BMT WBM

Lennox Head Seawall Upgrade Study

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BMT WBM Pty Ltd
Level 8, 200 Creek Street
Brisbane Qld 4000
Australia
PO Box 203, Spring Hill 4004

Tel: +61 7 3831 6744
Fax: + 61 7 3832 3627

ABN 54 010 830 421

www.bmtwbm.com.au

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Title:	Lennox Head Seawall Upgrade Study
Project Manager:	Malcolm Andrews
Author:	Malcolm Andrews, Craig Witt, Joris Jorissen
Client:	Ballina Shire Council
Client Contact:	Paul Busmanis
Client Reference:	

Synopsis: This report presents the findings of a study to improve the defence of the Lennox Head shoreline by assessing the structural competency of an historical buried rock seawall and then to develop a conceptual design for a structure suitable to resist future erosion including the predicted impacts of sea level rise.

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Contents

List of Figures

List of Tables

1 Introduction

1.1 Background

Lennox Head and Seven Mile Beach have a history of coastal erosion and foreshore protection works which began in earnest after the severe storms of 1967 and included an ad hoc seawall which was built over the ensuing decade with no clear design standard. There have been a number of previous studies which have investigated and reported on the erosion and various protection measures proposed and carried out including a Commission of Inquiry in Coastal Protection Works in 1992. In recent years coastal hazards and options for dealing with coastal threats have been formally assessed and a Coastal Zone Management Plan has been developed.

Leading on from this, BMT WBM has been commissioned by Ballina Shire Council to investigate the options for upgrading the shoreline defence at Lennox Head including a study to determine the competency of the existing historical seawall between Byron Street and the Lennox Head – Alstonville Surf Lifesaving Club. The Council's intent is to protect infrastructure under present day and future storm scenarios and identify any upgrading or seawall replacement required to provide a robust defence of the shoreline location. This is to be carried out with reference to the Coastal Zone Management Plan as well as maintaining a balance between amenity of the beach in front of the seawall and parkland between the seawall and Pacific Parade.

The study has been carried out in two main stages over a period of several years:

- Stage 1 of the study reviewed the essential coastal processes and seawall design parameters from previous coastal hazard studies and these were used to make an assessment of the structural capacity of the existing historical seawall structure between Byron Street and the Lennox Head – Alstonville Surf Lifesaving Club.
- Stage 2 has been informed by the results of stage 1 which indicated that the existing historical seawall had little structural capacity and needed to be replaced by a new terminal structure. A full range of structural options have been assessed and a recommendation of suitable structure types and seawall alignment made.

This report combines both studies and recommendations into a single report and includes the following Chapters:

- Chapter 1 – Background
- Chapter 2 – Review of Existing Information
- Chapter 3 – Coastal Hazard Assessment
- Chapter 4 - Characterisation of Existing Historical Seawall
- Chapter 5 – Structural Stability Assessment of Existing Seawall
- Chapter 6 – Background to Seawall Upgrade
- Chapter 7 – Coastal Hazards for Design Considerations
- Chapter 8 – Terminal Seawall Design Considerations

- Chapter 9 – Terminal Seawall Structure Options
- Chapter 10 – Conceptual Rock Seawall and Concrete Step Designs
- Chapter 11 – Implementation Plan.

1.2 Historical Protection Measures

Erosion protection works began at Lennox Head in 1967 after severe storms in the form of a ti-tree fence and ad hoc dumping of rock and soil on the eroded dune face (refer Figure 1-1 and Figure 1-2). These defences have been strengthened over the years particularly in the southern section when shoreline retreat has directly threatened development. The plan in Figure 1-3 shows a summary of defensive works that had been undertaken up until 1989.

Figure 1-1 Construction of Ti-Tree Fence following 1967 Storms

Figure 1-2 Broader View of Ti-Tree Fence

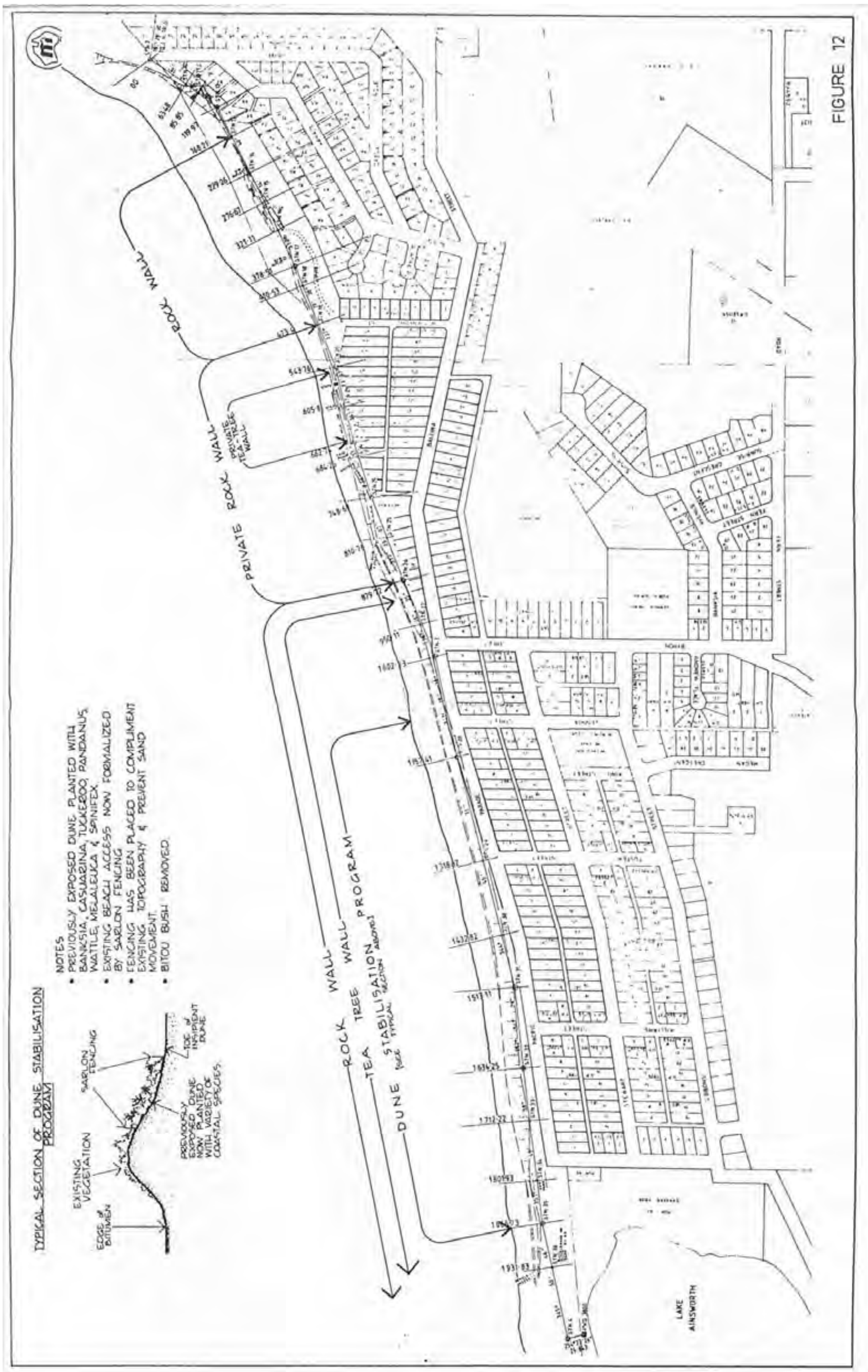


Figure 1-3 Early Lennox Head Seawalls (Ardill & Assoc. 1989)

2 Review of Existing Information

2.1 Previous studies and Commission of Inquiry

There have been many directly relevant previous investigations and reports relating to coastal erosion in the immediate study area and in the region generally. These are included in the Reference listing and referred to in relevant sections of this report. The two most notable major erosion events on record for northern NSW and southern Queensland, being June/July 1967 and 6th February 1974 and Lennox Head was severely impacted during these events.

The reference material of significance to the Ballina Coast includes an extensive range of previous studies of geology and geomorphology of the coastal systems of the region (Chapman et al 1982; Roy and Stephens 1980; Roy and Thom 1981; Stephens et al 1981; Thom et al 1978; PWD 1978). There have been a number of investigations associated with coastal erosion and protection measures at Lennox Head itself including an assessment of erosion trends and background studies for the 1993 Beach Management Plan carried out (PWD 1985; WRL 1986; Ardill & Associates 1989; Geomarine 1990; Commission of Inquiry 1992). Subsequently BMT WBM (formerly WBM Oceanics) carried out a Coastal Hazard Definition Study in 2003 further analysing photogrammetric profiles and defining short term storm bite and longer term recession and later Dean Patterson of WBM assessment of longer term impacts of sea level rise and the Richmond River breakwaters with newly developed shoreline evolution software (ECO MOD).

Lennox Head and Seven Mile Beach have experienced significant erosion over many years. There have been a number of previous studies which have investigated and reported on the erosion and various protection measures proposed and carried out. These are summarised below.

(a) Coastal Advice for Draft LEP (PWD, 1985)

- No detailed coastal process studies have been carried out;
- Preliminary photogrammetric analysis indicates historical recession rates of the Seven Mile Beach embayment of approximately 1m/yr;
- There is also geological evidence of long-term recession; and
- The Department adopted a 200m setback in the absence of a detailed study.

(b) Coastal Protection at Lennox Head (WRL, 1986)

- Recession rates indicated by aerial photographs between 1947 and 1981 are of the order of 1m/yr for the southern two-thirds of Seven Mile Beach;
- The rate is highest around the centre of the beach where rates of up to 1.5m/yr have been observed;
- The results of land survey carried out in 1879 confirm the long-term nature of the erosion;
- Protective works east of Raynors Lane and the shopping centre appear to have reduced the recession rate;
- The width of beach has narrowed;

- Cause of recession is not clear and may be related to:
  - Increase in Mean Sea Level;
  - Wave climate changes; and
  - Interference with sand supply (e.g. Breakwaters).
- Sea level changes are likely to be too slow and have a great deal of uncertainty;
- Richmond River breakwaters have had an effect on the movement of sand along the coast and the erosion at Lennox Head could be linked to this; and
- An 1879 survey pre-dates the start of the walls but there is no preceding or intervening data to draw any conclusions as to whether erosion is a continuation of natural processes or whether man's activities have altered the erosion rates.

(c) Statement of Environmental Effects for Proposed Seawall at Lennox Head (Ardill & Associates, 1989)

- Original subdivision has roadway reserve R1082 generally 100m wide from HWM as determined in 1884;
- In the location of DP11687 considerable erosion has occurred between 1922 and 1946 – no specific records of the erosion exists although there is some discussion in Newspaper articles;
- Photos around the late 1920's and 1940 indicates that significant erosion has occurred at the southern end of Lennox Head since that date with local reports of 50 – 60 yards lost;
- In 1942 residents built a rock wall using rock from the reef in response to the perceived threat;
- Council carried out foreshore protection works in 1950's in front of Lots 1 to 4;
- In the 60's a series of rock protection works were carried out;
- In 1967 cyclones caused concern at Lake Ainsworth for a brief period of time when the dune was blown out and foam and wash penetrated the fresh water lake;
- A newspaper reported only 9 feet separating the sea and the flooded lake whereas 10 years previous, 75 feet existed;
- Erosion in front of the Fitness Camps at Lake Ainsworth led to the construction of a 600 feet long rock wall to prevent a break through at the location of an ancient entrance;
- In July/August 1967 a ti-tree fence/wall was constructed which was exposed again in 1972 due to the effects of cyclone Wendy;
- Further erosion was again reported in 1974;
- A rock wall was constructed and earthworks carried out by Council between 1977 and 1980 from about Byron Street to north of Ross Street (see Figure 1-3);

  Assessment of erosion indicates:
  - Rock works at Lennox Head commenced in the early 1940's and masked natural movements;

  - Recession from top of bank on 1922 plan to top of scarp in 1947 is approximately 20m;
  - Recession of the erosion scarp is most pronounced at southern end where in excess of 40m occurred between May 1947 and June 1977;
  - The dunal system at Lake Ainsworth has receded by approximately 30m between 1947 and 1967 prior to the construction of the rock wall;
  - Accretion can be attributed to dune reshaping, reclamation and wall construction;
  - Recession distances between 1947 and 1987 range between 11m and 45m at rates of 0.6 to 1.5m/yr;
  - Mean recession rate 0.9m/yr; and
  - Hazard zone map includes provision for immediate impact of 50m plus long term recession at 0.9m/yr and an allowance for sea level rise.

(d) Environmental Impact Statement for Beach Management at Lennox Head (Geomarine 1990)

- Summary of coastal processes and history of erosion.

(e) Commission of Inquiry in Coastal Protection Works (1992)

- History of erosion and past remedial action; and
- Opinions of coastal specialists on processes and impacts of proposed works.

Following on from the above studies and the Commission of Inquiry, the Lennox Head Beach Management Plan was implemented in 1993.

The coastal hazards were updated in 2003 and 2013 by BMT WBM with the benefit of a history of photogrammetric profiles from 1947 to 2010.

2.2 Regional Geology

The coastal geology of the northern NSW coast has been well documented in many reports. The basis of these reports in respect to geology can be attributed to work carried out by Roy, 1973, and Thom et al, 1978, which described in detail the stratigraphy and morphology of the eastern coastline of Australia.

The processes that moulded the present coastal topography started some 60-80 million years ago when faulting and sea floor spreading along the South Eastern Continental margin formed the Tasman Sea and the Eastern Highlands. The basement rocks in the study area are metamorphosed sediment. These outcrop at Cape Byron, Broken Head and Lennox Head.

The bedrock hills rarely exceed elevations of 160 metres and in general, form discontinuous spurs that fall to less than 50 metres near the coast. Seaward extensions of these ridges shallowly underlie the coastal plain and in places outcrop on the coast or just offshore as rock reefs. The geophysical investigation indicates that the bedrock floors of the valleys between these ridges are in places up to 45 metres below the present coastal plain, the infilling material mainly consisting of Quaternary sediments. Tertiary basalt flows from the Mount Warning shield volcano infilled valleys in the Palaeozoic bedrock. Subsequent erosion has produced northeast trending bedrock spurs that terminate in the coastal plain. Quaternary geology in the coastal region of NSW is closely related to sea level changes caused by the episodic growth and decay of the polar ice sheets during the past million years.

The majority of the present day coastal sediments in the region were deposited during the latter stages of sea level rises (marine transgressions) and the following interglacial periods. At least seven major sea level fluctuations have occurred in the last 700,000 years (Thom and Chappell 1975). However, in the Ballina Coast region, it is believed that a majority of the present day sediments were laid down during the Holocene and late Pleistocene epochs. The Holocene deposits are the youngest and are the product of the last sea level rise in which its present position ($\pm$ 1 metre) was reached some 6,000 years ago. The older Pleistocene sediments are in the main related to a sea level rise 120,000 to 140,000 years ago (Thom et al 1978), which culminated in a sea level some 5 to 6 metres above the present situation. The resulting elevation of the Pleistocene sediments above present day sea level is probably the reason for their preservation during the initial erosional phase of the most recent marine transgression.

Two distinctive bay-barrier systems occur along the NSW coast. These have been termed the Inner Barrier and the Outer Barrier (Thom, 1965). The Pleistocene sand deposits are termed inner barrier sands. Sand deposited during the Holocene period are termed outer barrier sands.

A succession of high sea-level stands (interglacial transgressions) during the Pleistocene period resulted in the deposition of marine sediments in the valley embayments to form a coastal plain. These deposits accumulated within the nearshore, beach and dune areas, being 50 metres and more thick in the central area. Concentration of colloidal humic material from groundwater movement produced layers of dark-brown indurated sand. Over the inner edge of these coastal sand deposits, fluvial processes resulted in the deposition of silts and clays in the heads of coastal valleys.

At the beginning of the Holocene period, about 18,000 years ago, the postglacial transgression initiated a rapid rise in the sea level. During this event, marine quartzose sands, eroded and deposited on the exposed continental shelf during the preceding low sea-level stand, were remobilised and carried shorewards. Their accretion on the pre-existing Pleistocene shoreline resulted in the development of the present day longitudinal coastal barrier and beach complex. These deposits are termed the outer barrier sands. Thom et al (1978) considers that 7,000 years B.P. sea level was somewhere between 10m and 15m below present. The attainment of present day sea levels, approximately 6,000 years ago, would have drowned the previously postulated land bridges between existing outcrops of bedrock. Furthermore, this was the period of maximum onshore sand flux.

The major sediment movements since 6,000 years B.P. have been tidal delta building, gradual fluvial movements of sediment through the Richmond River generally during flood events and more significantly, the northerly littoral drift along the coast. Applying the general works of Thom (1974) it is likely that after 6,000 years B.P. the coastline was subject to rapid accretion, followed by a state of slow accretion, approximately 3,000-4,000 years B.P. Relative stability or slow recession has probably characterised the last 3,000 years to the present.

There is now a zone of active longshore and cross-shore beach sand movement which extends along the entire regional coastal system from the Clarence River to the northern entrance of Moreton Bay. This comprises contemporary Holocene sand extending from the most seaward depth of active seabed movement to the onshore limit of dune erosion during severe cyclones. In some areas where net accretion of the shoreline has occurred over the past 6,000 years, this active system is backed by earlier Holocene accumulations of dune sand, now beyond the immediate zone of potential erosion. In other areas where net erosion of the shoreline has occurred, the present active beach system may juxtapose directly against much older (Pleistocene) dune sand or bedrock.

In northern NSW and southern Queensland, onshore sand transport is also believed to have occurred during several earlier cycles of rising sea level and subsequent stillstands during the past million years. Some embayments had already been totally infilled with sand during the previous high sea level (last interglacial of Pleistocene age). These embayments could not act as coastal traps for sand during the Holocene, and sand transported onshore in such areas was removed by northward littoral drift. As such, accretion of mid-Holocene barriers was restricted to previously unfilled embayments. Many of these barriers in the southern sections of the regional coastal system have since been removed by erosion associated with the northward drift of sand. In contrast, relatively wide Holocene barriers remain in the northern areas within the Tweed Coast, Gold Coast and Stradbroke Islands. The present day extent of onshore beach/dune Holocene sand along the Ballina Shire coast is shown in Figure 2-1.

From many studies, it appears that three main “sedimentary units” (that is, clearly distinguishable types of sediment) are found in the depth zone from 0-30m.

- Inner nearshore sand
- Outer nearshore sand
- Inner shelf sand.

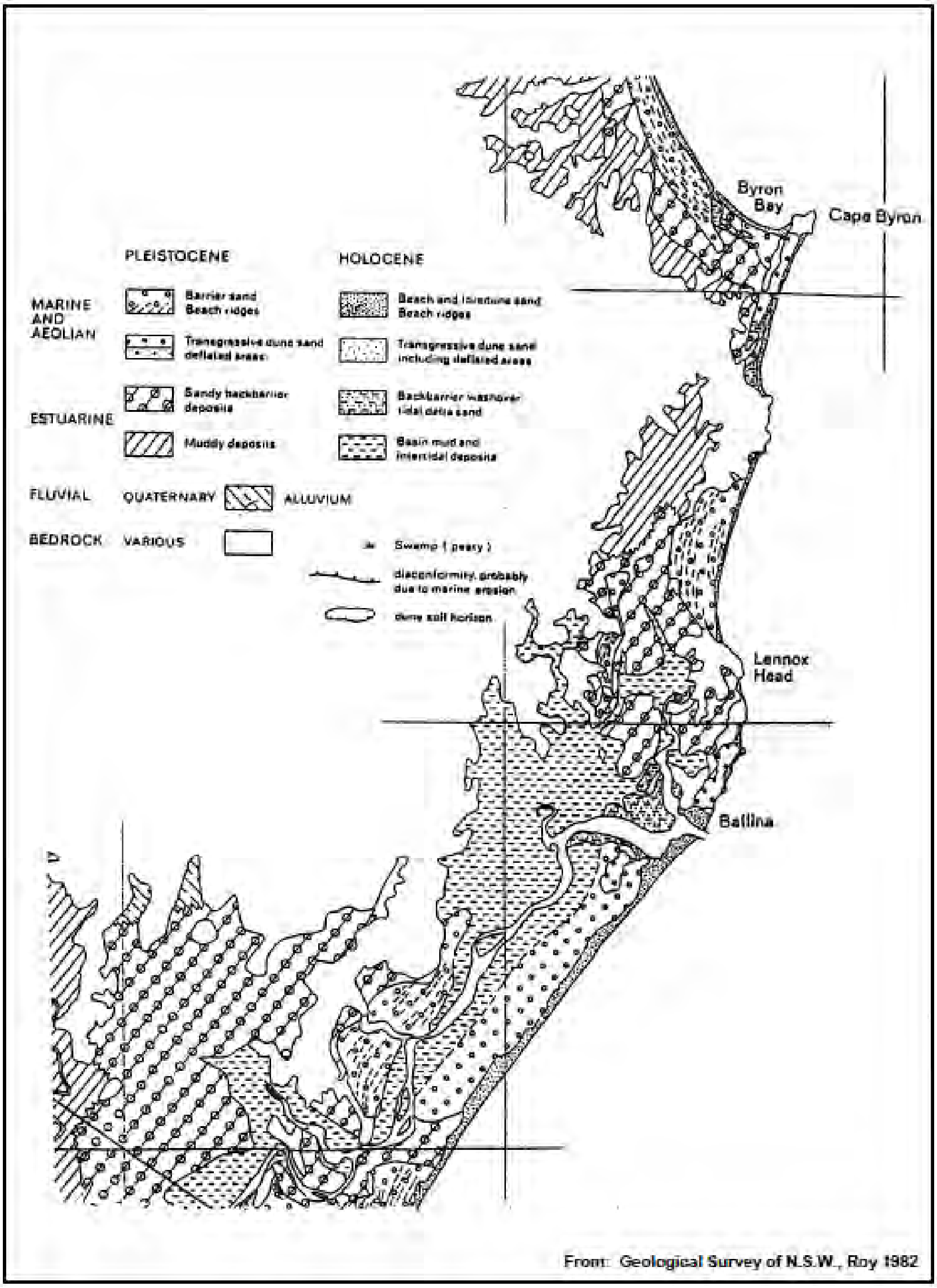


Figure 2-1 Ballina Shire Quaternary Geology

Of these, the inner shelf sediment unit, hardly interacts with nearshore processes. The inner nearshore sand unit is found in the upper part of the profile, generally from the shoreline to approximately 10 m depth. It indicates an almost continuous sediment pathway along the New South Wales and southern Queensland beaches, driven by the dominant southeast swell. The outer nearshore sediment unit is found in a continuous strip offshore of the inner nearshore unit, in depths of approximately 10-25m, moved by currents (East Australian Current and occasionally wind driven currents) while waves act as stirring agents, especially during storms. The net transport direction for this unit is less clear than for the inner nearshore unit.

In some cases, notably at Cape Byron and Point Lookout, the headland protrudes in such a way that the East Australian Current can impinge on the coast and interfere with the nearshore sediment transport. In such cases, a lobe of nearshore sand is found in deeper water south of the headland. This lobe can be seen as a loss to the littoral system.

In the case of Cape Byron, this offshore accumulation of inner and outer nearshore sand amounts to approximately 510,000,000 m^3 over the past 6,000 years, or an average yearly loss of approximately 50,000 m^3/yr (PWD 1978) to 84,000 m^3/yr (Delft Hydraulics 1992). At Point Danger, which protrudes much less sharply than Cape Byron and Point Lookout, no such lobe was found. Delft Hydraulics (1992) concluded that only the inner nearshore sand unit provides the clear pathway of longshore sand transport from Letitia Spit to Gold Coast beaches.

2.3 Shoreline Evolution

Where longshore sand transport is relatively strong and continuous along an extended coastal system, short and longer term patterns of beach erosion or accretion are influenced predominantly by any differentials in the longshore supply of sand. Where more sand is moved away to the north than is supplied from the south (or other sources), there will be a net loss of sand leading to shoreline retreat.

There is considerable evidence that this is the case along the coastal units of northern NSW including those embayments within the study area. This evidence is manifest in the form of:

- The typically crenulate shape of the beach units between the controlling bedrock headlands, with increased indentation (hook) at the southern end of each unit;
- A general absence of Holocene sand barriers south of around Bogangar except in small areas associated with river channels; and
- Evidence of continuing exposure in the contemporary beach/dune face at the southern ends of beach units of older Pleistocene hind-dune peat and clay deposits such as at Lennox Head and Suffolk Park.

Roy and Stephens (1980) report exposure of aboriginal middens at Byron Bay for which radiocarbon dating indicates average erosion rates there over the past 300 to 400 years of 0.25 to 0.35 m/yr.

At Lennox Head, there is a prominent nearshore reef comprised of peat overlying unconsolidated sand. A radiocarbon date on a tree stump extracted from the peat by PWD gave an average age of 3,765 (+/- 70) years B.P. (Geomarine, 1990). Accordingly this peat must have formed in a freshwater swamp behind a sand barrier (similar to the existing back beach swampy areas further to the north). This suggests that during present sea level conditions, a Holocene beach barrier existed seaward of the peat deposit (the reef) and has since been completely removed presumably by ongoing recession of the shoreline (Geomarine, 1999).

The surface morphology of the regional coastal zone is described in Thom et al (1978) and PWD (1978). The plan shape of the shoreline along the region reflects the dominant southeast swell conditions and northward net movement of beach sand. This manifests as a series of crenulate shaped embayments, more hooked at their southern ends and aligned more uniformly and relatively consistently at north-northeast (approx. 20°) at their northern ends.

Stephens, Roy and Jones (1981) reported such processes together with a conceptual model showing the mechanism by which the coastline has responded to the differential over recent geological times (refer Figure 4-4). This model showed at a regional scale that the greatest response in terms of progressive shoreline recession occurs at the southern end and extends further north over time. It illustrates the evolution of zeta form embayments between controlling headlands as a result of the longshore transport processes.

To the south of the Richmond River there are two major compartments. Shark Bay extends from Woody Head to Evans Head and is deeply embayed. The compartment between the Evans Head and the Richmond River is less embayed but on a similar general coastal alignment. The pocket beaches to the north of Ballina and Seven Mile Beach and Tallow Beach in Byron Shire are somewhat less embayed and on different general alignment.

The most dominant of the embayments is at Byron Bay in the lee of Cape Byron. In the coastal embayment between Byron Bay and Brunswick Heads, there is no Holocene back-beach barrier, but rather an extensive region of shore parallel beach ridges of Pleistocene age. These formed when barrier building sands moved onshore during a former (Pleistocene) period of sea level rise. A maximum of 13 ridges occur just to the north of Belongil Creek, the number reducing in the northerly direction, indicating a higher rate of progradation of the shoreline in the hook of the bay than elsewhere at that time.

In contrast, the Pleistocene ridge system north of Brunswick Heads is narrow and discontinuous, generally varying in height from +3m to +9m above sea level.

The exposure of Pleistocene sediments - sand rock reefs on the seabed and the dune scarp at the rear of the beach - indicates long term marine erosion and landward retreat of the shoreline in the Byron embayment, most significantly in the southern hook area north to Belongil Creek.

WBM (2000 and 2001) identified substantially less long term shoreline retreat at and north from Brunswick Heads.

Further north, through the Tweed and Gold Coast areas to Stradbroke and Moreton Islands, the shoreline shape has evolved as a series of crenulate embayments of size and orientation determined by both the prevailing wave climate and the relative positions of the controlling headlands. Where the headlands are closely spaced or aligned more or less north-south, the embayments are shallow and the shoreline relatively straight. Where the headlands are wide apart and/or oblique to north-south (e.g. Cudgen/Fingal/Point Danger), the embayment shape is unique to the particular circumstances, but adapted to accommodate the net longshore movement of sand through the region.

The width of the onshore Holocene dune sands increases with distance further north along the regional coastal system. While it is absent in the Ballina and Byron area, it begins to have significant width at around Bogangar, comprises the whole of Letitia Spit and widens out to typically 1000 metres around Broadbeach on the Gold Coast. The whole of South Stradbroke Island is

Holocene sand, as is the barrier unit forming the Eighteen Mile Swamp on North Stradbroke Island (some 2.5 km wide at its southern end).

Moreton Bay contains a vast quantity of coastal sands deposited there during the Holocene period. It has been estimated that there are about 1,400 million m^3 of sand in the South Passage delta and 4,000 million m^3 sand in the North Entrance tidal delta (Stephens 1992).

2.4 Richmond River Training Walls

The Richmond River is the only river of significance in the study region with respect to potential fluvial supply of sand to the coastal sand budget.

The lower reaches of the river and its entrance have undergone substantial change since the mid 1800's when it was first established as a port. Substantial dredging and river training works have been carried out since the late 1800's including:

- Dredging of the entrance channel and bar which continued until about 1974;
- River training works;
- Entrance breakwater construction; and
- Dredging of North Creek for navigation, water circulation and construction/reclamation since 1890.

Much of the dredged material was used for extensive reclamation in the Ballina region. A previous Sedimentological Study (PWD, 1993a) summarised the main features of a conceptual sediment dynamics model. Key features with respect to coastal processes include:

- The system is very stable with very little sediment movement other than fluvial reworking during major floods and some wave stirring and tidal movement near the entrance; and
- The active marine delta extends some 2,200m upstream of the entrance to the shoal near south-east Ballina (Kingsford Smith Drive) and into North Creek.

The net effect on sand supply/loss to the beach system over that time was not clearly or fully understood at the time. However, it is apparent that the River system is now supplying negligible quantities of sand. Furthermore, the extensive dredging works in the lower estuary and entrance for navigation and reclamation is likely to have resulted in a net loss with sand infeeding from the beaches to compensate for the material removed. The exact extent of such loss is unknown but is likely to be substantial given the extent of reclamation works in the Ballina region. Such works have now ceased such that ongoing losses to the river system will be minimised.

Various assessments of the impacts of the training walls and of dredging sand from the Richmond River entrance have been made by Dean Patterson of BMT WBM using the state-of-the-art Shoreline Evolution Model (SEM).

The model was designed initially to give a significantly improved estimate of recession due to sea level rise compared to predictions given by the Bruun Rule (1962). However, a key feature of the model is the inclusion of coastal structures such as headlands, reefs, river training walls, groynes and seawalls. This combined with regional longshore transport, onshore sediment supply rate, and calculation of cross shore and longshore transport driven by wave time series data, enables the assessment of realistic spatial variation in recession alongshore in response to sea level rise. The model is capable of assessing multiple beach units along sections of coastlines, providing for regional coastal processes and response to sea level rise.

Patterson (2007) analysed longshore transport rates along the whole region from Iluka to the Gold Coast and found a consistent progressively increasing pattern of net transport, with a contemporary rate at South Ballina of about 260,000m^3/year increasing to over 400,000m^3/yr at Tallow Beach. This gradient in longshore transport would cause long term progressive recession of the shoreline at Seven Mile Beach. However, it is also likely that there is some residual shoreward supply of sand into this beach system, thereby at least partially offsetting the tendency for shoreline recession.

Patterson (2009) modelled the impacts of the training walls on the beach system and found that there has been a long term reduction in the annual average rate of transport past the Richmond River and the downdrift beaches. It was shown that the rocky and pocket beach nature of the coastline between Ballina and Lennox Head are such that:

- Only limited erosion occurs due to the bedrock controls of the headlands and underlying reefs; and
- A substantial proportion of the sand losses caused by the training walls are transferred north relatively quickly to Lennox Head and Seven Mile Beach.

The modelling indicated that the downdrift erosion from the training walls had a major effect on Lennox Head over several decades from prior to 1947, decreasing since about 1980 as the longshore sand transport supply resumed. There is evidence of shoreline recovery ay Lennox Head over the past decades, although this is most probably an effect superimposed on a longer term trend of recession. The modelling also indicated that the effects of the training walls have not yet affected the beaches at Suffolk Park or Tallow Beach, although there is potential for the erosion to be felt there over the next 100 years.

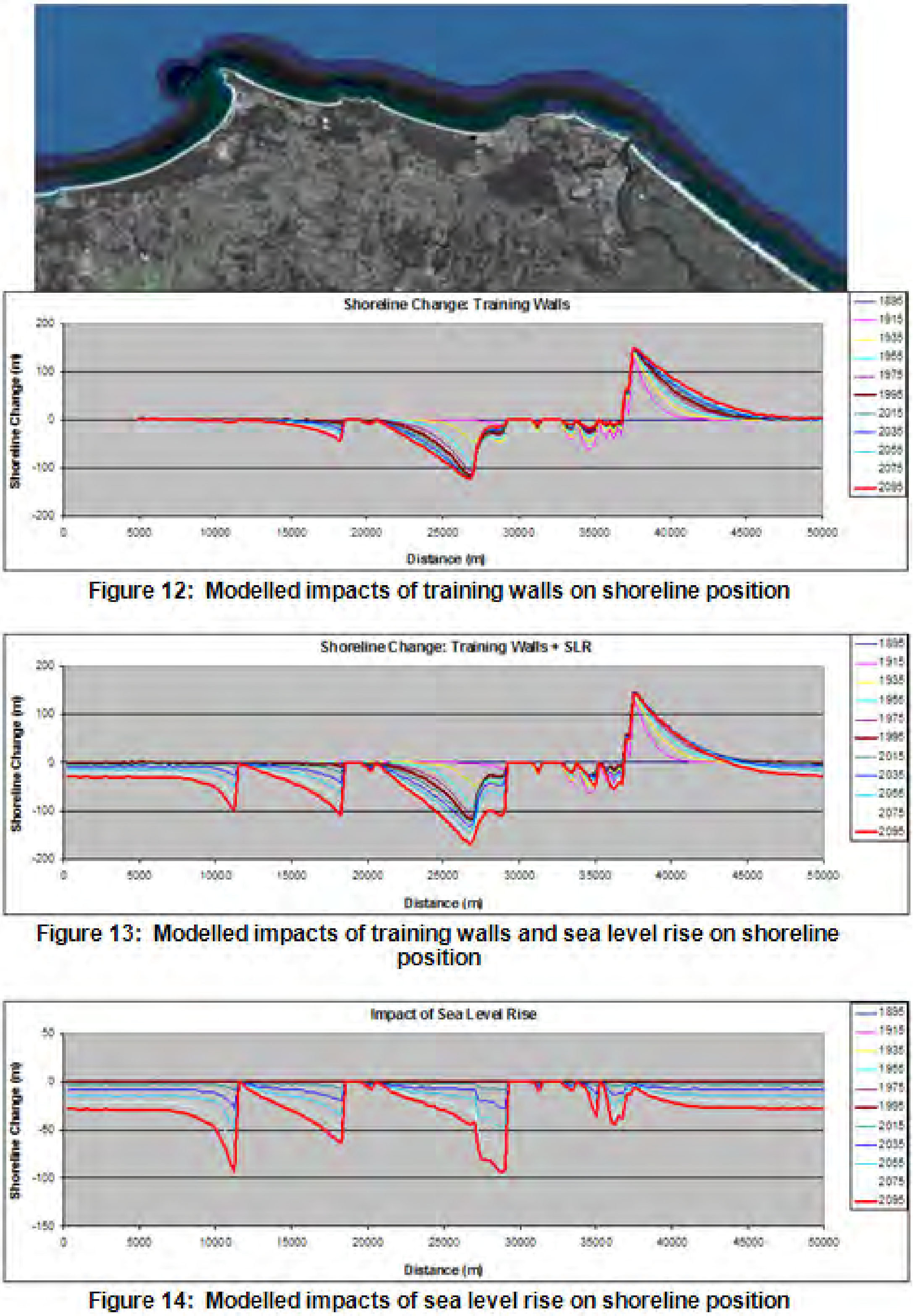


Figure 2-2 Modelled Impacts of Richmond River Training Walls (from Patterson, 2009)

An assessment of the impact of dredging the entrance channel to the Richmond River to improve navigation and not placing the sand in the active zone was also carried out by BMT WBM in 2013 with similar though less significant impacts.

Therefore, it is evident from these studies that any future works at the Richmond River mouth such as dredging or extensions to the walls will potentially impact on the beaches north to Lennox Head and Seven Mile Beach.

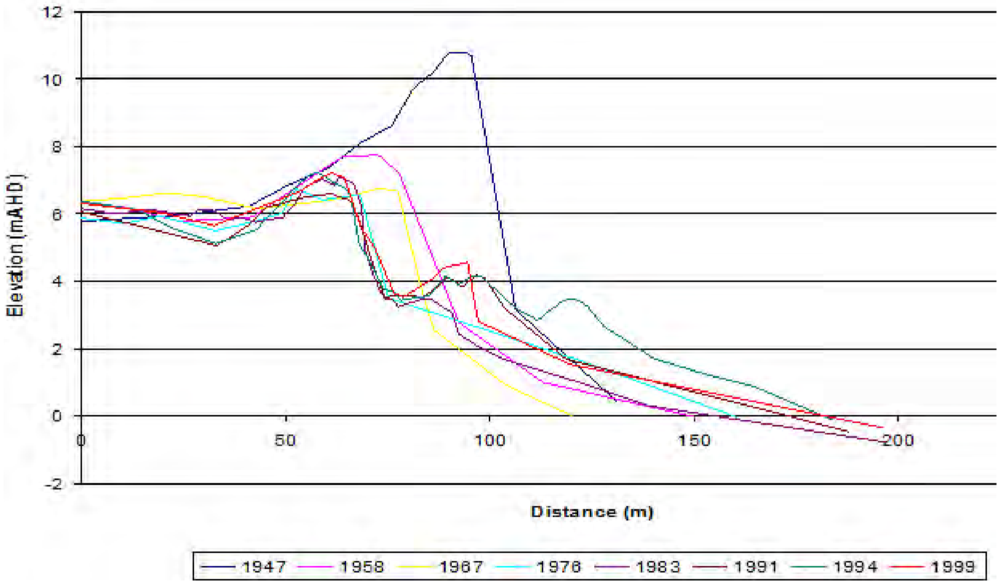


Figure 2-3 Photogrammetric Analysis of Beach and Dune Changes Seven Mile Beach (from WBM Oceanics Australia 2003)

2.5 Sand Mining

Sand mining for heavy minerals which took place extensively during the 1950s, 1960s and 1970s involved substantial relocation of dune sand and removal of the heavy mineral component. Some 1-3% of the sand volume within the mined area was removed. Most of this was from the dunal system landward of the contemporary active beach. Hence, its volumetric contribution to the regional sand loss from within the active littoral system cannot be quantified, but is relatively small.

As well, there is evidence of considerable relocation of dune sand associated with development of the coastal villages and roads in the region. Sand in the higher main dune areas was commonly pushed landward and/or along the coast to fill low hind-dune areas where the townships and roads now exist. The extent to which this sand would have been involved in the active beach/dune erosion/accretion processes over time to date is uncertain and cannot feasibly be quantified. However, it is likely to be relatively minor as most of the sand came from areas behind the apparent active dune scarp.

3 Coastal Hazards Assessments

3.1 Coastal Hazard Processes

The behaviour of the beaches in northern New South Wales is characterised by:

- Wave-induced longshore transport of sand, with a strong net transport to the north;
- Onshore/offshore movements of sand associated with relatively short term storm-related erosion and subsequent rebuilding of the beach and foredune;
- Wind-induced transport of sand from the beach to the back-beach dune system; and
- At some locations, effects of stream entrance movements and/or movements of beach sand into and from lower tidal estuary areas under the influence of tidal and flood flows.

There has most probably been some past shoreline retreat resulting from sea level rise in recent decades, expected to accelerate significantly in the future.

Any or all of these processes may be occurring at any time, depending on prevailing wave, wind and tide conditions. The resultant beach behaviour is one of constant change with substantial movements of the beach and foredune in the short to medium term (days/weeks/years) but only gradual progressive movements of the mean shoreline alignment in the longer term (decades/centuries).

The present study to define the coastline hazards for seawall design purposes at Lennox Heads requires:

- Adequate data and knowledge on the wave, tide and elevated water levels at the beach;
- Adequate data and knowledge on the processes of sand transport affecting the beach;
- Adequate data on beach, dune and shoreline changes over a substantial period of years;
- A good understanding of the spatial and temporal variability of the beach/dune system to facilitate proper interpretation of the available data;
- An understanding of the plan shape and elevation of the offshore reefs that protect part of the shoreline; and
- An assessment of the likely scour level at the toe of any proposed seawall and combined with surge and sea level rise an appropriate depth limited design wave height for the seawall.

It has been identified that there are no significant net inputs of sand to the Ballina Coast beach system other than that associated with longshore transport processes. The Holocene onshore supply is thought to have essentially ceased about 2000 years ago. Only the Richmond River has the potential to supply fluvial sand to the coast, but this is considered at present to be minimal.

As such, the available knowledge of the average annual longshore sand transport rates and differentials provides a most useful basis for assessment of the longer term progressive trends of shoreline change.

As well, cross-shore exchanges of sand occur within the overall sand budget with no net loss or gain of sand from the active system. There are short term transfers of sand from the beach/dune area to the nearshore profile in storms and progressive gradual return of the sand to shore over months to years by the action of day to day swell waves. Thus, the beach may appear eroded or exceptionally accreted without any overall net loss or gain of sand, depending on the occurrences or otherwise of storm events.

The available photogrammetric data provides a useful quantitative basis for confirming both the shorter term shoreline variability and any longer term retreat of the shoreline by both:

- Analysis of volumetric changes in the quantity of sand in the beach/dune system; and
- Measurement of any progressive longer term retreat of the predominant erosion escarpment in the main dune barrier.

However, because the photogrammetry covers only that part of the active profile visible above the water and does not identify sand moved temporarily to or from the nearshore profile, meaningful volumetric analyses require the context of each date of photography with respect to cross-shore sand movements associated with storm/cyclone erosion and beach re-accretion to be understood.

As well, any influence on volumetric analysis results of cyclic or short term effects such as the impacts of coastal works on parts of the beach system need to be identified and understood to avoid inappropriate extrapolation of patterns and trends to the future.

3.1.1 Wave Climate

3.1.1.1 Regional Wave Climate

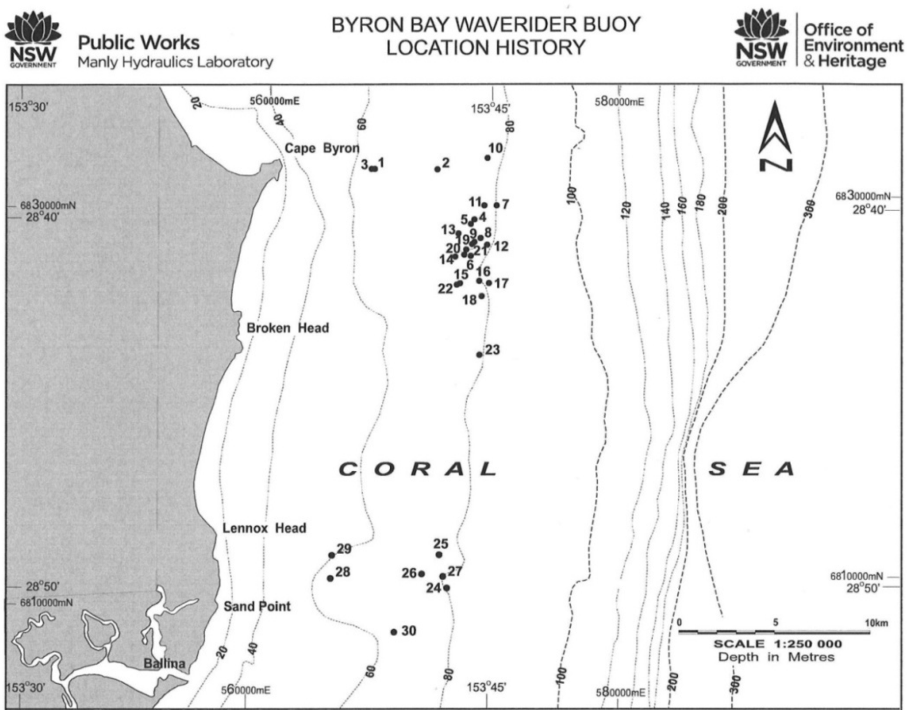
The regional wave climate is a dominant component of coastal processes. The deep water wave climate of the northern NSW coast comprises a highly variable wind wave climate superimposed on a persistent long period low to moderate energy swell predominantly from the southeast to east direction sectors. Typically, the swell may range up to 3-4m significant wave height with periods in the range 8 to 15 seconds. Prevailing wind waves are incident from a wider range of directions, predominantly the east to southeast sectors, consistent with the wind climate for the region, and range from small short period local ‘sea’ conditions to large storm and cyclone waves in excess of 6-7m significant wave height.

Wave data for Byron was provided for the study by MHL from the directional wave rider buoy moored in around 75-80 m water depth about 10 km offshore. The recorder location has been moved over the years in order to reduce the impacts of the East Australian Current on buoy stability and transmission/recorder failures that cause gaps in the data record. The locations are documented in Figure 3-1. Directional wave data has been recorded only since late 1999, with complete annual directional data sets available from January 2000.

Other wave data sources utilised primarily to extend and fill gaps in the Byron database include deep water WaveWatch III (WWIII) global wave model information since 1992 and British Meteorological Office (BMO) wave model information for the period 1989 to 1995. These data were cross-referenced against published data from the Brisbane recorder, offshore from Point

Lookout, although that information is known to be not sufficiently representative of prevailing conditions along the study region, particularly south of Cape Byron (Patterson 2007a).

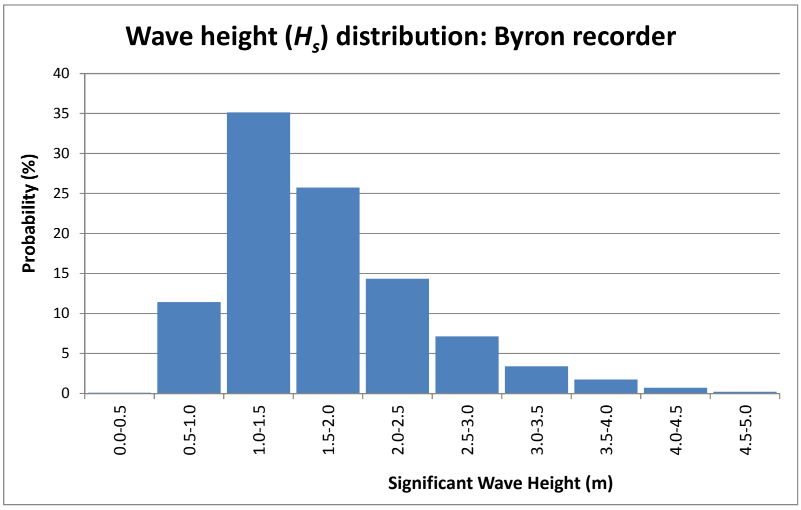
Basic wave parameter statistics derived from the recorded Byron data for years 2000 to 2012 are presented in Figure 3-2 in terms of significant wave height and spectral peak wave period and in Figure 3-3 for wave direction.



DEPLOYMENT LOCATION	Latitude (S)	Longitude (E)	GDA (Zone 56) Easting	GDA (Zone 56) Northing	WATER DEPTH (m)	First Date	Last Date
1	28°38'24"	153°41'18"	567280	6831690	64	14-Oct-1976	07-Jun-1978
2	28°38'24"	153°43'18"	570530	6831670	70	03-Aug-1978	13-Jun-1979
3	28°38'24"	153°41'12"	567110	6831690	62	08-Aug-1979	09-Aug-1983
4	28°39'48"	153°44'30"	572470	6829080	77	09-Aug-1983	13-Dec-1983
5	28°39'54"	153°44'24"	572310	6828890	77	07-Feb-1984	25-Sep-1984
6	28°40'48"	153°44'24"	572300	6827230	73	25-Sep-1984	30-Jun-1985
7	28°39'24"	153°45'12"	573620	6829810	80	27-Aug-1985	22-Nov-1985
8	28°40'18"	153°44'42"	572790	6828140	78	12-Dec-1985	24-Mar-1987
9	28°40'25"	153°44'31"	572480	6827950	78	24-Mar-1987	19-Nov-1987
10	28°38'05"	153°44'54"	573150	6832250	77	03-Dec-1987	07-Apr-1988
11	28°39'24"	153°44'49"	572980	6829800	77	18-May-1988	07-Nov-1988
12	28°40'30"	153°44'55"	573130	6827780	82	06-Dec-1988	08-Dec-1988
13	28°40'12"	153°44'00"	571650	6828350	72	10-Jan-1989	05-Aug-1989
14	28°40'49"	153°43'55"	571500	6827200	71	29-Aug-1989	14-Dec-1989
15	28°41'35"	153°44'03"	571730	6825790	74	07-Feb-1990	06-Dec-1990
16	28°41'30"	153°44'40"	572730	6825950	73	06-Dec-1990	08-May-1991
17	28°41'33"	153°44'59"	573240	6825840	78	29-May-1991	14-May-1992
18	28°41'55"	153°44'46"	572880	6825170	73	14-May-1992	18-Jun-1993
19	28°40'28"	153°44'26"	572360	6827850	73	23-Jun-1993	21-Jul-1993
20	28°40'46"	153°44'12"	571970	6827300	72	21-Jul-1993	11-Nov-1993
21	28°40'37"	153°44'15"	572060	6827570	72	01-Dec-1993	20-Jul-1994
22	28°41'36"	153°43'57"	571560	6825760	72	20-Jul-1994	05-Feb-1996
23	28°43'32"	153°44'40"	572700	6822180	72	05-Feb-1996	28-Nov-2001
24	28°50'09"	153°43'43"	571080	6809970	71	29-Nov-2000	23-Jan-2001
25	28°49'14"	153°43'38"	570950	6811670	71	10-Feb-2001	29-Aug-2003
26	28°49'44"	153°43'08"	570030	6810570	71	29-Aug-2003	12-Aug-2004
27	28°50'02"	153°43'24"	570570	6810200	71	12-Aug-2004	01-Jan-2005
28	28°49'36"	153°39'48"	564720	6811040	62	04-Feb-2005	11-Dec-2007
29	28°49'21"	153°39'56"	564940	6811500	62	11-Dec-2007	20-Aug-2008
30	28°51'14"	153°42'07"	568470	6808000	62	20-Aug-2008	Present

December 2011

Figure 3-1 Location History of the Byron Waverider Buoy (courtesy MHL)



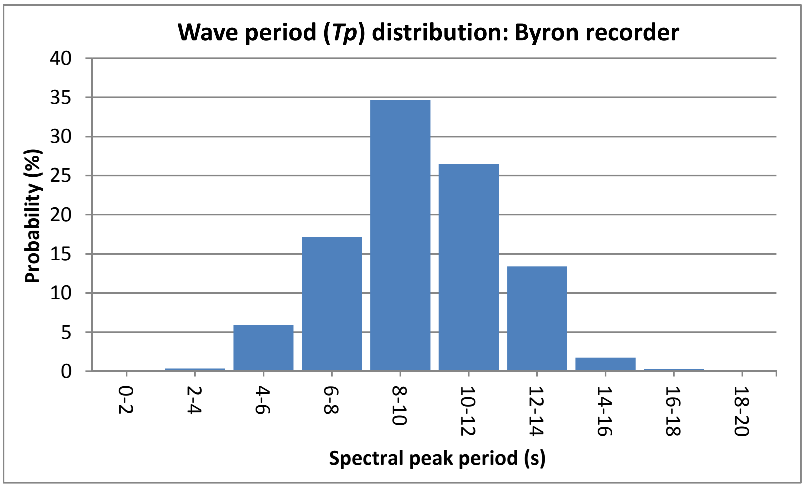


Figure 3-2 Occurrence Probability of H_s (top) and T_p (bottom) at Byron Recorder

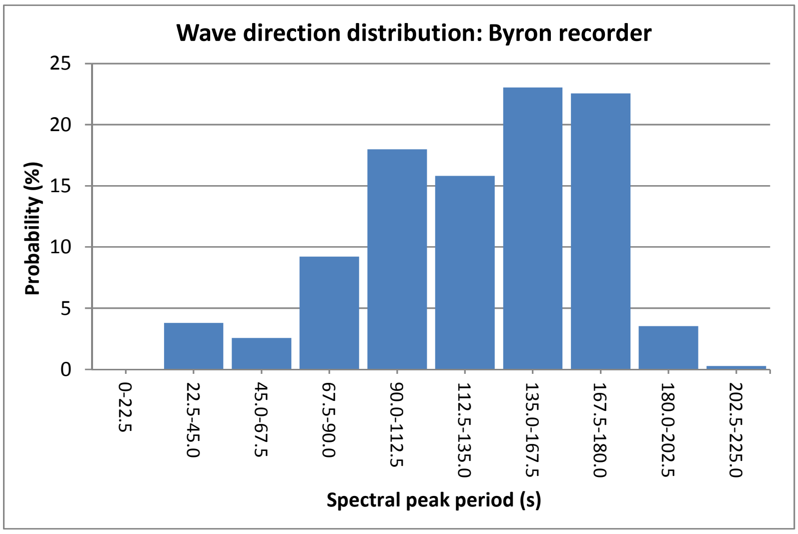


Figure 3-3 Occurrence Probability of Wave Direction at Byron Recorder

Figure 3-3 illustrates the predominance of southeast sector wave directions, showing that there is a relatively large proportion (approx. 26%) from directions south of southeast, in the range 157.5 to 202.5 degrees. Modal wave heights are 1.0-1.5m with spectral peak periods predominantly (~63%) in the range 8-12seconds.

The distribution of prevailing wave heights recorded at Byron is illustrated also in Figure 3-4, which shows their probability of exceedance calculated using the recorded data from January 2000 to July 2012. The median height is approximately 1.4m and 10%, 5% and 1% exceeded heights are 2.65m, 3.2m and 4m respectively.

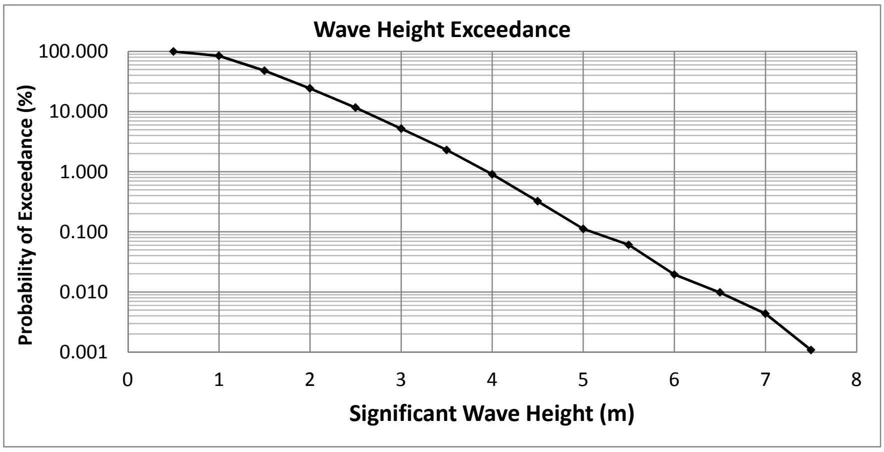


Figure 3-4 Exceedance Probability of Wave Height at Byron Recorder

3.1.1.2 Extreme Waves

Estimates of extreme deep water wave statistics have been calculated for the region from the set of peak storm wave height data collected by the Beach Protection Authority of Queensland over the period 1977 to 1999 (Allen and Callaghan 1999) for the recording site off Point Lookout, North Stradbroke Island. The two dominant types of storm wave, east coast low and tropical cyclone were considered. Table 3-1 shows their extreme wave estimates, also including the results for 1 hour exceedances at Byron from Kulmar *et al* (2005) based on the Byron data 1976 to 2004 for comparison, showing close agreement for the more extreme conditions. The Allen and Callaghan (1999) analysis results are also shown graphically, as plotted in Figure 3-5.

Table 3-1 Extreme Significant Wave Height Estimates

Average Return Interval (years)	East Coast Lows Hs (m)	Tropical Cyclones Hs (m)	Combined Hs (Allen & Callaghan) (m)	Byron1 hr Hs Kulmar *et al* (2005) (m)
2	4.85	3.89	5.02	5.4
5	5.67	4.60	5.83	6.0
10	6.10	5.20	6.29	6.3
20	6.47	5.83	6.71	6.7
50	6.90	6.73	7.28	7.3
100	7.20	7.46	7.75	7.6

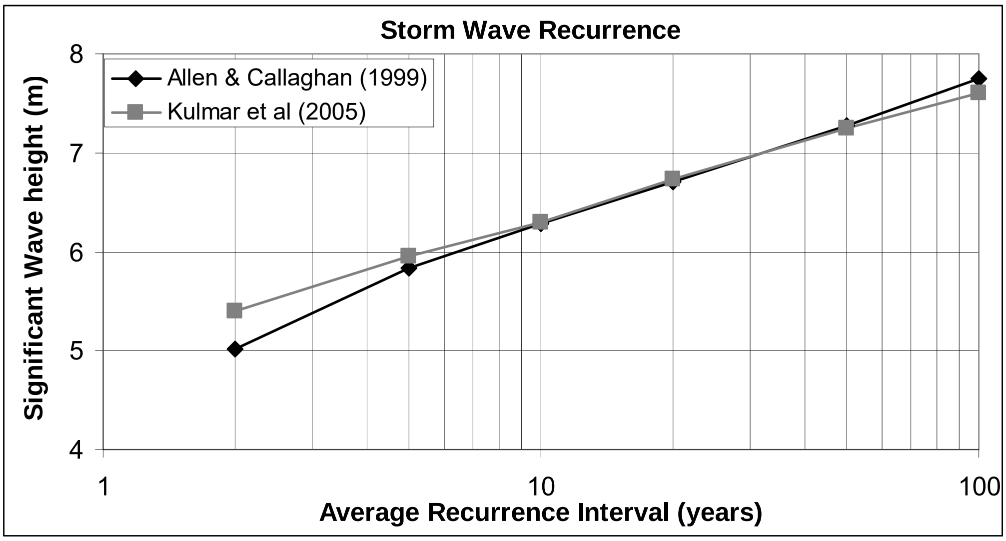


Figure 3-5 Storm Wave Recurrence Intervals for the Byron Region

Storm waves from easterly trough lows and tropical cyclones may approach from the east-northeast to south-southeast. The largest storm waves are associated with tropical cyclones and extra-tropical east coast lows. Large southerly swells often result from intense low pressure systems off the New South Wales coast.

These extreme wave height statistics correlate quire well with the highest recorded significant wave height (Hs) of 7.64m on 21st May 2009 (Figure 3-6). In that event, the recorded H_s of 6m was exceeded for 37 hours, 6.5m for 8 hours and 7m for 2.4 hours.

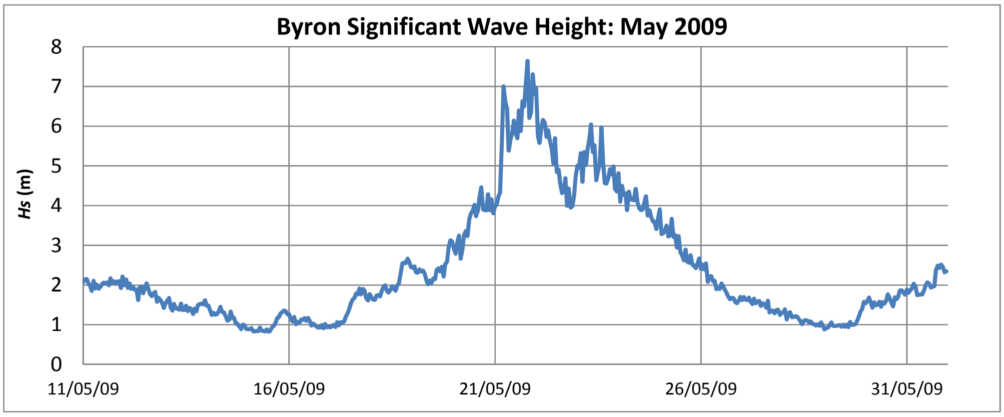


Figure 3-6 Storm Wave Recurrence Intervals for the Byron Region

However, it must be noted that the Byron recorder is located on the continental shelf where there is some effect of the seabed on the longer waves (> approx. 10s period) in the form of refraction, bed friction attenuation and shoaling. The larger storm event waves typically have a spectral peak period of 10-13 seconds and will be thus affected, with refraction having the greatest effect. At Byron, easterly waves will be affected negligibly due to the water depth and bathymetry shape involved while (for example) a recorded large (e.g. 7.5m) southeast wave of 13s period will be reduced by about 2.5% by refraction from deep water to the recorder. That is, in this example, the initial height in deep water would be about 7.7m. More southerly waves would be reduced further by refraction, around 8-10% for a south-southeast deep water direction.

Further, the earlier recorder locations prior to 1996 were susceptible to Waverider buoy interference by the East Australian Current such that it pulled under during large waves, thus experiencing drop-out failures and lost data during storm events. Caution is therefore needed in interpreting the results of statistical analyses from the recorded data.

Thus, while some uncertainty exists about extreme wave statistics relevant to this region, a 100 year ARI deep water design wave height of 7.5m has been adopted, with an indication that it has a mean duration in the range 1 to 6 hours.

3.1.2 Elevated Water Levels

In an open coastal situation, the components which contribute to elevated ocean water levels that influence beach erosion and potential foreshore overtopping and inundation during storms include:

- Astronomical tide;
- Inverted barometric setup;
- Wind setup;
- Wave setup; and
- Wave run-up.

Sea level rise will also contribute to elevated ocean water levels in the future, and must be considered in any assessment of shoreline recession and inundation hazards.

3.1.2.1 Astronomical Tide

Forces caused by the gravitational attraction of the Moon, the Sun and the Earth result in the periodic level changes in large bodies of water. The vertical rise and fall resulting from these forces is called the astronomical tide. Tides of the NSW coastline are classified as semi diurnal with significant diurnal inequalities, with two high tides and two low tides per day that are generally at different levels (i.e. the two high tide levels are different in any one day).

Astronomical tides are well understood and can be predicted on the basis of their harmonic constituents. The variation of Mean High Water Springs along the NSW coast derived from the constituents (M2+S2) is illustrated in Figure 3-7, indicating only a slight increase of about 10cm along about 1,000km of coastline. This is reasonably consistent with the recent analysis by Manly Hydraulics Laboratory (2011) which indicates an increase of slightly more than 20cm in the total tidal range, defined by MHL in terms of (M2+S2+1.2*K1+1.2*O1), corresponding to 10cm increase in the total amplitude. The predicted tidal levels derived from constituents for the Tweed-Byron region are shown in Table 3-2. There is only about 3cm difference between MHWS there and at Fort Denison in Sydney.

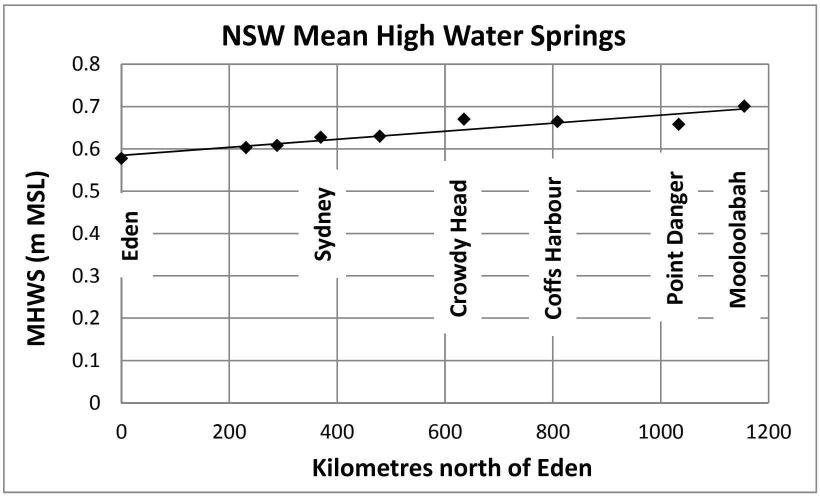


Figure 3-7 Mean High Water Springs Variation along the NSW Coast

Table 3-2 Tidal Statistics for Tweed-Byron Region

Tidal Plane	m AHD
Highest Astronomical Tide (HAT)	Approx. 1.0
Mean High Water Springs (MHWS)	0.66
Mean High Water Neaps (MHWN)	0.37
Mean Sea Level (MSL)	0.0
Mean Low Water Neaps (MLWN)	-0.37
Mean Low Water Springs (MLWS)	-0.66
Lowest Astronomical Tide (LAT)	-1.0

Derived from tidal constituents: Source Australian National Tide Tables

3.1.2.2 Storm Tides

DECCW (2010a) analysed average recurrence interval (ARI) water levels for use in coastal assessments in NSW. The design storm tide (tide plus surge) water levels applicable in the Tweed-Byron region are given in Table 3-3. The levels in Table 3-3 compare reasonably well with those derived from other studies for the local Gold Coast region by James Cook University (1977) and McInnes *et al* (2000), as outlined in Table 3-4.

For 2050 and 2100 assessments, the ocean levels also include projected sea level rise from year 1990 at the previous NSW Government's benchmarks of 0.4 m and 0.9 m by 2050 and 2100 respectively. Note also that the levels in Table 3-3 account for the sea level rise of 0.06 m that has been recorded already between 1990 to 2010 (DECCW 2010b). A small increase in storm surge heights (1–3cm) associated with future climate change has been projected by McInnes *et al* (2007). This has been incorporated into the assessment of elevated ocean water levels for future time periods in the coastal inundation hazard assessments.

Table 3-3 Design Elevated Water Levels (DECCW 2010)

ARI (years)	2010 (m AHD)	2050 (m AHD)	2100 (m AHD)
0.1	1.08	1.44	1.94
20	1.38	1.72	2.22
100	1.44	1.78	2.28

Table 3-4 Comparison of Design Storm Tide Levels from Various Studies

Recurrence Interval (years)	Fort Denison (DECCW 2010a) (mAHD)	Gold Coast (JCU 1977) (mAHD)	Gold Coast Seaway (McInnes *et al* (2000) (mAHD)	
			Including setup	Excluding setup
Immediate to 20yr	1.08	1.24		
50	1.38	1.30	1.9±0.1	1.1-1.2
100	1.44	1.35	2.1±0.1	1.3-1.4

3.1.2.3 Wave Set Up

As waves approach a beach across the surfzone they cause changes in the mean water level which is associated with gradients in the radiation stress of the wave train (i.e., the pressure force in excess of hydrostatic pressure caused by the presence of waves). Once waves have broken, kinetic energy is released and the mean water level is raised, sometimes substantially above the still water level. Maximum setup occurs at the beach face. The amount of setup depends on wave height, wave steepness and beach slope.

Although wave setup along the open coast shoreline is reasonably well understood there is growing evidence that propagation of wave setup through estuary entrances is minimal. Measurements documented by Hanslow and Nielsen (1993) from the Brunswick River entrance (NSW north coast) indicated that even when waves were breaking across the entrance, measurements of mean water surface extending up-river for some 200 to 300m showed only a very small transfer of wave setup. The maximum wave setup within the entrance was found to be less than 3% of the offshore wave height.

However, wave setup contributions to elevated water levels in the ocean can affect estuaries or stormwater discharge outlets by acting to impede the outflow of water during flood events. That is, the hydraulic gradient between outflowing flood waters and the ocean may be reduced where ocean levels are high, exacerbating flooding upstream in the estuary or stormwater system.

Normally, wave setup is incorporated in the calculations of wave run-up, the key factor considered herein in terms of inundation related to wave effects.

3.1.2.4 Wave Run-up

Wave run-up is the vertical distance on the shore that the uprush of water from a breaking wave reaches above the local mean sea level. It is the wave run-up mechanism that governs the volume of water that overtops a coastal barrier, for example, dunes, seawalls and entrance berms. Wave run-up levels are dependent upon factors including wave height, wave period, storm surge, beach slope and permeability, the roughness of the foreshore area and wave regularity. Run-up is more severe on steeper slopes and impervious materials, which means that steep-sloped grouted rock seawalls will generate much higher run-up than gently sloped beaches.

Wave run-up is variable due to the irregular nature of waves and is commonly assumed to have a Rayleigh statistical distribution matching that of the prevailing waves.

For inundation hazard definition, the rate and frequency of overtopping is an important consideration when determining the effectiveness of protection offered by existing seawalls, particularly with future sea level rise. Analyses of wave run-up levels and the associated potential for significant wave overtopping have been undertaken, including provision for sea level rise at 2050 and 2100.

3.1.3 Climate Change

3.1.3.1 Sea Level Rise

The previous NSW Sea Level Rise Policy Statement (DECCW 2009a) provides for an increase in mean sea level above 1990 levels of 0.4 m by 2050 and 0.9 m by 2100 but has been withdrawn. However, Council has adopted these values in its own policy. The Office of Environment and Heritage (OEH) has advised that an estimated sea level rise of 0.06 m between 1990 and present should also be considered in coastal assessments. The sea level rise provisions adopted for this study are thus 0.34m by 2050 and 0.84m by 2100.

3.2 Assessment of Hazards

Each of the hazards mentioned in previous sections are assessed in term of their impact on the existing and potential new seawall in this section.

3.2.1 Beach Erosion (Storm Bite)

3.2.1.1 Erosion Processes

During severe storms or a series of storms in succession, increased wave heights and elevated water levels results in wave attack of the beach berm and foredune region. Storm events generate high rates of transport of sand both:

- Offshore, with sand eroded from the beach face and transported to the nearshore seabed to form a sand bar roughly parallel to the shoreline; and
- Alongshore (i.e., along the beach) either upcoast or downcoast depending on wave direction, with gradients in the transport rates leading to erosion or accretion.

The result is erosion on the beach face and dune that may pose a hazard to back beach land and assets. The short term storm related cross shore sand transport and longshore drift occur simultaneously, the latter commonly leading to a significant shoreline erosion component immediately downdrift of headlands in cases where the sand supply into the beach compartment is less than the transport away to the north. Their effects are additive, although the beach itself (above mean sea level) will be observed to erode predominantly during storm events.

The extent of storm erosion that will occur under the same set of water level and wave conditions may vary. This is because the volume of erosion relates also to:

- The occurrence, location and strength of rip current cells, which promote seaward transport of sediment and may allow larger waves access to the beach face, resulting in further localised beach erosion;
- The state of the beach (eroded / accreted both on land and underwater) immediately prior to the storm; and
- Adjacent headlands or coastal structures that can modify local wave conditions and the supply of sand during the storm event.

On average, stable beaches exhibit a form of dynamic equilibrium. Following periods of large-scale short term erosion, the beach will tend to restore itself over time to an average or accreted state during favourable wave conditions. This recovery involves the shoreward return of sand from nearshore and/or, where the erosion resulted from alongshore losses, a sand supply from updrift that exceeds the transport away, commonly associated with headland bypassing processes.

On beaches that are in long term 'dynamic equilibrium', the amount of sand that returns to the beach is equal to the amount eroded during the storm. However, at beaches experiencing long term recession, not all the sand eroded may be returned and the eroded dune escarpment will move landward on average over time.

3.2.1.2 Storm Bite Assessment

During storms, increased wave heights and elevated water levels cause sand to be eroded from the upper beach/dune system (often termed ‘storm bite’) and transported in an offshore direction, typically forming one or more shore-parallel sand bars in the nearshore zone. As the sand bars build up, wave energy dissipation within the surfzone increases and wave attack at the beach face reduces. The severity of wave attack at the dune is dependent on wave height and elevated water level (the combination of tide, storm surge and wave setup) and preceding beach condition (i.e. if the beach is accreted or eroded prior to the storm). In addition, depending upon the orientation of the coastline relative to the direction of the incoming storm, the beach may either experience unimpeded wave power and severe erosion, or may be shadowed and protected from incoming wave energy.

During calmer weather, sand slowly moves onshore from the nearshore bars to the beach forming a wave-built berm and, subsequently, a wind-formed incipient foredune.

Typically, the cross-shore exchange of sand from the upper beach/dune area to the nearshore profile does not represent a net loss or gain of sand from the overall active beach system. While it may take several years, the sand eroded in the short-term during severe storms is returned to the beach and dune by the persistent action of swell waves and wind such that there is overall balance. In addition, for stable beaches, the longshore transport into and out of the compartment is equal over the long term, enabling an overall balance in the cycle of storm erosion and recovery.

The volume transported offshore from the beach and dune is known as the storm demand or storm bite. The amount of linear recession of the dune associated with such volume loss is related to the beach/dune profile and height. In this regard, the amount of beach rebuilding that may have occurred since the last storm and the volume of sand reserves in the dune system or nearshore areas are important.

The storm bite for a particular event is typically less if it follows previous erosion which has formed an offshore bar which causes waves to break and dissipate energy further offshore. Correspondingly, highly accreted beach/dune areas with depleted nearshore profiles tend to experience greater storm bite.

The extent of short term beach erosion can be influenced by the presence of bedrock, because it may restrict cross shore sand movement and dissipate significant amounts of wave energy. The extent may also be influenced by adjacent headlands or nearshore reefs coastal structures which can modify local wave conditions and the supply of sediment to the immediate downdrift beaches during a storm event.

Photogrammetric data provides information on changes to beach volume and the position of dunes over time. While inaccuracies can be common in older dates of photogrammetric data, all dates of photogrammetry were found to be accurate for analyses in this study. Photogrammetry provides data on changes above mean sea level, therefore consideration of longer term trends is based primarily on movements of the upper beach/dune system. However, the photographs present individual ‘snap-shots’ that describe beach state at one particular time. Knowledge of the timing and intensity of major historical storm erosion events is taken into account in interpreting the available data.

The photogrammetric data has been processed to calculate beach / dune volumes for each profile cross-section, and both average and cumulative volumes along representative sections of shoreline analysed. The envelope of volumetric variability in the photogrammetric data over a period of several years or decades may provide a measure of the potential storm bite volume even where the data does not relate to any particular storm event, provided any long term trends are taken into account. This takes account of both storm erosion and short term (months to years) variability due to alongshore fluctuations. As well, the horizontal distances to several specified level contour positions have been determined to indicate beach width variability and any movements of the dune face. For this study, distances to the +1.5m, +2.5m and +4m contours have been analysed, with movements in the +4m contour indicating any progressive shift over the long term in the extent of storm erosion, also an indicator of long term recession.

Review of photogrammetric processing methods by Hanslow (2007) concluded that both the horizontal movement of a selected dune contour position and the sub-aerial beach volume calculation have statistical significance to be appropriate for use in hazard assessments. Both of these methods have advantages and disadvantages. Both the sub-aerial beach volume data (cumulative block volumes, individual profile volumes) and dune contour position movements have been used to assess beach erosion potentials, as well as historical long term shoreline trends.

The results obtained in this way show that generally the largest volume losses along Seven Mile Beach were experienced in the period between 1958 and 1967. Figure 3-8 shows the maximum volume loss per length of beach, derived from two consecutive photogrammetry surveys.

Figure 3-8 shows that the maximum volume loss along those shoreline sections where the presence of bedrock in the active beach profile is limited (Profile 35 and beyond) are typically in the range 150-300m^3/m, consistent with experience at fully exposed ocean beaches elsewhere along the NSW coastline. However, south of Profile 35 the maximum volume losses are notably lower, particularly south of Byron Street (Profile 1 to 20) where the maximum historical volume loss is generally less than 50m^3/m. The lower volume losses along the section south of Profile 35 can be attributed to the shallow bedrock that is present along this section of the coast.

Along the study area (Profile 20 to 38), the historical volume losses vary along the shoreline. Along the southern most 600m section (between Byron Street and Williams Street), the maximum volume losses are generally around 100-120 m3/m. North of Williams Street, as the nearshore reef become less profound, the maximum volume losses increase to approximately 200m3/m at Lake Ainsworth.

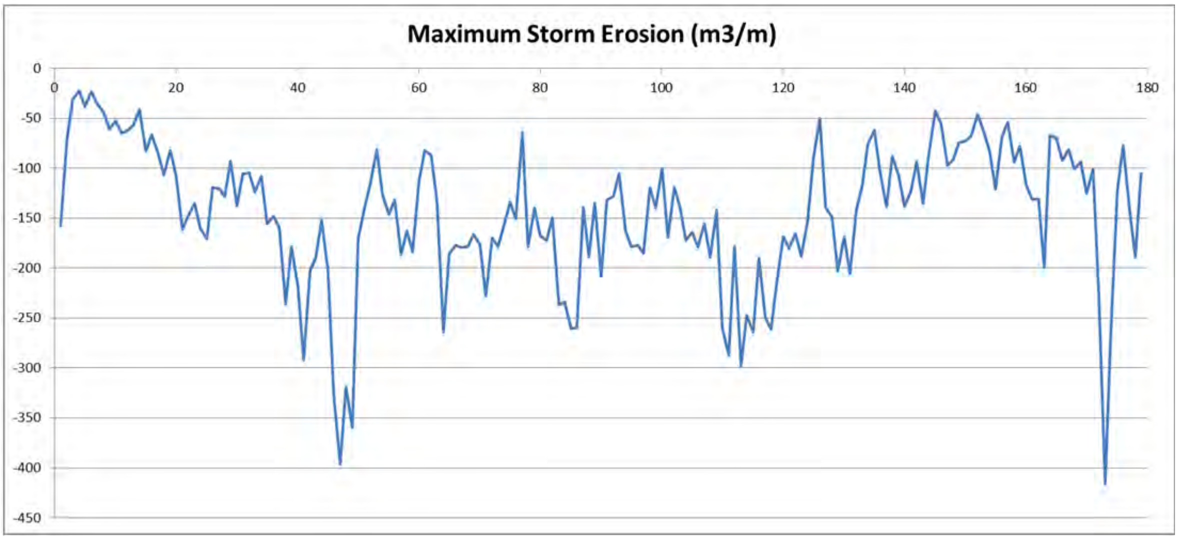


Figure 3-8 Historical Storm Bite Volumes along Seven Mile Beach

Each location is analysed on an individual basis. Generally, an attempt has been made to establish only one storm bite component for each location, based on the most recent accreted beach condition (typically 2007). Where uncertainty exists, it is feasible to adopt a range of values that may be incorporated into the probability spectrum in determining the erosion hazard lines.

3.2.2 Long Term Recession

The erosion hazard extents for the immediate, 2050 and 2100 planning times are based on the contemporary behaviour and forward projections of historical shoreline behaviour derived from the available data, together with analysis using either conventional coastal engineering methods or modelling of shoreline responses to sea level rise (SLR) and other likely climate change factors. The past behaviour comprises:

- Long term trends of shoreline change that relate to the geological evolution of the coastline regionally and will persist into the future;
- Short term storm erosion that will continue to affect the beaches much as it has to date;
- Short to medium term variability associated with variations in wave climate regime;
- Minor shoreline recession associated with the relatively slight sea level rise that has occurred over recent decades; and
- Anthropogenic influences such as coastal structures or sand mining interference with the beach/dune system.

Long term recession relates to the persistent and progressive existing trends of shoreline change that may be projected with reasonable confidence to the future. This needs to include also the projected progressive recession associated with future sea level rise. Superimposed on those trends are the reasonably well-defined cyclical effects of storm erosion and subsequent beach recovery.

3.2.2.1 Analysis of Historical Shoreline Recession

Historical shoreline recession trends may be identified most readily in the photogrammetry data in terms of either:

- Persistent progressive changes in the volume of sand contained in the beach/dune system; and/or
- Persistent and progressive changes in the position of the dune scarp.

Beaches experiencing long term recession are characterised by a persistent trend of reduction in the average sand volume and, often, a prominent back beach escarpment which moves landward over time. Net sand losses generally affect the nearshore area initially, typically due to alongshore gradients in the longshore sand transport rates. When the nearshore area has been depleted of sand progressively by longshore sand losses, the storm cut into the beach and dune will be unusually high and extend further landward than previously. In such a case, the beach will not recover to its former state.

Longshore sand losses create an overall net depletion of the active beach profile as retreat of the dune face, beach and nearshore profile down to a depth of about 10 metres, the typical limiting depth of longshore sand transport along the open coast. Thus, for a profile with dune height of 5 metres, only approximately one-third of the total volumetric sand loss occurs above mean sea level. This is an important factor in interpreting photogrammetric and survey data that only covers the upper beach/dune area.

It is feasible to identify such sand losses and thus the shoreline recession by analysis of the longshore sand transport rates. However, the database of recorded directional waves is limited and this approach is useful only where there is a significant transport gradient, typically along an extended coastline, Patterson 2007 found a substantial positive gradient in the longshore transport northward from the Clarence River to Point Danger of about 350,000-400,000m3/yr along 150km, corresponding to an average of about 2.3-2.7m3/m/yr. This would potentially lead to average shoreline recession for an active vertical zone of 15m (dune height of 5m to a littoral zone depth of 10m) of 0.15-0.18m/yr. However, it is likely that this is offset by some continuing shoreward sand supply to the beach system of at least 1m3/m/yr (Patterson 2013), reducing the average recession to less than 0.1m/yr. Further, the recession is not uniform along the coastline, being less immediately updrift (south) of headlands and greater downdrift (north).

Accordingly, long term recession rates at particular beaches are generally determined from analysis of volumetric and/or lineal movement trends derived from survey or photogrammetry data. Shoreline recession trends within the study region derived in that manner should be reasonably consistent with the regional average, but will vary depending on location relative to headland controls.

However, short to medium term variability due to wave climate variability may mask such a trend in data that is of insufficient length to isolate and identify the two processes. This is evident along the study region, most particularly in the embayment areas north of major headlands. In those cases, the underlying trend of change that could be extrapolated to the future may be difficult to quantify and needs to be interpreted in light of the patterns evident in the measured photogrammetry data and the best available knowledge of the prevailing wave conditions, to gain an understanding of the timing and extent of such variability. As well, natural short to medium term variability may be assessed using shoreline response modelling for the period of the input wave information.

Both the historical long term recession rates and provision for the variability must be incorporated into the assessment of long term recession in the future in combination with recession due to sea level rise, as outlined below.

The long term shoreline recession rate for Lennox Head has previously been assessed in the Ballina Shire Coastline Hazard Definition Study (WBM Oceanics Australia, 2003). This study provides a “best-estimate” long term recession rate of 0.5m/year.

For the assessment of the existing rock protection, the best-estimate” long term recession rate recommended of the Coastline Hazard Definition Study has been adopted for investigation of the likely future performance of the structure.

3.2.3 Sea Level Rise Impacts

3.2.3.1 Equilibrium profile (Bruun Rule) Concept

The study region beaches have evolved with mean sea level relatively constant at or near the present level over about 6,000 years to a condition of cross-shore dynamic equilibrium. That is, the profile shape across the beach/dune and nearshore areas to the lower shore-face has an equilibrium form about which cross-shore storm erosion and accretion seabed changes fluctuate. In principle, that equilibrium shape tends to be maintained relative to sea level as the sea level changes. This two-dimensional concept is demonstrated by the Bruun Rule, in Figure 3-9.

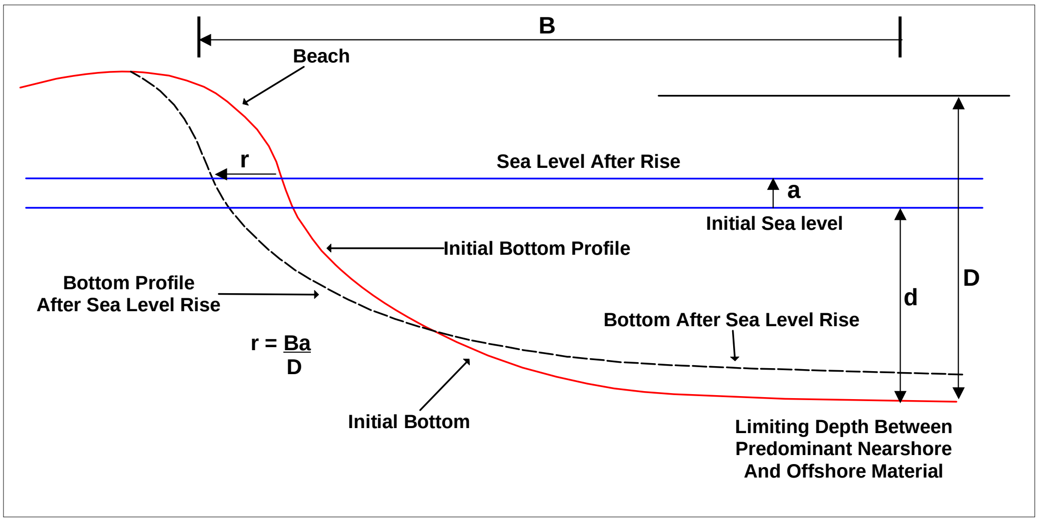


Figure 3-9 Bruun (1962) Concept of Recession due to Sea Level Rise

As the sea level rises, wave, tide and wind processes are occurring at a higher position at the beach face, with the beach and dune evolving to a more landward position to return to equilibrium with the new sea level. There is an upward and landward translation of the profile that is in equilibrium with the prevailing conditions at the new sea level position. Bruun (1962) has shown that the shoreline recession (r) may be estimated as Ba/D (as defined in Figure 3-1), where B/D represents the slope factor and the predicted recession is the slope factor times the sea level rise.

Application of this 'standard' simplified Bruun Rule has been highly contested within the coastal science community (e.g. Ranasinghe *et al*., 2007), often relating to the depth of closure to which the equilibrium shape is maintained. The depth of closure is generally adopted as the depth limit at which there is little or no potential for significant cross-shore exchanges of sand, but there has been conjecture surrounding what this depth may be. The DECCW (2010) *Coastal Risk Management Guide: Incorporating sea level rise benchmarks in coastal risk assessments* indicate the appropriate calculation of the depth of closure term required with the Bruun equation as follows: "*when using the 'Bruun Rule', use of the lower limit of profile closure (seaward limit of the Shoal Zone) as prescribed by Hallermeier (1981) is recommended in the absence of readily available information on active profile slopes at a location under consideration*". It has also been common practice along the NSW coastline to adopt generic active profile slope factors from the closure depth to the dune crest (Figure 3-10) in the range of 1:50 to 1:100.

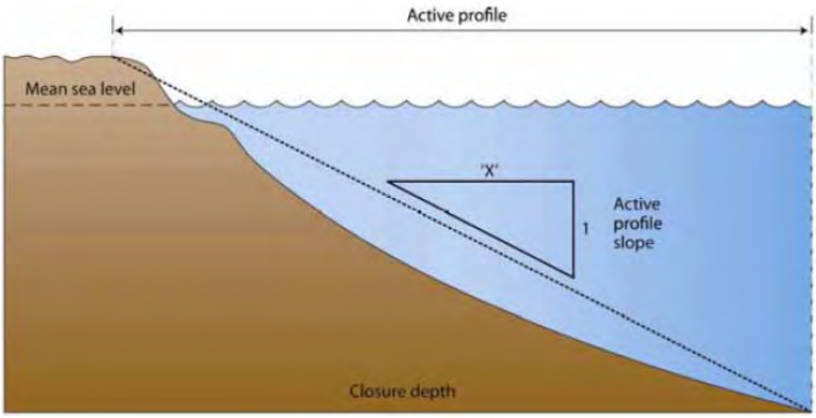


Figure 3-10 Idealised Schematic of the Active Profile Slope Applicable in the 'Bruun Rule' (from DECCW, 2010)

The previous coastline hazard assessments (WBM Oceanics Australia 2003, BMT WBM 2011) adopted a generic Bruun Rule slope factor of 50:1 for shoreline recession. For this assessment, this previously adopted Bruun Rule slope factor has also been adopted.

Thus, considering the adopted future sea level rise levels of 0.34m and 0.84m at 2050 and 2100 respectively, the Bruun Rule approach would yield recession provisions of 20 and 45m for the 2050 and 2100 planning timeframes respectively.

3.3 Dune Stability & Reduced Foundation Capacity

Immediately following storm erosion events on sand beaches, a near vertical erosion scarp of substantial height can be left in the dune or beach ridge. A zone of reduced foundation capacity can exist on the landward side of sand escarpments. This can impact on structures founded on sand within this zone and the sand escarpments pose a hazard associated with sudden collapse. Following such storm events, inspection of sand scarps should be undertaken to assess the need for restricting public access and the impact on structures.

Over time the near vertical erosion scarp will slump through a zone of slope adjustment to the natural angle of repose of the sand (approx. 1.5 Horizontal to 1.0 Vertical). Nielsen et al. (1992) outlined the zones within and behind the erosion escarpment on a dune face that are expected to slump or become unstable following a storm erosion event (see Figure 3-11), namely:

- *Zone of Slope Adjustment:* the area landward of the vertical erosion escarpment crest that may be expected to collapse after the storm event; and
- *Zone of Reduced Foundation Capacity:* the area landward of the zone of slope adjustment that is unstable being in proximity to the storm erosion and dune slumping.

Amongst other factors, the width of the zone of reduced foundation capacity behind the top of an erosion escarpment is dependent upon the angle of repose of the dune sand and the height of the dune above mean sea level. Table 3-5 provides an indicative guide to the width of the zone of reduced foundation capacity measured landward from the top of the erosion escarpment for various dune heights.

The defined zones should be added to the immediate, 2050 and 2100 year beach erosion hazard (i.e. taken to occur in a landward direction from the edge of the beach erosion extent). Climate change is not expected to modify soil stability, and thus the hazard extents remain relevant at the 2050 and 2100 year planning period.

The allowances in Table 3-5 assume a dunal system made up entirely of homogeneous sands (with an assumed angle of repose of 35 degrees) and makes no allowance for the presence of more structurally competent stratums, for example indurated sands and bedrock, nor do these allowances take account of water table gradients that may be present within the dunal system. Expert geotechnical engineering assessment is recommended to establish the structural stability of foundations located (or likely to be located) within the zone of reduced foundation capacity on a case by case basis.

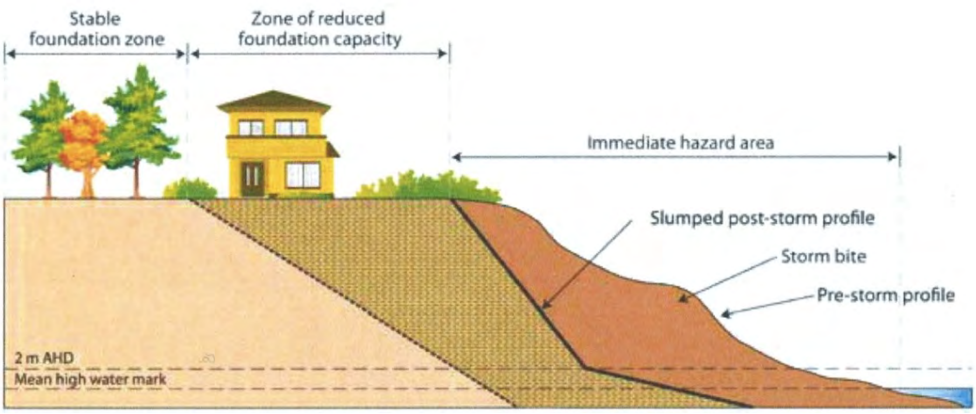


Figure 3-11 Design Profile and Zones of Instability for Storm Erosion (from DECCW 2010; after Nielsen et al 1992)

Table 3-5 Width of Zone of Reduced Foundation Capacity

RL of Dunal System (m AHD) [1]	Indicative width of Zone of Reduced Foundation Capacity (m) [2]
4	9.3
5	10.7
6	12.2
7	13.6
8	15.0
9	16.4
10	17.9

1 *Assumed that surface of dunal system is approximately level*

2 *Distance measured landward from the top of the erosion escarpment following slope readjustment.*

Following storm events where dune erosion has occurred, inspection of sand scarps in popular recreational beach areas should be undertaken to assess both the need for restricting public access and structural instability. The stability of existing and new building foundations in the vicinity of any erosion scarp will need to be assessed or designed by a qualified geotechnical engineer.

4 Characterisation of Existing Historical Seawall

4.1 Introduction

Subsequent to the severe shoreline erosion at Lennox Heads during 1966-67 significant community and Council effort was put into building defences against future erosion. These defences included a ti-tree fence and erosion scarp protection using a rock/soil mixture taken straight from the field to the beach. Anecdotal evidence suggests that these were ad-hoc measures with minimal design input and no as-built records kept. The structures were built using largely volunteer community workers assisted by Council in the form of equipment and materials. Subsequent beach recovery after the severe erosion covered the works with the current dunes extending well seaward of the structures.

Therefore, Council decided to use a combination of subsurface geophysical investigations to assess likely seawall location within the dune then and two trial physical excavations to uncover what remains of the original structures and calibrate the GPR assessment. This information would also be used to assess the likely ability to resist future erosion.

4.2 Initial Construction Technique

Many of the SE Queensland and northern NSW beaches suffered extensive erosion in 1966/7 due to a succession of cyclones and easterly trough lows. It appears that the immediate community response at Lennox Head to this dramatic erosion was to build a ti-tree fence. Figure 4-1 below shows the community involvement in this exercise that used 8000 poles that were jetted into the beach in July and August 1967. Figure 4-2 shows the fence and a facing of top soil and rock on the erosion scarp. Apparently this process of placing "paddock rock" on the erosion scarp was continued for up to 10 years with material containing more rock until it extended down to the ti-tree fence and in some locations included a trenched foundation toe.

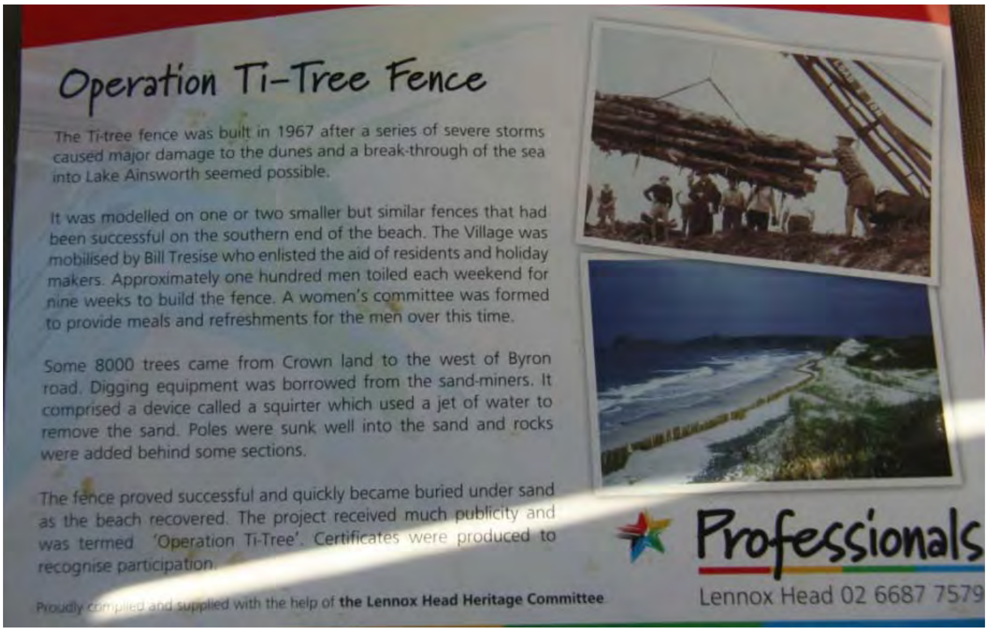


Figure 4-1 Operation Ti-Tree Fence Brochure

Figure 4-2 Ti-Tree Fence and rock placed on Erosion Scarp

4.3 Anecdotal Evidence

During the two days of excavation in October 2013 several people offered information regarding the construction methodology and materials used as well as showing photographs of the activity. In particular detailed information was obtained from Peter Thorpe, the retired Tintenbar Shire Engineer who supervised the work, as well as John Stewart and Trevor Newton who were community volunteers and/or council employees involved in the day-to-day construction. None of the people spoken to considered that this seawall was to an engineering standard but more an ad-hoc action to resist the immediate erosion threat. The wall was not exposed to significant further erosion as the beach recovered over time and currently a dune extends about 30m in front of the wall.

The following points were noted during the conversations.

Peter Thorpe:

- Joined Council late in 1967 after ti-tree fence was constructed;
- Supervised the rock seawall construction which was ad-hoc using rock “floaters” from local paddocks that were being sub-divided or built on;
- There was no design and no records were kept of where or when material was placed – it was seen as a general community benefit;
- Both the surface rock and top soil was deposited in front of the erosion scarp and the campaign continued intermittently for several years with the top soil providing good growing conditions for grasses which assisted in holding the material together;

- In some places the rock extended as far as the ti-tree fence – and in some places the ti-tree fence was broken off at ground level during rock placement;
- In some sections the rock was placed from RL 4m down to RL 0.5m with a trenched-in toe and about 1.5m to 2m thick;
- The seawall only extended to the Lennox Head – Alstonville Surf Lifesaving Club; and
- The seawall was never tested as the beach tended to accrete after the erosion event.

Trevor Newton:

- Worked with Council and involved in seawall work from time to time;
- Seawall not to any design just placing rock and associated to soil on the erosion scarp;
- Seawall extended to Lennox Head – Alstonville Surf Lifesaving Club – a different and better designed seawall was constructed by others at the Recreation Centre;
- The seawall was constructed over a period of 10 years; and
- Bigger rocks and better design for the seawall in front of Allens Parade.

John Stewart

- Volunteer during the building of the ti-tree fence – during 9 weekends;
- Posts about 100mm in dia. 10 feet long and jetted 4 feet into sand – sometimes peat intercepted; and
- Remembered very strong community spirit with ladies making refreshments.

4.4 Recent Site Investigations

Council decided to use a combination of subsurface geophysical investigations to assess likely seawall location within the dune then and physical excavation to uncover what remains of the original structures and calibrate the GPR assessment. This information would also be used to assess the likely ability to resist future erosion.

GBG Australia Pty Ltd (GBG) in conjunction with Georadar Research Pty Ltd was commissioned to carry out the subsurface geophysical investigations, including a ground penetrating radar (GPR) survey. BMT WBM Pty Ltd were engaged to carry out an engineering assessment of rock seawalls as they were excavated. The intention was then to replace disturbed rocks and recover with sand to the original condition.

The investigations were carried out in October 2013 and are described below.

4.4.1 Subsurface Geophysical Investigations

In October 2013, GBG Australia Pty Ltd (GBG) in conjunction with Georadar Research Pty Ltd carried out subsurface geophysical investigations, including ground penetrating radar (GPR) survey, across an approximately 1.4km section of coastal sand dune along the Lennox Head beach.

Ground penetrating radar (GPR) is a relatively new technique where radar frequencies are used to identify different densities of material in the soil profile. With experience and local calibration these features, such are water tables and rock concentrations, can be identified.

For this investigation data was acquired using a GSSI SIR3000 GPR data collection system with a 200MHz ground coupled antenna and referenced with a Real Time Kinematics Global Positioning System (RTK GPS)) was used for this investigation. The depth of penetration for the 200MHz was found to be about 5 m during an earlier trial in May 2013. Once the sea wall was located at a shallower depth than expected, the depth of penetration was reduced to approximately 2 m using a higher frequency 500MHz antenna.

After the initial survey on the 9th October of 3 lines at 25m spacing in front of the Surf Life Saving club, the main survey started at Line 1 at the southern end of the survey area and continued north at approximately 25m spacings to the south of the surf club. The morning of the 10th October, the survey continued north approximately 400m to the “Dog Track”.

The survey lines were completed in transverse direction (perpendicular to the beach), so as to cut across the assumed structural position of the sea wall. No lines were collected in a longitudinal direction. A total of 45 survey lines were collected at approximately 25 m spacing, where accessible. A typical sectional output is shown in Figure 4-3.

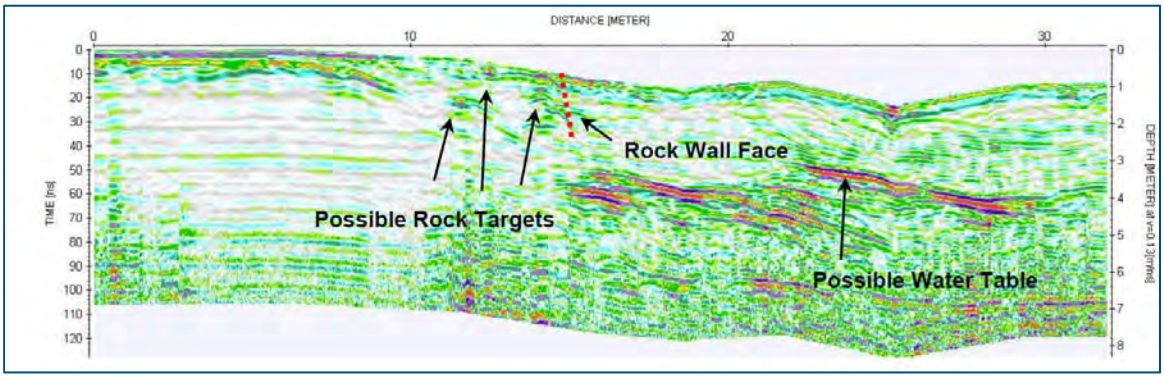


Figure 4-3 Sea Wall located on Line 37 (Excavation 1 surveyed with 200MHz)

The calibrated output of predicted seawall location is shown in Figure 4-4 and shows good comparison to the top of scarp as indicated by photogrammetry based on the 1967 photography.

Figure 4-4 Comparison of GPR predicted rock seawall (blue) and 1967 scarp line

4.4.2 Existing Seawall Excavations

As indicated above two trenches were excavated through the frontal dunes to expose the seawalls and allow an assessment of the engineering elements considered necessary for a competent rock seawall. The two locations of the excavations were:

- Just north of the Lennox Head – Alstonville Surf Lifesaving Club (Figures 3 to 6); and
- About 400m to the south near Williams Street (Figures 7 to 10).

A description of the seawall arrangement and material uncovered in the two trenches is below.

4.4.2.1 Excavation 1 – Just north of Lennox Head – Alstonville Surf Lifesaving Club

Interpretations of the initial ground penetrating radar transects by GBG staff indicated that there could be a gently sloping rock layer about 2m below the surface across the dune area (approx. 25m) and so it was decided to begin the excavation on the beach side of the proposed excavation alignment. However, no rock was found until near the low fence approx. 30m east of the dressing shed at the Lennox Head – Alstonville Surf Lifesaving Club. At this location old power poles overlay about 6 boulders of half a tonne each. The rocks were clumped together and located at a height of between +3 and +4mAHD.

The arrangement of the wall at this location did not fit the general description of the seawall that had been built and later discussions revealed that the seawall probably didn't extend past the Lennox Head – Alstonville Surf Lifesaving Club.

Figure 4-5 Trenching back from the beach

Figure 4-6 First rocks and buried power poles

Figure 4-7 Uncovered rocks near low fence at the Surfclub

Figure 4-8 Uncovered rocks near low fence at the Surfclub

4.4.2.2 Excavation 2 – Near Williams Street

Again interpretation of the initial ground penetrating radar transects by GBG staff indicated that there could be a gently sloping rock layer about 1m below the surface across the dune area (approx. 25m). This time it was decided to start from the western end of the section about 20m from the edge of Pacific Parade which was at a level of about +5mAHD. The beginning of a gently sloping rubble and red soil slope was immediately found and was tracked from the starting position for about 25m. The section had a reinforced toe which had a foundation level of about +2mAHD and was about 2m thick. The toe was excavated and consisted of about 6 rocks of around a half tonne in weight with a collection of smaller material. The red soil was not evidenced in this toe indicating that it may have been exposed to some wave action at higher tides.

The toe section was replaced for safety and a second excavation was made about 2m landward of the toe. This site also contained half tonne rock in a 1m thick zone of red soil and smaller rock.

Figure 4-9 Toe of Rubble Slope

Figure 4-10 Toe Rocks Exposed

Figure 4-11 Thickness of Rubble Slope (note red soil above sand)

Figure 4-12 Exposed Rocks in Existing Seawall

4.5 Structure Characterisation

Based on the review of the available information and the investigations carried out at the site, an estimate of the design characteristics of the existing seawall has been prepared. The primary points are indicated below and discussed in detail in Chapter 5.

- The estimated alignment of the existing rock protection is shown in Figure 4-4 and is based primarily on the GPR investigation after the results were calibrated to the excavated sections. The predicted crest and toe positions compare favourably with the positions of the RL2mAHD and RL4mAHD contours from the 1967 photogrammetry considering that the wall was built over many years and some beach recovery would have occurred in that time. The position of the seawall being on the alignment subsequent to a severe erosion event has meant that the structure has not been exposed to waves in recent decades. As such the location of the seawall is ideal in that it will only be exposed in extreme events and at other times will have a beach in front of the seawall providing a high level of amenity.
- Based on the two excavations and the GPR surveys it is noted that the crest is reasonably consistent but the toe elevation and the face slope of the seawall varies significantly. This is possibly explained by the fact that the wall was constructed over many tears and during that time the beach would have accreted (toe level increased) and the dune escarpment collapsed as it dried out (face slope flattened). Also it appears that the GPR survey may not have been able to penetrate to the toe level in some cases. It is not possible to define a typical cross section of the wall but in many areas it would include the crest at RL3-4m and the toe at RL1.5-2.5mAHD and the face slope at 1:4 as indicated in Figure 4-9. A typical design standard used in recent times might include a crest level at RL6 to prevent overtopping and a toe level of RL-1mAHD to prevent undermining during storms. The existing wall does not meet these standards.
- It is also noted that the face section of the seawall between the crest and the toe appears to be of a single layer of varying thickness which is consistent with the material being excavated from the field (new development land) and transported directly to site with no selection of rock size or grading. It was noted that the rock was basalt which is suitable for armour but that only a low percentage of the rock uncovered during the excavation was of a suitable armour size (approx.. 2 tonne or greater) and that some of these were not of suitable shape (i.e elongated rather than spherical or blocky). It is estimated that around 5% of the rock in the seawall may be suitable for use as armour in a future seawall and that a further 20% may be suitable as filter or fill material. Therefore, the existing seawall does not meet accepted notion of having “seawall armour” i.e. a selected grading of rock of a particular size and shape and laid in two layers with a filter layer behind and able to resist wave action.

In summary the existing seawall has a suitable alignment in that it is generally located well landward along a previous severe erosion escarpment. However the structure does not exhibit any of the currently accepted design elements being a suitable crest level, a suitable toe level (refer Table 4-1) and a suitable armour layer between these points. Further detailed analysis of these design elements is undertaken in Chapter 5.

Table 4-1 Predicted Existing Seawall Dimensions from GPR Survey

GPR Line No.	Length of Seawall (E to W)	Depth of Seawall (Crest to Toe)	Crest RL mAHD	Toe RL mAHD
1	4	1.0	5.0	4.0
2	4	0.8	4.0	3.2
3	4	0.9	3.9	3.0
4	5	1.2	3.2	2.0
5	2	0.5	4.5	4.0
6	4	1.0	3.8	2.8
7	3.5	0.6	4.2	3.6
8	8	1.4	3.5	2.1
9	11	1.9	3.9	2.0
10	9	3.2	3.7	0.5
11	6	1.4	3.0	1.6
13	7.5	2.1	4.5	2.4
17	6	1.8	3.0	1.2
18	6	2.0	3.4	1.4
19	8	1.0	3.0	2.0
20	8	0.9	3.2	2.3
21	14	2.2	3.6	1.4
22	10	1.7	3.8	2.1
23	10	2.0	5.0	3.0
27	7	1.9	3.8	1.9
29	7	1.0	3.8	2.8
31	8	2.7	5.2	2.5
33	9	2.0	4.0	2.0
35	5	1.0	4.0	3.0
36	6	0.6	4.6	4.0
37	6	1.6	5.4	3.8
38	6.5	1.5	4.8	3.3
39	8	1.3	4.5	3.2

Note: Line 1 is to the South and Line 39 is to the North (refer GBG Report)

5 Structural Stability Assessment of Existing Seawall

There are several basic components that need to be in place before a seawall can be accepted as being structurally adequate. These are listed below and explained in the following paragraphs:

- Crest level of sufficient height to resist overtopping to such an extent that the seawall is compromised by flow back through the wall or sensitive infrastructure behind the seawall is inundated;
- Toe rock depth and size adequate to resist undermining when beach sand is depleted and design storm occurs; and
- Armour size sufficient to resist design storm waves, including consideration of:
  - Armour rock size.
  - Armour face slope.
  - Armour thickness.
  - Armour filter layer.

The design of many of these elements is inter-related e.g. the armour face slope, armour size and overtopping rate are inter-related and the combination has to resist the design hydrodynamic loadings. The accurate prediction of the hydrodynamic loadings is a key component to the assessment of the structural stability of coastal structures. Design parameters for the existing seawall also include the expected beach profile changes during significant storms and likely profile changes in responds to sea level rise and/or sediment budget imbalances (i.e. long term recession).

This section describes the investigations undertaken to assess the existing and likely future performance of the seawall under storm attack. The adequacy of the existing seawall was assessed by assessing the likely stability of the structure under a range of design storm events for present-day conditions, and for the 2050 and 2100 planning horizons.

5.1 Seawall Design Conditions

Interpretation of the available information suggests that in 2013 the existing seawall was entirely buried and there was typically a dunal buffer of 10 to 20m in front of the structure. Erosion of sand during storm events will generally reduce this buffer and can cause a reduction of beach levels in front of the structure, which could lead to undermining of the seawall by reducing foundation support or exposing the components of the structure to excessive wave loading.

In NSW, a beach level of approximately -1.0mAHD is commonly adopted as an engineering rule of thumb for terminal structures located at the back of the active beach zone of exposed ocean beaches. This is based on stratigraphic evidence of historical scour levels and observed scour levels at structures during major storms (Nielsen et al. 1992; Foster et al. 1975).

At shorelines that are subject to a trend of recession, such as at Lennox Head, the pre-storm beach profile move landwards over time and the structure becomes progressively located further seaward in the active beach zone. This may lead to scour levels significantly below -1.0mAHD over time.

Future sea level rise will likely to exacerbate scour in front of the structure during storm events as the pre-storm beach profile will tend to move landwards and design water depths will increase (due to sea level rise), which in turn will allow higher waves to impact on the structure.

5.1.1 Storm Erosion Modelling

SBEACH modelling was undertaken to assess the beach levels in front of the existing seawall and provide estimates of the likely design loadings of the structure. SBEACH is a two-dimensional numerical cross-shore sediment transport modelling tool, developed by the US Army Corps of Engineers that simulates cross-shore beach, berm, and dune erosion produced by storm waves and water levels. SBEACH allows simulation of dune erosion in the presence of a hard bottom (ie. bed rock).

Beach profile response to a range of storms was modelled at two locations along the study area. The first profile location is at Lennox Street and considered representative for the shoreline section between Byron Street and Williams Street. The second location is at the Alstonville – Lennox Head Surf Lifesaving Club at the northern end of the study area. This beach profile locations is representative for the northern section of the study area (i.e. north of Williams Street).

Model calibration was performed by simulating a series of severe storms that occurred in 1967. Pre-storm beach cross-sections for these model calibration runs were based on photogrammetry profiles of 1958 (above 0mAHD). The extent and elevation of bed rock in the profiles was derived by analysing the LADS dataset analysis in conjunction with aerial photography.

Comparisons between modelled post-storm beach profiles and measured profiles, derived from photogrammetric survey data of 1967, are presented in Figure 5-1 and Figure 5-2. The figures show that the recorded storm beach profile (prior to construction of the seawall) is reproduced well by the model, as well as the recorded storm demand, determined from photogrammetric analysis (Refer to Table 5-1). The model calibration results also illustrate the effects of the nearshore bedrock on the storm erosion, with the storm volume at the Lennox Street profile being nearly half that at the surf club profile.

Table 5-1 Model Calibration of Storm Bite Volumes during 1967 storms

Location	Observed Volume Change above 0mAHD (m^3/m)	Modelled Volume Change above 0mAHD (m^3/m)
Lennox Street	95	107
Lennox Head Surf Club	177	211

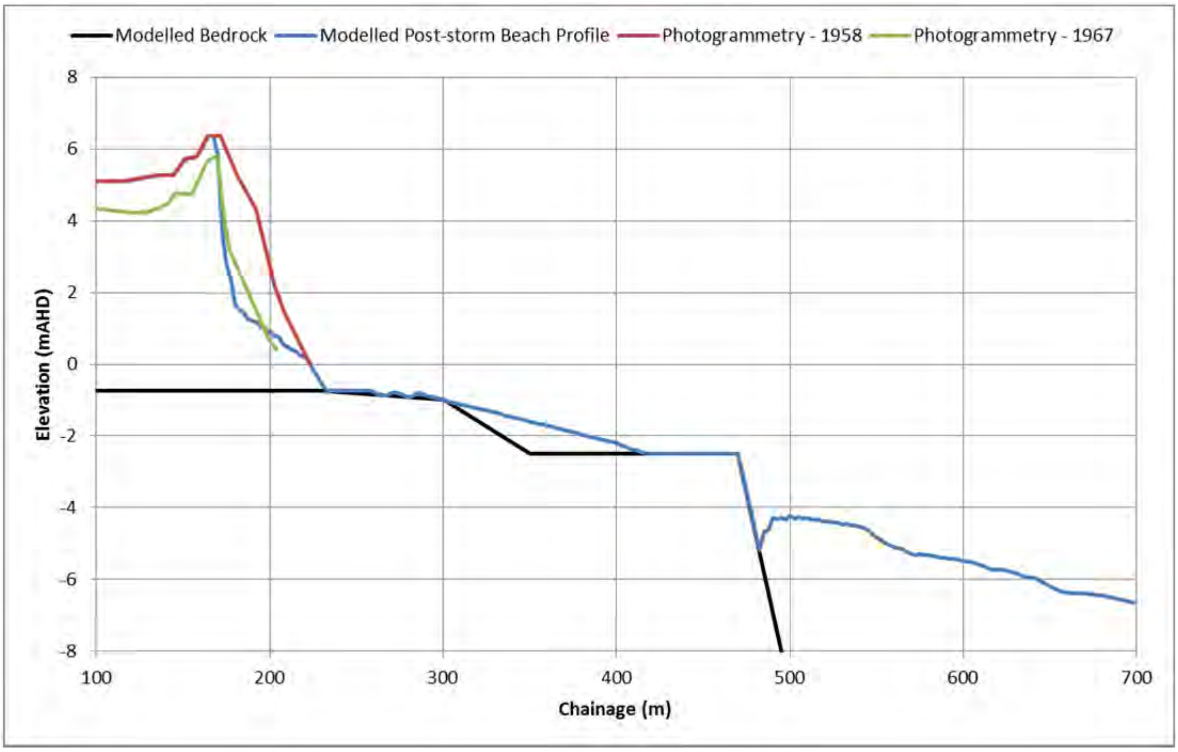


Figure 5-1 Model Calibration of Post-1967 event Beach Profile at Lennox Street

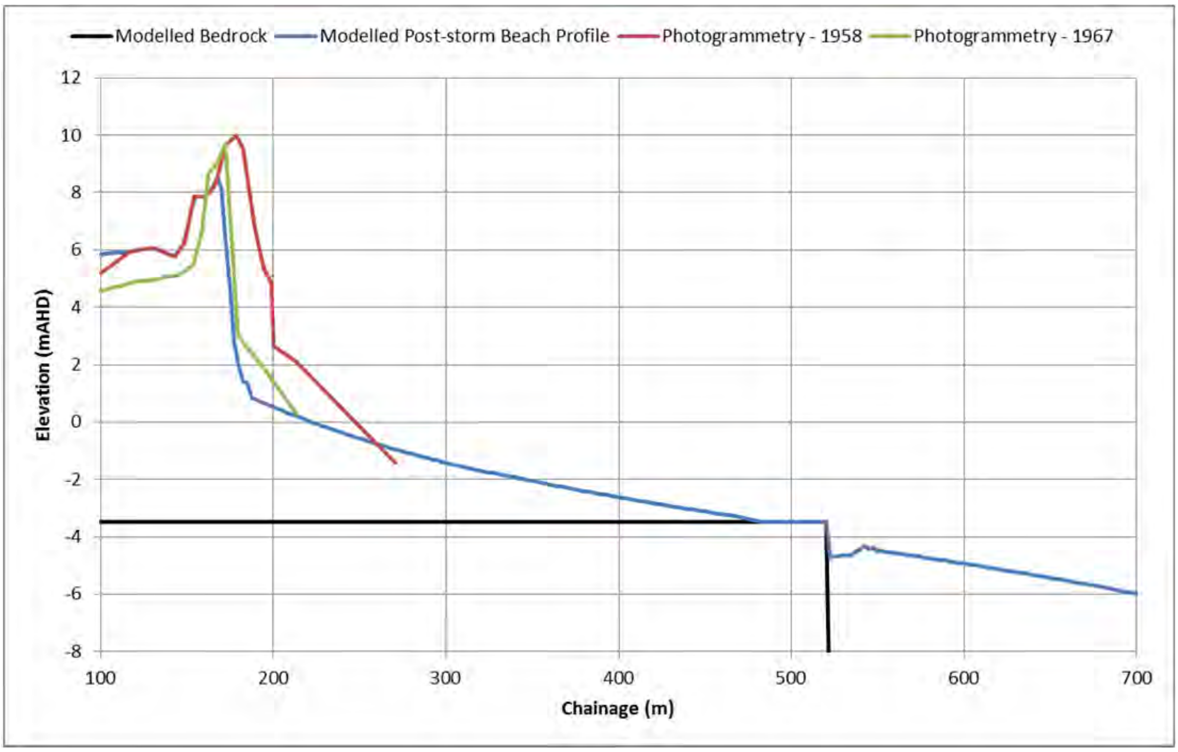


Figure 5-2 Model Calibration of Post-1967 event Beach Profile at Surf Club

5.1.1.1 Design Storm Erosion Modelling

A number of design storm events were simulated to assess the beach levels in front of the existing seawall, provide estimates of the likely design loadings of the structure and provide an assessment of the landward extent of the erosion escarpment in the situation without seawall. Modelling was undertaken for present-day conditions, as well as the 2050 and 2100 planning horizons.

Pre-storm beach cross sections for the assessment of present-day conditions were based on photogrammetry profiles of 2013. For the assessment of 2050 and 2100 planning horizons, an equilibrium beach profile approach was utilised to define the pre-storm cross sections. The 2050 and 2100 pre-storm cross sections are based on long term recession estimates documented in "Updated Coastal Hazard Areas for Ballina Shire – Stage 1" (BMT WBM, 2011). Water levels included surge and setup and seal level rise where appropriate and were taken from the modelled results (not empirical estimates) on the assumed profiles. The values are comparable with the empirical values used in the Coastline Hazard Definition Study (WBM, 2003).

Consistent with verified modelling undertaken by Carley and Cox (2003), the design erosion modelling was undertaken by simulating three sequential design storm events.

Figure 5-3 shows an example of output of the SBEACH modelling, being the modelled post-storm beach profile following a 1 in 100 year ARI design event for the scenario with an appropriate seawall in place. Table 5-2 to Table 5-7 present estimates of the beach levels in front of the existing seawall for a range of design storm events. The tables also present modelled design water levels (including wave effects) and wave heights at the toe of the structure. The water levels are generated by the model (not an input) and may appear conservative. However, model is widely used and provides the best available estimate of the landward position of the erosion escarpment in the situation without seawall.

Based on the scour modelling, it is concluded that the seawall is presently not at risk of failure during a 1 in 1 year ARI design storm event as it will unlikely be exposed during those conditions. However, during a 1 in 10 year ARI event, the structure is at risk of being undermined due to scour in front of the structure. If the seawall fails, the erosion escarpment under present-day conditions could extend up to approximately 12.5m landwards of the current seawall alignment during a 100 year design storm.

Under present-day conditions, assuming no further development between the seawall and Pacific Parade, the only infrastructure at risk due to storm erosion will be the car park at the end of Gibbon Street and any services located in this area. The road along Pacific Parade and the Surf Club building are presently not at risk of being affected by storm erosion for storm events up to the 100 year ARI event.

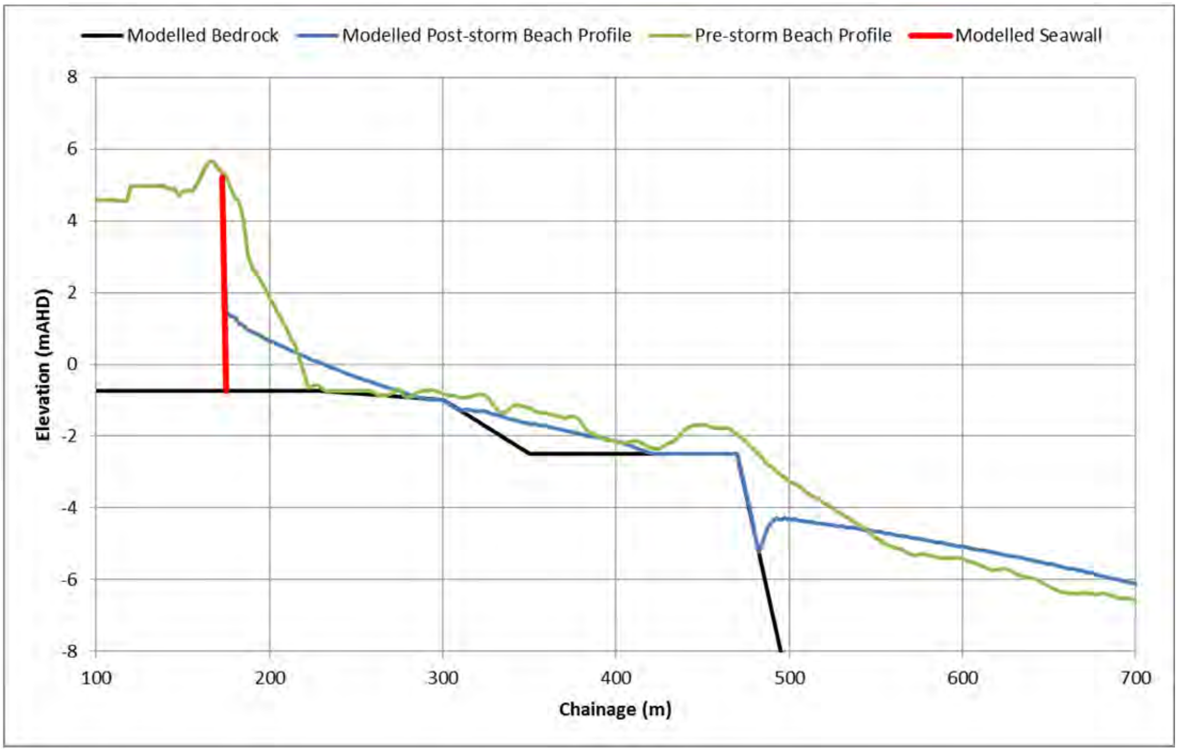


Figure 5-3 Modelled Post-Storm Beach Profile at Lennox Street for 100 year ARI event

Table 5-2 Model Predictions for Profile at Lennox Street – Present-day Conditions

Design Storm Event	Modelled Beach Level at toe of seawall (mAHD)	Modelled Scour Depth at toe of seawall (m)	Water Level (mAHD)	Significant Wave Height (m)	Escarpment Position (m relative to existing seawall)	Undermining
1 year ARI	4.64	N/A	N/A	N/A	12.50	No
10 year ARI	1.44	3.19	2.59	0.60	-7.50	Yes
100 year ARI	1.32	3.32	2.93	0.78	-12.50	Yes

Table 5-3 Model Predictions for Profile at Lennox Street – 2050 Conditions

Design Storm Event	Modelled Beach Level at toe of seawall (mAHD)	Modelled Scour Depth at toe of seawall (m)	Water Level (mAHD)	Significant Wave Height (m)	Undermining
1 year ARI	0.67	0.79	1.84	0.58	Yes
10 year ARI	0.25	1.21	2.78	1.30	Yes
100 year ARI	-0.04	1.49	3.14	1.68	Yes

Table 5-4 Model Predictions for Profile at Lennox Street – 2100 Conditions

Design Storm Event	Modelled Beach Level at toe of seawall (mAHD)	Modelled Scour Depth at toe of seawall (m)	Water Level (mAHD)	Significant Wave Height (m)	Undermining
1 year ARI	-0.28	0.28	2.20	1.29	Yes
10 year ARI	-0.39	0.39	2.70	1.70	Yes
100 year ARI	-0.63	0.38	3.08	1.88	Yes

Table 5-5 Model Predictions for Profile at Lennox Head Surf Club – Present-day Conditions

Design Storm Event	Modelled Beach Level at toe of seawall (mAHD)	Modelled Scour Depth at toe of seawall (m)	Water Level (mAHD)	Significant Wave Height (m)	Escarpment Position (m relative to existing seawall)	Undermining
1 year ARI	5.44	N/A	N/A	N/A	10.00	No
10 year ARI	0.75	4.69	2.63	0.7	-10.00	Yes
100 year ARI	0.73	4.71	3.03	0.9	-12.50	Yes

Table 5-6 Model Predictions for Profile at Lennox Head Surf Club – 2050 Conditions

Design Storm Event	Modelled Beach Level at toe of seawall (mAHD)	Modelled Scour Depth at toe of seawall (m)	Water Level (mAHD)	Significant Wave Height (m)	Undermining
1 year ARI	0.49	1.57	1.92	0.58	Yes
10 year ARI	0.13	1.92	2.83	1.28	Yes
100 year ARI	0.11	1.95	3.20	1.46	Yes

Table 5-7 Model Predictions for Profile at Lennox Head Surf Club – 2100 Conditions

Design Storm Event	Modelled Beach Level at toe of seawall (mAHD)	Modelled Scour Depth at toe of seawall (m)	Water Level (mAHD)	Significant Wave Height (m)	Undermining
1 year ARI	-1.00	1.07	2.21	1.46	Yes
10 year ARI	-1.43	1.97	2.73	2.17	Yes
100 year ARI	-1.48	1.61	3.10	2.28	Yes

5.2 Suitability of Crest and Toe Elevations

Based on the two excavations and the GPR surveys it was found that the crest varied typically between RL 3.0m AHD and RL 5.0m AHD. However the toe elevation varied widely between RL 0.5m AHD and RL 4.0m AHD however it should be noted that it appeared that the GPR survey did not penetrate to the toe in many sections possibly because of the frequency differences needed for different depth ranges. Consequently the face slope of the seawall varies significantly but is generally 1:3 to 1:4 which would be typical of the storm erosion scarp slope after it had slumped. In some sections to the north the predicted seawall is much flatter and near the toe shows a berm like shape suggesting it may have been placed after some beach accretion.

Therefore the seawall location predicted by the GPR survey appears consistent with the fact that the wall was constructed over many years and during that time the beach would have accreted (toe level increased) and the dune escarpment collapsed as it dried out (face slope flattened). It is not possible to define a typical cross section of the wall but in many areas it would include the crest at around RL 3-4mAHD and the toe at RL 2mAHD and the face slope at 1:4. This is similar to the photo in Figure 4-9. A typical design standard used in recent times might include a crest level at RL 6m AHD to prevent overtopping and a toe level of RL-1mAHD to prevent undermining during storms. The existing wall does not meet these standards.

5.3 Stability of Armour Layer

The armour rocks in a typical seawall design are selected, in conjunction with armour face slope, to resist the upward forces involved when the design wave impacts on the structure.

The size (weight) or rock armour is dependent on the level of exposure and face slope of the seawall. In northern NSW typical values ranging from 2-8 tonne are specified.

The existing wall has a very low percentage of this size rock and there was no evidence of layering of selected rock sizes.

5.4 Presence of Filter/Under Layer

The existing seawall does not have a geotextile layer or secondary armour under layer to protect the sand below the structure. As a result, it is considered likely that there will be significant sand losses from behind the wall when the wall is exposed to wave and tidal action. The generally low crest level would also allow overwash from wave action to back wash through the seawall potentially increasing sand losses. Therefore, if the existing seawall were to be exposed to wave action it would quickly settle and possibly loose rocks would be pulled out of the structure. It is likely that the entire structure would fail during a storm event.

5.5 Summary of Effectiveness of Existing Wall

The investigations and analyses to date indicate that existing seawall, does not satisfy any of the design conditions required of a seawall to resist wave attack. It is considered that the seawall still exists because it was initially constructed after severe storm erosion and the natural process of beach recovery has provided a natural buffer of sand in front of the seawall.

Design storm erosion analysis indicates that the seawall is presently not at risk of failure during a 1 in 1 year ARI design storm event as it will unlikely be exposed during those conditions. However, during a 1 in 10 year ARI event, the structure is at risk of being undermined due to scour in front of the structure. If the seawall fails, the erosion escarpment under present-day conditions could extend up to approximately 12.5m landwards of the current seawall alignment during a 100 year design storm.

Under present-day conditions, assuming no further development between the seawall and Pacific Parade, the only infrastructure at risk due to storm erosion will be the car park at the end of Gibbon Street and any services located in this area. The road along Pacific Parade and the Surf Club building are presently not at risk of being affected by storm erosion for storm events up to the 100 year ARI event.

However, if the seawall were to be considered as a terminal protection for any 50 year and 100 year future scenarios where it is likely to become more exposed to wave action as a result of shoreline changes due to sea level rise then it will be necessary to rebuild the seawall to a suitable design standard.

6 Background to Seawall Upgrade

The investigations and analyses carried out in Stage 1 of this study indicated that existing seawall, did not satisfy any of the design conditions required of a seawall to resist wave attack. It is considered that the seawall still exists because it was initially constructed after severe storm erosion and the natural process of beach recovery has provided a buffer of sand in front of the seawall since it was built.

Design storm erosion analysis indicated that the seawall is presently not at risk of failure during a 1 in 1 year ARI design storm event as it will unlikely be exposed during those conditions. However, during a 1 in 10 year ARI event, the structure is at risk of being undermined due to scour in front of the structure. If the seawall fails, the erosion escarpment under present-day conditions could extend up to approximately 12.5m landwards of the current seawall alignment during a 100 year design storm. It should be noted that if the existing dune in front of the seawall is reduced due to short or long term processes then the seawall may be exposed during lesser events and erosion after failure will be more extreme than described above.

Under present-day conditions, assuming no further development between the seawall and Pacific Parade, the only infrastructure at risk due to storm erosion will be the car park at the end of Gibbon Street and any services located in this area. The road along Pacific Parade and the Surf Lifesaving Club building are presently not at risk of being affected by storm erosion for storm events up to the 100 year ARI event.

However, if the seawall were to be considered as a terminal protection for any 50 year and 100 year future scenarios where it is likely to become more exposed to wave action as a result of shoreline changes due to sea level rise then it will be necessary to rebuild the seawall to a suitable design standard.

As Stage 2 of this study BMT WBM is to identify the works required to implement a suitable long-term terminal protection structure along this section of the coastline with the dual roles of protecting infrastructure whilst maintaining the optimal use of the beach in front of the structure as well as the parkland behind the structure but seaward of the esplanade.

This stage has four main components which are described in the following chapters:

- Assessment of likely future coastal hazards including climate change;
- Assessment of the suitability of various structural options for protection of infrastructure and enhancement of the beach amenity;
- Recommendation of a structure arrangement and alignment; and
- Development of an Implementation Plan.

7 Coastal Hazards for Design Considerations

7.1 Introduction

An understanding of the fundamental coastal hazard processes affecting the shoreline at Lennox Head is essential in developing a suitable implementation strategy to protect assets that are or may become subject to erosion threats.

The Lennox Head coastline is affected by a range of coastal hazards that will become potentially more acute or extensive in the future with climate change induced sea level rise. The key coastal hazards include:

- The erosion hazard, including components of immediate storm erosion, shoreline variability and future shoreline recession;
- Coastal inundation associated with wave run-up and overtopping of the dune barrier; and
- Dune zones of reduced foundation capacity.

These hazards have been assessed and mapped as part of the Ballina Shire Coastline Hazard Definition Study (WBM, 2003), and the 2011 update (BMT WBM, 2011).

The definition of coastal hazards inherently involves uncertainty relating to not only how prevailing oceanic conditions will manifest in the future and how reliably their effects on the shoreline can be determined, but also the considerable unknown factors involved and limitations in the available measured data. As such, the approach adopted in BMT WBM 2011 was to provide a band of feasible erosion extents, defined on hazard maps by lines representing the ‘best estimate’, ‘minimum’ and ‘maximum’ likely limits for the immediate, 2050 and 2100 planning periods. The ‘maximum’ and ‘minimum’ extents of the erosion hazard represent the range within which the erosion hazard is most likely to apply, as allowance for uncertainty inherent in the data interpretation and modelling, as well as other factors that are difficult to quantify reliably.

7.2 Coastal Erosion Hazards

During severe storms or a series of storms in succession, increased wave heights and elevated water levels results in wave attack of the beach berm and foredune region. Storm events generate high rates of transport of sand both:

- Offshore, with sand eroded from the beach face and transported to the nearshore seabed to form a sand bar roughly parallel to the shoreline; and
- Alongshore (i.e., along the beach) either upcoast or downcoast depending on wave direction, with gradients in the transport rates leading to erosion or accretion.

The result is erosion on the beach face and dune that may pose a hazard to back beach land and assets. The short term storm related cross shore sand transport and longshore drift occur simultaneously, the latter commonly leading to a significant shoreline erosion component immediately downdrift of headlands in cases where the sand supply into the beach compartment is less than the transport away to the north. Their effects are additive, although the beach itself (above mean sea level) will be observed to erode predominantly during storm events.

The beaches along Lennox Head experience considerable fluctuation associated with storm erosion and variability due to changes in the prevailing wave conditions, as evidenced by the substantial erosion experienced at Lennox Head in 1967. As well, there is a general regional trend of long term shoreline recession on which short to medium term variability is superimposed.

Thus, the 'immediate' erosion hazard extent represents the zone that could be affected by erosion in the immediate near future (e.g. over the next few years) in the event of one or more major erosion events, while the 2050 and 2100 extents incorporate a landward shift in the immediate hazard line in response to the long-term shoreline recession, including the effects of sea level rise.

Figure 7-2 presents the coastal erosion hazard extents mapping for the study area, based on BMT WBM (2011). It should be noted that these erosion hazard lines are derived on the assumption that the existing seawall does not provide any significant protection against erosion. The recent investigation into the structural capacity of the existing buried seawall (BMT WBM, 2014) indicated that the existing structure has a limited design standard in terms of providing protection against wave attack.

It should also be noted that there is a zone of reduced foundation capacity that extends landward of these erosion hazard lines (See also Figure 7-1). This zone of reduced foundation capacity represents the area where the bearing capacity of the dunes may become reduced as a result of erosion, potentially threatening the structural stability of structures (building, roads etc.) that are not adequately founded or piled.

Amongst other factors, the width of the zone of reduced foundation capacity behind the hazard lines is dependent upon the angle of repose of the dune sand and the height of the dune above mean sea level and at Lennox Head would typically be around 10 to 12m. Expert geotechnical engineering assessment is recommended to establish the structural stability of foundations located (or likely to be located) within the zone of reduced foundation capacity on a case by case basis. Importantly, the stability of the road foundation along Pacific Parade following severe erosion events, and the need for closing/restricting traffic during such times, will need to be assessed by a qualified geotechnical engineer.

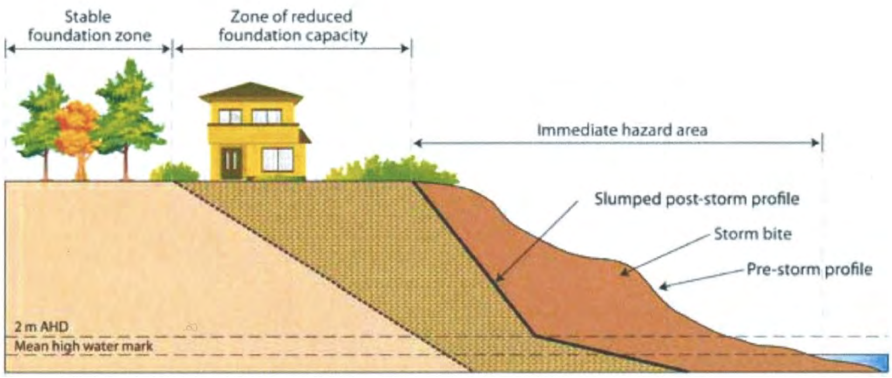


Figure 7-1 Design Profile and Zones of Instability for Storm Erosion (from DECCW 2010; after Nielsen et al 1992)

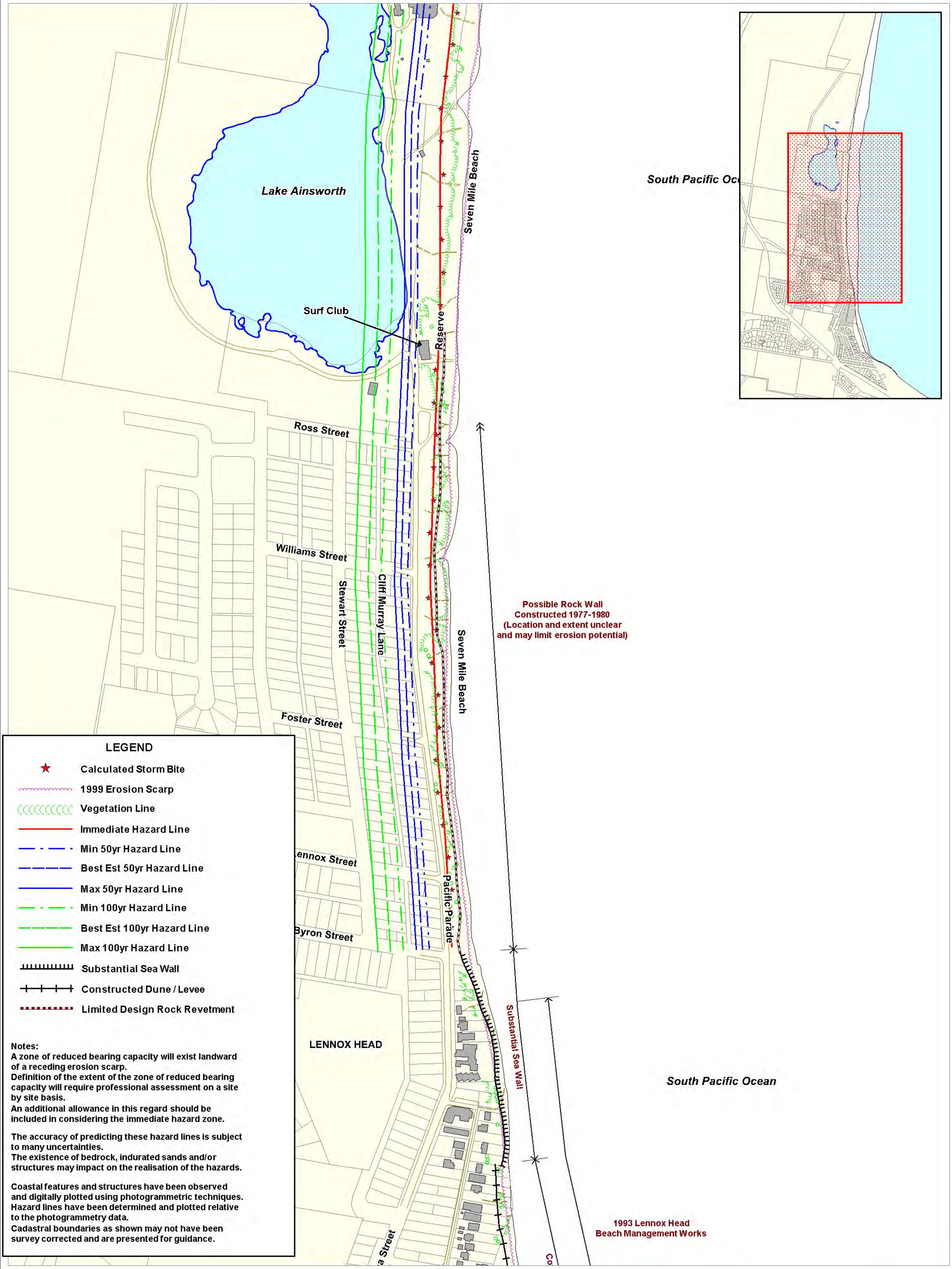
7.3 Coastal Inundation Hazards

Where the crest height of a dune is less than the wave run-up level, waves will overtop the shoreline and may cause inundation of the land behind. Consequently, this may present a hazard if the rate of overtopping can cause a significant impact to people or assets behind it.

BMT WBM (2003) assessed the potential for coastal inundation due to wave overtopping and concluded that episodic or infrequent overtopping may occur at the lower barriers (less than RL 5m AHD), but the extent of inundation from such processes is considered to be limited at present.

It is likely that Lennox Head will experience enhanced wave run-up and overtopping in the future, as sea level rises. Furthermore, in regions where future erosion may remove the whole of the frontal dune barrier (eg. Lake Ainsworth), extensive inundation of low areas could occur.

With regards to coastal inundation, it is important to note that assessment of wave overtopping in the 2003 report is based on the assumption that the shoreline comprises a natural beach/dune system. Where waves impact on shoreline protection structures (in particular vertical or steeply sloping surface such as seawalls), substantially higher wave run-up levels can be experienced. Therefore, wave overtopping is typically a key consideration in the design of such protection structures.



Title:

Lennox Head Erosion Hazard Zones

Figure: 7-2

Rev: A

N

0 150 300m

Approx. Scale

BMT WBM
www.bmtwbm.com.au

Filepath : I:\B20888_I_LennoxHeadSeawall\DRG\COA_001_Lennox_Hazard_Zones_111020.WOR

8 Terminal Seawall Design Considerations

8.1 Introduction

The purpose of a terminal seawall at Lennox Head would be to reduce the risk of coastal erosion causing damage and loss of the road (Pacific Parade), the surf club building (Lennox Head-Alstonville SLSC), private properties, beach accesses and public open space. It will also serve to prevent inundation if sea level rise is realised or if the dune in front of lake Ainsworth is breached. These factors will be key consideration in the seawall design and alignment.

Terminal seawalls are robust structures, built along the back of the beach, which provide a physical barrier separating the erodible material immediately behind the structure from wave and current forces acting on the beach itself, and thereby providing protection to the areas behind the structure. This protection is generally in the form of reducing shoreline recession and as such would be typically located as close as practicable to the infrastructure under threat in order to minimise the impact on coastal processes including beach amenity. In some cases the seawall will also reduce inundation by maintaining a sufficiently high dune (or seawall) to prevent the ingress of the sea e.g. around Lake Ainsworth. Both of these factors would be exacerbated by future sea level rise should it be realised.

Ultimately the structure would extend the entire 1.3km alongshore distance between the existing rock seawall at Byron Street to the northern end of Pacific Parade at Lake Ainsworth. It is envisaged that the structure would be constructed in a staged approach whereby sections of terminal seawall would be constructed only when sufficient erosion has occurred to warrent implementation. Following construction, it is anticipated that the structure could be largely concealed within a replenished dune and foreshore area and only re-exposed following a subsequent extreme erosion event. If sea level rise is realised the structure may become increasingly exposed over time.

Following such an event minor beach nourishment or sand scraping activities could be undertaken (if viable) to cover up exposed sections of the structure, minimise disruption to coastal processes and maintain beach amenity. The proposed structure would therefore form part of a hybrid shoreline protection solution, combining a concealed terminal structure, beach nourishment and ongoing dune and foreshore re-vegetation works. The Gold Coast City Council has adopted a similar shoreline management approach between Southport and Coolangatta.

The concept design phase of any terminal protection structure must include a number of site specific considerations:

- Geotechnical condition of the existing dune/foreshore (including location and depth of bedrock), historical changes and environmentally sensitive areas;
- Hydraulic conditions, including design wave and water levels;
- Structure alignment (horizontal and vertical);
- Crest elevation to limit wave overtopping to an acceptable level;
- Rock armour size to limit damage during a design event to an acceptable level;

- Specification of the filter layer(s) or the use of a geotextile layer;
- Scour and toe protection;
- Availability of materials;
- Access to the site and construction feasibility (including construction impacts such as traffic disruption and impacts on recreational beach use);
- Presence of any underground infrastructure such as cables, pipelines etc.);
- Public Access and Safety Principles (e.g. post construction beach access);
- Capital and maintenance costs;
- Financial and governance constraints (including budgetary limitations, approvability, lead time to implementation, maintenance commitments); and
- Foreshore Protection Principles (Assets to be protected, public safety, design life, staging).

8.2 Coastal Processes and Beach Amenity

Seawalls are commonly built with the intent of providing terminal protection against shoreline retreat and provide a physical barrier separating the erodible material immediately behind the structure from wave and current forces acting on the beach itself. They are typically constructed to allow for some flexible movement during storms but are designed to withstand severe wave attack for a typical design life of 50 years.

Where possible, seawalls should be continuous to prevent end effects and in smooth curves to prevent discontinuities that could threaten the overall integrity of the wall. They also have to be suitably founded for stability against scour at the toe of the structure, particularly on a receding shoreline.

While a properly designed and constructed seawall will protect the landward property from erosion, it effectively isolates the sand located behind the wall from the active beach system and may lead to other adverse consequences.

On a receding shoreline, the seawall effectively becomes progressively further seaward on the beach profile. This leads to a gradual increase in the quantity of sand effectively lost from the beach system, with:

- Lowering and eventual loss of the beach in front of the wall;
- Exacerbation of the erosion at the downdrift end of the wall where the sand losses are transferred and concentrated; and
- Loss of amenity in front of the seawall and exacerbated erosion threats at the ends.

Scour and lowering of the beach in front of the wall ultimately exposes it to higher wave attack and if not included in the design can lead to slumping and the need for ongoing maintenance. Such maintenance is typically in the form of topping up of the wall when the damage is small. However, where the seawall is not adequately designed or constructed, and effectively fails, complete reconstruction may be needed.

If amenity is an important issue in front of the seawall then the seawall needs to be located landward outside of the immediate erosion hazard zone or combined with a program of beach nourishment to maintain the beach in a suitable condition.

8.3 Hydrodynamic Design Conditions

The hydrodynamic design conditions are a key input for the design of any terminal protection structure.

An assessment of extreme wave and water level conditions has been undertaken as part of Stage 1 of this study (BMT WBM, 2014). These loadings are not only dependent on the local nearshore wave and water level conditions, but importantly influenced by the cross-shore positioning of the structure and the expected beach profile changes during severe storms and likely profile changes in responds to sea level rise and/or sediment budget imbalances (i.e. long term recession).

By reducing seaward protrusion, the scour levels in front of the structure may remain limited, which has a direct effect on the design requirements of the armour layer and the crest height (through wave runup and overtopping), as scour levels in front of the structure usually dictate the maximum depth limited breaking wave height. In addition, reducing seaward protrusion is also likely to reduce the foundation depth requirement of the structure to prevent undermining.

Future sea level rise will likely exacerbate scour in front of the structure during storm events as the pre-storm beach profile will tend to move landwards and design water depths will increase (also due to sea level rise), which in turn will allow higher waves to impact on the structure. As such, careful consideration of any changes in hydrodynamic design parameters during the design life of the structure will form an important step in the preparation of preliminary seawall design options for Lennox Head.

8.4 Resilience and Adaptability

The uncertainty inherent in the present projections of sea level rise and future shoreline behaviour at Lennox Head requires that terminal seawall works are resilient and adaptable to a broad range of potential shoreline scenarios, irrespective of the uncertainty.

Not all seawall types have the same degree of adaptability. A rock armoured revetment is more adaptable than a gravity structure or a rigid sloping wall due to its flexibility and ability to be “topped-up” to achieve increased height and resilience against increased toe scour and/or increased wave heights.

8.5 Design Life

Australian Standard AS 4997-2005 (“Guidelines for the design of maritime structures” recommends a design life of 50 years for a ‘normal commercial structure’. While the Standard does not formally apply to coastal structures such a terminal seawalls, the design shall consider a specific design life of 50 years from implementation.

8.6 Public Safety

It is required that the seawall design does not compromise public safety. To achieve this it would be necessary to ensure safe longshore and cross shore pedestrian access is provided in the vicinity of the structure. For a seawall located on a receding beach, it may be necessary to provide for a footpath on top of the structure in the future when a suitable beach width becomes unavailable.

Wave overtopping can be a safety issue during extreme events. Should the road along Pacific Parade need to remain operational during storm events, then limitation of wave overtopping by increasing seawall height or providing a wave barrier may become necessary.

Vertical seawall structures can exhibit a substantial vertical face when exposed by a receding beach and this may become hazardous for foreshore users (falling down large vertical drop) and swimmers/surfers returning to shore (collision and drowning hazard).

8.7 Timing of the Works

As mentioned, it is envisaged that the structure would be constructed in a staged approach from the south with sections of seawall constructed as required and only when significant erosion has triggered the implementation strategy.

In Section 3 of this report the storm bite was assessed by investigating both the maximum volume loss per length of beach, derived from two consecutive photogrammetry surveys and also by modelling actual shoreline recession during storms with the existing historic seawall in place. Along the study area, the historical volume losses vary along the shoreline. Along the southern most 600m section (between Byron Street and Williams Street), the maximum volume losses are generally around 100-120 m3/m. North of Williams Street, as the nearshore reef become less pronounced, the maximum volume losses increase to approximately 200m3/m at Lake Ainsworth and this relates to 20-40m of shoreline recession. The modelling for a 1:100 year storm indicated a shoreline recession of around 12.5m.

Based on these investigations and allowing for construction i.e. room to excavate the dune and build the seawall, the trigger distances have been developed by BMT WBM and shown in the last column of Table 8-1. This indicates a buffer width of 30m, measured from the road boundary to the top of the frontal dune (when beach and dune is in a general accreted state), for the shoreline section between Byron Street and Williams Street, 40m for the section between Williams Street and Ross Street and 45m between Ross Street and Lake Ainsworth. The recommended trigger for implementation of a terminal protection at the Surf Club is 45m. It is noted that no trigger has been defined for the car park at the end of Ross Street, as the CZMP recommends no specific management action to protect this asset. These are relative to the beach alignment as shown in the 2013 photogrammetry.

It should be noted that the natural beach alignment to the north of the existing seawall is in a smooth curve which currently runs increasingly seaward of the road. Should this alignment be followed by the proposed seawall then buffer distances may reduce and the triggers distances reached relatively quickly.

The trigger width requirements are intended to provide a sufficient buffer to give Council adequate time to implement the required seawall section prior to encountering the unacceptable impacts of erosion hazards such as:

- Impact to the structural integrity of Pacific Parade; and
- Permanent loss of beach and shoreline assets (e.g. Surf Lifesaving Club building).

The available buffer distance and the anticipated progression of future shoreline recession suggest that the trigger distances may be reached at different times throughout the study area. Consequently, it is anticipated that the terminal protection wall could be constructed in distinct phases, most likely from south to north and could potentially be separated by many years, depending long term on erosion trends.

Table 8-1 Available Buffer Width at Key Assets

Asset	Distance between Asset and Foredune in 2013	Recommended Trigger distance for enhanced erosion management
Lennox Head-Alstonville Surf Club Building	51m	45m
Car park near Ross Street	29m	NA
Road along Pacific Parade at Ross Street	57m	45m
Road along Pacific Parade at Williams Street	50m	40m
Road along Pacific Parade at Foster Street	53m	40m
Road along Pacific Parade at Lennox Street	34m	30m
Road along Pacific Parade at Byron Street	38m	30m

8.8 Seawall Alignment Considerations

8.8.1 Recommendations of the Coastal Zone management Plan

The Coastal Zone Management Plan 2016 (CZMP) recommends that the alignment of the seawall follow the 'line of protection' landward of which property or infrastructure is to be protected and that the seawall is aligned as far landward as possible to maximise the potential to retain a sandy beach seaward of the wall.

Given that the historical seawall has proven to be ineffective (Section 4) the CZMP recommends that the seawall alignment considerations include the following:

- For the area between Byron Street and the Surf Lifesaving Club, it is recommended that the seawall be as close to Pacific Parade as possible while balancing the future amenity provided by the beach with that provided by the grassed public open space between the beach and the roadway. The key considerations relating to the alignment include the following:

    - The further seaward the alignment of the seawall, the sooner beach amenity can be expected to be reduced due to the estimated shoreline recession occurring “against” the seawall. These reductions in beach amenity will be in the form of a gradual increasing frequency with which public access along and general use of the beach will be restricted due to combinations of low sand supply and tide levels. Such effects already occur on occasions as a result of the seawall south of Byron Street.
    - The further landward the alignment of the seawall, the less grassed public open space will ultimately remain once shoreline recession has occurred up to the seawall.
    - It is likely that the unique amenity provided by the beach will be valued more highly by the community than the amenity provided by the grassed public open space, thus indicating an alignment as close to Pacific Parade as possible is preferred.
- The existing Surf Lifesaving Club building and the proposed replacement just south of this does not protrude substantially further seaward of the general alignment of Pacific Parade so the wall alignment could extend seaward of this structure.
- Between the Surf Lifesaving Club and the Lake Ainsworth Sport and Recreation Centre, the primary protection requirements are preventing a breakthrough to Lake Ainsworth and maintaining access between those centres. Accordingly, the seawall could be located further landward here; however consideration of increased interaction of seawater with the freshwater ecosystem of the lake will need to be considered.
- At the Lake Ainsworth Sport and Recreation Centre, the seawall works will need to be integrated with the existing upgraded seawall in front of the centre constructed in 1997.

8.8.2 Coastal Engineering Considerations

From coastal engineering and ease of construction perspectives, and incorporating the CZMP considerations, the following points can be made:

- The seawall will generally need to form a smooth curved alignment from the existing seawall to the Surf Lifesaving Club to prevent differentials in sand transport in the future if the wall becomes exposed.
- The Ground Penetrating Radar (GPR) survey carried out in conjunction with the investigations of the historical seawall was able to determine the crest and toe alignment of this seawall and the distance from crest to toe (refer Table 4-1) generally indicated the width of a typical seawall construction. It is noted that this alignment is seaward of the 2010 storm bite assessment by about 10m. This alignment is a good indicator of the beach response to a severe erosion event.
- If the seawall were to extend from -2m AHD to +5m AHD, with a standard 1: 1.5 face slope design, then the horizontal extent of the wall from toe to back of the crest would be about 20m. Therefore, allowing 10m landward of the crest for construction vehicles and materials storage a distance of 30m from threatened infrastructure will be required to enable construction. This supports the suggested trigger values in Table 8-1.
- As indicated in the CZMP construction costs would be reduced by building after an erosion event when a significant volume of sand would be moved offshore. However it should be noted

that this sand will begin to move onshore again immediately after the storm and may need to be restrained by a bund (or collected and stored) during the construction period and then used as beach nourishment in front of the seawall after completion.

- Building after storm erosion will also facilitate an effective tie in to the existing seawall;
- Alternatively building the seawall at a more normal time with sand accreted might have advantages in that seawall construction would be in dry conditions and excavated sand could be placed seaward to provide a cheaper bund that sheet steel piling.
- Ultimately the choice of when the seawall is built may well be driven by funding availability or the consequence of a dramatic event.

It should also be noted that while the historical seawall was considered not to be of a design standard, there will likely be a considerable number of rocks that could be re-used for the new seawall.

8.8.3 Alignment Options and Recommendation

Based on the above discussions and in particular the considerations outlined in the CZMP it is considered that the following options should be considered:

- Adopt the alignment of the existing historical seawall (approximately the 1967 erosion scarp). This has the advantage of being the alignment of a known severe erosion event and would allow the removal and reuse of the historical seawall rocks at the same time as building the new seawall. Also it follows the smooth curved alignment of an eroded beach and fits in with the alignment of the existing seawalls to the south. However it is a relatively seaward alignment when considering further long term shoreline recession and would likely require beach significant nourishment in the long term for beach amenity.
- Adopt the alignment of the calculated storm bite as shown in Figures 2-3 and 2-4 of the CZMP (refer Figure 8-1 and Figure 8-2). This alignment has two advantages in that it is significantly landward of the historical seawall alignment and it still passes seawards of the Surf Lifesaving Club and the Lake Ainsworth Sport and Recreation Centre. The major disadvantage of this alignment is the discontinuity at the existing seawall near Byron Street which is about 30m seawards of this possible alignment.
- A hybrid alignment which transitions from the historical seawall in the south to the calculated storm bite in the north i.e. transitioning in the zone from Byron Street to Foster Street. This option combines the best aspects of the previous options and would provide the most favourable option for maintaining beach amenity to the southern section of the beach.

It is considered that the hybrid alignment option above provides the best outcome and would easily lend itself to phased implementation with the initial phase approximately following the historical seawall alignment allowing recovery of the materials from that seawall. This is also supported by the trigger point analysis which indicated that the Byron Street to Foster Street should be the first phase of seawall extension.

These options are presented in Figure 8-3 below.

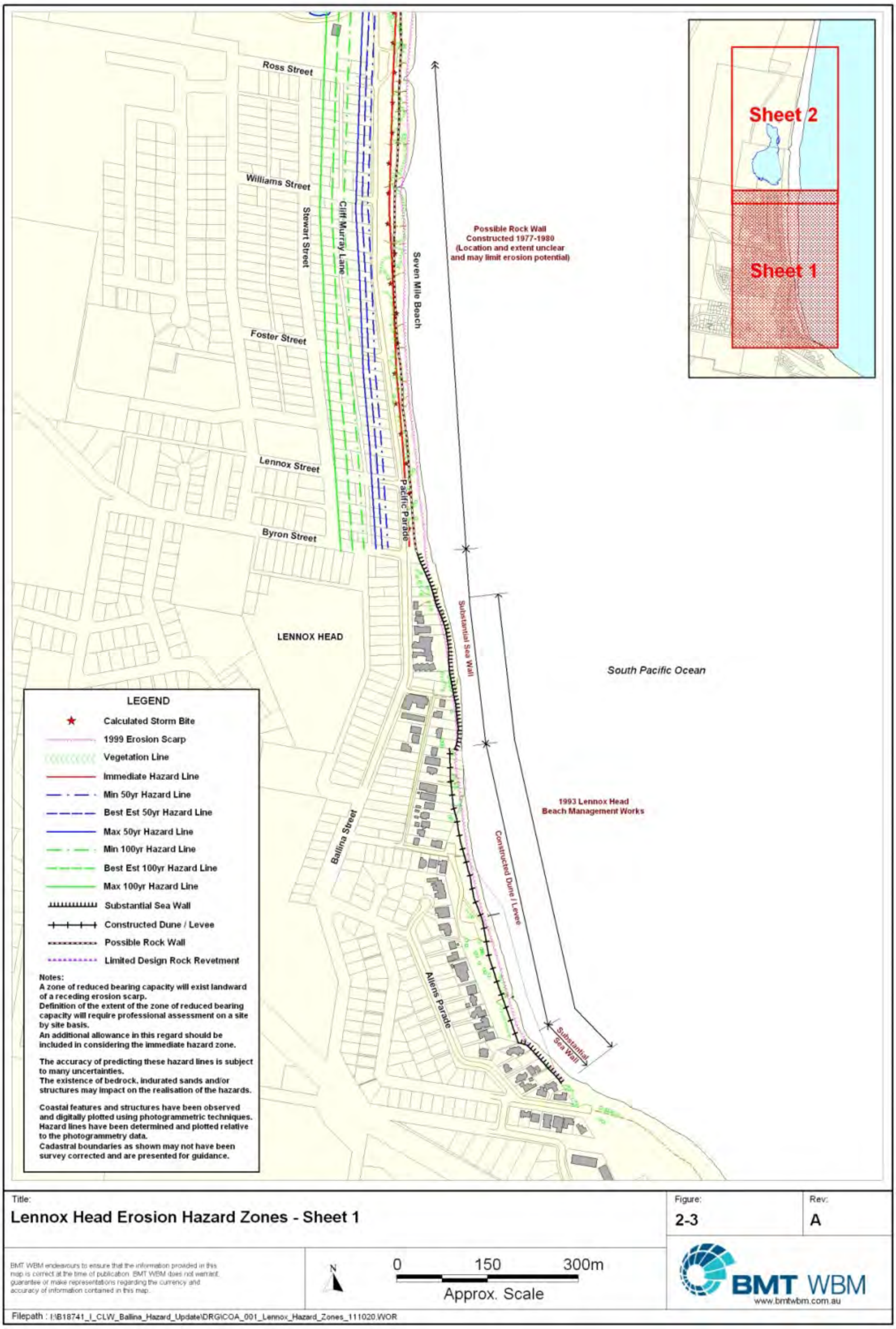


Figure 8-1 CZMP Hazard Lines Fig 2-3

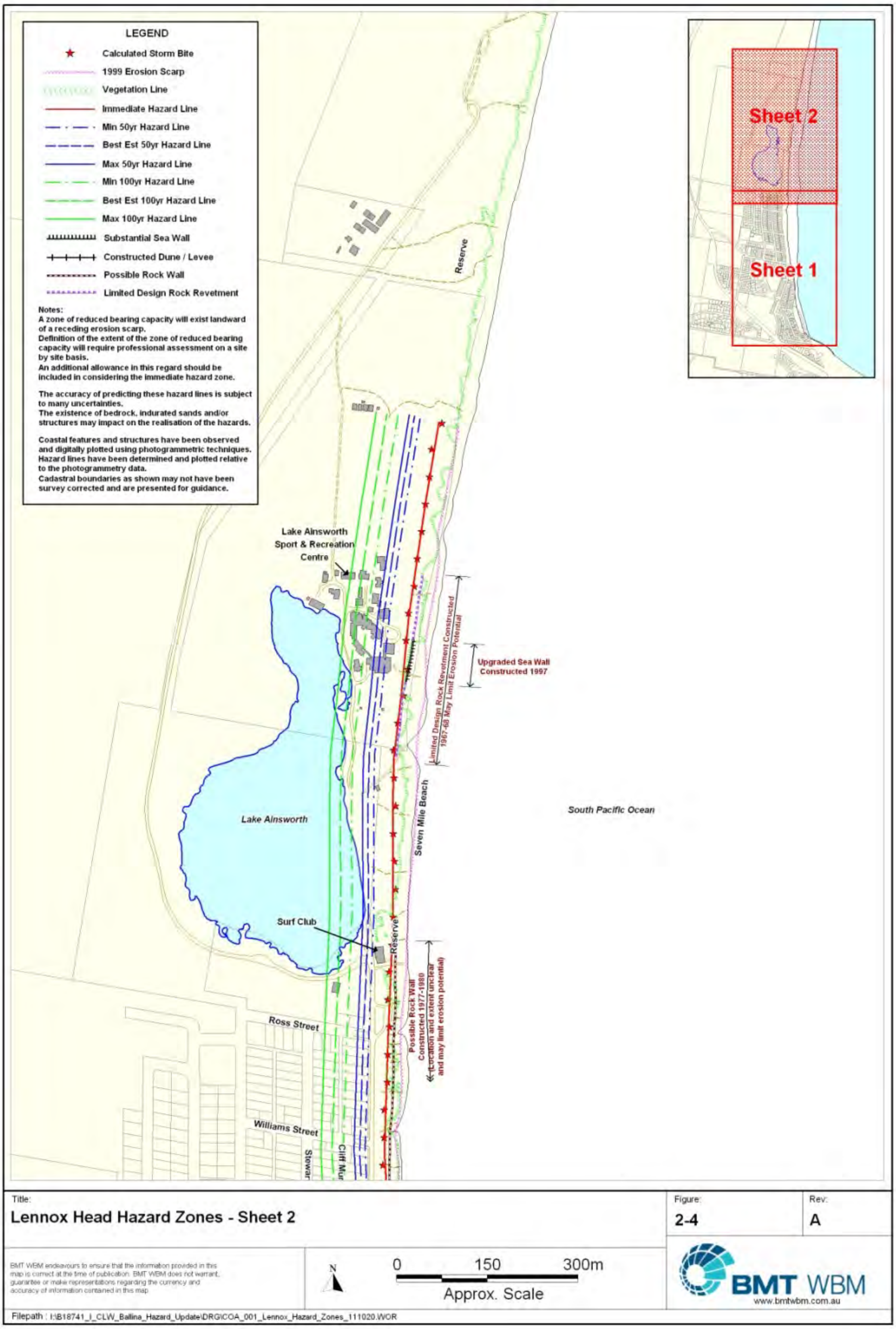


Figure 8-2 CZMP Hazard Lines Fig 2-4

Title:

Lennox Head Seawall Alignment Options

Figure: 8-3

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9 Terminal Seawall Option Assessment

9.1 Introduction

There are many types of seawalls, built from a wide range of materials, including rock, concrete, steel, timber and geotextile. Some structures are better suited than others depending upon their site-specific purpose, the varying physical forces they are designed to withstand and the availability of construction materials.

The following sections provide a general description of common types of seawalls and discuss their main advantages and disadvantages with respect to the potential application at Lennox Head.

9.2 Rock Revetment

Rock revetments are flexible, sloping rock rubble structures that are widely used as terminal protection measure in high wave energy environments such as the northern New South Wales coast. Conventional rock rubble revetments typically comprise a rock armour layer, one or more rock underlayers, a filter layer (usually a geotextile) and a buried rock toe to prevent undermining of the structure. Rock revetments generally have a slope of between 1V:1.5H and 1V:3.0H. A conventional rock armoured seawall is shown in Figure 9-1.

Rock for use in revetments must have high durability and high density and the design of rock revetments is often controlled by the size, shape and quality of rock available from nearby quarries. While not specifically investigated as part of this investigation, quarries where large quantities of suitable rock (primarily basalt) are known to exist within the Ballina region. However, availability will need to be checked at the final design stage.

Rock revetments are flexible structures that can tolerate a significant degree of displacement and shifting. Typically, the design conditions permit the movement of some 10% of the armour units and 2% damage during the design event.

They can be easily upgraded if required by topping up the structure. As such, these structures are highly adaptable to changing conditions. This is considered a positive for application at Lennox Head, as there are significant uncertainties regarding the long term future behaviour of the shoreline and the effects of sea level rise on the hydrodynamic design loadings.

Advantages

The advantages of this type of terminal protection include:

- High degree of coastal protection
- Proven design concept in high energy environments
- Low maintenance and high durability
- Adaptable and flexible design (able to accommodate different settlement and adaptable to unforeseen beach erosion and sea level rise)
- Relatively cost effective (rock can be sourced locally)
- Reuse of existing rock possible

- Aesthetics (consistent appearance to existing structure to the south, more natural looking than alternatives)
- Effectively absorbs wave energy (reduced wave overtopping and scour).

Disadvantages

The disadvantages of this type of terminal protection include:

- Beach access restricted to stairways
- Reduction of recreational space (beach and foreshore)
- Large footprint (more likely to be accepted by community)
- Higher construction impacts (traffic disturbance to supply rock, large excavation area)
- Longer lead time (sourcing of rock, longer construction duration)
- Collision hazard for surfers and swimmers.

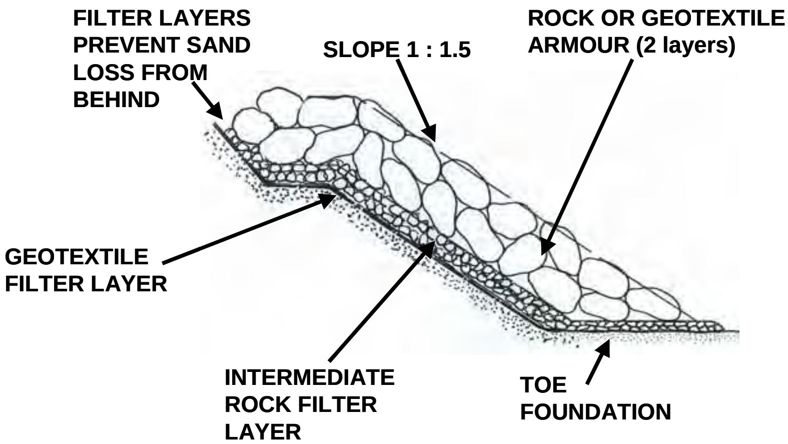

Figure 9-1 Design Section of a Typical Rock Seawall

Figure 9-2 Existing Rock Revetment at Lennox Head South

Figure 9-3 Construction of a Rock Revetment at Stockton Beach in 1987 (Photo courtesy OEH)

9.3 Randomly-Placed Concrete Revetment

Irregular concrete blocks are similar to rock revetments, but instead of rock, the armour layer consists of randomly-placed, precast concrete units, often applied in a single layer thickness. Most concrete armour units are well-engineered, robust designs that are protected by patents and attract a royalty fees to use.

Concrete units are an attractive alternative if suitable rock is not available locally or cannot be supplied at a competitive price. Typically, this form of seawall is more suited to large design waves (Hs > 5m at structure).

For study area, the capital costs of this type of structure are estimated to be at least 50% higher than a comparable structure made of rock.

Advantages

The advantages of this type of terminal protection include:

- High degree of coastal protection
- Proven design concept in high energy environment
- Durable
- Adaptable and flexible design (able to accommodate different settlement and adaptable to unforeseen beach erosion and sea level rise)
- Effectively absorbs wave energy (reduced wave overtopping and scour)
- Smaller footprint than rock wall.

Disadvantages

The disadvantages of this type of terminal protection include:

- Expensive for Lennox Head conditions
- Aesthetics (less likely to be accepted by community)
- Beach access restricted to stairways
- Reduction of recreational space (beach and foreshore)
- More difficult to modify/upgrade, compared to rock revetment
- More difficult to repair, compared to rock revetment.

Figure 9-4 Randomly-Place Concrete Revetment (Photo courtesy Xbloc)

9.4 Geotextile Containers Seawall

Sloping sandbag revetments are being used increasingly as shoreline erosion protection works. This type of seawall comprises sand-filled geotextile containers or bags. Sand-filled geotextile container revetments generally have a recommended slope of 1V:1.5H and can be used in low to moderately severe wave attack with design wave heights up to around 1.6m (This would require a geotextile container size of 2.5 m^3).

Due to the high energy environment at Lennox Head, it is unlikely that geotextile containers can used along the site, unless the structure can be placed far enough landward to minimise excessive wave exposure and a relatively short lifespan of the structure is accepted.

Should a geotextile container solution be feasible, then it should be noted that separate access ways may need to be installed at regular intervals to provide safe pedestrian access as the required 2.5 m^3 containers will be too large for the public to safely walk down and over the seawall. These access ways may be separated stairway structures or integrated within the revetment itself by placing smaller geotextile containers on the larger 2.5m3 containers to provide a trafficable access way.

Advantages

The advantages of this type of terminal protection include:

- Smaller footprint than rock wall
- Shorter construction duration
- Lower construction impacts
- Aesthetics
- Softer option (collision).

Disadvantages

The disadvantages of this type of terminal protection include:

- Unstable in large waves conditions (**unlikely to be feasible at Lennox Head)**; subject to more detailed analysis)
- Low durability (prone to vandalism and damage by plant root penetration, deteriorates under UV exposure)
- Higher maintenance
- More difficult to modify/upgrade, compared to rock revetment
- More difficult to repair, compared to rock revetment
- Absorbs less wave energy (larger wave overtopping and scour).

9.5 Stepped Monolithic Concrete Seawall

Rigid sloping walls made of precast reinforced concrete slabs are popular on promenades, especially where there is very heavy pedestrian traffic, such as on main tourist beaches. Landscape features such as seating, plantation pots and staircases can be incorporated into the design of these structures, providing an attractive public open space between the beach and the foreshore. They typically have slopes of between 1V:1.5H to 1V:3.0H.

An example of a stepped concrete seawall is the seawall at Manly Beach, NSW (refer Figure 9-5). This seawall has large bleachers with trafficable stairways between the bleachers. This type of seawall has the advantage of being relatively narrow, but is generally highly reflective to incoming waves. Consequently, stepped concrete seawalls may experience additional loss of beach in front of the wall during storm events or exhibit a slower recovery thereafter. For higher steps such as those required for Lennox Head the structure will need to be supported on piles for long term stability.

Advantages

The advantages of this type of terminal protection include:

- High degree of coastal protection
- High durability

- Maintains beach amenity during most periods
- Improved beach access, compared to rock revetment
- Low maintenance.

Disadvantages

The disadvantages of this type of terminal protection include:

- Danger to public if wave reach the steps
- Capital cost substantially higher than rock revetment
- Difficult to remove if no longer needed/desired
- Rigid structure could be prone to catastrophic failure during storm event
- More difficult to modify/upgrade, compared to rock revetment
- Stability of structure is sensitive to toe erosion and settlement of pile foundations
- Wave reflection may lead to additional loss of beach in front of the wall.

Figure 9-5 Stepped Seawall at Manly Beach, NSW (Photo from WRL, 2012)

9.6 Vertical Piled Seawall

Vertical piled seawalls are relatively thin vertical structures driven into the seabed. These structures rely on the depth of penetration into the soil substrata for stability against horizontal loads.

This type of seawall can be made of intersecting reinforced concrete piles (secant pile structure) or steel sheetpiles (or a combination of king piles and sheetpiles). Sheet piling can only be used in areas where the subsurface conditions are relatively free of boulder and bedrock, making it unsuitable for application at Lennox Head (likely presence of bedrock at relatively shallow depths).

Secant pile structures are formed as a combination of large diameter unreinforced piles interspersed with smaller diameter reinforced concrete piles which are bored to overlap into the unreinforced pile sections. An example of this type of seawall is the secant pile wall recently built around the Cudgen Headland SLSC at Kingscliff (See Figure 9-6).

At Lennox Head, it is likely the adequate embedment of a secant pile wall can be achieved at relatively small depths by embedding the piles in the bedrock (subject to detailed geotechnical investigation). Notwithstanding this, it is likely that anchoring will be required.

A piled wall has the advantages of being narrow, thereby located relatively landward and becoming exposed in the profile later than the other seawall options for the same functional cross-shore criteria. However, once exposed, scour in front of the structure may be increased during storm events and recovery of the beach delayed due to the highly reflective nature of vertical structures. When exposed, these structures can exhibit a substantial vertical face that can become hazardous for foreshore users (falling down large vertical drop) and swimmers/surfers returning to shore (collision and drowning hazard) and provide a major obstacle in terms of beach access.

Advantages

The advantages of this type of terminal protection include:

- Small footprint
- Relatively cost effective
- Can be constructed without major excavations
- Shorter lead time (quick to construct)
- High degree of flexibility in terms of implementation length (sections more easily determined by shoreline protection need).

Disadvantages

The disadvantages of this type of terminal protection include:

- Unproven design concept in high energy environments
- Public safety issues (vertical drop)
- Beach amenity impacts
- Reduced beach access
- Prone to sand leakages (difficult to inspect during construction)
- Wave reflection may lead to additional loss of beach in front of the wall
- More difficult to modify/upgrade, compared to rock revetment
- Difficult to remove if no longer needed/desired.

Figure 9-6 Secant Pile Seawall at Kingscliff Beach

9.7 Gravity Structures

Concrete and blockwork gravity walls are common as promenades on major beaches, such as Bondi Beach in Sydney. Their small footprint (compared with sloping seawalls) maximises the space available landward and seaward of the structure.

Gravity seawalls depend primarily on shearing resistance along the base of the structure to support the applied loads and as such rely mostly on the weight of the structure to provide the required stability against wave action. They require strong foundation soils to adequately support their weight. At Lennox Head, a gravity wall would likely be founded on a filter bed of riprap which is directly on the underlying bedrock to avoid differential settlement of the structure. An example of a gravity wall structure is presented in Figure 9-7.

Similar to vertical piled seawalls, gravity structures are usually also highly reflective to incoming waves once exposed. This may result in increased scour during storm events and consequently delayed recovery of the beach thereafter.

Advantages

The advantages of this type of terminal protection include:

- High degree of coastal protection
- Low maintenance
- High durability.

Disadvantages

The disadvantages of this type of terminal protection include:

- Public safety issues (vertical drop)
- Beach access restricted to stairways

- More complex engineering
- Significantly higher capital cost, compared to rock revetment
- More difficult to modify/upgrade, compared to rock revetment
- Rigid structure could be prone to catastrophic failure during storm event
- Difficult to remove if no longer needed/desired.

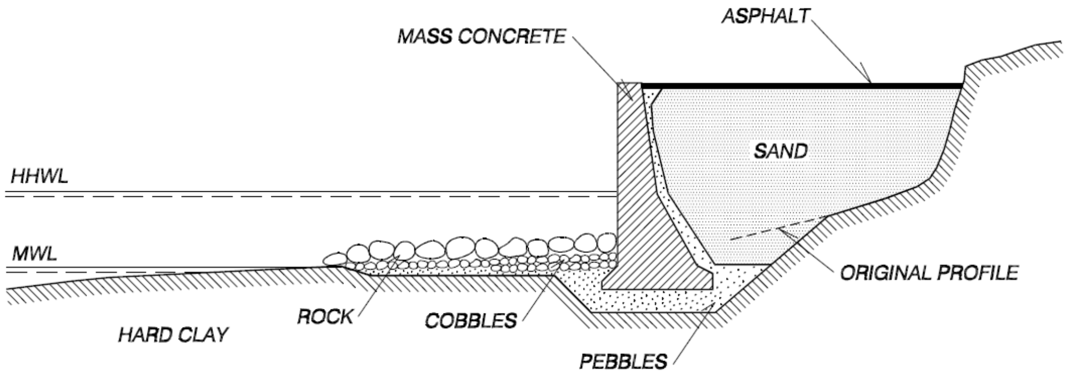

Figure 9-7 Example of a Vertical Gravity Seawall (from USACE, 2006)

9.8 Summary of Alternative Terminal Seawall Options

The sections above describe six alternative terminal protection options. Their potential strength and weaknesses have been assessed qualitatively with respect to the following:

- Level of protection
- Durability
- Adaptability
- Maintenance Requirements
- Public safety
- Beach Amenity
- Degree of wave absorption (loss of beach and wave overtopping issues)
- Implementation lead time
- Construction impacts
- Construction footprint (Spatial requirements for implementation)
- Capital costs.

A summary of key advantages and disadvantages of each option with respect to the potential application at Lennox Head are summarised in Table 9-1 with a shortlisting of rock or stepped seawall based on the relative advantages and disadvantages of each type in the Lennox Head situation.

Table 9-1 Summary of Alternative Terminal Seawall Options for Lennox Head

Seawall Option	Level of protection	Durability	Adaptability	Maintenance	Public safety	Beach Amenity	Wave Absorption	Lead time	Construction Impacts	Construction Footprint	Capital Costs	Overall Suitability
Rock Seawall	✓✓	✓	✓✓	✓	✖	✓	✓	✖	✖	✖	✓	✓
Concrete Unit Seawall	✓✓	✓	✓	✓	✖	✖	✓	✖	✓	✓	✖	✖
Geotextile Containers	✖✖	✖	✖	✖	✓	✓	✖	✓	✓	✓	✓	✖
Stepped Seawall	✓	✓✓	✓	✓✓	✓	✓✓	✖	✓	✓	✓	✖	✓
Vertical Piles Seawall	✓	✓✓	✖	✖	✖✖	✖	✖✖	✓✓	✓✓	✓✓	✓✓	✖
Gravity Structure	✓	✓✓	✖	✓✓	✖✖	✓	✖✖	✓	✓	✓	✖✖	✖

Note: Two ticks better than one tick and two crosses worse than one cross

10 Conceptual Seawall and Step Designs

10.1 General Concept and Alignment

The traditional terminal protection on high energy beaches i.e. beaches exposed to ocean waves has been the rock seawall. This type of structure has been used for over 100 years in Australia and has proven to be robust and able to be maintained and / or upgraded with larger rocks or to higher crest levels if necessary. The disadvantage of these structures is that access from the protected land behind the seawall to the beach across the rock seawall is difficult and dangerous particularly where the dunes are high as in Lennox Head or where a beach no longer exists in front of the seawall.

Many methods have been trialled to provide better beach access with timber or aluminium steps (popular in Queensland where storm surge protection requires high dunes) and angled access paths (e.g. southern Lennox Head) being the most popular. In recent times the provision of monolithic concrete steps has become more popular (refer Figure 9-5). In particular they have been proposed and used on less exposed beaches or where the danger of large waves impacting users of the steps is low.

The full range of options available for Lennox Head has been assessed in Section 9 and the option of providing a rock seawall with steps at reasonable access points has been found to be an acceptable concept. In association with car parking adjacent to Pacific Parade, the steps will maintain access to the beach at most times with the possible exception of storm conditions. Previous studies have used an interval between access points of about 400metres and this appears acceptable for Lennox Head as it would locate the first set of steps in the vicinity of Williams Street. For safety these access points should be include signage regarding the danger if the waves impact on the stairs during storm conditions.

The recommended alignment of the new seawall has been developed in Section 8.8.3 and the regional alignment is shown in Figure 8-3. The final detailed alignment will be subject to survey of the existing seawall to the south, the adopted final design of the seawall and other environmental and social considerations. An approximate final location showing crest and likely toe alignments is given in Figure 10-1.

A description of the conceptual designs for a rock seawall and a stepped seawall follows.

LEGEND

- Crest of Existing Seawall
- Crest fo Historic Seawall
- 2010 Immediate Hazard Line
- Crest of New Seawall
- Approx. Toe of Seawall

Title:

Lennox Head Initial Seawall Alignment Byron Street to Foster Street

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10.2 Seawall Concept Design

The analysis in Section 9.8 identified a traditional rock rubble mound seawall combined with a stepped monolithic concrete structure founded on piles as suitable option for Lennox Head. Previously in Section 3 the conceptual design conditions included a foundation level of -1m or -2m AHD (depending on extent of long term beach erosion, level of nearshore bedrock and sea level rise) with a crest level of 5m AHD. It should be noted that recent seawall designs, where future sand supply is not assured i.e. updrift of headlands, are using -2m AHD as the structure foundation level. This then dictates much more severe design conditions and greater rock armour mass. It has been assessed that significant overtopping of the crest at RL 5m AHD will be unlikely but this will be calculated in the final design. If necessary a 1m high wave barrier can be incorporated at the crest.

Therefore, the likely design requirements for the upgraded seawall at Lennox head will be as follows. These will need to be checked at the time of final design but are in agreement with recent detailed designs for a rock seawall Kingscliff (WRL 2014) where a lack of beach nourishment is expected to result in lower beach levels. The following initial design parameters are suggested:

- Foundation level of -2m AHD;
- Crest elevation of 5m AHD plus 1m wave barrier to reduce overtopping;
- Rock armour mass of up to 13 tonne may be required on a rock face slope of 1:1.5.

At this stage it is considered preferable that a face slope of 1:1.5 be adopted at Lennox Head to reduce the footprint on to the beach. Armour rock mass could be reduced by having a flatter face slope but this will extend the toe further seawards reducing long term amenity. Further details will be needed in final design for tie in to existing walls and proposed concrete steps as well as end treatments to prevent outflanking if storm erosion occurs.

However, for Lennox Head the following points are noted:

- Currently there are approximately 750m of existing rock seawall immediately north of the headland with a foundation level of about -1m AHD;
- This existing seawall will be the first to suffer from any reduced sand supply in the future and as such its foundations may need to be improved or the seawall rebuilt; and
- Bedrock is present in the area and appears to be at levels of -1m to -2m AHD and this may form a suitable non-erodible base for the seawall.

In summary it is considered that nominating the foundation level of the seawall at -2m AHD or firm bedrock is a reasonable option, though initially more expensive, as it provides surety of defence into the future. A conceptual design is shown in Figure 10-2.

Final design will need to include an investigation of sound bedrock in the area (note that this was visually assessed in Section 5-1) and assessment of armour rock mass based on the resultant depth limited wave height at the seawall. In the worst case scenario it can be expected that future beach sand levels will recede to -2m AHD resulting in a relatively massive structure compared to current standards.

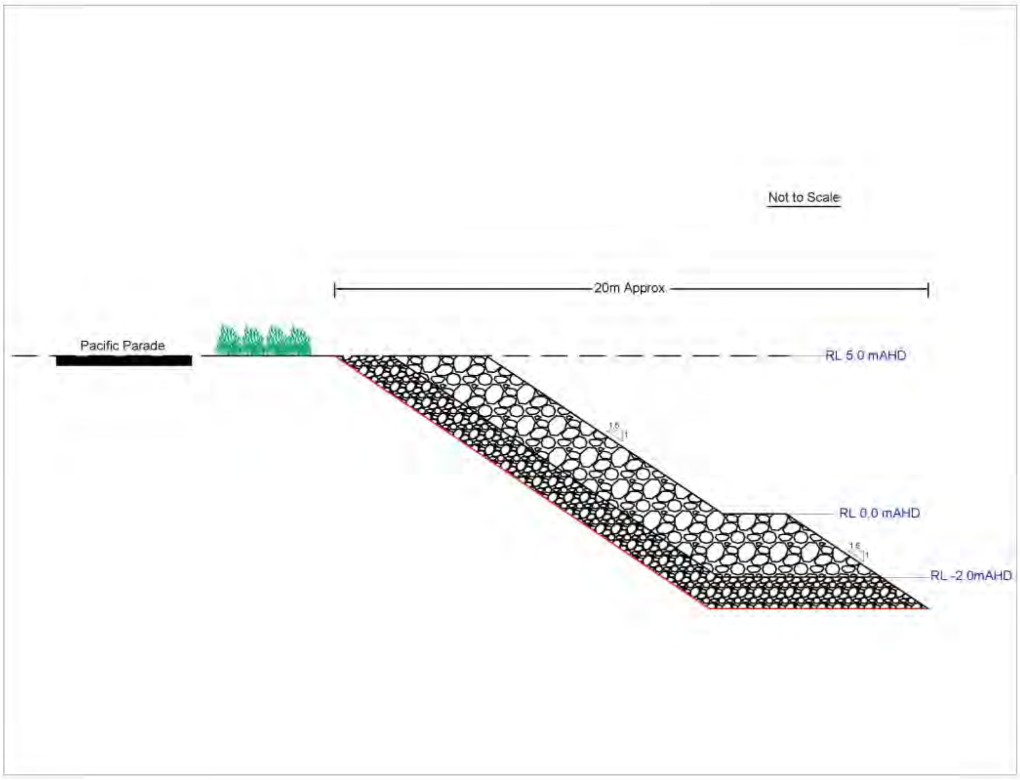

Figure 10-2 Conceptual rock seawall design

10.3 Step Concepts

Introducing steps into seawalls to improve access is a recent addition to shoreline protection options for high energy environments although it has been used for low energy environments for some time. A popular structure at Manly Beach is shown in Figure 9-5 and a typical conceptual design is given in Figure 10-3. These steps have the obvious advantage of allowing access to the beach while providing platforms for recreation at many levels above the beach or ocean (depending on tide and beach level). The disadvantage of these steps is the danger involved in trying to access the ocean during storm events if waves are impacting the structure. If used it would be necessary for Council to provide safety signage as well as considering the blocking of access during large storms.

The following parameters are considered appropriate for the conceptual design of stairs at Lennox Head:

- Base of concrete stairs at -2m AHD or bedrock and supported on reinforced concrete piles;
- Crest elevation of 5m AHD (optional wave barrier to 6m AHD);
- Concrete and reinforcement designed for the harsh marine environment; and
- Step face slope of 1:1.5 so that the stairs can be integrated into the rock seawall sections.

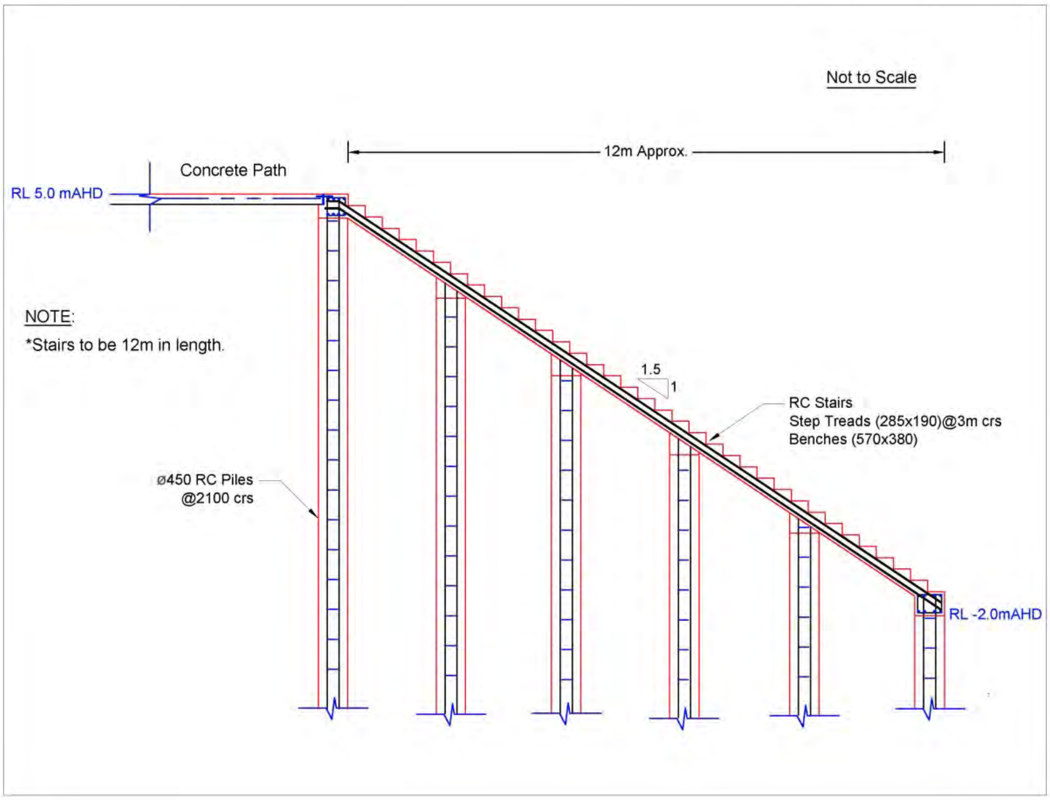

Figure 10-3 Conceptual concrete steps design

10.4 Typical Seawall / Step Integration Detail

By maintaining the 1:1.5 face slope, which suits normal tread / riser standards, the stepped structure can easily be integrated into the rock seawall as shown in Figure 10-4. The rock seawall and step interface would need to incorporate design aspects to prevent any outflanking of the structures in severe events.

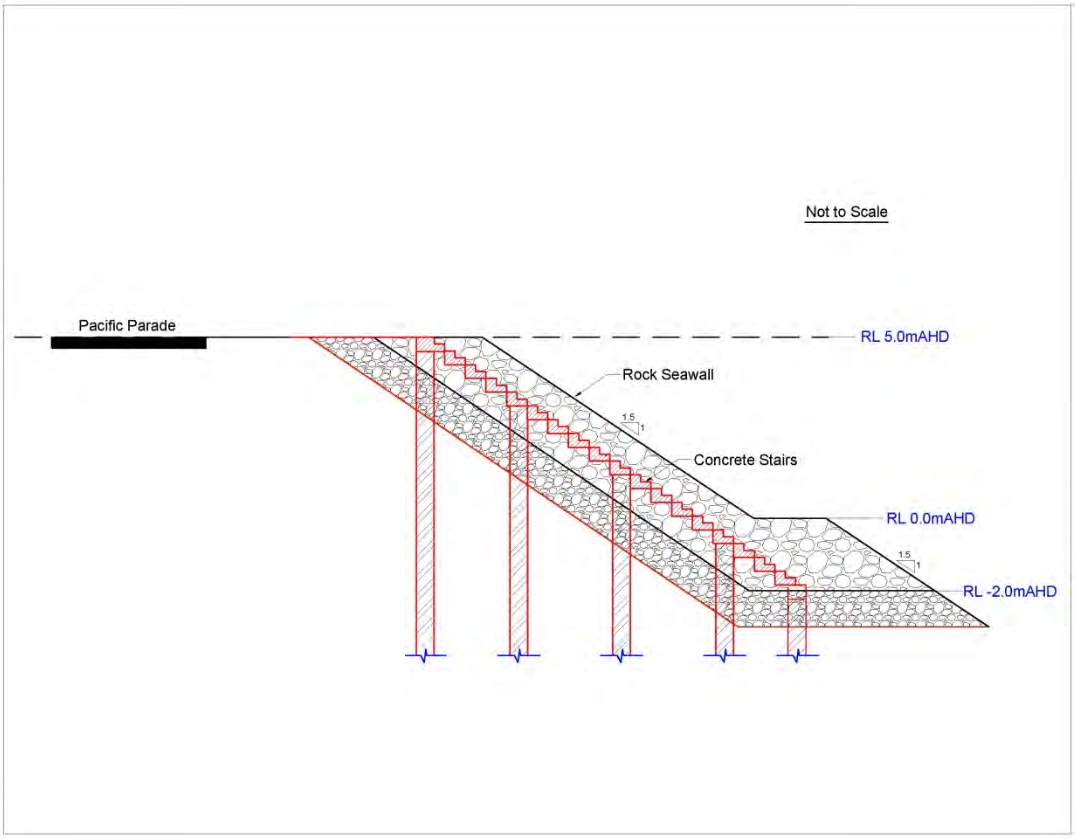

Figure 10-4 Conceptual Seawall with Steps Section

11 Implementation Plan

The implementation plan has been divided into 3 Stages as shown in Figure 11-1 and described below:

- Stage 1 – Seawall from Byron Street to Foster Street;
- Stage 2 – Steps and seawall for a distance as defined by trigger distances at the time; and
- Stage 3 – Completion of the seawall to link up with the recent seawall at the Lake Ainsworth Sport and Recreational Centre including steps and associated carparks at the Lennox Head – Alstonville Surf Lifesaving Club and approximately 300m centres to the south.

11.1 Stage 1 – Byron Street to Foster Street

Section 8 has indicated that the infrastructure between Byron Street and Foster Street is within the trigger distance zone and should be built as soon as possible. It is noted however, that the CZMP recommends that construction commence immediately after storm erosion to reduce the volume of sand excavation for construction of the seawall. As noted previously, if funding is available before this storm erosion occurs then construction in dry conditions within the normal dune may prove viable. It is recommended that this seawall be built on the alignment shown in Figure 10-1, at a time when a final design has been completed, funds are available and sufficient storm erosion has occurred. It not considered that stairs are required in this initial section (approx. 300 m) as the existing seawall south of Byron Street has access ramps and access over the dune will be available just north of the end of the new seawall until Stage 2 is commenced.

An estimated cost for this seawall, based on recent estimates for a similar design standard seawall at Kingscliff of $18,000 per metre would be $6,000,000 (2016) allowing $5,400,000 for the seawall and $600,000 for excavation, recovery of historical rocks, bunding and post-construction landscaping. Ongoing maintenance costs would be about 1% on the initial cost per year or $60,000 per year excluding inflation.

In regards to timing it is recommended that the new seawall be constructed as soon as possible and at least with the next 5 years i.e. before 2022.

11.2 Stage 2 – North of Foster Street

The length of the next section of seawall will be dependent on how soon trigger distances are reached. This will be largely determined by the reliability of an ongoing supply of sand to the area, whether the newly completed seawall produces accelerated erosion as an end effect and whether climate change impacts are realised. A determination of when this will occur is not possible at this time but it is important that monitoring of the dune scarp location every year and post storms would give a good indication of when trigger points are being approached.

It is recommended that the provision of steps and associated car parking be considered at Foster Street if a seawall is planned to be built further to the north. The steps could be about 20-50m in length to provide access and an area for recreation. The cost of the seawall and steps at current day prices will be about $18,000/m however estimating costs well into the future (greater than 10 years) is not possible to predict and will need to be assessed at some point in time when the trigger distances are being approached.

11.3 Stage 3 – Completion of Seawall

The final stage will be the completion of the seawall to connect to the existing seawall protecting the Lake Ainsworth Sport and Recreation Centre including steps and associated carparks at the Lennox Head – Alstonville Surf Lifesaving Club and approximately 300m centres to the south. It is not possible at this time to predict when this might be needed, if staging is required or the cost.

Title:

Proposed New Seawall Construction Stages

Figure: 11-1

Rev: A

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12 References

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Appendix A Ground Penetrating Radar Report (GBG)

North Parramatta, NSW. 2151
Tel: 02 9890 2122. Fax: 02 9890 2922
E-mail: info@gbgoz.com.au

A.B.N. 77 009 550 869

GEORADAR RESEARCH Pty Ltd A.C.N 003 776 743
412 Eastbank Road
Coramba, via Coffs Harbour, NSW 2450
Tel: 02 66-54-4162 Mobile: 0428 66 2399
E-mail: richard.yelf@georadar.com.au

1st November 2013 GBGA Ref: GBG1579

Attention: Mr. Paul Busmanis
Engineering Works Manager
Civil Services Group
Ballina Shire Council,
PO Box 450,
Ballina, NSW. 2478.

SUBSURFACE INVESTIGATION USING GROUND PENETRATING RADAR (GPR) TO LOCATE A BURIED SEAWALL STRUCTURE AT LENNOX HEAD, NSW.

1. INTRODUCTION

GBG Australia Pty Ltd (GBG) in conjunction with Georadar Research Pty Ltd carried out a subsurface geophysical investigations during the 9th and 10th October 2013 across an approximately 1.4km section of coastal sand dune along Lennox Head beach, NSW.

A small geophysical investigation trial had been conducted by Georadar Research Pty Ltd in May 2013. GBG Australia Pty Ltd carried out additional investigation north and south of the Lennox Head Surf Life Saving club using Ground Penetrating Radar (GPR), referenced with a Real Time Kinematics Global Positioning System (RTK GPS). The results from the trial suggested a sea wall is buried at approximately 3.5m depth. Subsequent excavation showed this was not the case.

Figure 1 overleaf shows the extent of the survey area.

Figure 1: Extent of the GPR survey.

2. THEORY OF TECHNIQUES

Ground Penetrating Radar (GPR) is a non-destructive, geophysical technique for rapidly imaging the shallow subsurface (up to 10 m depth).

GPR is normally operated in the reflection mode, like an echo sounder, except that impulses of electromagnetic energy are transmitted rather than sound waves. These impulses are of very short duration (each pulse has a rise time of typically 1-5 nanoseconds) and contain a wide spectrum of frequencies, typically in the range between 100 MHz and 1 GHz.

Signals are reflected back from interfaces in the ground where there is a contrast in the dielectric properties between two adjacent layers. The target depth is proportional to the time taken for the signal to travel down and back from a given layer. A radar-gram profile is built up of multiple scans collected along a selected line path. A sample radar-gram has been selected from the data and shown below in Figure 2. The recorded reflections can be analysed in terms of shape, travel time, signal amplitude and phase to provide information about a target's size, depth and orientation. Note that the higher the frequency of antennae used, the higher resolution is achieved at the expense of depth of penetration.

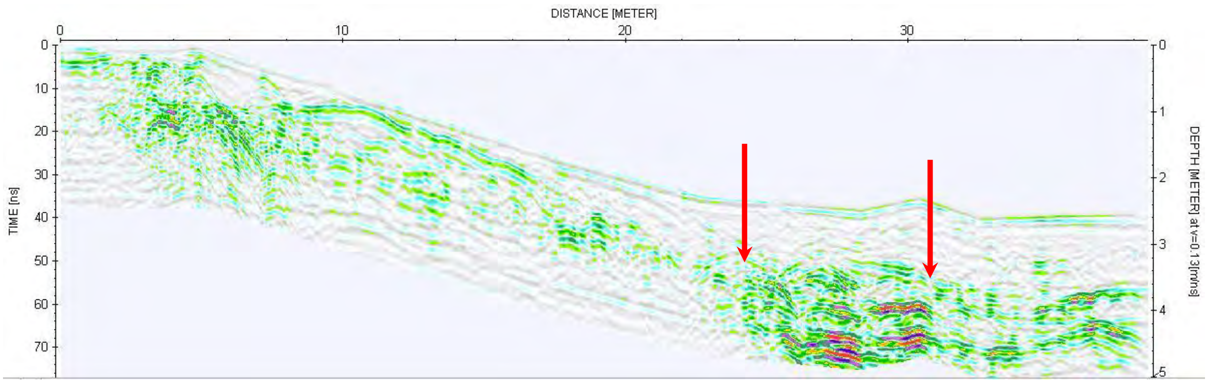

Figure 2: Sample Radargram showing the Sea Wall extent of profile 19 (500MHz).

The GPR method has limitations which are inherent to the geophysics of the technique. These include that:

- There must be good coupling between the transmitting and receiving antennae and the ground. Profiles collected over thick undergrowth or rough ground, building rubble / rocky ground, fill, reinforced concrete slabs and some paving materials may adversely affect the data. In these cases the reliability of the information obtained may be compromised. Clay significantly absorbs radar energy and hence the GPR method is generally not very successful in soils with high clay content. Also a rocky subsurface introduces significant clutter into the GPR response which may make the detection of boundary layers difficult.

- GPR profiles must be collected on a continuous line. Profiles cannot be collected across vertical breaks in slope such as steps or where there are obstacles such as fences.

3. METHODOLOGY

The data for this investigation was acquired using a GSSI SIR3000 GPR data collection system. A 200MHz ground coupled antenna was used for this investigation. The depth of penetration for the 200MHz was 5 m during the trial. Once the sea wall was located at a shallower depth than expected, the depth of penetration was reduced to approximately 2 m using a higher frequency 500MHz antenna.

After the initial survey on the 9th October of 3 lines at 25m spacing in front of the Surf Life Saving club, the main survey started at Line 1 at the southern end of the survey area and continued north at approximately 25m spacings to the south of the surf club. The morning of the 10th October, the survey continued north approximately 400m to the “Dog Track”.

The survey lines were completed in transverse direction (perpendicular to the beach), so as to cut across the assumed structural position of the sea wall. No lines were collected in a longitudinal direction. A total of 45 survey lines were collected at approximately 25 m spacing, where accessible.

On-site quality assurance and interpretation of the data was conducted by viewing the raw profile lines in real time. Processing and analysis was carried out at our Sydney office. Data processing involved: - static correction, to correct the signal to the surface, and background removal, to remove static noise bands and hence enhance the returned signal, and correction for elevation (topography).

4. RESULTS AND DISCUSSION

The results of the investigation have been plotted using autoCAD on drawing GBGA1579-01 at a scale of 1:4000, and GBGA1579-02 and 03 at a scale of 1:1000, attached to this report as a PDF file. The results of the survey appear as an outline depicting GBG Australia's interpretation of the sea wall extent overlain on a Google Earth Image. The positions of our survey lines, collected with RTK GPS are also shown.

GBG1579-04 shows every 2D GPR cross section collected on site at Lennox Head, identifying, where possible, the sea wall in red. This has also been attached to this report as a PDF file. Note: The red hatch line represents GBG Australia's Interpretation of the walls position only, not the depth of the wall.

An initial survey was conducted in front of Lennox Head Surf Lifesaving club on Line 37. A target was chosen based on the previous assumptions (May 2013), which was then excavated. Based on those assumptions, no sea wall was located, so further excavation continued approximately 10 m inland before a sea wall was found at approximately 800 mm beneath ground level. The RTK GPS recorded the Reduced Level (RL) of the top of the wall at 4.722 m. The RL for the bottom of the wall was 4.04 m, giving an approximate thickness of wall of 700 mm. The largest rock that was excavated on Line 37 was 1000 mm x 800 mm x 400 mm, however no excavation continued behind the wall face to determine the nature of/if any back fill. The radar data shows the wall face of approximately 1000 mm thickness, with several targets behind it suggesting possible rock or fill. Refer to Figures 3 and 5.

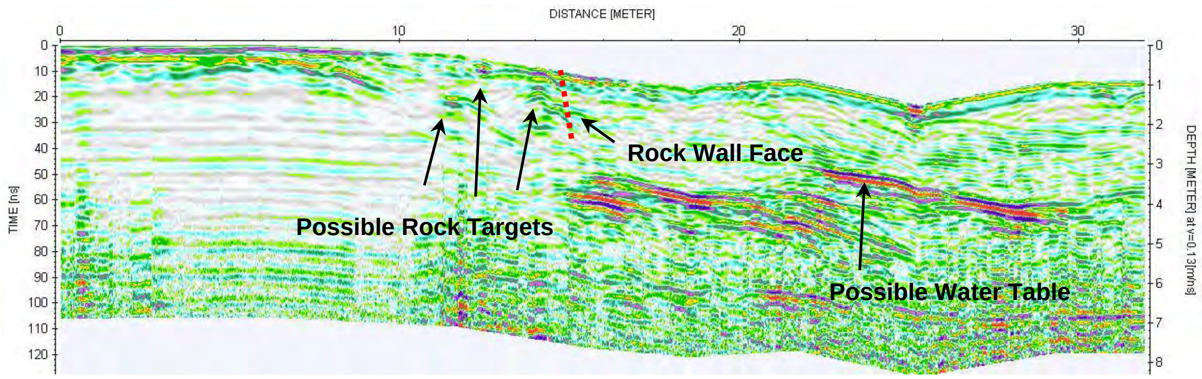

Figure 3: Sea Wall located on Line 37 (Excavation 1 surveyed with 200MHz)

A second excavation, approximately 400 m south of the Surf Club, yielded a sea wall consisting of a rock wall front, back by approximately 10m of rocky fill. The rock wall fill that was seen after shallow excavation (200-300 mm) was of smaller size in comparison to the rock wall face, however, as shallow excavation was not carried out further, an average size was not to be determined. The RL to the top of the wall front was 3.63 m, with an RL for the bottom of the wall at 2.1 m giving an approximate wall face thickness of 1.5 m. Refer to Figure 6.

The location of the sea wall proved difficult to locate with GPR. The smaller rocky fill behind the sea wall front created clutter within the data set, which in turn, made interpreting the contrasting signals between the sand fill and the larger rocks visible at the wall front more difficult. Furthermore, a strong signal was reflected at approximately 2-4 m depth (see Figure 3). This reflection was mistaken for the sea wall during the initial trial in May 2013, thus causing the wrong area to be targeted during the first excavation. It has been assumed this strong reflection is due to a possible water table, layer of organic material or high salinity moisture.

The GPR data shows the average depth to our interpreted sea wall ranges from 0.3 meters to 1.2 meters beneath the coastal sand dune, including the rocky fill. This is based on the correlation with two excavations carried out on the 9^{th}-10^{th} October.

Not all lines were able to locate the sea wall. For example, as can be seen from drawing GBG1579-01, lines 14, 15 and 16 did not extend to the sea wall. This was often due to fencing and/or thick vegetation.

Lines collected north of the Lennox Head Surf Lifesaving Club proved difficult to locate the sea wall, however were visually located. See Figure 4 below. Tougher terrain and a very uneven antenna ground coupling created relatively poor data resolution. The survey line extent of 42, 43 and 44 were carried out fully with GPR but were unable to be collected with GPS due to a large amount of trees in the area, however, once out of the vegetated area, the remainder of the line was able to be collected. On drawing GBGA1579-01, Lines 42, 43 and 44 have had an assumed GPR profile line orientation mapped where GPS was not available.

Figure 4: Rock wall visible in forested area north of Lennox Head Surf Club.

Figure 5: GBG Australia staff, Jack Ellis, collecting GPS points at the sea wall exposure on 9th October. Line 37.

Figure 6: Rock exposure visible following the second excavation on 10th October, adjacent to Line 18.

5. CONCLUSIONS

GBG Australia was tasked with mapping the location of Lennox Head sea wall at pre-located transverse profiles running approximately 1.4km along the beach in central Lennox Head. The survey was undertaken with GPR and referenced using RTK GPS to relocate our survey lines. The results can be seen on drawing GBGA1579-01 to 03.

GBG Australia's interpretation of a sea wall target, based on the two (2) excavations on site, has been identified on the majority of the GPR lines collected. These can be seen in GBGA1579-04. A number of lines have not identified the sea wall, most notably due to inaccessibility or rough terrain.

Disclaimer: - Many of the findings in this report are drawn from the interpretation of electrical and electromagnetic signals in conjunction with calibration and correlation carried out on site. The conclusions drawn represent the best professional opinions of the authors, based on their experience and results of excavations on similar materials.

I hope that this provides you with the information required. If you require clarification on any points arising from this investigation do not hesitate to contact me.

For and on behalf of

GBG AUSTRALIA PTY LTD

JACK ELLIS

Technical Officer/Geophysicist

Attachments: GBGA1579-01 to 04

GBG Australia. 18 Fennell Street North Parramatta NSW. 2151 Australia. Tel: (02) 9890 2122 Fax: (02) 9890 2922

Appendix B Predicted Rock Seawall Sections (GBG)

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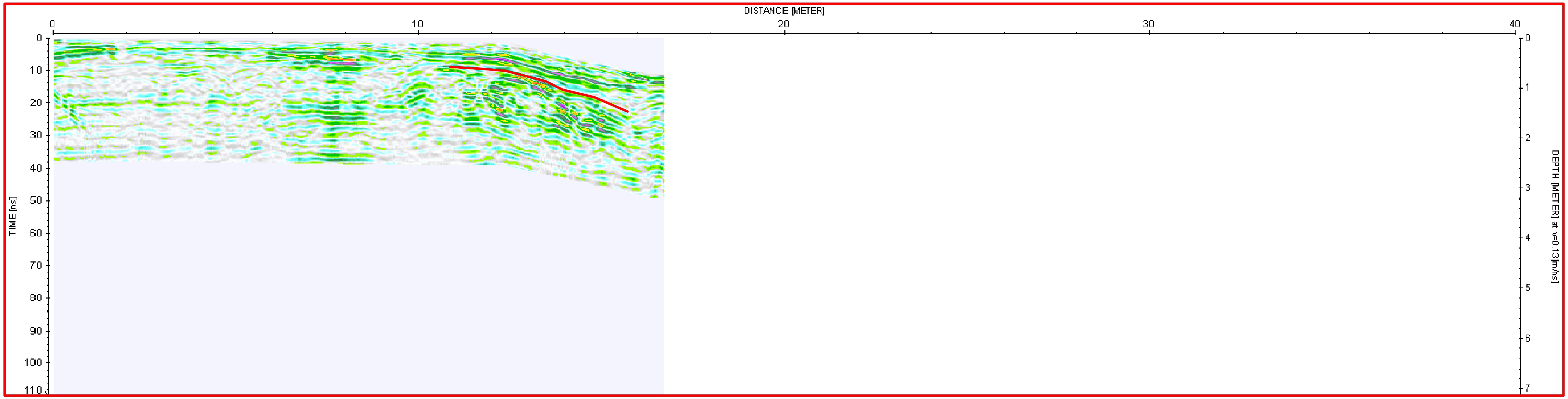

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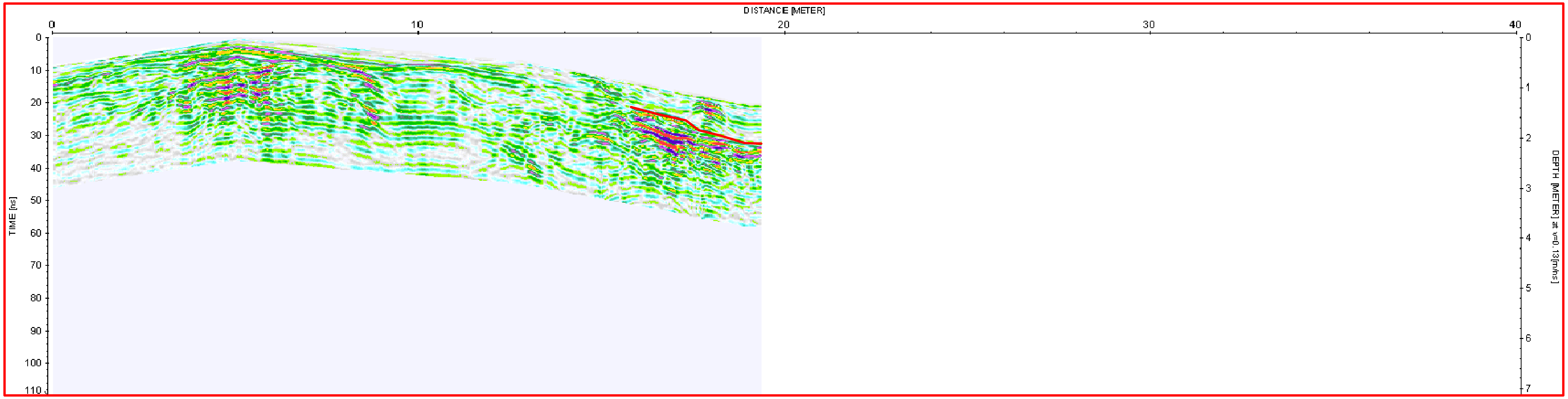

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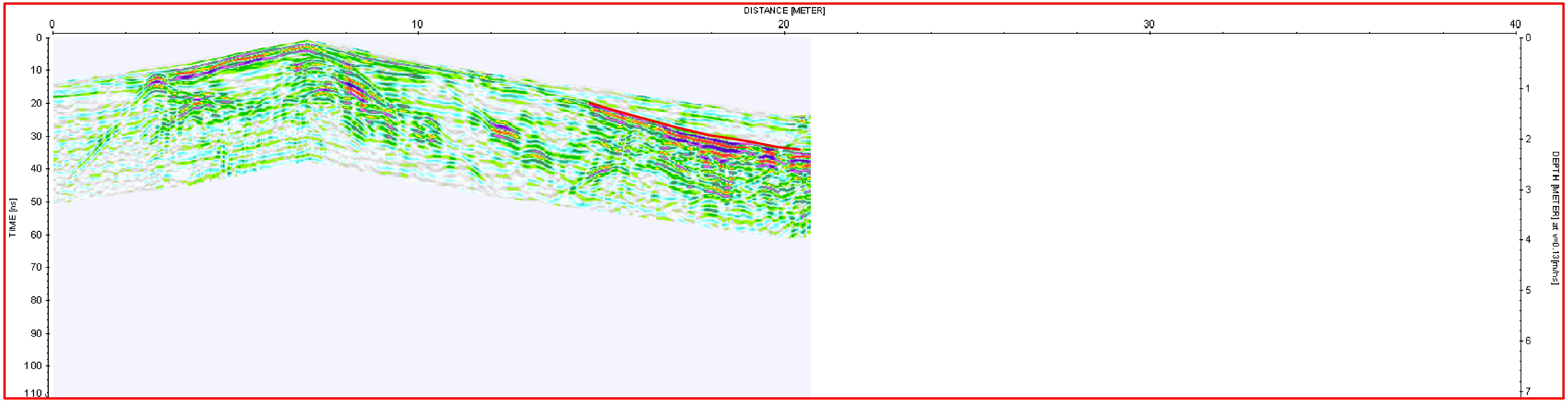

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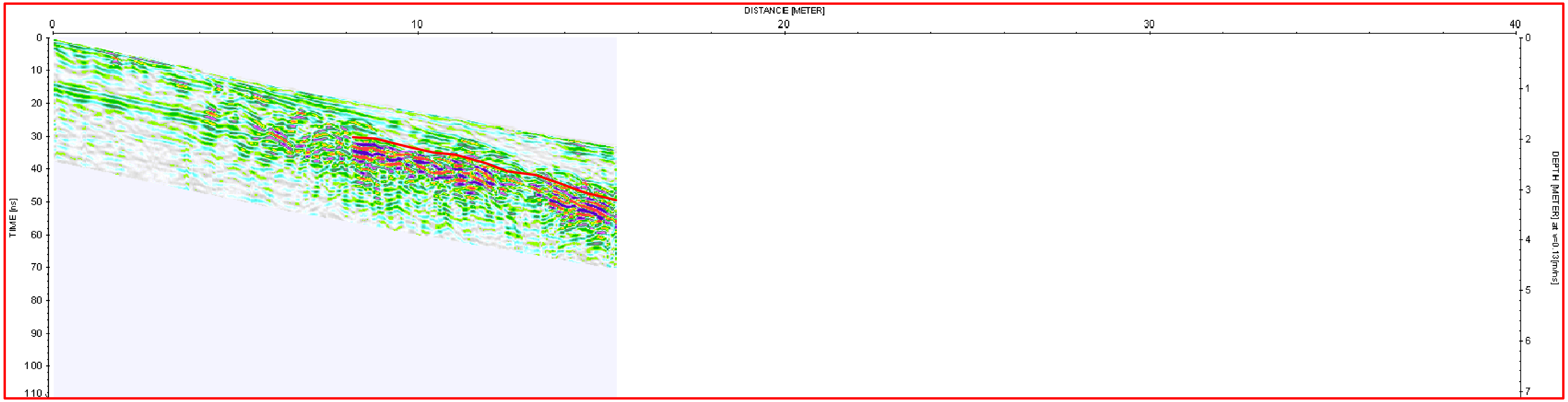

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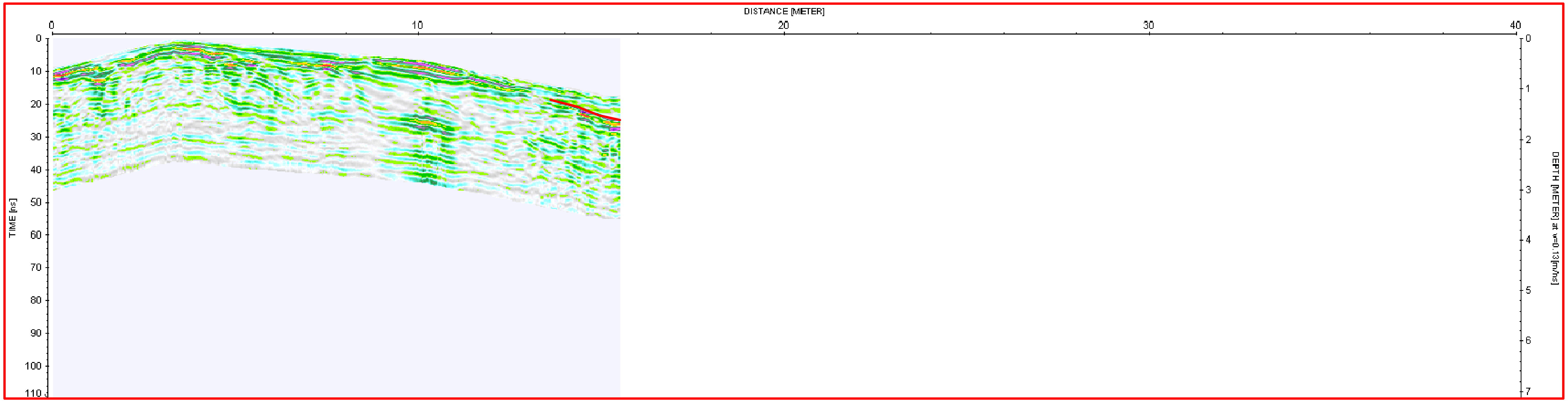

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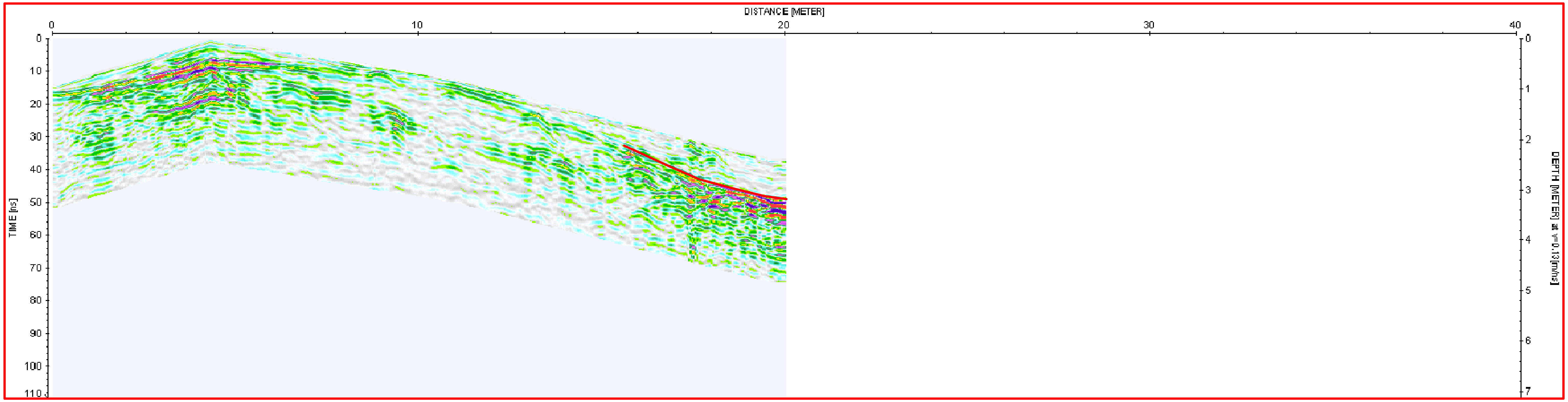
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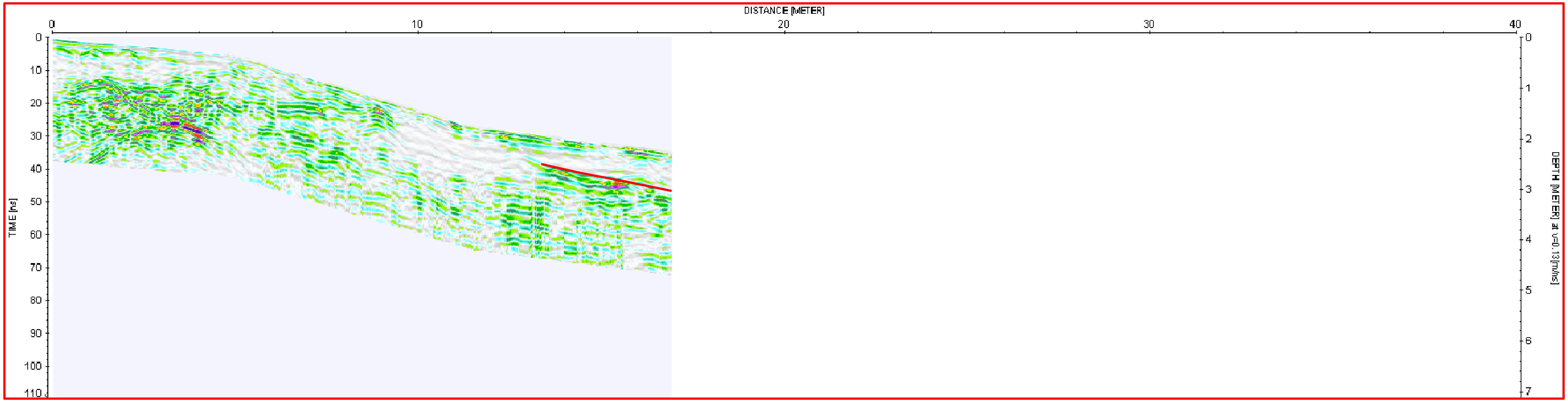
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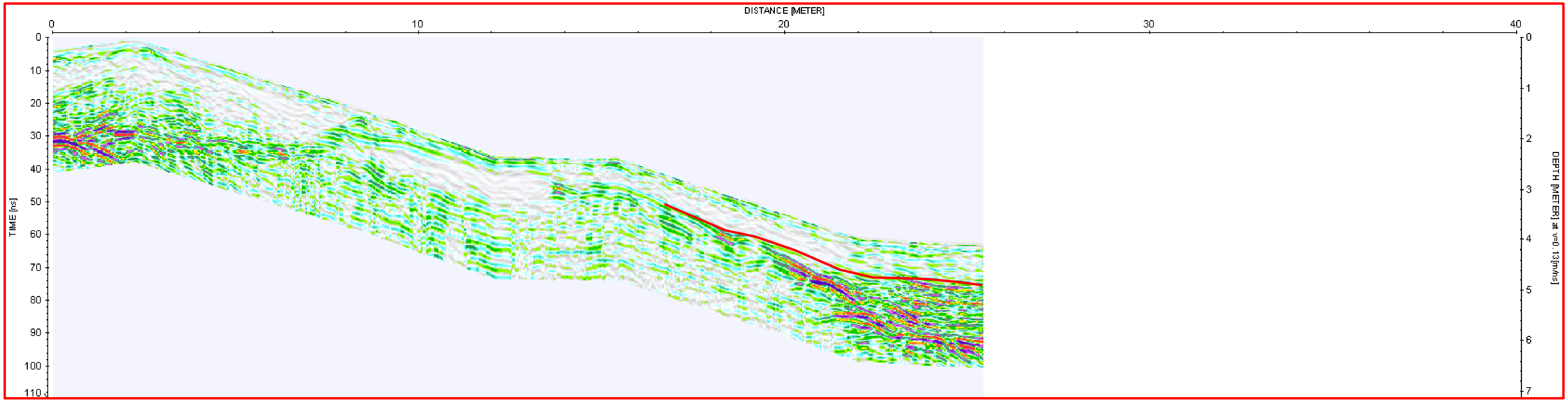

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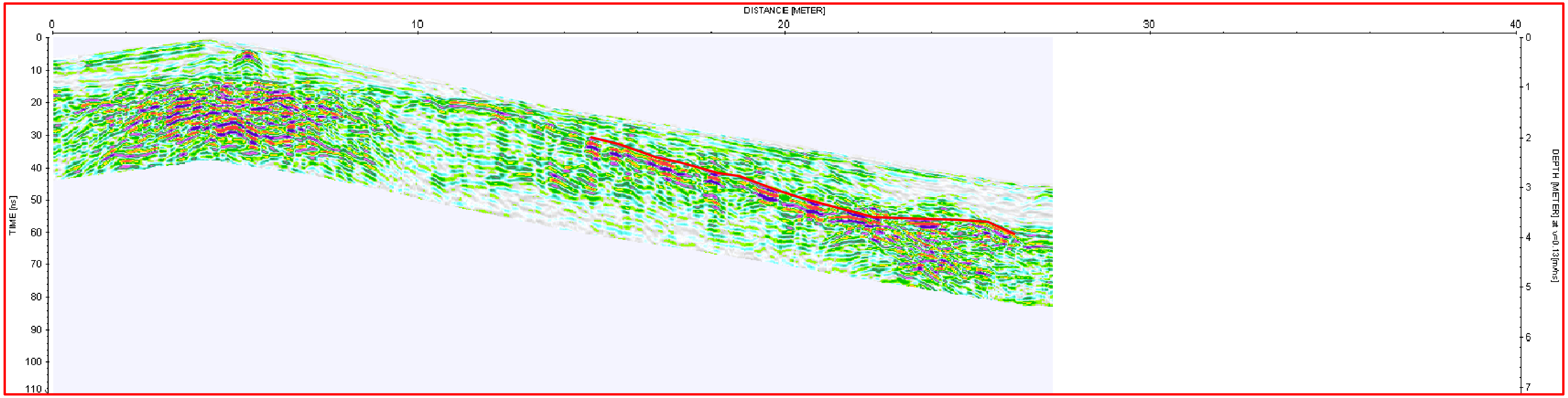

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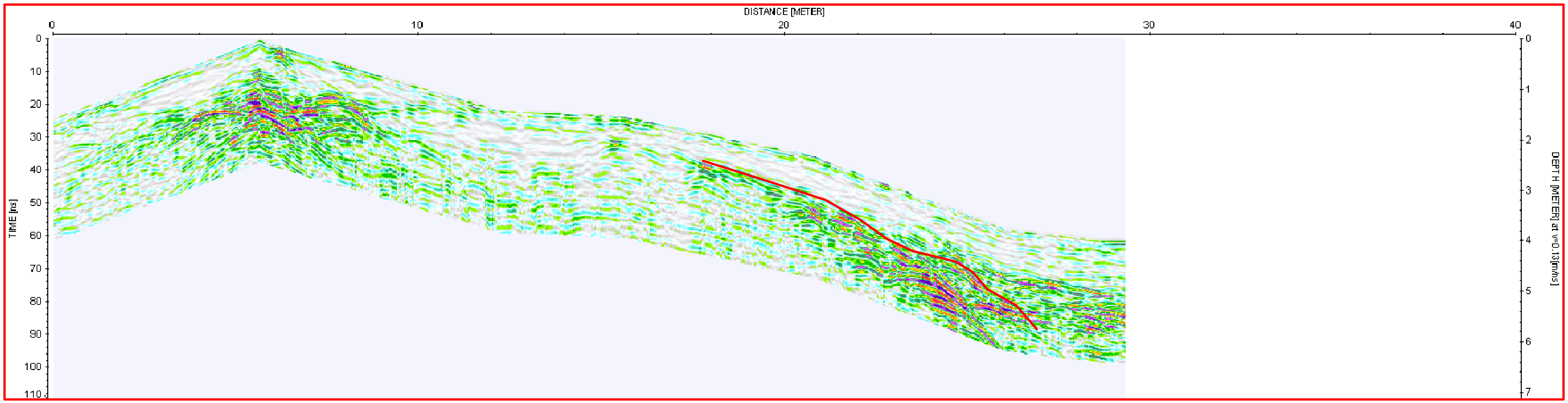

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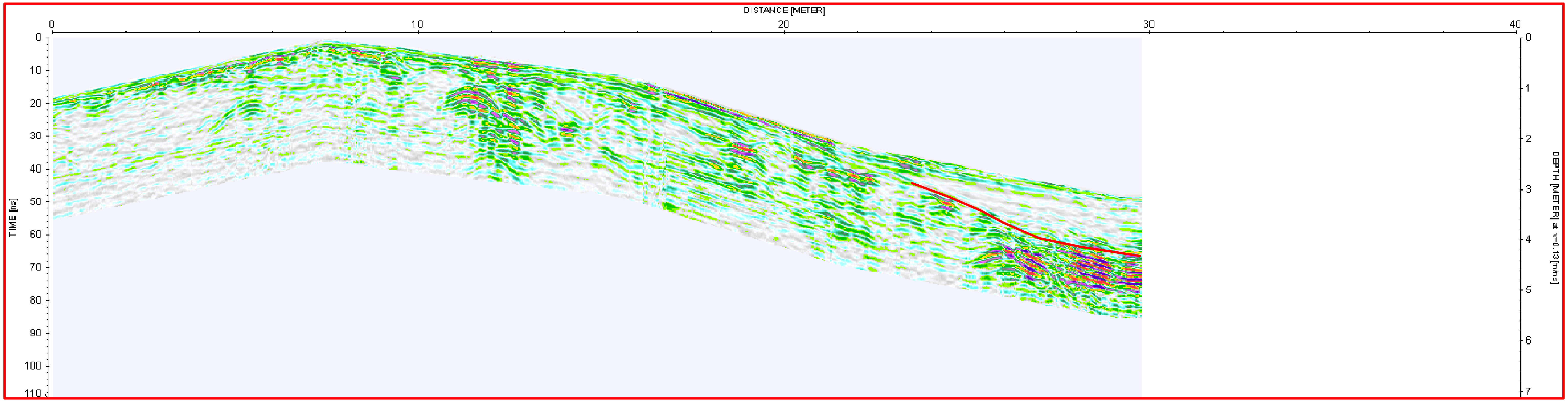

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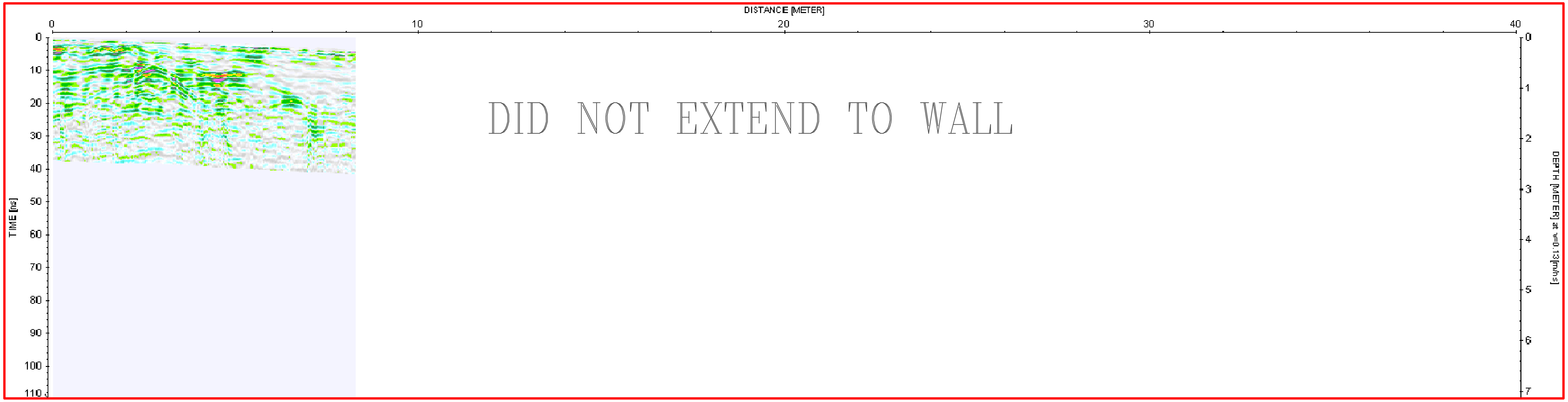
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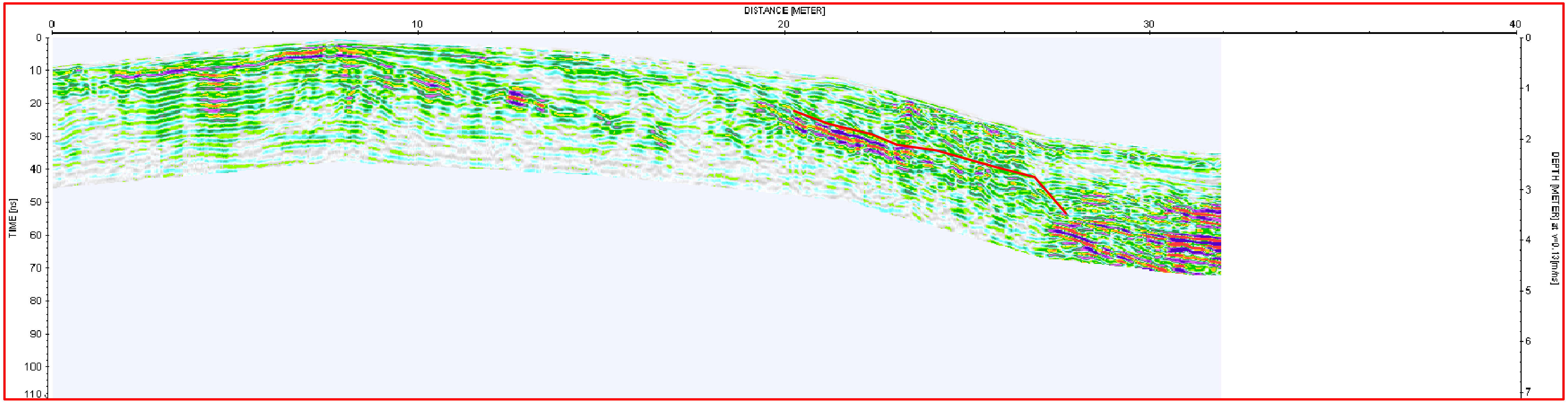

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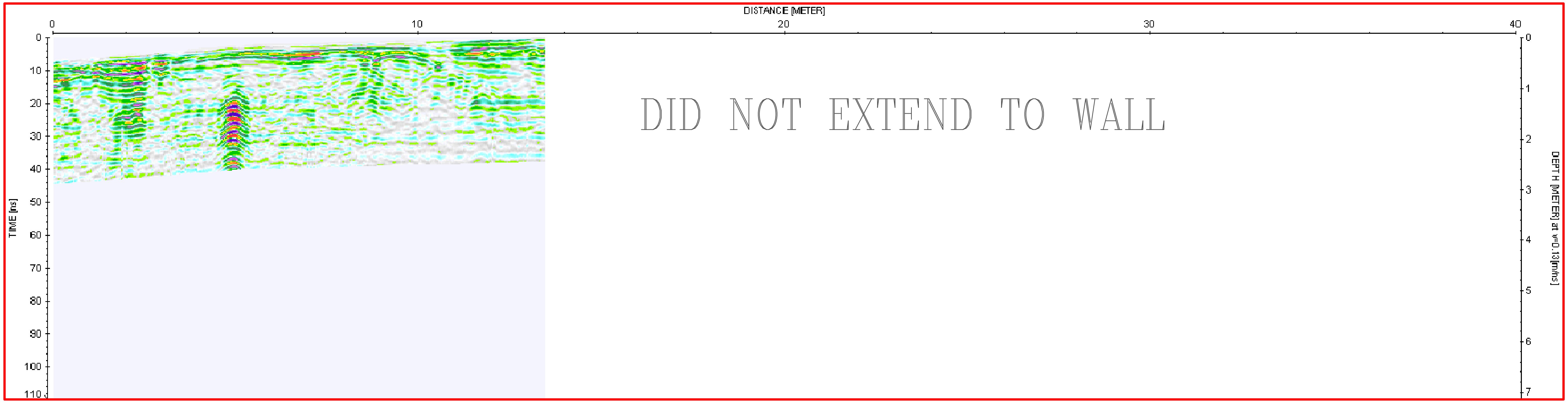

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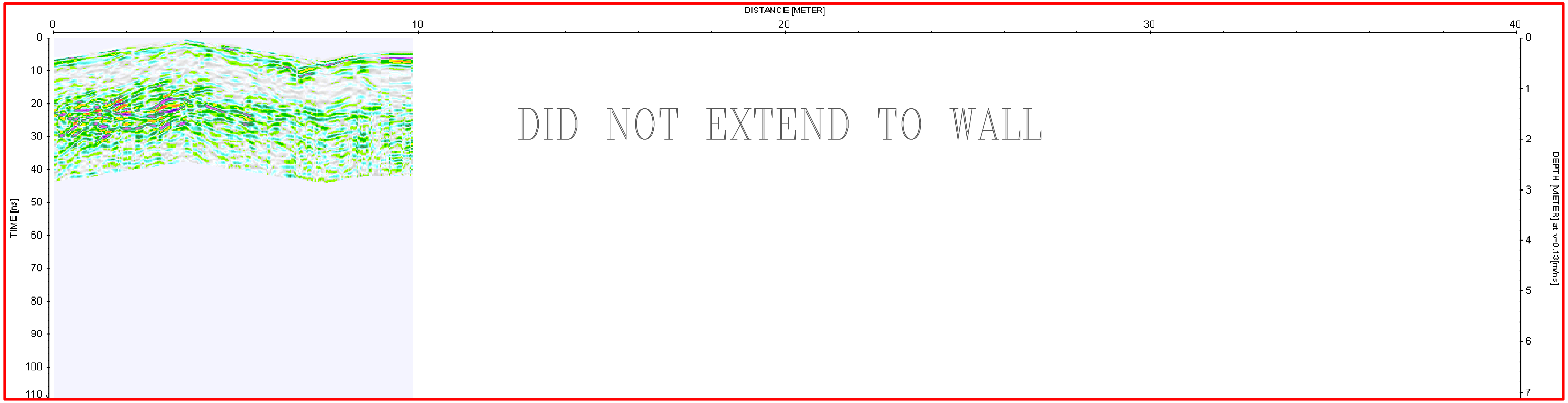
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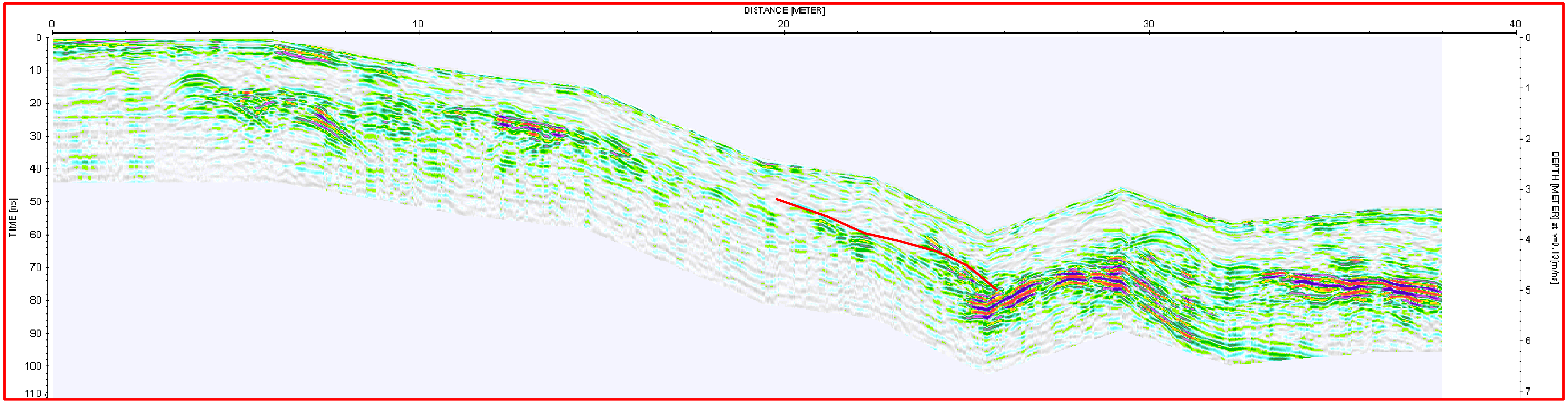
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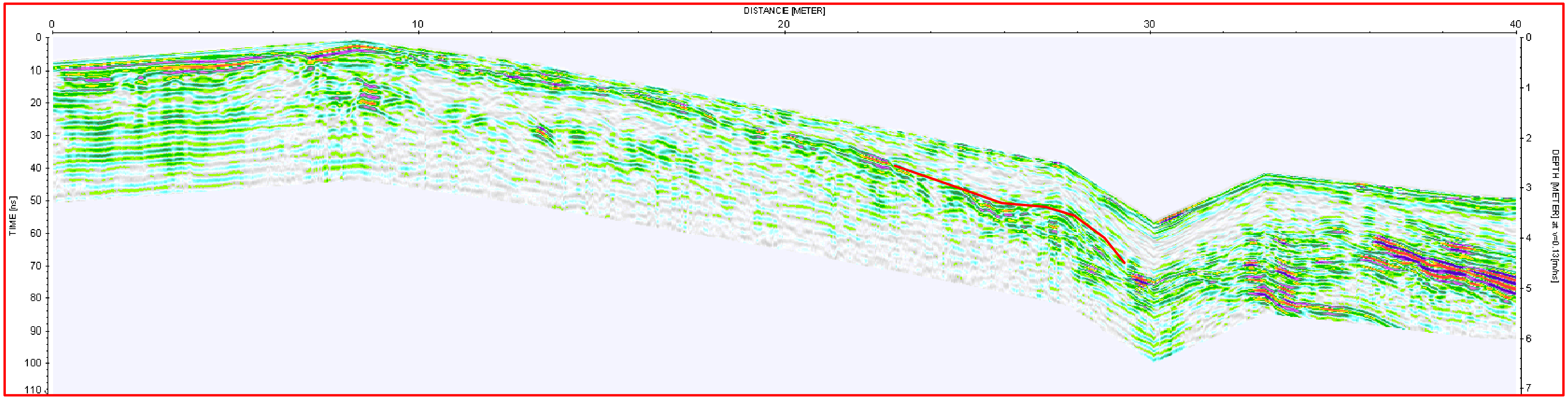

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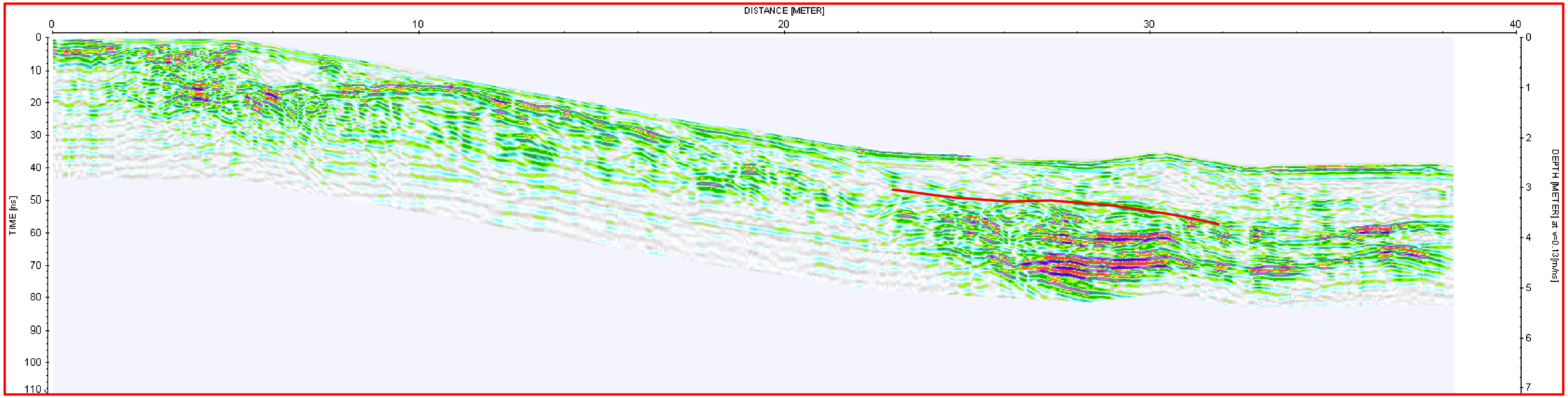
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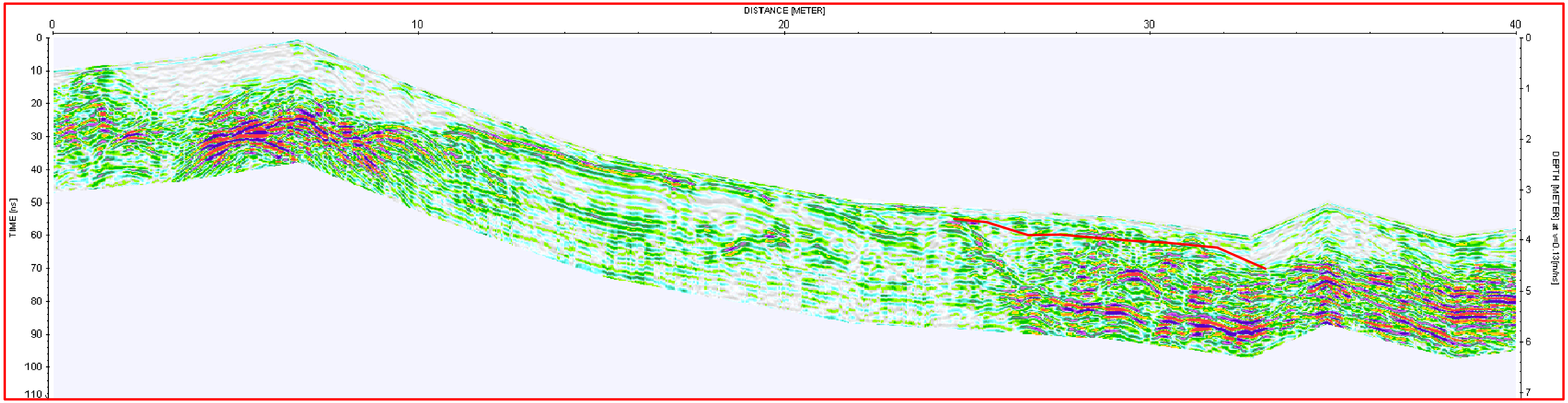

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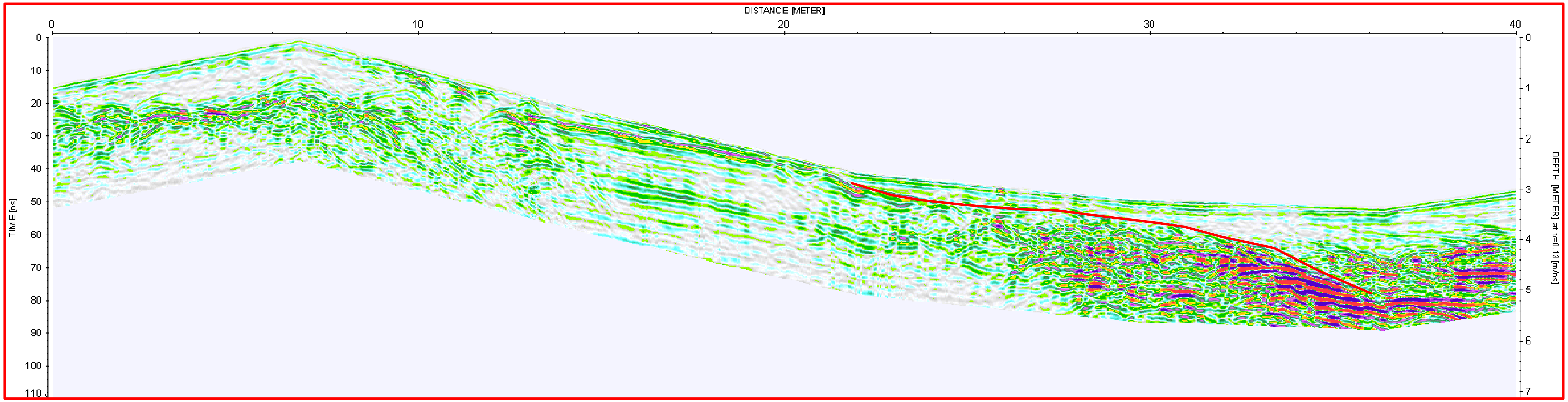

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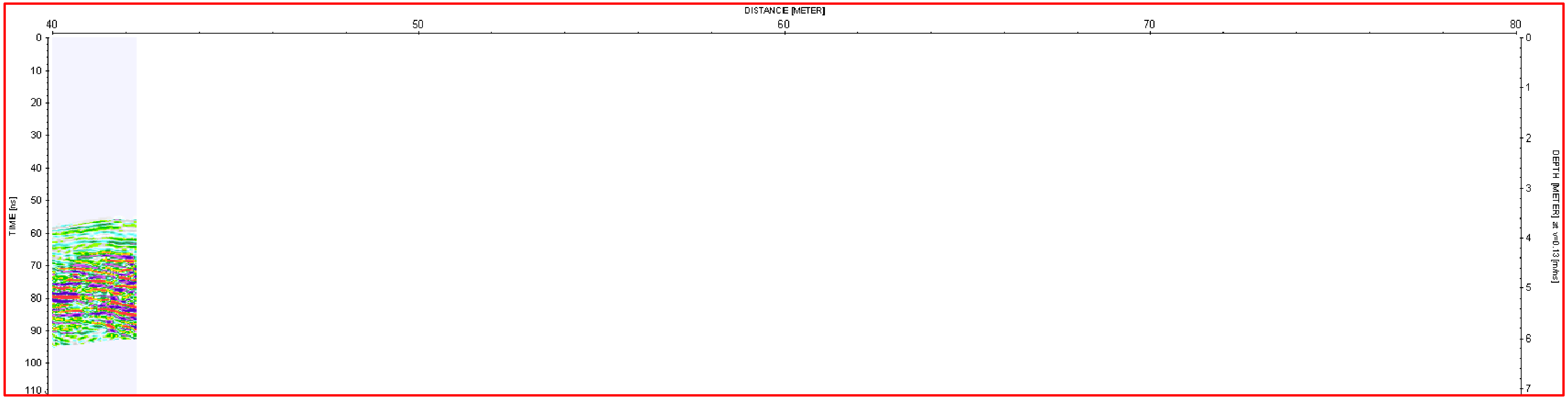

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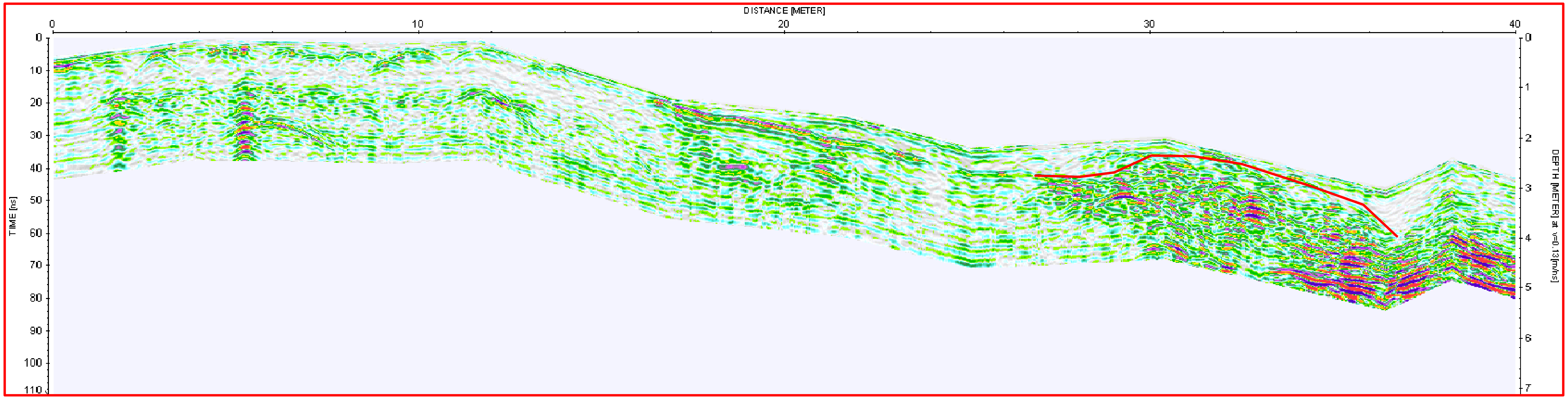

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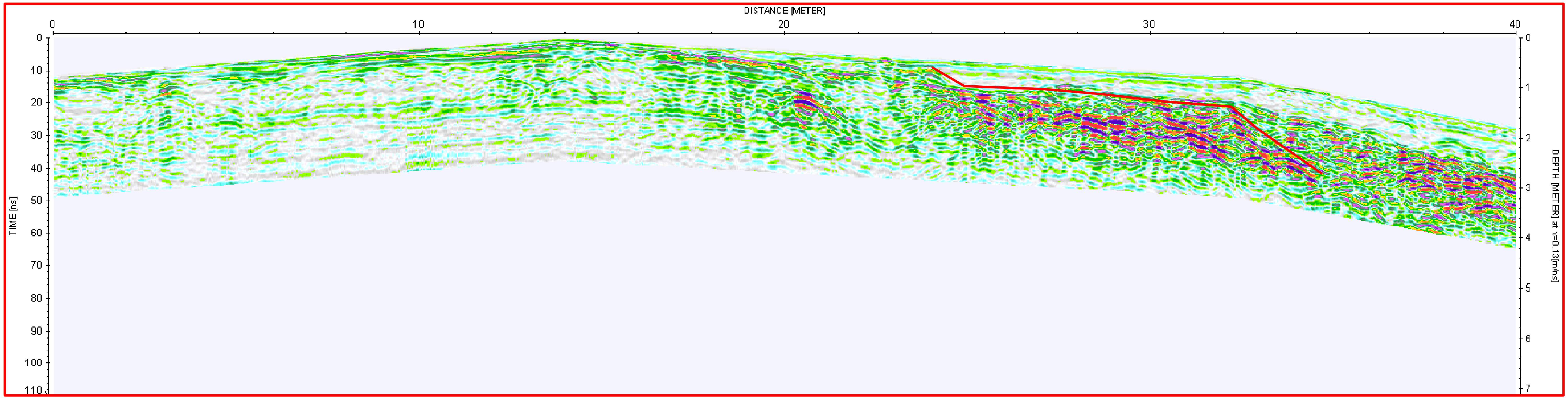
LINE 23



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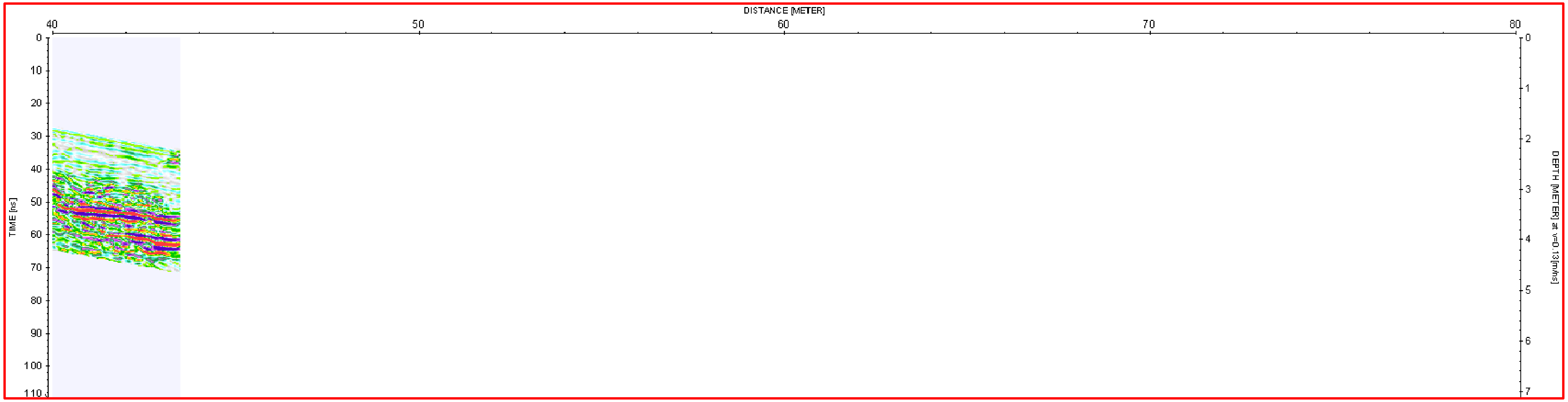
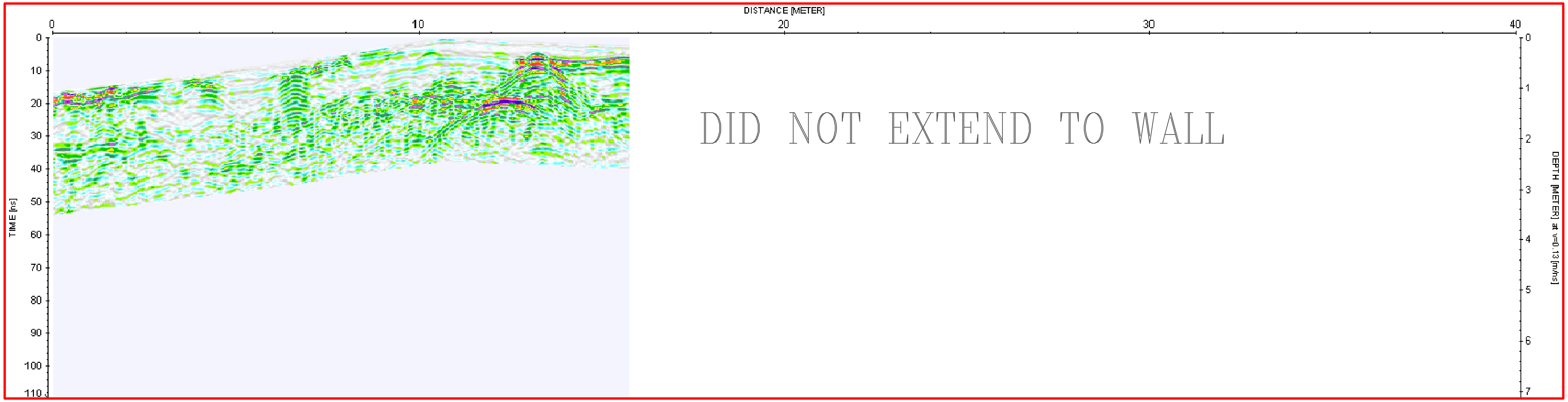
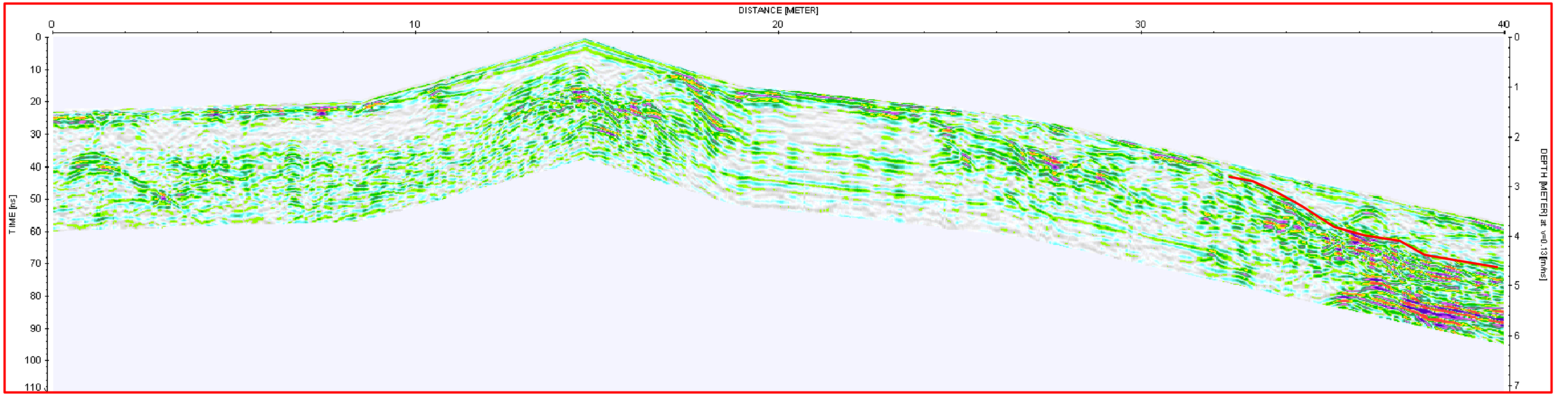


LINE 26



DISTANCE [METER]
0
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13 [m/ns]
0
1
2
3
4
5
6
7
DID NOT EXTEND TO WALL

LINE 27



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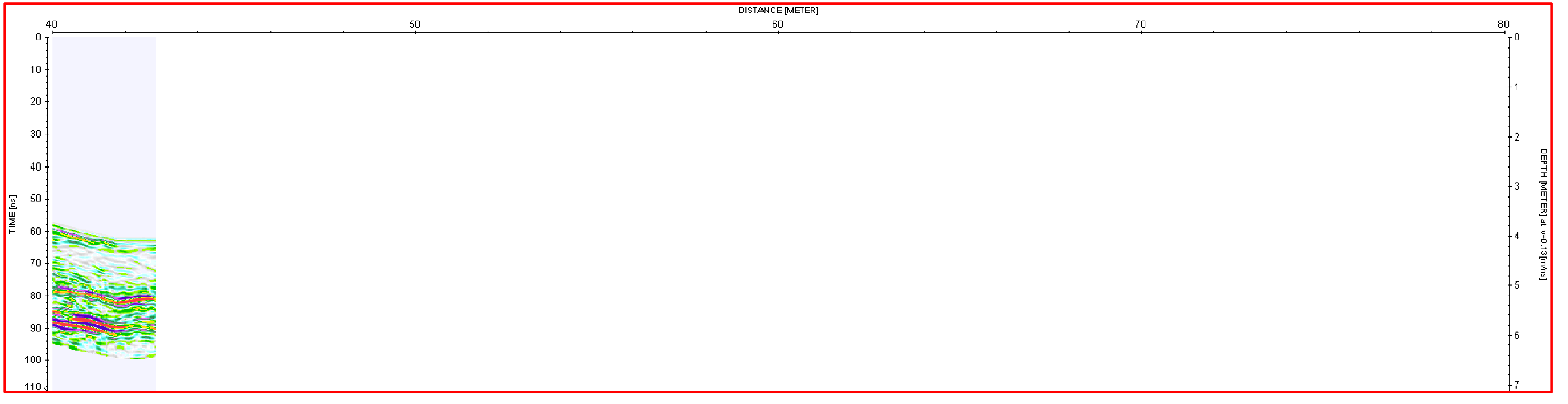

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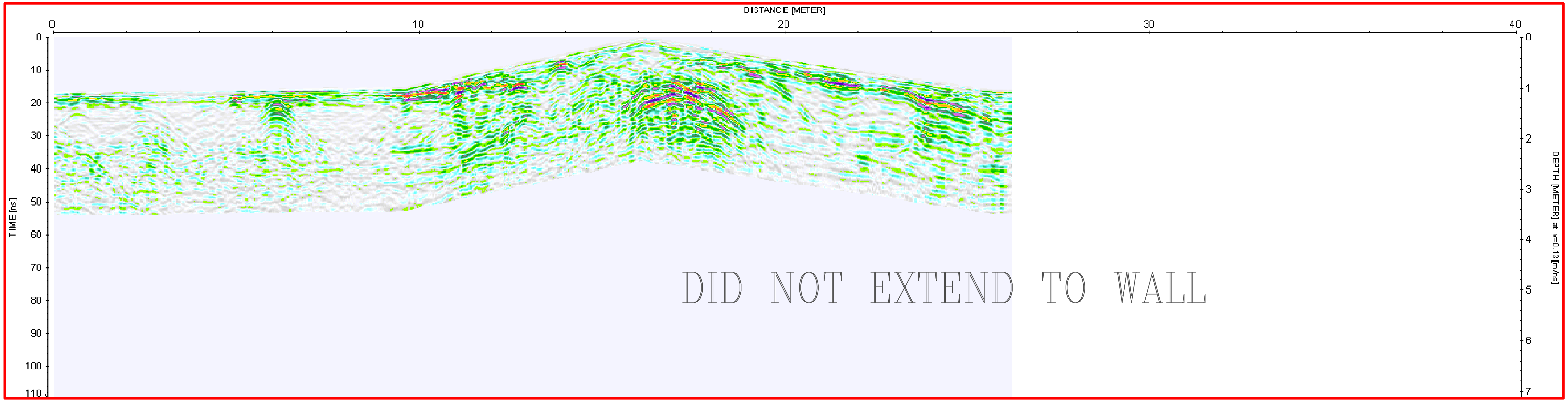

DISTANCE [METER]
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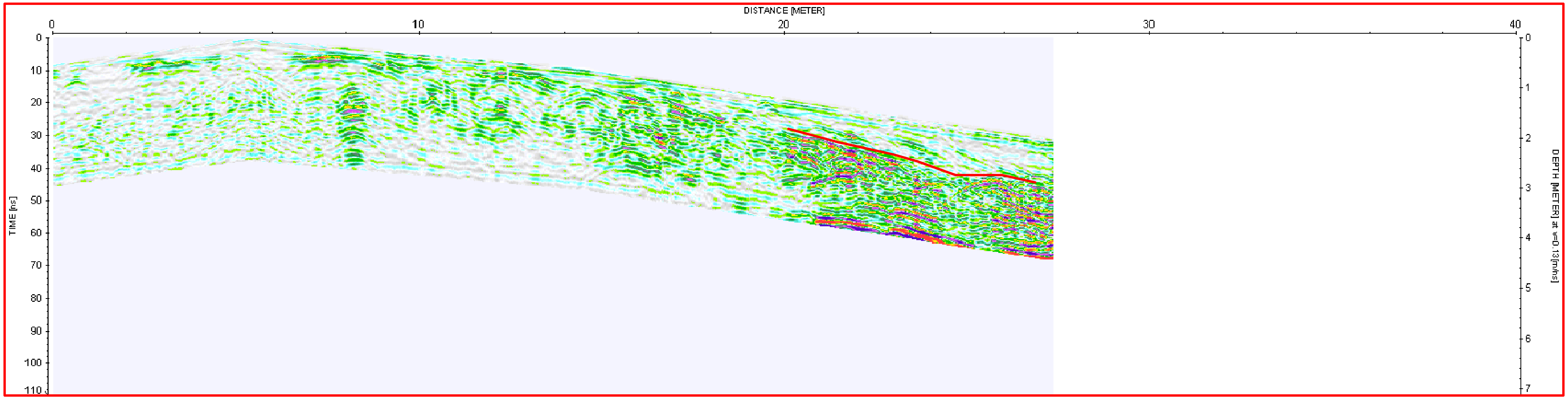
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7
DID NOT EXTEND TO WALL

LINE 29


DISTANCE [METER]
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10
20
30
40
TIME [ns]
0
10
20
30
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50
60
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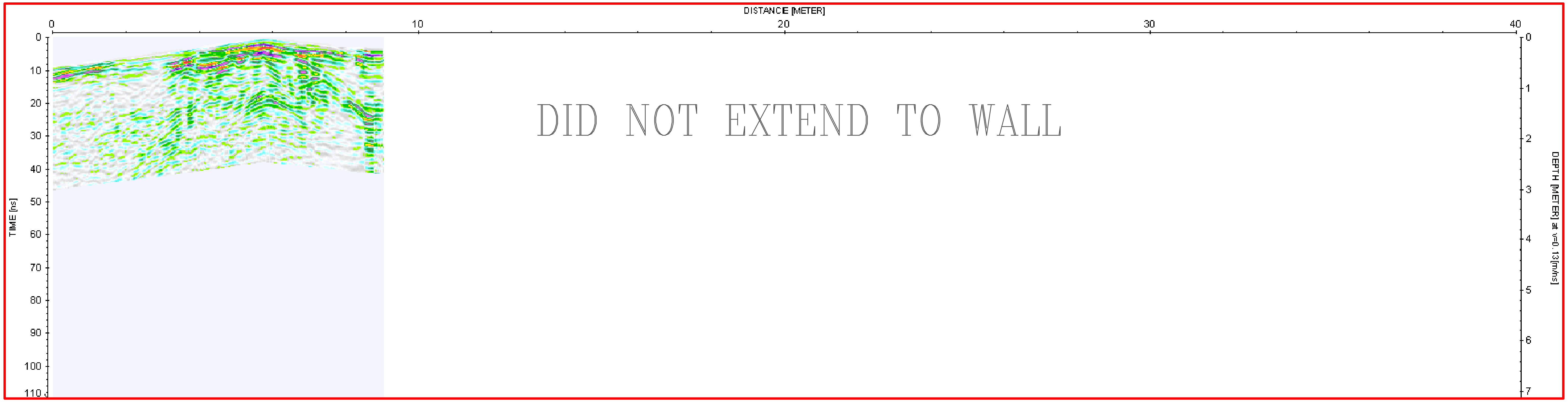
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7

LINE 30



DISTANCE [METER]
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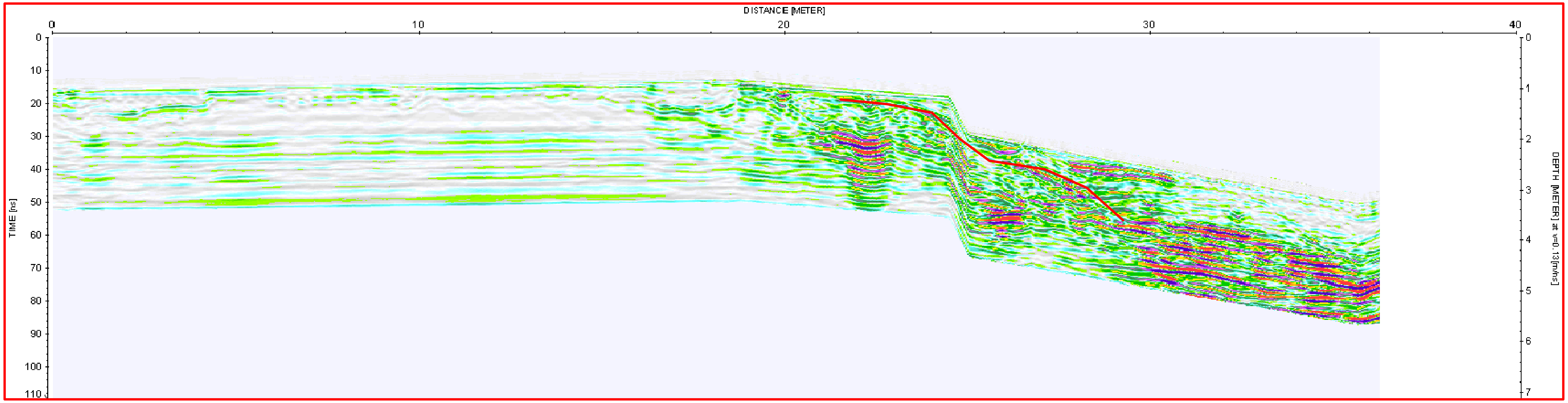
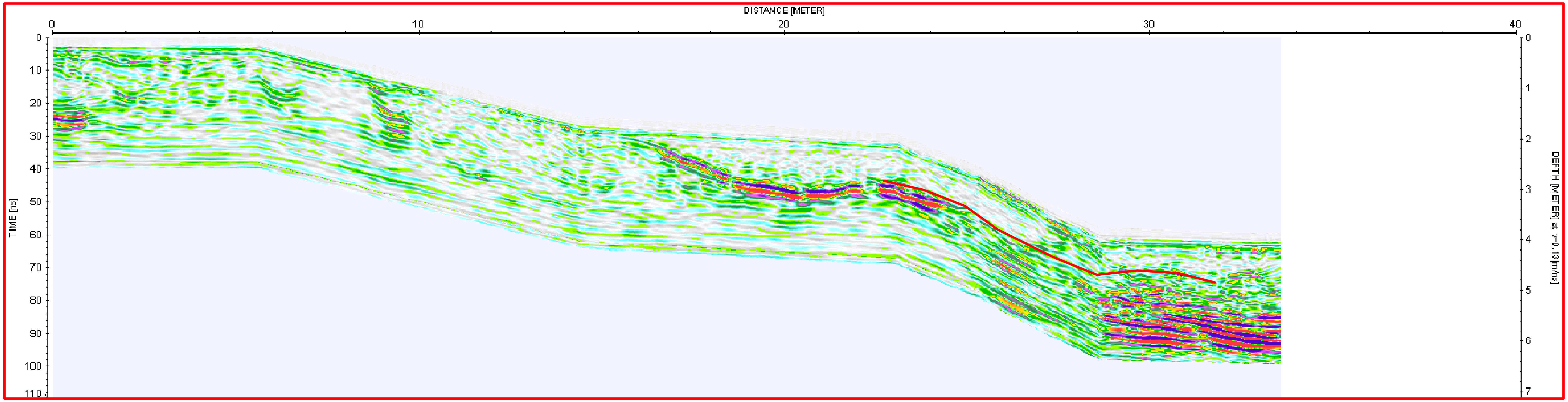
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7
DID NOT EXTEND TO WALL

LINE 31



DISTANCE [METER]
0
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7

LINE 33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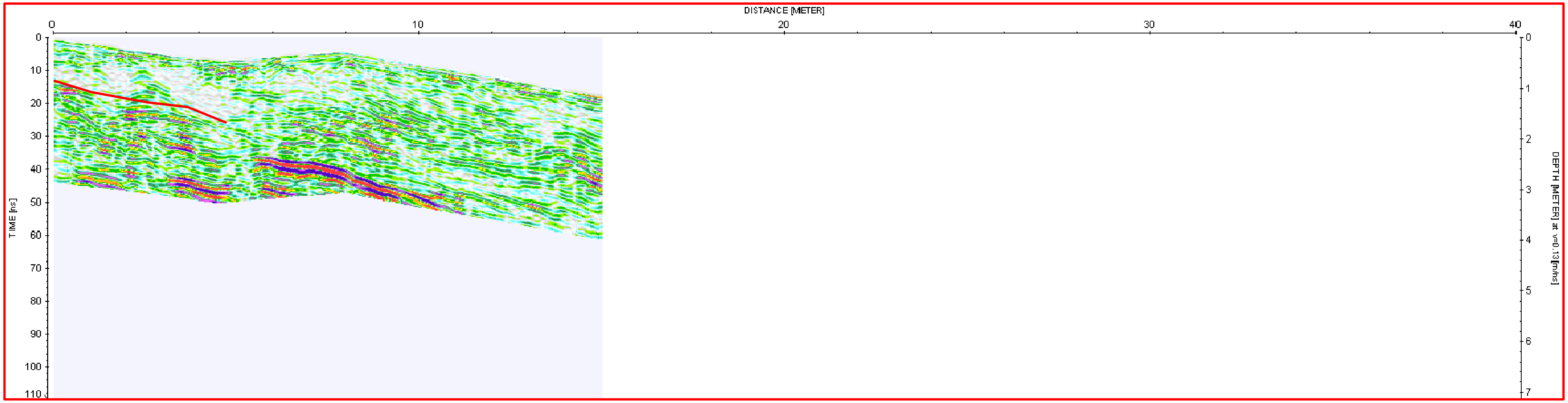
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7

LINE 35


DISTANCE [METER]
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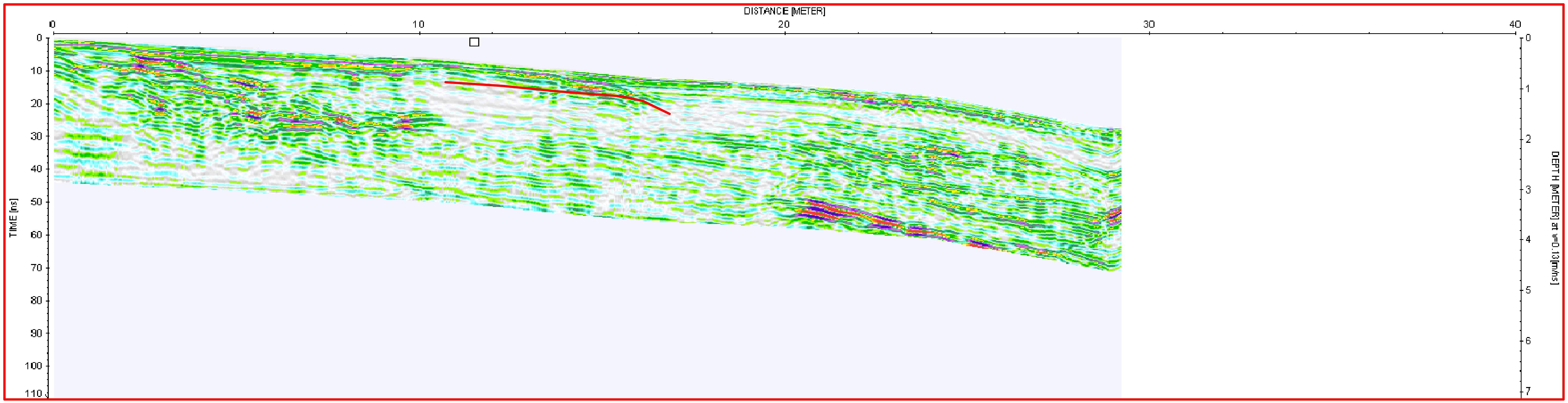
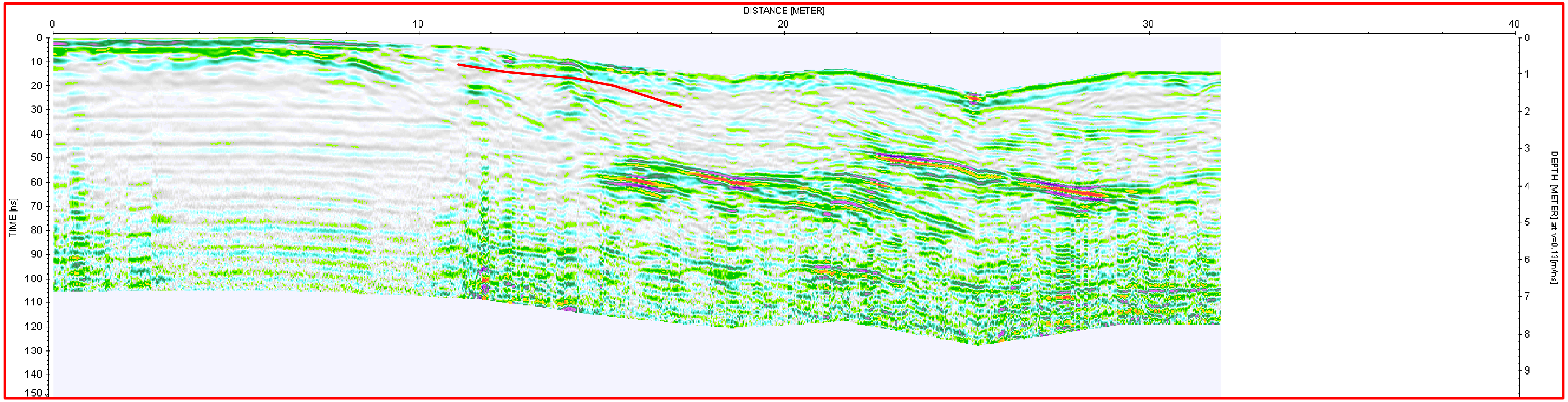
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7

LINE 36



DISTANCE [METER]
0
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7

LINE 37



DISTANCE [METER]
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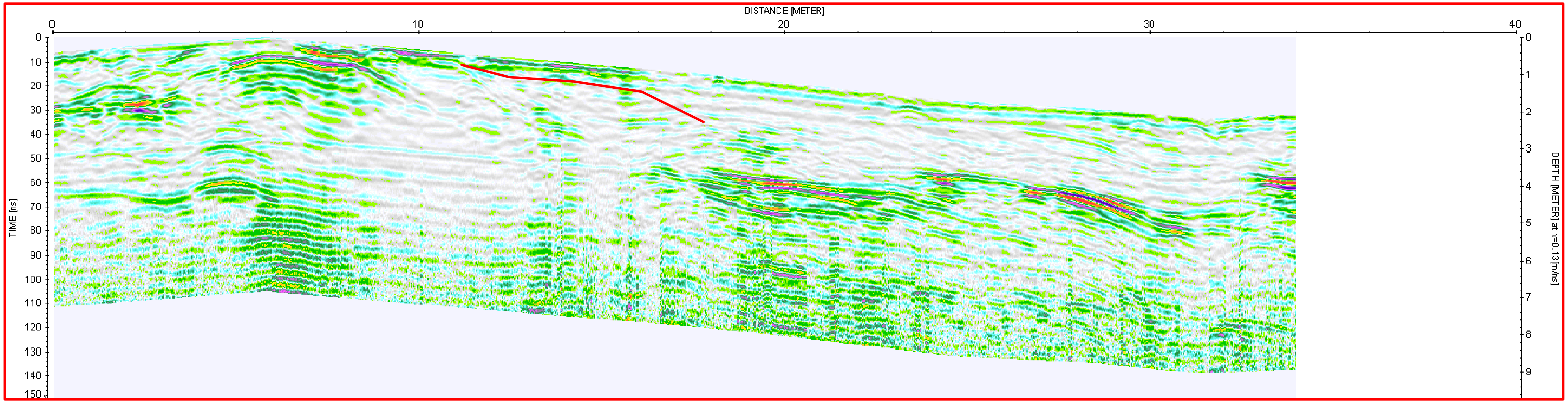
30
40
TIME [ns]
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7
8
9

LINE 38



DISTANCE [METER]
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10
20
30
40
TIME [ns]
0
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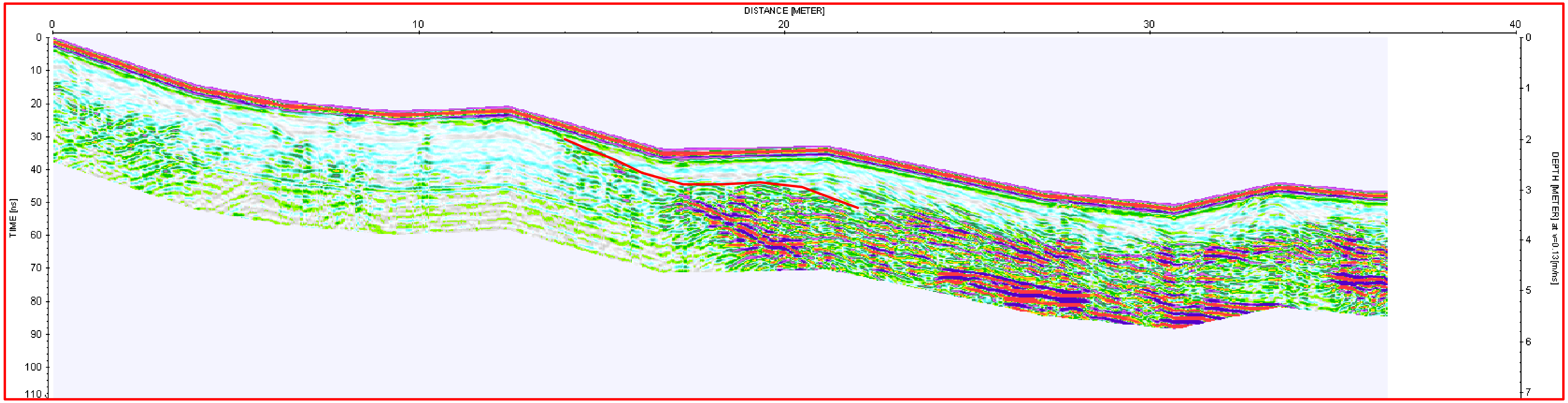
20
30
40
50
60
70
80
90
100
110
120
130
140
150
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7
8
9

LINE 39


DISTANCE [METER]
0
10
20
30
40
TIME [ns]
0
10
20
30
40
50
60
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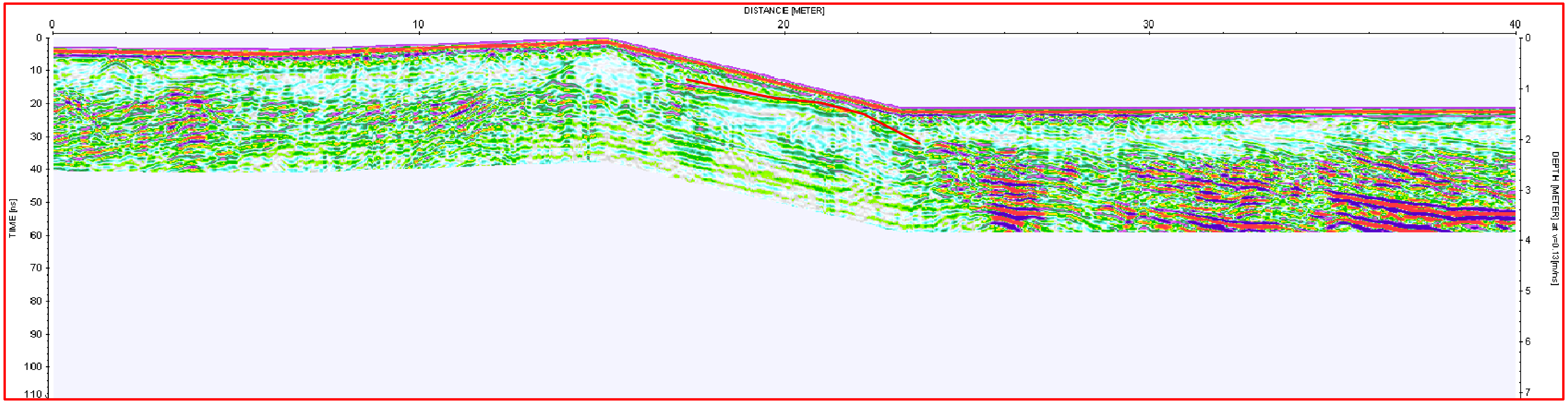
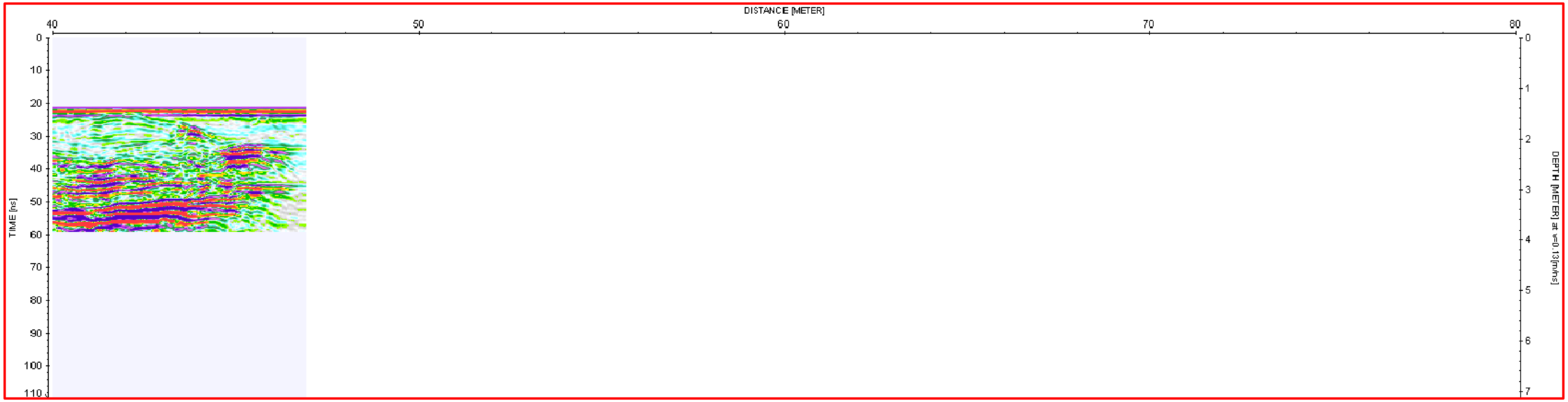
100
110
DEPTH [METER] at v=0.13[m/ns]
0
1
2
3
4
5
6
7

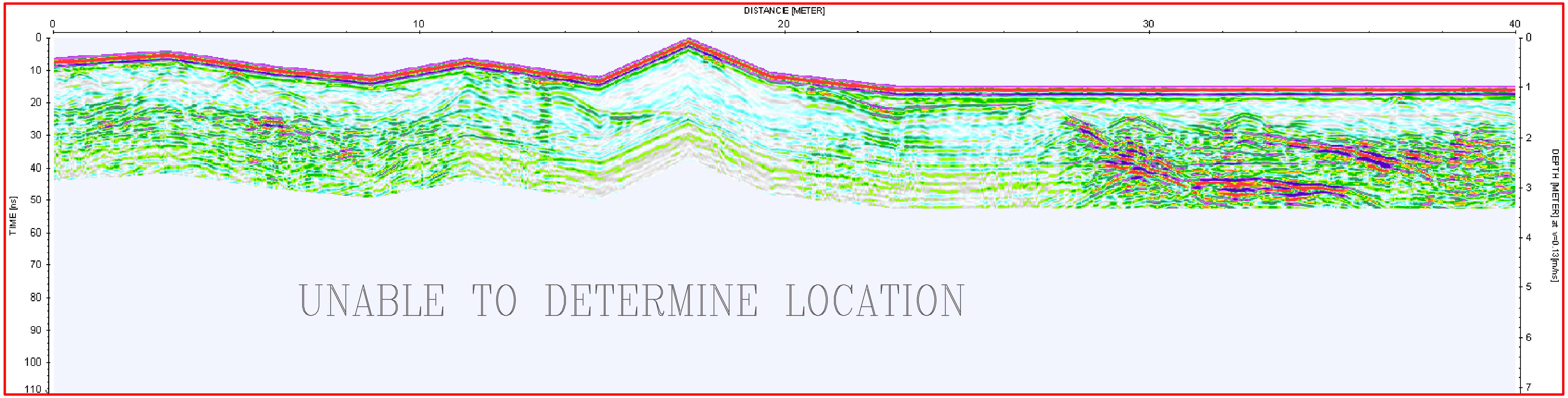
LINE 42



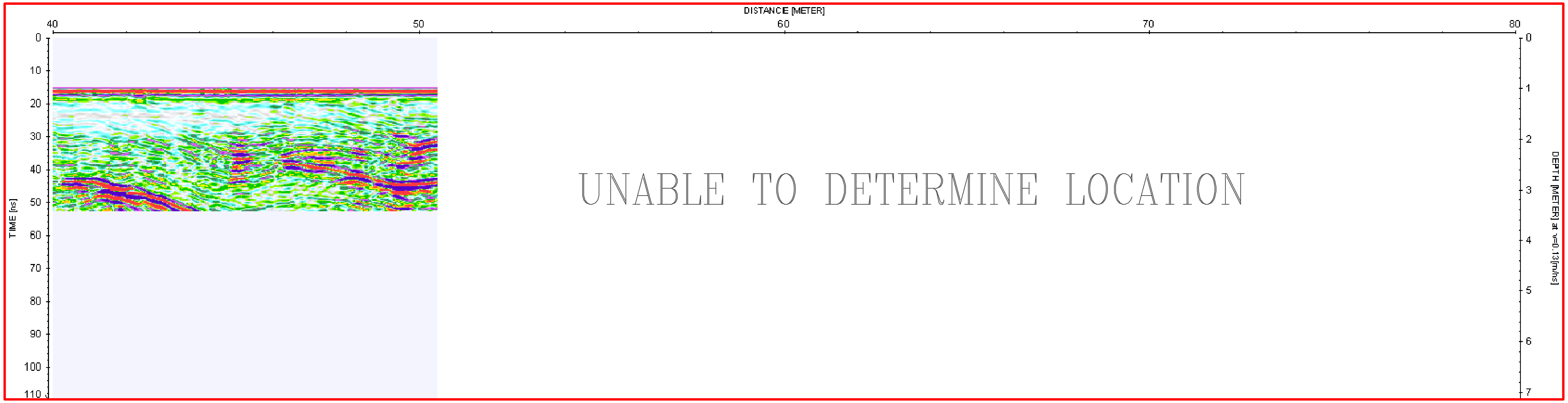
LINE 42 CONT.



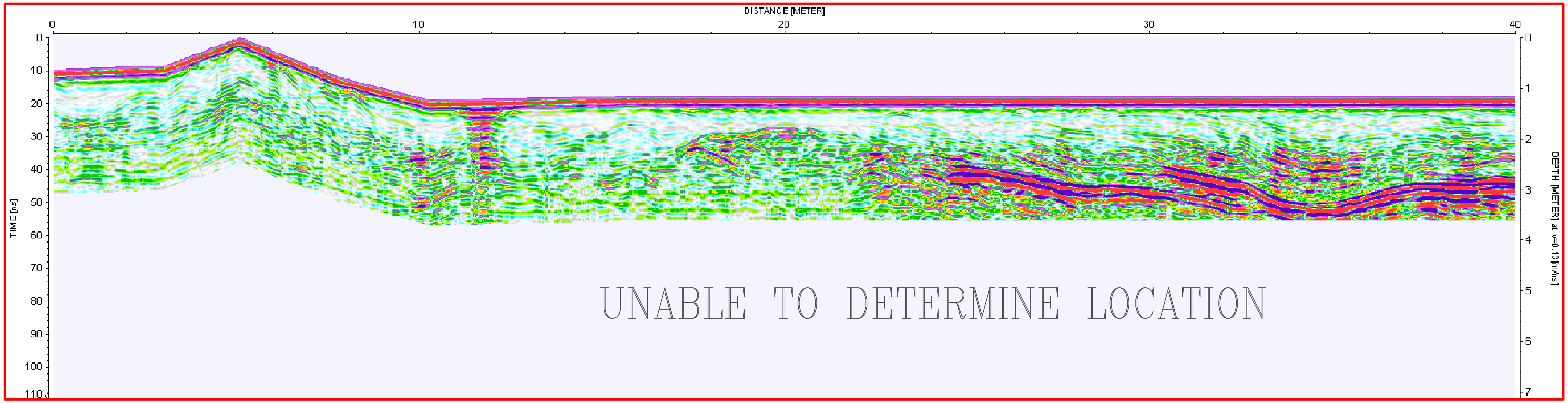
LINE 43



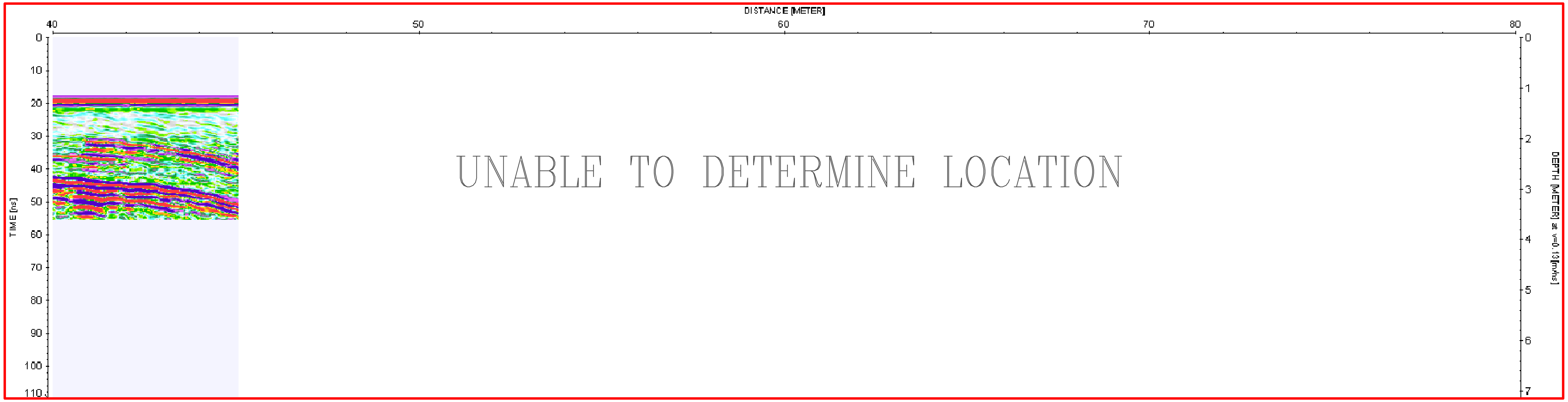
LINE 43 CONT.



LINE 44



LINE 44 CONT.



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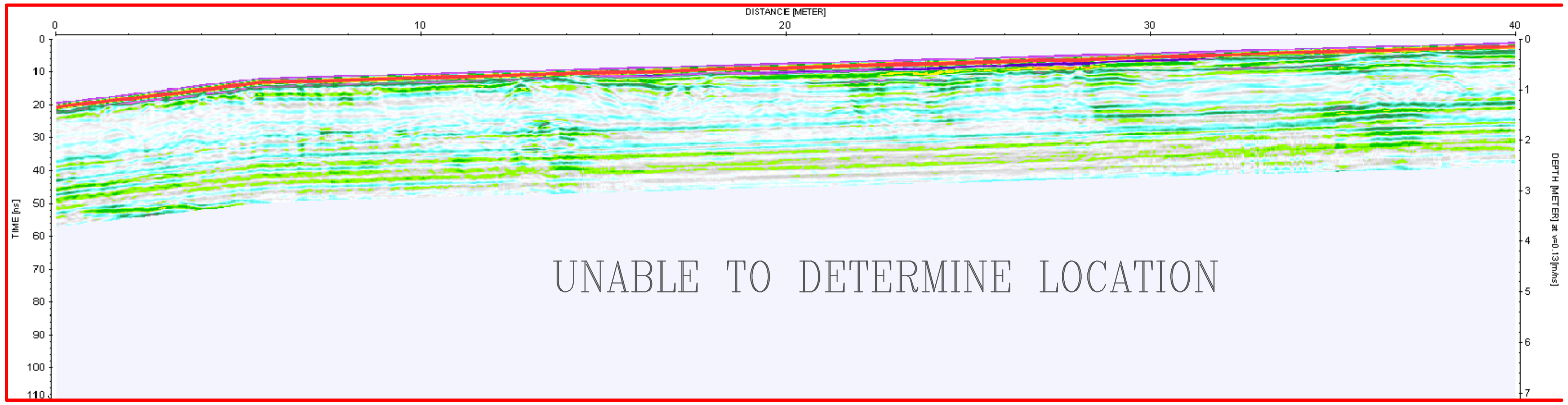
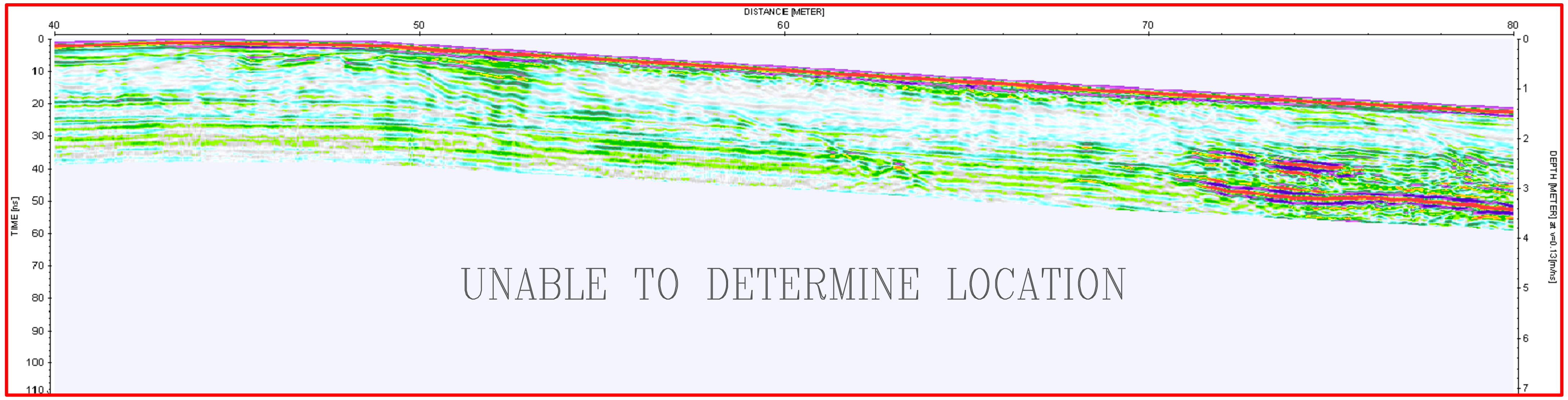
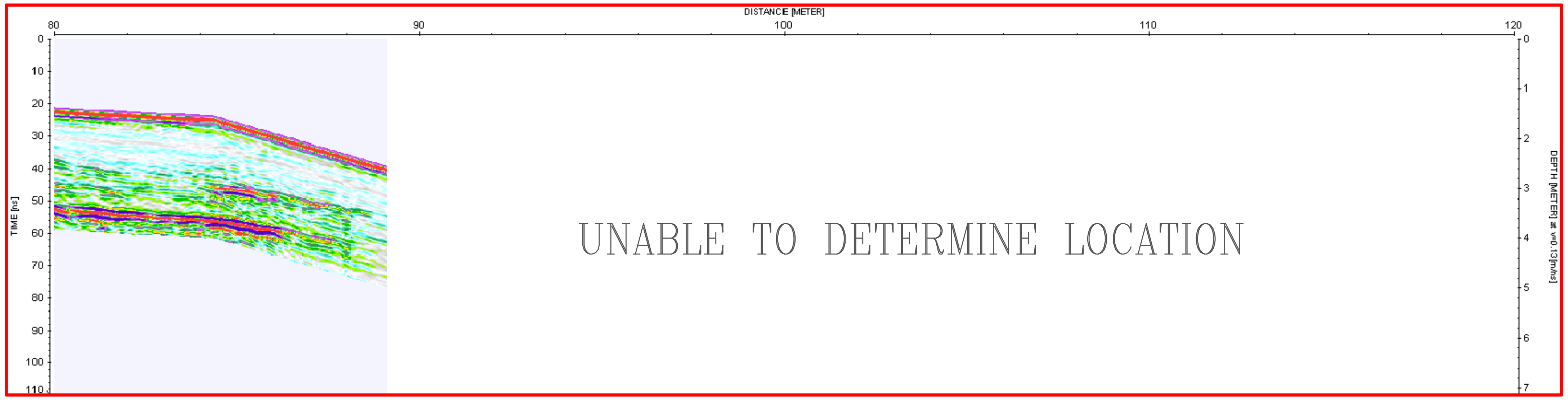


LINE 45 CONT.

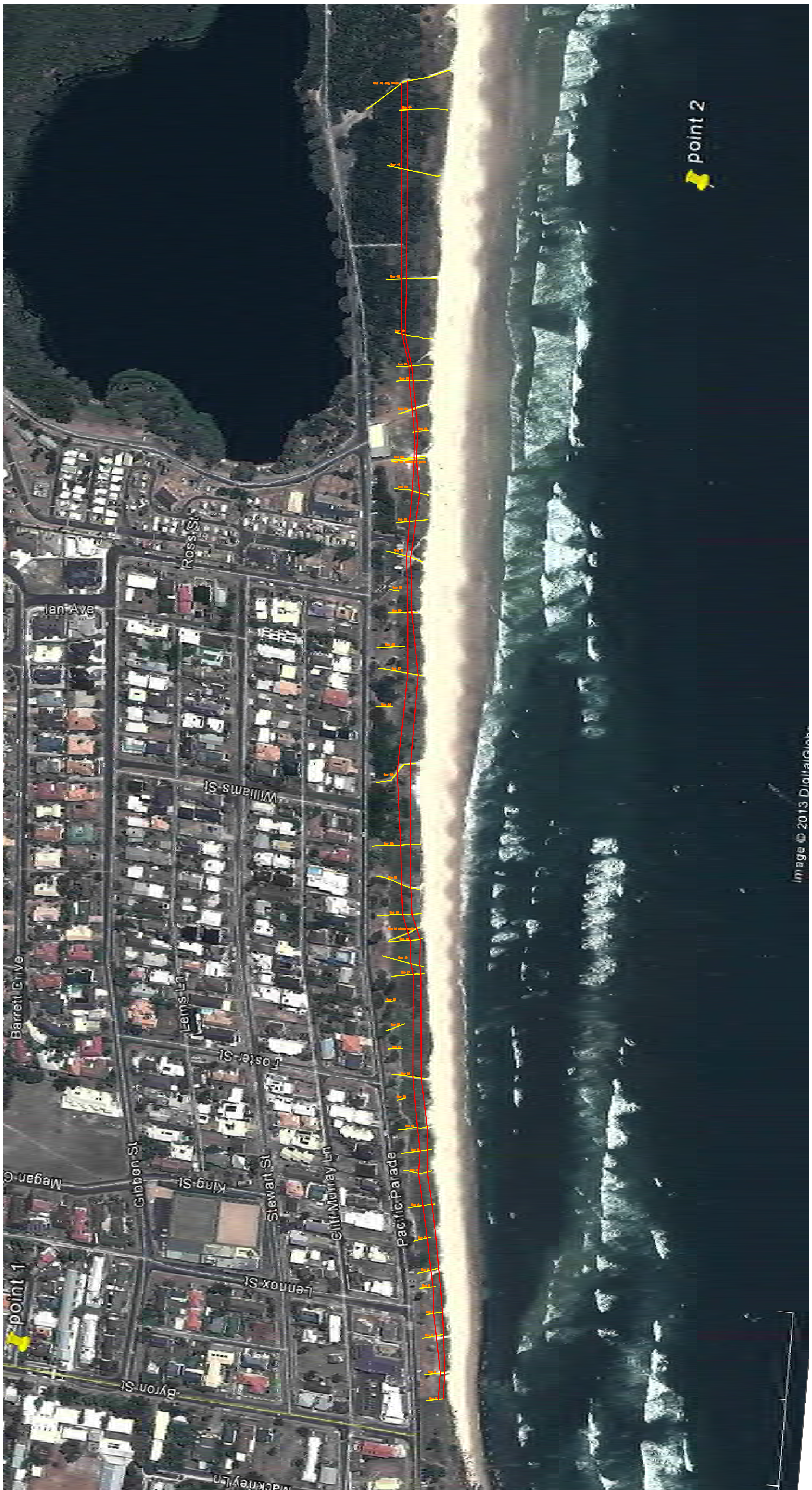


LINE 45 CONT.



GBG Australia

GROUND PENETRATING RADAR SURVEY – LENNOX HEAD



Legend

GPR PROFILE

SEA WALL EXTENT

Line 34 LINE NUMBER

CLIENT: CIVIL SERVICES GROUP, BALLINA SHIRE COUNCIL			
TITLE: GPR SURVEY TO LOCATE A BURIED SEA WALL LENNOX HEAD, NSW			
DRAWN: J.E	PROJECT MANAGER: J.E	CADFILE: GBGA1579	
SCALE: 1: 4000	DATE: 01 NOV 13	DRG No: GBGA1579–01	
		REV:	A3

GBG Australia Advanced Subsurface Investigations

18 Fennell Street
North Parramatta
NSW 2151
Australia

Telephone:
(02) 98902122
Fax:
(02) 98902922

Email:
info@gbgoz.com.au
Website:
www.gbgoz.com.au

BMT WBM Bangalow

6/20 Byron Street, Bangalow 2479
Tel +61 2 6687 0466 Fax +61 2 66870422
Email bmtwbm@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Brisbane

Level 8, 200 Creek Street, Brisbane 4000
PO Box 203, Spring Hill QLD 4004
Tel +61 7 3831 6744 Fax +61 7 3832 3627
Email bmtwbm@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Denver

8200 S. Akron Street, #B120
Centennial, Denver Colorado 80112 USA
Tel +1 303 792 9814 Fax +1 303 792 9742
Email denver@bmtwbm.com
Web www.bmtwbm.com

BMT WBM London

International House, 1st Floor
St Katharine's Way, London E1W 1AY
Email london@bmtwbm.co.uk
Web www.bmtwbm.com

BMT WBM Mackay

PO Box 4447, Mackay QLD 4740
Tel +61 7 4953 5144 Fax +61 7 4953 5132
Email mackay@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Melbourne

Level 5, 99 King Street, Melbourne 3000
PO Box 604, Collins Street West VIC 8007
Tel +61 3 8620 6100 Fax +61 3 8620 6105
Email melbourne@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Newcastle

126 Belford Street, Broadmeadow 2292
PO Box 266, Broadmeadow NSW 2292
Tel +61 2 4940 8882 Fax +61 2 4940 8887
Email newcastle@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Perth

Level 3, 20 Parkland Road, Osborne, WA 6017
PO Box 1027, Innaloo WA 6918
Tel +61 8 9328 2029 Fax +61 8 9486 7588
Email perth@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Sydney

Level 1, 256-258 Norton Street, Leichhardt 2040
PO Box 194, Leichhardt NSW 2040
Tel +61 2 8987 2900 Fax +61 2 8987 2999
Email sydney@bmtwbm.com.au
Web www.bmtwbm.com.au

BMT WBM Vancouver

Suite 401, 611 Alexander Street
Vancouver British Columbia V6A 1E1 Canada
Tel +1 604 683 5777 Fax +1 604 608 3232
Email vancouver@bmtwbm.com
Web www.bmtwbm.com