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3655

KIWI POINT QUARRY
NAURANGA GORGE
REPORT ON THE
POTENTIAL DEVELOPMENT OF THE QUARRY

For: Holcim New Zealand Limited
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Date: July 2015

TABLE OF CONTENTS

1. Introduction

This report is a compilation of the work completed on the investigation, analysis and assessment of the drilling and surface mapping programme carried out during 2014. The drilling and some of the surface mapping was carried out by Ormiston Associates Ltd with Geoscience Consulting (NZ) Ltd. carrying out the detailed structural mapping. Their analysis of this latter work is incorporated into this report.

Plate 1
General view of Kiwi Point Quarry looking to the Northwest with the main northern batter at the right and western batter at the left.

From the results of the above work, an assessment of the potential quarry excavation has been completed and a design drawn up. This has been incorporated into a model by Precision Aerial Surveys Ltd and the potential volumes of rock so developed measured by them.

2. Site Description

Kiwi Point Quarry is located on the southern side of State Highway 1 within the Ngauranga Gorge and approximately 5 kilometres north from Wellington City (Figure 1).

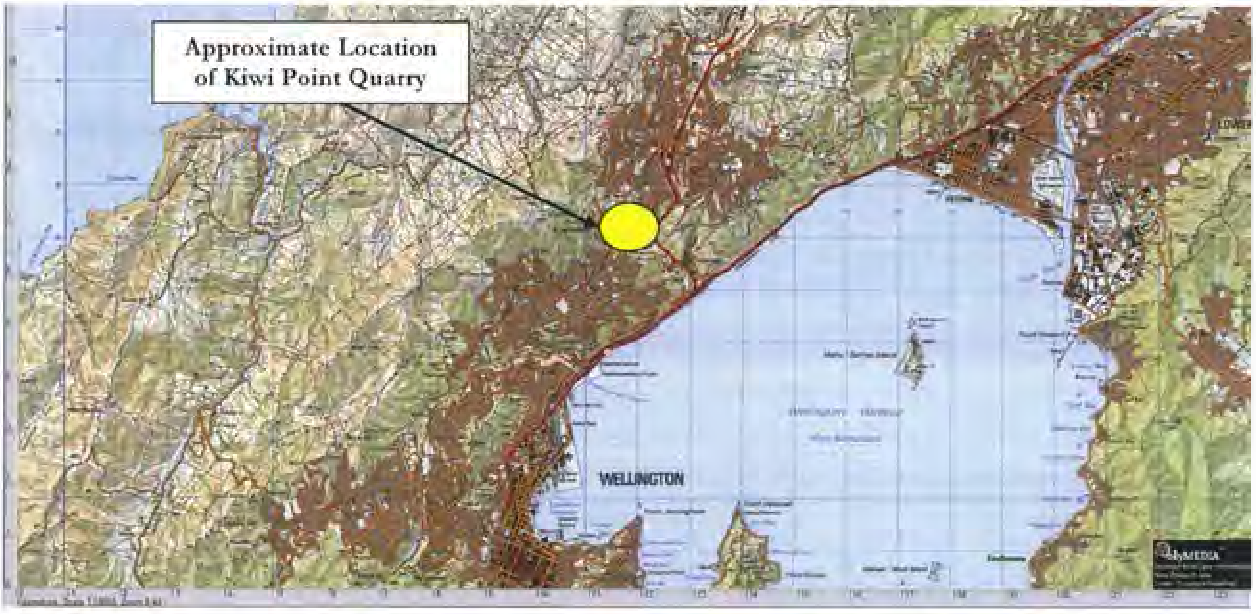


Figure 1.
Location Map, Kiwi Point Quarry.

The current quarry has been excavated from south to north into a ridge resulting in the formation of a batter slope rising steeply to the north. The quarry floor is currently being excavated to win aggregate production.

3. Geology

Reference to Geological Map 22 (Begg J.G, Mazengarb C, Geology of the Wellington Area, 1:50,000, IGNS 1996) indicates the quarry site is underlain by Torlesse Complex within the Wellington Belt of Triassic age (215 to 205 million years old) and comprising interbedded Sandstone and Argillite (mudstone). The lithology is commonly referred to as 'Greywacke'.

Observation of the Northern Quarry batter slope and Southern Area batter slope (Plate 2) indicates bedding defined by black argillite beds within the sandstone dominant lithology.

Plate 2
Northern Batter exposing slightly weathered to fresh sandstone dominant greywacke with bedding dipping to the left as defined by black argillite beds.

3.1 Weathering Profile

Slightly weathered to fresh greywacke is overlain by a variable depth of moderately to completely weathered greywacke that is discoloured brown (Plate 3). Weathering develops from the surface down and therefore forms an 'onion skin' generally following the surface topography. Weathering can also be observed close to prominent joints/faults which allow water to penetrate deeply into the rock mass. **Highly weathered greywacke** is discoloured and more than 50% of the rock mass is changed to soil. **Moderately weathered greywacke** is described as discoloured and discontinuity surfaces have discolouration which penetrates slightly into the rock mass. The moderately weathered rock mass is significantly weaker than the fresh rock.

For the purposes of this exercise we have assumed moderately weathered greywacke is overburden although there may be a market for this material from time to time (Plate 3).

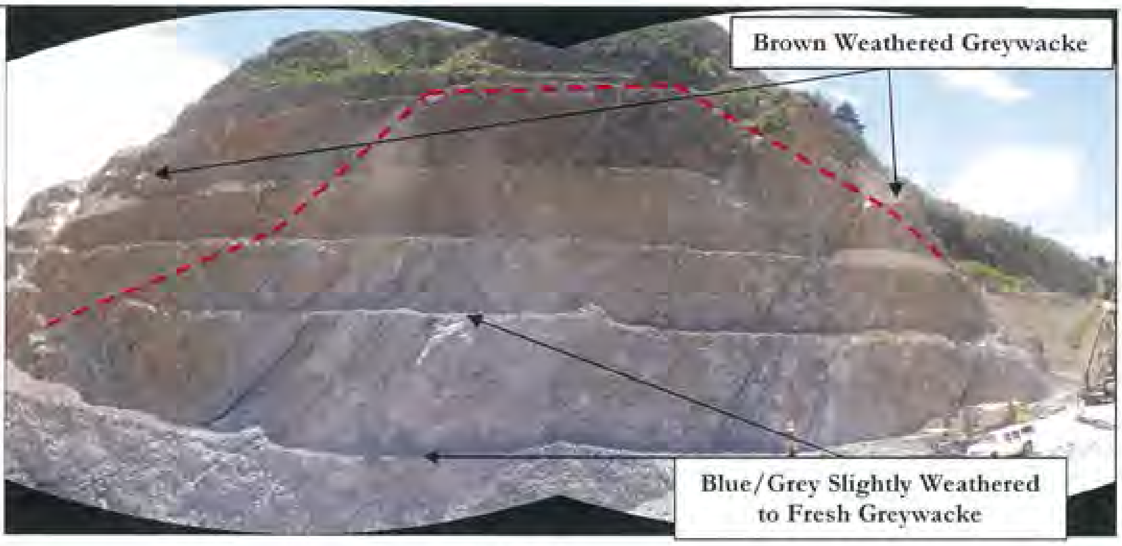


Plate 3
View of Northern Batter showing brown weather veneer forming an onion skin appearance overlying the slightly weathered to fresh (blue/grey) greywacke. Dashed red line denotes the approximate location of the contact between weathered and slightly weathered.

The approximate contact exposed within the northern and western batters between brown weathered materials (overburden) overlying slightly weathered to fresh greywacke is identified on the appended quarry plan Drawing No. 3655-1, Sheet 1, Revision 1 and on Plate 3 above.

The approximate location of the contact between overburden and greywacke as observed in the quarry batters and from available borehole logs is identified on Cross Sections A-A' (Drawing No. 3655-2, Rev. 1), and E – E' (Drawing No. 3655-6). Subsurface conditions have been extrapolated between the boreholes and opinions and recommendations are based on this assumption. However, even though such inference is made no guarantee can be made as to the validity of such inferences or assumptions due to the inherent variability of natural soil, rock and fill deposits both laterally and with depth. Consequently, variations between the boreholes may exist and may vary away from our cross sections.

4. Ultimate Pit Crest Constraints

Although the pit has been excavated to RL 65 metres the pit has along the northern and north western faces been backfilled to approximately RL 90 metres. Geoscience (Reference 1) has calculated the impact of placing this fill on the stability of the faces rising to the north and west and has demonstrated that under static conditions this fill slightly increases the factor of safety for the faces but that the difference becomes more significant under earthquake loading.

4.1 Northern Batter

The ultimate pit northern batter slope crest is currently restricted to the existing batter crest within residual to completely weathered soils at an elevation of approximately RL 210m (See Drawing 3655-1, Sheet 1, Revision 1). We also understand that without a variation to the land use consent, the quarry batter crest cannot be pushed back closer to the northern boundary of the property. There is also a minimum 20 metre buffer requirement between any future ultimate pit crest and residential property boundaries to the north. Geoscience (Reference 1) has recommended a maximum slope batter angle of 40° for this face. Current batters are at 76° or more with an overall slope, including benches, of more than 60°. It is evident therefore that unless the face is cut back at the top to flatten both the batters and the overall slope there will be continual failure of benches and batters on the face. As the benches are generally less than 5 metres wide and, for the most part, significantly less than this due to previous slope failures, the ability to mitigate such failures is limited. Furthermore, previous slope failures on the slope have, in some places, eliminated the benches entirely (see Plates 4 & 5).

Plate 4
View to the east of northern batters and benches. Note that benches are inaccessible for maintenance and debris accumulated on remnant benches allowing any further Rockfall material to spill down the face.

Geoscience has also carried out an assessment of the potential for large scale deep seated landslides (Reference 1, pages 12 to 14). This assessment has identified that while the probability of mass failure of the face is low for up to a 1 in 500 year earthquake, the stability of the moderately weathered upper layers of rock is not. Loose rock could possibly fall in an earthquake of a 1 in 50 year magnitude in sufficient quantities to overwhelm the benches and fall to the base of the face (Factor of Safety 1.07 or greater). For a 1 in 500 year earthquake such failures are probable (FoS 0.83 to 0.93).

Plate 5
Bench failure on lower western end of northern batter removing all access to the bench for maintenance. Failure inferred to be stress release on Joint Set 1.

4.2 Western Batter

The potential western extent of the existing western pit batter crest is limited by the access road and a Wellington City Council sewer running within the road formation. It is understood that relocation of this amenity is being investigated to enable the quarry footprint to be extended to the limits imposed by the topography alone. It is understood that while it might be possible to relocate the sewer, it is not possible to relocate the road.

In the northwest corner of the face (see Plate 6) joint Set 2 dips out of the western batter slope at approximately 45° and at an angle that is less than the existing 66° batter slope forming kinematically unstable large blocks perched on the batter slope that have been undermined by excavation as illustrated in the following Plates 6 & 7.

Plate 6
View to the west of the western batter showing Joint Set 2 dipping out of the face and remnant undermined kinematically unstable blocks perched on the batter slope.

Geoscience has assessed the potential for failure on this southeast facing batter (Ref 1 Table 1 page 11). They have identified that planar failure is likely at batter slopes greater than 35° and wedge failure at batters greater than 40°. These are apparent in Plates 6 & 7. A fault running up the intersection between the northern and western faces (see Drawing No. 3655-1, Sheet 1, Rev 1) complicates the structure of the situation (see Plate 5).

Plate 7
Joint Set 2 and potentially unstable blocks perched on the batter slope.

As the face swings to a north-south orientation the joint orientation steepens so that the recommended batter angles steepen slightly to 40° (Reference 1, Table 6, page 17).

4.3 Southern Batter

The structure along this face all dips into the face so that the risk of both wedge and planar failures is unlikely to exist. The southern batter will be cut to a level which will intersect with the south facing side of the previously excavated valley floor. This is at about RL 80 metres. Backfilling will be above the crest of the excavation. This face has a recommended batter of 45° based on the risk of topping failure (reference 1, page 17). As this risk can, during the working life of the quarry, be managed the design batters have been set at 65° with 5 metres benches at 15 metres intervals.

4.4 Eastern Batter

The eastern batter is all below RL 90 metres and will be backfilled following excavation. Geoscience has recommended (Ref 1 page 11) batter angles of 40° due to the risk of planar sliding. As this risk can, in the short term, be managed during the working of the face a design batter of 55° with 5 metre benches at 15 metre intervals is proposed.

5. Pit Remediation and Expansion

5.1 Northern Face

The northern face is over steep and does not have adequate benches on it. Analysis has determined that there is a risk of failure of the moderately weathered rock forming the upper approximately 70 metres of the face with 1 in 50 year earthquake and a high risk of failure with a 1 in 500 year event. To improve this stability it will be necessary to cut back the crest, lower the batter angles and widen the benches. Currently the excavation to the north is limited by planning constraints; physical constraints are provided by the property boundary and the presence of the telecommunications tower (see Drawing No. 3655 -1, Sheet 1, Rev. 1).

By cutting the face crest back to the edge of the telecommunications tower site as shown on Drawing No. 3655-1, Sheet 1, Rev. 1 a more stable configuration on the face can be achieved with an initial 40° batter increasing to 50° after the bench at RL 180 metres as shown on Cross Section A-A' (Drawing No. 3655-2, Sheet 2, Rev. 1). This batter configuration will daylight on the existing face at RL 105 metres. Below this level it will be necessary to support the existing face by building up the current fill to this level or slightly above.

While this face configuration is still steeper than that recommended by Geoscience, it will at least meet the recommendations for avoidance of planar failure. As this type of failure has the potential for the largest scale of face collapse it is anticipated that by achieving these batter slopes, the amount of potential rock fall will be substantially reduced.

5.2 Western Face

The main part of the western face can be excavated to reduce the batter angle to those recommended by Geoscience, however, at the intersection of the Northern and Western faces this is not possible for the full height of the face. Above RL 135 metres it is possible to flatten the batters to about 35° but below this level the constraints of the existing excavation and the access road require the batters be steepened to 45° (see Drawing No.'s 3655-1, 3655-3, 3655-5 and 3655-6). The proposed backfill will provide support up to RL 105 metres but for the batters between RLs 105 and 135 metres it may be necessary to consider other forms of remediation such as rock bolting. This will only be apparent once the final faces are exposed.

The fill proposed to extend to RL 90 metres will allow the batters to be steepened below this level to 45° as potential for failure above this angle can be managed during the operation of the quarry.

5.3 Southern Face

The Southern face will be subject to toppling failure only. As the crest of this face is below RL 90 metres, this type of failure can be managed during the operation of the quarry. Backfilling will support the face in the long term eliminating any risk of failure.

As the pit will be filled to the crest of the face no rock catch berms will be required on this face.

5.4 Eastern Face

The Eastern Face has not been specifically considered by Geoscience. They do, however, consider the jointing on the north face for west facing batters (Ref 1 Table 1, page 11). They note that the only form of failure likely is from wedge failure. There are no known joint intersections likely to form such features on this face. As fill will be placed against the total height of this face it is considered that batters can be cut to 55° without undue short term risk. Any failures that occur during the excavation can be managed by the operation.

As the pit will be filled to the crest of the face, no rock catch berms will be required on this face.

6. Rock Trap Design Sizing

There is an ongoing risk for failures occurring on the batter slope ranging from individual boulders to partial bench failures. The full northern face height is approximately 100 metres and there is potential for debris to travel the full face height. We would expect material to be slowed by loose debris on benches but there is a risk for debris to travel the full face height. This risk is increased with failures initiated by earthquake events.

Analysis of the potential for rockfall has been carried out using the computer programme Rockfall from Rocscience Ltd. A detailed description of the analyses carried out and the assessments from them are presented in Appendix 1.

This analysis has confirmed that the potential for inundation from rockfall decreases as the height from which the fall originates decreases (Appendix 1, cross sections 2a to 7a). Also that the distance that the rocks will travel from the base of the face is related to the nature of the benches and the size of the rockfall. The rocks will travel further when the benches are clean compared with when they have talus on them (see sections 2a, b, c and 8a, b, c). The design of the fill surface will also impact on the distance rocks will travel from the base of the face. By raising the fill level to RL 9

8m and leaving a 5 metre deep trench at the toe of the batter most of the rocks falling from the face will be captured and by constructing a 2.0 metre high barrier wall on the crest of the trench the risk of inundation can be reduced to a level where it can be considered to be minimal.

7. Pit Expansion and Deepening

To maximise the production from the pit a design for the deepening to the floor has been completed. This design incorporates the recommendations for batter slope design provided by Geoscience and is presented in a digital format prepared by Precision Aerial Surveys Ltd (PAS). The design is attached as Drawing Number 3655-7 at the rear of this report.

Additional to the resource available from below the floor is material that will be derived from the flattening of the batters on all the existing faces.

The calculation of the resources available from the quarry completed by PAS is based on topographical data provided by Beca in February 2014. Any estimate of the current resources will need to take into account sales since that date.

The estimated quantities are presented in Table 1 below.

Table 1
Estimated quantities of material (PAS).

Item	Volume (m^3)	Dominant rock Type	Assumed Density (t/m^3)	Tonnes
Pit	581,500	Blue & Blue Brown	2.6	1,512,000
Walls	425,300	Brown	2.4	1,021,000
Total	1,006,823	Mixed	-	2,533,000
Fill	798,800		1.6	1,278,000

8. Conclusions and Recommendations

8.1 Northern Batter

1. The northern batter is over-steepened, unstable and poses a high risk to the current pit floor access road, personnel and machinery.

2. We recommend application to WCC to vary the land use consent to allow advancing the batter crest further north by 30metres. Although it would be preferable to extend the quarry zone to within 20 metres of the quarry boundary the presence of the telecommunications tower might prevent this.

3. A more stable face than at present has been designed and it is recommended that fill be placed at the toe of the face to RL 105 metres. Any fill benches formed should be at least 20 metres wide.

4. As part of this construction we recommend buffer zones be formed at the toe of the face with raised berms to limit the spread of any rock falling from the face. This is discussed further in paragraph 17 below.

8.2 Intersection of Northern and Western Faces

5. On the northern face above RL 135 metres it will be possible to form batters that comply with the recommended stable angles. Below this level the constraints of the previous excavation limit the ability to flatten the batter angles sufficiently

6. It is recommended that following excavation of this face the need for specific remedial measures such as rock bolting be reviewed for the interval between the fill to be placed to RL 105 metres and the flatter batters above RL 135 metres.

7. To avoid the risk of rockfall impacting on any structures to be placed on the filled quarry floor, rock catch berms have been recommended.

8.3 Western Batter

8. It has been possible to design a face with batters which meet the recommended slopes of 40°.

9. To avoid the risk of rockfall impacting on any structures to be placed on the filled quarry floor, rock catch berms have been recommended.

8.4 Southern Batter

10 It has been possible to design a face to meet the recommended batter angle of 65

11 As fill will be placed to the crest of the face there are no long term concerns regarding the stability of the face.

8.5 Eastern Face

12 As fill will be placed against the eastern face once excavated it is considered that the design face can be excavated slightly steeper than the recommended batter angle of 50°. A batter angle of 55 has been used in the design.

13 It is considered that any potential rockfall from this face during excavation can be managed during the operation of the quarry. Backfill will provide support to stabilise this face in the longer term.

8.6 Overburden

14 Recontouring of the current batter slopes will produce significant volumes of material that will need to be disposed of either by sale or backfilling the existing quarry floor. It is estimated that some 800,000 bench m3 (1,300,000 tonnes) of this material will be generated from the proposed redesign of the quarry. If recompacted on site, this will fill a volume of some one million cubic metres.

8.7 Resources

15 Recontouring of the quarry batter slopes will produce some 430,000 bench m^3 (1,000,000 tonnes) of rock. This will be predominantly brown in nature but with limited quantities of blue-brown rock.

16 By deepening the pit a further resource, primarily blue but with a significant proportion of blue-brown rock, will be produced. The quantities estimated are some 600,000 bench m3 (1,500,000 tonnes).

8.8 Rockfall Analysis and Fill Design

17 Analysis of the stability of the quarry faces by Geoscience has identified the likelihood of failures occurring on the faces particularly under conditions of earthquake acceleration. Such rockfalls will impact on the potential use of the land to be developed form backfilling the quarry.

18 To assess the risk of inundation onto the platform to be constructed on the quarry floor, a computer programme "Rocfall" has been used to assess the relative impact of various situations.

19 Results from this analysis have shown that the risk of inundation increases with the height on the face, from increased size of the fall and the nature of the benches on the face.

20 A design for the toe fill has been achieved to reduce this risk to negligible proportions. This design consist of a toe bund to RL 105 minimum, a bund along the outside edge of the main bund, raising the fill on the lower platform to RL 98 metres and leaving a 5 metres deep trench along the base of the toe bund. A rockcatch fence at least 2 metres high along the crest of this trench will augment the effect to prevent the spread of rock falling from the face.

9. Reference

1 Geoscience Consulting (NZ) Ltd, 24 February 2015, Slope Stability Review, Kiwi Point Quarry, Ngauranga Gorge, Wellington.

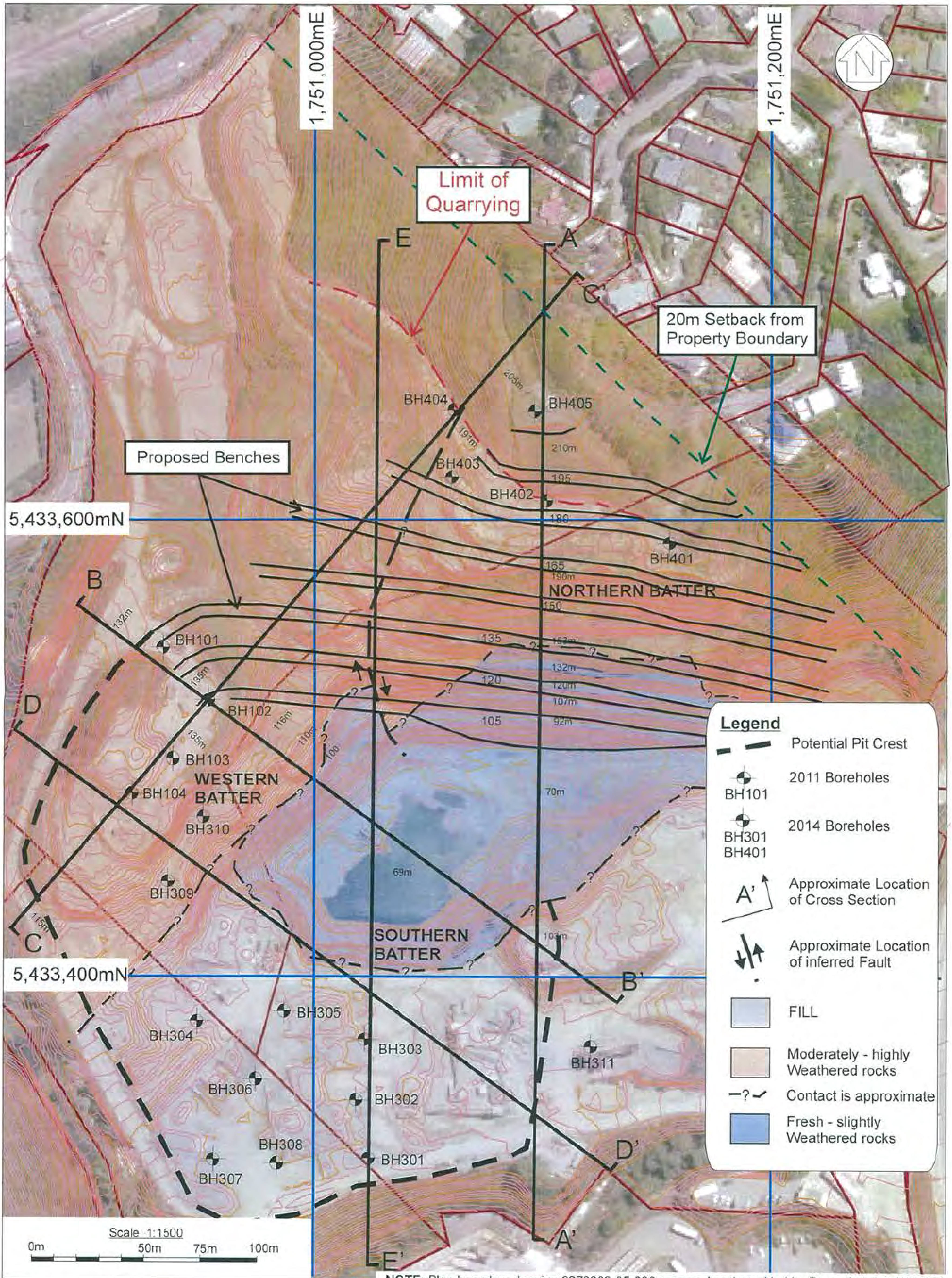
Prepared by

Sandy Ormiston MSc MAusIMM

And

Simon Carryer MSc FAusIMM

July 2015



NOTE: Plan based on drawing 3272389-95-006 prepared and provided by Beca, dated 03/02/2014. Refer to Drawing No. 3655-7 for the Ultimate Quarry Development Plan, dated 14/07/2015.

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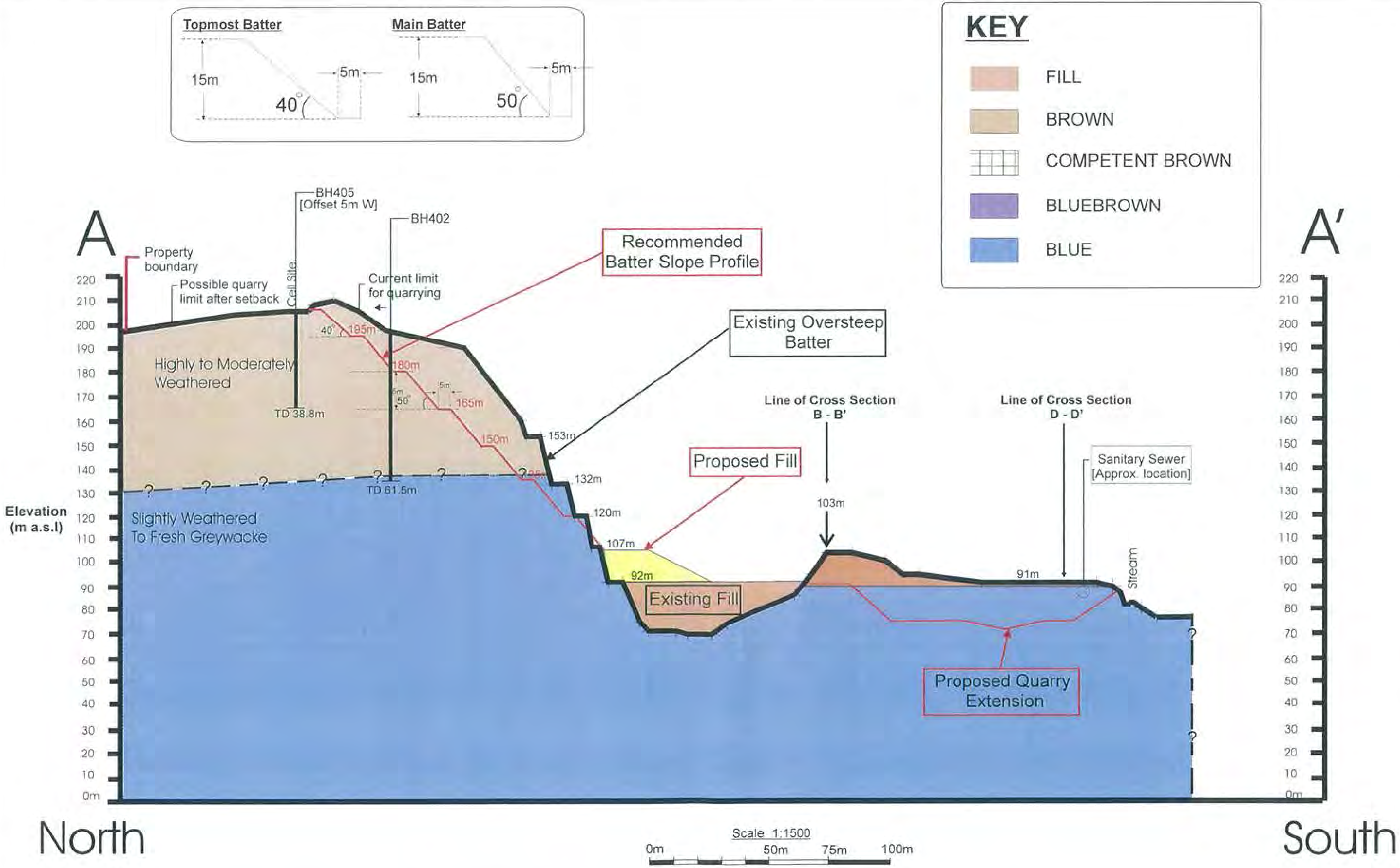
CLIENT:	Holcim NZ Ltd
LOCATION:	Kiwi Point Quarry
TITLE:	Site Plan - General Geology & Proposed Northern Batter

SCALE: 1:1500 @ A3
DRAWN: TCF
DATE: 14 July 2015
CHECKED: AWO

DRAWING NO
3655-1
SHEET 1 OF 7



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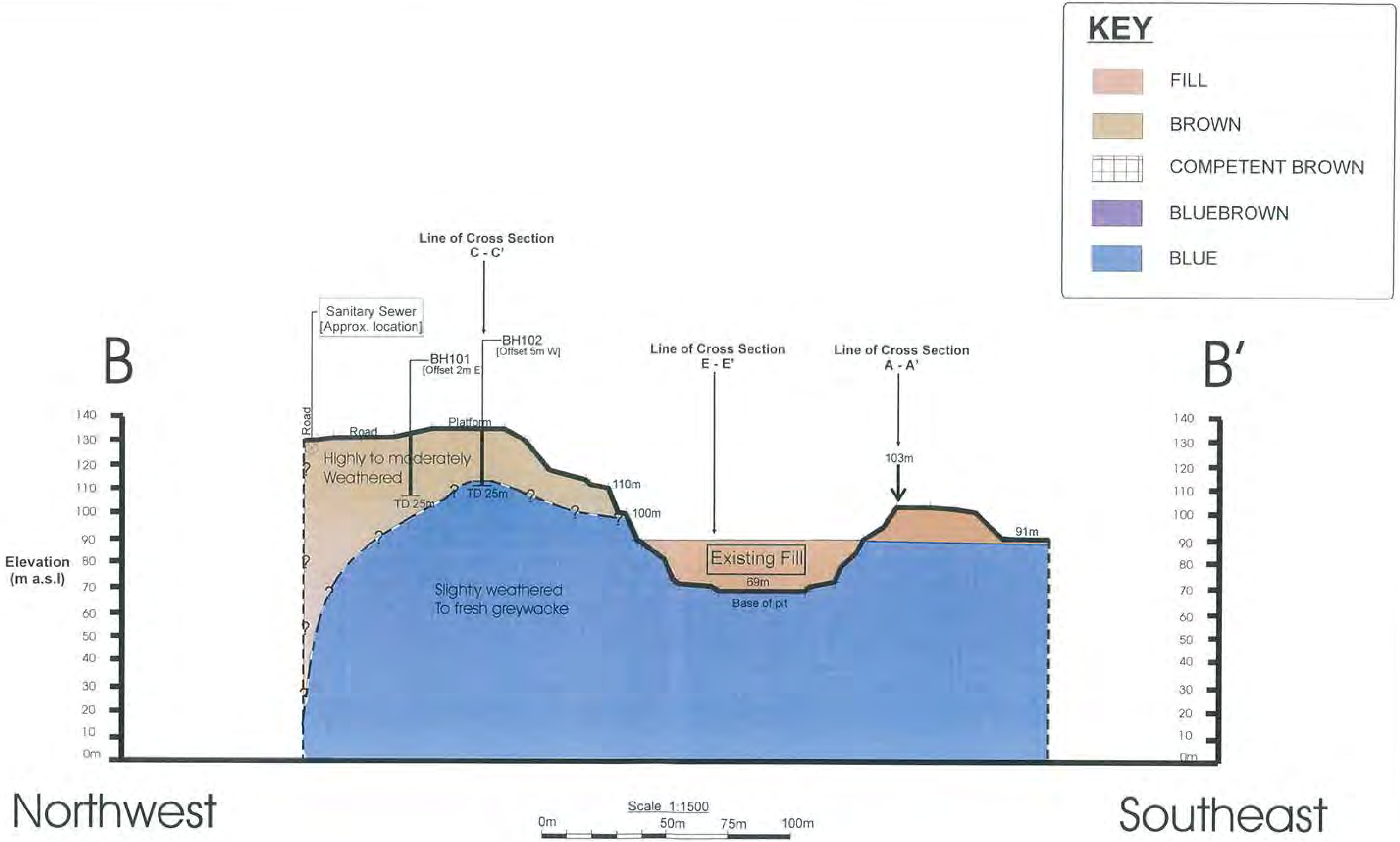
CLIENT: Holcim NZ Ltd.
LOCATION: Kiwi Point Quarry
TITLE: Section A - A'

SCALE: 1:1500 @ A3
DRAWN: TCF & SJC
DATE: 14 July 2015
CHECKED: AWO

DRAWING NO
3655-2 Rev 1



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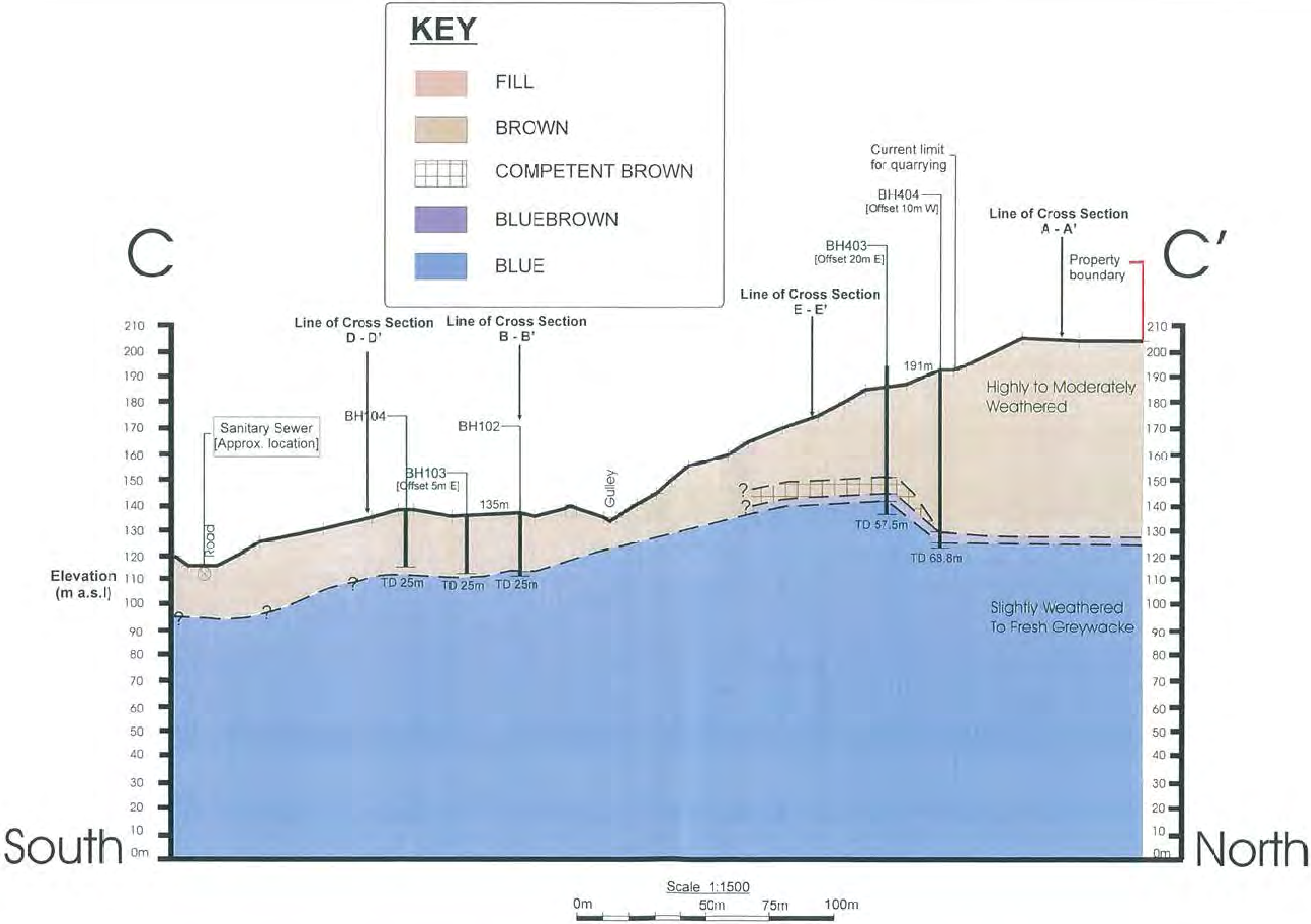
CLIENT:	Holcim NZ Ltd.
LOCATION:	Kiwi Point Quarry
TITLE:	Section B - B'

SCALE: 1:1500 @ A3
DRAWN: TCF
DATE: 14 July 2015
CHECKED: AWO

DRAWING NO
3655-3

SHEET 3 OF 7



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CLIENT: Holcim NZ Ltd.
LOCATION: Kiwi Point Quarry
TITLE: Section C - C'

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DRAWN: TCF
DATE: 14 July 2015
CHECKED: AWO

DRAWING NO
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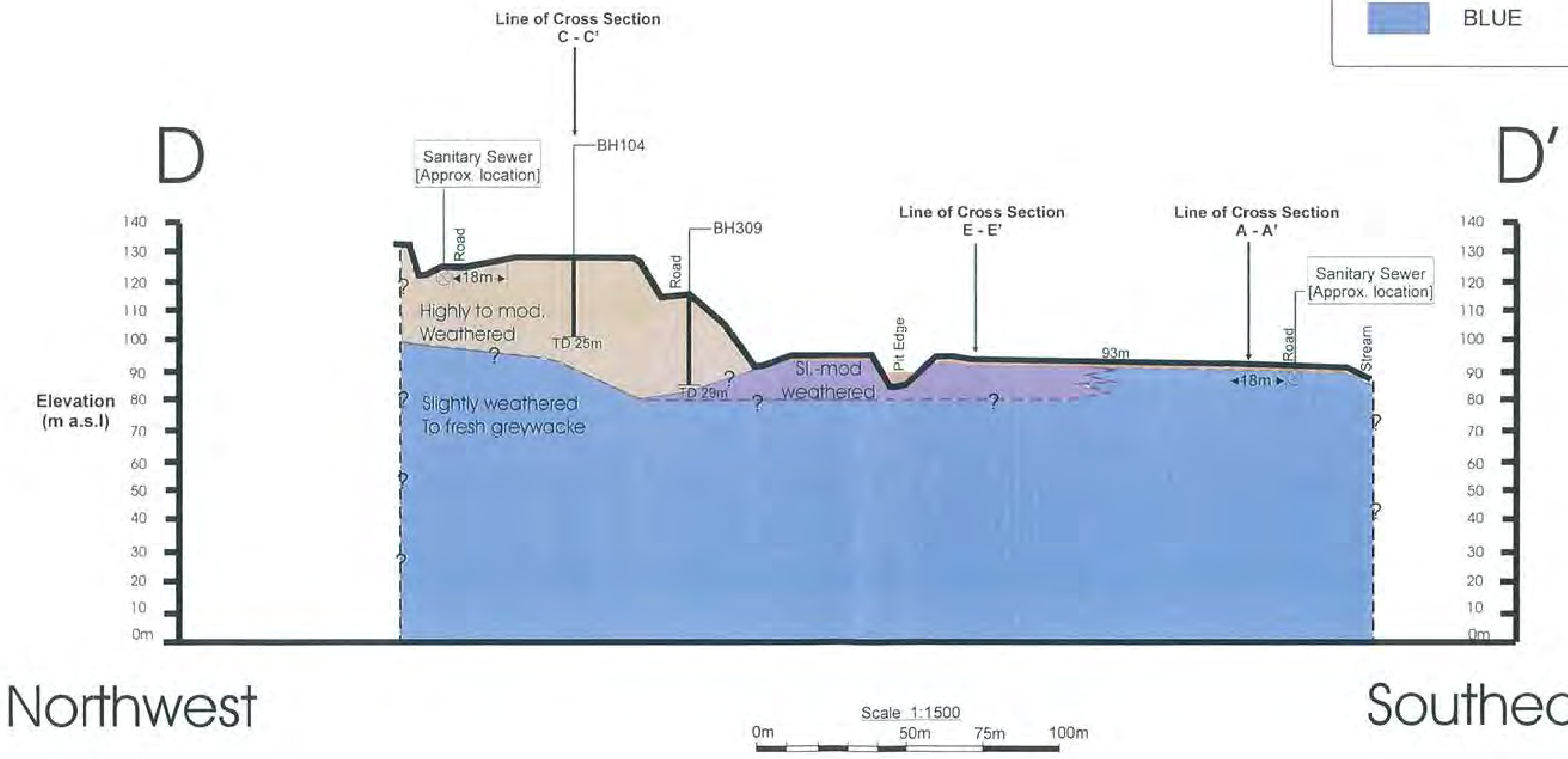
SHEET 4 OF 7



KEY

- FILL
- BROWN
- COMPETENT BROWN
- BLUEBROWN
- BLUE

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LOCATION: Kiwi Point Quarry
TITLE: Section D - D'

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DATE: 14 July 2015
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DRAWING NO
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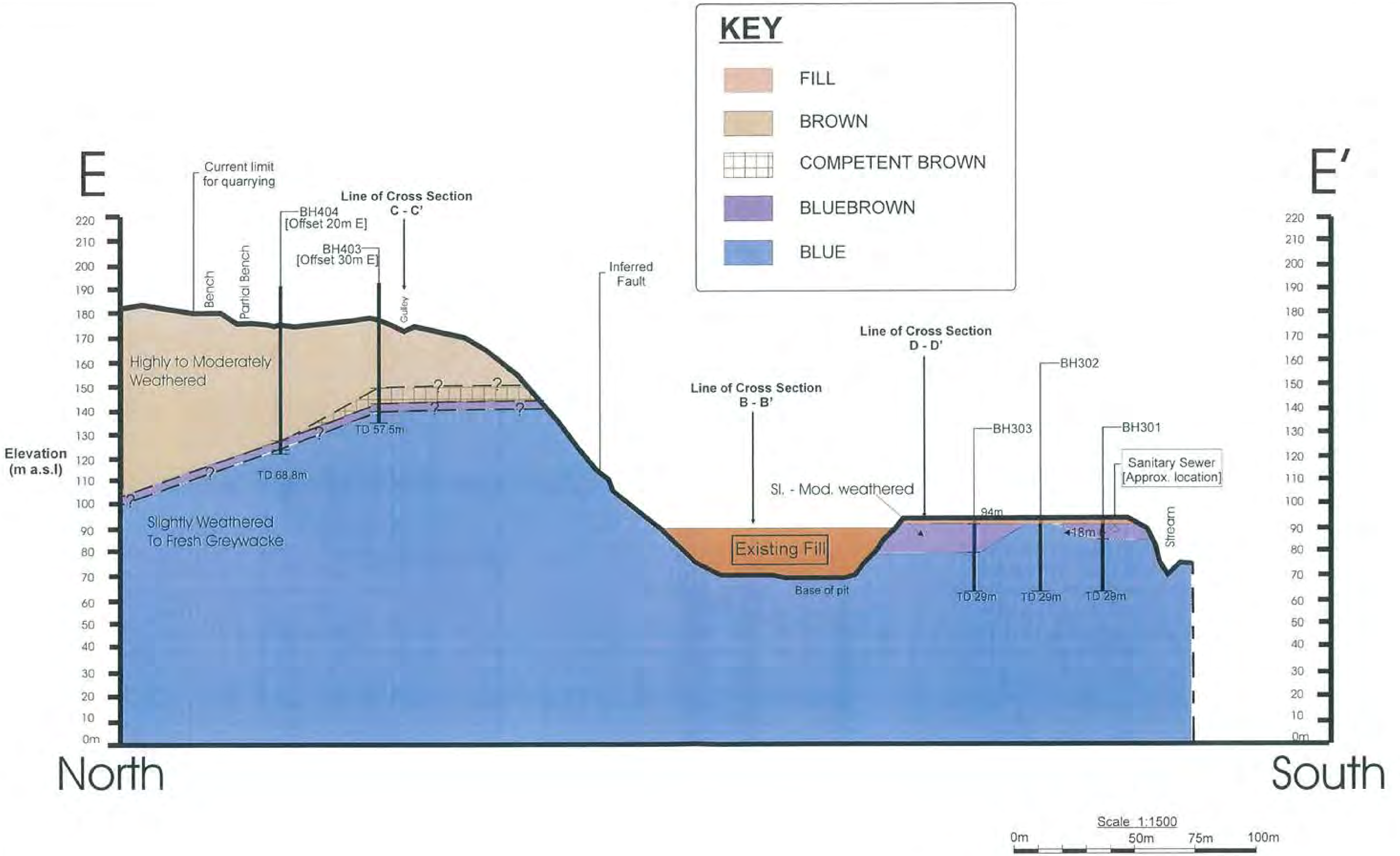
SHEET 5 OF 7



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CLIENT: Holcim NZ Ltd.
LOCATION: Kiwi Point Quarry
TITLE: Section E - E'

SCALE: 1:1500 @ A3
DRAWN: TCF
DATE: 14 July 2015
CHECKED: AWO

DRAWING NO
3655-6

NOTE: Plan based on drawing prepared and provided by Precision Aerial Surveys Ltd. D&G Ref: 6010-01, Dated May 2015.

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CLIENT: Holcim NZ Ltd.
LOCATION: Kiwi Point Quarry
TITLE: Ultimate Quarry Development Plan

SCALE: 1:2000 @ A3
DRAWN: TCF
DATE: 14 July 2015
CHECKED: SJC

DRAWING NO
3655-7

APPENDIX 1

RocFall Analysis

TABLE OF CONTENTS

APPENDIX 1
ROCKFALL ANALYSIS

1. Introduction

The computer programme "RocFall" from RocScience Inc has been used to test a number of scenarios to assess the risk of inundation onto the proposed building platform at the base of the north face of the Kiwi Point Quarry. The face configuration used is that proposed from the parameters provided by Geoscience Consulting (NZ) Ltd (Reference 1) providing for 15 metre high batters at 50° with 5 metre wide benches. This configuration provides an overall face slope of approximately 45°. A 20 metre wide toe bund is incorporated in the face design and a 2 metre high berm is constructed on the outer edge of this to limit debris from the face rolling over onto the building platform below. Following derivation of the results from the initial analyses described above, modifications to the design of the toe have been considered and the most effective result decided to minimise the distance of rocks bouncing from the face toe has been modelled. This will reduce the risk of impact on buildings constructed on the platform below the face to minimal.

2. Input Parameters

Initially the model was tested using static conditions and subsequently it was again tested under dynamic conditions for a 1 in 50 year earthquake and a 1 in 500 year earthquake. The results of a fall originating from each of the benches were compared to derive the worst case situation. The rock characteristics used have been derived from tables provided by RocScience Inc. and are for "weathered rock" from the top of the face and "bedrock" for the rest of the benches/faces. Three sizes of rock have been used in the model – small (2.56 kg), medium (780 kg) and large (3,120 kg). The results were analysed for single rocks, small falls (10 rocks total or 12 tonnes), large falls (50 rocks total or 70 tonnes) and huge falls (100 rocks total or 130 tonnes). A range of rock shapes were used in the testing.

Two different slope situations have been tested, being with clean bedrock benches and with talus on the benches. In all scenarios the faces were assumed to be clean of any talus.

The parameters used are set out in full on the RocFall Analysis Information Sheets at the end of this appendix.

For analysis under the dynamic situations of a 1 in 50 year earthquake (SLS) and a 1 in 500 year earthquake (ULS) the magnitude of the peak horizontal velocity of the rocks had to be derived empirically. The rationale for the derivation of peak horizontal velocities for the Kiwi Point site is described in **Section 3** of this report below.

3. Rationale for Selection of Peak Initial Horizontal Velocity

The Serviceability Limit State (SLS) is indicated as a 1 in 50 year earthquake where 0.13g acceleration is expected, and which will result in any structures directly affected being able to remain operational.

The Ultimate Limit State (ULS) is indicated as a 1 in 50 year earthquake where 0.35g acceleration is expected, and whereby any person within or adjacent to a structure are not endangered. These criteria are in conformation to NZ Standard 1170.5:2004 including Supplement 1.

These expected acceleration values have been provided by Geoscience Consulting (NZ) Ltd (Reference 1) and these accelerations used as a basis for the modelling of rockfall from the face.

GeoNet Strong Motion data records all large ground movements from stations located in key areas around the country. Analysis of selected data sites shows that the largest peak ground accelerations (PGA) measured were during the 2010/2011 Christchurch earthquake sequence with the most relevant results plotted below. Locations summarised in **Table 1** are considered the most similar to the Kiwi Point Quarry site, where measurements have been recorded and on rock slopes considered to likely provide similar effects to the Kiwi Point Quarry site.

Table 1: Summary of large strong motion earthquakes in Christchurch.

Event/Date	Mw(i)	Epicentre Distance (km)(ii)	Peak Velocity (m/s)	PGA(iii)
Panorama Rd 13/06/2011	6.0	1km	1.17m/s	0.82g
Godley Drive 13/06/2011	6.0	7km	1.69m/s	1.86g
Heathcote Valley 21/02/2011	6.2	2km	0.97m/s	1.47g

Notes:

(i) M_w = Measured Earthquake magnitude

(ii) Epicentre Distance from station.

(iii) PGA – Peak Ground Acceleration

Motion data summarised in **Table 1** above indicates a maximum PGA of between 0.82g and 1.86g experienced during magnitude 6.0 to 6.2 earthquakes. These accelerations are between 2.37 and 5.31 times larger than what is anticipated to be experienced at Kiwi Point Quarry for a 1 in 500 year earthquake.

The associated horizontal velocities recorded for the motions presented in **Table 1**, were between 0.97 m/s and 1.69 m/s and as such, we can assume that the maximum horizontal velocity expected for a 1 in 50 and 1 in 500 year earthquake at the Kiwi Point Quarry to be smaller than this, due to lower expected Peak Ground Accelerations.

To choose the initial horizontal velocities for use in the RocFall model of the Kiwi Point Quarry north face, we have chosen four (4) accelerogram stations from sites with similar geological characteristics to the Kiwi Point Quarry slope and have plotted Peak Ground Acceleration against Peak Horizontal Velocity for all data recorded from these sites. Accelerograms chosen are Makara Bunker (Wellington), Wainuiomata Hill (Wellington), Godley Drive (Christchurch) and Panorama Road (Christchurch). By extrapolation of these graphs, presented at the end of this Appendix, figures summarised in **Table 2** have been derived for Average Peak Horizontal Velocity as the input parameter into RocFall for modelling dynamic situations resulting from earthquake acceleration of the rock face.

Table 2 – Assessed Peak Horizontal Velocities for Chosen Sites.

Location	Peak Horizontal Velocity (m/s)	
	SLS (0.13g)	ULS (0.35g)
Makara Bunker	0.171 m/s	1.127 m/s
Wainuiomata Hill	0.081 m/s	0.277 m/s
Godley Drive	0.106 m/s	0.306 m/s
Panorama Road	0.127 m/s	0.419 m/s
	Average: 0.105 m/s	**Average: 0.334 m/s**

4. Results

The results of the individual analyses are presented in the Analytical Sections following the analysis graphs for the Derivation of Peak Horizontal Velocity at the end of this Appendix.

4.1 Static Conditions, "Clean Hard Bedrock" Benches

A variety of scenarios were tested for static conditions with "clean hard bedrock" as described below.

4.1.1 From Top of Face

Six analyses are presented in the sections attached to the end of this Appendix. These (Sections 1a to 1c) confirm that as the size of the individual rock increases the distance travelled from the toe will increase. Also that for all rock sizes the initial protection design, as analysed, is inadequate.

For small, large and huge rockfalls, of mixed sized rocks (Sections 2a to 2c), the number of rocks breaching the protection barrier increases with the size of the rockfall.

4.1.2 From First Bench

This scenario (Sections 3a to 3c) shows that small rockfalls are held within the containment, however for large and huge rockfalls, the initial protection design is breached and runout distances have not been reduced compared with rockfalls from the top of the slope (Sections 2a to 2c).

4.1.3 From Second Bench

This scenario (Sections 4a to 4c) shows that the initial protection design is breached by all rockfalls, however, the runout distance and amount of rocks decreases for the large and huge falls.

4.1.4 From Third Bench

This scenario (Sections 5a to 5c) shows that the design protection will contain the majority of rocks from all size rockfalls but the distance travelled is very little different from the bench above.

4.1.5 From Fourth Bench

This scenario (Sections 6a and 6c) shows that for a small and huge rockfalls (12 tonnes and 130 tonnes), all rocks will be contained, but for the large rockfall (70 tonnes) the occasional rock, although having limited runout distance, will still breach the design containment.

4.1.6 From Fifth Bench

This scenario (Section 7a) shows that large rockfalls from the lowest bench will be contained by the design containment.

4.2 Static Conditions, Dirty Benches

To test the effect of talus collecting on the benches, the worst case scenario determined from the above analyses to be from the top of the face has been modelled for each of the three sizes of rockfall (Sections 8a to 8c). These analyses show that although the talus has a significant effect in reducing the number of rocks breaching the design containment, the large and huge rockfalls (70 tonne and 130 tonne) still breach the containment albeit with less runout than for clean benches (Sections 2a to 2c).

4.3 Dynamic Conditions

To test the effect of an initial horizontal velocity on the falling rocks as would be imparted by an earthquake, the rockfalls from the top of the face were analysed for both clean and dirty benches. The scenarios analysed are for peak initial horizontal velocities that would be imparted from a 1 in 50 year earthquake (SLS) and a 1 in 500 year earthquake (ULS).

4.3.1 Rockfall from 1 in 50 year Earthquake, Clean Benches

The analyses presented (Sections 9a to 9c) have been compared with the analyses for static conditions (Section 2a to 2c). These show that under earthquake conditions (SLS), large and huge rockfalls are not significantly different than for static conditions. It is interesting to note that the runout and amount of rocks for a small (12 tonne) in a 1 in 50 year scenario decreases compared to static conditions.

4.3.2 Rockfall from a 1 in 50 year earthquake, Dirty Benches

The analyses presented (Section 10a to 10c) show that there is a decrease in runout and amount of rocks breaching the containment in all size rockfalls, compared to a clean bench SLS scenario (9a to 9c). There seems to be no significant change in runout or amount of rocks breaching the containment for all rockfalls compared to the dirty bench, static condition (Section 8a to 8c).

4.3.3 Rockfall from a 1 in 500 year Earthquake, Clean Benches

This analyses shows that small falls would appear to have fewer rocks breaching the design containment than for the static condition (Section 11a). Large rockfalls (70 tonne) show significantly greater numbers of rocks breaching the containment and having much greater outrun than compared to both static and SLS conditions (Sections 11b). In a huge rockfall (130 tonne) the number of rocks and runout distance are reasonably similar to both static and SLS conditions.

4.3.4 Rockfall from a 1 in 500 year earthquake, Dirty Benches

The analyses presented (Sections 12a to 12c) show that compared with the static conditions and dirty bench (Section 8a to 8c), there is no significant difference in runout or rocks breaching the containment for all rockfalls. Compared to a Clean Bedrock scenario (ULS) (Section 11a to 11c) there is a significant decrease in rockfall runout distance and the number of rocks breaching the containment.

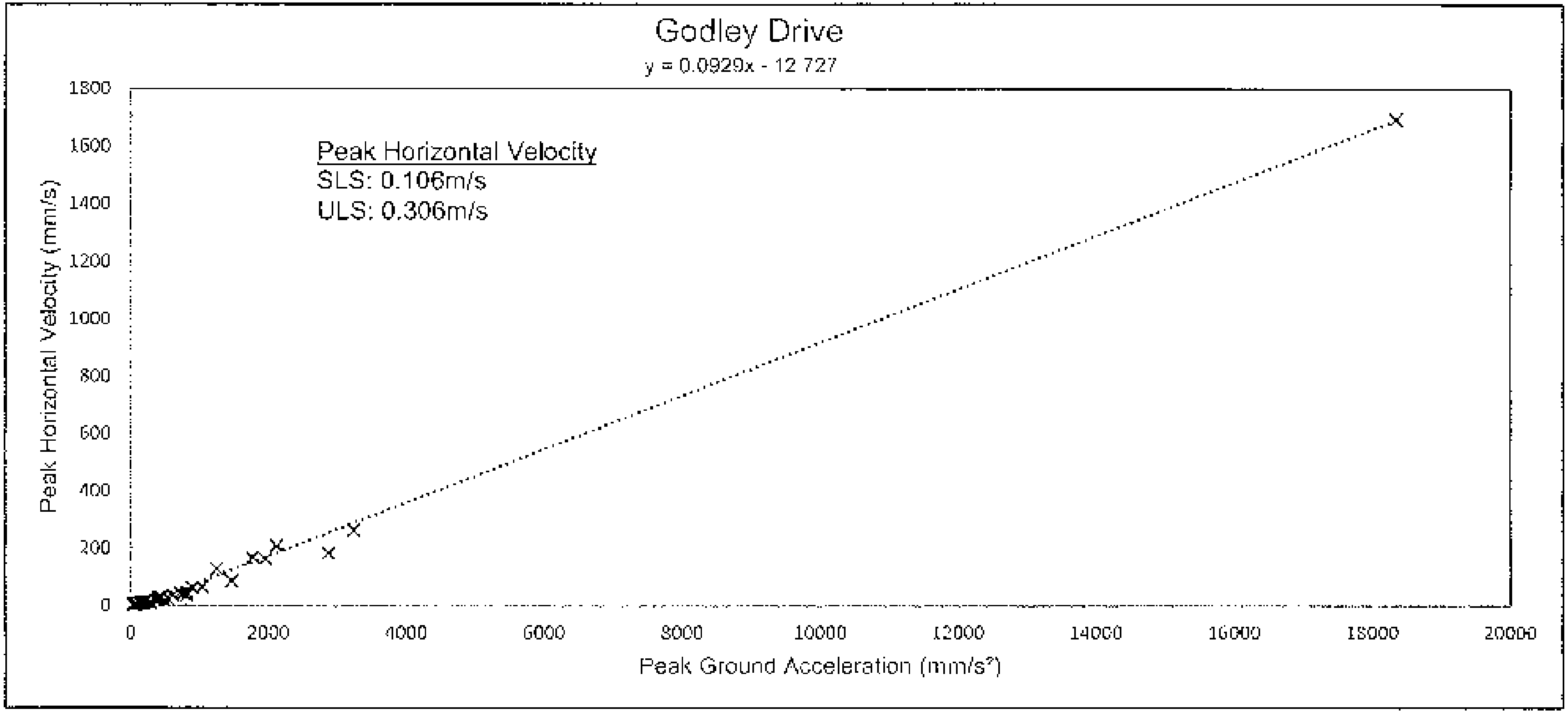
5. Redesign of Containment Structure

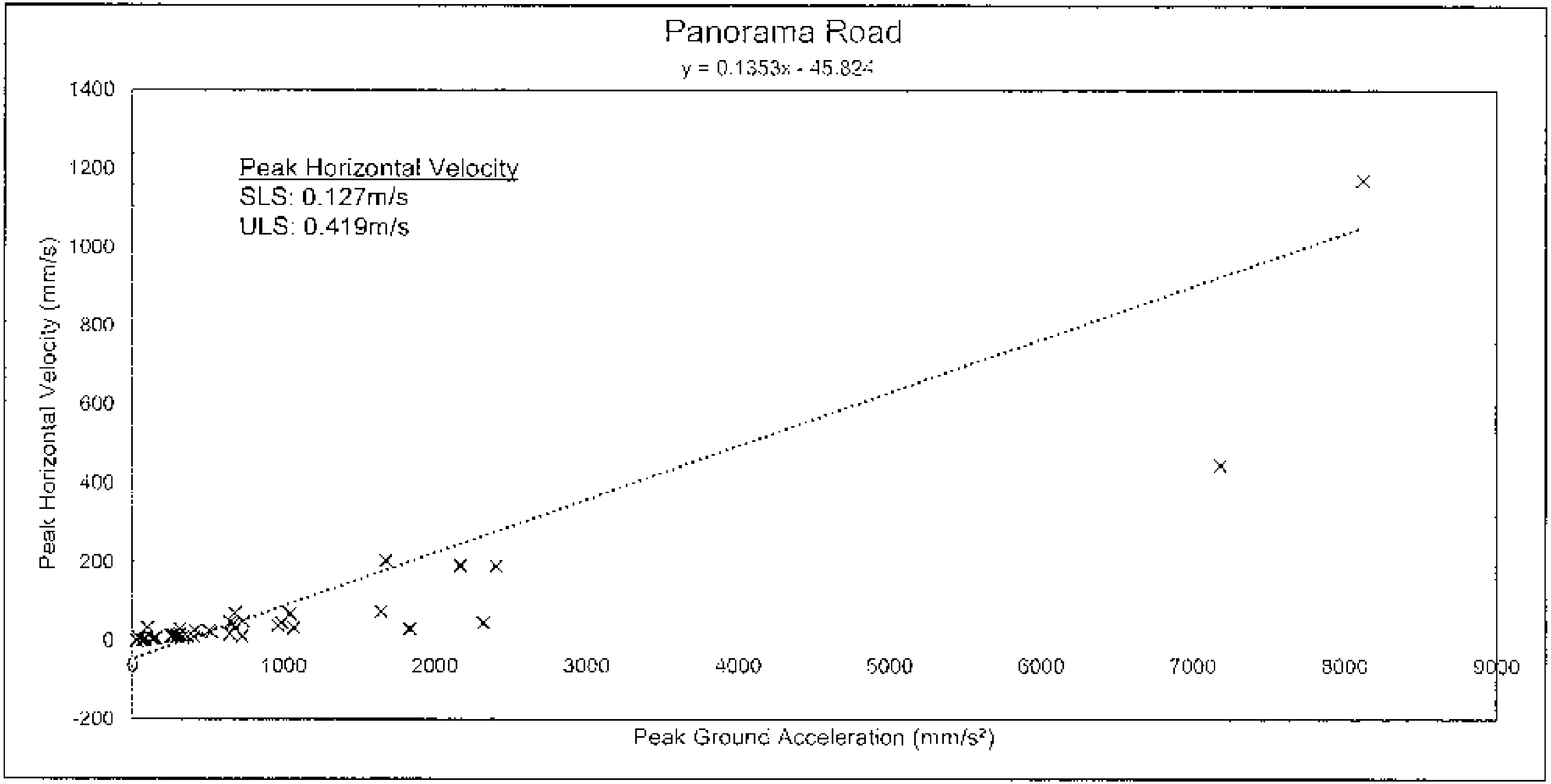
As the results of the analyses show that the initial design of the fill at the toe of the face will not contain any but the smallest of rockfalls, a variety of different designs were tested using the worst case scenario of a fall from the top of the face in a 1 in 500 year earthquake (ULS), with clean hard bedrock benches and talus covered, dirty benches. The result of these analyses has provided a satisfactory design as presented in Section 13a and 14a. To test the risk of inundation from an even greater rockfall situation resulting from 200 tonne and 300 tonne falls have been analysed and presented as Sections 13b, 13c, 14b and 14c.

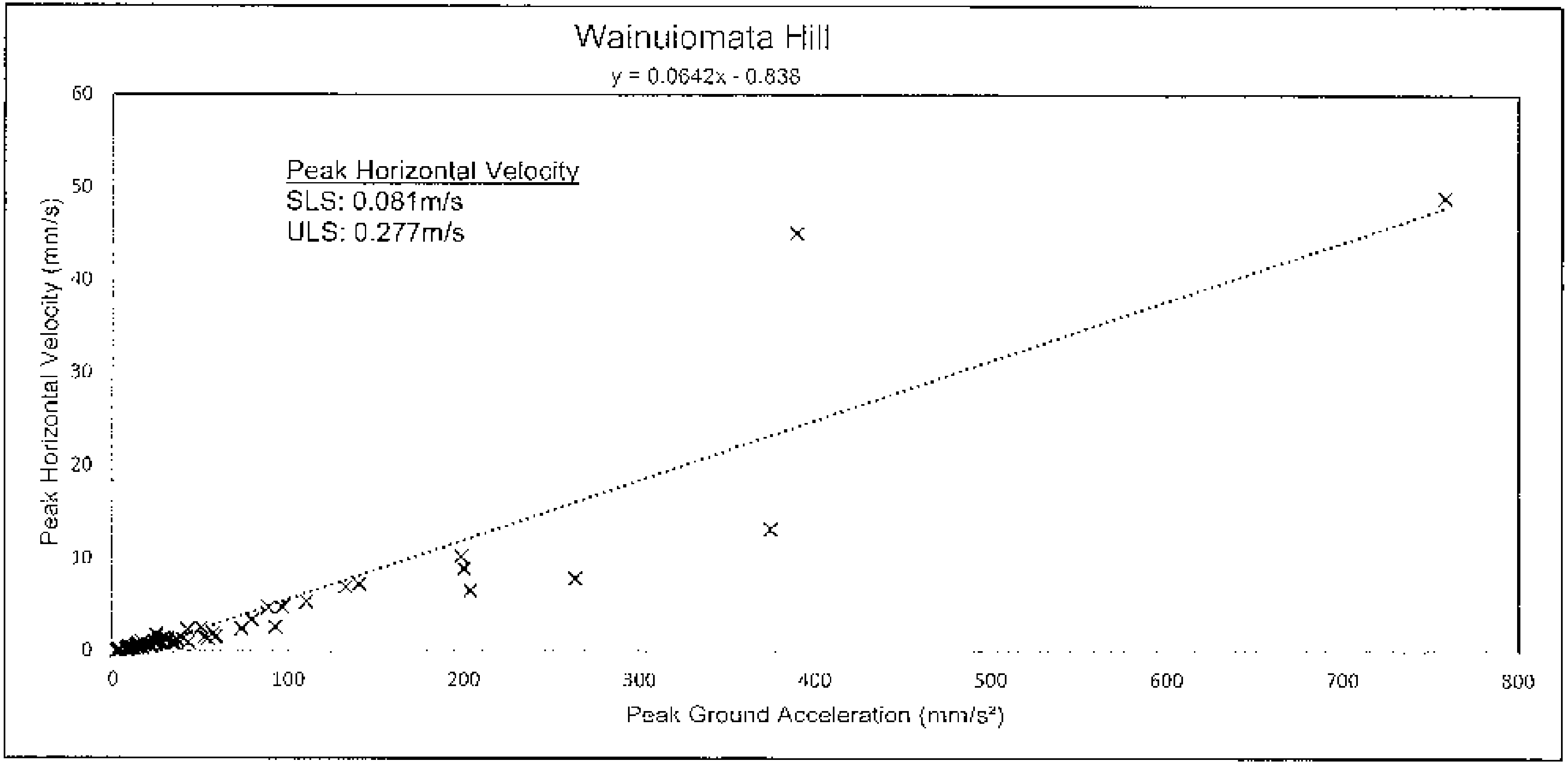
These results show that the ultimate design for a containment structure as presented in these sections (Sections 13a to 13c and 14a to 14c) can provide for the containment of the majority of all falls that are considered by us to be potentially likely from this face.

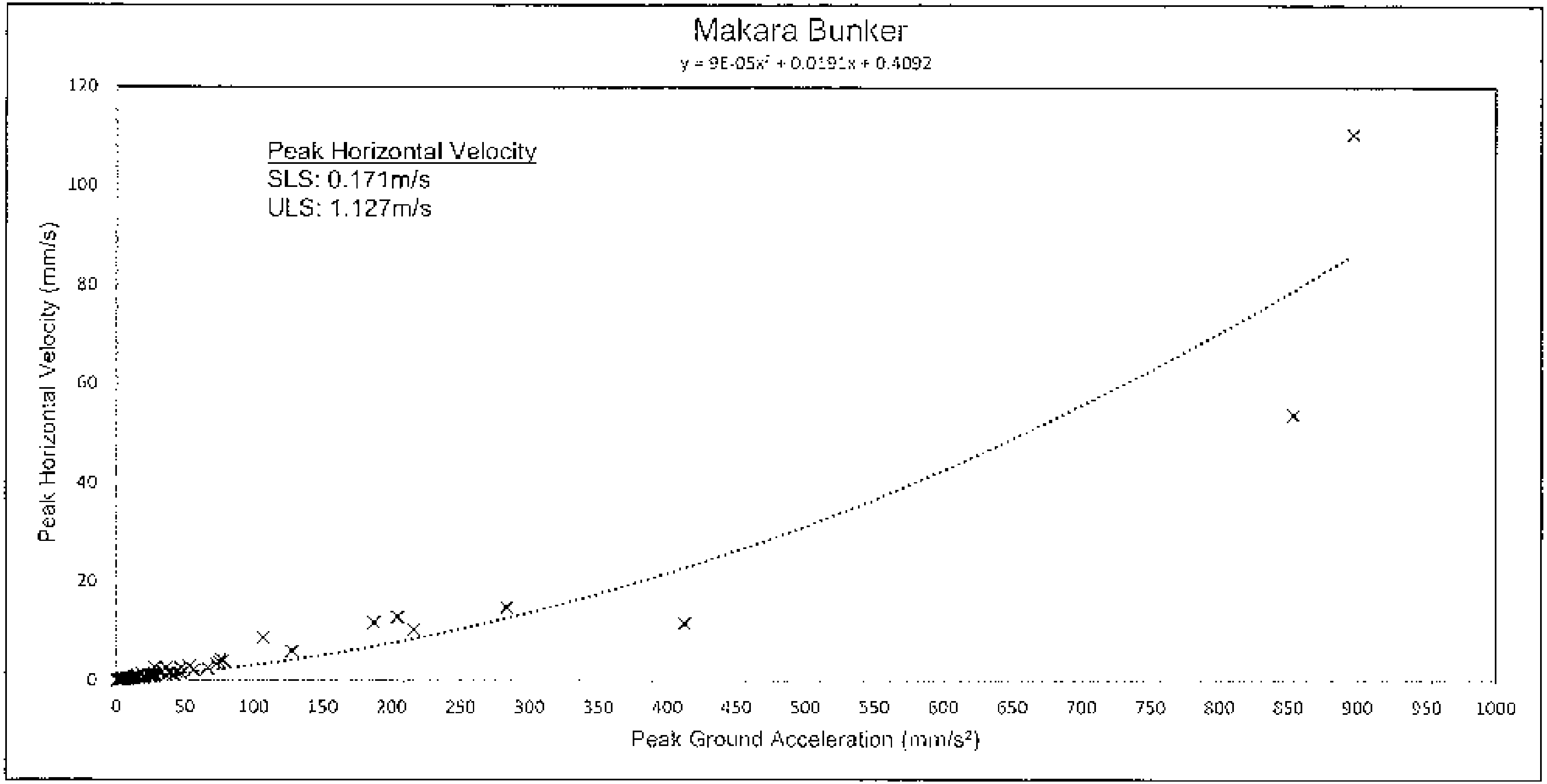
6. Summary and Conclusions

1. Analysis of potentially likely rockfalls from the north face of the Kiwi Point Quarry as redesigned using the recommendations provided by Geoscience Consulting (NZ) Ltd have been carried out using the programme "RocFall" provided by RocScience Inc.
2. Analysis has been undertaken on a slope under 'worst case' scenario, clean hard bedrock conditions.
3. The analyses have tested the risk of inundation of the building platform proposed at the base of this face under both static and dynamic conditions.
4. The dynamic conditions tested are for a 1 in 50 year (SLS) earthquake and a 1 in 500 year (ULS) earthquake. The horizontal velocities anticipated to result from these events have been derived using measured data from four sites considered to be similar to that of the Kiwi Point Quarry. The figures used are the average of these four results being 0.105 m/sec (SLS) and 0.334 m/sec (ULS).
5. Testing under both static and dynamic conditions has shown that the risk for inundation of the building platform will increase with the size of the rockfall and also from the height on the face from where it originates. The risk does not seem to increase drastically with the size of the earthquake, however a large rockfall (70 tonnes) in a ULS earthquake caused a significant increase in runout and amount of rocks breaching the containment.
6. To reduce the risk of inundation of the building platform the toe containment structures at the toe of the northern face have been redesigned from that originally proposed. The redesigned containment structure is shown from the analyses carried out to reduce the risk of inundation of the building platform to very low levels.


Godley Drive
y = 0.0929x - 12 727
Peak Horizontal Velocity
SLS: 0.106m/s
ULS: 0.306m/s
Peak Horizontal Velocity (mm/s)
Peak Ground Acceleration (mm/s²)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000


Panorama Road
y = 0.1353x - 45.824
Peak Horizontal Velocity
SLS: 0.127m/s
ULS: 0.419m/s
Peak Horizontal Velocity (mm/s)
1400
1200
1000
800
600
400
200
0
-200
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
Peak Ground Acceleration (mm/s²)

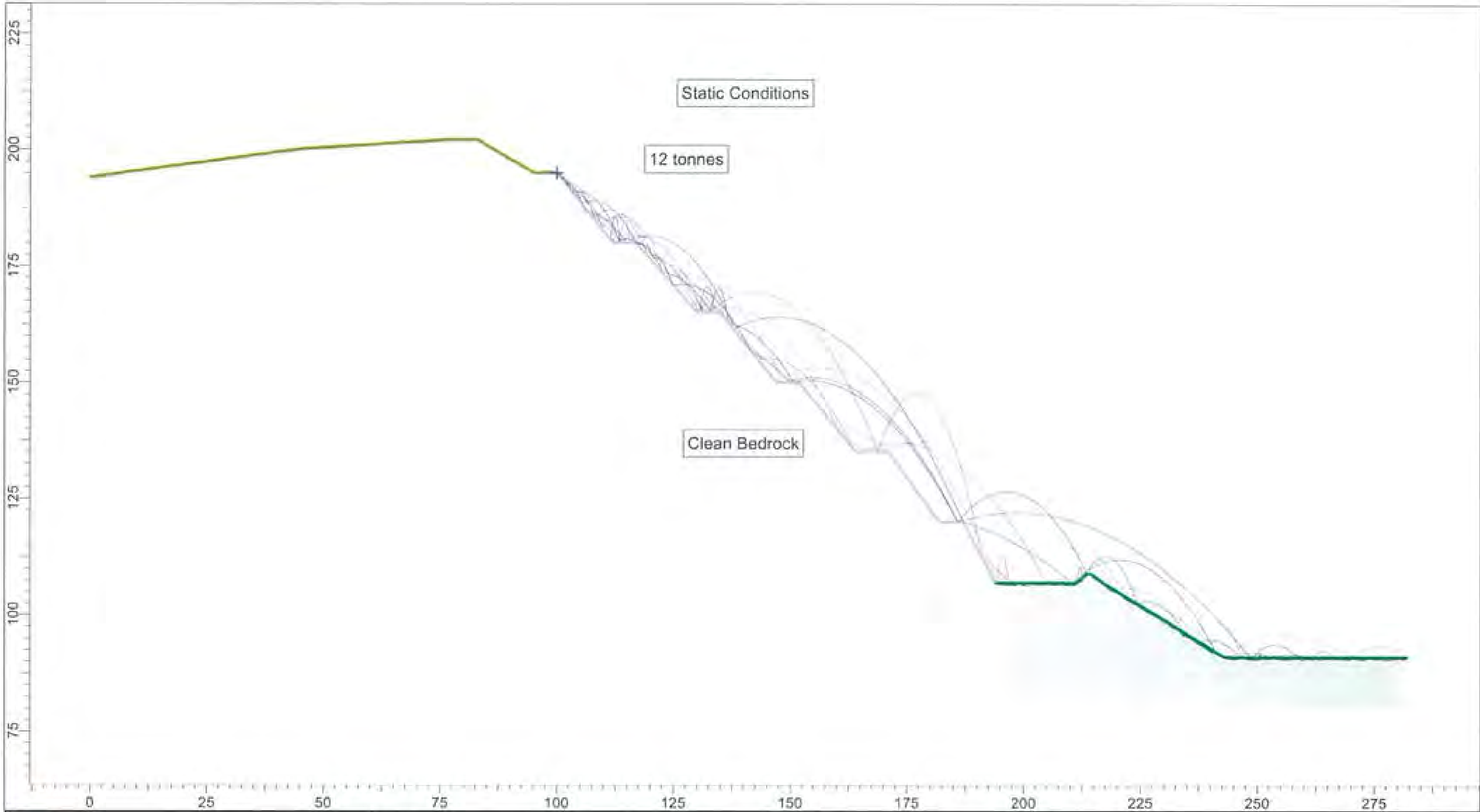

Wainuiomata Hill
y = 0.0642x - 0.838
Peak Horizontal Velocity
SLS: 0.081m/s
ULS: 0.277m/s
Peak Horizontal Velocity (mm/s)
Peak Ground Acceleration (mm/s²)
0
10
20
30
40
50
60
0
100
200
300
400
500
600
700
800


Makara Bunker
y = 9E-05x² + 0.0191x + 0.4092
Peak Horizontal Velocity
SLS: 0.171m/s
ULS: 1.127m/s
Peak Horizontal Velocity (mm/s)
Peak Ground Acceleration (mm/s²)
0
20
40
60
80
100
120
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
950
1000

ANALYTICAL SECTIONS FOR

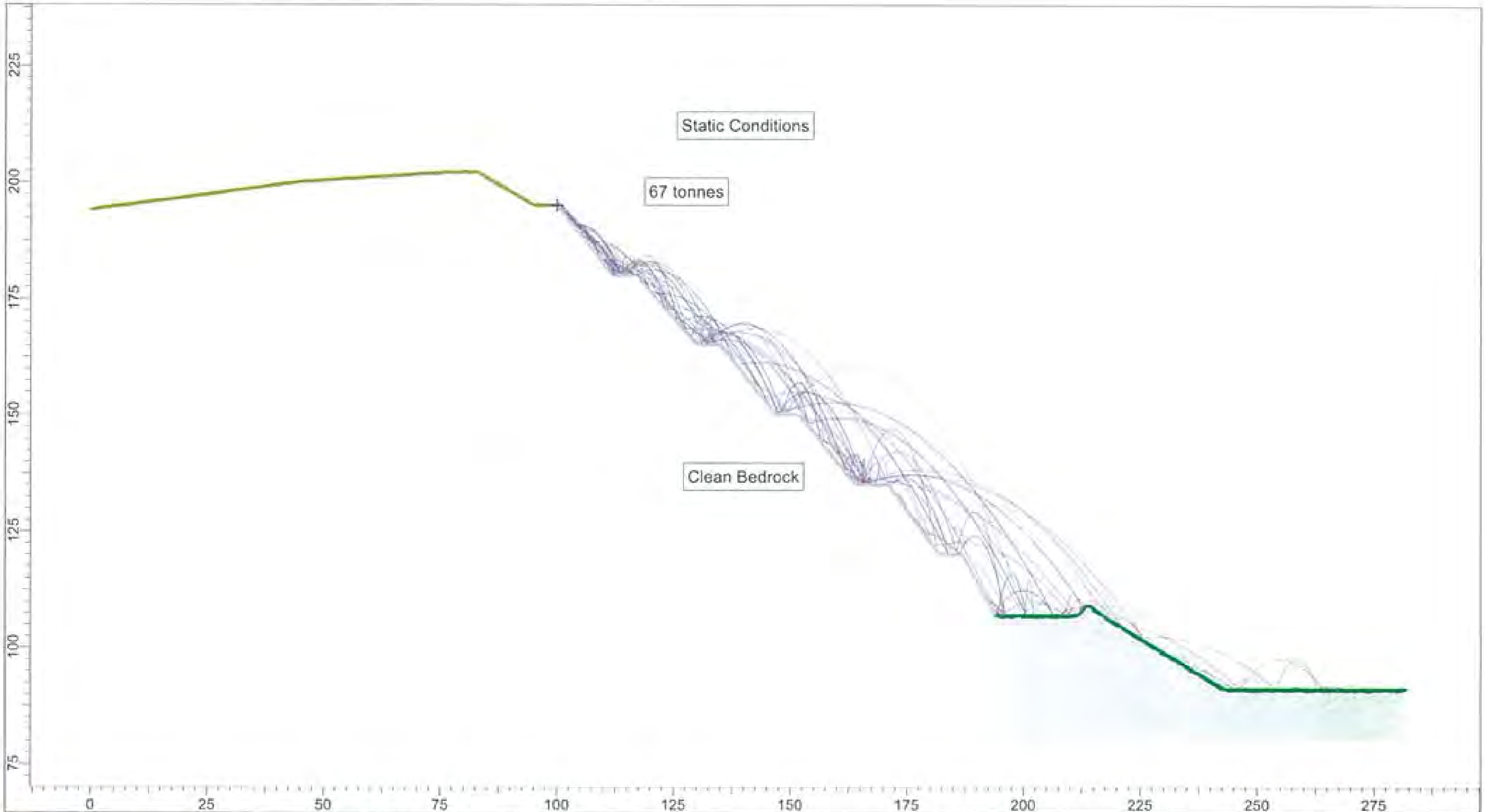
KIWI POINT QUARRY, NORTH FACE.

Static Conditions



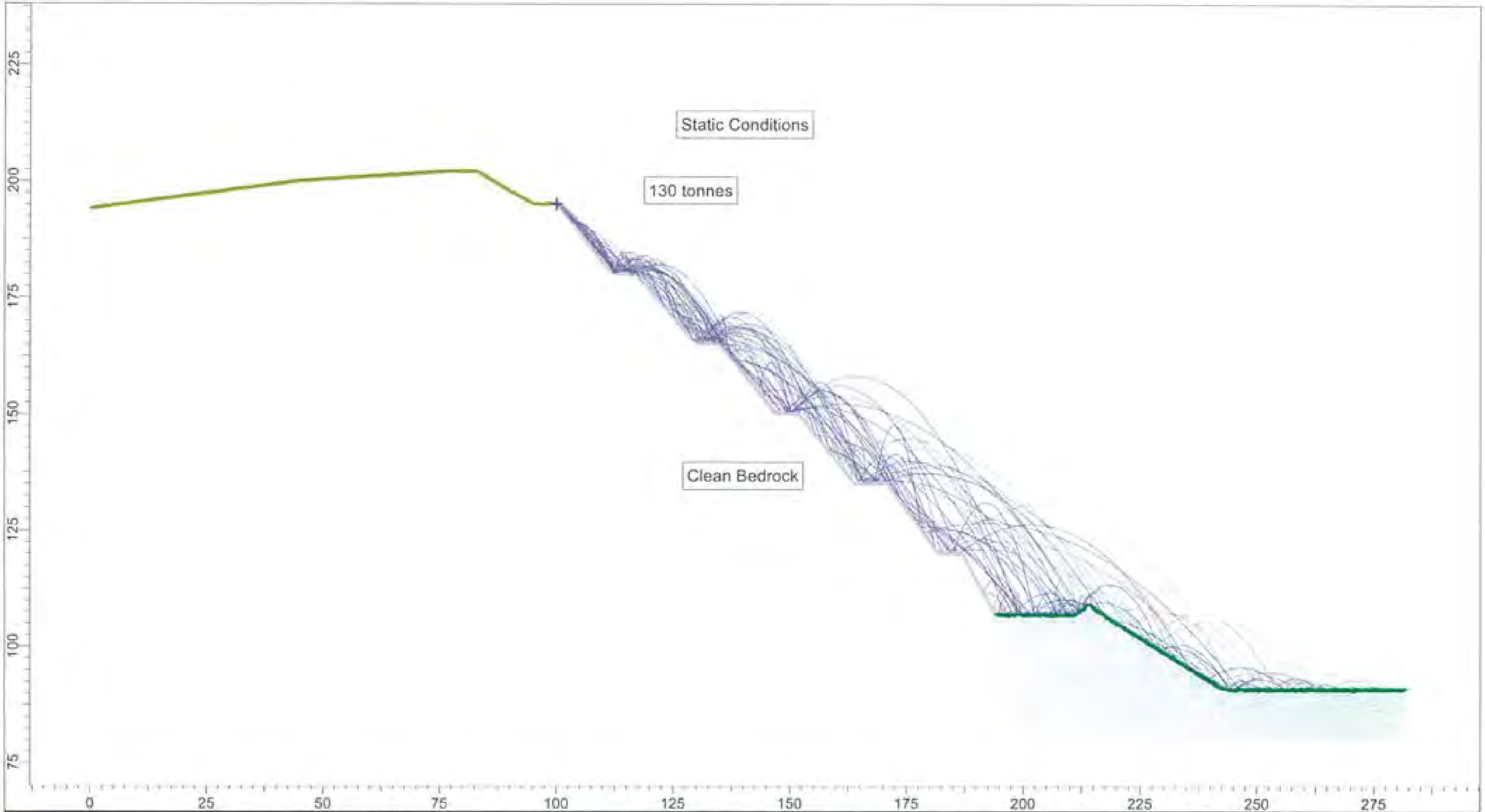
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	2a Clean Bedrock, 12t, static.fal5

ROCFALL 5.013



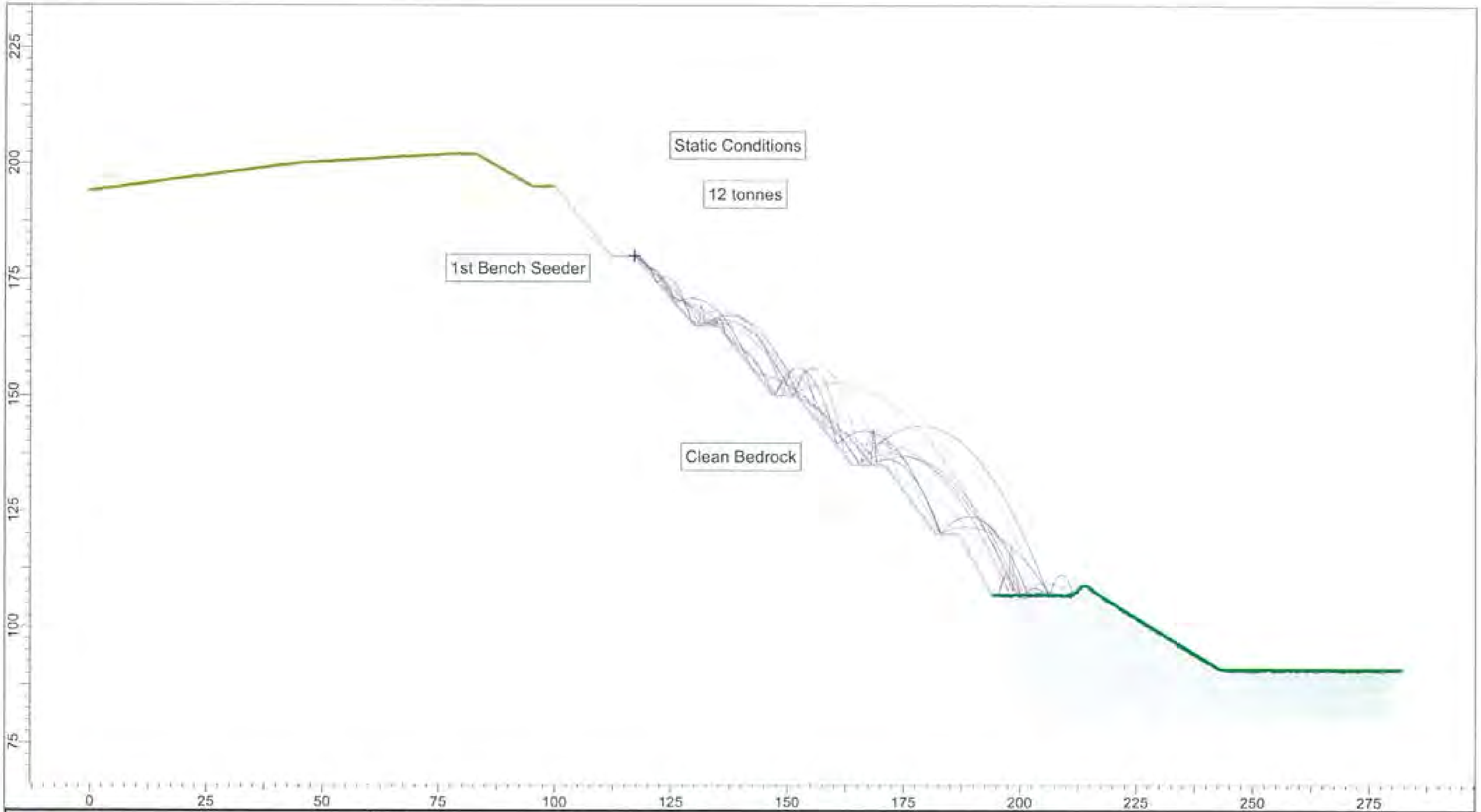
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	2b Clean Bedrock, 67t, static.fal5

ROCFALL 5.013



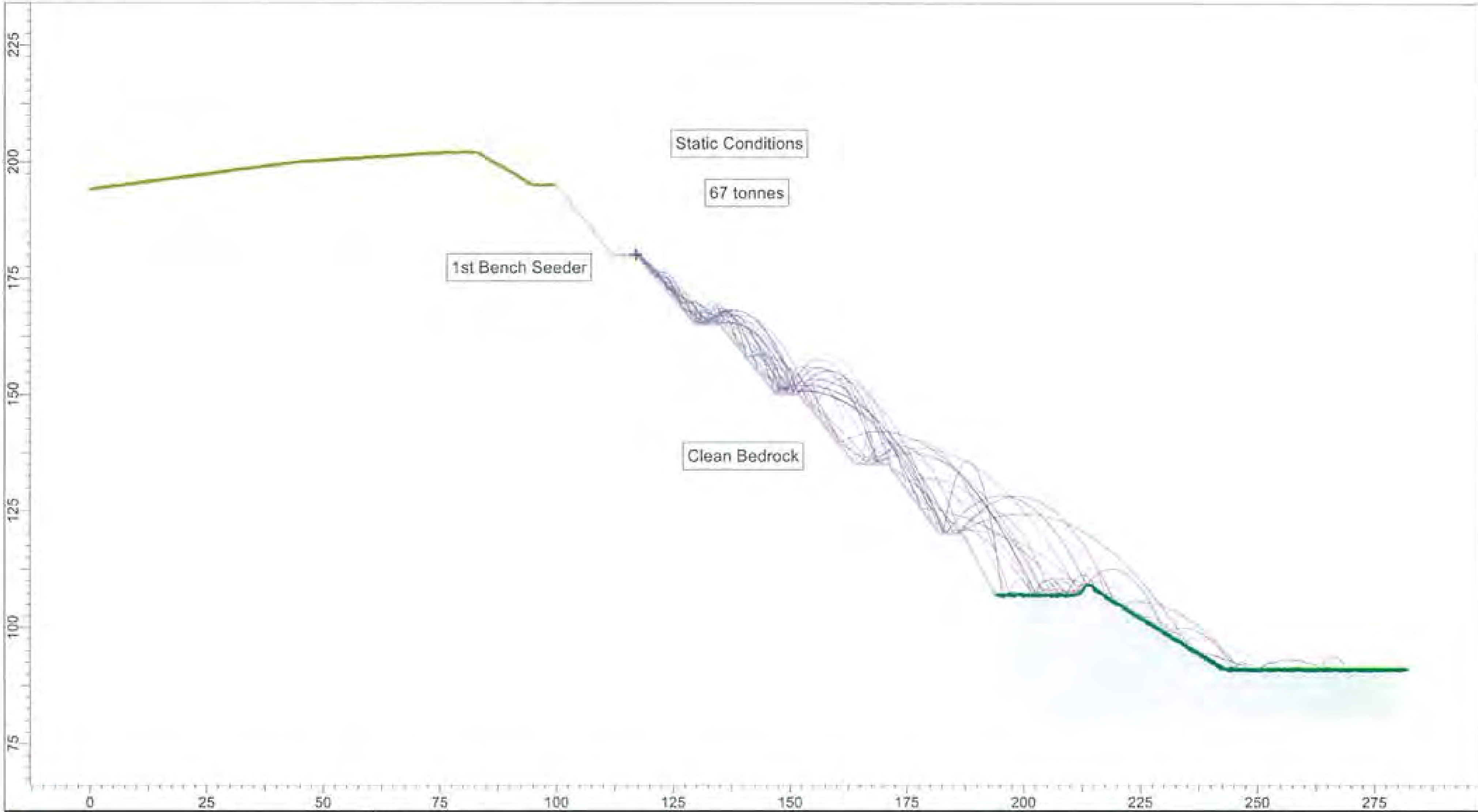
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	2c Clean Bedrock, 130t, static.fal5

ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	3a Clean Bedrock, 12t, static, first bench.fal5

ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	3b Clean Bedrock, 67t, static, first bench.fal5

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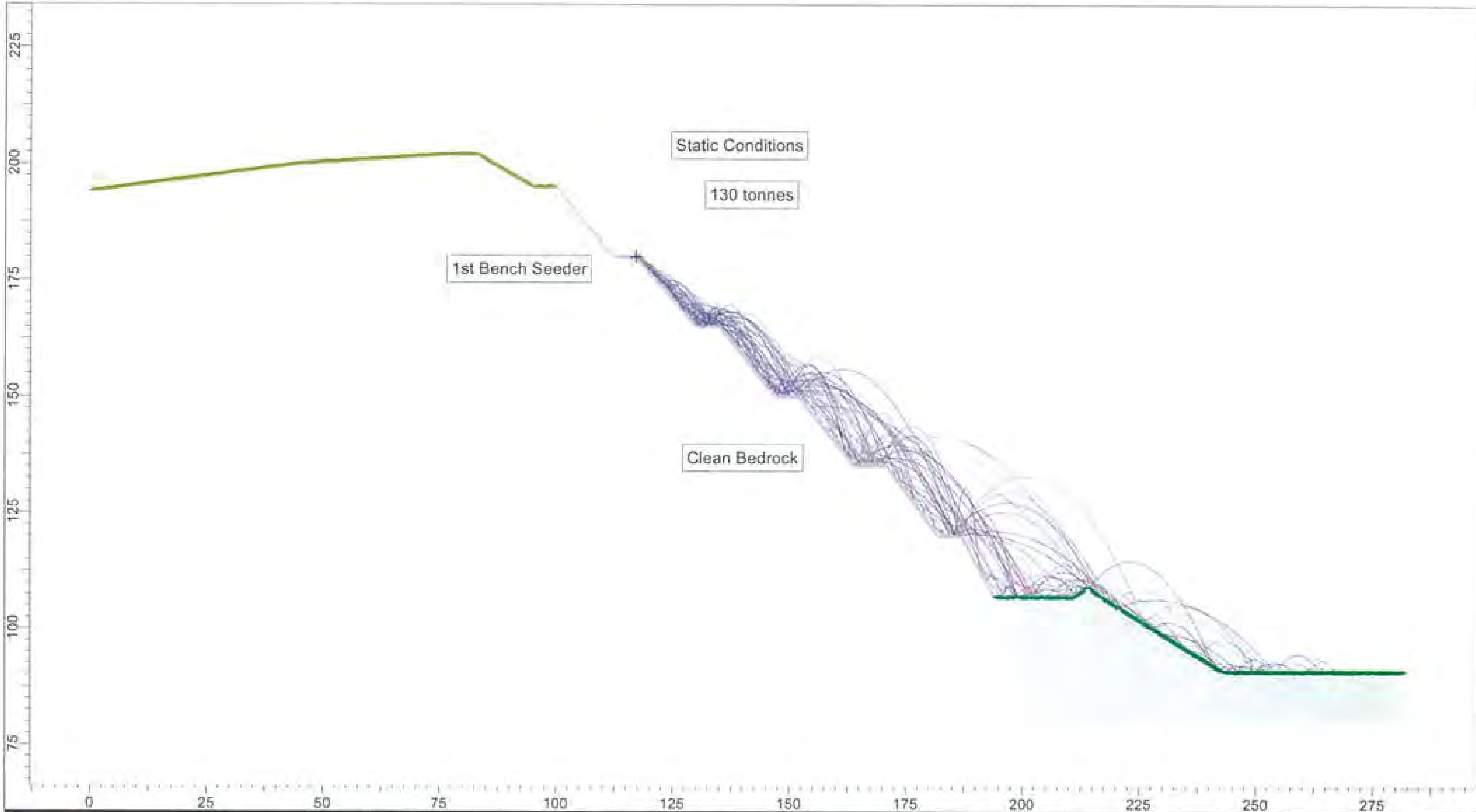
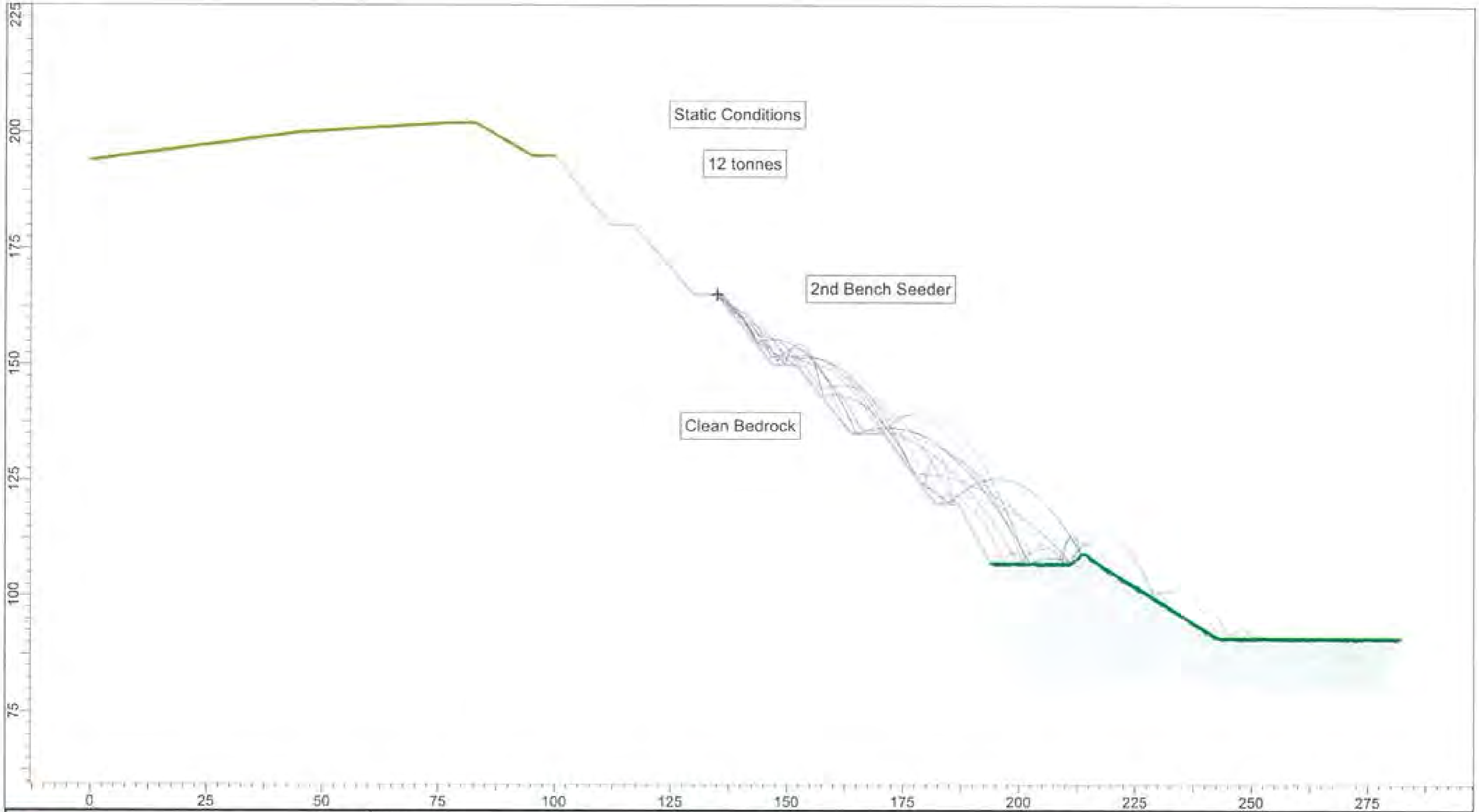


Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	3c Clean Bedrock, 130t, static, first bench.fal5

ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	4a Clean Bedrock, 12t, static, second bench.fal5

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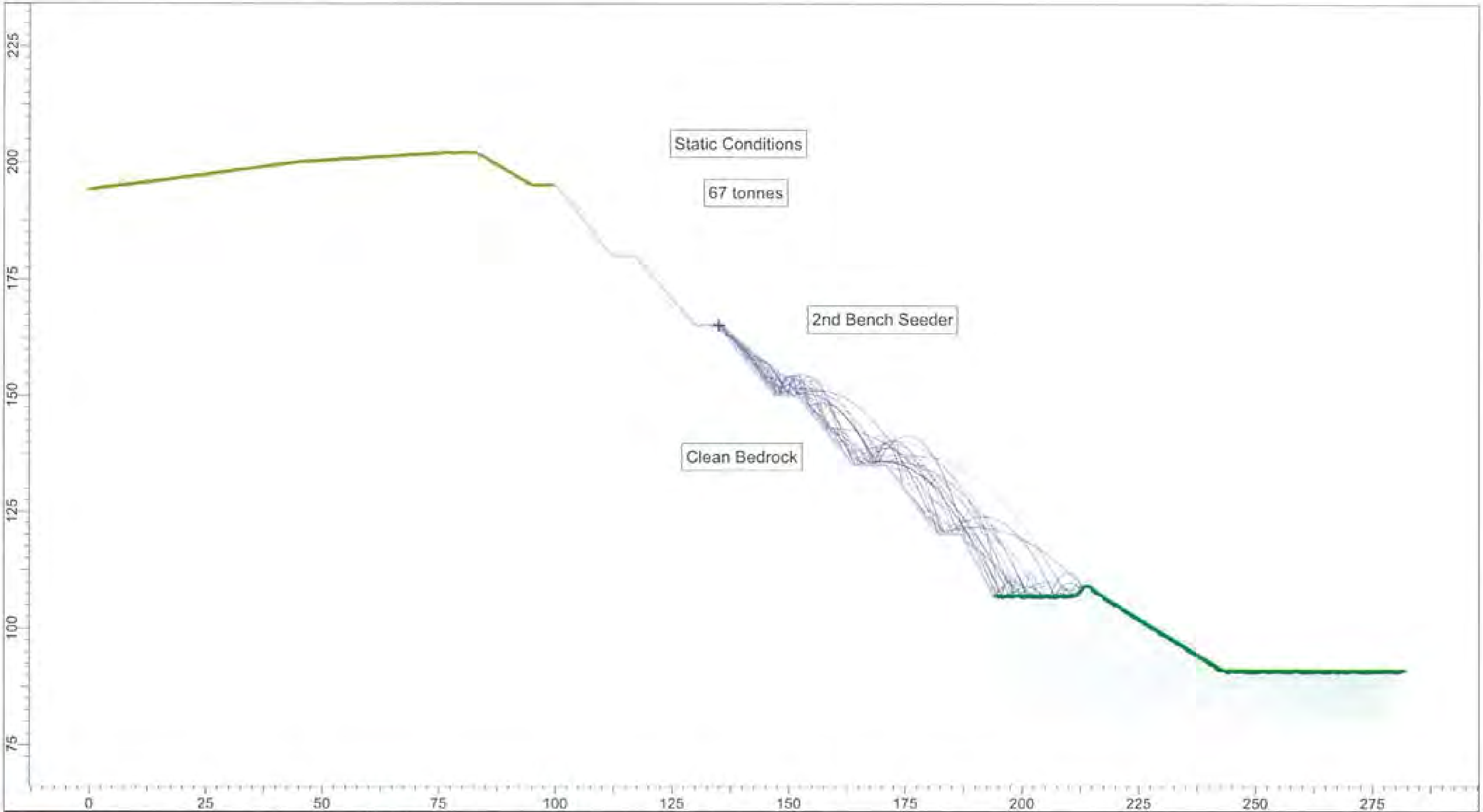

Project	Kiwi Point Quarry, Wellington
Analysis Description	Rock Fall Analysis
Drawn By	S Carryer & A Fell
Company	Ormiston Associates Ltd
Date	24/6/2015
File Name	4b Clean Bedrock, 67t, static, second bench.fal5

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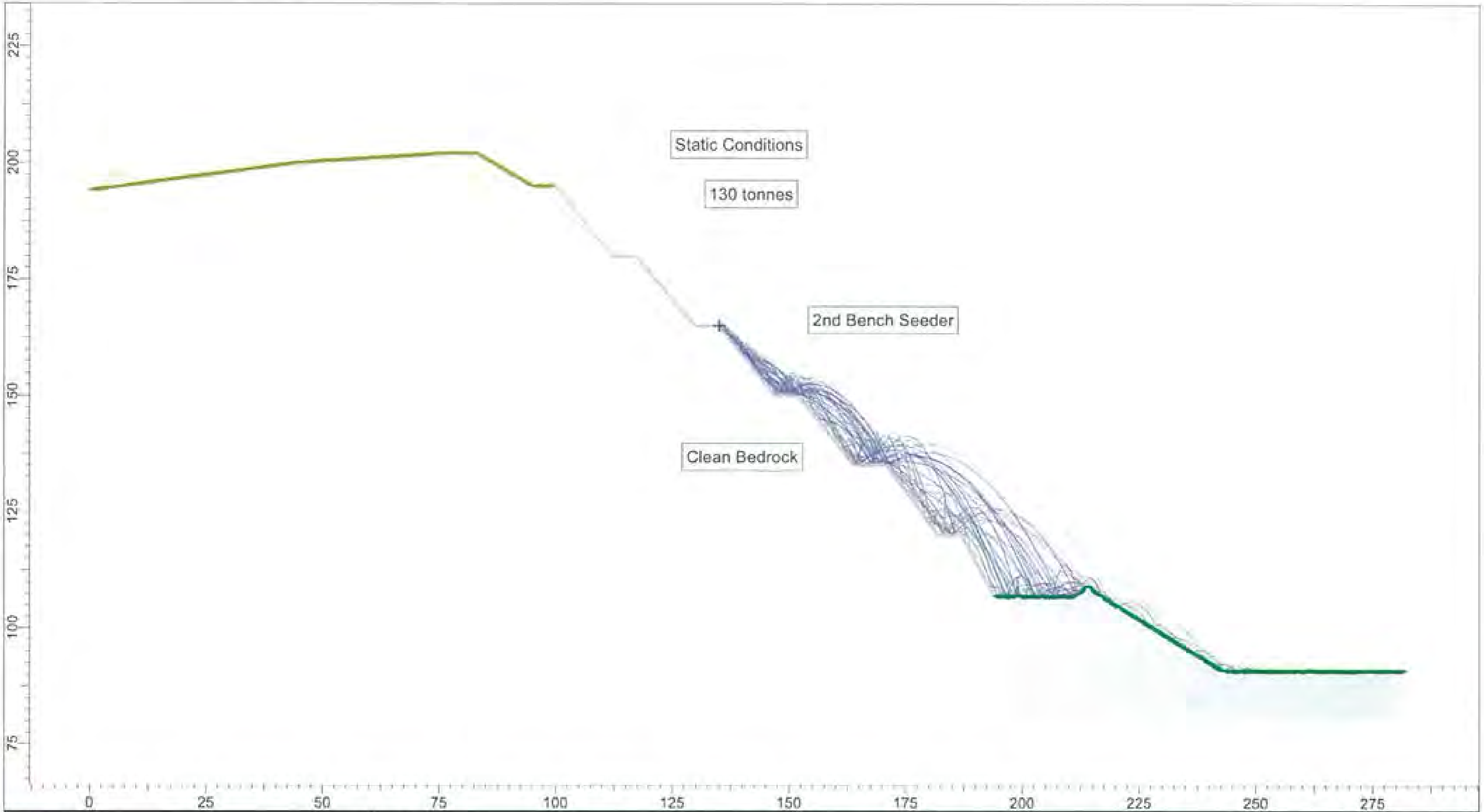

Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	4c Clean Bedrock, 130t, static, second bench.fal5

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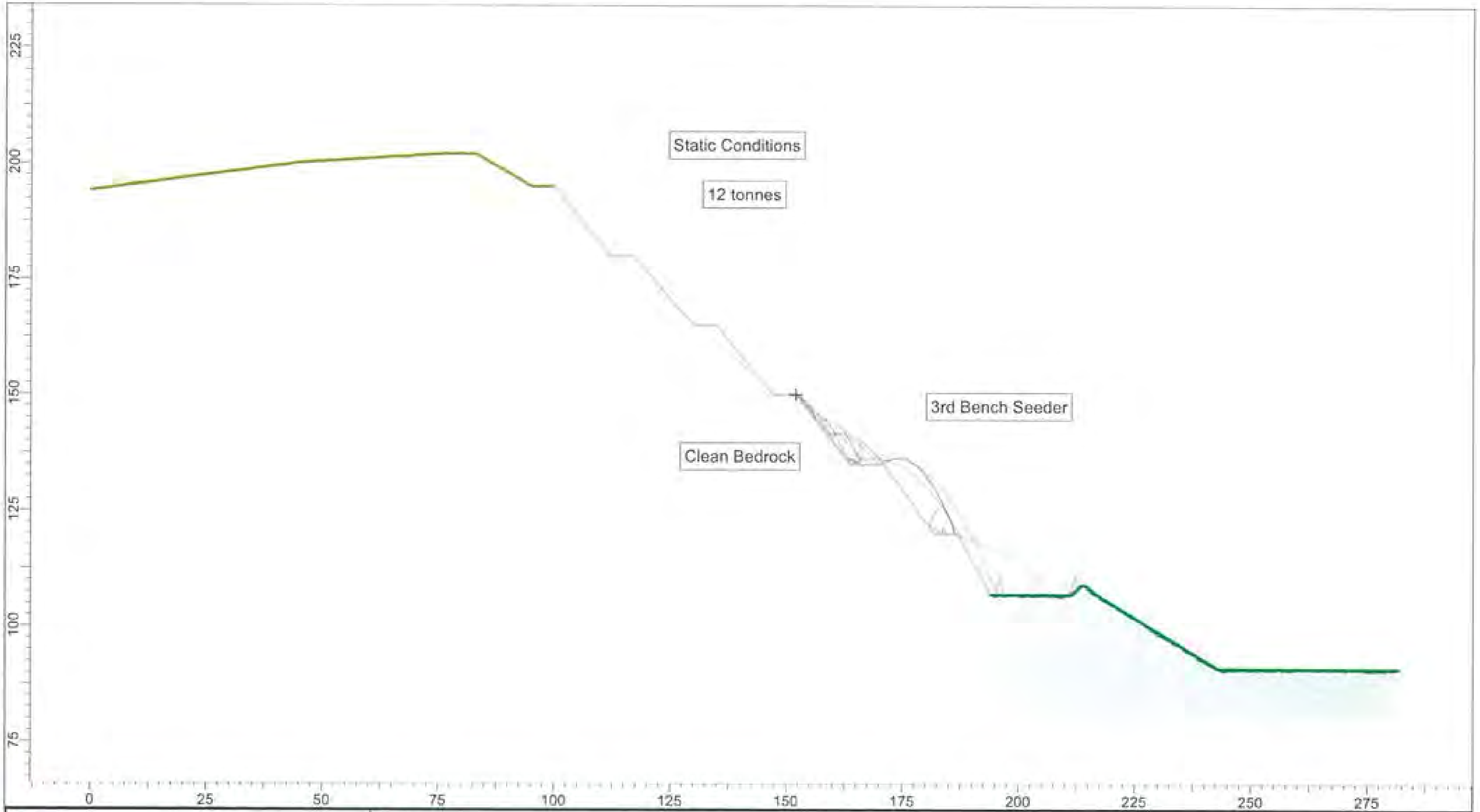

Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	5a Clean Bedrock, 12t, static, third bench.fal5

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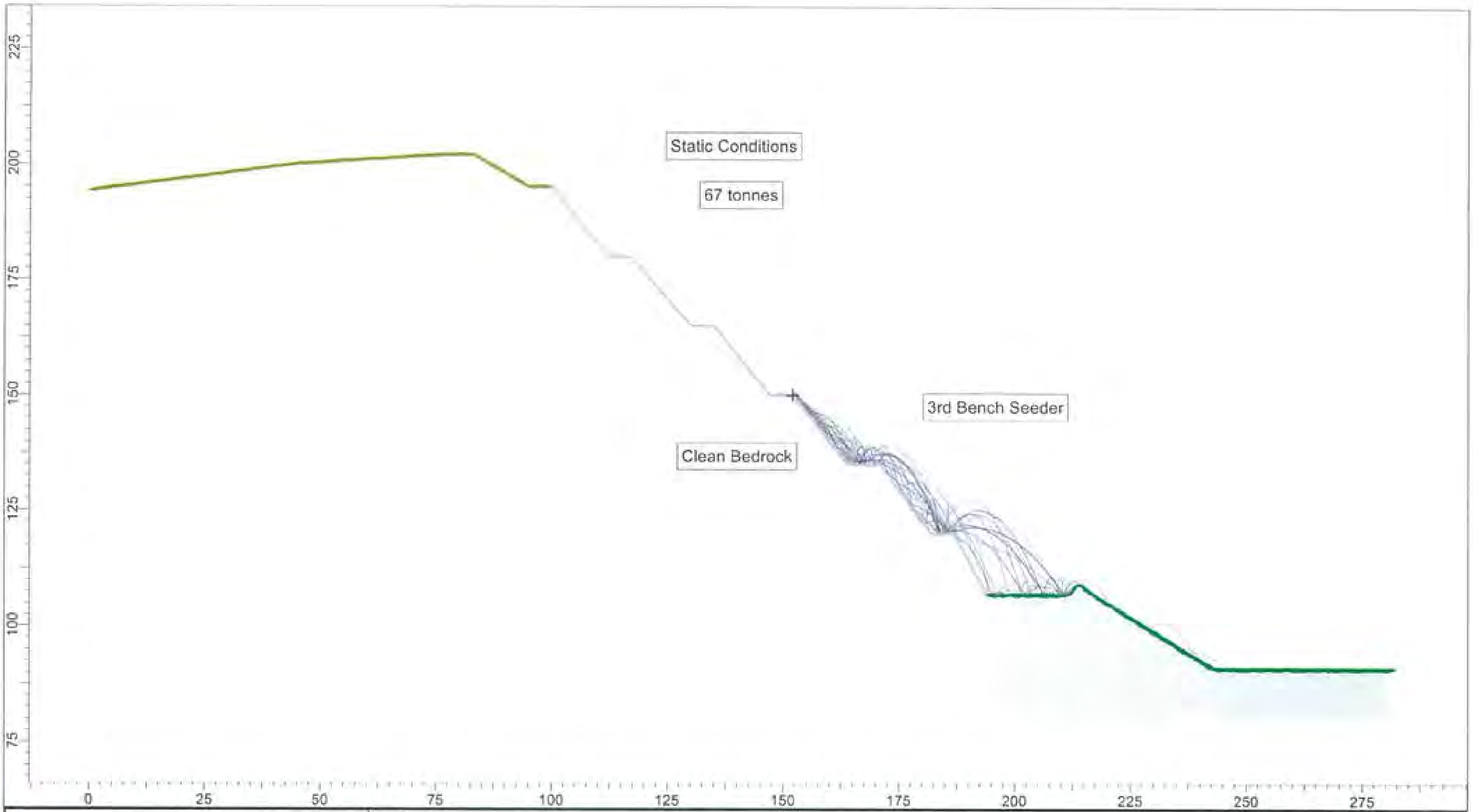

Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	5b Clean Bedrock, 67t, static, third bench.fal5

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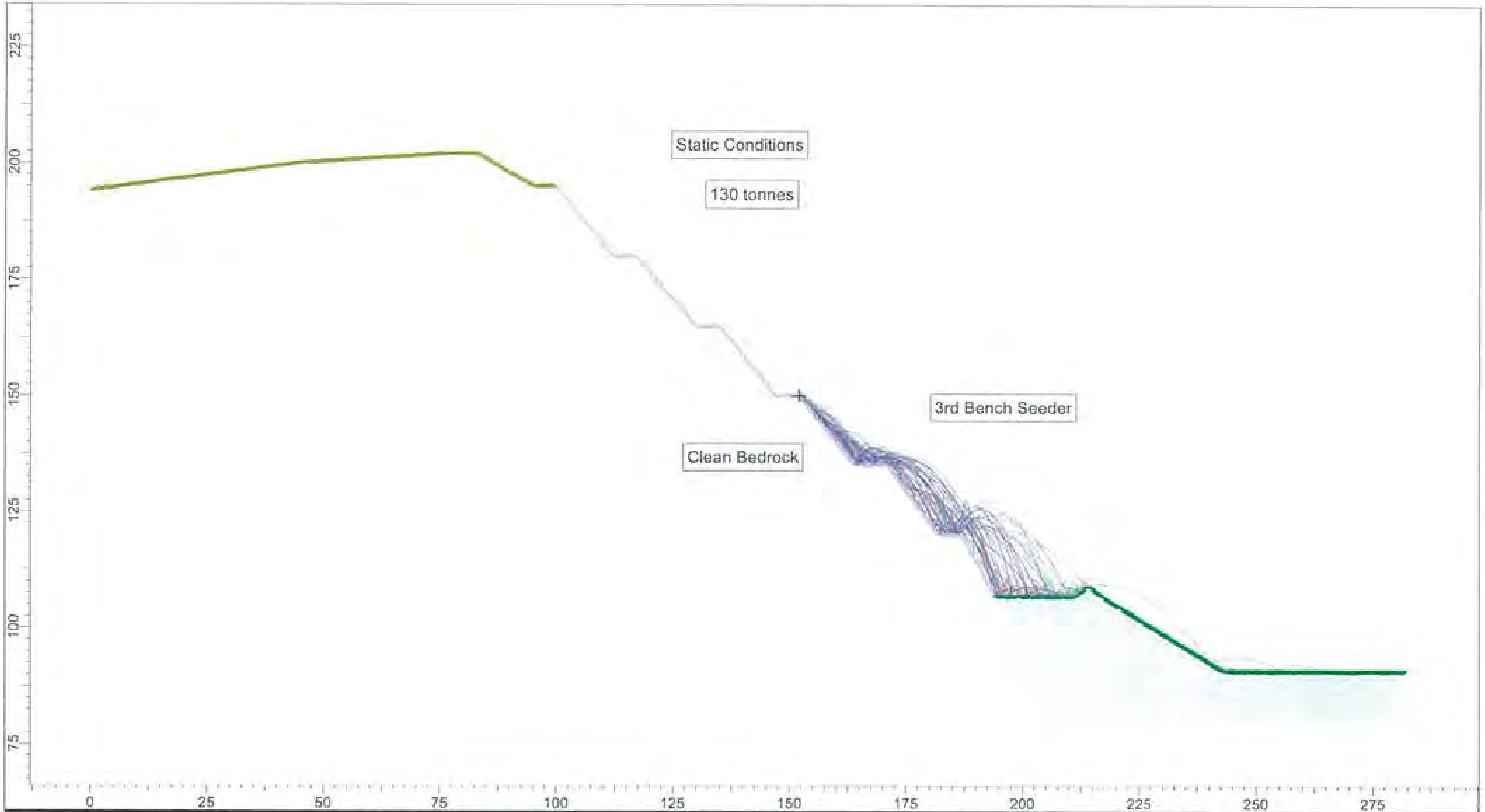
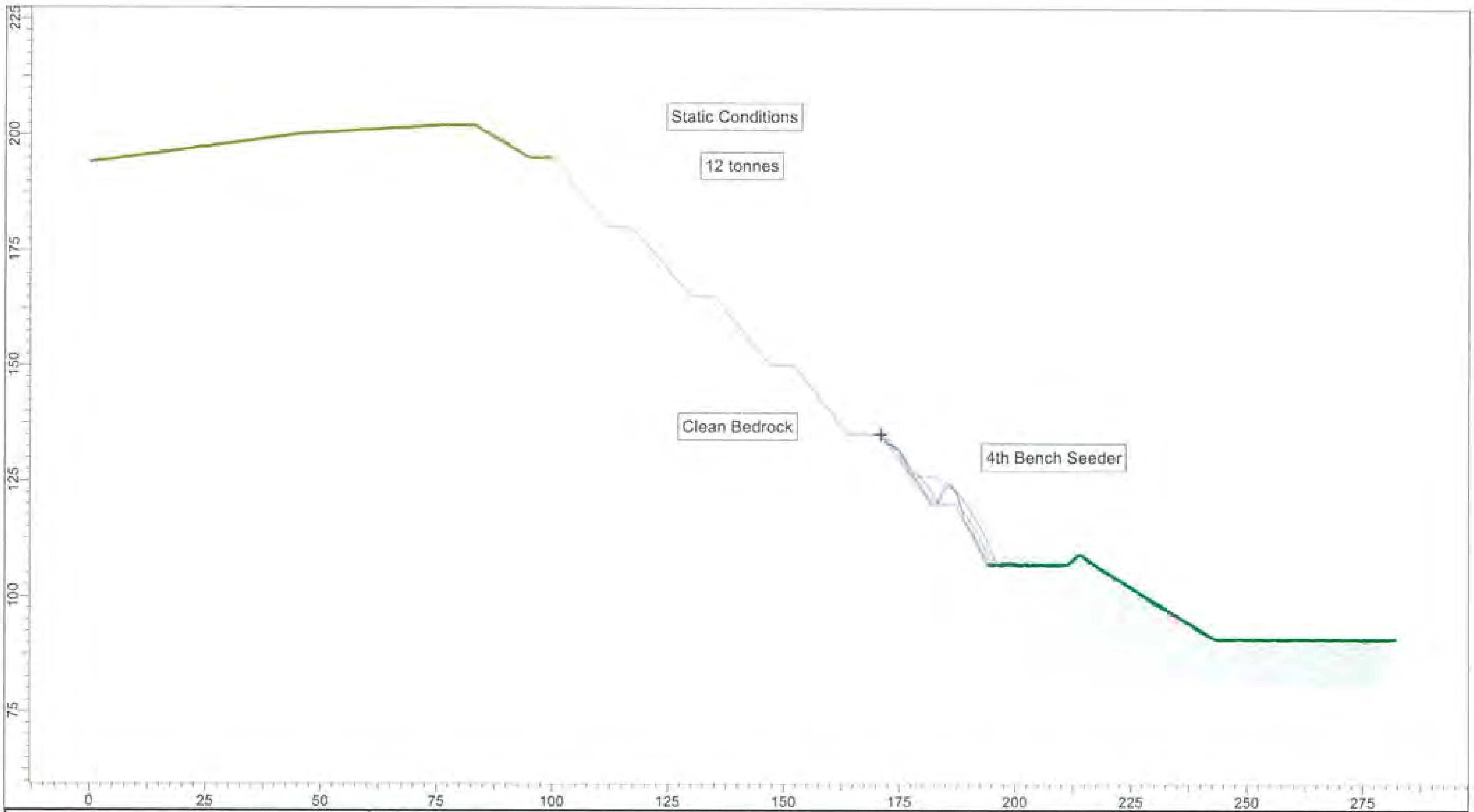
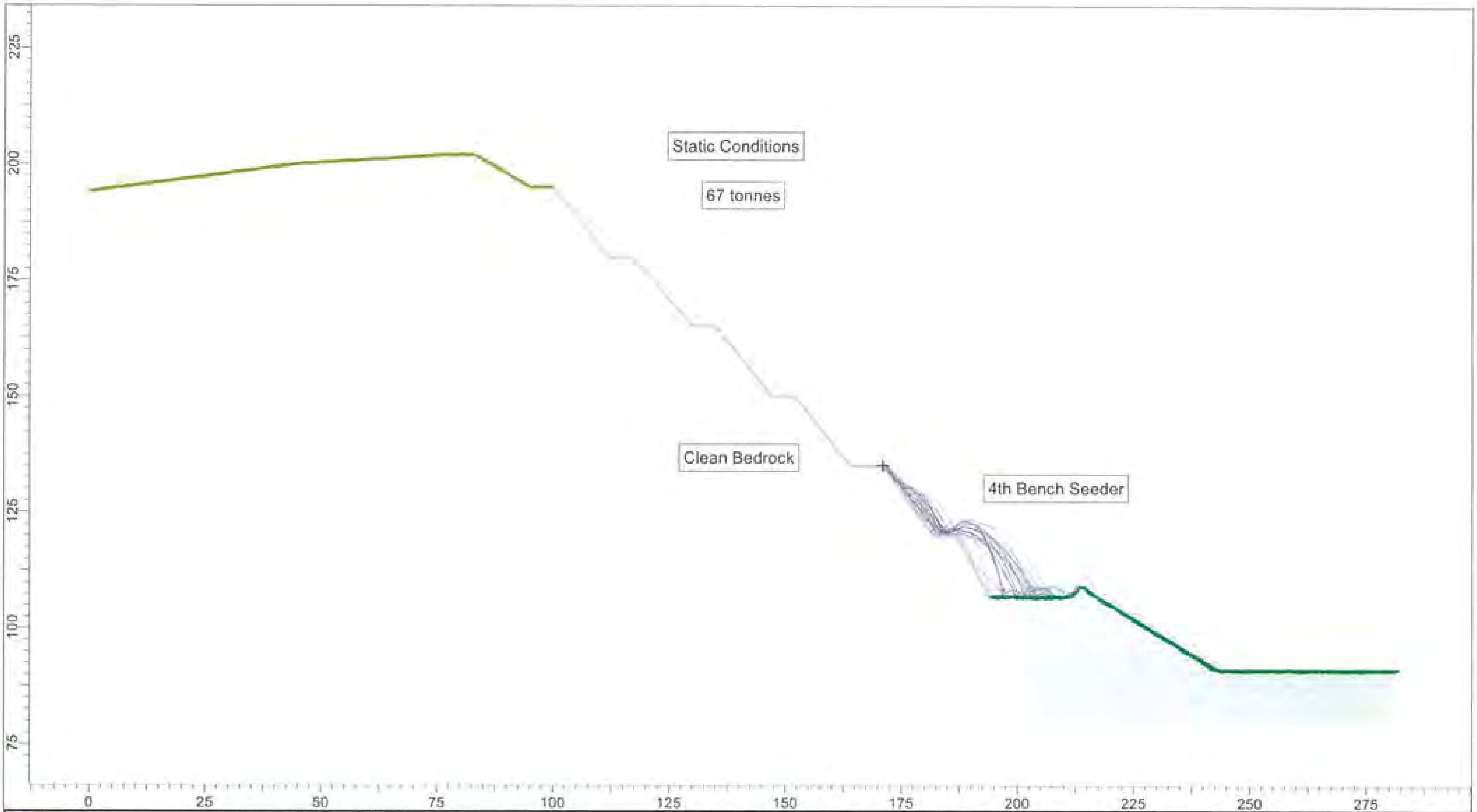


rocscience	Project	Kiwi Point Quarry, Wellington	
	Analysis Description	Rock Fall Analysis	
	Drawn By	S Carryer & A Fell	Company: Ormiston Associates Ltd
ROCFALL 5.013	Date	24/6/2015	File Name: 5c Clean Bedrock, 130t, static, third bench.fal5



Project	Kiwi Point Quarry, Wellington
Analysis Description	Rock Fall Analysis
Drawn By	S Carryer & A Fell
Company	Ormiston Associates Ltd
Date	24/6/2015
File Name	6a Clean Bedrock, 12t, static, fourth bench.fal5

ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	6b Clean Bedrock, 67t, static, fourth bench.fal5

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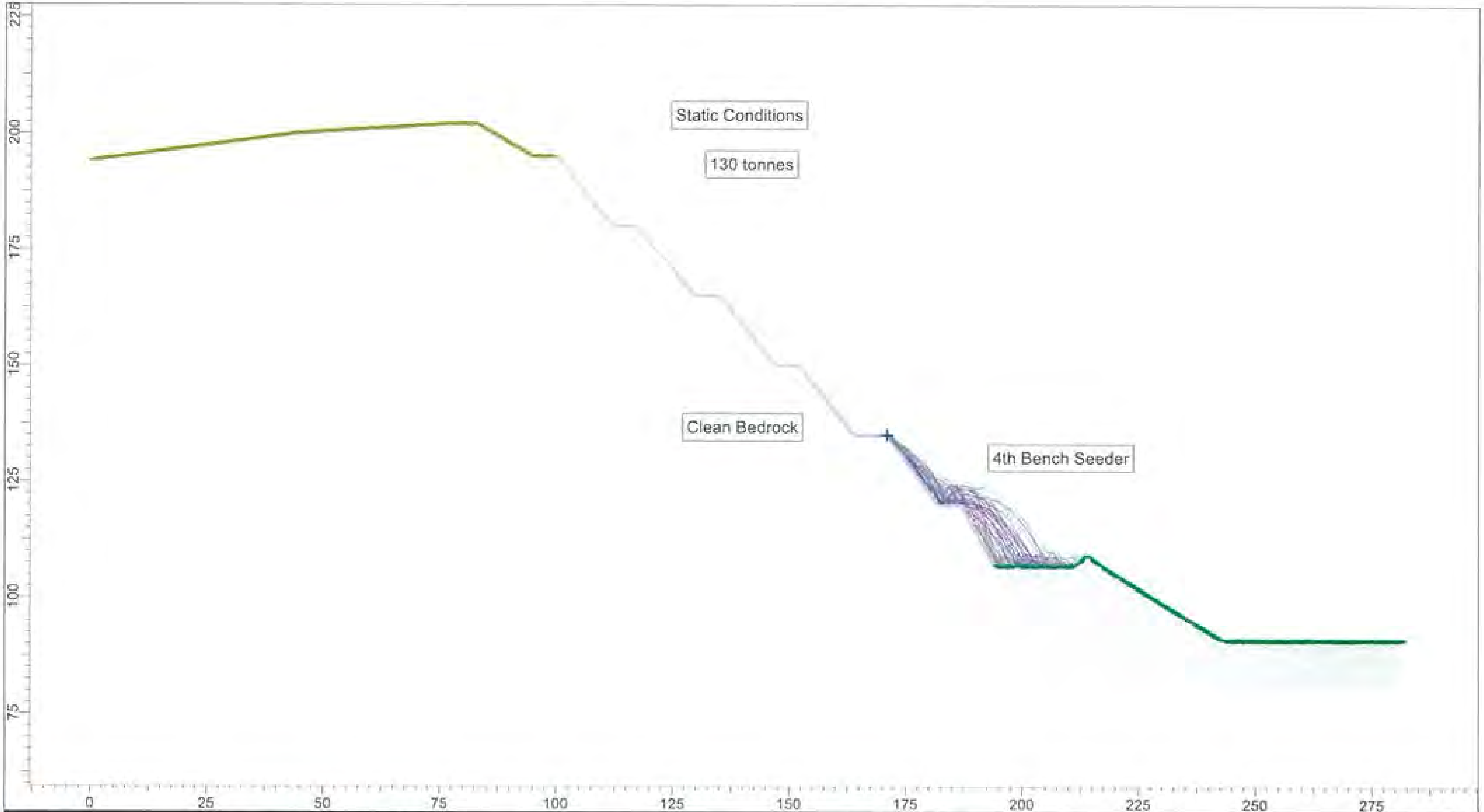

Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	6c Clean Bedrock, 130t, static, fourth bench.fal5

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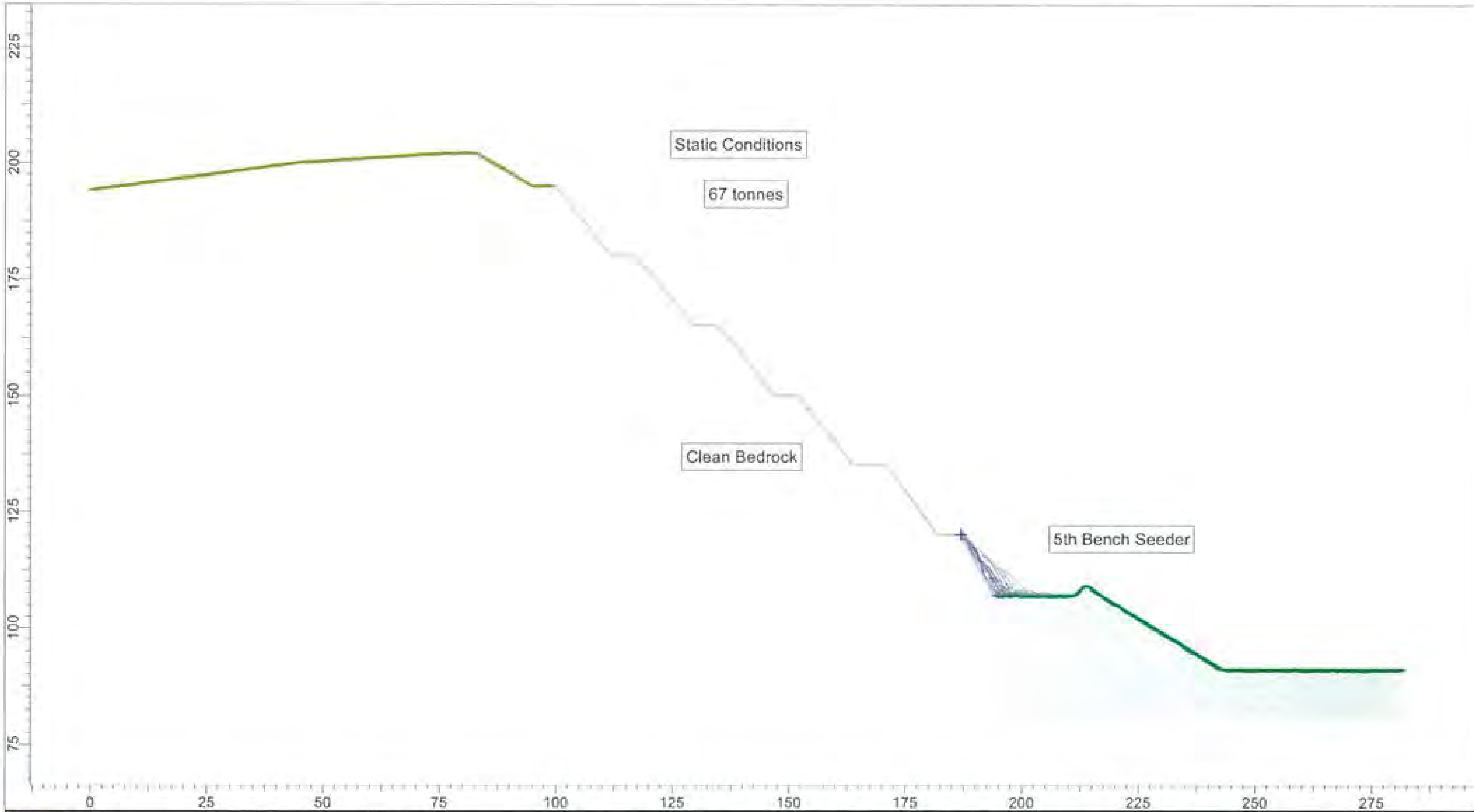
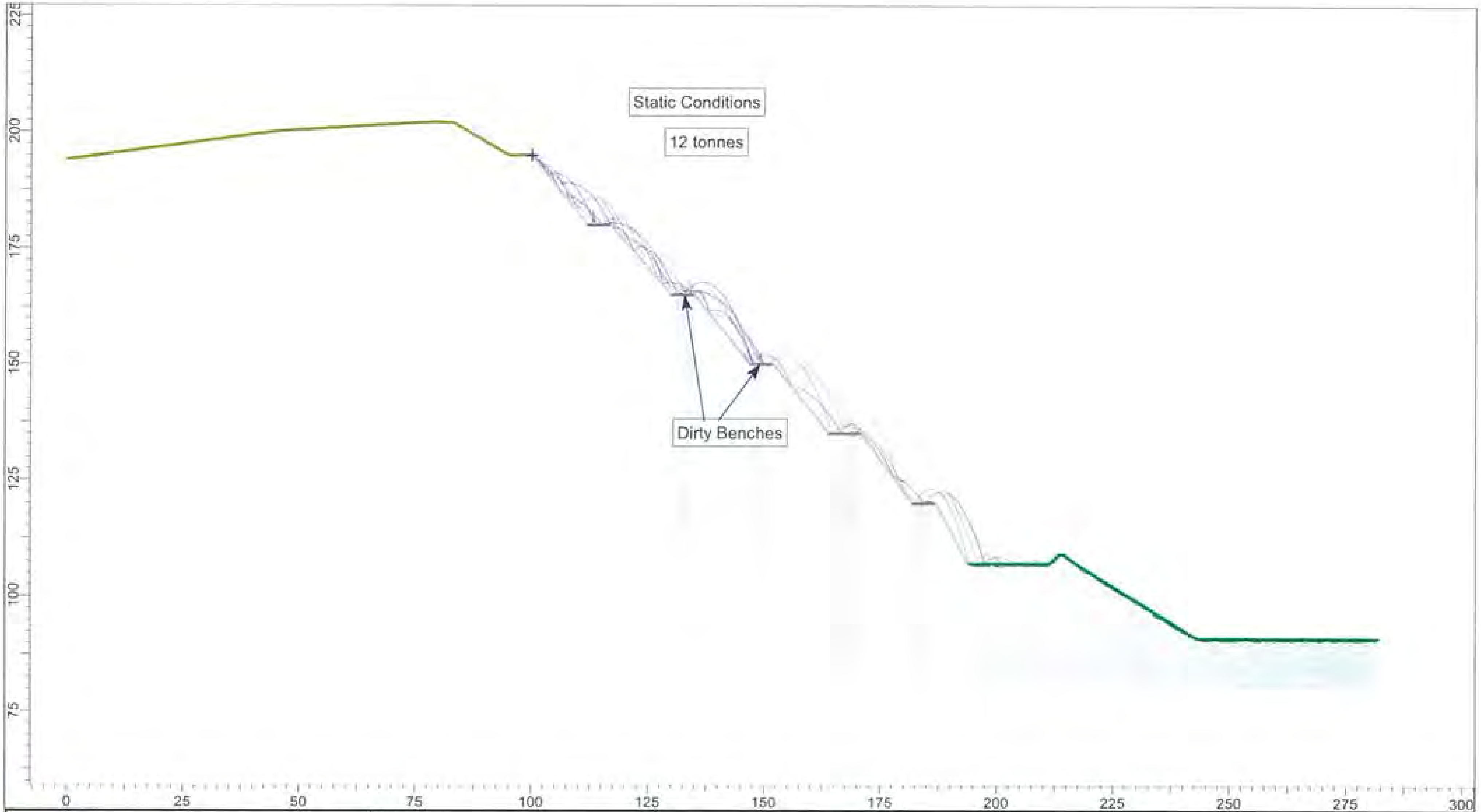
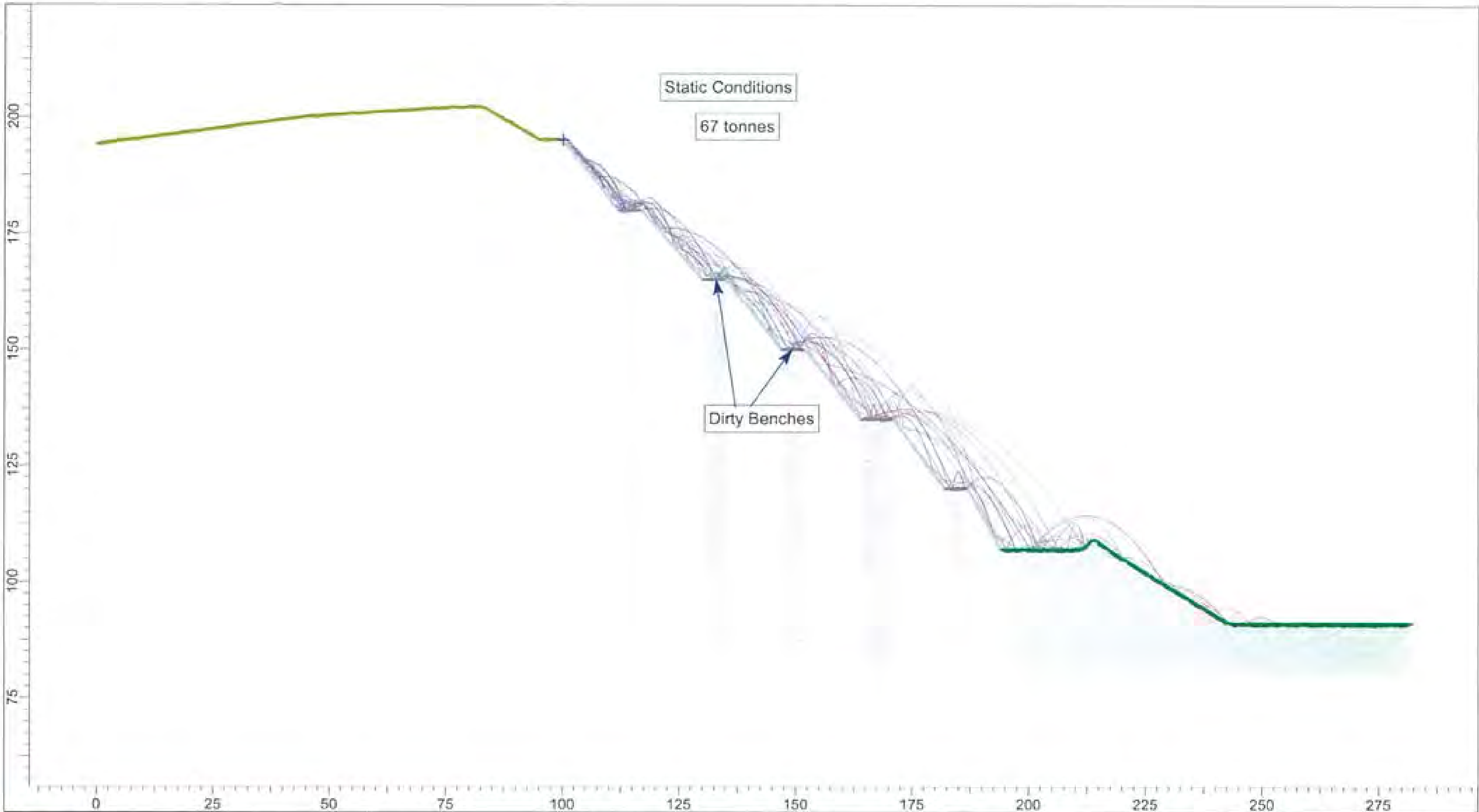


Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	7a Clean Bedrock, 67t, static, fifth bench.fal5

ROCFALL 5.013

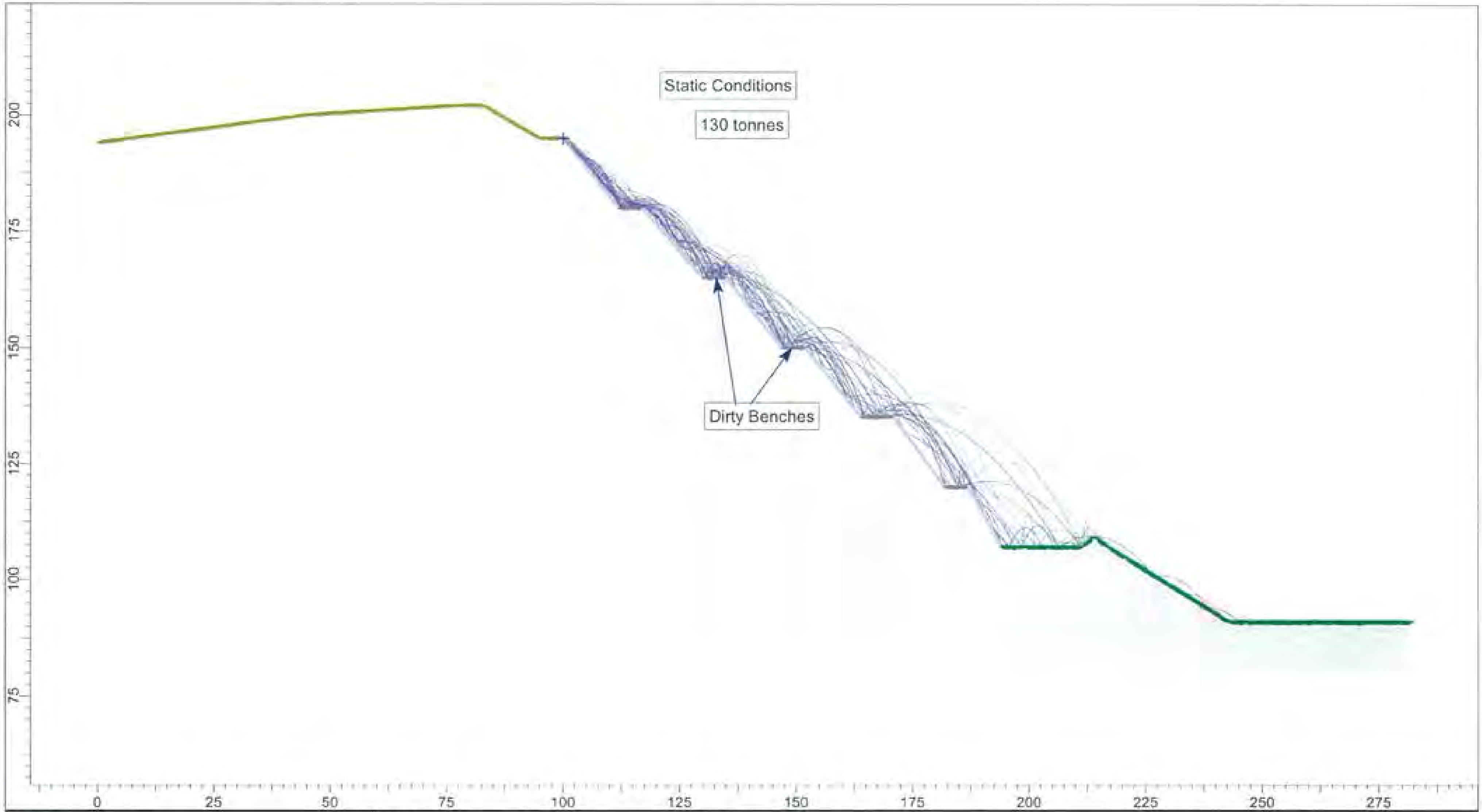


rocscience	Project	Kiwi Point Quarry, Wellington
	Analysis Description	Rock Fall Analysis
	Drawn By: S Carryer & A Fell	Company: Ormiston Associates Ltd
ROCFALL 5.013	Date: 24/6/15	File Name: 8a Dirty Bench, static, 12t.fal5



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/15	File Name	8b Dirty Bench, static, 67t.fal5

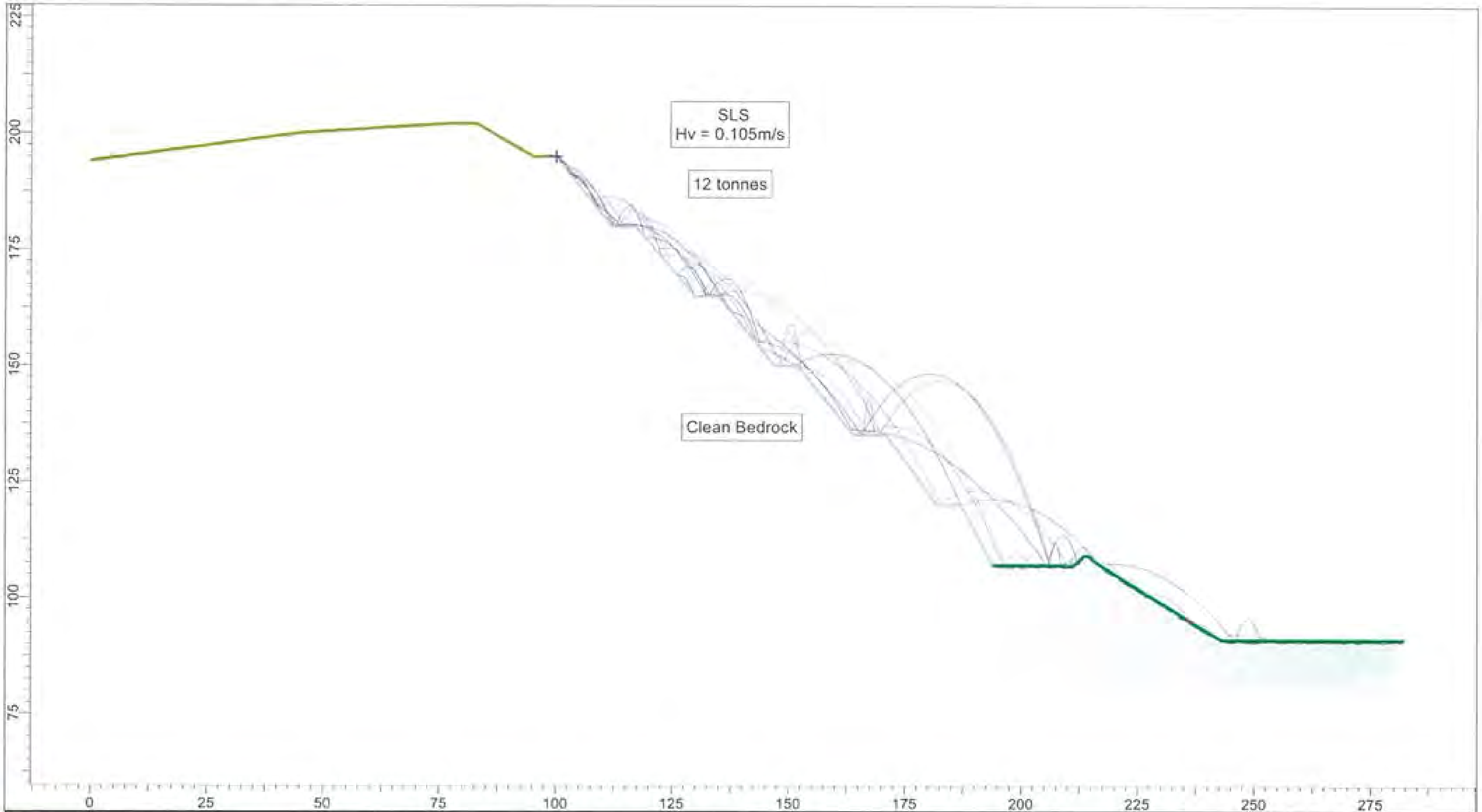
ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/15	File Name	8c Dirty Bench, static, 130t.fal5

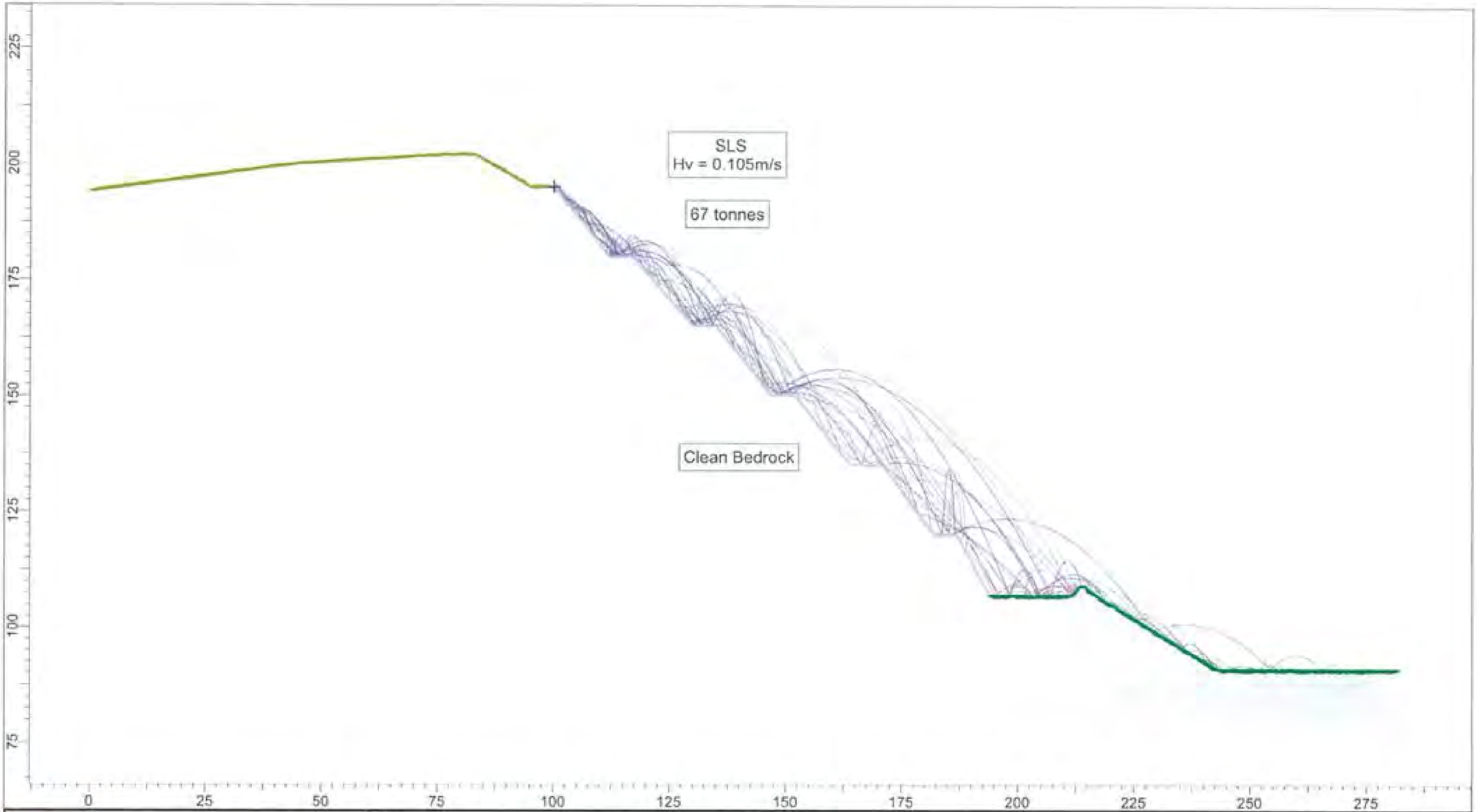
ROCFALL 5.013

Dynamic Conditions (SLS)



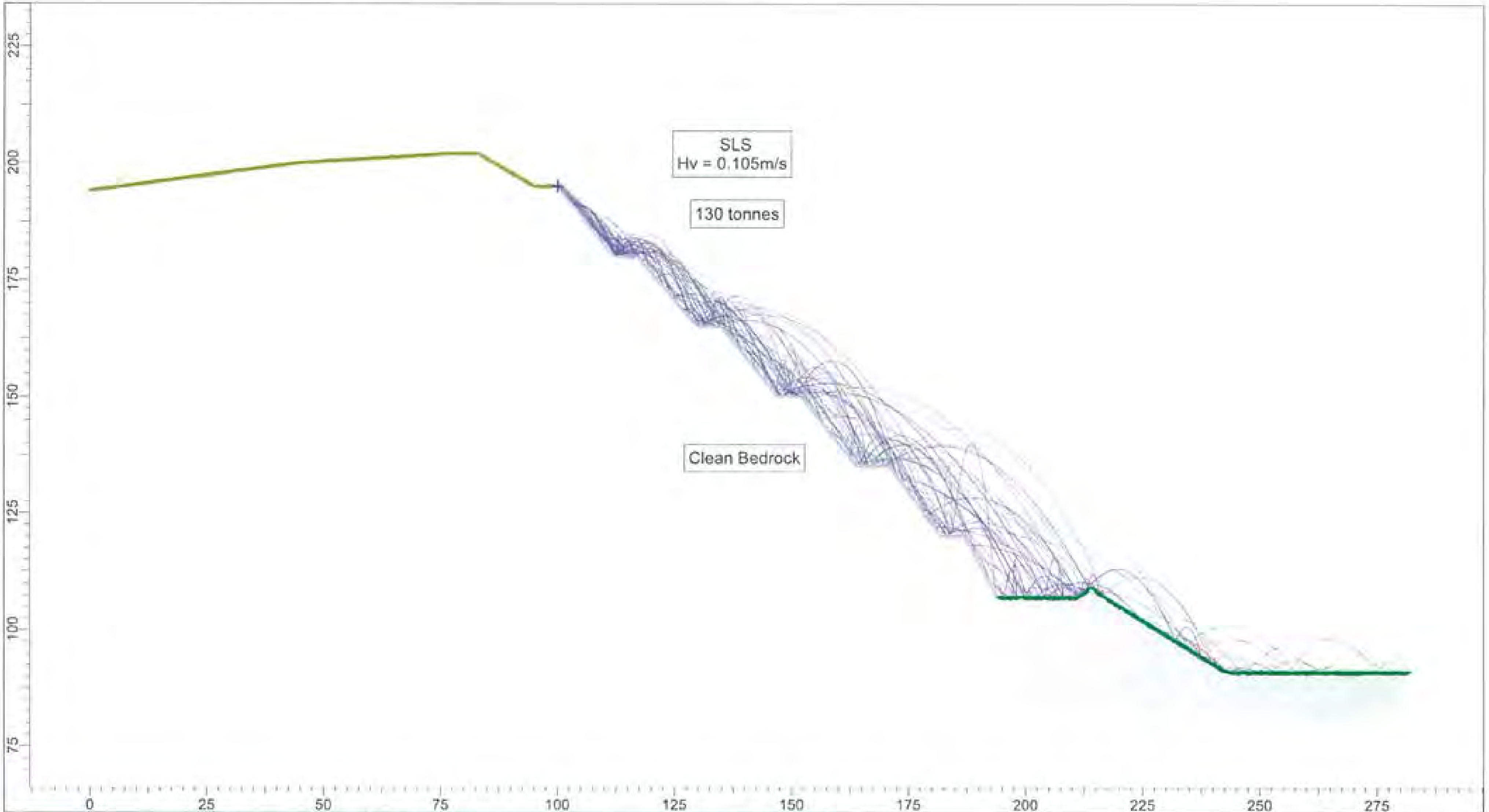
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	9a Clean Bedrock, 12t, SLS.fal5

ROCFALL 5.013



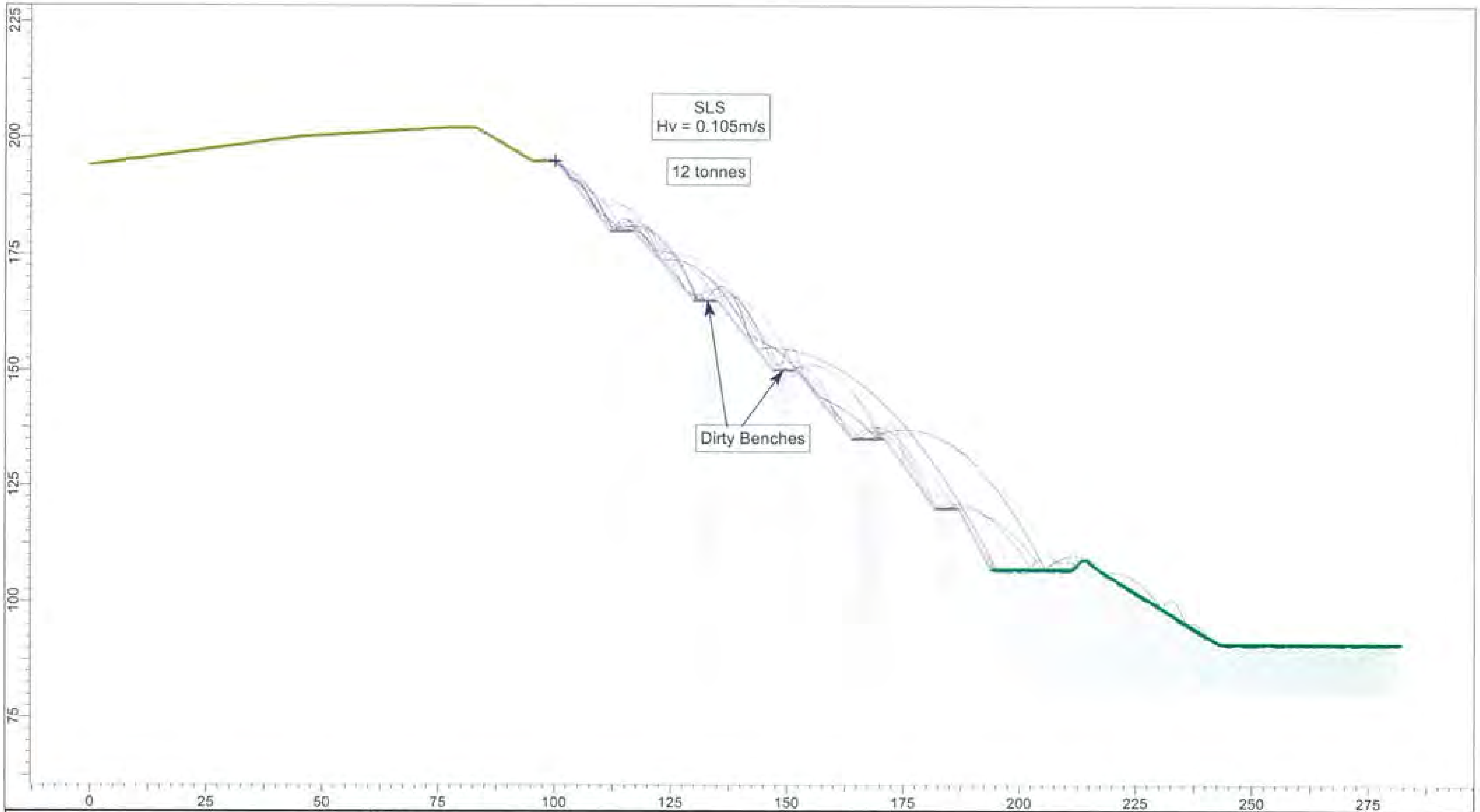
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	9b Clean Bedrock, 67t, SLS.fal5

ROCFALL 5.013



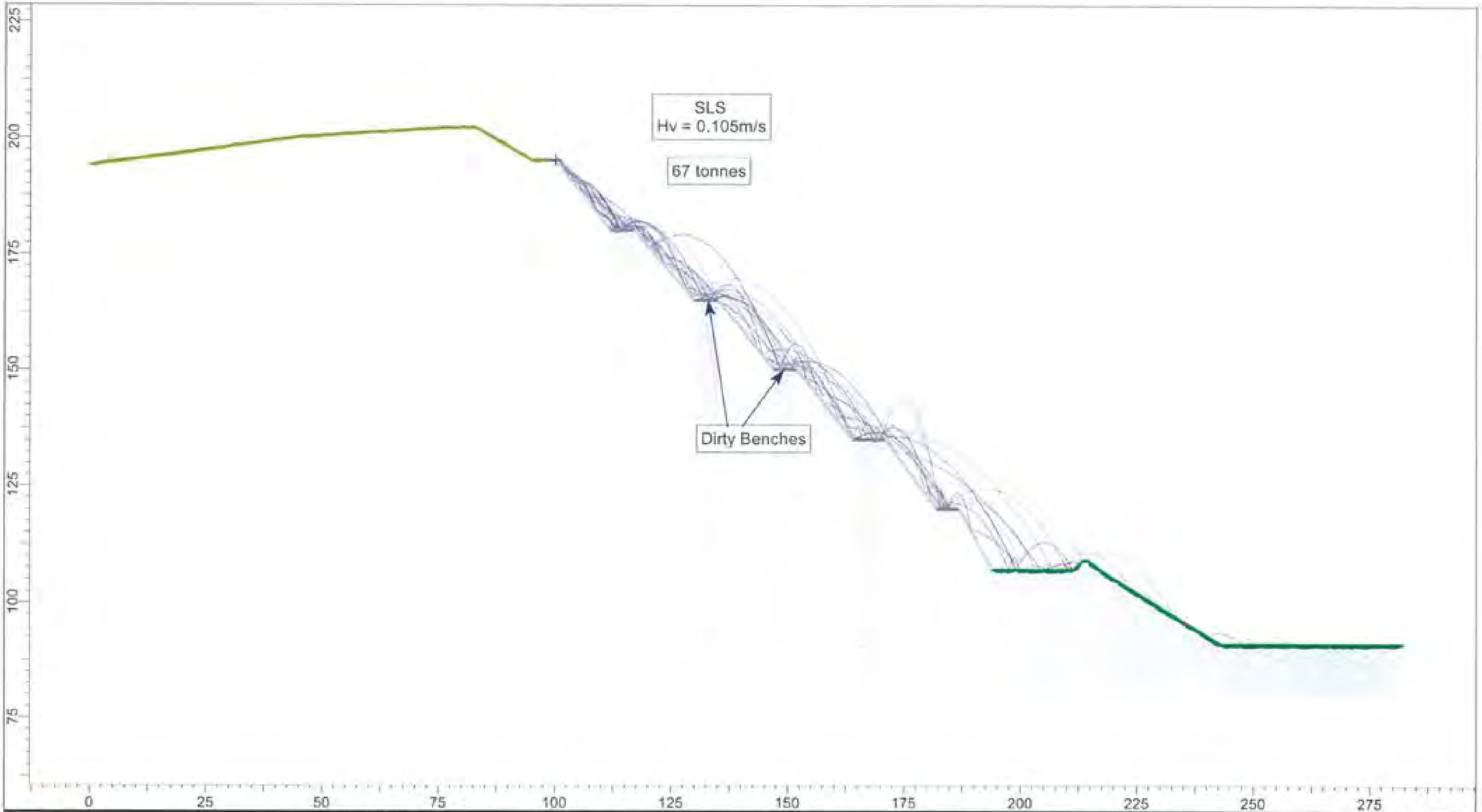
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	9c Clean Bedrock, 130t, SLS.fal5

ROCFALL 5.013

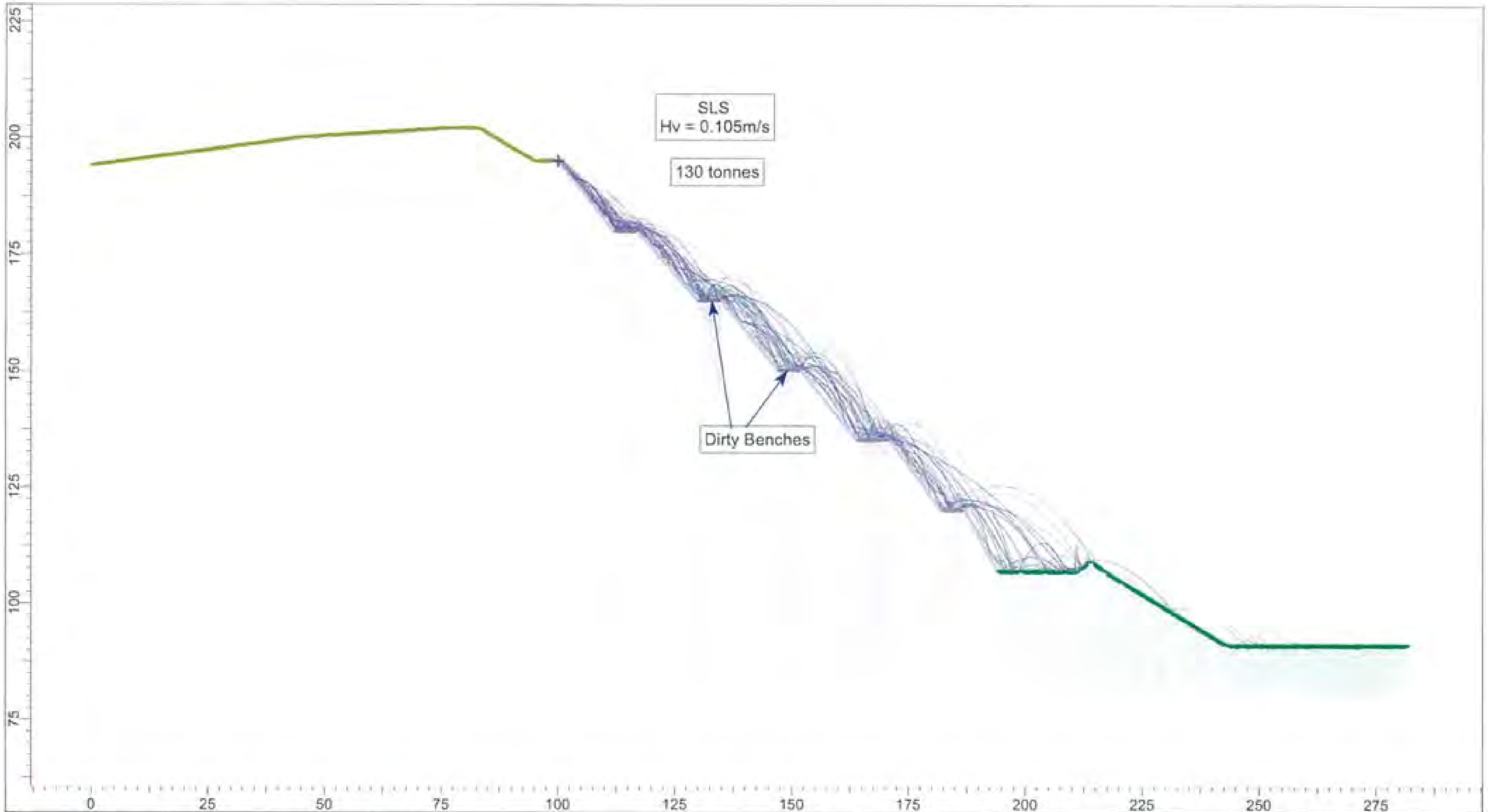


Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/15	File Name	10a Dirty Bench, 12t, SLS.fal5

ROCFALL 5.013



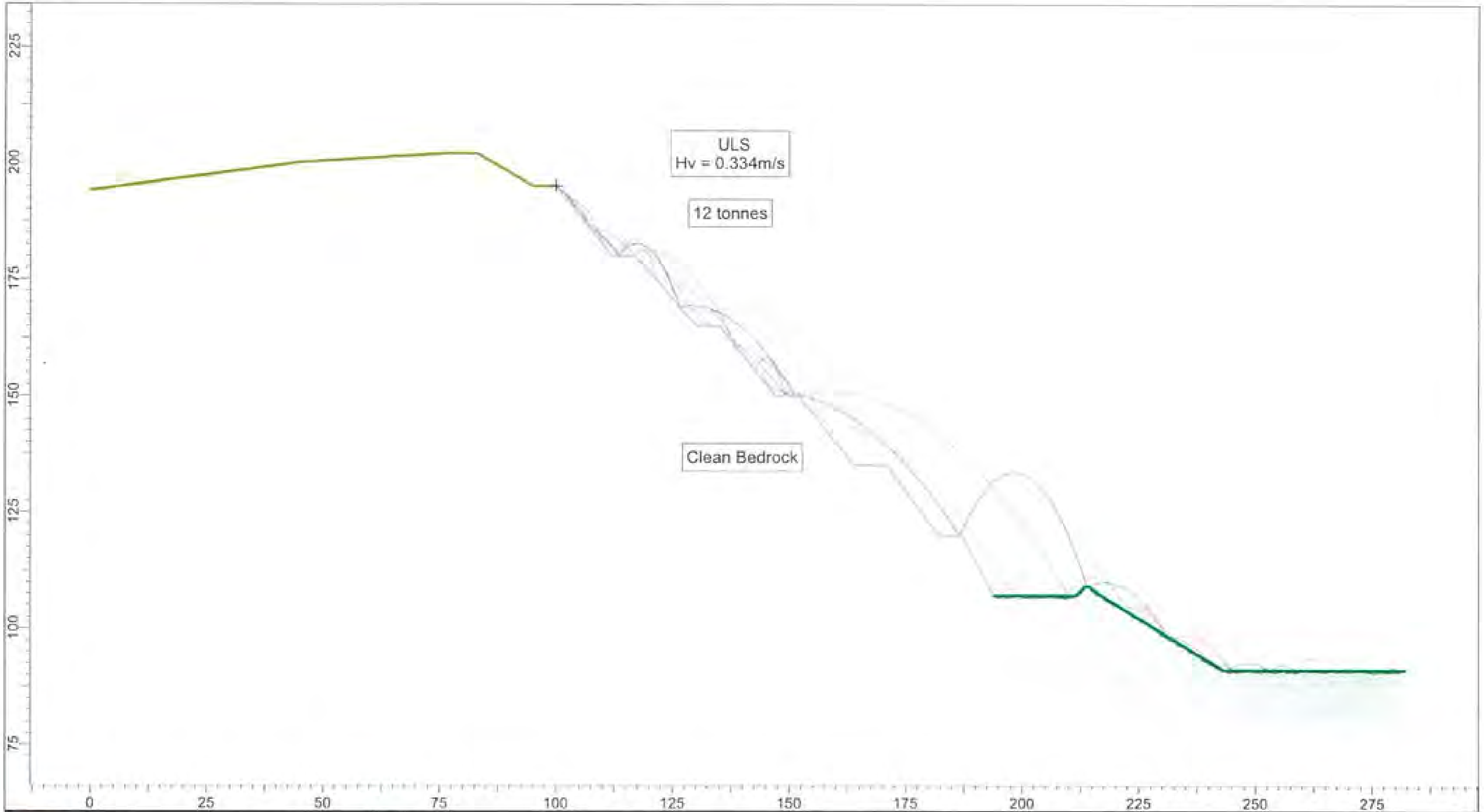
rocscience	Project	Kiwi Point Quarry, Wellington	
	Analysis Description	Rock Fall Analysis	
	Drawn By	S Carryer & A Fell	Company: Ormiston Associates Ltd
ROCFALL 5.013	Date	24/6/15	File Name: 10b Dirty Bench, 67t, SLS.fal5



Project	Kiwi Point Quarry, Wellington	
Analysis Description	Rock Fall Analysis	
Drawn By	S Carryer & A Fell	Company: Ormiston Associates Ltd
Date	24/6/15	File Name: 10c Dirty Bench, 130t, SLS.fal5

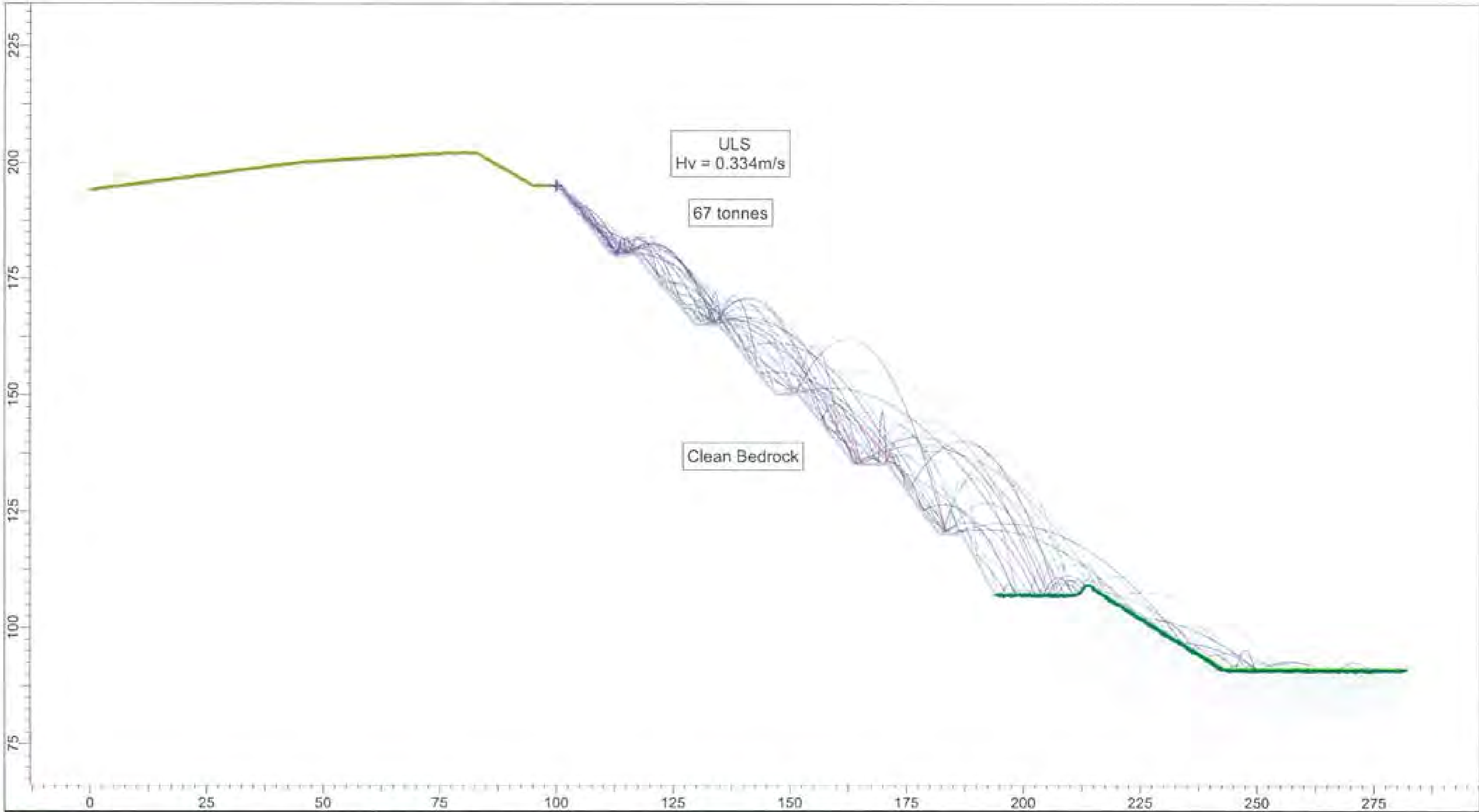
ROCFALL 5.013

Dynamic Conditions (ULS)



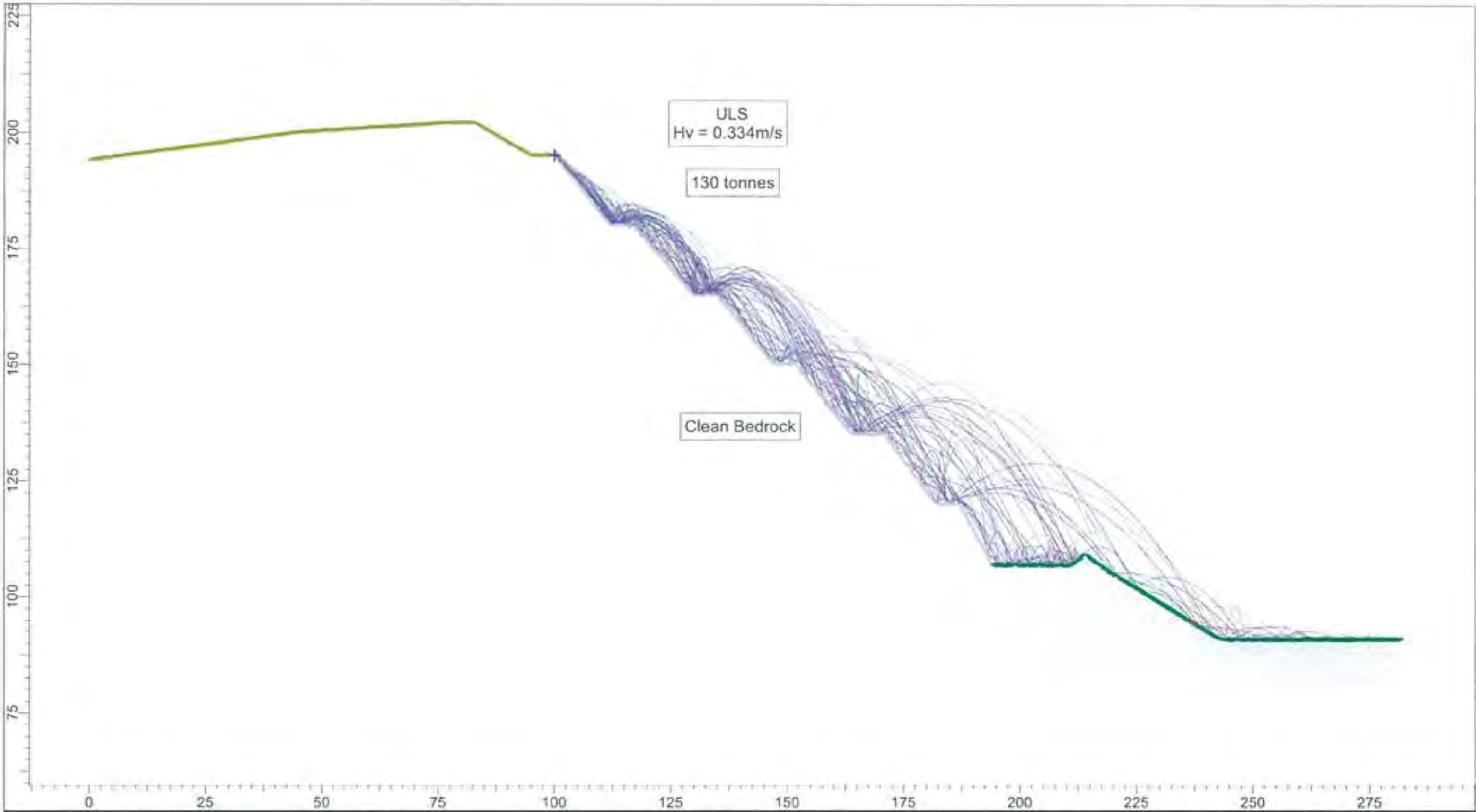
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	11a Clean Bedrock, 12t, ULS.fal5

ROCFALL 5.013



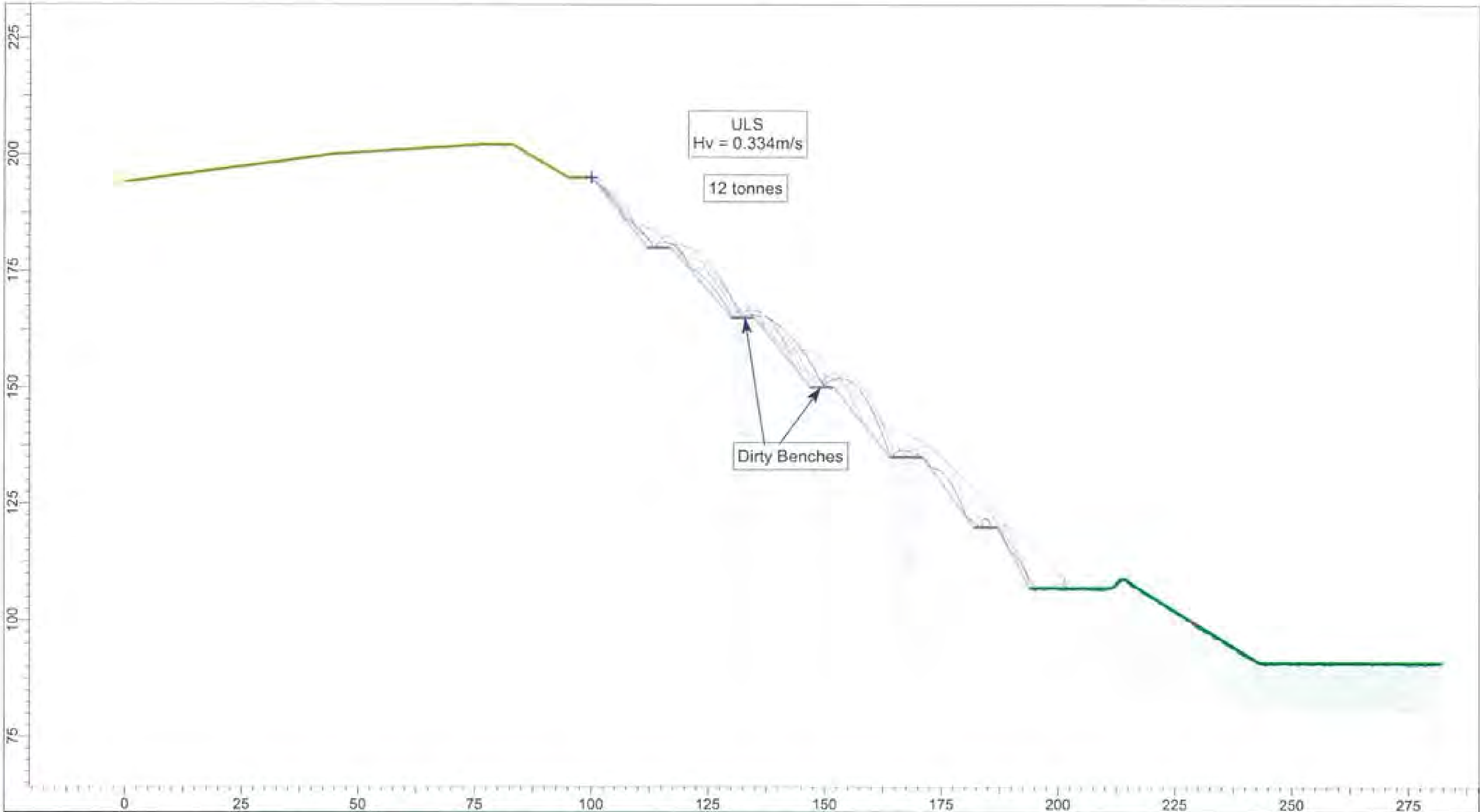
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/2015	File Name	11b Clean Bedrock, 67t, ULS.fal5

ROCFALL 5.013



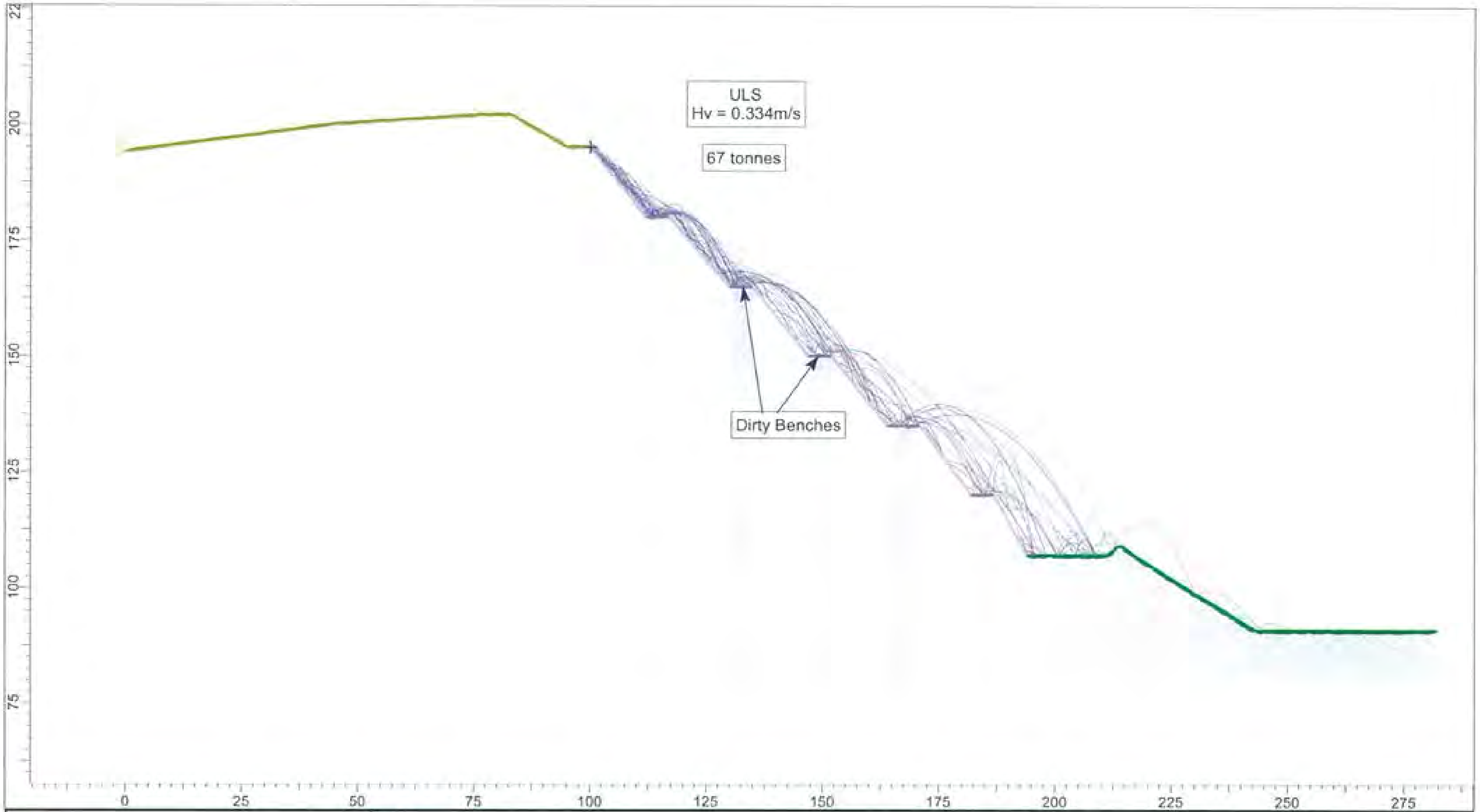
Project	Kiwi Point Quarry, Wellington
Analysis Description	Rock Fall Analysis
Drawn By	S Carryer & A Fell
Company	Ormiston Associates Ltd
Date	24/6/2015
File Name	11c Clean Bedrock, 130t, ULS.fal5

ROCFALL 5.013



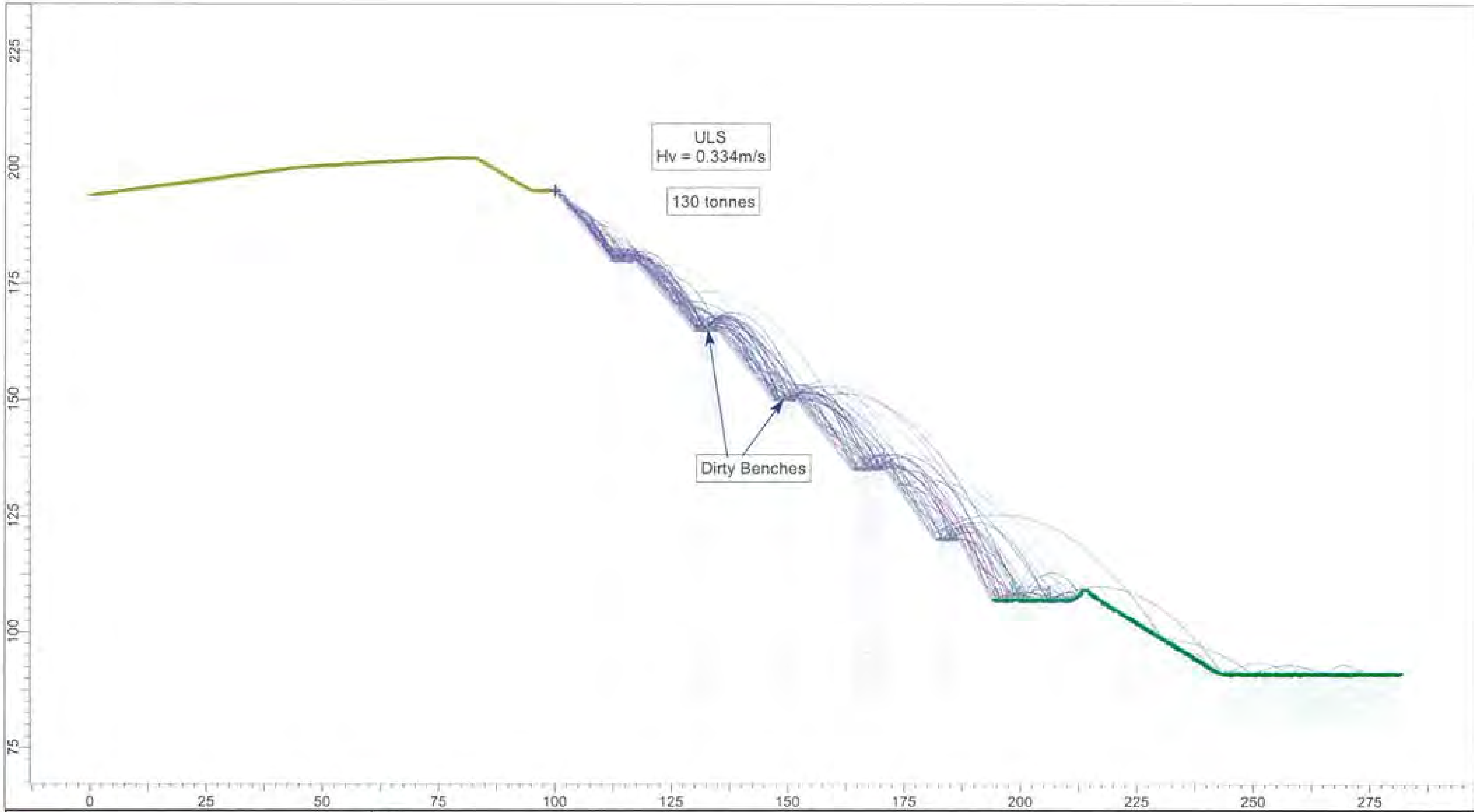
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd
Date	24/6/15	File Name	12a Dirty Bench, 12t, ULS.fal5

ROCFALL 5.013



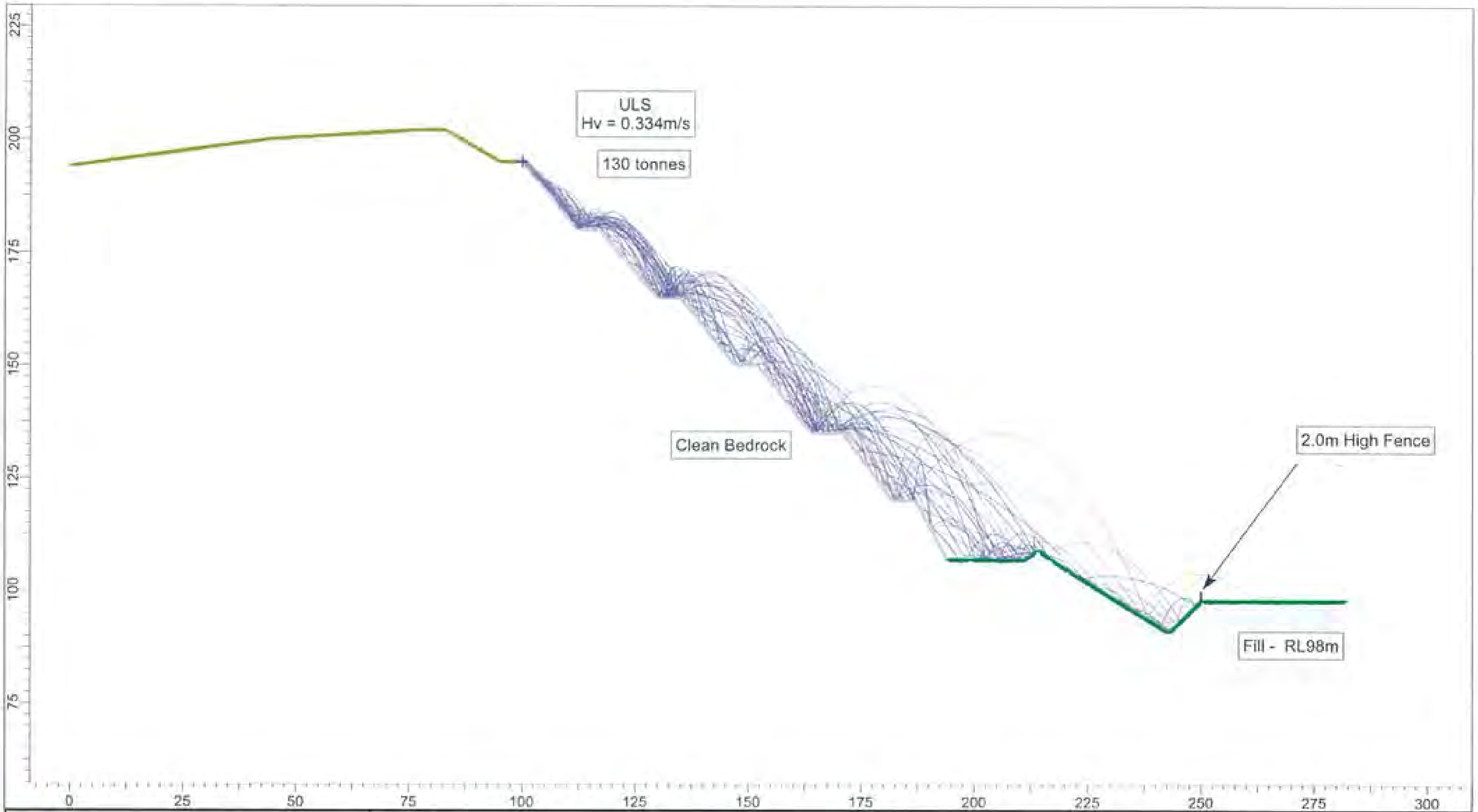
Project	Kiwi Point Quarry, Wellington
Analysis Description	Rock Fall Analysis
Drawn By	S Carryer & A Fell
Company	Ormiston Associates Ltd
Date	24/6/15
File Name	12b Dirty Bench, 67t, ULS.fal5

ROCFALL 5.013



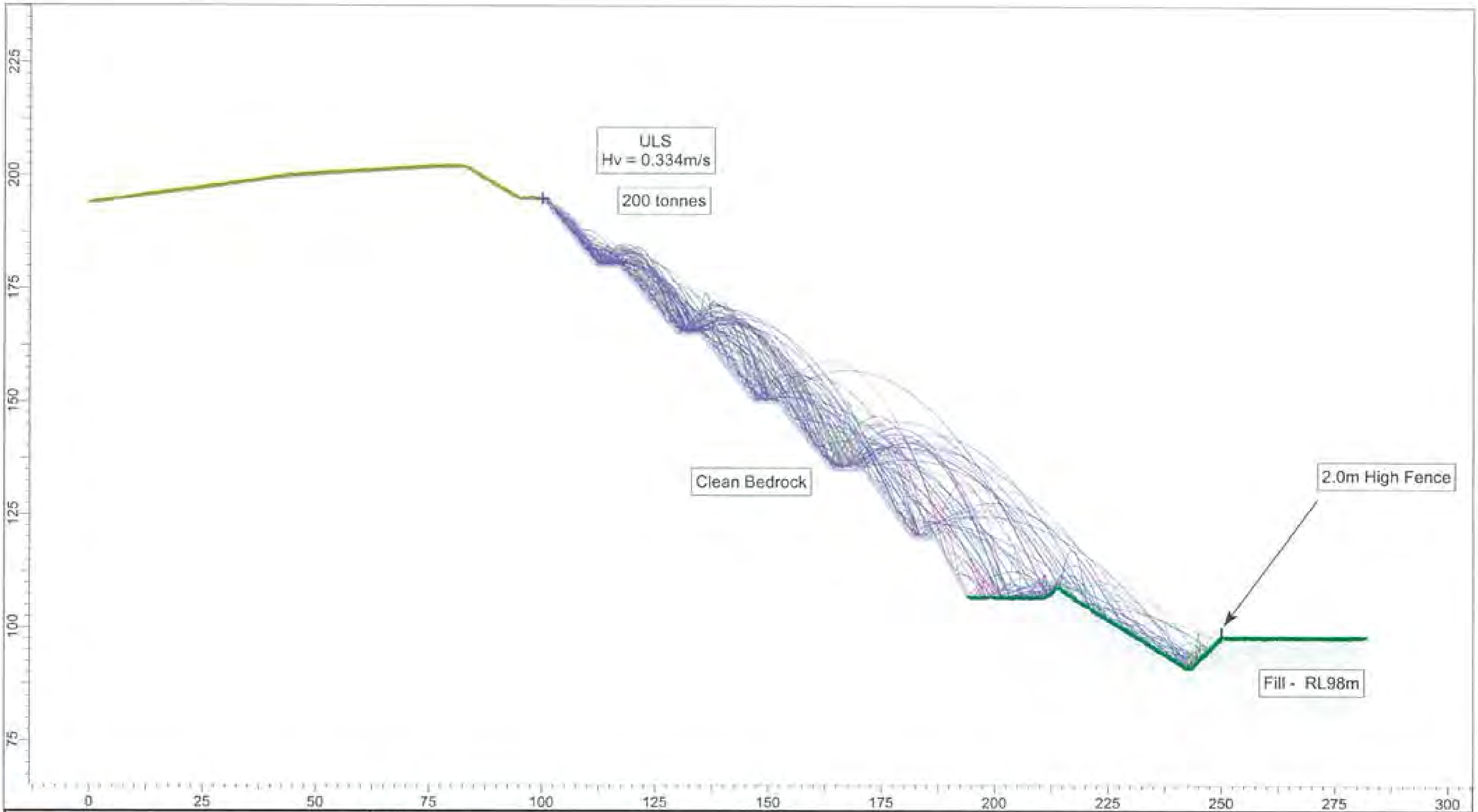
Project	Kiwi Point Quarry, Wellington
Analysis Description	Rock Fall Analysis
Drawn By	S Carryer & A Fell
Company	Ormiston Associates Ltd
Date	24/6/15
File Name	12c Dirty Bench, 130t, ULS.fal5

ROCFALL 5.013



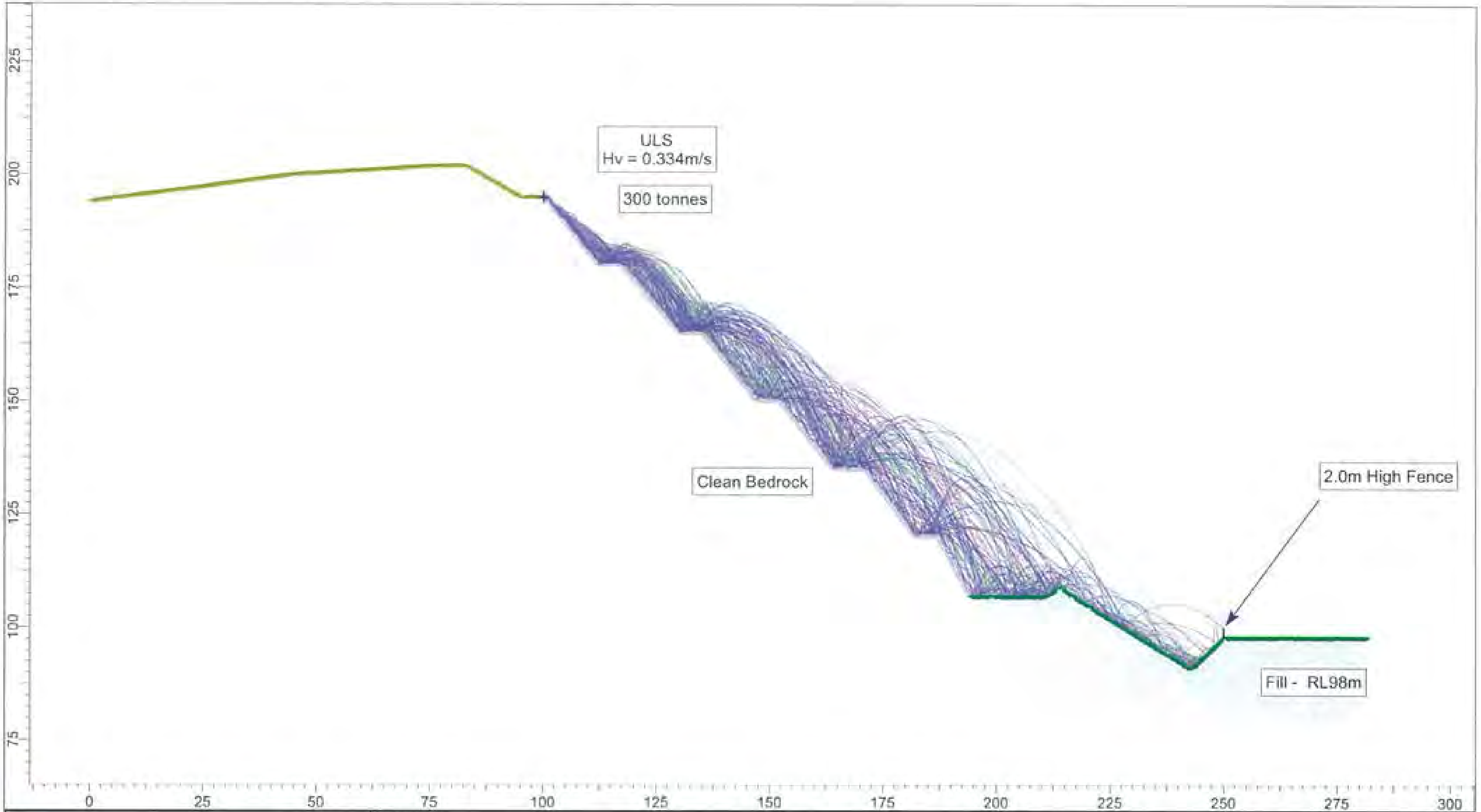
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd.
Date	24/06/2015	File Name	13a Clean bedrock, (ULS), 130t, v-notch + fence.fal5

ROCFALL 5.013



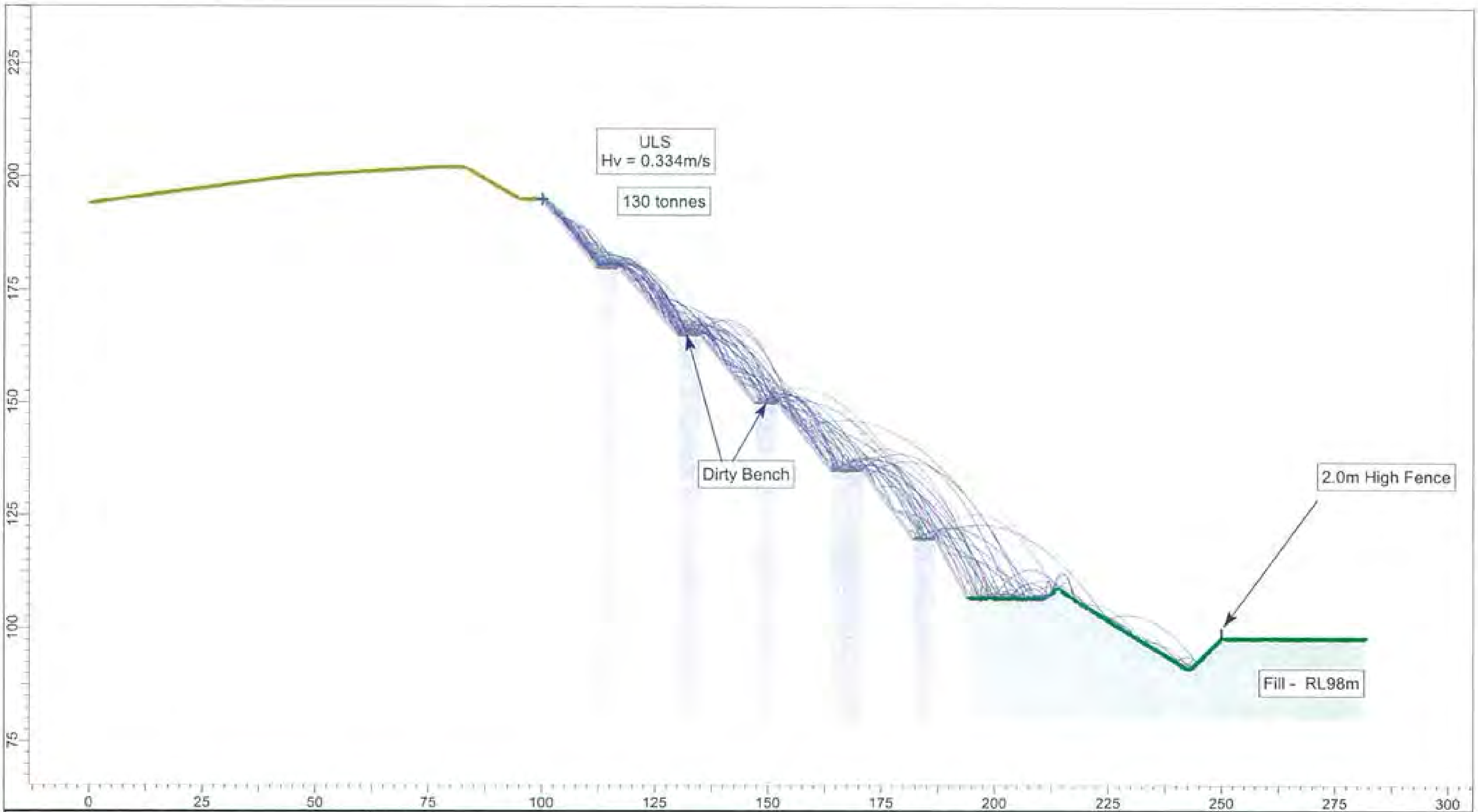
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd.
Date	24/06/2015	File Name	13b Clean bedrock, (ULS), 200t, v-notch + fence.fal5

ROCFALL 5.013



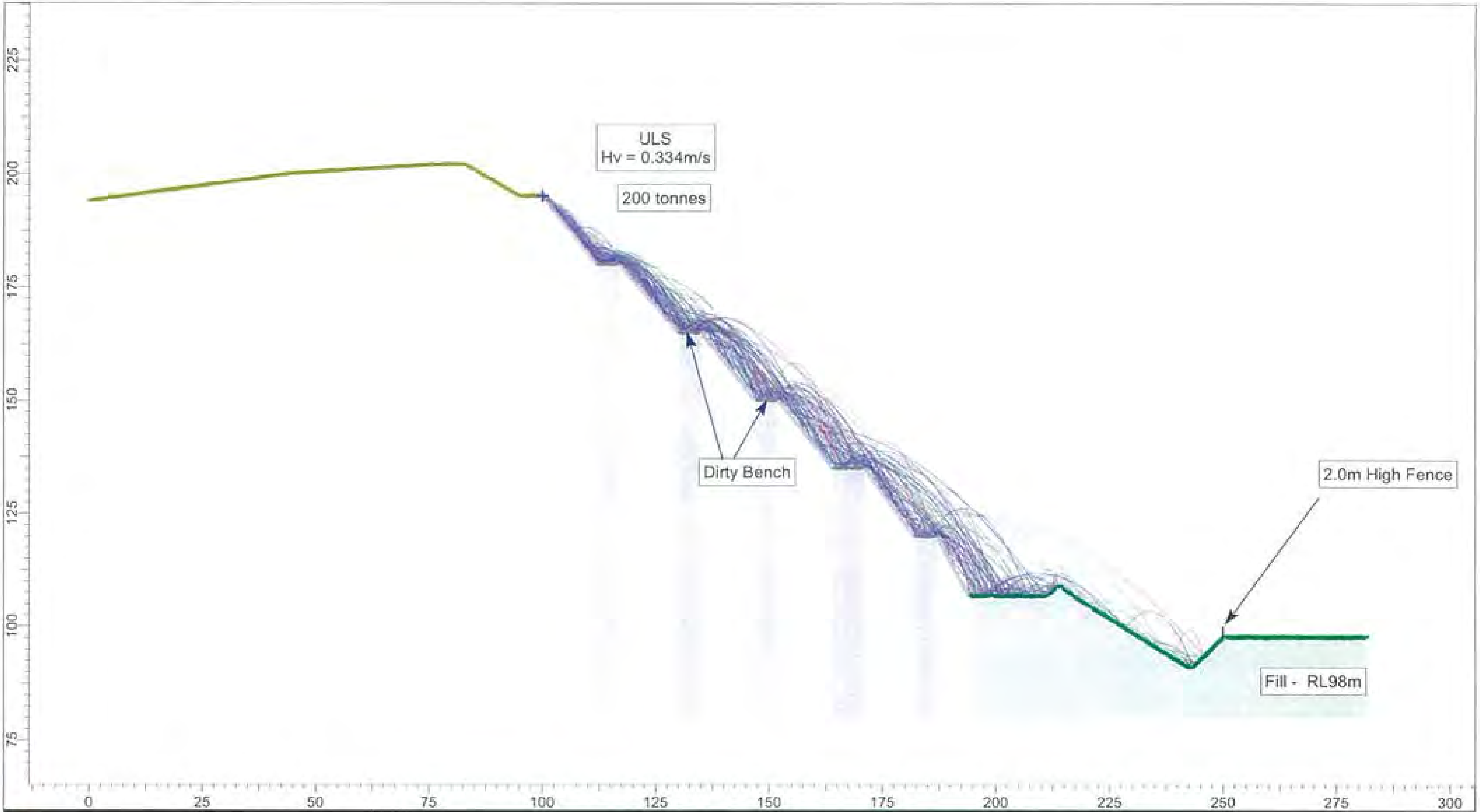
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd.
Date	24/06/2015	File Name	13c Clean bedrock, (ULS), 300t, v-notch + fence.fal5

ROCFALL 5.013



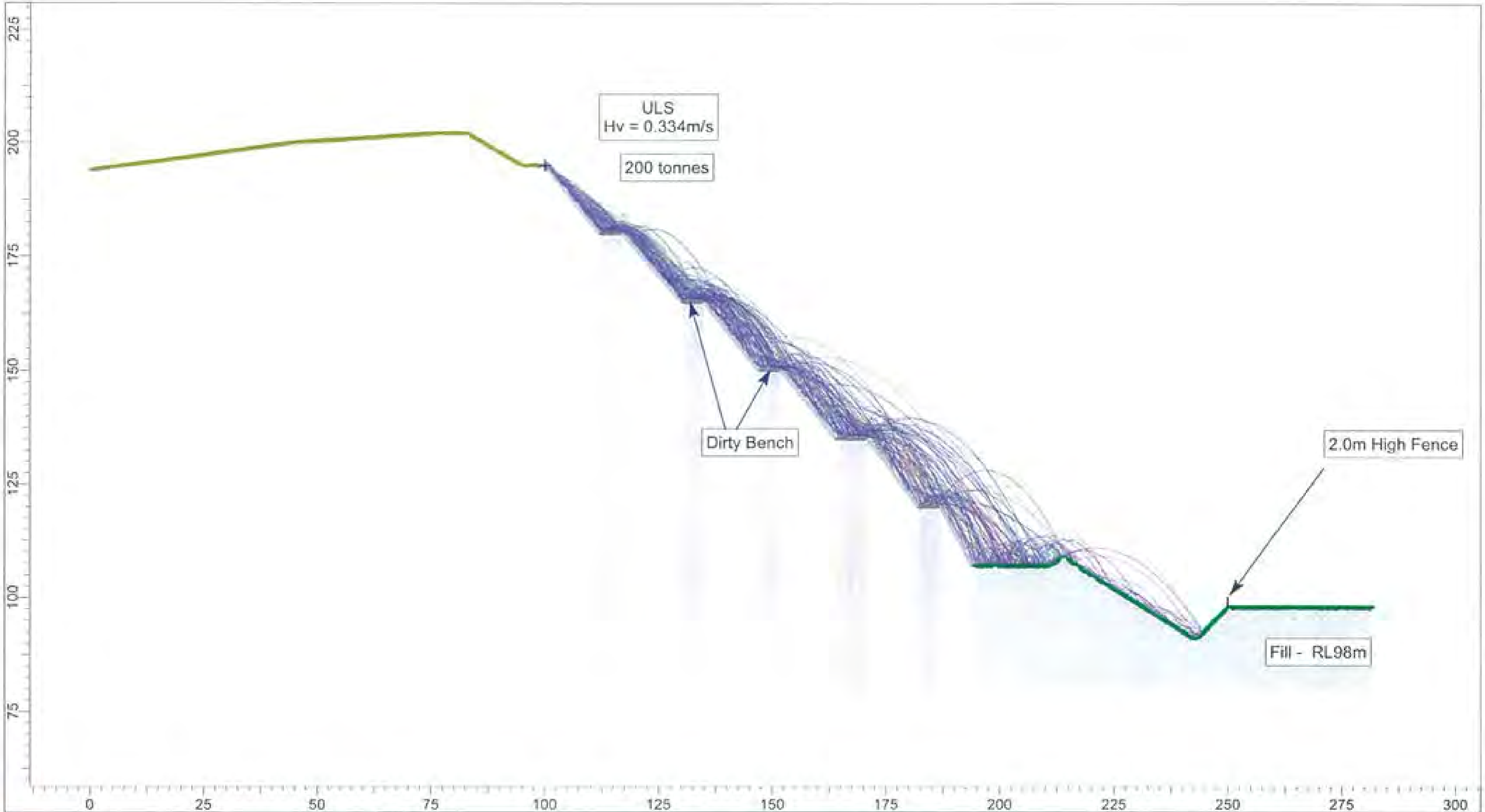
Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd.
Date	24/06/2015	File Name	14a Dirty Bench, (ULS), 130t, v-notch + fence.fal5

ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd.
Date	24/06/2015	File Name	14b Dirty Bench, (ULS), 200t, v-notch + fence.fal5

ROCFALL 5.013



Project	Kiwi Point Quarry, Wellington		
Analysis Description	Rock Fall Analysis		
Drawn By	S Carryer & A Fell	Company	Ormiston Associates Ltd.
Date	24/06/2015	File Name	14c Dirty Bench, (ULS), 300t, v-notch + fence.fal5

ROCFALL 5.013