POWERFUEL

Portland energy recovery facility

Environmental statement
Addendum
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7.1 Preliminary slope stability assessment

Powerfuel Portland Ltd

Portland Energy Recovery Facility

Preliminary Slope Stability Assessment

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ARUP

Contents

1 Introduction

1.1 Purpose and scope

This preliminary slope stability assessment has been prepared by Ove Arup and Partners Ltd (Arup) on behalf of Powerfuel Portland Ltd (Powerfuel) to assess the stability of the slope adjacent to the proposed Energy Recovery Facility (ERF) development on a site located within Portland Port on the Isle of Portland, Dorset.

This report has been produced to support the planning application for the proposed ERF development prior to detailed ground investigation being carried out and prior to detailed engineering design of the layout and structures within the development.

The intention of this report is to consider the site context, site history and existing information on the ground conditions and:

- assess the current risks at the site associated with slope stability;
- assess the likely impact of the proposed development on slope stability; and
- assess the suitability of the site in principle for the proposed ERF development

This report assumes that a more detailed assessment will be carried out once the results from site-specific ground investigation are available and the design of the proposed development has been further developed.

Sources of information reviewed as part of this assessment are listed in full in the References section and include historical ground investigation data and reports related to the site and surrounding area and other publicly available information. A site walkover was not considered necessary for this preliminary report and has not been undertaken as part of this assessment.

1.2 Proposed development

The proposed development will comprise an ERF that has been designed nominally to treat 183,000 tonnes of refuse-derived fuel (RDF) per year and the capacity to export 15.2 MWe of electricity to the grid. It will be a mass burn facility, using boiler and moving grate technology with a high efficiency steam boiler and high efficiency turbine.

The RDF will be stored in a bunker, envisaged to be approximately 40m long and 20m wide. The depth of the bunker is likely to be around 5m, and for the purposes of this report is it assumed that this may require a temporary excavation up to around 8m deep. The proposed building will enclose the RDF bale storage area in the fuel hall and waste bunker, tipping hall, cranes, conveyors, feed hopper, furnace boiler, condenser units and turbine/generator. The building height is likely to vary from approximately 19m in the area containing the tipping hall and bunker to 47m in the area containing the furnace and boiler.

In addition to the large excavation for the bunker, the development will require various other smaller excavations, for example utilities trenches, surface water storage tanks and foundations. These other excavations are considered negligible in relation to global slope stability and are not discussed further in this report.

2 Site context

2.1 Site description

The main development site is located in the northeast of the Isle of Portland, a peninsula island on the Dorset coast, within Portland Port approximately 600m east of the villages of Fortuneswell and Castletown as shown in Figure 1.

The Isle of Portland is formed by an escarpment which rises to a height of approximately 140m above Ordnance Datum (mOD), forming cliffs along the eastern side of the island.

The development site is roughly triangular in shape and comprises reclaimed land which has been developed for various uses since the 1800s. The site is currently vacant (all previous buildings have been demolished) and the groundcover is predominantly hardstanding.

Figure 1: Site location

The main development site is bounded to the east by overland fuel pipelines which supply marine fuel from Portland Bunkers fuel storage area in the nearby cliffs. Beyond the pipelines is the shingle shoreline of Balaclava Bay, which extends south from the Portland Harbour breakwaters. To the southwest is Incline

Road, a private road actively used by port traffic, and a former railway which runs along the toe of the slope. Beyond the former railway is a steeply rising hillside supporting grassland, scrub and woodland habitat. Existing operational port development lies to the north and northwest of the site.

2.2 Topography

The development site is relatively flat with an elevation of approximately 7mOD. The site is bounded to the southwest by a hillside which rises inland to a height of approximately 140mOD.

The hillside comprises an upper steep escarpment of limestone/sandstone over a shallower slope formed of landslip deposits over the underlying bedrock with a slope angle of around 8°. Towards the base of the hillside the slope steepens to a gradient of approximately 30°.

The former railway which runs towards the toe of the slope was constructed within a slight cut into the hillside around 10m above the elevation of the development site.

2.3 Site history

A detailed description of the site history can be found in the Geoenvironmental and Geotechnical Desk Study report [1] that formed technical appendix I1 to the Environmental Statement. Reference has also been made to www.portlandhistory.co.uk . The elements considered to be of most relevance to slope stability are described below.

The 1805 OS map (Figure 2) indicates the site prior to development of the harbour and shows the coastline to be approximately in the position of the current steep slope above the site.

The Admiralty Incline Railway was built in 1848-49 to transport stone from the quarries at the top of the Isle to Portland Nore, for use in construction of the breakwater arms of Portland Harbour. Construction of the breakwater arms continued until around 1872 and included placing fill in front of the steep slope to create the development site (Figure 3).

By 1903 a railway had been built on the hillside above the site (Figure 4). This appears to be on a combination of filled embankment, cuttings into the slope and free-standing viaduct.

The development site was used for a variety of uses by the Royal Navy after construction of the breakwaters, including buildings relating to the Admiralty Underwater Weapons Establishment. These were substantial industrial buildings that included basement structures. These buildings have been demolished over the last 20 years or so. Former basements have been backfilled with demolition arisings. In 2016/17, the main road was realigned along the base of the cliff along the western site boundary creating the current vacant site. The last of the stockpiled demolition rubble was cleared from the site in 2018.

Figure 2: 1805 OS Map (British Library, supplied under the Open Government License v1.0)

Figure 3: 1864 OS map (extract from Groundsure report [10] – copyright OS)

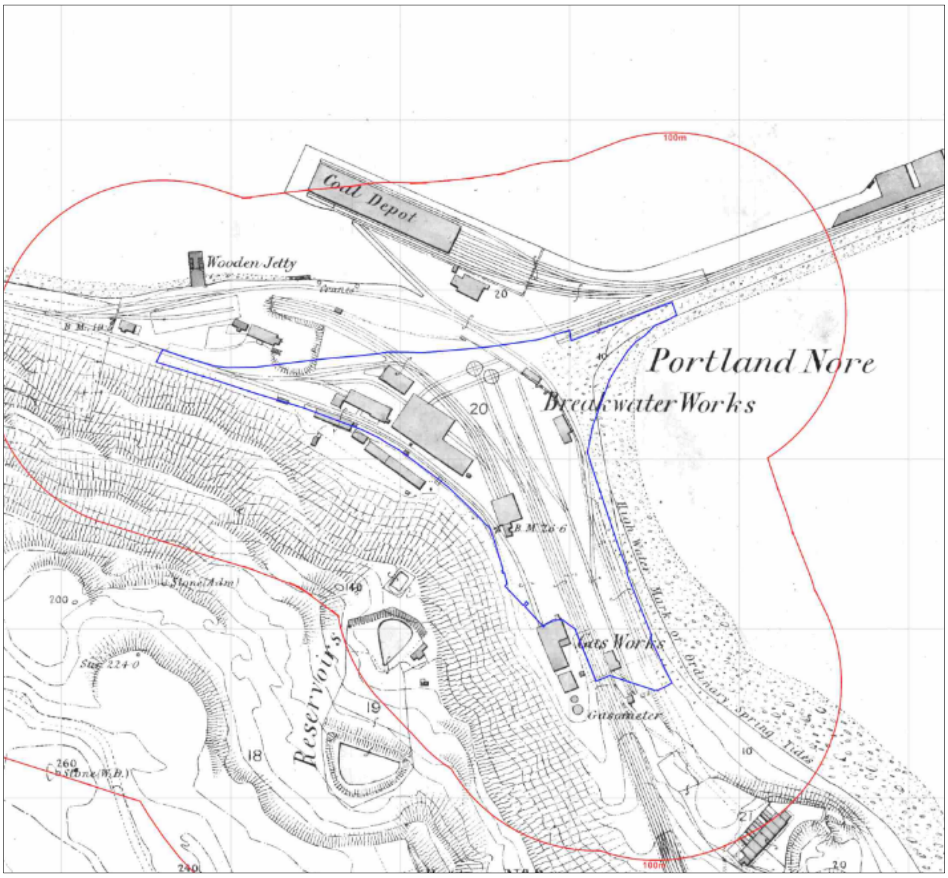
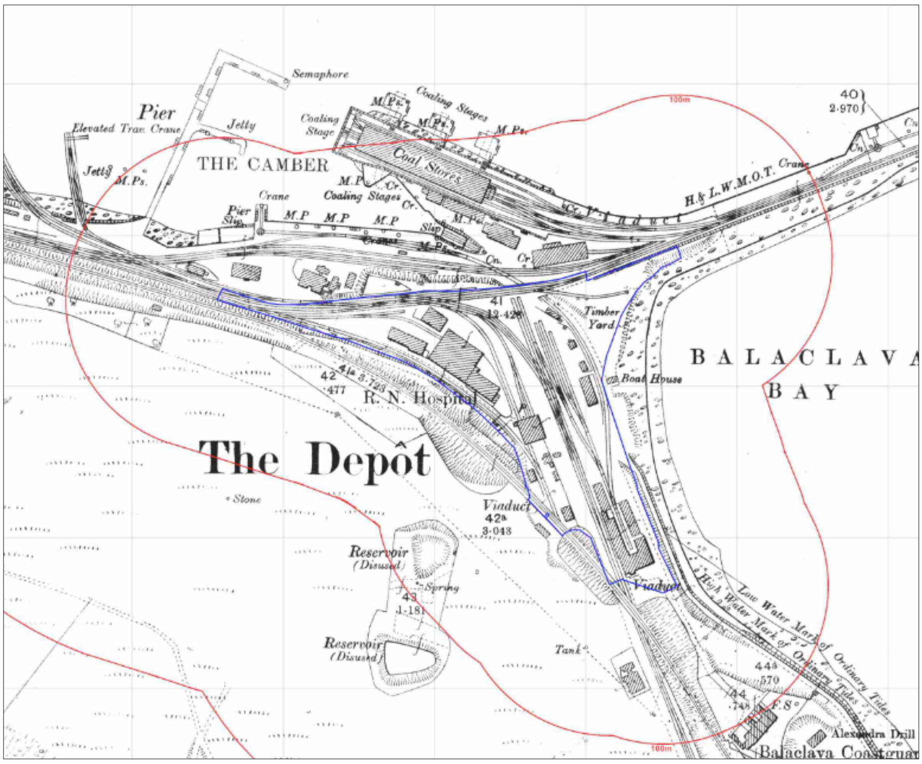


Figure 4: 1903 OS map (extract from Groundsure report [10] – copyright Ordnance Survey)



3 Geology

3.1 Regional geology

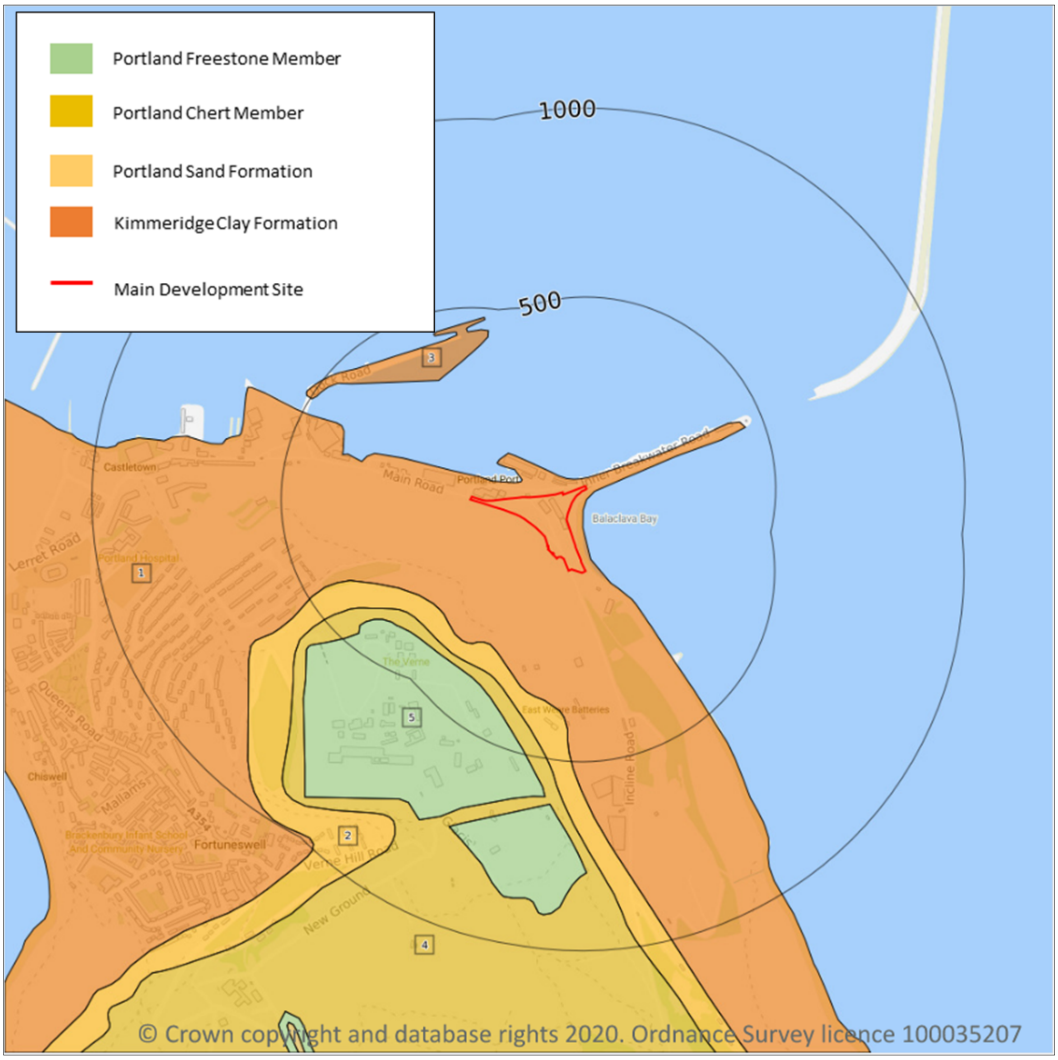
The Isle of Portland is formed of Jurassic sedimentary strata. The geological sequence within the cliffs above the site is shown in Figure 5 and comprises:

- Portland Stone Formation (Portland Freestone Member and Portland Chert Member) comprising limestone with oolitic bands and frequent nodules of chert;
- Portland Sand Formation comprising a silty to very silty mudstone with argillaceous bands of limestone and a variably sandy siltstone with some thin bands of nodular argillaceous limestone and layers of silty mudstone;
- Kimmeridge Clay Formation a calcareous mudstone with thin siltstone beds.

The Portland Stone Formation form the cliffs along the escarpment to the southwest of the site and overlies the Portland Sand which sub-crops at the base of the steep cliffs. The Kimmeridge Clay forms the lower slopes and extends towards the coast.

Landslip or colluvial deposits overlie the Kimmeridge Clay on the slopes and comprise variable deposits of gravel, cobbles and boulders of limestone, chert or siltstone with a varying sand and silty clay matrix. These deposits were formed as a result of the natural weathering and degradation of the cliffs.

Figure 5: Bedrock geology from Groundsure [10]



3.2 Geology at the site

A ground investigation was completed within the development site in 2009 by RPS [9]. The ground conditions within the site were recorded to comprise:

- Made ground approximately 5 to 8m thick comprising a mixture of firm, locally firm to stiff clay, gravelly clays, silty sands and gravels. Limited anthropogenic materials were recorded indicating that the material is largely reworked natural materials used to form the original port development in the 1800s;
- Superficial deposits were recorded in one location in the northeast corner of the site. These comprised grey and brown sands and gravels at a depth of 5m bgl to approximately 12m bgl and were considered likely to be Marine Gravel deposits;
- A weathered zone of Kimmeridge Clay was identified in two boreholes in the north of the site as a thin layer of firm to stiff grey clay containing limestone gravels resting above the Kimmeridge Clay bedrock. The top of the

Kimmeridge Clay was identified as depths from 5m to 12m bgl and was proven to a maximum depth of 21m bgl. The strata encountered largely comprised mudstones with occasional bands of stiff clay.

No site-specific ground investigation has been undertaken on the hillside above the development site, however historical British Geological Survey (BGS) borehole logs on the slope indicate the thickness of the landslip deposits to be up to at least 5m thick, with some boreholes recording thicknesses of approximately 13m.

The BGS logs indicate the landslip deposits comprise soft to firm clay with boulders, cobbles and gravels of chert and limestone.

Evidence from neighbouring sites indicates the presence of a disturbed zone at the top of the Kimmeridge Clay that contains slickensided shear surfaces [9]. This is associated with historical landslips.

3.3 Groundwater

Limited groundwater level monitoring was undertaken on the main development site by RPS. Groundwater was encountered between depths of 7.18 m and 7.88m bgl within the Kimmeridge Clay and at a depth of approximately 7.7m bgl in the superficial deposits in the northeast of the site.

One historical BGS borehole on the hillside recorded groundwater at a depth of 10.4m bgl towards the base of the landslip deposits

Historical maps presented in the Arup 2020 desk study [1], which formed Appendix I1 to the ES, indicate the presence of some springs on the slope above the site.

4 Historical slope stability

4.1 Slope failure mechanisms

Very slow natural movements occur within the colluvium along the slopes. The movement is understood to be aided by coastal erosion which removes some of the weight from the toe of the slope [3].

The stability of the natural slopes is considered to be controlled by the slope angle of the upper surface of the Kimmeridge Clay and by the presence of water within the slope [4].

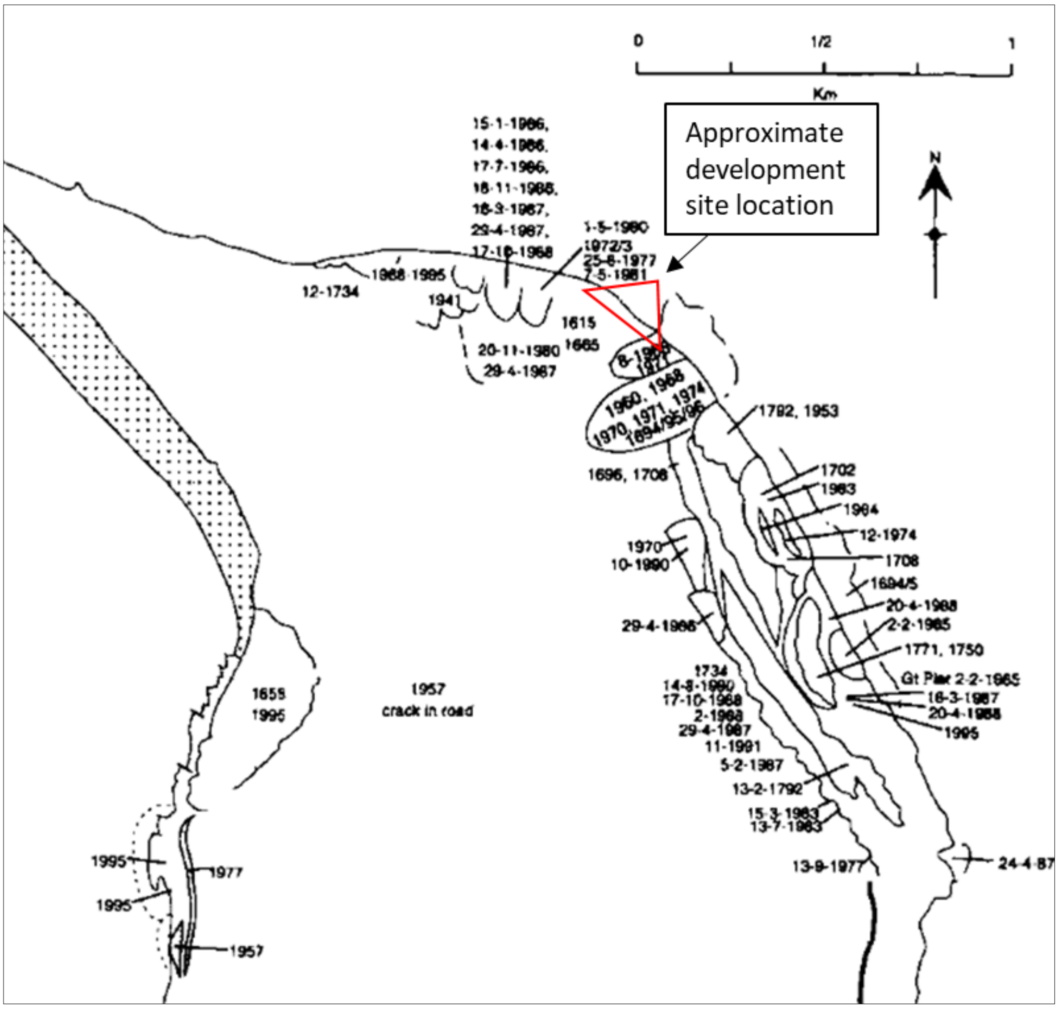
There are considered to be three possible modes of slope failure [4]:

- Deep-seated slumps which occur within the colluvium and fill material;
- Along soft clays on the interface between the Kimmeridge Clay and overlying colluvium;
- Re-activation of very deep-seated rotational failures through the Kimmeridge mudstone, at depths below the colluvium/Kimmeridge Clay interface.

4.2 Historical slope failures

The Isle of Portland has a history of landslips with records of slips along the coastline existing from the 1665 to the present day [3]. The distribution of recorded historical landslip locations is shown in Figure 6.

Figure 6: Location of known landslips, adapted from [3]



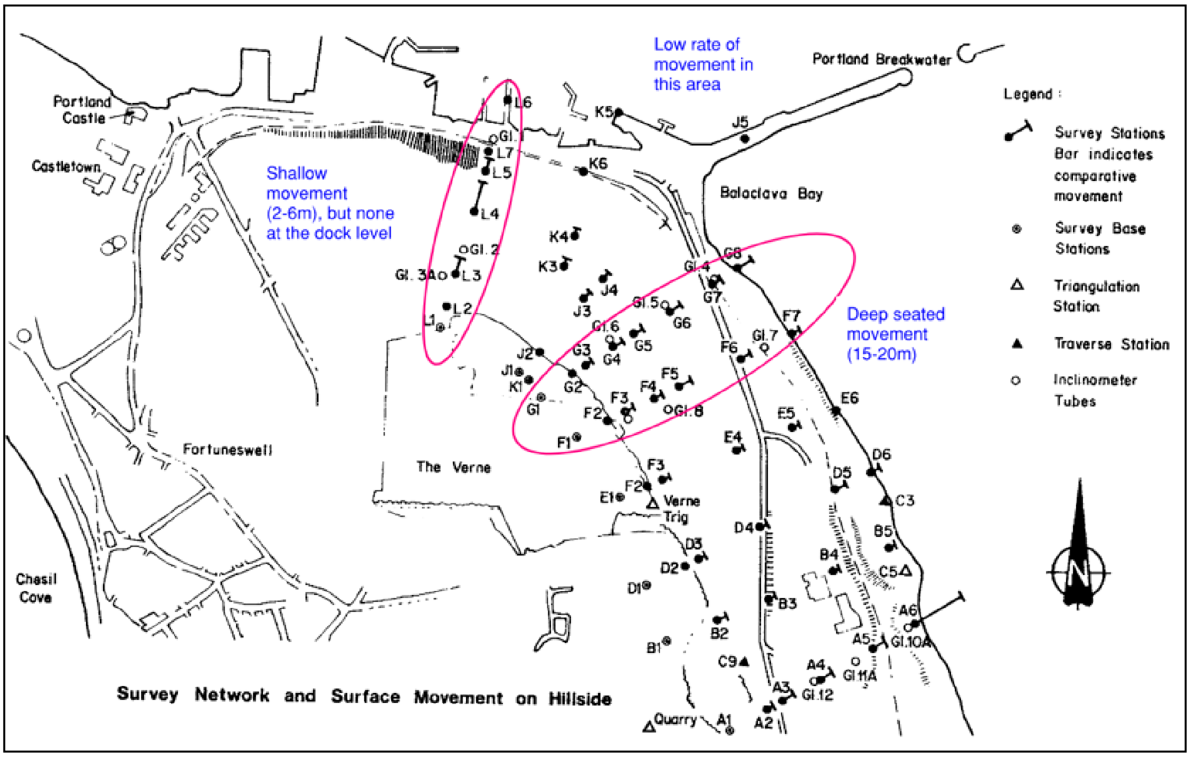
On the slopes to the west of the site there are records of four landslips that have a well-defined deep seated shallow circular form [3]. Individual shallow movements have been recorded at rates between 3.5-9mm per year in the last 50 years. The development of the harbour resulted in extensive cut and fill at the base of the slope and the dredging of the harbour entrances also removed weight from the toe of the slope.

The only slips indicated to be immediately above the development site are recorded to have occurred in the 1600s, however, it appears that the exact location of these slips are not known with any precision and were just in this general area of the coast. In this part of the Isle of Portland undercutting of the toe of the slope by sea erosion is considered to be a predominant control on the slope movements. However, due to the development site at the base of the slope being formed of reclaimed land, the site will be protecting the slope from future coastal erosion.

The coastal slopes to the south of the site, adjacent to the site known as Upper Osprey, forms the most active landslip area on the Isle of Portland. The largest landslide is reported to have occurred in 1792 following a period of high rainfall and comprised a massive, deep seated slip [4]. Several more recent failures have occurred within this area, and they are predominantly considered to be as a result of poorly executed earthworks and a failure to control water flows properly rather than natural instability of the slope [4].

Surface movement monitoring was undertaken along the north-east cost of Portland between 1977 and 1988 [5] as shown in Figure 7. The results indicated that shallow movements were occurring on the slopes to the west of the development site and deep-seated movement was occurring on the slopes to the south. While the survey points on the slope above the development site were of a limited number, they indicate a low rate of movement in this area.

Figure 7: Surface movement monitoring 1977 to 1988, adapted from [5]



The records of historical slope movements along the northeast coast of the Isle of Portland, suggests that the slopes above the development site are in a different setting to the areas to the south where the main landslides on Portland have occurred.

4.3 Recent evidence of slope movements

No site-specific settlement monitoring data is available for the slope above the development site. However, LiDAR data is published by the Environment Agency and updated annually. This provides elevation data that can be compared over time to identify indications of relative displacement. The elevation accuracy is only stated as +/-15cm and the quality of the data is affected significantly by vegetation, but nevertheless the LiDAR data can provide some information on relative movements.

Due to the vegetation on the slope there is considerable 'noise' in the available LiDAR data. A comparison of available data over 12 years from 1998-2010 did not identify any consistent differential movement in any areas of the slope above the development site.

Anecdotal evidence from the Port also indicates that there has been no record of any recent slope movements adjacent to the site, with recent slope movements only recorded on the slopes to the north and to the south at Upper Osprey. The former rail embankment that runs along the side of the site at the toe of the slope has been in place for over 100 years and does not appear to have been affected by large-scale slope movements directly above the site.

5 Ground model

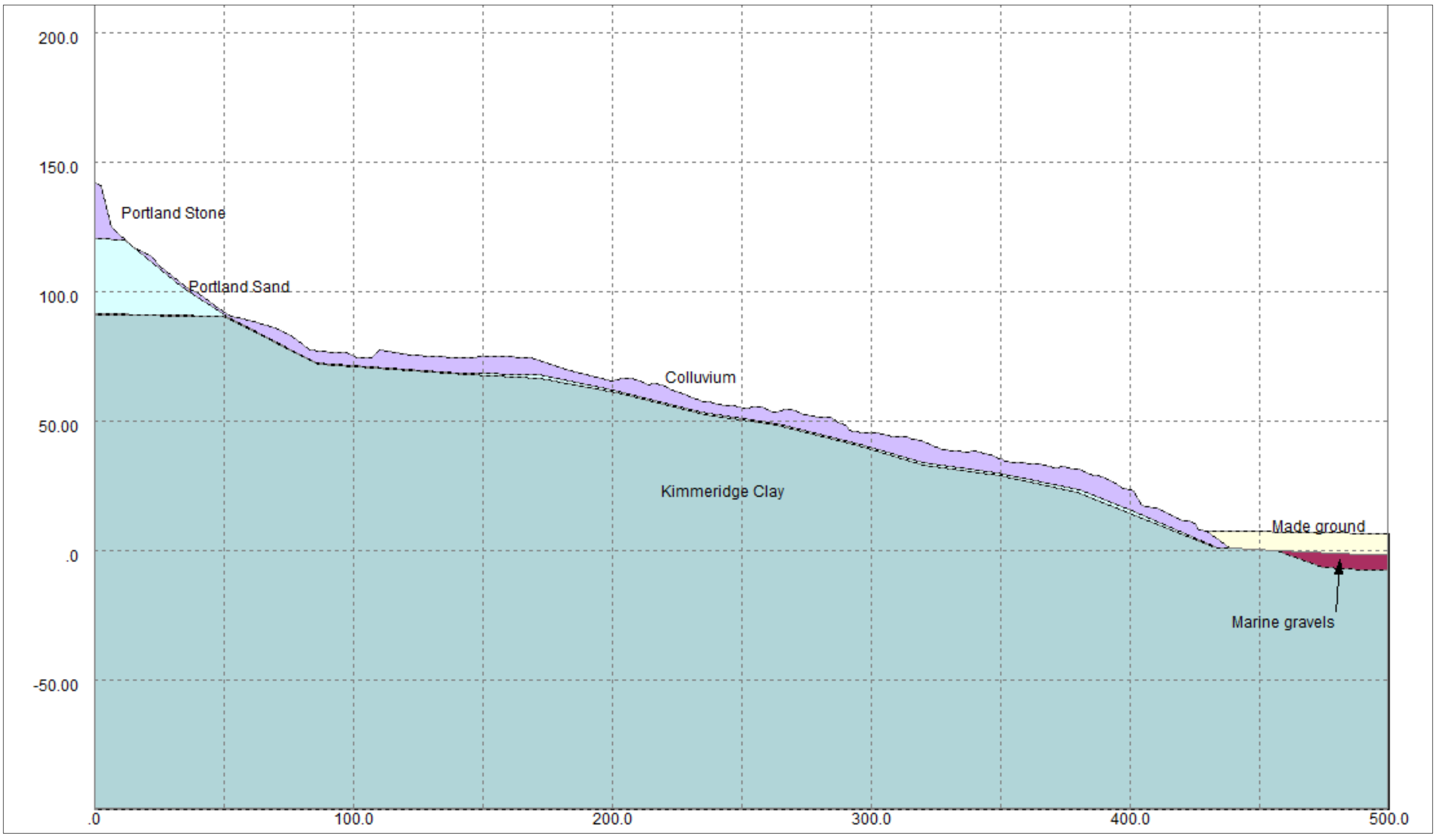
5.1 Stratigraphy

The section shown in Figure 8 indicates the assumed stratigraphy at the site. Note that this is based on limited existing ground investigation data and is heavily influenced by published papers [3], [5], [7] and ground investigation reports on nearby sites [4], [9].

The following main assumptions are made:

- Prior to the port development the coastline was at the toe of the existing slope
- Historically, erosion at the toe of the slope initiated slope instability and allowed a disturbed zone to develop at the top of the Kimmeridge Clay
- The colluvium layer is of limited thickness on the slope (5m or so)
- Colluvium would have been eroded from the toe of the slope and therefore no colluvium or disturbed zone is present beneath the made ground used to raise the ground for the port development. This is supported by borehole evidence from the site.
- The placement of fill for the port development will have provided weight at the toe of the slope and had a buttressing effect against further slope movement. The fill would also act to protect the toe of the slope from further coastal erosion.

Figure 8: Assumed site stratigraphy



5.2 Geotechnical parameters

5.2.1 Made ground

The made ground at the toe of the slope is variable in nature, likely derived from reworked soils that were excavated during the early development of the port/harbour. For the purposes of the following assessment, no attempt has been made to subdivide the made ground into fine and coarse parts. All of the made ground is given strength parameters typical of a well graded general fill.

5.2.2 Marine gravels

The sand and gravel encountered in a single borehole on the site (RT-2) has been interpreted as marine gravel. It is assumed to be a medium dense well-graded coarse soil.

5.2.3 Colluvium

The colluvium encountered on neighbouring sites is highly variable in nature, but in general can be considered a clay with varying amounts of gravels and boulders. For the purpose of strength parameters, the colluvium is assumed to be representative of a gravelly clay of low to intermediate plasticity. Laboratory testing data from neighbouring sites indicates relatively high peak and residual shear strengths. This is likely to be related to the presence of lithorelicts (fragments of rock) within the clay.

5.2.4 Disturbed Kimmeridge Clay

The upper surface of the Kimmeridge Clay is generally highly disturbed and in places softened due to past slope movements and the presence of groundwater. Polished ('slickensided') shear surfaces are commonly encountered on neighbouring sites. The presence of these shear surfaces means that it is appropriate to consider residual strength parameters. Data from neighbouring sites indicates these residual parameters to be very low.

5.2.5 Kimmeridge Clay

The Kimmeridge Clay is generally encountered as a weak rock (mudstone), although the upper part appears to be weathered to a clay directly beneath the development site.

5.2.6 Groundwater

The ground at the toe of the slope is assumed to be saturated at least to mean sea level. On the slope, on neighbouring sites, groundwater is typically recorded just below the top surface of the Kimmeridge Clay. Perched groundwater is also found at various levels within the colluvium.

A combination of piezometric levels and 'r_u' values have been used in the slope stability assessment.

5.2.7 Parameters used in preliminary assessment

The parameters provided in Table 1 have been assumed in the preliminary assessment presented in the following section. The parameters are based on data presented for neighbouring sites and on the professional judgement of the author. They are intended to be used for the preliminary assessment only to determine likely failure modes and potential impact of changes.

Table 1: Ground conditions parameters used for preliminary assessment

Stratum	Unit weight	Effective angle of shearing resistance	Effective cohesion	Groundwater
Made ground	19	33	2	$R_u = 0.3$
Marine Gravels	18	35	0	Piezometric
Colluvium	20	34	0	$R_u = 0.0$
Disturbed Kimmeridge Clay	20	15	10	$R_u = 0.1$
Kimmeridge Clay	22	35	20	Piezometric

6 Slope assessment

6.1 Proposed earthworks

The proposed ground level for the development will be similar to the existing site at approximately 7mAOD.

The RDF waste bunker will extend to a depth of approximately 5mbgl (2mOD) beneath the centre of the building. The depth of the excavation required to construct the bunker may extend up to 8mbgl which is anticipated to be towards the base of the made ground and geological boundary with the top of the Kimmeridge Clay. Based on the available ground conditions information it is anticipated that the base of the excavation will be below groundwater level.

6.2 Slope analysis

6.2.1 General approach

Assessment of slope stability has used Oasys SLOPE software to explore potential failure surfaces and changes to factor of safety that may occur in the future. Shallow superficial failures have been ignored (by specifying a minimum weight of slip). To simplify the assessment, only circular slip surfaces have been explored, although it is recognised that actual slips are likely to be non-circular, elongated along the disturbed zone at the top of the Kimmeridge Clay. The analysis method used is the Bishop method with variably inclined interslice forces. All analyses use unfactored soil parameters and give a global factor of safety on shear strength.

A ’global factor of safety’ is the ratio of the restoring forces to the disturbing forces. A slope with a factor of safety of less than 1.0 would be unstable. Many natural slopes will have a factor of safety in the range 1.0-1.2 and can be considered marginally stable, i.e. they may become unstable under certain conditions such as sustained wet weather. A factor of safety in the range 1.3-1.5 is commonly sought for newly engineered slopes.

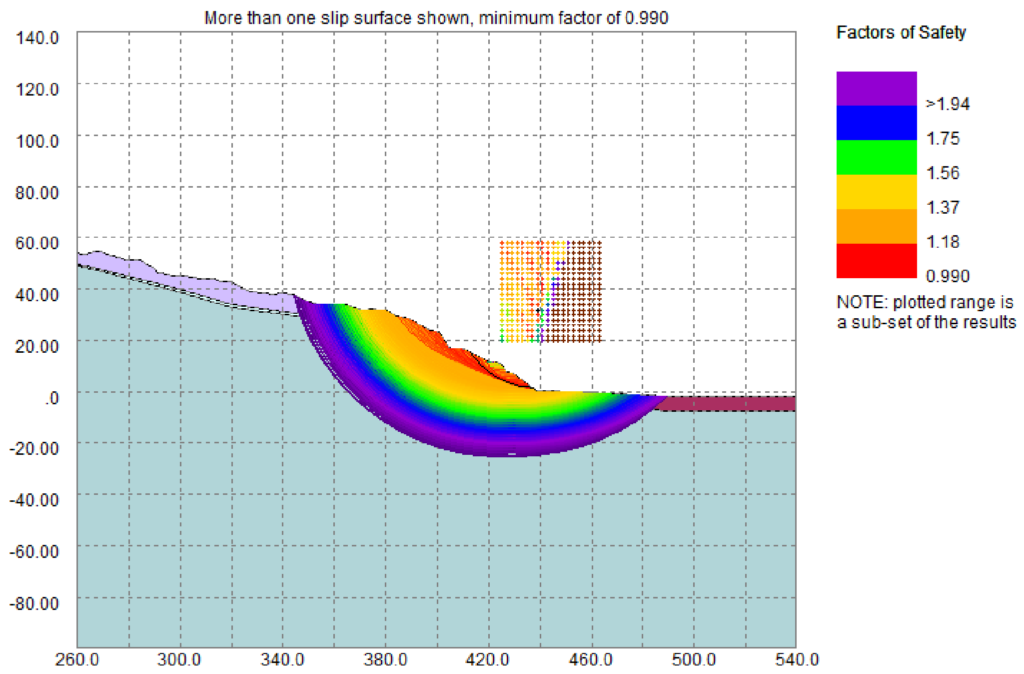
Without detailed information on the stratigraphy, geotechnical properties of each soil layer and groundwater conditions the absolute factor of safety may not be calculated with certainty. However, for existing slopes, it may be appropriate to consider the relative change in factor of safety. For example, if a slope is thought to be marginally stable, but with no evidence of recent instability, implementing drainage to increase the factor of safety by, say, 0.1 may be considered acceptable, rather than targeting a particular absolute factor of safety.

The following series of assessments was intended to consider the likely relative changes in factor safety of the slope over time.

6.2.2 Original situation

The first analysis (presented in Figure 9) was carried out to check that the chosen parameters give the likely failure mode with a factor of safety close to unity (1.0). The assumed original topography is without the existing made ground. Slope failures are likely to have frequently occurred along this part of the coastline. The minimum factor of safety is found for slip circles tangential to the disturbed zone, which is as expected.

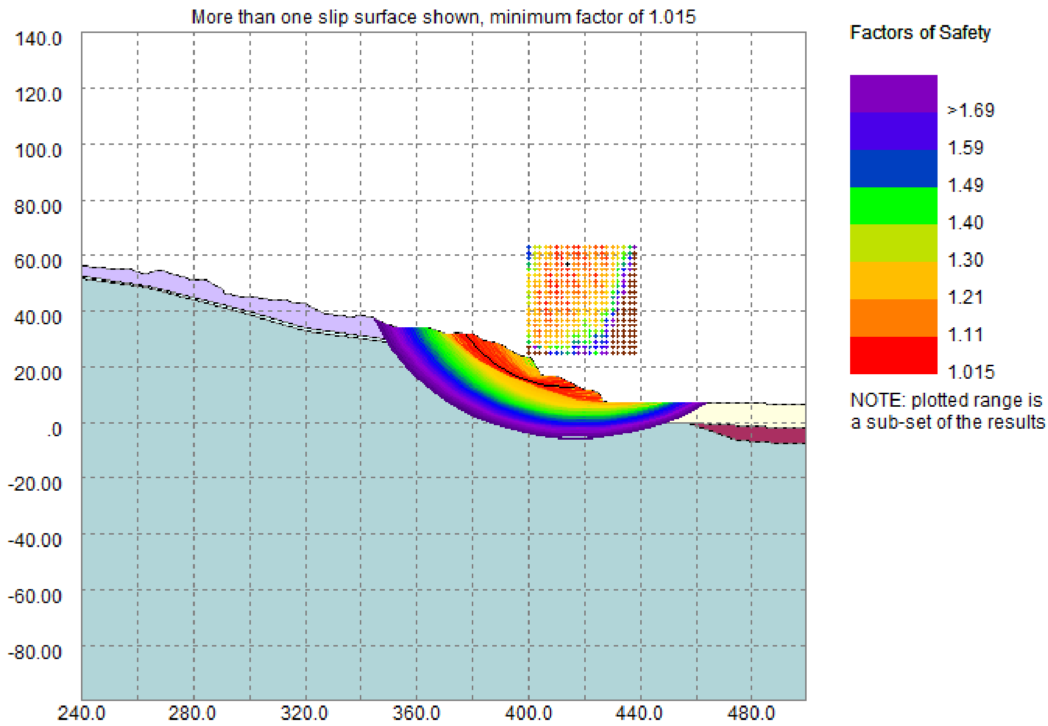
Figure 9: Slope analysis for original situation



6.2.3 Current situation

After the previous analysis had provided reassurance that the chosen strength parameters are reasonable, an analysis was carried out with the existing made ground in place (presented in Figure 10). The assessment indicates that the stability of the slope above the site is likely to be marginal, again primarily due to the presence of the disturbed zone. The buttressing effect of the made ground at the toe of the slope is seen to significantly increase the factor of the safety of deeper slips that may affect the development site itself.

Figure 10: Slope analysis for current situation



6.2.4 Proposed excavation

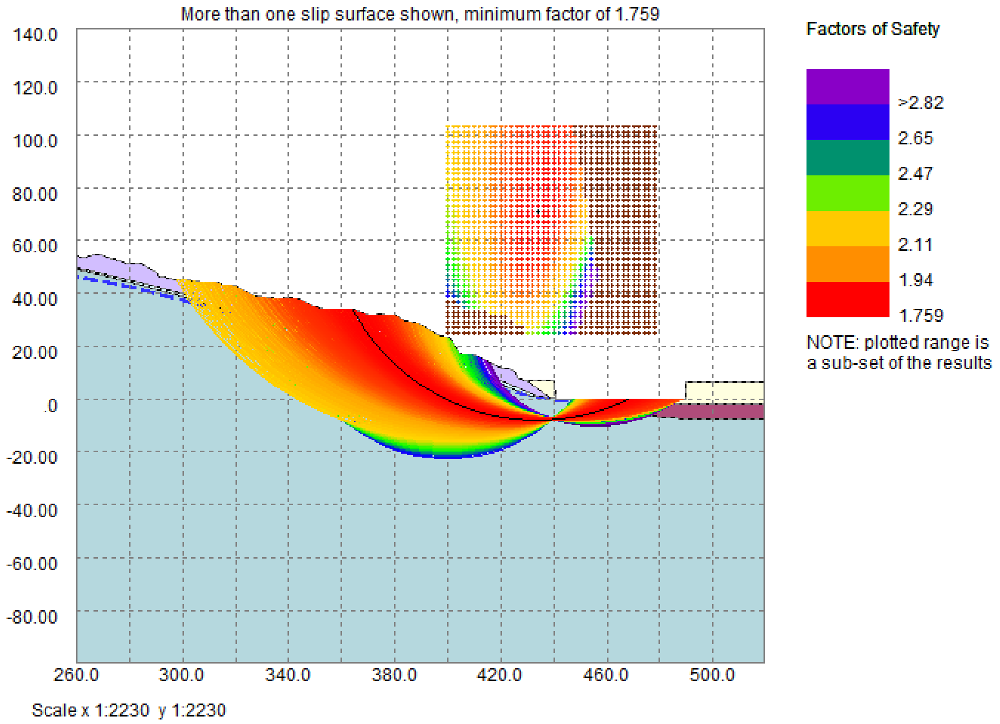
The next analysis considered the effect of the excavation at the toe of the slope.

Several assumptions have been made:

- The excavation would not be carried out using battered slopes, but with a robust embedded retaining wall that would form part of the permanent structure.
- The retaining wall would extend to at least around -8mOD and would prevent any slip circles above this level.
- A reduction in slope stability would therefore be primarily due to the loss of weight at the toe of the slope.

The results of the analysis are shown in Figure 11.

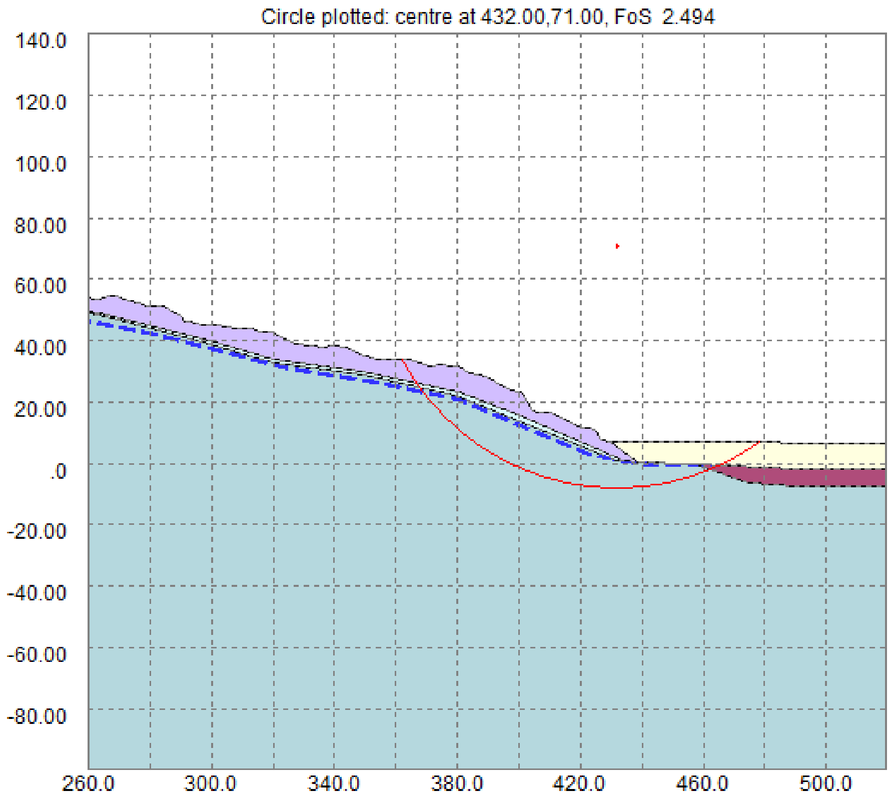
Figure 11: Slope analysis for proposed excavation



It can be seen that the slip circles must go much deeper within the undisturbed Kimmeridge Clay (to pass beneath the embedded retaining wall) and the factor of safety is significantly higher than expected for shallow slips on the slope above the site.

Using the minimum slip circle geometry in the above analysis, a check was made against the current situation, as shown in Figure 12.

Figure 12: Current situation showing minimum slip circle geometry

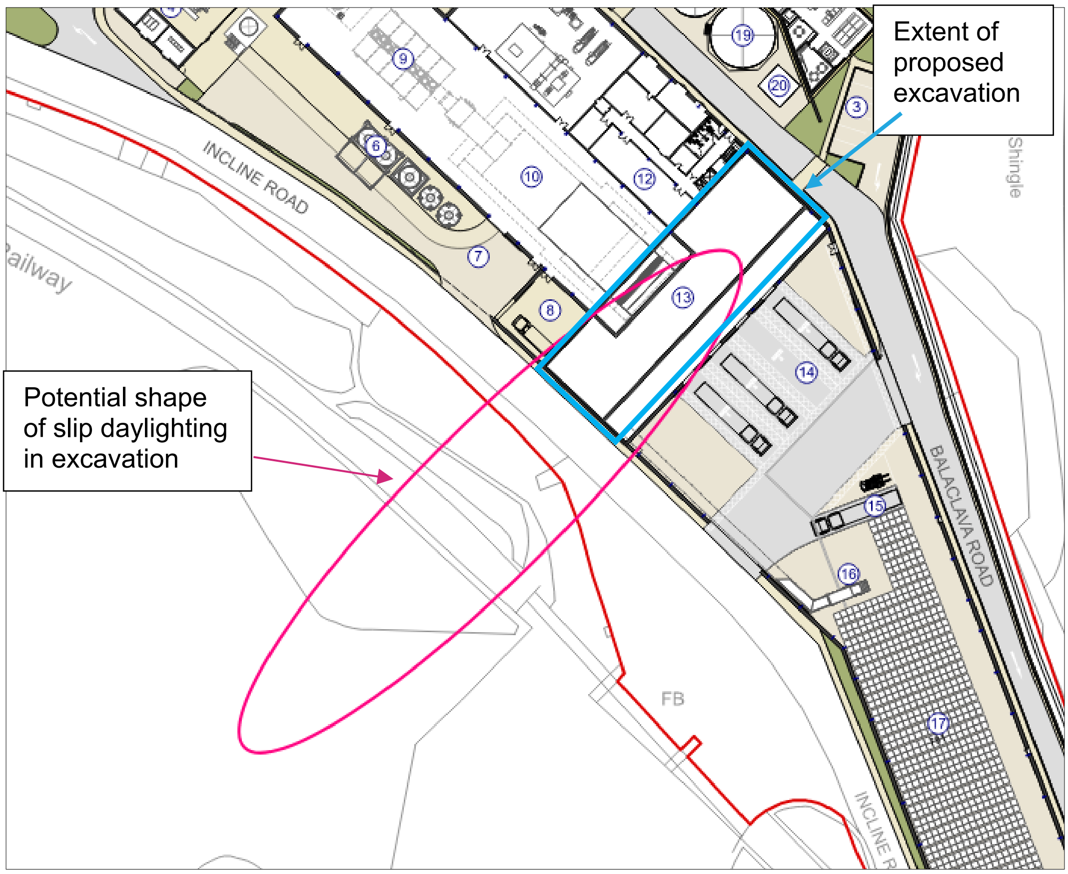


This suggests that the removal of weight at the toe of the slope would have a significant effect on factor of safety for this slip circle (from FOS 2.5 to FOS 1.8).

However, the following should be noted:

- The width of the excavation perpendicular to the slope (20m) is very narrow in relation to the potential slip circle indicated.
- The sketch shown in Figure 13 indicates the potential shape of slip that would be needed for this to occur.
- The deeper-seated slips that have been recorded historically to the west and south of this area are at least 80 to 100m wide and this suggests that a very narrow, elongated slip is highly unlikely.
- The actual FOS would be much higher than the FOS 1.8 from the analysis, due to the considerable 3-D effects of such an elongated slip.
- Notwithstanding the above, a FOS of 1.8 would not normally be of concern (refer to discussion of absolute FOS in Section 6.2.1).

Figure 13: Potential shape of slip needed for significant effect on factor of safety



7 Conclusions

The following conclusions are made based on the desk study assessment and stability analyses presented above:

- The north and eastern parts of the Isle of Portland have been significantly affected by slope instability in the past, and there is evidence of ongoing movement.
- The development site is at the northeastern corner of the Isle and historical records and the geomorphology of the slope indicates that it has been affected by past slope movements above the western and southern parts of the development site. The slope above the central part of the site has less evidence of historical instability, but there is historical monitoring data that indicates a low rate of creep at shallow depth.
- Prior to filling to existing ground levels for the original port development, the toe of the slope would have been exposed to coastal erosion processes that would have acted to destabilise the slope. The fill currently on the site provides toe weight and a buttressing effect to the slope and will also provide protection from future erosion.
- The development site, and particularly the central part of the site where excavation is proposed, is therefore situated in a position that has a lower risk of instability than neighbouring areas such as at Upper Osprey to the south.
- The development site in its current condition is very unlikely to be affected by deep-seated instability.
- The slope above the development site is known to be affected by progressive creep movements that affect the shallow surface soils. The rate of movement is potentially a few millimetres per year.
- It is likely that the rate of movement will accelerate during periods of wet weather and at some time in the future this may lead to sudden shallow slope movement, likely along pre-existing shear surfaces within the upper 5m of the slope.
- Sudden shallow slope movement could result in debris at the toe of the slope that could affect or partially block the highway. However, it is noted that the Port do not have records of such slips occurring in the past.
- The proposed excavation is orientated parallel to the slope and hence is narrow in relation to potential slope instability.
- Embedded retaining walls will be used to carry out the excavation, and these will prevent shallower slips from occurring.
- The excavation will result in a significant removal of weight at the toe of the slope. This will reduce the factor of safety of potential deep-seated failures passing beneath the embedded retaining walls. However, the removal of weight will only go back to the original state of stress before the site was filled for the port development. After construction of the structures and buildings

over the pit area, the total weight at the toe of the slope is likely to be similar to or more than existing.

- The geometry of such a slip would be very elongate and hence there would be considerable 3-D effects compared to the infinite slope assumed in the analyses. Even discounting these effects, the factor of safety is likely to be acceptable.
- Deep-seated slips would need to pass through the undisturbed Kimmeridge Clay and are considered highly unlikely unless there are relict shear surfaces from ancient landslide events.
- This preliminary assessment has made various assumptions that should be confirmed by further work, including site-specific ground investigation. However, in summary it is concluded that the proposed development is unlikely to have any significant effect on the stability of the hillside above.
- From this assessment it is considered that there is no reason in principle why development of the proposed ERF cannot safely occur (in relation to slope stability) with careful mitigation.

8 Recommendations

Ground investigation is required to confirm the assumptions made in this assessment:

- Confirm the thickness of colluvium and nature of disturbed zone on the slope, assuming that it is practicable to access the slope. It is likely to be possible to position boreholes on the former railway line.
- Position boreholes at the toe of the slope, on the edge of the highway, to confirm ground conditions in this zone.
- Boreholes across the development site to confirm the thickness of made ground, presence of marine gravels, absence of disturbed zone and absence of shear surfaces within the underlying Kimmeridge Clay.
- The borehole techniques should be designed to allow detailed logging of the soils, in particular evidence of existing polished shear surfaces.
- Laboratory testing to explore effective stress parameters, including residual shear strengths.
- Piezometers installed in boreholes at discrete depths to confirm piezometric profiles.
- Inclinometers installed on the railway and on the side of the highway to confirm current depths and rates of any slope movement. These can then be maintained during operation of the facility to provide early warning of any change in rate of movement.

The engineering process will include the following:

- Using the results of the ground investigation, carry out a detailed slope stability assessment to confirm the assumptions presented in this report. This should include consideration of potential non-circular slip surfaces.
- Design of the proposed excavation and embedded retaining wall with consideration of the potential for destabilisation of the adjacent slope.
- Developing a long-term monitoring strategy to mitigate the risk of shallow slope instability on the development, including potential blockage of the highway.

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