressions between the ribs of the carbonate platform.
- **Fore-reef deposits:** These deposits form the margin of the carbonate platform as a talus slope extending into deeper water. They are expected to consist mainly of coarse, reef-derived material, including coral rubble, cobbles and blocks shed from the reef front, and are limited in extent, forming a relatively narrow apron along the platform edge.

The reclamation fill may also be thicker where it overlies the natural depressions, while remaining relatively shallow across the intact ribs of the platform.

Additionally, within foreshore areas, a shallow mantle of carbonate beach deposits is expected to overlie the carbonate platform surface.

Figure 3.2 below illustrates the geomorphic features summarised above. Figure 3.3 presents a schematic cross section of the inferred subsurface conditions based on review of geomorphic features.

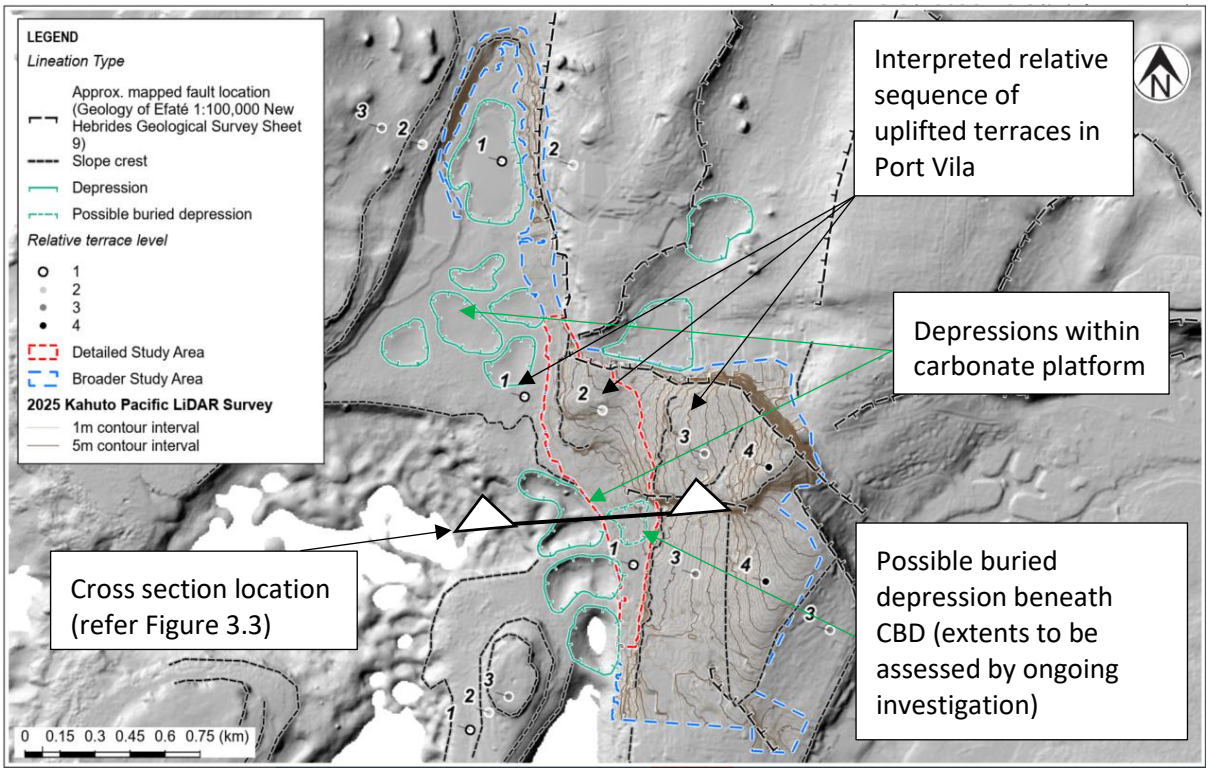


Figure 3.2: Mapping of distinct geomorphic features within the broader Study Area, including slope breaks and intra-reef depressions. Base layer sourced from 2012 Efaté LiDAR and bathymetric DEM (Geoscience Australia, 2012-2013)

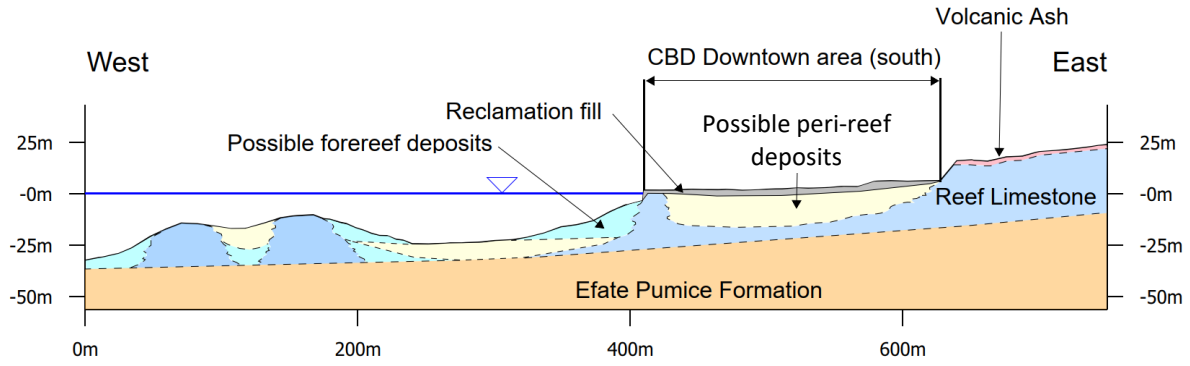


Figure 3.3: Schematic geological cross section through south CBD downtown / waterfront area (west – east).

3.3 Geological model for the Study Area

The geological units encountered in the investigations that form the basis for the 3D ground model for the Study Area are summarised in approximate stratigraphic order from youngest (uppermost) to oldest in Table 3.1.

Summary descriptions and the spatial distribution of these units across the Study Area is described in the subsections below.

Table 3.1: Summary of geological units

Geological unit	Sub-unit
Unit 1: Fill	Unit 1a: Reclamation fill
	Unit 1b: Surficial fill
Unit 2: Volcanic ash	
Unit 3: Peri-reef deposits	Unit 3a: Intra-reef deposits
	Unit 3b: Fore-reef deposits
Unit 4: Reef Limestone Formation	Unit 4a: Recent Reef Limestone
	Unit 4b: Raised Reef Limestone
Unit 5: Efaté Pumice Formation	Unit 5a: Weathered Efaté Pumice Formation
	Unit 5b: Unweathered Efaté Pumice Formation

The 3D ground model was developed using the software package 'Leapfrog' (produced by Seequent Limited). This involved interpretation of historic geotechnical information and recent investigations, including 11 machine boreholes completed by T+TI in 2025 / 2026, supplemented by an additional 90 historic investigations (52 machine boreholes and 38 test pits), field observations, and ground surface and bathymetric survey profiles. This 3D geological model formed the basis for subdividing the Port Vila CBD into the four ground condition zones (Zone A to Zone D) described in Section 3.4.

To develop the ground model, it was necessary to include user generated points and lines to guide the geological surfaces. These user-defined points and lines were placed away from investigation locations, so that the ground model remains consistent with available factual data.

An overview of the broad geological model and a West to East geological cross section through Government Hill and the downtown CBD are presented below in Figure 3.4.

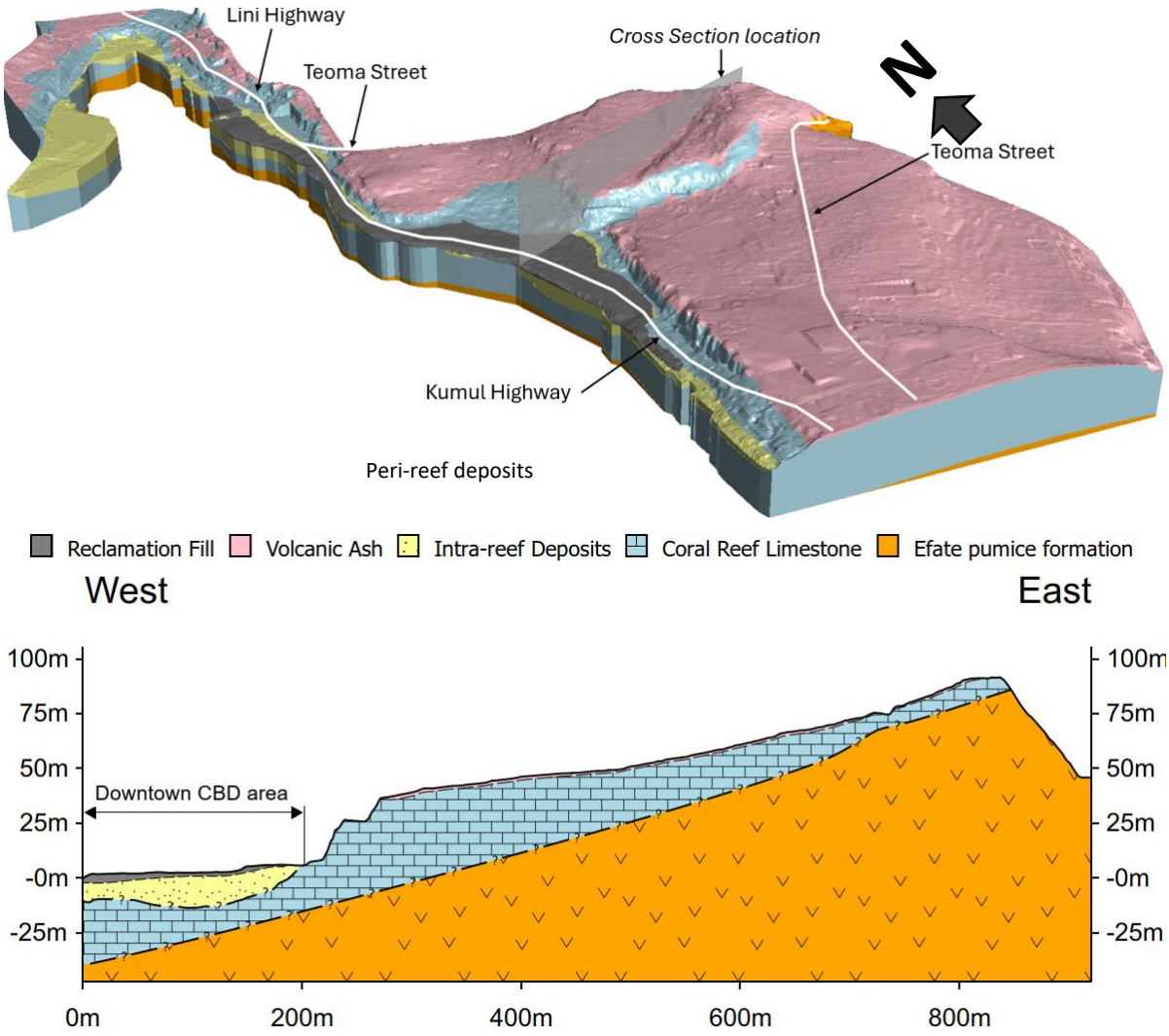


Figure 3.4: 3D geological Model and illustrative cross section of the east-west geology across the CBD

3.4 Lithostratigraphy of the Study Area

On the basis of variations in these units identified in the 3D ground model, the Study Area has been divided into four geologically distinct zones (Zones A to D), shown on Figure 3.5.

In broad terms, the four zones reflect two contrasting settings. Zones A and B occupy the low-lying reclaimed coastal margin, where Reclamation Fill overlies Peri-reef Deposits, which in turn overlie the Reef Limestone Formation. Zones C and D occupy higher ground where the Reef Limestone Formation lies at or near the surface, capped only by a thin veneer of surficial fill and Volcanic Ash.

3.4.1 Fill

Two distinct fill types have been identified, with each type associated with particular zones. Reclamation Fill was placed to raise the ground surface above the existing sea level and is bounded to the west by the seawall. It was encountered only within Zones A and B, where it is typically 1 to 6 m thick. The groundwater table within the Reclamation Fill sits at approximately 1.5 m below ground level and is tidally influenced for at least 200 m inland.

Surficial fill, by contrast, is confined to Zones C and D, where it is localised to areas of development in which the ground has been modified. It is generally 0 to 1 m thick and is generally not expected to exceed 2 m, even in raised areas.

3.4.2 Volcanic Ash

A relatively thin layer of Volcanic Ash overlies the Reef Limestone Formation within Zones C and D only. It is typically 0–2 m thick, but because the top of the underlying Reef Limestone is undulating, this thickness is highly variable. Near the Teoma Street / Port Vila Municipal Stadium area, the volcanic ash appears to be in excess of 4 m thick, likely due to erosion from adjacent hillsides and redeposition in low-lying areas. The volcanic ash deposits are often organic-rich.

3.4.3 Peri-reef deposits

The peri-reef deposits comprise interlayered intra-reef and forereef deposits and are primarily confined to Zone A, where they underlie the Reclamation Fill and overlie the Reef Limestone Formation. However, localised peri-reef deposits may be present in some areas across Zone B. The depositional setting of these deposits is described in Section 3.2. They are expected to be highly variable in both composition and continuity, and are typically 5 to 16 m thick in Zone A. Where present in Zone B, the deposits are expected to be shallow, typically in the order of 1 to 5 m thick.

3.4.4 Reef Limestone Formation

The Reef Limestone Formation underlies all four zones at variable depth and forms the principal competent unit across the Study Area. It comprises a range of limestone types of differing character and age, reflecting the various environments in which they were deposited, and ranges in elevation from approximately -30 mRL to almost 100 mRL across Port Vila. It is generally assumed to be in the order of 20 to 30+ m thick across most of the Study Area, reducing to between 5 and 20 m thick in Zone D.

Cavities formed by karstic dissolution are known to occur within the raised reef limestone platforms, although their prevalence and spatial distribution across the Study Area are not yet well understood. The Reef Limestone Formation is generally free-draining, and as such in Zones C and D, the groundwater table is expected to lie at significant depth, with topography exerting limited influence on its level.

3.4.5 Efaté Pumice Formation

The Efaté Pumice Formation forms the bedrock beneath Port Vila and comprises well-sorted tuffaceous sandstone and siltstone beds originally deposited in a submarine environment. Based on limited investigation, a weathered soil horizon several metres thick is present overlying unweathered rock. The characteristic interbedded light and dark grey beds are expected to persist for hundreds of metres below ground level, and the formation – within the Study Area – dips west at approximately 15 degrees.

The depth to the top of the Efaté Pumice Formation is not well understood across the Study Area: it exceeds 30 m below ground level in Zones A, B and C, but rises to between 5 and 20 m depth in Zone D.

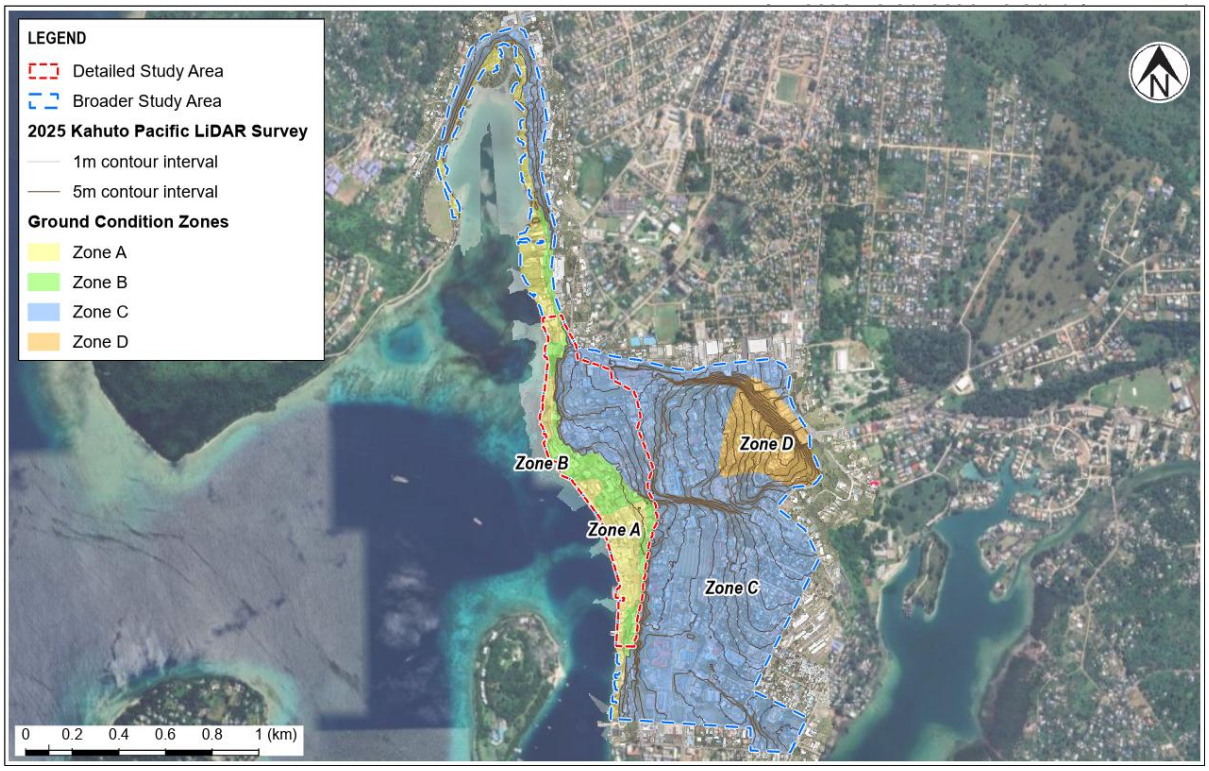


Figure 3.5: Ground condition zoning of the broader Study Area.

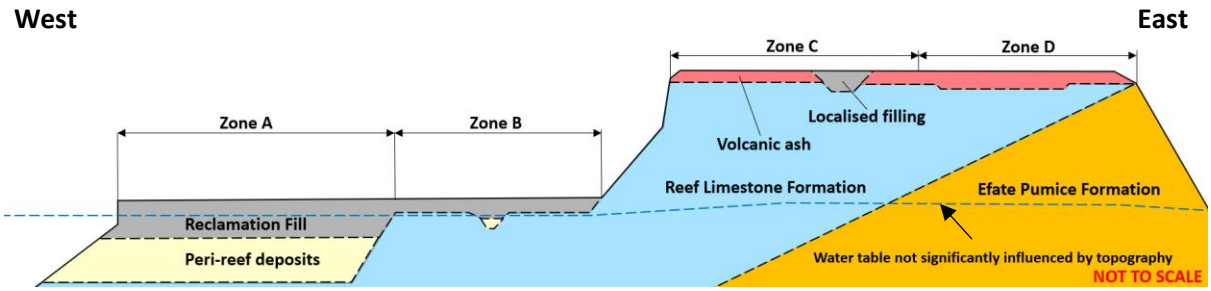


Figure 3.6: Schematic diagram showing a simplified ground profile of each ground conditions zone across Port Vila.

Table 3.2: Litho-stratigraphy of the Port Vila area

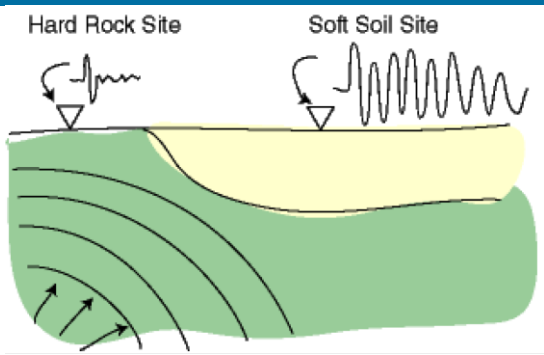
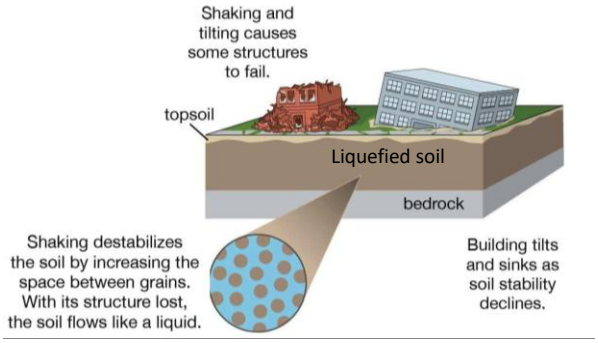
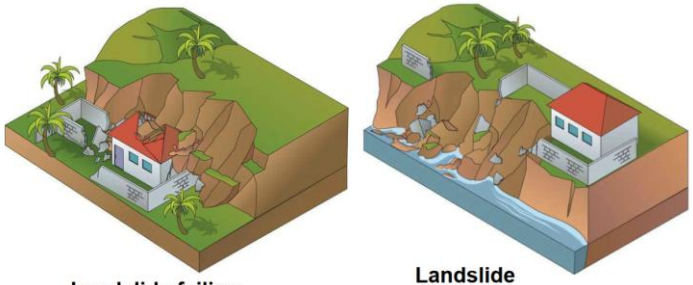
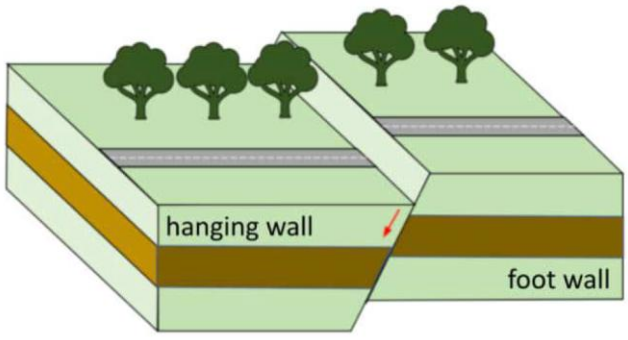
Geological unit	Sub-unit	Generalised soil / rock description	Indicative range of thickness (m)				Indicative lower to upper quartile range [average]		
			Zone A	Zone B	Zone C	Zone D	SPT N value [A]	UCS (MPa) [B]	V_s (m/s) [C]
Groundwater		The regional groundwater table is not significantly influenced by topography and is governed by sea level in low-lying areas.	1 to 5 m below ground	1 to 5 m below ground	Expected > 10 m below ground	Expected > 30 m below ground	-	-	-
Unit 1: Fill	Unit 1a: Reclamation Fill	Sandy GRAVEL, some cobbles / boulders and some silt; brown and white. Loose to medium dense, moist, poorly graded. Clasts, sub-angular, coral.	1 to 6	0 to 3	0 to 2	0 to 2	7 to 22 [16]	-	170[F]
	Unit 1b: Surficial fill	Sandy GRAVEL, some silt; brown and white. Loosely packed, dry; gravel, fine to coarse, sub-angular, coral, wood and concrete. May contain materials that are unsuitable for foundation support or reuse as engineering fill, such as organic matter or buried rubbish.					4 to 17 [8]	-	-
Unit 2: Volcanic ash		Silt, some clay, trace rootlets; dark reddish-brown. Stiff, moist, medium plasticity.	-	-	0 to 2[D]	0 to 2	-	-	-
Unit 3: Peri-reef Deposits	Intra-reef deposits	Gravelly SAND, some silt, minor cobbles with interbeds of silty SAND, minor gravel; grey. Loose to medium dense, moist, well graded. Clasts, sub-rounded, coral and shell fragments.	5 to 20	0 to 5	-	-	6 to 19 [10]	-	240 to 360 [310]
	Fore-reef deposits	Cobbly GRAVEL, some boulders, minor sand; white. Medium dense to very dense, moist, poorly graded. Clasts, sub-angular to sub-rounded, coral.					9 to 46 [19]	-	-
Unit 4: Reef Limestone Formation	Recent reef limestone	Unweathered, white, coral LIMESTONE. Extremely weak to weak, moderately to well cemented. With interbeds of uncemented to weakly cemented carbonate soils similar to intra-reef deposits described above.	10 to 30	> 30	> 30	5 to 20	23 to >50 [>50]	3 – 8 [5]	540 to 1000 [760]
	Raised reef limestone	Highly weathered to unweathered, creamy white stained yellowish brown and orange-brown, coral LIMESTONE. Extremely weak to weak, well cemented.							
Unit 5: Efaté Pumice Formation	Weathered	Completely weathered to highly weathered, orange brown, TUFF. Extremely weak, pumiceous. Soil description: SILT, some clay; brown. Stiff to very stiff, moist, high plasticity.	Hundreds of metres thick (Not encountered)	Hundreds of metres thick (Not encountered)	Hundreds of metres thick (Not encountered)	5 – 6[E]	8 to 14 [10]	-	250[F]
	Unweathered	Slightly weathered to unweathered, grey banded dark grey, TUFF. Very weak to weak, unwelded, pumiceous				Hundreds of metres thick	>50	-	550[F]

Note:
[A] SPT N value – the result of a Standard Penetration Test, a routine field test in which a sampler is driven into the base of a borehole by a hammer of standardised weight dropped from a set height. The N value is the number of hammer blows needed to drive it 300 mm and gives a measure of how dense or firm the soil is, i.e. a higher value means denser / firmer ground.
[B] UCS – Unconfined Compressive Strength, the maximum squeezing (compressive) pressure a sample of rock can withstand before it fails, whether by shearing or crushing, measured in megapascals (MPa). It is a standard indicator of intact rock strength.
[C] vs – shear wave velocity, the speed (in metres per second) at which shear seismic waves travel through the ground, measured from field (downhole geophysical) testing. Stiffer ground transmits these waves faster, so vs is widely used as a measure of ground stiffness and is a key input for earthquake (seismic) design.
[D] In the Teoma Street area, Volcanic Ash soil appears to be significantly deeper, possibly 4 to 8 m, based on non-intrusive surface geophysical testing. We understand this area is known for poor drainage and based on the surface morphology likely acts as a sediment trap for soils washed from the surrounding hillside.
[E] The base of the weathered Efaté Pumice Formation was reached in only two boreholes on Government Hill, where drilling continued through it into the underlying unweathered Efaté Pumice Formation.
[F] Insufficient test data to provide indicative range.

3.5 Earthquake induced geotechnical hazards

The dominant earthquake induced geotechnical hazards for the ground conditions identified within the broader Study Area are summarised in Table 3.3 and form the key geotechnical components assessed in this study. These hazards are assessed in detail in Section 4 to Section 7.

Table 3.3: Summary of geohazards

Geohazard Type	Summary of geohazard	Illustrations of geohazards
Ground shaking	Strong ground movement during an earthquake. Shaking can be locally amplified (made stronger) by the underlying soil and rock profile (site amplification effects).	
Liquefaction / Subsidence	Loose, water-saturated soils temporarily behaving like a liquid during earthquake shaking, causing loss of soil strength, leading to ground settlement and building tilt.	
Slope instability	Landslides that may undermine structures on crests or impact those at the toe of slopes.	
Fault surface rupture	Sharp displacement of the ground capable of tilting / shearing overlying structures.	

There are several other geotechnical hazards that are likely to be relevant for developments within the Study Area. These include but are not limited to:

- Bearing capacity and ground settlement under gravity, live and/ or wind loading. This hazard may be particularly important in areas that are predominantly underlain by soft/ loose Reclamation Fill and/ or Intra-reef deposits.
- High groundwater table in low lying areas (such as Zone A and Zone B). Any excavations that extend beneath the ground water table will likely require temporary groundwater cutoff measures (e.g., sheet piling, secant piles) and/or dewatering. Pile foundations extending below ground water table may require casing.
- Karstic features in the Coral Limestone. These features may significantly reduce the available bearing capacity for shallow foundations and may result in sudden collapse of voids, causing significant differential settlement and structural damage.
- Shrink swell behaviour of soil due to seasonal variation in soil moisture. This may be important in areas underlain by Volcanic Ash, where potential seasonal ground deformation needs to be accounted for in the foundation design.

These hazards have not been assessed in this report but should be considered for site-specific design, where relevant.

4 Ground shaking hazard

4.1 Regional tectonic setting

Vanuatu is located in a highly complex and tectonically active region of the South Pacific. The Vanuatu islands extend approximately 1200 km along a north-northwest to south-southeast trend between latitudes 10° and 22° S from the Solomon arc in the north to the Matthew–Hunter Ridge in the south. The tectonic setting for the islands is predominantly governed by the eastward subduction of the Australian Plate beneath the Pacific Plate along the New Hebrides trench (or the Vanuatu Subduction Zone (VSZ)).

The estimated convergence rate along the subduction zone varies from approximately 35 mm/yr in the central segment, where the D'Entrecasteaux Ridge on the subducting plate collides with the arc, increasing to approximately 120 mm/yr towards the northern and southern ends of the VSZ (Johnson et al, 2021; Suckale and Grünthal, 2009).

The geometry and kinematics of the subduction system are further complicated by the active back-arc spreading within the North Fiji Basin and the interaction among the several microplates. This complex tectonic setting gives rise to a combination of thrust, strike-slip and extensional faulting regimes across the region.

4.2 Historical earthquakes

Given the regional tectonic setting, large magnitude earthquakes have frequently occurred near Port Vila. Since 1907, 51 magnitude 7 or larger earthquakes have been recorded within 500 km of Port Vila, with the largest recorded event being a magnitude 8.1 earthquake in 1920.

The most recent, a magnitude 7.3 earthquake approximately 25 km from Port Vila which occurred on 17 December 2024, resulted in loss of lives and significant damage to land, infrastructure, and buildings. While strong motion stations are not available in Port Vila to record the intensity of shaking, very severe to violent shaking was reported in Port Vila based on felt reports, with a peak ground acceleration of approximately 0.4 to 0.6g estimated based on felt reports (U.S. Geological Survey, 2024).

Figure 4.1 shows historical earthquakes greater than magnitude 5 recorded since 1905 near Port Vila including the location of the magnitude 7.3 2024 Port Vila earthquake. Earthquakes along the plate boundary dominate the historically recorded events, with earthquakes along the interface between the Australian and Pacific Plate shown in green and earthquakes within the subducting Australian Plate shown in blue.

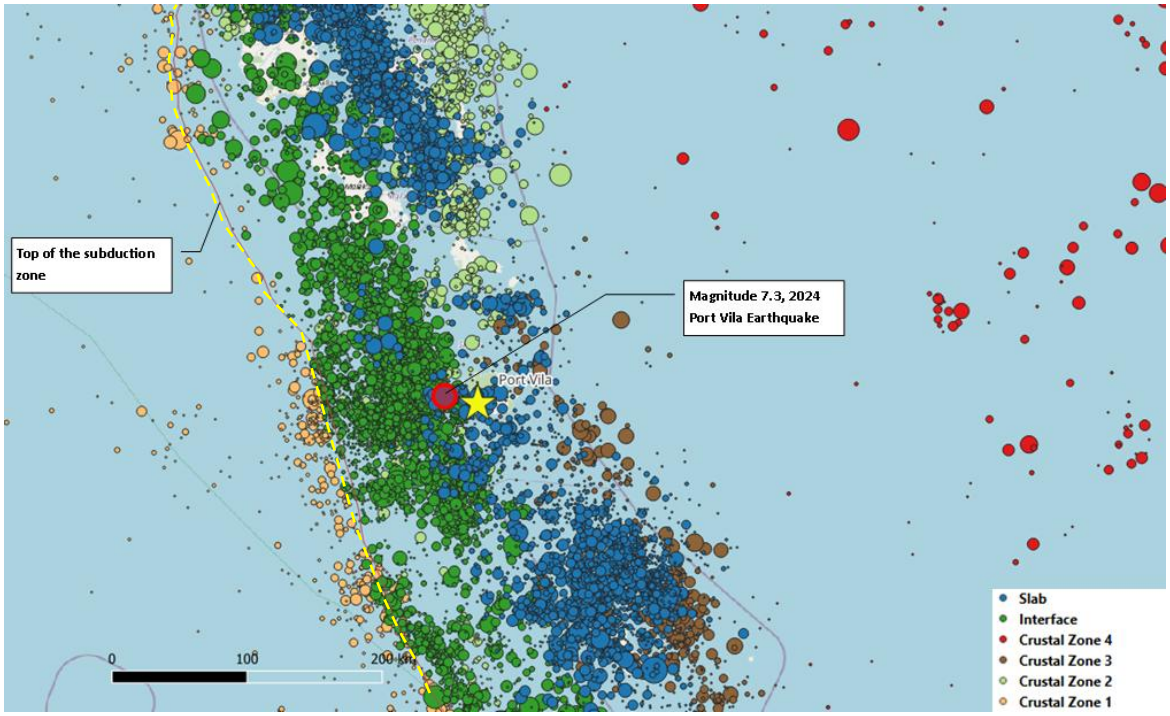


Figure 4.1: Historical earthquakes greater than magnitude 5 since 1905 near Port Vila. The earthquakes shown have been compiled from ISC-GEM Global Instrumental Earthquake Catalogue, ISC Reviewed Bulletin and ANSS Comprehensive Earthquake Catalogue.

4.3 Seismic hazard assessment

There are limited publicly available seismic hazard studies for Vanuatu to inform the assessment of shaking hazard across the Study Area. Recent information available for Port Vila includes:

- Probabilistic seismic hazard assessment of the southern Pacific Islands by Johnson et al. (2021), which includes Port Vila. This assessment provides seismic hazard for a rock site for 1 in 500 annual exceedance probability (AEP) and 1 in 2500 AEP.
- Global Earthquake Model (GEM) Global Seismic Hazard Map (version 2023.1) which provides high-level estimate of peak ground acceleration (PGA) for a rock site for 1 in 500 AEP (Johnson et al., 2023).
- Probabilistic Seismic Hazard Model for Vanuatu by Suckale and Grünthal (2009) which present seismic hazard for 1 in 500 AEP.

There has been a trend of increasing estimates of ground shaking with time as more data becomes available in the South Pacific region. These increases are driven by increase in size and quality of recorded earthquakes for the region, improved understanding of the VSZ and improved models to predict ground shaking from an earthquake rupture.

Recently, the Vanuatu Building Code Edition 2025 was revised, including updated hazard for Port Vila. The Vanuatu Building Code predominantly refers to the New Zealand Standard for Earthquake Actions (NZS 1170.5:2004) but prescribes a hazard factor of 0.7 for Port Vila.

As part of the commission for the EEETA Programme, a seismic hazard assessment (SHA) has been undertaken for the Study Area to estimate ground shaking. This SHA follows current modelling practice for assessment of seismic hazard, including the development of a region-specific seismic source model and use of recently released ground motion models.

The methodology and results from the seismic hazard assessment is presented in the T+TI EEETA Port Vila Seismic Hazard Assessment report dated 10 June 2026, which is provided in Appendix D. Earthquake loading parameters based on this SHA for geotechnical assessment are summarised below.

4.4 Earthquake actions for geotechnical design

For the SHA, a representative time-averaged shear wave velocity over the upper 30 m (V_{s30}) was assessed based on available geophysical testing within each assessed ground zone (refer to Appendix A). The assessed V_{s30} values are as follows (refer to the SHA report for additional detail).

- Zone A – Mean V_{s30} of 375 m/s
- Zone B – Mean V_{s30} of 450 m/s
- Zone C – Mean V_{s30} of 600 m/s
- Zone D – Mean V_{s30} of 450 m/s

While the above values are considered as representative for each zone, it is likely that V_{s30} will vary within each zone. Based on geophysical testing completed to date, V_{s30} was generally assessed to vary from 300 m/s to 760 m/s. Hence, Table 4.1 presents earthquake actions for geotechnical design across a range of V_{s30} values to illustrate sensitivity. The variability in PGA with V_{s30} at various AEPs is also illustrated in Figure 4.2.

It is important that V_{s30} is confirmed through site-specific testing, particularly for high importance structures and/or structures where the shaking hazard is likely to be sensitive to the adopted V_{s30} value. Refer to the SHA report for further discussion and recommendations on sensitivity to the site V_{s30} value.

Table 4.1: Ground motion parameters for geotechnical assessment

Annual Exceedance Probability	PGA (g)						Earthquake magnitude
	V_{s30} 300 m/s	V_{s30} 375 m/s	V_{s30} 450 m/s	V_{s30} 500 m/s	V_{s30} 600 m/s	V_{s30} 760 m/s	
1 in 25 AEP	0.34	0.32	0.31	0.3	0.28	0.26	7.1
1 in 100 AEP	0.65	0.65	0.64	0.63	0.61	0.57	7.3
1 in 250 AEP	0.95	0.97	0.97	0.96	0.93	0.88	7.5
1 in 500 AEP	1.23	1.26	1.28	1.28	1.26	1.2	7.5
1 in 1000 AEP	1.55	1.62	1.66	1.67	1.65	1.58	7.6
1 in 2500 AEP	2.06	2.18	2.27	2.31	2.3	2.22	7.6

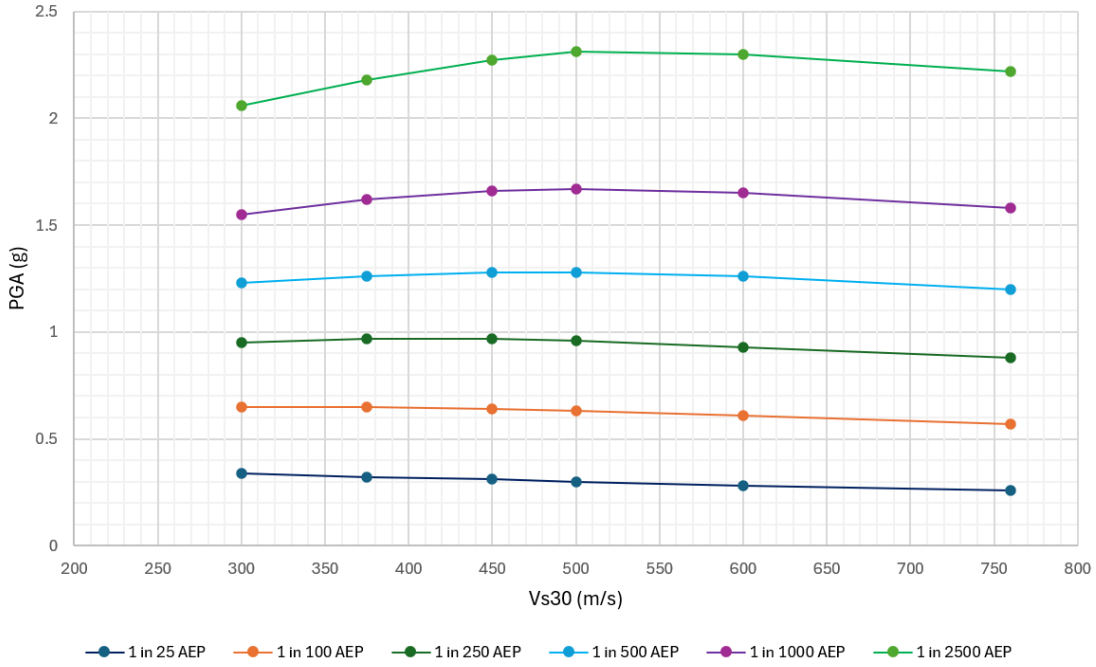


Figure 4.2: PGA sensitivity to Vs_{30} for various levels of earthquake shaking (AEP).

The high ground motion is largely driven by the proximity to the VSZ and the potential for this significant seismic source to generate large magnitude earthquakes (> M_w 7) at frequent intervals. Figure 4.3 presents a comparison of the PGA estimated in this SHA for a range of V_{s30} values against regions in New Zealand and Solomon Islands that are in proximity to significant subduction zones.

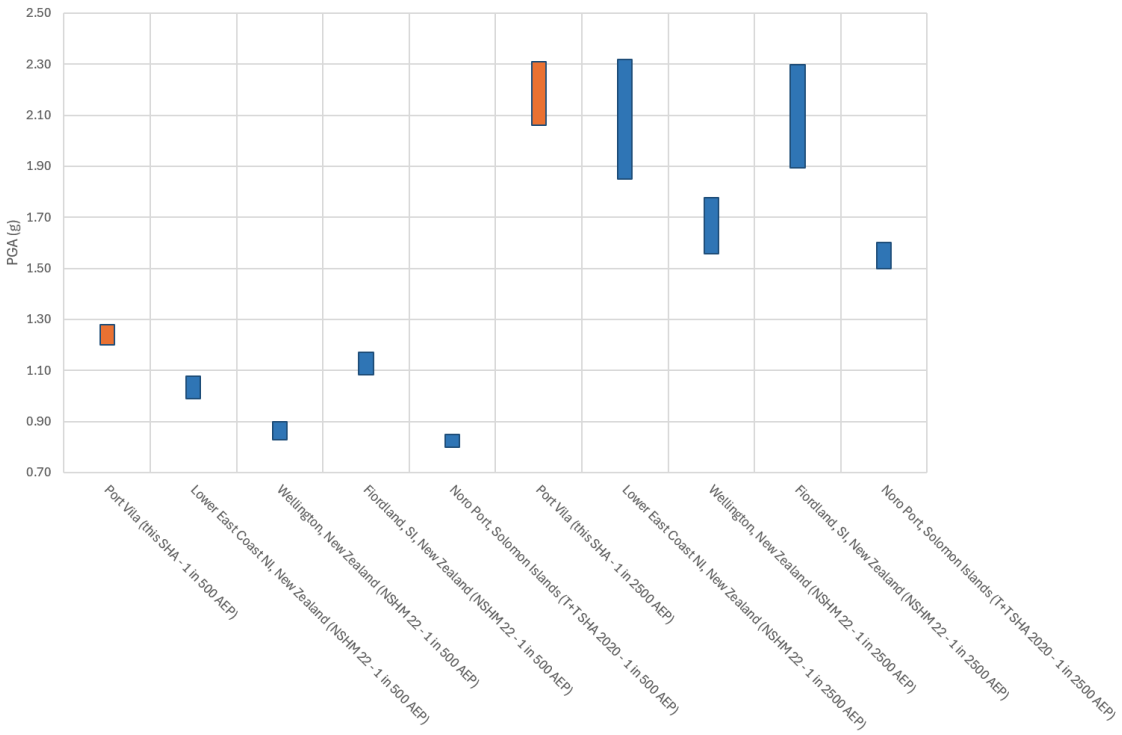


Figure 4.3: Comparison of PGA for 1 in 500 and 1 in 2500 AEP for regions in New Zealand and the Solomon Islands that are in close proximity to significant subduction zone.

4.5 Residual uncertainties

There is inherent uncertainty in the estimated ground shaking hazard for Port Vila CBD. The ground shaking hazard presented above is assessed based on currently available information relevant for the Study Area. Earthquake hazard science is a constantly evolving field and in particular the understanding of seismic hazard in the South Pacific region is evolving rapidly with increasing quantum of data and an improved understanding of seismic sources. Accordingly, it is possible that the results of future research may indicate a seismic hazard that is lower or higher than the hazard estimated by this SHA. A discussion of potential further work that may improve the assessed seismic hazard is provided in the T+TI EEETA Port Vila Seismic Hazard Assessment report.

4.6 Key conclusions and recommendations

The assessed ground shaking hazard for the Study Area is very high in a global context. The assessed ground shaking is driven by the proximity to the VSZ, which has the capability to generate large magnitude earthquakes at frequent intervals. The assessed hazard is generally similar to most recent global models available for the region and other areas around the world near significant subduction zones.

The ground shaking hazard estimated is generally higher than the values outlined in the Vanuatu Building Code Edition 2025 (VBC), though broadly similar to earlier draft numbers released for public consultation.

The seismic hazard assessed specifically for the study area and used as the basis for this geotechnical assessment is based on currently available science. However, it is acknowledged that the design loads considered for the recovery and rebuild efforts will require a balanced consideration of available science, economic factors and practicalities of enforcing the seismic demands on the recovery design.

Broadly, the seismic actions from the Vanuatu Building Code represent a regulatory minimum. The ground shaking hazard presented above, and the SHA provided in Appendix D present the assessment of seismic hazard based on currently available science.

For design, it is recommended that:

1 Where lower values are specified in the Port Vila CBD SHA compared to VBC, the basis for adopting the lower design actions than regulatory minimum should be specifically justified on a project-by-project basis.

2 Where higher values are specified in the Port Vila CBD SHA compared to VBC, design actions adopted should be confirmed on a project-by-project basis. There may be merit in adopting higher actions from the SHA based on the importance level, performance requirements and the expected level of resilience for the structure.

5 Liquefaction hazard

5.1 Liquefaction mechanism

Liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial loss of shear strength. This loss of shear strength can result in settlement and/or horizontal movement (lateral spreading) of the soil mass, and reduced bearing capacity for structural foundations.

The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, plasticity of the soil, and elevation of the groundwater table.

5.2 Historical observation of ground damage due to liquefaction

As noted in Section 4.2, large magnitude earthquakes occur frequently in the Vanuatu region.

Two recent large earthquakes (M_w7+) with ground shaking felt in Port Vila are:

- 2002, M_w 7.2, located 43 km west of Efaté at a depth of 18 km. USGS estimated Modified Mercalli Intensity (MMI) in Efaté generally ranged from MMI 6.5 in the west to MMI 5.5 in the east. The peak ground acceleration (PGA) was likely around 0.3g based on the reported MMI for the area.
- 2024, M_w 7.3, located 24 km west of Efaté at a depth of 54 km. USGS reported estimate MMI in Efaté generally ranged from MMI 7.5 to 9. The PGA was likely 0.4 to 0.6g based on the reported MMI for the area.

The reported damage from the 2002 earthquake included (Garaebiti et al., 2002):

- Subsidence and rotation of the seawall and foundation failure for several structures on reclaimed areas of the city waterfront.
- Severely damaged bridges around Port Vila.
- Cracks were observed in the ground surface in the CBD. Separation between concrete slabs along the sea wall. A separation of 50 to 100 mm was observed on construction joints.

It is important to note that some of the above damage may have occurred in the absence of liquefaction (such as shaking induced failure of structures and foundations, softening of material not susceptible to liquefaction etc.).

Liquefaction and lateral spreading damage observed during reconnaissance by GNS Science (2025) included:

- Ground cracking and settlement at Port Vila Port that may be due to liquefaction and lateral spreading within the Reclamation Fill. However, there was no evidence of surface ejecta at the time of observation.
- Ground settlement and ground cracking generally parallel to the sea wall. The subsidence and evidence of lateral ground deformation was likely caused by liquefaction and lateral spreading within the Reclamation Fill and marine sediments in the central and southern CBD area. In the southern CBD area, it was observed that the newly constructed seawall reduced the density of ground cracking, and the magnitude and extent of lateral spreading compared to areas with the older seawall.
- Evidence of liquefaction was generally not observed in elevated terrace areas generally underlain by Reef Limestone and Efaté Pumice Formation.

5.3 Liquefaction assessment methodology and results

A quantitative liquefaction assessment has been undertaken using geotechnical and geophysical testing completed to date over the Study Area. The assessment methodology and results are summarised in the sections below. The quantitative assessment results have also been compared against observations of liquefaction induced damage, with commentary on consistency between available analytical methods and actual performance.

5.3.1 Ground shaking hazard

The ground shaking hazard used for the liquefaction is based on the SHA completed for the Port Vila CBD, summarised in Section 4.4.

A range of PGA values have been considered to assess the level of shaking at which liquefaction is likely to trigger.

5.3.2 Groundwater level

A representative groundwater level at 1.5 m below ground level has been adopted for the liquefaction assessment. This groundwater level is based on typical observations in the lower lying terrace that is most likely to be underlain by soils susceptible to liquefaction.

5.3.3 Liquefaction susceptibility assessment and results

Generally, sands, non-plastic silts and low permeability sand-silt-gravel mixtures are susceptible to liquefaction. The following assessment criteria have been used to assess the susceptibility of the underlying soils across the Study Area.

- **Geological criteria:** Moderately weathered to unweathered rock is not susceptible to liquefaction, where the material behaviour is still predominantly controlled by the parent rock structure;
- **Groundwater level:** Soils above the groundwater table are not susceptible to liquefaction; and
- **Plasticity of the soils**: Soils that are moderately to highly plastic (with a plasticity index greater than 12) are not susceptible to liquefaction.

Soils that are highly permeable (such as clean gravels to cobbles with minor to insignificant sand and fines) are considered as unlikely to liquefy. Excess porewater pressure generation in highly permeable soils likely to be limited. Hence, these soils were considered as not susceptible for the simplified liquefaction triggering assessment.

Table 5.1 presents the assessed susceptibility of each geological unit and sub-units underlying the Study Area. This assessment was based on review of available published information, description of the soils recovered in the geotechnical investigations and laboratory testing completed for the Study Area. The delineation of ground zones considered are shown in Appendix A.

Table 5.1: Liquefaction susceptibility

Geological Unit		Assessed material behaviour	Assessed susceptibility
Unit 1: Fill	Reclamation Fill (Unit 1a)	Reclamation Fill is likely to have been placed in an uncontrolled manner. Predominantly comprises sandy and gravelly soils, below the groundwater table. Hence assessed as susceptible.	Susceptible to liquefaction
	Surficial Fill (Unit 1b)	Cohesionless soils located above the water table. Soils unlikely to be saturated and hence assessed as not susceptible.	Not susceptible
Unit 2: Volcanic Ash		Predominantly comprises silty clay that is moderately to highly plastic.	Not susceptible
Unit 3: Peri-reef	Intra-reef (Unit 3a)	Predominantly saturated, sandy and gravelly soils.	Susceptible to liquefaction
		Continuous bands of poorly cemented coral reef cobbles. This layer is likely to be highly permeable, and hence assessed as not susceptible.	Not susceptible
	Forereef (unit 3b)	Predominantly saturated, cobbly gravels with some boulders and minor sand. This layer is likely to be highly permeable, and hence assessed as not susceptible.	Not susceptible
Unit 4: Reef Limestone Formation		Slightly weathered to unweathered Limestone, extremely weak to weak. Not susceptible.	Not susceptible
Unit 5: Efaté Pumice Formation		Weathered Efaté Pumice Formation, predominantly comprising silt with some clay. Moderately to highly plastic and hence considered to be not susceptible.	Not susceptible
		Slightly weathered to unweathered Sandstone and Siltstone, extremely weak to weak. Not susceptible.	Not susceptible

5.3.4 Liquefaction triggering assessment and results

Soils which are susceptible to liquefaction require a certain level of earthquake shaking (trigger) to liquefy. Denser soils require more intense and/or longer duration of shaking (higher trigger) than less dense soil.

Liquefaction triggering analyses have been undertaken to assess the potential for liquefaction to occur for various levels of ground shaking. Triggering analyses has been undertaken using:

- SPT N-values from boreholes using the Idriss and Boulanger (2008) triggering method; and
- Shear wave velocity measurements from downhole testing undertaken for the Study using the Kayen et al. (2013) and Andrus et al. (2004) triggering methods.

The above simplified empirical procedures were developed from case histories comprising predominantly Holocene, hard-grained alluvial deposits. The susceptible soils within the Reclamation Fill and Intra-reef deposits contain a high proportion of gravels and/or crushable soils. There are no well-established relationships between SPT N-values/ shear wave velocity and cyclic resistance for these specific soils, as such there is a higher degree of uncertainty in the assessment.

- In crushable soils, the measured SPT N-value may underestimate liquefaction resistance due to crushing of the soil under the SPT hammer.

- In gravelly soils, the measured shear wave velocity may overestimate liquefaction resistance based on documented case history (CentrePort, Wellington; Cubrinovski et al., 2017)

Figure 5.1 and Figure 5.2 present summaries of the liquefaction triggering results. Detailed liquefaction calculations are not included in this report. However, this can be provided as a separate technical memo should this be required.

Figure 5.1 shows the typical depth and thickness of the liquefiable material at 1 in 250 AEP level of earthquake shaking. At this selected level of earthquake shaking, any material susceptible to liquefaction will trigger. The figure presents triggering results using the Kayen et al. (2013) method using measured shear wave velocity data only, as the results using the Andrus et al. (2004) were broadly similar.

Figure 5.2 presents the one dimensional free-field settlement (S_{v1d}) assessed using the Ishihara and Yoshimine (1992) relationship for SPT N-value based triggering assessment and the Cetin et al. (2009) probabilistic relationship for shear wave velocity based triggering assessment. Results are shown for various levels of earthquake shaking. S_{v1d} provides an indication of severity of liquefaction and the level of shaking when liquefaction tends to trigger.

The triggering assessment shows that:

- Reclamation Fill and Intra-reef deposits with SPT N-values greater than 30, or a depth normalised shear wave velocity (V_{s1}) of 250 m/s or greater, are generally too dense to liquefy.
- Looser soils than above are likely to trigger at PGA exceeding 0.2 to 0.6g, approximately corresponding to 1 in 25 to 1 in 100 AEP levels of earthquake shaking. There is a high probability of these levels of earthquake shaking occurring within the design life of a building.
- A cumulative liquefiable thickness of 10 m to 15 m was estimated for triggering assessment using SPT N-values. A cumulative liquefiable thickness of approximately 5 m was estimated for triggering assessment using V_{s1}.
- Liquefaction triggering assessment using both SPT N-values and V_{s1} indicate potential for shallow soils (typically in the upper 5 to 10 m) to liquefy at PGA exceeding 0.2g. It is important to consider potential reduction in strength and stiffness of these shallow soil layers for shallow foundation design.
- Estimated S_{v1d} ranged from 150 mm to 500 mm for triggering assessment using SPT N-values and from 100 mm to 250 mm for triggering assessment using V_{s1} data. The S_{v1d} provides a sense of liquefaction vulnerability. S_{v1d} is an estimate of free-field ground settlement and is unlikely to be an accurate measure of actual settlement of structures.

The use of SPT N-values for liquefaction assessment underlying the Study Area is considered conservative. SPT N-values likely underestimate the density of crushable soils, and the discrete measurements likely overpredict the cumulative thickness of liquefiable layers by not accounting for the interlayered nature of susceptible and non-susceptible deposits. S_{v1d} assessed using V_{s1} appears to be more consistent with liquefaction-induced damage observed in recent earthquakes. This is discussed further in Section 5.5.

In addition, liquefiable layers within the Peri-reef deposits (Unit 3a) are typically non-continuous interlayered with not-susceptible layers. The system response effects of interlayered deposits are not accounted for in simplified liquefaction triggering procedures adopted in this study. The interlayered nature of these deposits is likely to improve the liquefaction resistance of this unit as a whole.

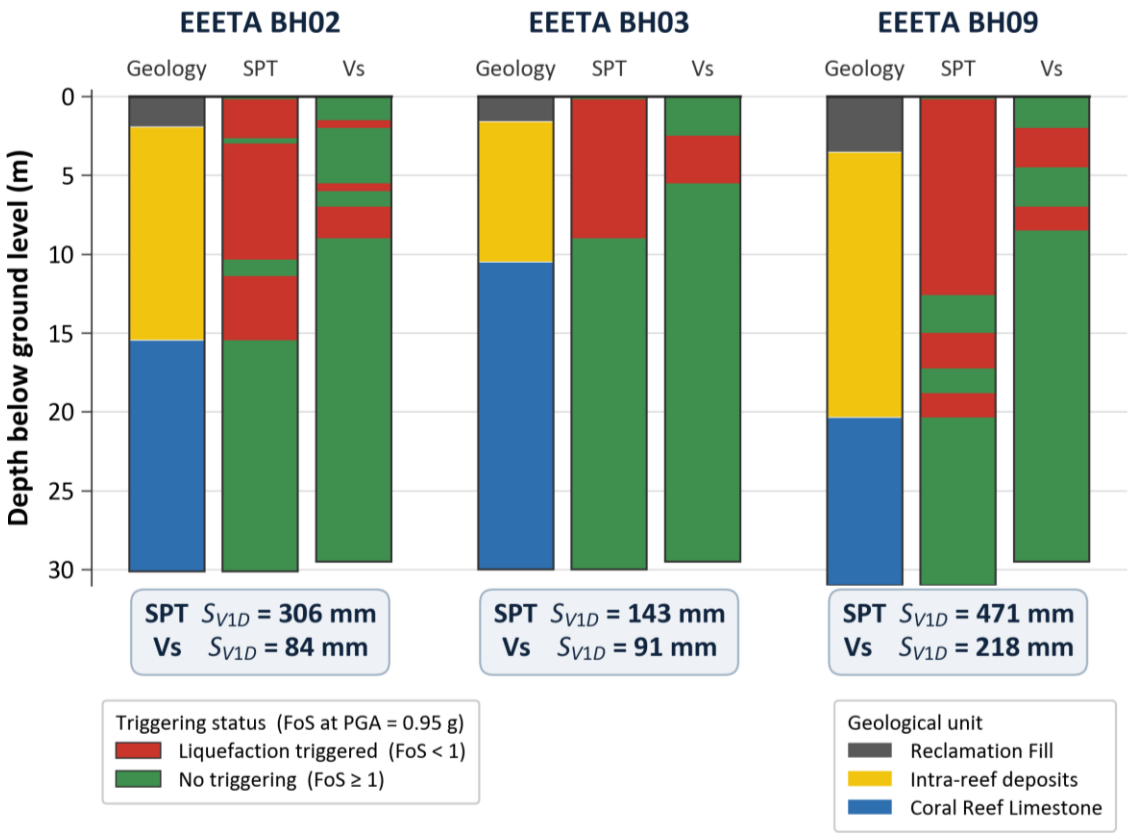


Figure 5.1: Liquefaction triggering assessment using SPT N-values and measured shear wave velocity for a 1 in 250 year design event; PGA = 0.95g.

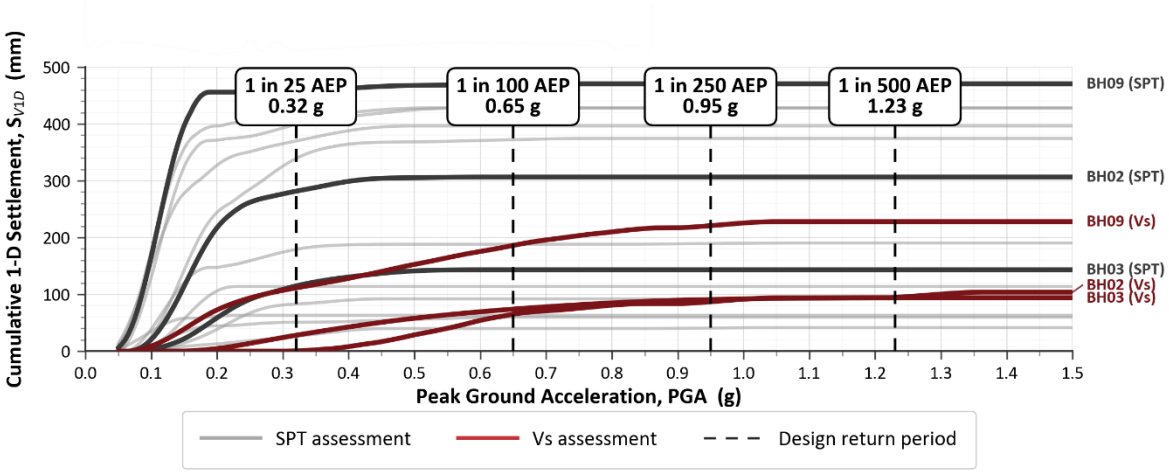


Figure 5.2: One dimensional free-field settlement assessed using SPT N-values and Vs measurements from downhole testing over various levels of earthquake shaking. Light grey lines are other non-paired boreholes with SPT N-value measurements available across the Study Area.

5.4 Liquefaction consequences

Typical consequences due to liquefaction include:

- Slope instability or lateral spreading due to presence of a free-face or sloping ground resulting in lateral and vertical ground deformation. This may result in ground cracking, tilting of structures or tearing of structures due to lateral stretch across the structural footprint;
- Ground settlement due to liquefaction induced ejecta, volumetric consolidation and/or shear induced building settlement. This may result in tilting of structures, significant cracks in the structure, rupture of service connections etc;
- Reduction in bearing capacity for shallow foundations, negative skin friction loads on pile foundations and uplift loads on buried structures;
- Potential increase in flooding hazard due to area wide ground settlement and/or increase in groundwater table relative to ground surface level.

Some of the characteristic features and effects of lateral spreading and liquefaction are illustrated in Figure 5.3 and Figure 5.4.



Figure 5.3: Characteristic features and effects of lateral spreading illustrated on a Christchurch case history (modified from Cubrinovski (2019), Ralph B. Peck Lecture). Obtained from NZGS/ MBIE Module 3.

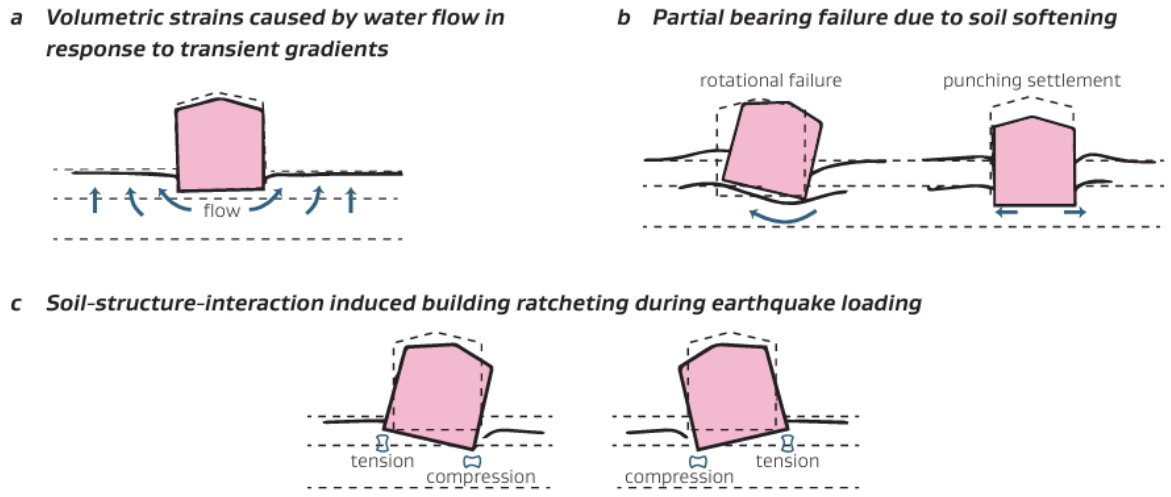


Figure 5.4: Liquefaction induced building displacement mechanisms – figure obtained from Bray and Dashti (2014).

The Port Vila CBD is bound by a coastal sea wall towards the west (approx. 3 m retained height), with uncontrolled reclamation along parts of the shoreline. In these areas, where there is continuous liquefiable layers present, there is potential for liquefaction induced lateral spread mechanisms to form. Lateral spread may result in vertical and lateral ground deformation that extend a significant distance inland (typically up to 200 m to 300 m inland).

In Zone A, and localised areas in Zone B, that are underlain by liquefiable soils, liquefaction induced ground settlement is also likely to occur. The actual ground and building settlement at a site will vary and will depend on:

- Thickness and depth of the liquefiable soil,
- Foundation type and load imposed on the underlying ground, and
- Level of earthquake shaking.

The consequences of liquefaction can be managed through good design practice and careful detailing. A broad level approach to mitigate effects of liquefaction are discussed in Section 5.6 and is provided in the Geotechnical Guidance Document.

5.5 Comparison of liquefaction assessment against observed damage

The observations from the 2024 Port Vila earthquake and the 2002 earthquake are generally consistent with the quantitative liquefaction assessment.

The recorded observations indicate evidence of ground settlement and lateral deformation in the CBD area that are likely to be a result of liquefaction and lateral spreading towards the coastline. There is minimal evidence of liquefaction ejecta. This may indicate the effect of a competent non-liquefiable crust that supresses surface manifestation of liquefaction, or that the underlying soils contain minimal fine-grained soils to be carried up to the surface as ejecta.

Based on comparison of the quantitative analyses against observed damage due to liquefaction, we note that:

- The observations support the assessment that only the Reclamation Fill and cohesionless soils within Intra-reef deposits are likely to be susceptible to liquefaction. These soils are generally limited to the low-lying areas within Zone A and potentially localised areas in Zone B.
- The quantitative analyses indicate liquefaction triggering in susceptible soils is expected at 0.2 to 0.6g levels of earthquake shaking (loosest soils triggering at the lower end of this range). It is likely that the 2024 Port Vila earthquake generated this level of shaking at Port Vila CBD and, hence the majority of susceptible material will likely have triggered in that event.
- The observed damage indicates that the quantitative assessment using SPT N-values is likely to be conservative as the observed damage was generally less than predicted. The assessment using V_s collected through downhole testing appear to be more consistent with the level of observed damage.

Observation of liquefaction induced damage following significant earthquakes provide highly useful information to calibrate analytical assessments. However, it is important to note that due to lack of ground motion sensors in Port Vila there is uncertainty in the actual intensity and duration of ground motion that was experienced across the Study Area due to the 2002 and 2024 earthquakes. Liquefaction is highly sensitive to both intensity and duration of ground motion, and as such, liquefaction induced damage observed from past events (or lack of damage) is not a complete indication of future damage that can occur.

Nevertheless, the recovery and rebuild efforts provide a significant opportunity to establish a database to record high quality information to support future design works. These are discussed further in Section 5.6.

5.6 Key conclusions and recommendations

Liquefaction hazard for the Study Area indicates that loose, susceptible soils within the Reclamation Fill and Intra-reef deposits have the potential to liquefy at PGAs between 0.2g to 0.6g. These deposits are generally present in Zone A, with potential for localised deposits in Zone B. This assessment is consistent with observed land damage likely due to liquefaction from the 2002 and 2024 earthquakes.

Liquefaction, and liquefaction induced lateral spreading near the coastline, may result in significant ground deformation. As such, it is important that foundations are appropriately selected and designed to minimise liquefaction induced effects.

Good design practice for resilient structures typically includes:

- Early collaboration between geotechnical and structural engineer and collaboration in selecting a suitable foundation form, ground improvement measures and tolerable deformation.
- Use of well-connected, regular foundation system that can tolerate levels of ground deformation that may occur and/or systems that can be re-levelled readily.
- Flexibility in service connections.

For sites that are underlain by liquefiable soils, mitigation measures should be assessed based on the proposed structural form and likely loads that will be imposed on the underlying soils. A general strategy for liquefaction assessment and foundation selection to mitigate against liquefaction induced damage is provided below. Further guidance is provided in the Geotechnical Guidance Document. The New Zealand Earthquake Geotechnical Engineering Practice: Module 3 and Module 4 also provide useful guidance for liquefaction assessment and foundation design.

A site-specific liquefaction assessment and foundation design will generally comprise:

- **Desktop assessment** – Based on available geological information assess whether liquefaction hazard is relevant for the site. Typically, liquefaction hazard is only likely to be relevant in low-lying areas underlain by Reclamation Fill and/ or Intra-reef deposits.
- **Performance level** – Confirm performance level for the structure, including the AEP for design earthquake shaking and tolerable levels of ground deformation.
- **Site-specific liquefaction assessment** – Undertake a site-specific simplified liquefaction assessment where liquefaction hazard is relevant. This is typically required (but not limited to) for structures supported on shallow foundations overlying Reclamation Fill and/or Intra-reef deposits. Carry out site-specific investigations as required. Based on liquefaction assessment undertaken in this study.
  - Simplified methods using V_s appear to be more consistent with observations of liquefaction induced ground damage. It is recommended that geophysical testing is considered for any additional geotechnical investigations, when appropriate.
  - Simplified methods using SPT N-values are likely to be conservative. It is recommended that any site-specific liquefaction assessment is calibrated against observations that may be available in the area following the 2024 and 2002 earthquakes.
- **More rigorous assessment** – For high importance structures, there may be merit in carrying out rigorous laboratory testing such as cyclic simple shear or cyclic triaxial testing and/ or more rigorous liquefaction assessment.

  Rigorous assessment often requires collection of undisturbed samples and sophisticated laboratory testing. The testing provides a means to directly measure the cyclic resistance of the underlying soils and may show that the cyclic resistance of the underlying soils is greater than typical for hard-grained sands. These test results, along with available geophysical testing, could be used to form specific relationships between V_s and liquefaction resistance.

  Given the cost of obtaining undisturbed samples and undertaking sophisticated laboratory testing, these methods are likely only appropriate for high importance or critical structures.
- **Selection and design suitable foundations** – Design foundation systems and ground improvement measures required to mitigate liquefaction effects and achieve the required performance level. It is recommended that both the project geotechnical and structure engineer are involved in the selection and design of appropriate foundation and/ or ground improvement measures. In particular, buildings within 200 m to 300 m of the coast may be prone to lateral ground deformation due to lateral spreading. The selected foundation system should consider potential for lateral displacement and lateral stretch. For buildings located within this zone, generally well-connected foundation systems are recommended to tolerate potential ground movement.

6 Slope hazard

6.1 Background

The 17 December 2024 Port Vila earthquake triggered numerous slope failures across the Study Area and the wider Port Vila area. The most prominent slope-related ground damage was localised rockfall, sourced predominantly from the steep, uplifted coastal cliffs that bound the CBD along the north.

These rockfalls were concentrated on the upper parts of the slopes, near the crest, where slope angles generally exceeded 45° and were consistently associated with cut excavations made into the hillside to allow development. By contrast, natural, undisturbed slopes generally performed well, with few earthquake-induced failures observed. Comparable rockfall activity from these slopes was also recorded following the January 2002 earthquake.

This section assesses the slope hazard within the Study Area, and includes:

- Landslide mechanisms and triggering processes relevant to Port Vila;
- A summary of the GNS Efaté-wide landslide hazard model;
- An areawide susceptibility assessment for the Study Area to develop runout and slope regression susceptibility zones, including residual uncertainties of the assessment.

The intended purpose of this assessment is to characterise the slope hazard at a level suitable for area-wide land-use planning and to support pragmatic risk management for developments potentially exposed to slope hazards.

6.2 Landslide hazard mechanisms

6.2.1 Landslide types

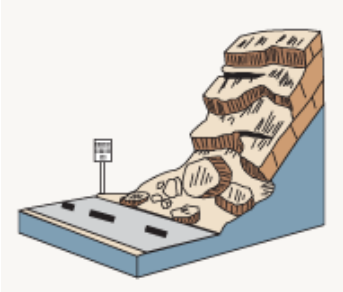
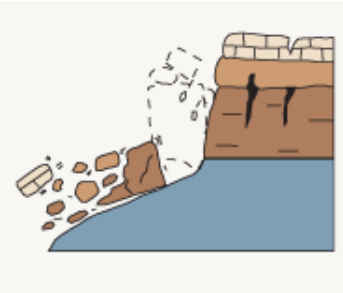
Landslides are broadly defined as “the movement of a mass of rock, debris, or earth down a slope”, and are commonly distinguished by their type of movement and the material in which they occur (Cruden & Varnes, 1996; Hungr et al.,2014).

Based on this classification system, field and desktop landslide mapping completed by T+TI and GNS (Massey et al., 2025a) indicate that the predominant landslide types, observed following the 17 December 2024 earthquake, within the Study Area typically comprised the following types (defined in Table 6.1):

- **Falls**: Detaching limestone blocks from very steep, modified slopes was the typical failure mechanism for most of the observed landslides in Port Vila. Soil overlying the detached blocks also fell, accumulating as a talus slope or pile of debris at the base of the slope.
- **Topples:** Vertical to subvertical weaknesses in the limestone rock mass allow blocks to topple out of the slopes around some parts of Port Vila, generally where the limestone rock is more massive, i.e. contains less defects.

Other kinds of landslides were generally not observed within the Study Area, following the 17 December 2024 earthquake, likely due to the material types, strengths, significant depth to groundwater, and topography.

Table 6.1: Definitions of landslide types observed in Port Vila

Types of Landslips		Description (New Zealand Geotechnical Society, 2025)
Falls		Rock: The landslide body forms an irregular surface of jumbled rock, sloping away from the scarp. If the rockfall is large, the debris may contain depressions, and the toe area will likely be an irregular pile of debris or talus. Depending on slope height, angle and surface characteristics, individual rock blocks may bounce or roll for a considerable distance downslope, particularly when earthquake induced. Soil: Detachment of soil fragments or blocks of cohesive (cemented or unsaturated) soil. Fallen material forms a heap adjacent to scarp with an irregular surface
Topples		Topple failures involve the forward rotation and movement of a mass of rock out of a slope. This kind of slope failure generally occurs around an axis (or point) at or near the base of the block of rock. Toppling failures can occur in both natural and cut slopes in rock with regularly spaced fractures or bedding planes that strike parallel or subparallel to the slope, and dip steeply into the face. A topple often results in the formation of a debris wedge or debris cone at the base of the slope. New talus cones are unvegetated; old talus cones can have weeds and even trees growing on them.

6.2.2 Triggering mechanism

Although factors such as geology and topography are the primary factors dictating whether slope instability can occur, it usually takes a triggering event, such as seismic shaking, for a landslide to be initiated. In the case of the types of landslides observed in Port Vila, the expected triggering mechanisms include:

- Ground shaking due to earthquakes, resulting in fall of already loose material and generating further loose rocks. This is expected to be the principal triggering mechanism within the Study Area.
- Jacking / physical loosening by vegetation growth between natural fractures in the rock.
- Elevated water pressures within joints and along geological contacts due to intense rainfall.

6.2.3 Consequences

Consequences of rockfall witnessed in Port Vila can occur both upslope and downslope of the rockfall source.

Upslope of the source, regression of the slope crest can result in land loss and undermining of structures and roads.

Downslope of the source, boulders and debris can impact structures and inundate downslope areas. Boulders can cause severe damage to structures given sufficient size and velocity. These hazards are illustrated in the example images of the rockfall due to the 2010/2011 Canterbury Earthquake Sequence in Figure 6.1 and Figure 6.2 below.



Figure 6.1: Rockfall causing undermining of houses in Christchurch, New Zealand (The Guardian, 2011).



Figure 6.2: Inundation of downslope areas (a) and damage to house (b) due to rockfall in Christchurch, New Zealand (Massey et al., 2014).

6.3 Summary of GNS assessment

Following the December 2024 earthquake, GNS conducted an Efaté-wide landslide study (Massey et al., 2025b). The study used a combination of desktop mapping of over 3,500 landslides across the island and machine learning trained to multiple factors including geology, earthquake shaking, elevation, slope angle, local slope relief, slope aspect, slope curvature, topographic wetness and roughness, and fault proximity.

The key findings of their assessment – in relation to the Study Area – are summarised below:

- The Earthquake Induced Landslide (EIL) model results are represented in a 12 m square grid, providing a broad coverage, high level analysis.
- GNS analysed landslide susceptibility and EIL runout hazard for both wet and dry ground conditions. Fahrböschung (travel) angles of 24 degrees (wet) and 38 degrees (dry) were adopted, with obstacle heights scaled to the size of the landslide.
- Locations of infrastructure and building assets were used to assign a risk rating of negligible, low or high risk.
- Limitations due to the scale and coverage of the study means smaller topographic features could not be determined by the model. Furthermore, in some instances the model suggests a hazard area in locations where the local geometry would not be sufficiently steep.
- The model results for the wet condition from the GNS study have been used as a high-level comparison for the landslide susceptibility assessment detailed in Section 6.4.

The Port Vila area results – from the GNS study – are presented in Figure 6.3 below.

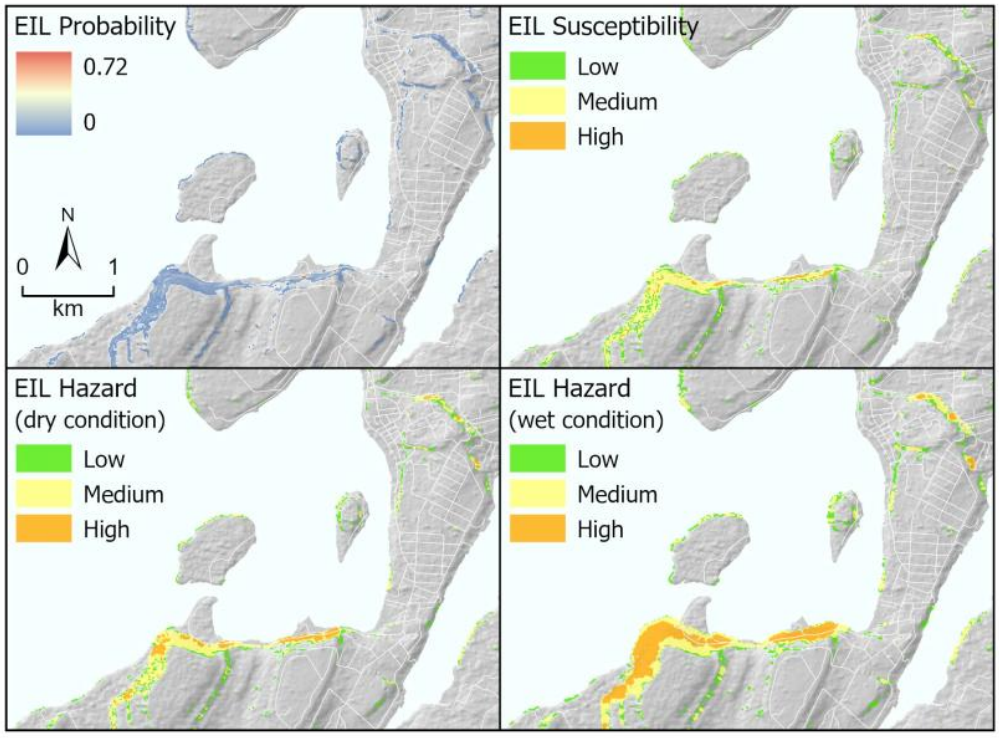


Figure 4.5 Example landslide hazard-inundation area model output for both dry and wet debris avalanches at 12 m resolution. Note that the EIL Probability ranges from 0 to 0.72 and represents the range estimated for the entire island of Efate. In this example map, the hazard classes only go up to 'High', as there are no 'Very High' classes present in this area.

Figure 6.3: Excerpt of Figure 4.5 of Massey et al. (2025b) showing summary of Earthquake Induced Landslide (EIL) model results in the Port Vila area.

6.4 Susceptibility assessment

6.4.1 Background

A landslide susceptibility assessment generally following the guidance provided in the 2024 Landslide Planning Guidance (de Vilder et al., 2024) was undertaken, at a level of detail equivalent to a Level A study as defined in the guidance. This assessment focused on rockfall and rock toppling, the dominant landslide mechanism observed within the Study Area due to the 2024 Port Vila earthquake, and includes:

1 **Runout envelope**: This is the land downslope of a landslide source area, which can be inundated with debris when a landslide occurs. The runout envelope is defined by the angle from the extent of the debris to the crest of the slope, commonly referred to as the Fahrböschung angle.

2 **Regression envelope**: This is the area typically upslope of a potential rockfall source area or over steepened slope, where ground deformation could be expected due to block drop out or tilting.

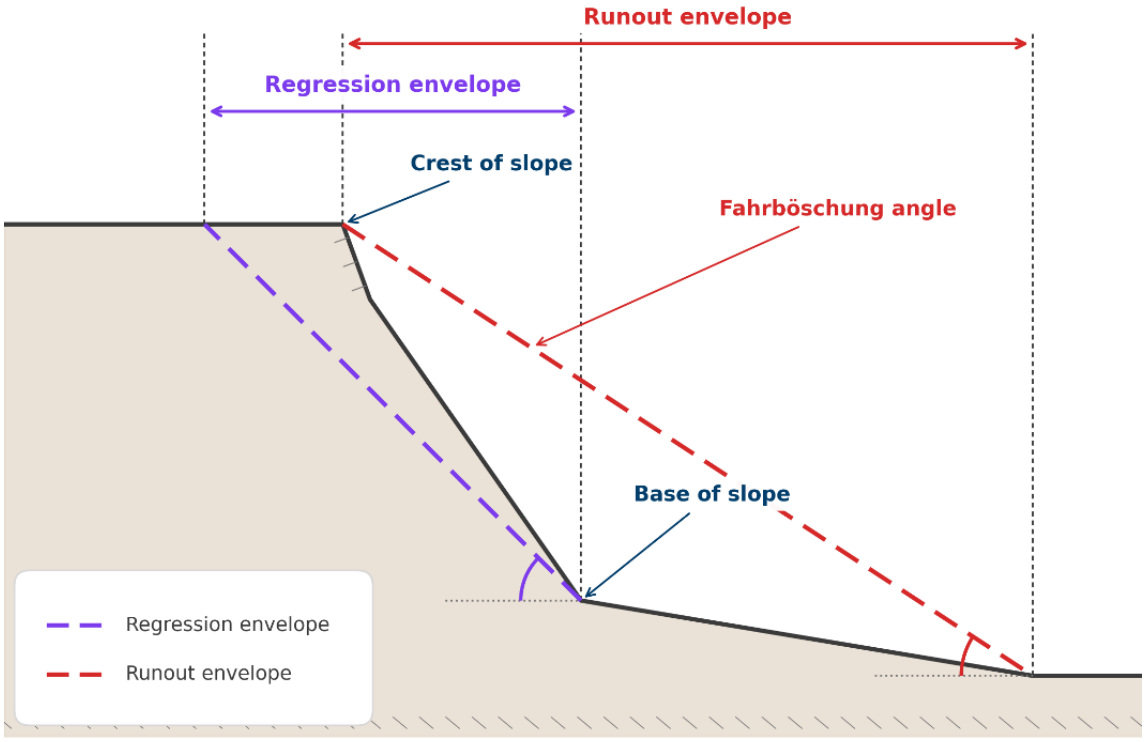


Figure 6.4: Illustration of regression and runout envelope definitions

6.4.2 Runout methodology

A summary of the methodology used to determine the susceptibility zoning for rockfall runout is provided below, with detailed results outlined in the Runout Technical Memo (T+TI, 2026).

1 **Back-analysis of model input parameters,** using RocFall2, based on 9 No. landslide sources and debris runout mapped (Massey et al., 2025a) following the 17 December 2024 earthquake. The following assumptions have been made for the back analysis:

   a Pre-earthquake 2D slope profiles extracted from the 2012 LiDAR dataset (Geoscience Australia, 2012-2013) along sections representative of the slope failure. In some instances, the post-earthquake 2025 LiDAR dataset (Kahuto Pacific, 2025) was used where the 2012 LiDAR was not representative of the slope prior to the rockfall.

   b Landslide sources were generally assumed from crest of the slope failure.

   c Landslide runouts obstructed by structures or altered prior to mapping were excluded.

   d At each landslide location, the material parameters were calibrated so that the maximum modelled runout matched the runout observed in the mapped landslides. These parameters were generally consistent with experience in similar materials and the Rocfall 2 software reference library.

   e Each landslide was modelled at an order of magnitude comparable to that observed, using 1,000 boulders for larger rockfalls and 100 boulders for smaller rockfalls.

   f An initial horizontal velocity to simulate the earthquake was also applied.

   g Typical block shapes and sizes approximately 1 m in diameter were selected on the basis of engineering judgement from the field observations undertaken.

2 **Typical slope material parameters were adopted** for each of the following materials:

   a Bare rock slopes – typically seen in cuts or landslides exposing the limestone surface.

   b Vegetated slopes – landslides occurring in vegetated slopes where slope materials caused more dampening of boulder bouncing, velocities and rotation.

   c Heavily vegetated slopes – landslides in vegetated slopes where vegetation remained in situ and boulders were deflected by vegetation, causing further dampening of boulder velocities and rotation.

   d Road – downslope areas representing a sealed road surface.

   e Earth – unpaved downslope areas including roadside berms or unsealed yards.

3 **Forward modelling of representative locations** across the Study Area (17 in total) to assess potential rockfall runout for a representative (not necessarily absolute maximum) event for the Study Area. The additional locations provide a larger variety of slope geometries compared to the back analysed sections. The following assumptions have been made, in addition to those already summarised in the back-analysis outlined above:

   a Representative model locations were selected to increase the spatial distribution of analysis and the variety of slope geometries analysed.

   b Back-analysis parameters are representative of other slopes within the Study Area, where rockfall did not occur during the 17 December 2024 earthquake.

   c All models were run with approximately 1000 boulders, consistent with the New Zealand Ministry of Business, Innovation & Employment (MBIE) (2016).

   For each location, the modelled runout was summarised using the distance reached by only 5% of the boulders, the distance reached by only 1%, and the distance reached by the single furthest boulder (0.1%), as shown in Figure 6.5.

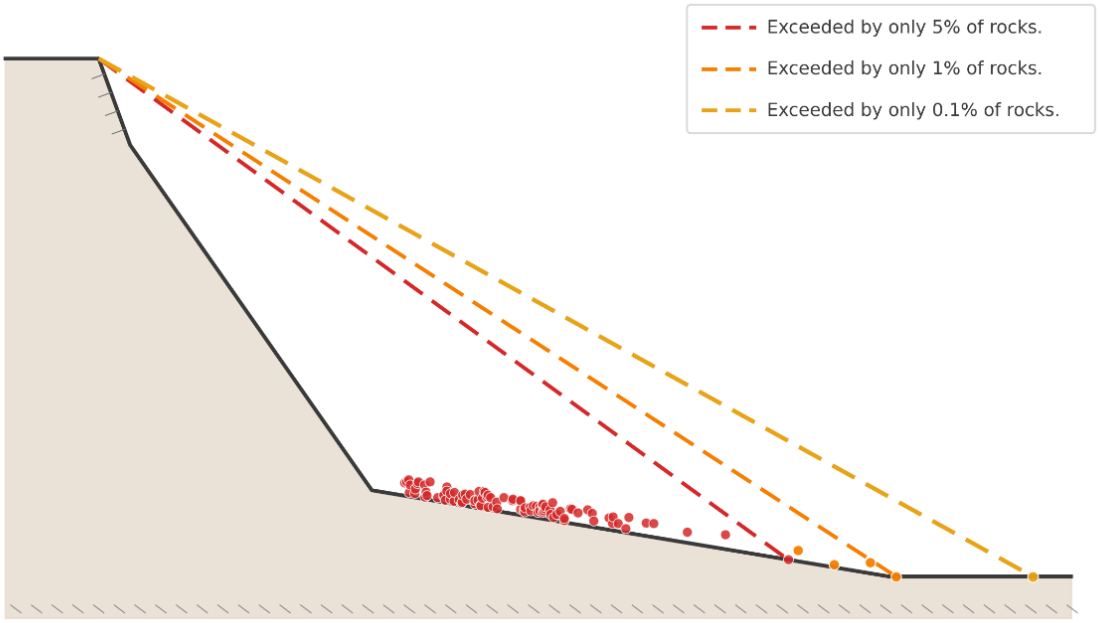


Figure 6.5: Schematic diagram of runout envelopes exceeded by only 5%, 1% and 0.1% of rocks. Coloured dots illustrate the proportion of rocks that come to rest in each runout envelope.

4 **Adopted two representative runout envelopes**, based conservatively on the results calculated in point 3, summarised below:

   a **36° runout envelope:** The area within which ~95% of fallen boulders come to rest. Highest susceptibility (most material ends up within this envelope). This means that in a rockfall event involving 100 boulders, about 5 boulders would be expected to travel beyond this envelope.

   b **29° runout envelope:** The area which only approximately 0.1% of boulders exceed. This means that only about one boulder in every 1,000 would be expected to travel beyond this envelope (in the modelling, the single furthest-travelled boulder).

   An intermediate tier was considered but not adopted, as the additional runout envelope did not provide meaningful differentiation at the mapping scale.

5 **Potential rockfall sources were mapped where the slope angle exceeded 45 degrees** for at least a 10 m width, on slopes at least 4 m high. Localised, property-scale features were excluded from this assessment as these are considered beyond the level of detail of this assessment. These should be considered on a site-by-site basis.

6 **Used a GIS-based line of sight (LOS) method to project the runout envelopes** (adopted above) from all mapped rockfall sources. At source points along the top of crest (generated at 1 m intervals) a viewshed analysis is undertaken, using the 2025 LiDAR 1 m DEM, combined with the adopted runout envelopes.

6.4.3 Slope regression methodology

Based on our experience with similar materials and our field observations of slopes across Port Vila, we consider the potential for slope regression to be generally limited, for the following reasons:

- The soil layer over the rock is shallow, so the potential for deep soil failures is limited.
- Groundwater is deep, so water pressure is unlikely to significantly contribute to failure.
- The rock mass contains few continuous weakness planes along which blocks could detach.
- The rock itself is cemented and largely intact.

Accordingly, we used a simplified, qualitative approach to flag the areas most likely to be affected by slope regression, as follows:

- Regression zones have been adopted for areas that sit above an assumed rockfall source.
- From the base of those slopes and rockfall source areas, an envelope has been projected back into the slope at 45° to define the zone potentially affected by regression, with a minimum width of 5 m and a maximum width of 15 m. The 15 m width was pragmatically adopted to reflect the maximum spacing of defects typically observed in the rock and the largest rock blocks that could realistically detach and fail. A GIS-based LOS approach was used to generate these areas, generally consistent with the approach described in Section 6.4.2 point 6.

This slope regression assessment is relevant for the steep, cemented and largely intact limestone slopes assessed within the Study Area, and should not be applied elsewhere.

6.4.4 Residual uncertainties in the assessment

Residual uncertainties should be considered when using this assessment, including:

- There is limited recorded information within Port Vila relating to historic rockfalls, beyond those related to the 17 December 2024 earthquake.
- While the LiDAR used in this study was high-resolution, inaccuracies in slope angle may arise where the ground surface is obscured by vegetation and buildings. Limited site observations have been undertaken to verify the accuracy of the ground profiles.
- Source areas have been assumed on slopes of sufficient height (4 m) and width (10 m) above 45°. Rockfall sources have not been verified at all locations. Rockfall is still possible on slopes too small or too shallow to be included in this study. Slope deformation could occur within other areas, however is likely limited to shallow soil failures or limited ground displacement. Localised, property-scale features were excluded from this assessment as these are considered beyond the level of detail of this assessment and will be sensitive to local-scale factors. These should be considered on a site-by-site basis.
- Not all slopes could be analysed in this study, and the zones derived have been based on median values. Site specific analysis may predict more precise values for rockfall runout.
- The runouts and slope regressions modelled in this study are for rockfall only. Shallow soil failures and deep-seated slope failures have not been analysed as they were not the typical landslide type. While other landslide types were not observed within the Study Area, this does not preclude the possibility that such failures could occur. This assessment has focused on the landslide types that were observed and are known to occur within the Study Area.
- The slope regression assessment is qualitative only, based on limited field observations.
- Future changes in slope geometry have not been considered in this analysis and can create, reduce, or eliminate hazards.

6.5 Key conclusions and recommendations

Based on our susceptibility assessments undertaken above and review of the GNS Efaté-wide assessment, we consider that the following runout and slope regression susceptibility zones, presented on Figure 6.7 with more detailed maps presented in Appendix B, should be adopted within the Geotechnical Guidance Document:

1 **Runout susceptibility**: the zoning of the area downslope of rockfall sources according to how far fallen boulders are expected to travel. Two levels of susceptibility are adopted:
   a **High:** the area within the 36° runout envelope, in which approximately 95% of fallen boulders are expected to come to rest.
   b **Medium:** the area between the 36° and 29° runout envelopes. Approximately 5% of fallen boulders are expected to travel into this area, and only approximately 0.1% (about one boulder in a thousand) are expected to travel beyond it.
2 **Runout – Awareness overlay**: The GNS wet debris zone extends beyond the limits of our assessment and considers scenarios and landslide types effectively outside the scope of this assessment. The GNS wet-condition envelope (24°) projects further downslope than our 29° envelope because it represents saturated, debris-avalanche-type runout – a different and more mobile mechanism than the dry rockfall modelled in this assessment. We recommend it is conservatively adopted only as a runout awareness overlay, considered at a planning level for critical infrastructure and structures which are of higher importance/occupancy or where post-disaster functionality is required.
3 **Slope regression – Susceptible:** a zone applied above identified rockfall source areas, projected at approximately 45° back from the base of the slope or source area and limited to a minimum of 5 m and maximum of 15 m horizontal from the slope crest. Within this zone, progressive retreat of the slope crest may occur over time as material is lost from the face. We recommend it is adopted as a slope regression susceptibility overlay, considered at a planning level when siting structures close to the crest of steep slopes and rockfall source areas.

Before new development proceeds in the susceptibility areas, a site-specific assessment should be undertaken to confirm whether, and to what extent, the proposed development is likely to be affected by rockfall. Where mitigation is required to reduce risk to tolerable levels, the measures likely to be most useful are listed below in general order of preference, based on what we consider to be practical and cost-effective for the Port Vila CBD. The Geotechnical Guidance Document provides further information to support this process:

1 **Scaling and trimming** – removal of loose, overhanging or unstable material from the slope face to reduce the source of rockfall.
2 **Catch ditch** – an excavated ditch at the toe of the slope to intercept and retain falling material.
3 **Bund** – an earth bund formed at the toe of the slope to contain rockfall and debris.
4 **Catch fence** – a flexible rockfall barrier (catch fence) installed to arrest falling material.
5 **Slope modification** – regrading or battering of the slope to a flatter, more stable geometry.
6 **Setback** – locating development away from the crest and toe of the slope. This could be considered, but is likely to result in a reduction in the land available for development.
7 **Slope meshing or anchoring** – for critical or high-importance structures, draped mesh and/or anchoring of the slope face could be considered. However, this is likely to be very expensive and to require specialist international expertise.

It should be noted that rockfall and slope regression remain possible outside the zones mapped in this report due to limitations in data and modelling.

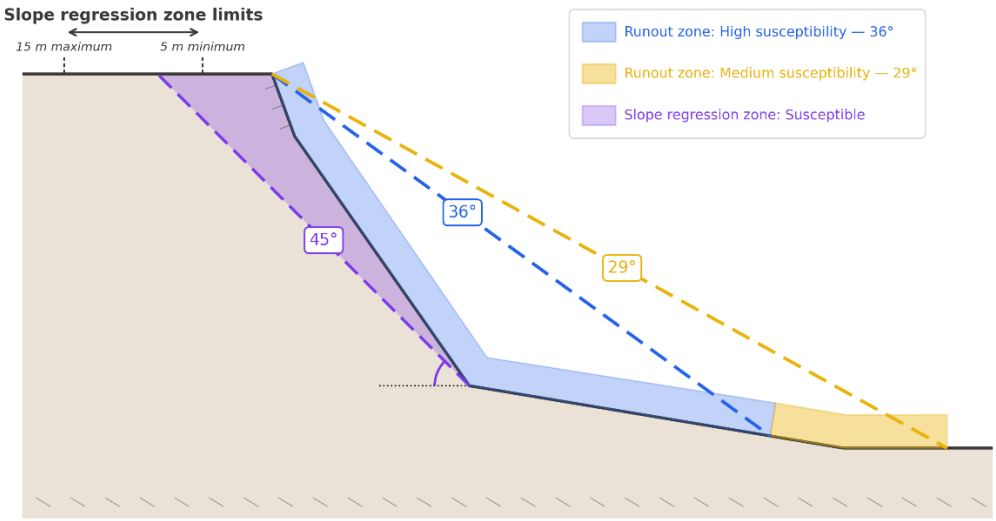


Figure 6.6: Schematic diagram presenting hazard zones

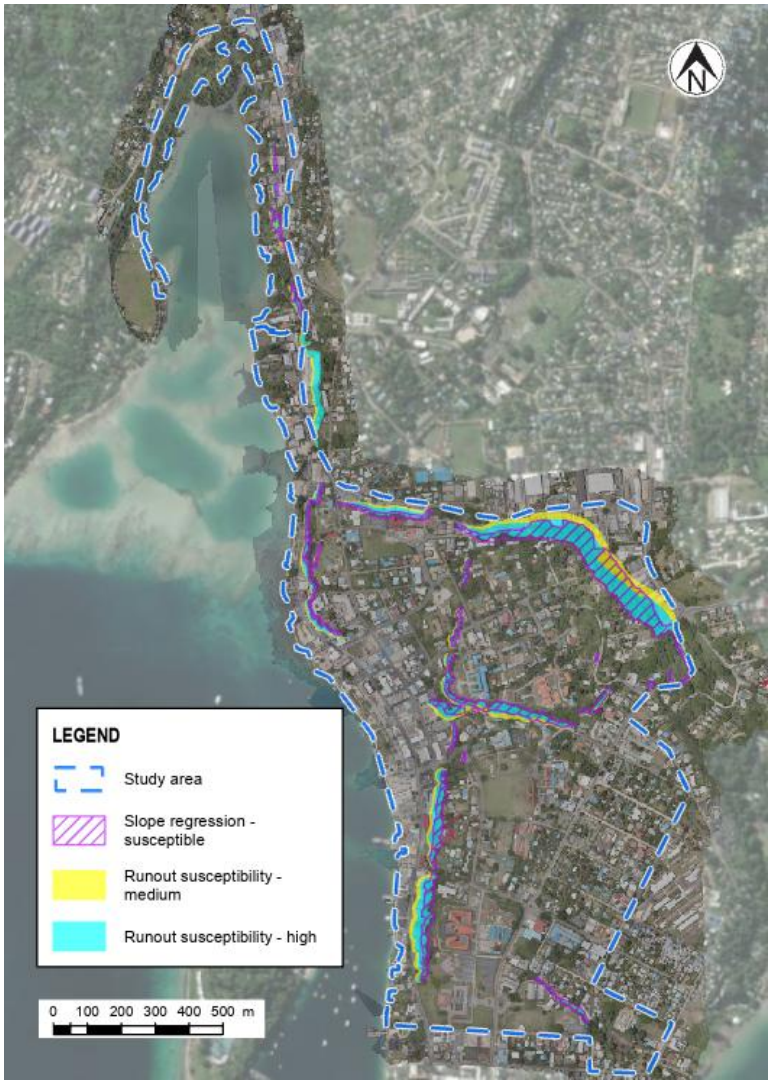


Figure 6.7: Plan view of the Study Area showing slope susceptibility zones (detailed maps in Appendix B).

7 Fault surface rupture hazard

7.1 Background

The most prominent ground deformation feature observed in the Port Vila CBD following the 17 December 2024 earthquake was an east–west trending zone of sharp subsidence extending from Rue de Paris toward the harbour waterfront. Its cause has not been directly confirmed. However, its linear geometry and proximity to mapped and inferred faults, together with the results of the GNS Science assessment, 3D geological model and LiDAR comparison, indicate that fault-related ground deformation should be considered in future rebuilding and development.

The fault surface rupture hazard considered here relates specifically to several short, upper-plate faults (mapped or inferred) that may deform the ground surface within the Study Area.

This high-level assessment comprises:

- Review of the possible causes of ground deformation within the central CBD, to establish whether fault surface rupture has occurred and, in turn, whether it could occur on other features across the Study Area.
- Refinement of possible fault locations through LiDAR mapping.
- Consideration of single-event displacement and recurrence interval parameters.
- Development of a fault deformation overlay.
- Outline of the high-level implications for development.

This assessment is intended to support pragmatic rebuilding and future development planning. It is not a detailed active-fault investigation and does not remove the need for site-specific assessment where development is proposed within or near the fault deformation overlay shown in Figure 7.11 and Appendix C.

7.2 Fault rupture terminology

A fault is a break or zone of weakness in the Earth's crust along which the ground on one side has moved relative to the other. A fault may remain locked for long periods, then slip suddenly during an earthquake. If that slip reaches the ground surface, it may form a visible step, crack, tear or zone of warping. This is referred to as surface fault rupture. The following terms are used in this section:

- **Primary fault rupture** means surface rupture on the fault that generated the earthquake.
- **Secondary fault rupture** means deformation on nearby faults caused by the main rupture.
- **Triggered slip** means movement on a pre-existing fault caused by earthquake shaking or stress changes from an earthquake on another fault. Triggered slip may produce permanent ground deformation even though the fault is not the primary earthquake source.
- **Normal faulting** means fault movement where ground lying above the fault (hanging wall) has moved downward relative to the ground lying below the fault (footwall).
- **Active fault**, for the purpose of this adopted methodology, means a fault that has moved in the recent geological past and could move again. The current New Zealand Ministry for the Environment (MfE) framework (Kerr et al., 2003) defines an active fault as one that has deformed the ground surface within the last 125,000 years; faults with average recurrence intervals greater than 125,000 years are generally not subject to planning controls under that framework.

Figure 7.1 illustrates the distinction between primary rupture, secondary rupture, and triggered slip.

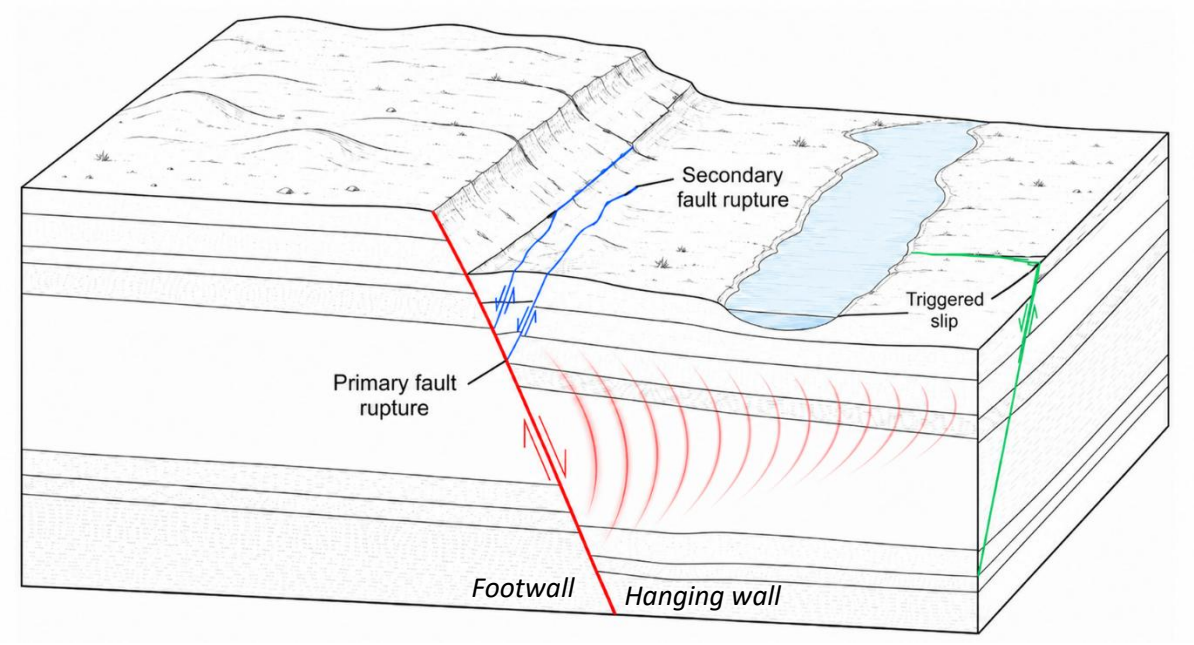


Figure 7.1: Schematic diagram showing primary and secondary rupture, and triggered slip.

7.3 Central CBD ground deformation from the 17 December 2024 earthquake

An approximately 200 m long, east–west trending zone of subsidence was observed in the central CBD following the 17 December 2024 earthquake, shown on Figure 7.2 and Photograph 7.1. Vertical displacement along the feature was approximately 100 to 400 mm. The feature caused significant damage to several structures, including the National Bank of Vanuatu (NBV), a single-level store, the Pacific Energy service station, and minor damage to the seawall.

Two mechanisms could plausibly explain the deformation:

1 **Earthquake-induced soft-ground settlement**, meaning permanent settlement of loose or soft ground caused by shaking, including dynamic compaction and/or liquefaction-induced settlement; or

2 **Triggered slip**, meaning minor movement on a pre-existing fault caused by stress changes or shaking from the December 2024 earthquake.

These mechanisms are not mutually exclusive in the lower CBD, where reclamation fill and intra-reef deposits susceptible to liquefaction are present.

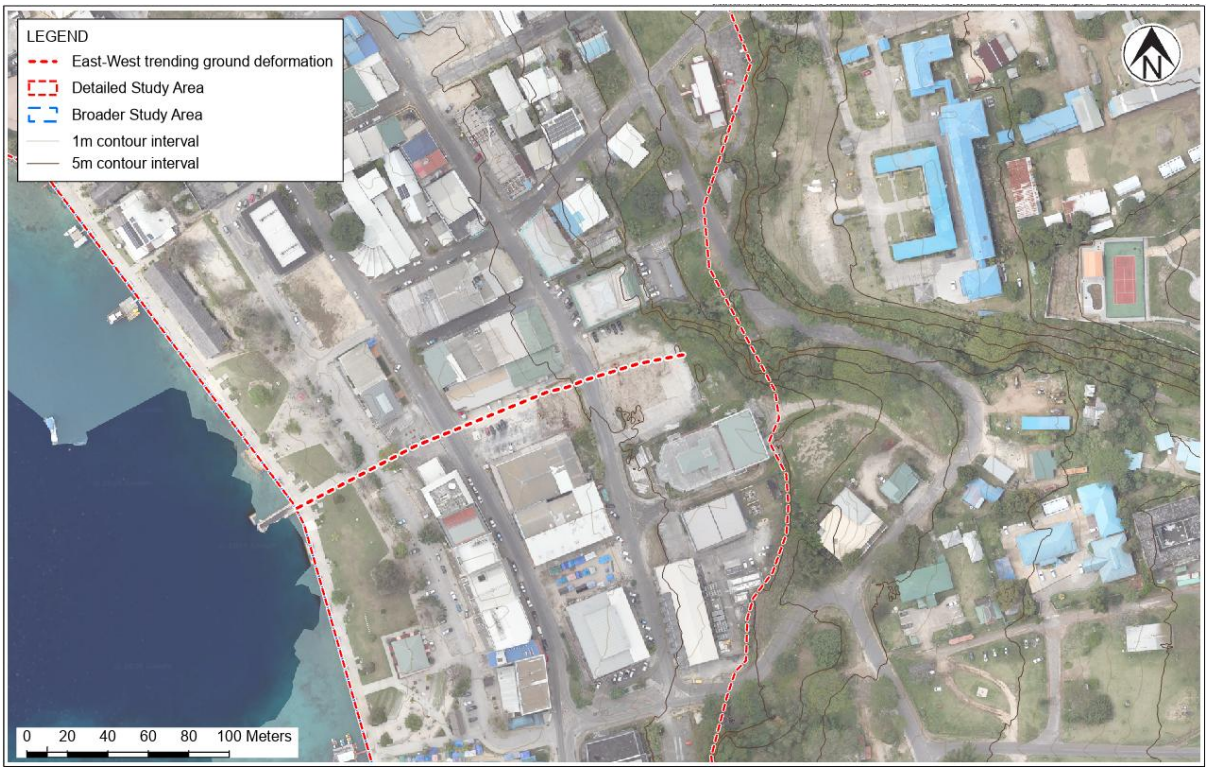


Figure 7.2: Location of the east–west trending ground deformation feature observed in the central CBD. Post-earthquake drone photography taken in November 2025, following demolition of the store at the central carpark and the NBV building.



Photograph 7.1: Looking east along trend of ground deformation through the downtown CBD area.

7.3.1 GNS interpretation

GNS assessed the central CBD deformation in its 2025 surface fault rupture and liquefaction hazard report (Langridge et al., 2025). The key points from that assessment are:

- The approximately 200 m long east–west cracking could represent an extension of the northwest–southeast trending "Palace Fault" toward the coastline.
- The Palace Fault may be a possible active fault under the adopted New Zealand active-fault criterion, based on the interpretation that Government Hill may be capped by a Marine Isotope Stage 5e (MIS 5e) marine terrace, broadly dated to approximately 115,000 to 128,000 years before present.
- GNS could not confirm whether the cracking within the CBD represents faulting, settlement or gravitational movement, but recommended that it be treated as potentially fault-related when planning the CBD rebuild.
- GNS also concluded that possible fault-related ground-deformation features mapped after the December 2024 earthquake were generally short, up to approximately 0.3 km, and had small displacements, up to approximately 0.4 m. Given the depth and intra-slab source of the earthquake, GNS considered such deformation, if fault-related, to be more consistent with secondary faulting or triggered slip than primary rupture.

We consider this a reasonable conservative starting point. However, the additional project-specific ground model and LiDAR difference assessment provide further constraints on the likely mechanism and extent of deformation.

7.3.2 3D geological model evidence

The 3D ground model described in Section 3.3 provides important evidence because it shows a sharp and laterally persistent contrast in ground conditions across the central CBD deformation feature.

The feature is constrained by 12 machine boreholes within approximately 20 m of the observed deformation and by four geophysical lines with testing at approximately 20 m spacing. These data show:

- **North of the deformation feature:** shallow Reef Limestone Formation, generally encountered at approximately 0 to 2 m depth.
- **South of the deformation feature:** reclamation fill overlying deep intra-reef deposits, comprising loose to medium dense, cohesionless sand and gravelly sand extending to approximately 18 m below ground level. As discussed in Section 5.3.3, these deposits are susceptible to liquefaction.

A distinctive vuggy limestone sub unit within the Reef Limestone Formation was also encountered across much of the Study Area. "Vuggy" means the limestone contains visible cavities or voids. This unit is interpreted as a marker horizon associated with a distinct period of coral growth. Where paired boreholes are located on either side of the central deformation feature, the marker horizon appears vertically offset by approximately 9.5 to 12.5 m. This scale and consistency of offset is unlikely to be explained solely by local paleo-topography or erosion. We interpret the offset as evidence of geological fault displacement across this boundary.

Figure 7.3 shows the sharp contrast in ground conditions along the east–west deformation feature.

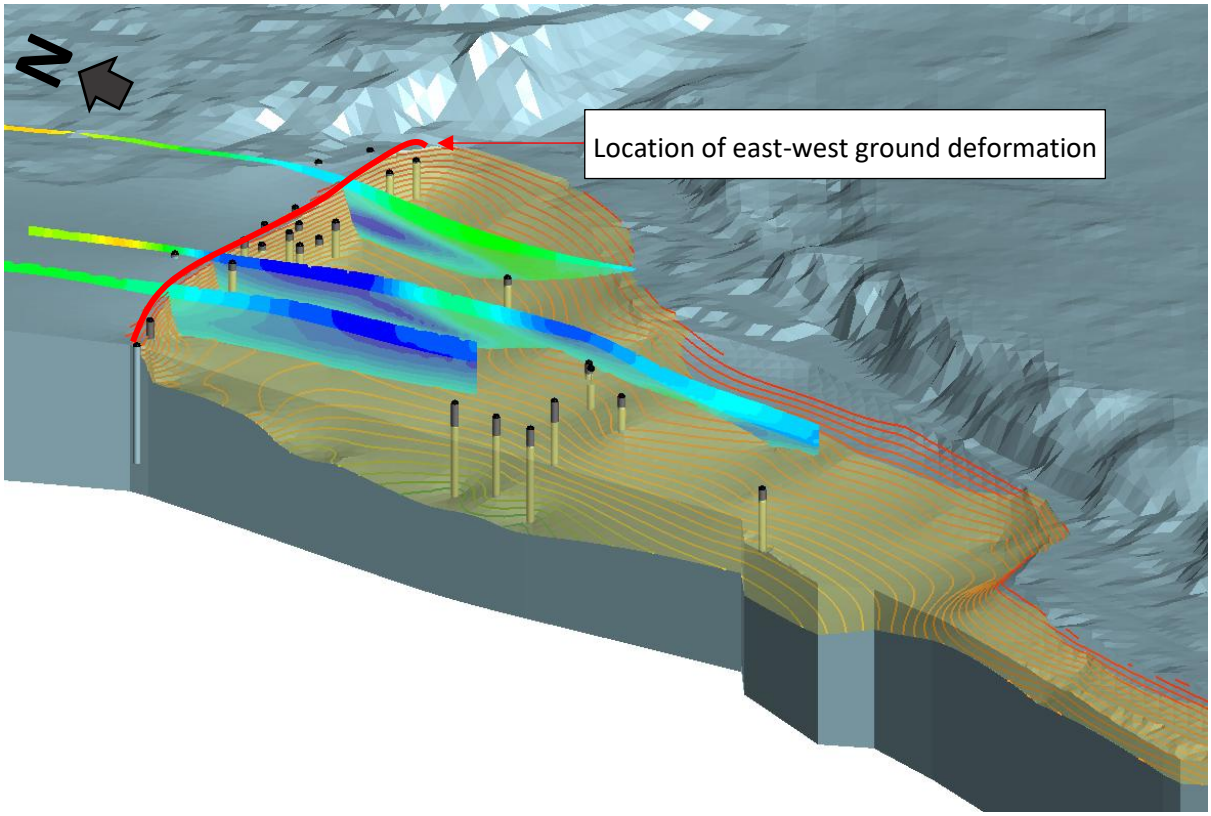


Figure 7.3: Excerpt from the 3D geological model showing the sharp contrast in ground conditions along the east–west trending ground deformation feature. Bluish-grey shading = Reef Limestone Formation.

7.3.3 LiDAR vertical difference evidence

LiDAR data provide an independent way to compare ground levels before and after the 17 December 2024 earthquake. The 2025 Kahuto Pacific drone LiDAR and photogrammetry survey was compared with the 2012 Geoscience Australia airborne LiDAR dataset to assess vertical ground movement across the Study Area.

The two datasets were collected using different benchmarks and survey methods. Expected regional tectonic deformation also affects the comparison. To reduce these effects, local control points were selected at locations where position and elevation were judged to have remained reasonably stable between surveys. The 2012 dataset was then adjusted to align with the 2025 survey. Figure 7.4 presents the LiDAR vertical difference interpretation. The LiDAR comparison could not reliably resolve horizontal movement from the available hard structures visible in both datasets.

Based on this comparison, we consider that:

- Subsidence extends farther east than the initially mapped 200 m feature shown in Figure 7.4, likely by approximately 400 to 500 m, and aligns with the base of the west-northwest–east-southeast trending Palace Fault escarpment.
- In the lower CBD, at approximately 1 to 5 m reduced level (m RL), subsidence adjacent to the feature is of the order of 200 to 400 mm. This subsidence reduces to approximately 100 mm over a horizontal distance of 200 m towards the south.
- In the upper area, at approximately 20 to 40 m RL, subsidence adjacent to the escarpment is of the order of 200 mm. This subsidence reduces to approximately 100 mm over a horizontal distance of 200 m towards the south.

- The upper area is underlain by Reef Limestone Formation over Efaté Pumice Formation to significant depth, and is not expected to contain soils susceptible to liquefaction or volumetric consolidation under earthquake loading.
- The largest subsidence in the lower CBD occurs near the east–west deformation feature and near the seawall, where liquefaction-susceptible intra-reef deposits are known to be present.

The comparison is subject to uncertainty. The 2012 dataset is relatively sparse in places, control points were selected by judgement, and LiDAR vertical accuracy is typically of the order of ±100 mm, potentially greater for the 2012 survey. These limitations affect the precision of individual displacement values. However, we consider the overall subsidence pattern sufficiently reliable for this high-level assessment.

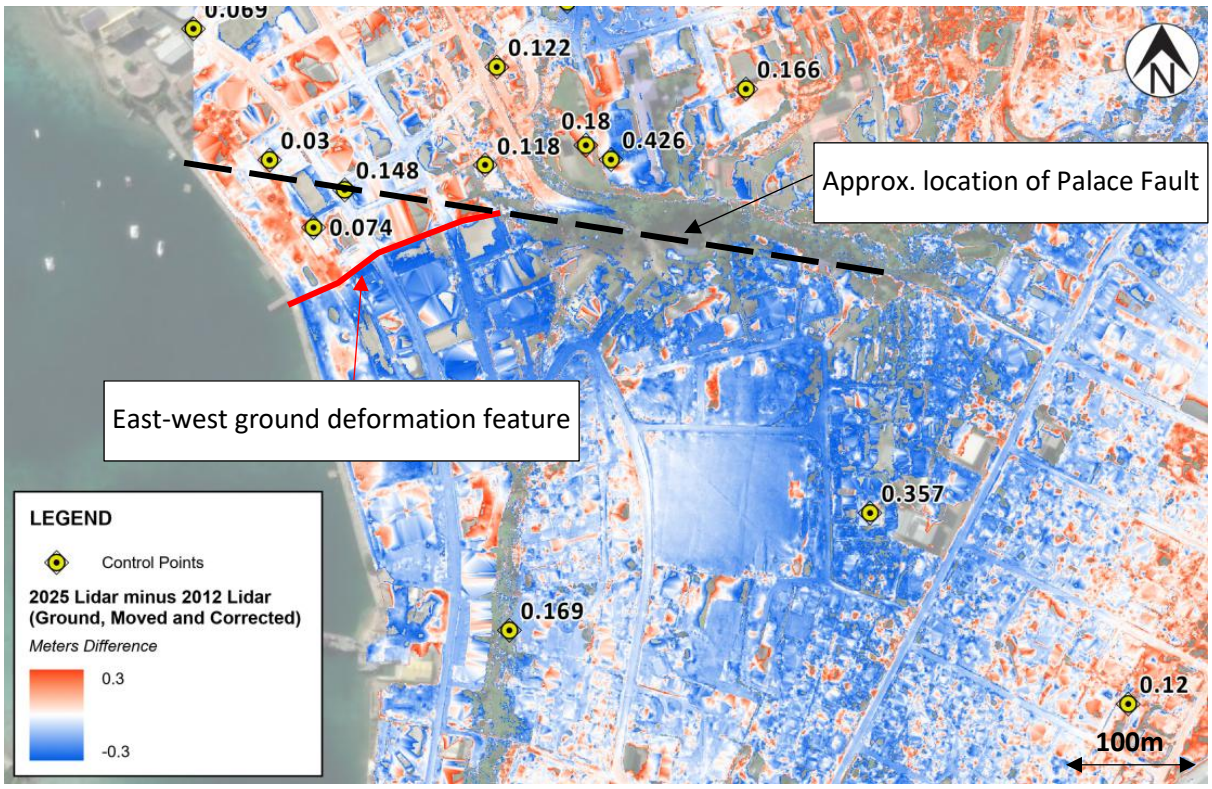


Figure 7.4: Approximation of vertical difference between the 2012 and 2025 LiDAR datasets. Blue = lower in 2025; red = higher in 2025. Yellow dots show elevation correction adjustments, in metres, at each control point location.

7.3.4 Tank Hill fault exposure and geophysical evidence

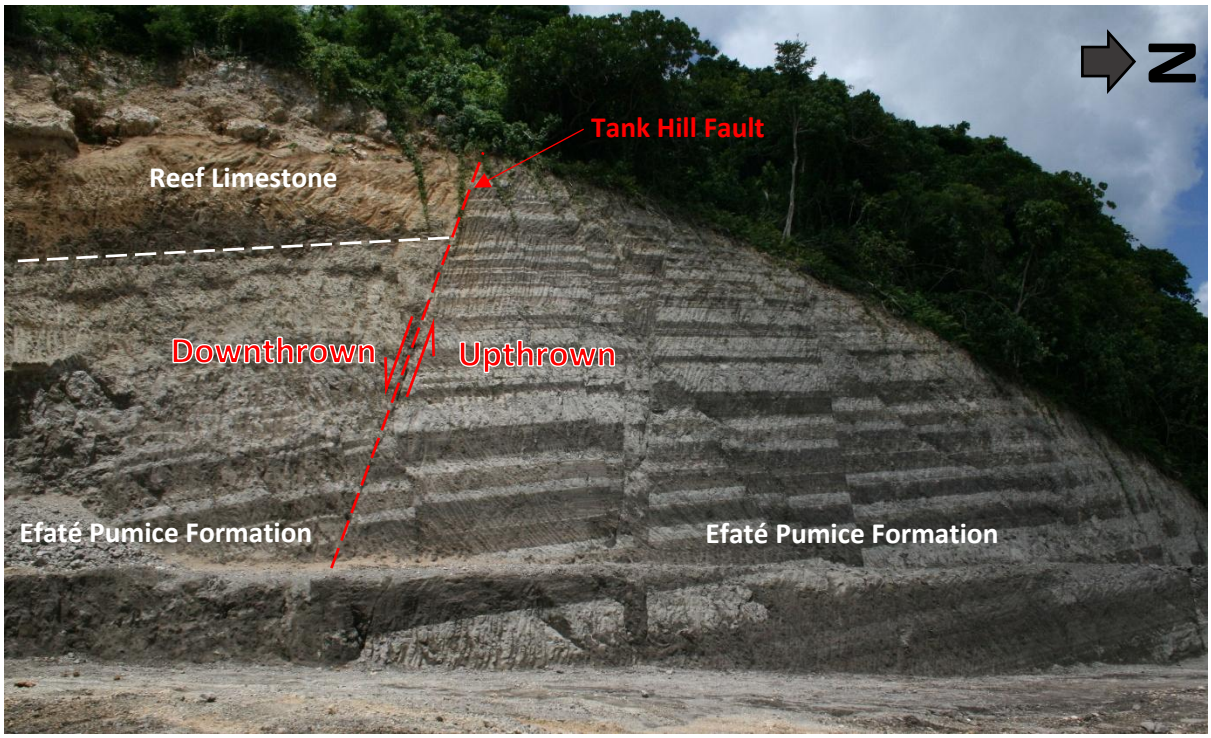
In 2014, a fault was exposed during excavation of a slope near the Teoma Road roundabout. This fault is referred to informally in this report as the Tank Hill Fault. The exposure showed a steeply dipping normal fault, with the southern side downthrown relative to the northern side. The measured fault orientation was strike/dip 154/72°. Several smaller secondary faults were observed on the northern side. On the downthrown southern side, Reef Limestone Formation overlies the older Efaté Pumice Formation, indicating that the limestone cap has been vertically displaced by movement on the fault.

Two additional lines of evidence support the interpreted continuation of this fault toward the lower CBD:

- A multi-channel analysis of surface waves (MASW) geophysical line at Joint Court Ontop, referred to as MASW Line 11b, shows a sharp change in ground stiffness across the inferred fault line; and
- Projecting the measured fault orientation from the Teoma Road exposure, calibrated to the MASW result, places the fault trace within approximately 20 m of the east–west ground deformation feature in the lower CBD.

We consider the sharp contrast in ground conditions within the lower CBD to be more consistent with an escarpment formed by the Tank Hill Fault than by the Palace Fault alone. However, between the Presidential Palace and the Teoma Road roundabout, neither field observations nor the LiDAR comparison identified clear December 2024 ground deformation along the projected Tank Hill Fault trace.

Photograph 7.2 shows the fault exposure near the Teoma Road roundabout, and Figure 7.5 shows the projected Tank Hill Fault trace toward the lower CBD.



Photograph 7.2: Photograph of the fault exposure in the excavation near Teoma Road roundabout, taken in 2014.

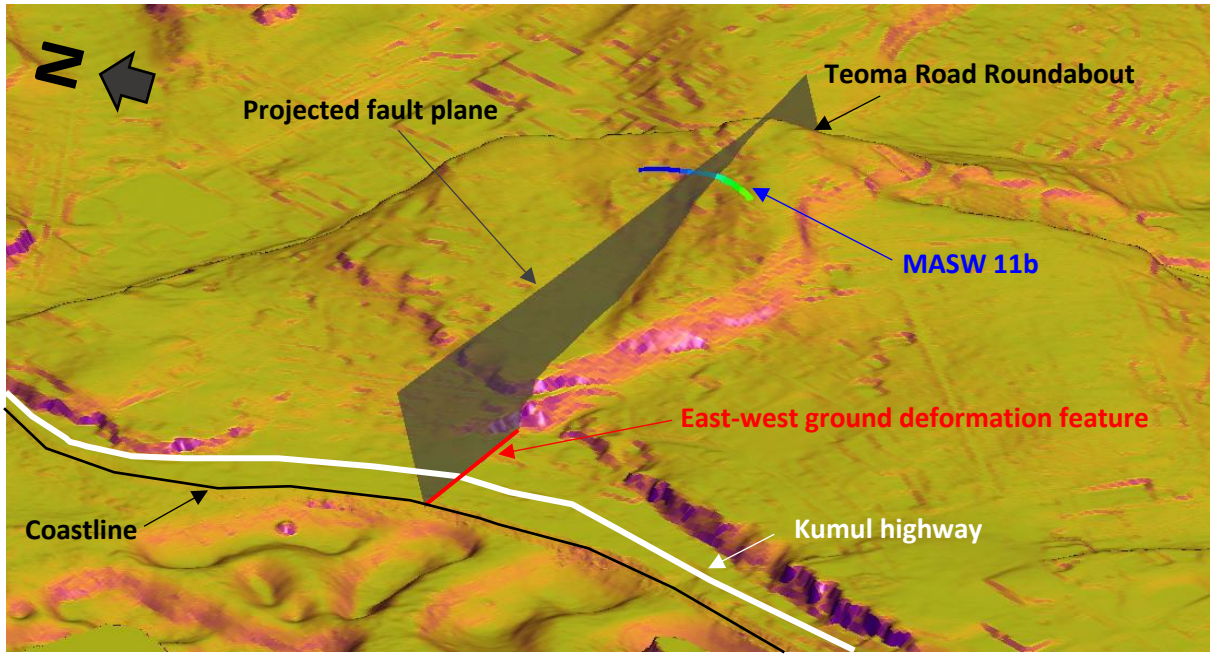


Figure 7.5: 3D view showing the Tank Hill fault trace projected from Teoma Road roundabout exposure (Photograph 7.2), through MASW 11b to the lower CBD area where the ground deformation has occurred.

7.4 Interpretation of the December 2024 deformation mechanism

This subsection brings the evidence together and presents the preferred interpretation. The interpretation is necessarily cautious because no trenching or direct dating has been completed.

We interpret the December 2024 central CBD deformation as follows:

1 In the lower CBD, the observed deformation aligns with a sharp boundary between shallow Reef Limestone Formation to the north and deeper, loose reclamation and intra-reef sediments to the south. The vuggy limestone marker horizon is vertically offset by approximately 9.5 to 12.5 m across this boundary, indicating that fault displacement has accumulated over geological time.

2 The deformation extends beyond the lower CBD into an upper area underlain by Reef Limestone Formation and Efaté Pumice Formation, where liquefaction or volumetric consolidation under earthquake loading are not expected to explain the observed subsidence. Soft-ground settlement alone is therefore not a sufficient explanation for the overall deformation pattern.

3 The lower CBD deformation aligns closely with the projected Tank Hill Fault. The extended deformation to the east aligns with the base of the Palace Fault escarpment. We interpret the event as movement of a compartmentalised block along pre-existing planes of weakness associated with these faults.

4 The 17 December 2024 earthquake source was not one of these mapped CBD faults. The observed deformation is therefore interpreted as triggered slip rather than confirmed primary surface rupture.

5 In the lower CBD, particularly near the seawall and within the liquefaction-susceptible intra-reef deposits, fault-related subsidence and liquefaction-induced settlement may both have contributed to the total observed ground movement. The relative contribution of each mechanism cannot be separated with confidence from the available data.

The interpreted east-west ground deformation feature relative to the inferred Tank Hill and Palace Fault traces is shown in Figure 7.6. Several other ground elevation anomalies can be observed in Figure 7.6 that are unrelated to the December 2024 earthquake. These instead reflect pre-existing development and associated earthworks (such as the seawall, carpark and presidential palace) or increased error in the datasets along steep, vegetated slopes.

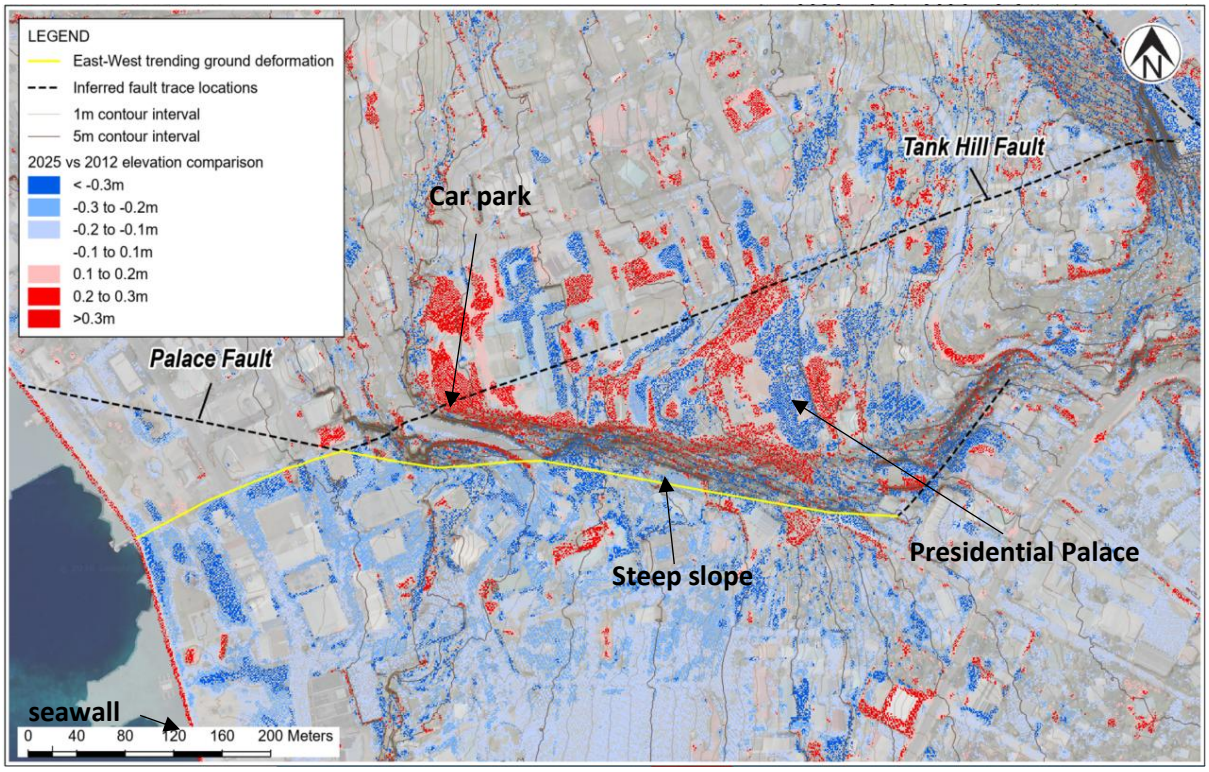


Figure 7.6: East-west ground deformation feature shown in yellow relative to inferred geological fault locations, with ground subsidence south of the feature shown in various shades of blue.

7.5 Fault trace mapping and activity assessment

This subsection reviews the mapped faults in the Study Area and explains which have been carried forward into the fault deformation overlay.

The published geological map (Ash et al., 1978) shows several features mapped as faults or probable small faults within the Study Area. These features were mapped at regional scale and do not align precisely with the 2012 and 2025 LiDAR datasets. We have remapped the inferred fault traces for this study using the 2025 LiDAR, the 3D geological model and field observations.

Within the Study Area, the published map shows five features mapped as faults and two further features mapped as “lineaments, probably small fault”. On review, we consider the two lineaments, L-1 and L-2, and one north–south mapped feature to be more likely uplifted sea cliffs separating marine terrace levels than active fault traces. These features trend north–south and are geomorphically similar to other uplifted coastal scarps in Port Vila that have not been mapped as faults. They are therefore not carried forward into the fault deformation overlay.

The remaining roughly east–west escarpment features are interpreted as possible fault-related features. They appear to form part of a broader set of linear escarpments extending several kilometres eastward toward the Teouma River Valley. Figure 7.7 and Figure 7.8 show the mapped features and wider escarpment pattern.

Our inferred fault trace mapping assumes that, where a clear escarpment is present, the fault trace is located near the base of the escarpment. This is considered reasonable for this high-level assessment because the intact coral reef limestone is relatively resistant to erosion. However, where erosion or modification has occurred, the true fault trace may lie upslope or mid-scarp. In lower-lying areas where younger deposits may bury the original scarp, the fault trace is treated as a well-defined extended trace rather than a directly exposed trace. Figure 7.9 shows the LiDAR-based inferred fault trace mapping adopted for this assessment.

The fault activity assessment is based on three main lines of evidence:

1 Partial lengths of at least two mapped faults, interpreted as the Tank Hill Fault and Palace Fault, experienced surface deformation during the 17 December 2024 earthquake as triggered slip. This confirms that these structures, or structures coincident with them, can accommodate surface deformation during strong earthquakes. It does not confirm that they are capable of generating primary surface rupture earthquakes.

2 GNS Science inferred possible fault activity from uranium-thorium (U/Th) dating of uplifted reefs and marine terraces in the Port Havannah area. U/Th dating is a geochronological method used to estimate the age of carbonate materials such as coral reef limestone. GNS interpreted the uppermost terrace at Government Hill, at approximately greater than 70 m RL, as a possible MIS 5e surface, broadly dated to about 115,000 to 128,000 years before present.

3 Lecolle et al. (1990) reported additional U/Th dates at Point d'Arbel, approximately 2 km west of Port Vila. These indicate a first terrace at approximately 10 m RL dated to 39.2 thousand years before present (39.2 ka), and a second terrace at approximately 25 m RL dated to 125 ka. These dates do not align well with the Port Havannah interpretation and should be treated with caution, but they do not remove the possibility that the east–west faults have been active within the time period relevant to the adopted MfE framework.

Based on the available evidence, we consider it appropriate to treat the mapped east–west fault features carried forward in this assessment as possibly active under the adopted New Zealand methodology. The activity of individual faults remains uncertain and should be confirmed by more detailed investigation if development decisions depend on the result.

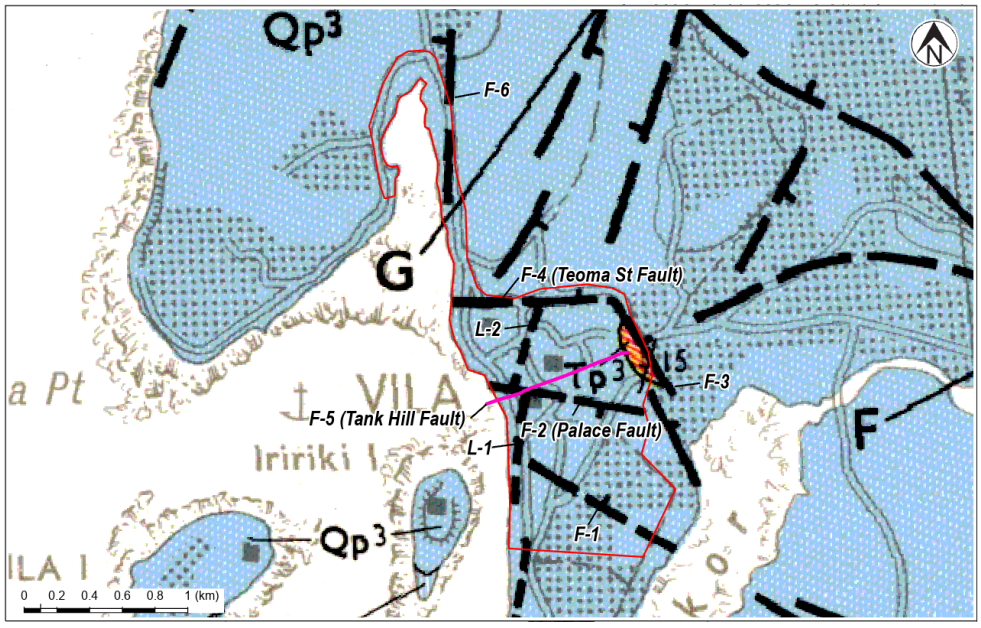


Figure 7.7: Basemap of *Geology of Efaté 1:100,000 New Hebrides Geological Survey Sheet 9 with T+TI markups of the faults and lineaments within the Study Area.*

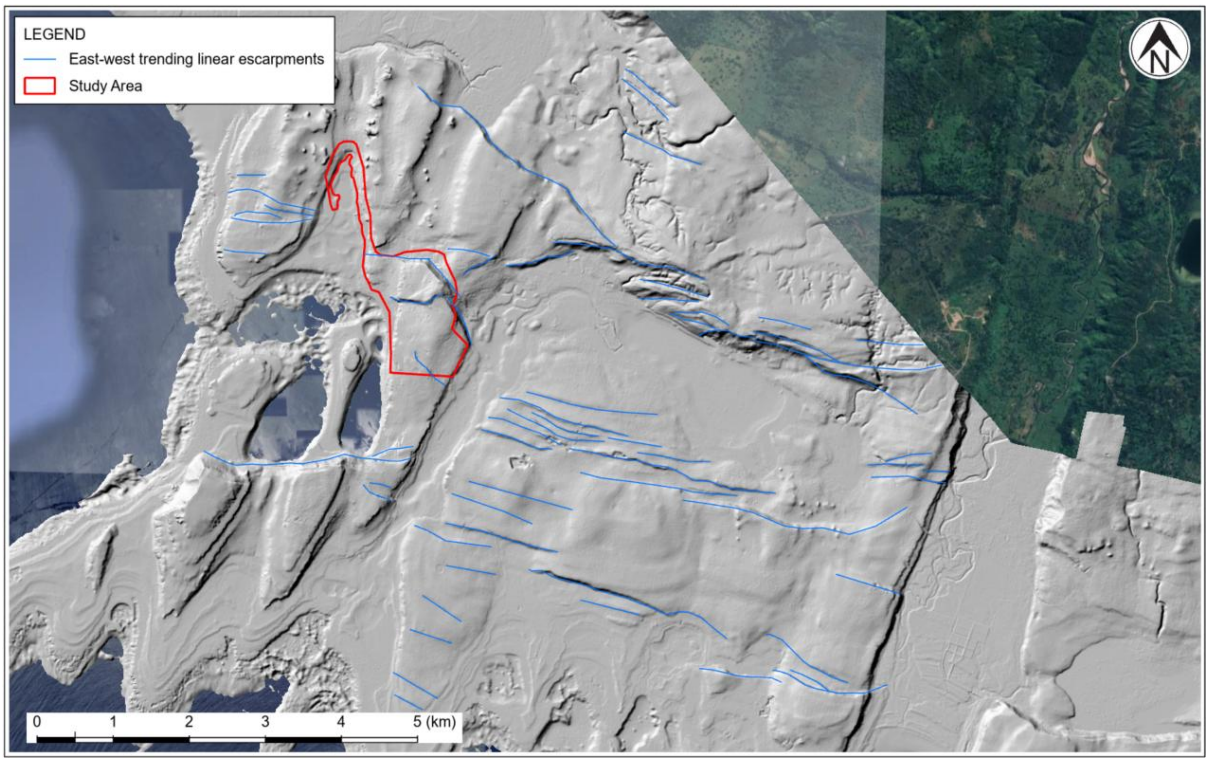


Figure 7.8: East–west trending linear escarpments shown on hillshade generated from the 2012 LiDAR DEM.

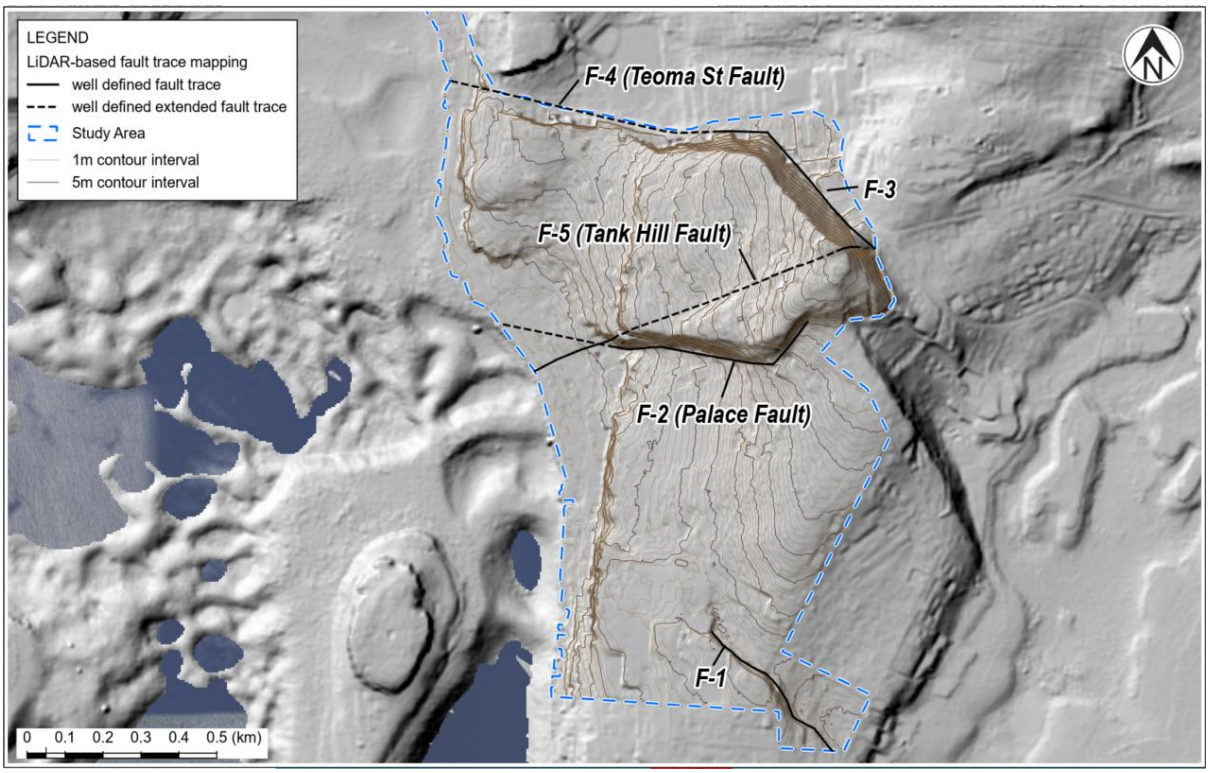


Figure 7.9: LiDAR-based mapping of inferred fault trace locations within the Study Area.

7.6 Indicative displacement and recurrence parameters

This subsection provides preliminary numerical parameters for screening-level planning and design consideration. These estimates are based on empirical relationships and geological inference, not direct palaeoseismic investigation, and should be used carefully with explicit consideration given to how sensitive design and decision-making outcomes could be to uncertainties in the estimated parameters.

Single-event displacement means the amount of permanent ground movement that occurs during one surface-rupture event. For this high-level assessment, single-event displacement has been estimated using the empirical surface-rupture displacement relationships of Wells and Coppersmith (1994). This method is commonly used for preliminary engineering assessments where direct fault-specific data are not available. The method has substantial scatter, particularly for normal faults, and the results should be treated as indicative only.

Based on aerial imagery and hillshade models derived from the 2012 LiDAR dataset, we consider an indicative rupture length of approximately 10 km to be representative for the east–west fault features that appear to continue beyond the Study Area. Using this rupture length and the Wells and Coppersmith (1994) normal-fault relationships (refer Figure 7.10) gives:

- **Average single-event displacement** expected along the whole rupture: approx. 180 mm.
- **Maximum single-event displacement** expected anywhere on the rupture: approx. 340 mm.

These values are of the same order as the 200 to 400 mm subsidence observed in parts of the lower CBD following the 17 December 2024 earthquake.

The recurrence interval is the average time between surface ruptures or surface-deformation events on a fault. A preliminary recurrence interval has been estimated by dividing the indicative single-event displacement by an indicative slip rate.

The indicative slip rate is based on the following assumptions:

- The 39.2 ka terrace unit dated by Lecolle et al. (1990) at Point d'Arbel may correlate with the vuggy limestone marker unit identified across the lower CBD;
- The vuggy limestone marker is vertically offset by approximately 9.5 to 12.5 m across the central CBD deformation feature; and
- The offset divided by the inferred age of the marker provides an indicative long-term vertical slip rate.

Using these assumptions gives an indicative slip rate of approximately 0.25 to 0.32 mm/year. Combining this slip rate with the indicative single-event displacement gives an approximate recurrence interval of 600 to 1,400 years. Under the adopted MfE framework, this would fall within Recurrence Interval Class I, which is defined as less than or equal to 2,000 years.

This recurrence interval estimate is sensitive to the assumed age correlation, the measured offset, the selected rupture length and whether the 2024 deformation represents the dominant displacement mechanism. It is based on one interpreted offset across one fault and should not be assumed to apply equally to all mapped faults in the Study Area.

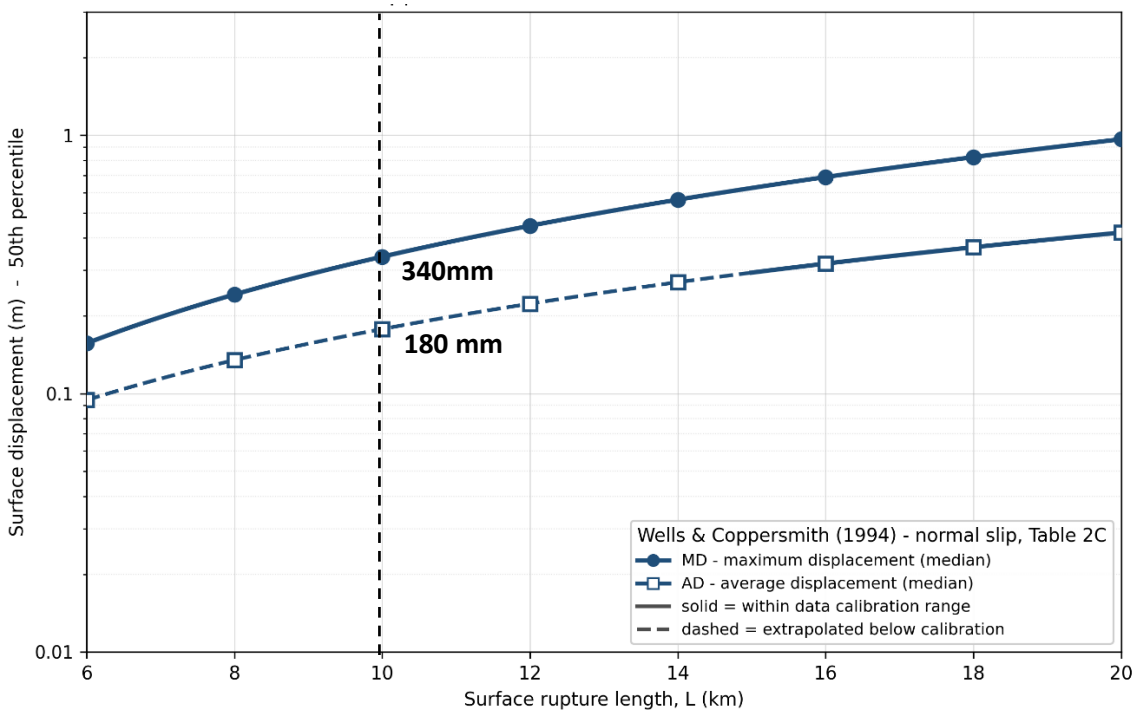


Figure 7.10: Empirical surface-rupture displacement relationships for 6 to 20 km surface rupture lengths, derived from Wells and Coppersmith (1994).

7.7 Fault deformation overlay

This subsection explains how the mapped fault information could be used in practice. The fault deformation overlay is a trigger for assessment and risk management, not a recommendation that development be automatically prohibited.

The MfE guideline recommends a minimum Fault Avoidance Zone (FAZ) buffer of 20 m either side of a known fault trace or likely rupture zone. In the absence of a Vanuatu-specific active-fault planning framework, we recommend a fault deformation overlay for the Study Area based on this minimum buffer, adapted to the local mapping uncertainty.

The recommended fault deformation overlay comprises the greater of:

- A 20 m width either side of the mapped or inferred fault trace; and
- Where an escarpment is present, extension of the overlay to the crest of the escarpment to account for the possibility that the true fault trace lies mid-scarp rather than at the mapped base.

This approach reflects the relatively well-defined nature of the visible escarpments, the tightly constrained deformation observed in December 2024, and the sharp subsurface contrast identified in the 3D geological model. Where the fault trace is buried beneath younger lower-elevation deposits, the trace is classified for this assessment as well-defined extended.

Figure 7.11 shows the recommended fault deformation overlay across the Study Area. The detailed fault deformation overlay is presented in Appendix C.

Future development within or near the overlay should not be automatically precluded. The overlay should trigger assessment of credible deformation effects, consideration of within-site avoidance where practicable, and design for robustness where the residual risk can be made tolerable. Avoidance of the overlay is recommended where effects cannot reasonably be accommodated.

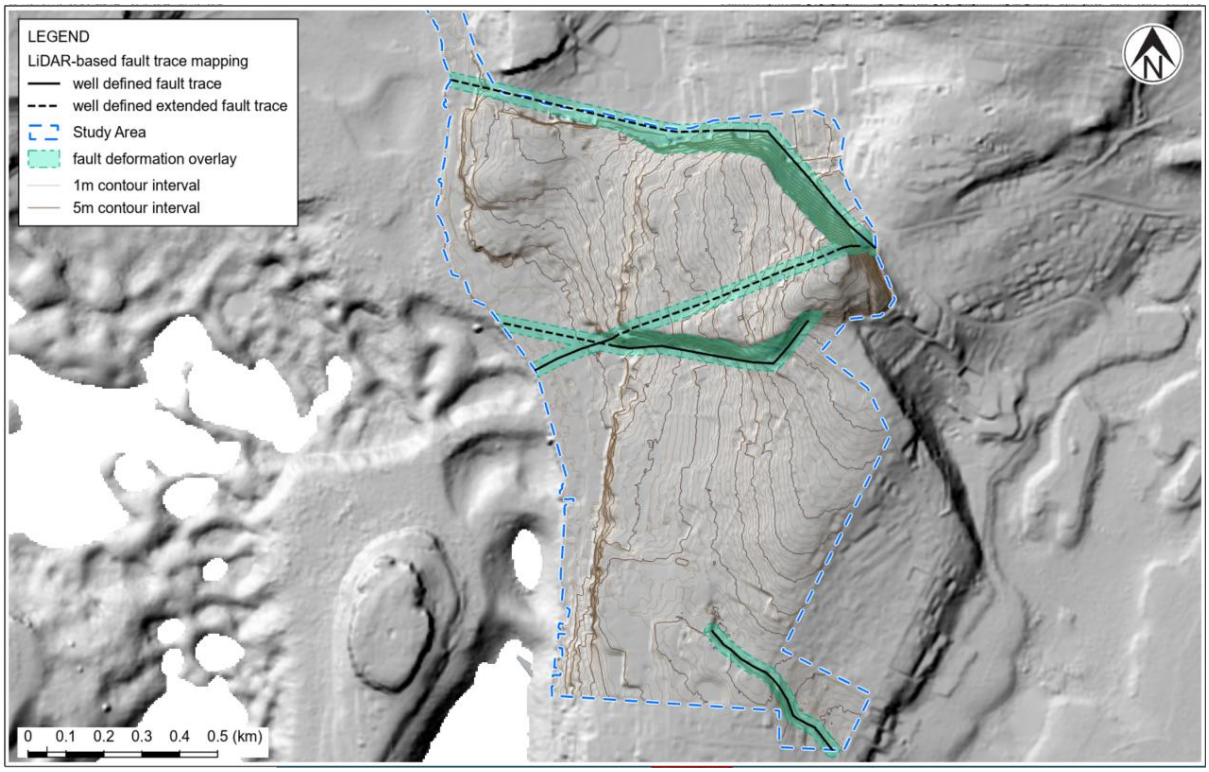


Figure 7.11: Summary of the recommended fault deformation overlay across the Study Area.

7.8 Residual uncertainties

The following residual uncertainties should be considered when using this assessment:

- No fault trenching, detailed palaeoseismic logging or direct dating of faulted units has been undertaken within the Study Area. Fault activity, single-event displacement and recurrence interval therefore remain high-level estimates.
- It has not been established whether the mapped faults are capable of generating primary surface rupture, or whether their dominant future hazard is triggered slip during earthquakes on other sources.
- In the lower CBD, the deformation feature coincides with liquefaction-susceptible intra-reef deposits and increased subsidence near the seawall. The relative contributions of fault displacement and liquefaction-induced settlement cannot be separated with confidence.
- The classification of faults as possibly active relies on inferred marine terrace ages and correlations. The Port Havannah and Point d'Arbel U/Th dating interpretations do not align well, and neither directly dates the faulted surfaces within the Study Area.
- Only partial lengths of two faults are interpreted to have moved during the December 2024 event. The activity of other mapped east–west features is inferred from their similar geomorphic expression and apparent continuity.
- The single-event displacement estimate is based on the Wells and Coppersmith (1994) empirical relationships and an assumed rupture length of approximately 10 km. A different rupture length would change the estimated displacement.
- The indicative slip rate and recurrence interval depend on the unconfirmed assumption that the offset vuggy limestone marker correlates with the 39.2 ka unit dated at Point d'Arbel.

- Fault traces have been mapped from LiDAR, field observations and the 3D geological model. Where erosion, colluvial burial, reclamation or excavation has modified the ground surface, the true trace may differ from the mapped trace.
- The LiDAR vertical difference assessment is affected by differences in survey method, control-point selection, the sparse 2012 dataset, vegetation, anthropogenic changes to ground levels, and typical LiDAR vertical accuracy. These limitations mainly affect individual displacement values rather than the overall interpreted trend.
- The fault deformation overlay is not an absolute boundary between hazardous and non-hazardous land. Local deformation may occur outside the mapped overlay, and parts of the overlay may be shown by more detailed assessment to have lower or higher hazard than currently inferred.

7.9 Key conclusions and recommendations

Based on our assessment undertaken, we conclude that:

1 The central CBD ground deformation observed after the 17 December 2024 earthquake was most likely caused by triggered slip or secondary movement on pre-existing upper-plate faults associated with the Tank Hill Fault and Palace Fault. In the lower CBD, liquefaction-induced settlement may also have contributed to the total observed subsidence.

2 The deformation was not primary rupture of the December 2024 earthquake source fault, which was a moderately deep intra-slab event west of Port Vila.

3 The east–west mapped fault features carried forward in this assessment should be treated as possibly active against the <125,000-year active-fault criterion used as a technical benchmark by GNS and in New Zealand practice. This does not constitute a Vanuatu statutory classification.

4 Indicative empirical assessment using an approximately 10 km rupture length gives average and maximum single-event displacement estimates of approximately 180 mm and 340 mm respectively. These values are broadly consistent with the order of displacement observed in December 2024, but remain first-order estimates.

5 An indicative slip rate of approximately 0.25 to 0.32 mm/year and recurrence interval of approximately 600 to 1,400 years has been inferred from the offset vuggy limestone marker and assumed age correlation. If expressed using New Zealand recurrence interval class terminology, this would fall within Recurrence Interval Class I. This classification should be treated as preliminary because the age correlation and offset history are not confirmed.

6 The fault deformation overlay shown on Figure 7.11 and Appendix C has been adopted in the Geotechnical Guidance Document. The overlay should comprise the greater of a 20 m width either side of the mapped or inferred trace, or extension to the crest of a visible natural escarpment where the trace could plausibly lie upslope or mid-scarp.

There is likely merit in gaining consensus with specialists in the field (such as GNS or Geoscience Australia) on the presence of these faults, and the associated activity and single event displacement that can be expected on these features.

For future development within or near the fault deformation overlay, the Geotechnical Guidance Document presents a potential approach to risk management. These are some of the principles that have formed the basis of the guidance:

1 **Confirm whether the proposed works intersect the fault deformation overlay.**

2 **Avoid the fault deformation overlay within the site where practicable.** If the development layout allows, structures, critical foundations and important services should be positioned away from the overlay within the same site. This is preferred where it can be achieved without creating other material risks or impractical development outcomes.

3 **Assess how deformation could affect the development.** Consider whether the foundation and structural design that would otherwise be specified can tolerate differential vertical ground deformation of the order of 200 to 400 mm, together with potential cracking, tilt and localised differential settlement. In some cases a robust foundation may already be required for other reasons (e.g. liquefaction or soft ground), and this may already provide a solution which can accommodate the potential fault-induced ground displacement.

4 **Design for robustness to accommodate deformation and accept residual risk**. For ordinary structures, a realistic objective may be life safety and collapse prevention rather than avoidance of all damage or immediate post-earthquake functionality. Measures may include well-tied foundations, regular structural form, detailing that can tolerate differential movement, flexible service connections, and foundation systems that can be inspected and, where feasible, re-levelled.

5 **Apply a higher standard for critical infrastructure and post-disaster facilities.** Critical infrastructure and facilities requiring post-disaster functionality should be located outside the overlay where practicable. If that is not practicable, the potential deformation should be included in the geotechnical and structural design, and the residual risk should be accepted only where it is consistent with the required performance objective.

6 **Avoid the overlay where effects cannot be made tolerable**. If the expected deformation cannot be reasonably accommodated by siting, foundation design, structural detailing or service design, the overlay area should be avoided for that development or use.

It has not been established that fault surface rupture poses a greater risk than ground shaking or liquefaction in the Study Area. The fault deformation overlay should therefore be considered on balance alongside other hazards such as shaking, liquefaction, slope hazard assessments, tsunamis etc (e.g. it may be counterproductive to move a building to avoid the fault deformation overlay if this means the building is more exposed to liquefaction or slope hazard).

8 Summary of geotechnical hazard assessment and key opportunities for further work

8.1 Overview

This report presents a geotechnical assessment of ground shaking, liquefaction, slope instability and fault surface rupture hazard for Port Vila CBD to support recovery efforts following the 2024 Port Vila earthquake. The assessment is underpinned by the geotechnical investigations, geophysical surveys, LiDAR analysis and 3D geological modelling completed for the Study, and by a seismic hazard assessment completed specifically for the Study Area.

This report presents a description of each earthquake induced geotechnical hazard, the methodology used for the area-wide assessment and key outcomes and recommendations for further work. Figures presenting areas susceptible to each hazard considered are provided in Appendix A, Appendix B, and Appendix C, with key conclusions and recommendations summarised in Sections 4 to 7.

It is important to note that the hazards presented in this report can be managed and mitigated through careful planning, design and construction to achieve resilient rebuilding of the Port Vila CBD.

The hazards are manageable through three measures applied together:

1 Investigating the ground before building, so that designs match the actual site conditions.

2 Pragmatic engineering, such as strong, well-tied foundations that can tolerate some ground movement and/ or ground improvement as required.

3 Risk-based planning, applying greater scrutiny to large, important or critical facilities. The figures within the appendices of this report show where each hazard needs to be considered and what level of assessment is appropriate.

The geotechnical assessment presented in this report has been used as the technical basis for the development of the Geotechnical Guidance Document, which sets out in practical terms what investigations, foundations and design measures are appropriate for different types of buildings in each ground zone. The zoning maps and data will also be made available digitally to support planning and consenting decisions.

This report is an area-wide study, it does not remove the need for site-specific investigation, but it provides a consistent framework to support a more resilient rebuild of Port Vila.

8.2 Summary of geotechnical hazards

Table 8.1 presents a summary of the geotechnical hazard assessment presented in Sections 4 to 7 for ground shaking, liquefaction, slope instability and fault surface rupture hazard. The discussion presented in Sections 4 to 7 provides a more thorough discussion of hazard mechanism, assessment methodology, outcomes of the assessment and assumptions and uncertainties. As such, it is important that this section is reviewed in detail to attain a full understanding of the work presented in this report, with the table below providing a broad level summary.

Table 8.1: Summary of assessed geotechnical hazards

Geohazard	Assessed areas that are susceptible	Description of the hazard
Ground shaking hazard	Full Study Area (refer Appendix A and Appendix D)	The ground shaking hazard has been estimated based on seismic hazard assessment specific to the Study Area (provided in Appendix D). Port Vila has a high ground shaking hazard in a global context, predominantly driven by large magnitude earthquakes along the Vanuatu Subduction Zone and within the subducting plate. There is a high probability (> 30% in a 50-year period) of earthquake events that result in very strong shaking across Port Vila CBD. Peak ground acceleration (PGA) and earthquake shaking for short and/or stiff structures have relatively small variability over the Study Area. However, shaking for tall and/or flexible structures is more influenced by underlying ground conditions. Areas underlain by softer soils (such as Zone A) are likely to further amplify earthquake shaking acting on tall and/or flexible structures. For these structures, site-specific geotechnical and geophysical testing should be undertaken to confirm the site subsoil conditions.
Liquefaction hazard	Zone A and B (refer Appendix A)	Reclamation Fill and Intra-reef deposits underlying Zone A and in localised areas in Zone B are assessed to be susceptible to liquefaction. Loose to medium dense soils within this zone are likely to liquefy at PGA of 0.2 to 0.6g, which is equivalent to 1 in 25 to 1 in 100 AEP levels of earthquake shaking. Areas near the coast within Zones A and B are also susceptible to lateral spread. Liquefaction and liquefaction-induced lateral spreading can result in ground settlement and lateral ground deformation. Site-specific liquefaction assessment should be undertaken for developments in Zones A and B, particularly for high-importance structures and structures near the coastline. Simplified assessment undertaken in this study using analytical methods indicate liquefaction assessment using SPT data alone are likely to be conservative. Where practical, shear wave velocity based assessment, calibrated against observed performance in the 2002 and 2024 earthquakes, is recommended. Vulnerability of structures to ground deformation will vary based on the form, weight and location of the structure. Nevertheless, high-level resilient design measures that can be adopted include, early geotechnical-structural collaboration in selecting suitable foundation system, well-tied and re-levellable foundation systems, and flexible service connections.

Geohazard	Assessed areas that are susceptible	Description of the hazard
Slope instability	Slope susceptibility zones (refer Appendix B)	Steep slopes (generally at angles greater than 45°) and man-made slopes (such as cutting into the hillside for development area) are likely to be susceptible to slope instability due to earthquake shaking, high groundwater pressure following significant rain events and/or due to natural erosion. Based on observed performance following the Port Vila earthquake, the dominant slope instability mechanisms within susceptible areas are assessed to be rock falls or topples from steep slopes. Figures provided in Appendix B present assessed slope runout and regression envelopes based on assessed slope instability for slopes greater than 4 m in height and 10 m in width. The slope runout zones vary between 10 m and 40 m over the Study Area, while the slope regression is generally limited to 5 m to 15 m in width at the crest of the slope. This assessment together with the GNS wet-condition runout extent will be used to support the development of the Geotechnical Guidance Document and to present an awareness overlay for critical infrastructure planning. Development within the susceptibility zones should undertake a site-specific slope assessment to confirm whether the hazard affects the site and, where required, pragmatic mitigation (such as setbacks, catch areas or local protection) can be used to reduce risk to tolerable levels. It is also recommended that a coordinated landslide register be established (PVCC, PWD and the Department of Geology and Mines) to record future events and progressively refine the slope hazard zoning.
Fault surface rupture hazard	Fault deformation overlay (refer Appendix C)	The ground deformation observed in the central CBD after the 17 December 2024 earthquake is interpreted as triggered slip on pre-existing faults (the Tank Hill and Palace Faults), possibly combined with liquefaction-induced settlement in the lower CBD. It was not primary rupture of the earthquake source fault. The east-west trending faults mapped across the Study Area should be treated as possibly active. Indicative assessment suggests single-event displacements of the order of 180 to 340 mm, comparable to the deformation observed in December 2024. This is of a similar order to the ground deformation which is typically addressed in liquefaction-resistant design, suggesting that designing foundations to accommodate liquefaction-induced ground movements would also help manage fault-related displacement. A fault deformation overlay is recommended around the mapped fault traces (the greater of 20 m either side of the trace, or extension to the crest of a visible escarpment). Development within the overlay should not be automatically precluded; the overlay instead should trigger site-specific assessment and design for ground deformation (e.g. well-tied, robust foundations). Critical infrastructure and post-disaster facilities should be located outside the overlay where practicable.

8.3 Opportunities

The recovery and rebuild efforts present several valuable opportunities to improve the understanding of geotechnical hazards and support the planning, design and construction of a resilient CBD. It is recommended that this document and the Geotechnical Guidance Document are treated as a living document that is updated as more information becomes available.

Opportunities that are worth consideration include:

- A public geotechnical and geophysical investigation database. A shared database can be useful for high-level strategic planning, to support recovery following significant earthquake events and to lower costs at project level.
- Network of ground motion stations to accurately measure ground motions across Port Vila and inform future refinement of seismic hazard for the region.
- Regular LiDAR surveys to provide an accurate baseline to inform recovery efforts following significant earthquake events or other natural disasters.
- Development of a reconnaissance framework and a database to systematically record ground deformation and structural damage following significant natural disasters.
- Assessment of the impacts on linear infrastructure and measures to improve resilience to future disaster events.
- Sharing of global lessons and best practices and training for key stakeholders.
- Creating a landslide database and recording process led by an agreed lead agency (working with PVCC, PWD and the Department of Geology and Mines) and using a standard landslide data process, with clear ownership to avoid duplication of effort and overlapping responsibilities. Maintaining such a landslide inventory would progressively improve the understanding and management of slope hazard across Port Vila by:
  - Recording the location, type, size, trigger and runout distance of future landslide events;
  - Building a local evidence base on landslide frequency, magnitude and distribution;
  - Enabling future refinement of the landslide zoning;
  - Supporting more consistent planning decisions about where landslide risk should be avoided, mitigated or managed; and
  - Informing emergency response, infrastructure planning and site-specific geotechnical assessments.

  This type of long-term landslide recording has been undertaken in New Zealand in recent years and has allowed landslide hazard models and land-use planning responses to be progressively refined as additional event data becomes available.
- Targeted further work to confirm fault activity and characterise displacement history, which could include:
  - Targeted field reconnaissance of mapped fault traces and marine terrace surfaces;
  - Dating of offset reef, terrace or other suitable materials, including U/Th dating of coral or other calcareous material where practicable;
  - Logging of existing or future exposures, and trenching where feasible, to assess fault activity and displacement history; and
  - Refinement of the fault deformation overlay as new mapping, dating and site-specific investigation data become available.

9 Applicability

This report has been prepared for the exclusive use of our client Egis NZ Limited, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

The geological model presented in this report is based on information obtained from project-specific and historic investigations undertaken at point locations. The nature and continuity of the stratigraphy away from borehole locations may vary from those assumed in this report.

Earthquakes are unique and impose different levels of shaking in different directions on different sites. The results of the geohazard assessments presented within this report are based on regional seismic demand and published analysis methods. It is important to understand actual performance may vary from that calculated.

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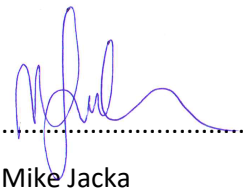

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