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PRELIMINARY INVESTIGATION OF THE
INFLUENCE OF MATAHINA POWER STATION ON
**RIVER BANK STABILITY ALONG THE
RANGITAIKI RIVER**



June 1988

 **WORKS**

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Prepared for
Electricity Corporation of New Zealand Ltd
Hamilton

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SUMMARY

- 1 The flow regime and morphology of the Rangitaiki River have been affected by a number of man-made and natural changes including the diversion and shortening below Thornton, training of the river between stopbanks downstream of Te Teko, alteration of bed gradients by the March 1987 earthquake and modification of flow regime by construction and operation of the Matahina Power Station. This report presents a preliminary assessment of the factors, including the single peak daily operating regime of the Matahina Power Station, which influence the stability of the river banks in the lower reaches of the Rangitaiki River.

In addition to the daily discharge regime the Matahina Power Station may also affect downstream bank stability by bed degradation and/or attenuation of floods but analysis of these influences was beyond the scope of the present investigation.

- 2 The single peak daily operating regime has only characterised about 33% of the time since commissioning of the dam in 1967. However the relative predominance of this regime since 1981 suggests it has been sufficiently dominant in more recent times for any effect on bank stability to be evident.
- 3 The major influence of the present operating regime on river flow is the daily fluctuation in water level, which typically ranges from 0.5-1.2 m. While some increase in velocity occurs over the operating range, available data from the Te Teko gauging site suggests this change is not significant. Thus the effect of the operating regime on bank stability/erosion would appear to be restricted to any weakening of bank strength and related instability associated with the water level fluctuations.
- 4 A field investigation by jet boat in August 1987 identified 29 major sites of present bank instability/erosion downstream of Matahina Dam (excluding instability resulting exclusively from earthquake damage in March 1987). The length of affected bank at these sites ranged from 20-1300m, being most commonly less than 200 m.
- 5 A field assessment of erosion/instability mechanisms revealed that processes related to fluvial entrainment (i.e. river flow erosion) appear to be the primary cause of most of the bank instability problems. Draw-down effects and other processes likely to be aggravated by the single peak daily operating regime were identified as the primary factor at only 1 of the 29 sites - and the rate of bank retreat of that site is relatively slow.
- 6 Field and laboratory investigations of bank sediments indicate that the banks are primarily composed of silty sands and sandy silts - though clayey silts were also observed at some sites. Simple estimates of permeability (obtained using the empirical relationship developed by Hazen) suggested the materials are generally of low permeability. The estimated permeability of the sands ranged from $1 - 10 \times 10^{-6}$ m/s - with some clayey silts less than 3×10^{-8} m/s. The results suggest the banks could be vulnerable to draw-down effects and other weakening processes related to water level fluctuations under some operating regimes.

However further more detailed work would be needed to quantify the operating limits.

- 7 A historical analysis of bank retreat at site 6 was conducted using aerial photographs covering the period from 1945 - 1987. This analysis strongly suggested that significant bank retreat related to fluvial entrainment processes only occurs during major floods (probably $>Q_5$), with little to no erosion occurring in periods without such flows - regardless of the operating regime of the Matahina Power Station. This analysis provides further support for the conclusion from (limited) flow data that the present operating regime does not generally aggravate those bank instability problems primarily related to fluvial entrainment processes.
- 8 Overall, it appears that the present operating regime is not a significant factor in bank instability problems downstream of the dam. However this report represents only a preliminary assessment and further more detailed work would be enquired to confirm this conclusion. Further work would also be required to quantify safe operating limits should any change from the present operating regime be considered in the future.
- 9 A detailed assessment of the twice daily peak discharge power station operating regimes on river bank stability was not possible within this report. Trial of this operation in 1980 ceased after concern was expressed regarding river bank stability. However, when considered in relation to observations on bank instability and erosion associated with the single peak operation it is not clear to what extent the instability problems observed in 1980 were related to the double peak operating mode. A brief comparison of the single and double peak regimes identified a possible mechanism whereby the double peak daily operating mode might be expected to have a greater influence on bank instability. However, it is equally possible that the concerns in 1980 arose largely from a heightened awareness of existing problems - particularly as many of the sites then causing concern have been identified in this study as primarily related to river flow erosion.

The lower reaches of the Rangitaiki River meander across the alluvial Rangitaiki Plains passing through the townships of Te Teko and Edgecumbe, discharging immediately downstream of Thornton into the Bay of Plenty. A number of man-made and natural changes have influenced the flow regime and morphology of the river in this region over the last century. These include the diversion of the river downstream of Thornton, the training of the river between stop banks (with some river straightening) downstream of Te Teko, and the recent change in bed gradients in the vicinity of Edgecumbe due to aerial subsidence resulting from the March 1987 earthquake. However, perhaps the most significant single modification to the river flow regime across the plains has been the construction of Matahina Dam, and power station which was commissioned in 1967, at a site 9 km upstream of Te Teko.

The major effect of the power station has been the daily fluctuations in water level associated with variable power generation requirements. The present operation is characterised by a single peak daily fluctuation. A two peak daily dam discharge, which has economic advantages over other generating regimes, was experimented with briefly in 1980. However, this regime was abandoned after various reports suggested it was initiating and/or aggravating bank instability problems. No detailed investigations were undertaken at that time to define the extent to which the operating regime influenced bank instability.

At a meeting of MWD/Electricorp on the 1 July 1987, the engineering services of the Ministry of Works and Development, Hamilton, were offered to undertake a two phase investigation to better define the geotechnical constraints on power generation.

PHASE I - To review existing data related to the operation of Matahina Power Station and the associated impact on the Rangitaiki River bank stability. The geotechnical constraints on the dam's operation are to be established.

PHASE II - To provide criteria defining the operating river level fluctuations permissible such that river bank instability is not induced.

Approval for the Phase I investigation was received on the 7 July 1987 with the subsequent field work performed on the 27 and 28 August 1987. The scope of the present study is thus primarily concerned with assessing the influence of the operating regime on bank erosion downstream of the dam - particularly to determine whether the regime is likely to have initiated, or be severely aggravating, any of the existing instability problems. The initial content of the investigation, comprised of a site inspection and appraisal of existing power station operation data, was extended (approval received 7 September 1987) to incorporate analysis of aerial photography and laboratory testing of bank materials. All correspondence is presented in Appendix A.

2.0

METHODOLOGY

The influence of the present Matahina Power Station operating regime on bank instability problems along the Rangitaiki River was assessed using the following procedures:

i Analysis of the Flow Regime

A brief comparison of pre and post dam flow regimes was undertaken by examining annual stage-time plots and/or hydrographs for the period 1960-1987. The major operating regimes since commencement of dam operation in 1967 were identified and briefly discussed. The proportion of time that each of these regimes has operated since commissioning of the station was also assessed visually from the flow data. A more detailed analysis of flow regime was beyond the scope of the present study.

ii Field Investigations

A field inspection of bank instability problems between Matahina Power Station and the mouth of the Rangitaiki River was conducted from a jet boat. All sites of instability related to flow regime (as opposed to earthquake damage) were identified and examined. On the basis of these observations a qualitative assessment was made, for each site, of the processes largely responsible for bank instability. (The theoretical framework for this qualitative assessment is discussed in section 4.2). The influence of the present operating regime on these processes, and hence on the bank instability, was then assessed.

iii Laboratory Analysis

During the field inspection a total of seven samples of bank materials were taken for laboratory analysis. The primary purpose of these samples was to estimate the relative permeabilities of different bank forming materials encountered. The samples were texturally analysed and grading curves for each established.

These grading curves were then used in association with existing empirical procedures to estimate permeabilities. One sample was also subjected to a staged consolidated triaxial test. This information enabled a preliminary assessment of the susceptibility of the various bank forming materials to instability related to draw-down mechanisms.

iv Analysis of Aerial Photography

The rate of bank erosion both prior, and subsequent, to dam construction was examined at a site of major instability by comparing historical aerial photographs. This analysis, while limited, was undertaken to provide some quantitative measurements with which to test the qualitative assessment derived from the earlier analyses (ie:- iii above).

3/5 of 58... considering the size of the river.

3.0 FLOW REGIME

3.1 Operating Regimes

Examination of stage-time plots and hydrographs for the period since commissioning of Matahina Power Station in 1967 reveals three broadly distinct flow regimes:

- Single Peak Daily Regime (present regime)

At the time of the field survey Matahina Power Station was a peak load station, with maximum power generation typically occurring at 1800 - 1900 hrs. This operating pattern results in single peak daily fluctuations in water level below the dam. The size of the water level fluctuations generally depends on available water (ie, storage plus inflows). Typically the fluctuations range from 0.5 - 1.2 m at the Te Teko gauging station (see stage time plots for 1982 and 1984 in Appendix B). As the minimum permissible operating output is 22 MW (about 40 m³/s) and the maximum output about 72 MW (approximately 130 m³/s) the maximum potential water level fluctuation associated with power generation is about 1.9 m (determined using rating curve 014 for the Te Teko site - though some adjustment of this curve is necessary following the recent earthquake (Mr R Murray, MWD, Rotorua, pers comm)). However, the size of the fluctuations is to some extent limited by present operating regulations which restrict particularly the maximum rate of draw-down (currently limited to a maximum load decrease of 8 MW per hr, about 14 m³/s per hour). The largest daily fluctuations evident in the stage record, that are attributable to power generation, are all about 1.4 - 1.6 m. At times of minimal storage and low lake inflows the dam generally operates at the minimum permissible level (22 MW - about 40 m³/s) all day and little to no fluctuations in discharge or water level are evident downstream (eg, see hydrograph for late April 1986, Appendix B).

- Two Peak Daily Regime

A two peak daily operating regime was briefly experimented with in 1980. The water level fluctuations associated with this regime were larger than those typical of the current regime - being commonly 1.4 - 1.7 m. (See further discussion in Section 4.3.3). However, this regime was abandoned following reports that it was initiating severe bank instability and operated for a total period of less than 2.5 months (see stage time plot for August - early October 1980 in Appendix B).

- Run of River Flow Regime

Historically the dam has not always been a peak load station and has operated for long periods simply according to river flow (ie, water generally discharged at the dam at much the same rate as inflow to the lake). During such periods there were generally no, or only minor (<0.4 - 0.5 m), daily fluctuations in downstream water level associated with power generation (eg, see stage time plot for 1978 in Appendix B).

3.2 Comparison with Pre-Dam Regime

The major contrast with pre-dam conditions is the daily water level fluctuation associated with the single and two peak daily regimes. The run of river flow regime is essentially identical to the pre dam conditions, the only major difference in flow regime being any attenuation of flood discharges effected by the dam.

An investigation of the extent to which operation of the dam has attenuated flood flows since 1967 was beyond the scope of the present investigation. However, it is known that the reservoir level has been lowered to attenuate some floods occurring since 1967 (see Table IV, Callander and Duder, 1979).

The extent of any flood attenuation depends on the volume of storage made available at the lake prior to arrival of the flood. The original scheme report for the Rangitaiki River flood protection noted that the dam reservoir could reduce flood peaks by 4 - 5,000 ft³/s (113 - 142 m³/s) if lowered adequately before arrival of the flood (BOPCC, 1968, volume 1 p 32) and estimated the dam would reduce the 100 year return period flow from 906 m³/s (entering the reservoir) to 793 m³/s (exiting the dam). A recent flood frequency analysis undertaken using post-dam flows suggests Q100 may now be only about 697 m³/s - but notes this conclusion is only tentative as the data gives a variety of results depending on the statistical distribution assumed to best represent the flood population. This difficulty is possibly due to the relative paucity of significant floods since 1971 (see figure 6.2).

3.3 Frequency of the Various Operating Regimes

Table 3.1 gives an approximate breakdown of the proportion of time that each of the above regimes has operated since commissioning of the dam in 1967. This breakdown is based on a visual examination of the stage-time plots and hydrographs in Appendix B. In constructing the table, extended periods of low flow (where there are generally no water level fluctuations) and floods have been grouped with the run of river regime. Although not strictly correct, this simplification was accepted since the primary purpose of the subdivision was to identify the proportion of time that water level fluctuations due to power station operation have been experienced.

The data indicates that the recent single peak daily operating regime has been of primary significance since 1981. In the preceding 14 years the regime characterised only 14% of the total period (operating for a significant duration only in the years 1967 - 69 and 1972) compared to nearly 70% of the period since mid 1981. Overall, significant daily water level fluctuations associated with operation of the Matahina Power Station have characterised about one third of the period since commissioning of the number one turbine in January 1967.

TABLE 3.1

Proportion of time characterised by each operating regime since commissioning of Matahina Power Station in 1967 (up to April 1987 inclusive)

| Year | Proportion of Time (%) | | |
|---------|------------------------|--------------------|---------------------|
| | Single Peak Regime | Double Peak Regime | 'River Flow' Regime |
| 1967 | 59 | - | 41 |
| 1968 | 42 | - | 58 |
| 1969 | 16 | - | 84 |
| 1970 | 5 | - | 95 |
| 1971 | 5 | - | 95 |
| 1972 | 37 | - | 63 |
| 1973 | 10 | - | 90 |
| 1974 | 3 | - | 97 |
| 1975 | 6 | - | 94 |
| 1976 | 2 | - | 98 |
| 1977 | - | - | 100 ¹ |
| 1978 | 1 | - | 99 |
| 1979 | 3 | - | 97 |
| 1980 | 5 | 19 | 76 |
| 1981 | 37 | - | 63 |
| 1982 | 90 ² | - | 10 ² |
| 1983 | 10 ² | - | 90 ² |
| 1984 | 65 | - | 25 |
| 1985 | 80 ² | - | 20 ² |
| 1986 | 90 | - | 10 |
| 1987 | 70 ^{2,3} | - | 30 ^{2,3} |
| <hr/> | | | |
| Average | 31 | 1 | 68 |

1 Break in record of about 13 days in April.

2 Significant periods of low flows.

3 Available record only covered period to May 1987 at time of analysis.

4.0 FIELD INVESTIGATIONS

4.1 Introduction

In order to evaluate the principle causes of bank instability below Matahina Dam, it was necessary to identify and map all the major sites of bank instability in this reach. Some previous mapping had been undertaken (BOPCC, 1980) but this was not comprehensive and considerable bank protection has also been installed since this earlier exercise. Hence, a jet boat inspection (Dahm, Edwards and Jennings MWD and Roberts BOPCC) was undertaken along the full length of the reach. As well as the mapping, a qualitative assessment was made of the principle mechanisms affecting bank instability at each site. This chapter presents the theoretical framework for the field investigation and a summary and discussion of results. The detailed discussion of each site is presented in Appendix C.

4.2 Theoretical Background

4.2.1 Processes Of Bank Erosion

Thorne (1982) noted that the processes causing river bank retreat can be grouped into two major categories according to whether they are primarily related to :

- fluvial entrainment (erosive removal of sediment by river currents) or to
- subaerial/subaqueous weakening and weathering

The processes of fluvial entrainment are those which cause bank retreat by direct entrainment from the surface of the channel bank or bed. Most commonly these processes effect bank retreat by erosive removal of sediments from the lower bank, undermining overlying material, (e.g. Pizzuto, 1984) and/or scour of the adjacent bed causing an increase in steepness (ie increased bank angle and height) and hence decreased bank stability.

The processes of weakening and weathering are those which give rise to bank instability by reducing the strength of the bank. The most significant of these processes are typically those which operate within the bank to reduce its strength. Such processes include positive pore water pressure, seepage forces, saturation and cycles of wetting and drying. The importance of such processes in causing bank instability has been demonstrated in a number of studies (e.g. Springer et al 1985; Ullrich et al, 1986).

The above categorisation of the processes causing bank retreat is complicated by the fact that, in nature, no single process ever operates entirely alone to produce erosion (Thorne, 1982). For instance, some processes of weakening and weathering can make a bank more susceptible to fluvial entrainment processes. Nonetheless, the classification produces a very useful means of categorising field observations. Hence in the field examination, the major emphasis was placed on identifying whether the bank instability noted was primarily attributable to the processes related to fluvial entrainment or to weakening - rather than attempting a more detailed resolution of the mechanisms.

4.2.2 Evaluation Of The Influence of Matahina Power Station

In order to evaluate the significance of the operation of Matahina Power Station on bank stability, it was necessary to briefly consider the possible effects of the dam on each of the broad categories of erosion processes noted above.

With regard to the processes of fluvial entrainment, the power station regime could conceivably exacerbate channel bank instability by :

- i inducing channel bed degradation and/or
- ii increasing the frequency and/or magnitude of velocities capable of causing erosion

No information is available to gauge the possible extent and influence of bed degradation. However Callander and Duder (1979) noted an average annual entrainment of 181,500 m³ of bedload and 144,500 m³ of suspended load. Hence the reduction in sediment load has been significant and may well have initiated some morphologic change downstream (particularly in the reach just below the dam). Such change would most probably take the form of degradation or channel widening within the relatively short period since the dam was constructed. Determination of the nature and extent of such change was beyond the scope of the present study.

Clearly, the 'river flow' regime (see section 3.1) which was the predominant regime for the period 1969-81 and 1983 would not have exacerbated velocities. Rather, any effect of this regime would be the opposite - through any attenuation of floods effected by reservoir storage. The single and double peak regimes (see section 3.1) could however, exacerbate velocities as a consequence of the increase in frequency of medium flows (100 - 130m³/s associated with the daily generating peaks. Nonetheless, this seems unlikely since the stage-average velocity plot for the Te Teko gauging site (prior to the recent earthquake) indicates little change in average velocity for the flow range of 70 - 130 m³/s. Moreover, the aerial photograph investigations (see chapter 6) suggest fluvial entrainment has not been significantly affected by the increased frequency of such flows - with most entrainment initiated erosion appearing to occur during major floods.

While the fluctuations in water level associated with the single and double daily peak generating regimes may not significantly exacerbate velocity-related erosion they could be expected to exacerbate weakening processes. For instance, these fluctuations could induce saturated loading of the banks, positive pore water pressures, seepage forces, piping erosion (particularly in stratified banks) and cycles of wetting and drying - all of which effects (grouped as draw-down effects in this report) act to decrease soil strength and hence, bank stability. Springer et al (1985) found that such draw-down effects were a very significant cause of bank failures in their comprehensive study of stream banks in the Ohio River system.

Thus, theoretical considerations suggest that those sites most likely to be affected by the operation of the Matahina Station are those where weakening processes play a significant role in bank instability.

Distinguishing the relative role of fluvial entrainment and weakening processes in the field is complex, and hence to some extent subjective, given the complex inter-relationships which occur between these processes in nature. Nonetheless, significant discrimination is possible by noting the nature and location of the instability zones and other local conditions. For instance, in those regions where bank instability occurs largely on the outside of bends, it is highly probable that fluvial entrainment processes are the primary cause of the instability as these regions are characterised by higher near bank velocities. Also secondary helical flow induced in these zones by differential water levels across the channel results in greater bed scour near the base of the bank (particularly during flood conditions - the holes in these zones tend to fill in during low and normal flows).

As Simons and Li (1982) note, field discrimination is also aided by the fact that the fluvial entrainment and weakening processes tend to act on different regions of the bank. Weakening processes associated with draw-down effects tend to act primarily at the water surface. Hence, these forces, where dominant, tend to result in a shelf or berm forming just below water level (figure 4.1b). Simons and Li (1982) note that the effects of erosion of this type are limited to a relatively narrow zone - usually extending no more than 3 - 5 m landward. Hence, along the Rangitaiki River, any bank instability due primarily to draw-down effects should be characterised by a bench at or just below minimum daily water level. Discrimination of such sites was aided by the fact that our field visit took place at a time of near minimum discharge (discharge ranged from about 46-60 m³/s during the field visit).

In contrast to the draw-down effects, the distribution of velocity and shear stress in open channel flow causes the maximum tractive shear stress (and hence fluvial entrainment type erosion) to act on the bank approximately two thirds of the depth below the water surface (Simons and Li, 1982). For this reason, fluvial entrainment related processes tend to act with greatest intensity near the base of the banks (figure 4.1c).

Thus, despite the complex interrelationship of the processes causing bank instability, it is believed that an adequate theoretical framework exists for this study to enable recognition of the primary group of processes (ie fluvial entrainment or draw-down related) responsible for bank instability at each site. Nonetheless, care was also taken to identify sites where processes not presently significant might become important with a different operating regime.

4.3 Results

4.3.1 General

The location of the major sites of instability (other than earthquake damage) noted in the field investigation of 27 and 28 August 1987 are shown in figures 4.2 and 4.3. The principle details of the instability at each of these sites are summarised in Table 4.1. Detailed discussion of each of the sites is presented in Appendix C.

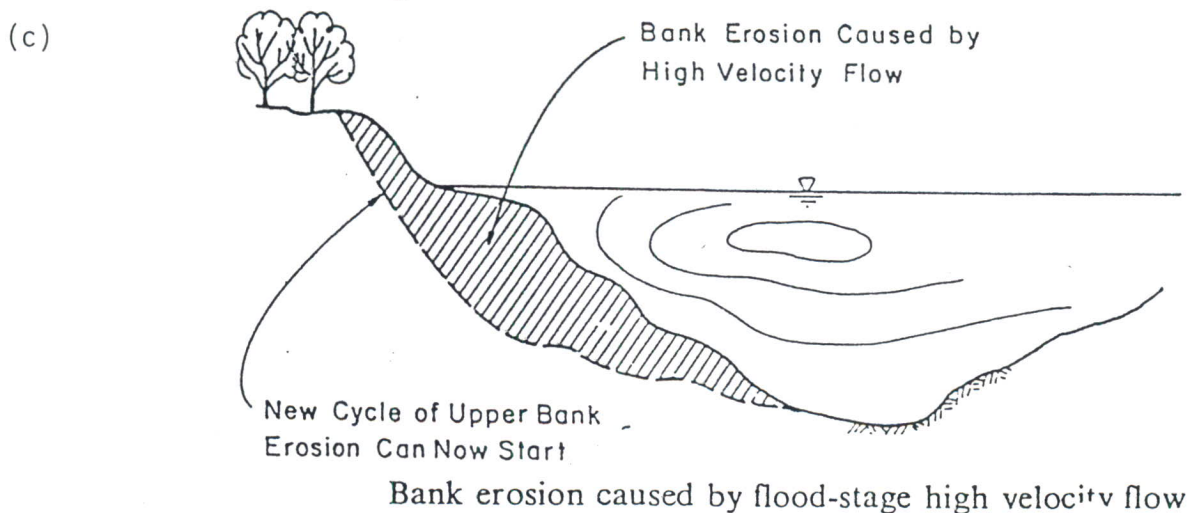
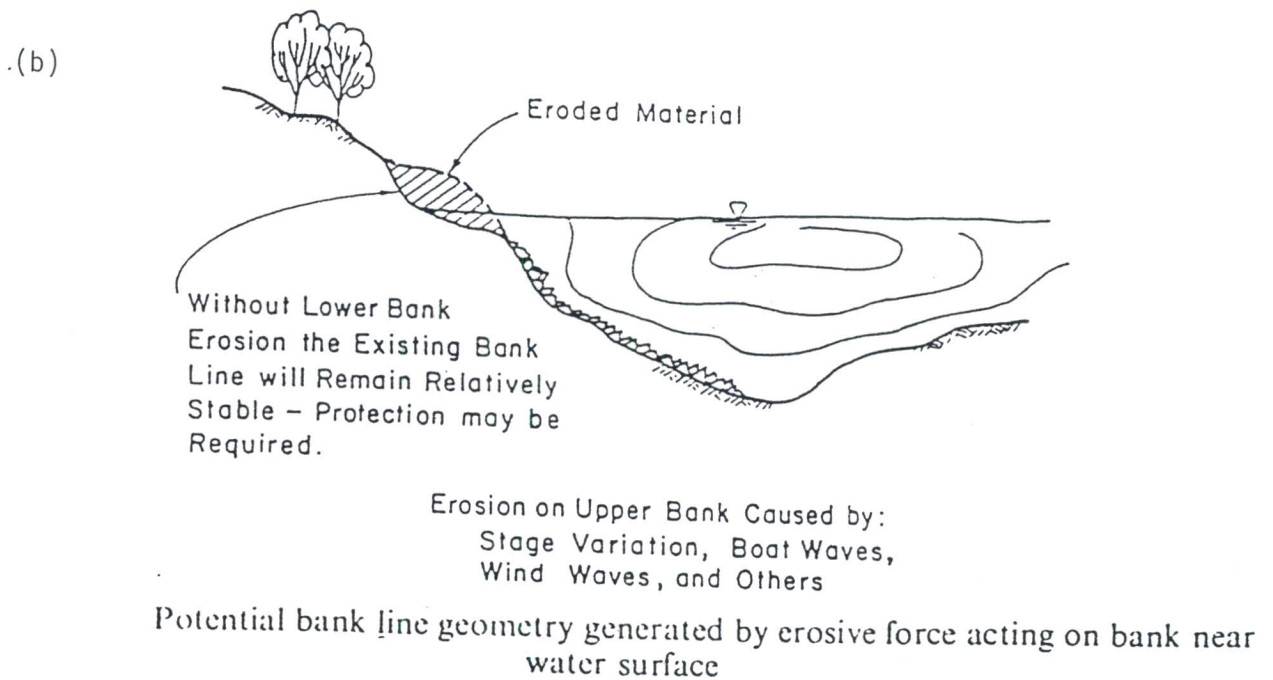
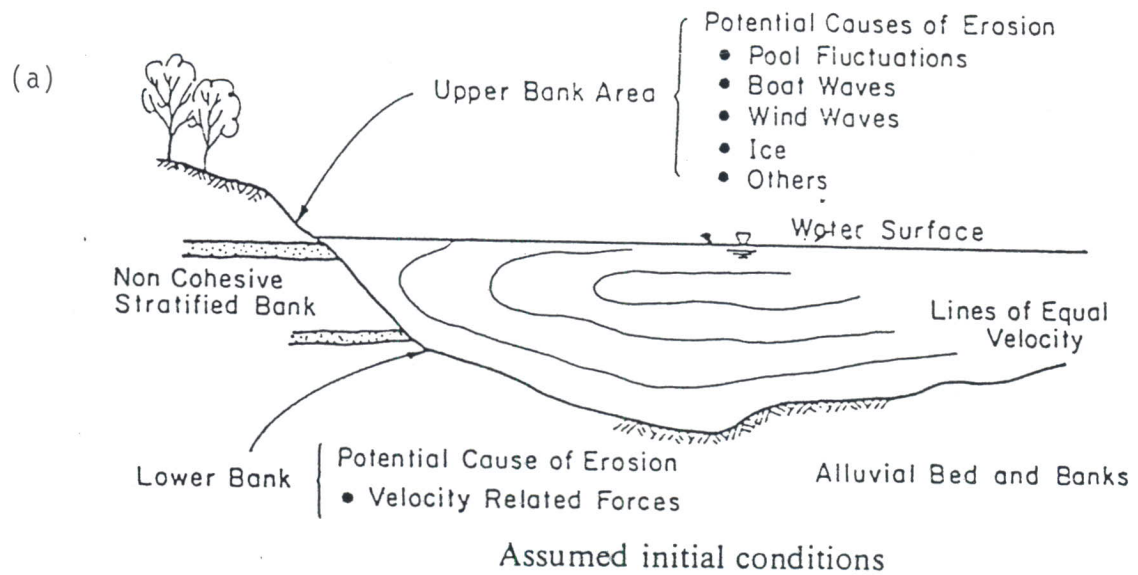


Figure 4.1 Difference in bank profile geometry generated by erosion related to stage variations and wave action from that generated by velocity-related erosion. (Taken from Simmons and Li, 1982).



TE TEKŌ

FIG 4.2



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FIG 4.3

| Site | Distance From Entrance (km -approx) | Length of Affected Bank(m) | Primary Erosion Process | Comment |
|------|-------------------------------------|----------------------------|-------------------------|---|
| 1 | 34.5 | 90 | Fluvial Entrainment | Local tree debris could also be a factor at one location. |
| 2 | 34.1 | 20 | Fluvial Entrainment | Possibly aggravated by February 1987 earthquake. |
| 3 | 33.2 | 150 | Fluvial Entrainment | |
| 4 | 31.8 | 200 | Fluvial Entrainment | Ignimbrite exposed at base of bank limits erosion. |
| 5 | 31.2 | 200 | Fluvial Entrainment | |
| 6 | 30.7 | 800 | Fluvial Entrainment | Seepage forces could occur but are not currently important. |
| 7 | 29.0 | 20 | Fluvial Entrainment | Seepage forces may be a secondary contributing factor. |
| 8 | 28.5 | 100 | Fluvial Entrainment | Draw-down effects could occur but are not currently significant. |
| 9 | 28.2 | (100) | Fluvial Entrainment | Bank erosion currently restricted by willow protection. |
| 10 | 27.8 | 100 | Fluvial Entrainment | Draw-down effects could occur but are not currently significant. |
| 11 | 27 | 250 | Fluvial Entrainment | |
| 12 | 26.8 | 30 | Fluvial Entrainment | |
| 13 | 25.4 | 300 | Fluvial Entrainment | Seepage forces may be a contributing factor, but unlikely. |
| 14 | 25.2 | 100 | Fluvial Entrainment | |
| 15 | 24.9 | 30 | Fluvial Entrainment | Seepage forces active but rate of retreat is controlled by fluvial entrainment. |

Table 4.1 Summary of field investigations.

| Site | Distance From Entrance (Km - approx) | Length of Affected Bank(m) | Primary Erosion Process | Comment |
|------|---|----------------------------------|------------------------------|--|
| 16 | 24.5 | 80 | Fluvial Entrainment | Draw-down related effects may play a minor contributory role. |
| 17 | 23.6 | 200 | Fluvial Entrainment | |
| 18 | 17.1 | 300 | Fluvial Entrainment | Velocities increased by March 1987 earthquake. |
| 19 | 16.5 | 150-200 | Fluvial Entrainment | Undermining of rock protected banks as a consequence of increased velocities since March 1987 earthquake. |
| 20 | 15.7 | 100 | Draw-down related Effects | Limited rate of bank retreat. |
| 21 | 14.5 | 200 | Fluvial Entrainment | Draw-down effects could occur but not currently significant. |
| 22 | 13.8 | Patches | ? | Fluvial entrainment, stock and draw-down effects could all be important |
| 23 | 13.1 | 150 | Fluvial Entrainment | Velocities decreased by recent earthquake at all sites below site 21. |
| 24 | 9.3 | 60 | Fluvial Entrainment | |
| 25 | 8.6 | Patches | Fluvial Entrainment | Draw-down effects may be a factor. |
| 26 | 7.0 | 100 | Fluvial Entrainment | May have been aggravated by recent earthquake. |
| 27 | 4.5 | 1000 m | ? | Minor damage - may be due to draw-down effects, stock or, less likely, to steepening of the bank due to channel bed scour. |
| 28 | 2.4 | 400 m | Fluvial Entrainment | |
| 29 | 1.8 | 1300 m | ? | Wave lap erosion probably the primary mechanism. May also be influenced by current scour and tidally induced draw-down effects. Artificial cut - ie not natural channel. |

Table 4.1 (contd.) Summary of field investigations.

In all, a total of 29 sites of active instability were identified, with the length of affected bank ranging from 20 m to over 1300 m - but most commonly being less than 200 m.

Fluvial entrainment related processes were identified as being by far the most common mechanisms causing bank instability - being the primary factor at at least 22 sites and either the, or one of the, major factors at a further 5 sites (Table 4.1). By contrast, draw-down and other related effects were identified as being the most probable primary cause of instability at only one site - although they might also be a significant factor at a further 5 sites (Table 4.1). Other factors identified as probably being significant at 1 or more sites were wave-lap erosion (probably the major factor at site 29), increased velocities due to the recent earthquake (sites 18 and 19) and tree stumps and/or debris (possible secondary role at sites 1, 18 and 19).

In some cases ground cracking and slumping as a result of the 2nd March 1987 earthquake complicated field observation. It was necessary to consider the age and apparent activity of instability in order to distinguish between earthquake damage and river erosion processes.

4.3.2 Effect of Present Operating Regime of the Matahina Power Station

The dominance of fluvial entrainment related processes (Table 4.1) suggests that the present operating regime at the Matahina Station is not a significant factor in the majority of bank instability problems in the lower Rangitaiki River.

As noted in section 4.2, it is unlikely that the present operating regime significantly affects near bank velocities. Even at those sites where draw-down related processes could be important (ie 3 - 17 % of all sites), it is by no means certain that the draw-down effects associated with power station operation are responsible for the instability. For instance, draw-down during floods (when the banks are likely to be saturated from surface infiltration) could be the primary factor - as could, in the lower reaches, draw-down effects due to tidal fluctuations. Hence, while draw-down effects due to the present operating regime may be a significant factor at up to 3 - 17 % of all the sites of bank instability, it is also equally possible that they are not a significant influence at any of the sites. Further, more detailed field investigations would be required to determine the relative influence of Matahina Power Station, floods, and tidal fluctuations.

4.3.3 Implications for Operational Flexibility

Clearly, even if the present single peak daily regime does not play an important role in bank instability, it does not necessarily follow that a markedly different regime would not have a significant influence. For instance the actions taken at the time of the 1980 double peak daily regime suggest a widespread view that this regime aggravated bank instability.

While a detailed analysis of the double peak regime was beyond the scope of this study the hydrographs in figure 4.4 compare the regime with typical to large fluctuations of the single peak daily regime.

In particular, two significant differences are evident;

- with the two peak daily regime there is a lesser period of low flow between peaks and a higher frequency of high flows (with the peaks often coalescing to give a sustained period of high stage - e.g. 29 and 30/9/80, Figure 4.4)
- the two peak daily regime is typically characterised by larger amplitude fluctuations. This may be largely a function of the present restrictions on the scale of draw-down. However, as the rates of draw-down do not appear markedly dissimilar on those occasions when similar sized peaks are evident (e.g. compare 29/8/80 and 29/8/84 - Figure 4.4) water supply (or other) factors may also be responsible.

If the double peak daily regime did markedly aggravate bank instability then the effect may have been due to the greater sustained duration of high stages and the lesser period between flow peaks - these factors possibly resulting in greater water ingress into the bank than was able to drain between peaks, leading to draw-down related instability.

Unfortunately the field measurements required to determine whether or not such effects did occur were not undertaken at the time of the 1980 regime. Moreover it is worth noting that many of the sites of instability reported at that time (BOPCC, 1980) are identified in this report as being most probably related primarily to erosion by river currents and not significantly influenced by the operating regime at the Matahina Power Station. Thus it is also possible that the double peak regime simply drew attention to what was an already existing problem.

Regardless of the effect of the 1980 regime, the evidence in this report suggests that a more rigorous regime than the present single peak daily operation can probably be operated without markedly aggravating existing instability or initiating erosion at other sites. However as many of the present erosion problems directly influence regions of the stopbank (and the stopbank forms the river bank at a number of other sites) it is imperative that the vulnerability of the banks to any major change in daily regime be properly assessed by an appropriate field programme if any change is ever contemplated. Moreover it should be noted that the recent lowering of bed levels downstream of Kokohinau due to the March 1987 earthquake may increase the vulnerability of some banks in this region to draw-down effects - as water levels now influence higher regions of the bank (generally composed of fine sediments) than formerly. Field investigations to measure important hydraulic parameters of the in situ banks would be the most appropriate means of quantifying the operational constraints. Such measurements could be used to evaluate various optional regimes or to design the most appropriate regime.

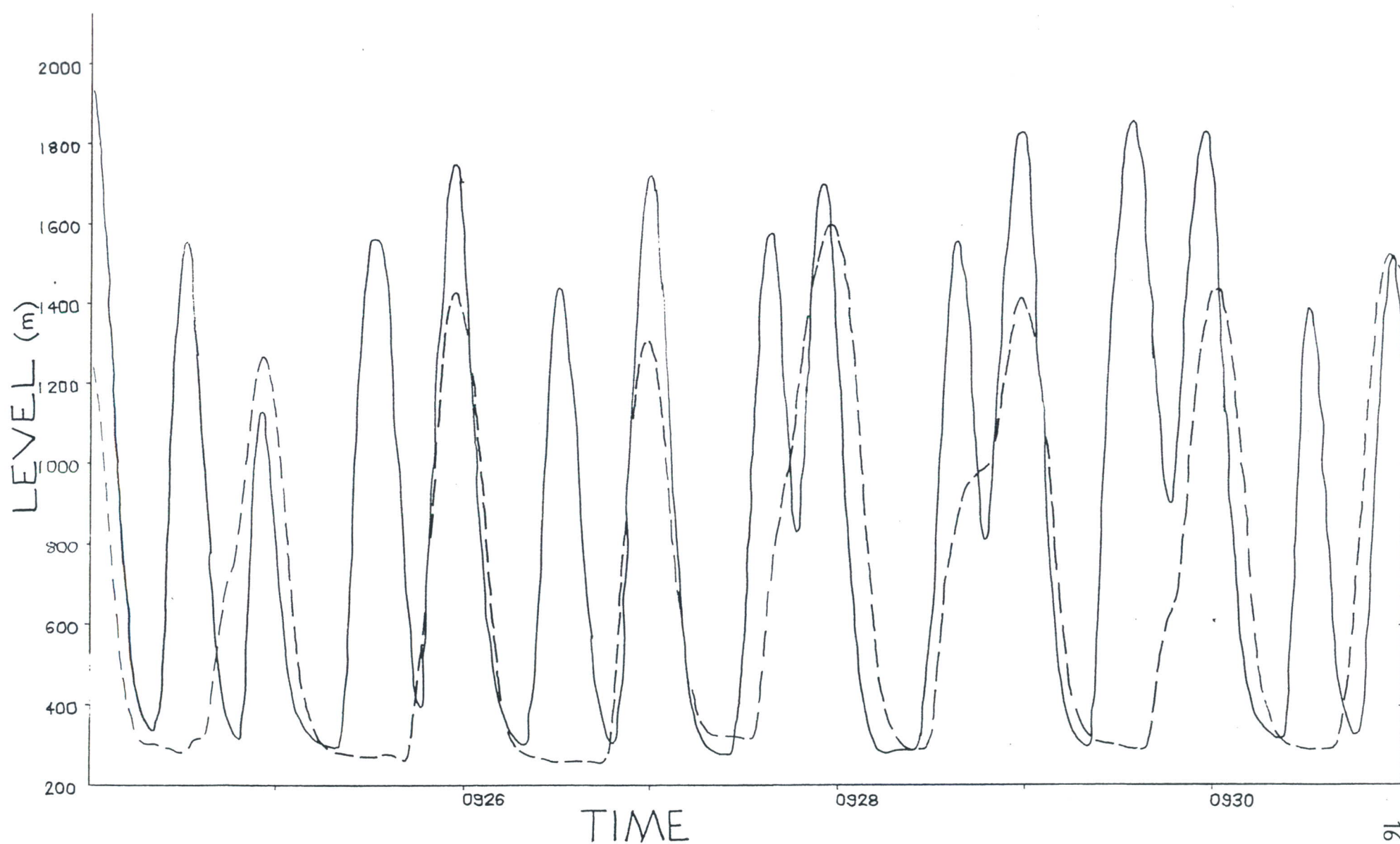


Figure 4.4 : Comparison of double peak daily regime (solid line) with single peak daily operating regime (dashed line). (Double peak data from period 24-30 September 1980 and single peak data from period 24-30 September 1984).

5.0 LABORATORY INVESTIGATIONS

As part of the field investigation work performed on the 27 and 28 August 1987, disturbed sampling was undertaken at sites 10, 16, 20 and 28. All samples were subjected to a particle size analysis with this data used to estimate the insitu permeability of the bank materials. In the case of the sand taken from Site 20, the material was also recompacted and subjected to a staged consolidated triaxial test. Table 5.1 is a summary of the laboratory test results derived for each site. Additionally, grading curves and triaxial test results are presented in Appendix D.

Each of the test sites are discussed separately below with reference made to the test results obtained and their implications :

a Site 10 - 27.8 km Distance

The site is located on the left hand bank of the river and on the outside of a bend.

The presence of readily erodible cohesionless material at the bank base and the clear evidence of undermined slab type failures suggests the instability is primarily related to fluvial entrainment processes (Appendix C). The site was selected as it was considered to be fairly typical of the reaches of the river upstream of Te Teko. Three samples were taken for analysis (Figure C12).

The textural analyses indicate that the bank is composed of interbedded layers of alluvial sands and silts (Table 5.1). Calculated permeabilities indicate that the silts have very low permeabilities with the permeability of the sands, while low, about 30 times higher. With reference to section 3.1, water levels up to 1.6 m above the low river level at the time of the field survey could be expected through power generation. Thus, perching of the watertable could occur in the Sample 2 sand above the silt layer at the base of the bank (figure C12). Hence, it is possible that some power station operating regimes could induce bank instability by draw-down related effects, even though such effects do not appear to significantly influence bank instability with the present operating regime. Further, more detailed investigations would be required to define safety limits.

b Site 16 - 24.5 km Distance

The site is located on the left hand bank of the river and on the outside of the approach to a minor bend. The site stratigraphy consists of alternating cohesive silts and non-cohesive sands. Erosive removal of the sands with consequent undermining of the overlying cohesive silt layers was evident at the site (see Appendix C for detailed site discussion). The site was selected for sampling as the interbedded layers of sands and silts composing the bank suggested that draw-down effects could exert a significant influence on bank stability with some operating regimes.

| Site No. | Sample No. | Height of Sampling Location Above Water Level [m] | PARTICLE SIZE ANALYSIS RESULTS | | | | | | Approx. Permeability [x10 ⁻⁶ m/s] | Permeability Level | C _u [D ₆₀ /D ₁₀] | Degree of Uniformity | Classification |
|----------|------------|---|--------------------------------|------|------|--------|----------------------|--------|--|--------------------|--|--|----------------|
| | | | Composition [%] | | | | D ₁₀ [mm] | | | | | | |
| | | | Clay | Silt | Sand | Gravel | | | | | | | |
| 10 | 2 | 1.0 | 4 | 14 | 82 | 0 | 0.021 | 4.4 | Low | 8.1 | Moderately Uniform | Light Brown Moderately Uniform Fine To Medium SAND With some Silt And Very Rare Clay | |
| | 3 | 1.2 | 4 | 15 | 71 | 10 | 0.012 | 1.4 | Low | 56.7 | Non Uniform Very Well Graded | Very Dark Grey Fine To Coarse Very Well sorted SAND With Some Silt, are Gravel, And Very Rare Clay | |
| | 4 | 2.0 | 8 | 67 | 25 | 0 | 0.0024 | 0.058 | Very Low | 19.2 | Non Uniform Well Graded | Brown Well Graded CLAYEY SILT With Some Organic Content | |
| 16 | 5 | 1.4 | 7 | 78 | 15 | 0 | 0.0050 | 0.25 | Low | 7.8 | Moderately Uniform | Brown Moderately Uniform SILT With Some Sand And are Clay | |
| | 6 | 0.0 | 4 | 18 | 78 | 0 | 0.0185 | 3.4 | Low | 7.6 | Moderately Uniform | Very Dark Greyish Brown Moderately Uniform SAND With Rare Silt And Very Rare Clay | |
| 20 | 7 | 1.2 | 4 | 12 | 84 | 0 | 0.032 | 10. | Medium | 4.5 | Uniform | Light Greyish Brown Pumiceous Uniform Fine To Medium SAND With Rare Silt and Very Rare Clay | |
| 28 | 1 | 0.0 | 21 | 79 | 0 | 0 | <0.0017 | <0.029 | Very Low | >5.3 | - | Greyish Brown Organic CLAYEY SILT | |

* Permeability Determined Using Hazen's Experimental Law.

0 Assessment based on levels of permeability as contained In Lambe and Whitman (1969)

TABLE 5.1
SUMMARY OF RESULTS DERIVED FROM LABORATORY TESTING OF DISTURBED SAMPLES

The particle size analyses confirmed the permeability of the silts to be slightly lower than those of the fine - medium sands (sand permeability estimated to be 13 times that of the silt). Hence, it is possible that draw-down effects (e.g. piping) associated with water level fluctuations could influence bank instability of this site - though any contribution of these processes under the present operating regime is minor (see site discussion in Appendix C).

c Site 20 - 15.7 km Distance

The site is located on the right hand bank of the river and on the inside of a bend. The presence of an extensive shallow bench at the toe of the slope is indicative that basal scour is not a major erosion mechanism (see detailed site discussion Appendix C). The site was chosen for sampling as it was the only observed section of bank erosion where erosion did not appear to be the dominant mechanism, thereby suggesting that the instability could be primarily attributable to draw-down related processes.

As the exposed bank materials appeared very uniform through the full bank depth, only one soil sample was taken. The particle size analysis indicated a uniform fine to medium sand having a predicted permeability higher than any other material sampled (Table 5.1). Recomposition of the sand into a mould was performed to permit triaxial testing. Drained strength parameters of $c' = 0$ kPa and $\phi' = 35^\circ$ were obtained. The recomposed density was deliberately low as the insitu sand density was not known and it was thought desirable to err on the lower side of the actual soil strength. Though a value of $c' = 0$ kPa may have been expected, it is noted that the presence of vertical banks at the site (Figure C31, Appendix C) suggests that some small cohesion component of strength may have been present but destroyed in the sampling process. The presence of an underlying "heavier silt" layer at or below water level, reported in BOPCC (1980), was not observed at the time of the field inspection due to the presence of slump debris. The presence of such material could obviously tend to perch the watertable and thus, promote instability due to draw-down related processes.

To perform a definitive rapid draw-down stability analysis, the nature of the slope would need to be known precisely; stratigraphy, permeability, piezometric surface, soil strength parameters cross sectional shape, being some of the parameters. As the aforementioned parameters are not all known in this case, a computer based analysis was not thought appropriate. The draw-down stability charts of Morgenstern's were used for a rough appraisal assuming a small effective cohesion. The results indicated a bank at incipient failure under significant draw-down conditions. Though Thorne (1982) would suggest that such an application of Morgenstern's charts is of limited value, the preliminary indications are considered useful.

As the bank material is non cohesive, shallow slip surfaces resembling slab type failures are expected, consistent with the fact that no deep seated rotational failure mechanisms were observed on site (extensive tension cracking observed at the top of the bank is thought to be due to earthquake damage). Thus, it is considered that instability problems at the site may well be principally caused by water level fluctuations - due either to power station operation and/or floods.

However, the presence of a debris slope extending more than halfway up the bank with some vegetative cover suggests the site is of low activity - the debris tending to stabilise the toe of the bank.

d **Site 28 - 2.4 km Distance**

The site is located on the left hand bank of the river and on the outside of a curve. The reach is tidal with significant tidal fluctuations (order of 1.0 m). The bank materials are generally fine grained, with a particularly low permeability clayey silt at water level (Appendix C).

The particle size analysis results confirm the field impression that the clayey silt material has a low permeability - this material having the lowest predicted permeability of all the samples tested. The permeability is expected to be considerably less than that of the overlying silty sands which probably have a permeability similar to that of the other sands in Table 5.1 (ie $1.4 - 4.4 \times 10^{-6}$ cm/s). Though the depth profile adjacent to the bank would suggest active basal erosion, it is possible that perching of the watertable above the clayey silt due to water level fluctuations could also influence bank stability. The finer grained bank materials in the lower bank profile are consistent with the lower energy regime and may have been deposited as a consequence of salinity - induced flocculation.

6.1 Introduction

In order to provide some initial measurements to supplement the field investigations, a limited aerial photograph analysis was undertaken to examine the influence of the dam on the rate of bank retreat. Initially, it was hoped to compare the rate of bank retreat at 2-3 sites over 15-20 year periods prior and subsequent to commissioning of the dam in 1967. However, for various reasons, this proved to be beyond the scope of the present investigation. The most important complication was the very high frequency of extreme flood events in the 14 year period between 1957 and 1972. This 14 year period included 3 flows in excess of Q_{30} (including one flow estimated at Q_{70}) and 6 flows greater than or equal to $Q_7 - Q_8$ [By way of contrast, the subsequent 16 years to the time of this report contained only one significant event, the 1983 flood - estimated to have a 5 year return period].

Thus, it was necessary to subdivide both the pre and post-dam regimes to separate this period with a high frequency of flood flows from both prior and subsequent periods. This necessitated a minimum of 5 aerial photographs per site investigated - which effectively limited the present investigation to one site.

Site 6 (See figure 4.2) was chosen for the analysis. The primary reason for the choice of this site is the evidence that bank retreat is primarily related to fluvial entrainment rather than draw-down related processes. (The 8-10 m height of the banks suggests they are unlikely to be significantly affected by loading effects associated with fluctuating water levels, although there is evidence that seepage effects could occur - see site discussion in Appendix 3). Given the apparent primary importance of erosion in bank instability along the river (see discussion in Section 4.3 and Appendix 3), it was considered that the effect of the dam on erosion was the most appropriate emphasis for this preliminary analysis.

The large scale of the bank retreat and the absence of bank protection measures were also further reasons for choice of the site. Sites with lower banks further downstream were often complicated by relatively small scale bank retreat, the presence of willows (complicating accurate positioning of the shoreline) and rock protection (date of placement unsure at some sites - though it is probable that this could be determined from BOPCC records if required). Although these complications could probably be overcome in a more detailed investigation, they were beyond the scope of the present study.

6.2 Data

In choosing aerial photographs to evaluate erosion at the site, it was desired to :

- subdivide the pre and post-dam regimes. This was obtained using a photograph 5231/35 from survey 1906 (N.Z. Aerial Mapping Ltd) flown in 15.10.66 - barely 2 months prior to commissioning of the first turbine.

- obtain at least 20 years record both prior and subsequent to commissioning of the station. Photo 684/33 of survey 256 (NZ Aerial Mapping Ltd) flown on 19.03.45, provided a record of nearly 22 years bank retreat prior to commissioning. Photo Q15 of survey 8732 (NZ Aerial Mapping Ltd) flown 13.03.87 provided just over 20 years record since commissioning of the station.
- distinguish the period 1957-67 within the pre-dam regime and the period 1967-71 within the post-dam regime (the period 1957-71 having a high frequency of large flood events as noted above). This subdivision was undertaken so that the relative impact of high flows could be assessed for both pre and post-dam regimes.

The post-dam regime was subdivided using photo 4654/30 from survey 3580 (NZ Aerial Mapping Ltd) flown on 22.09.72. Photo 2010/8 from survey 596 (NZ Aerial Mapping Ltd) flown on 17.10.52 was used to subdivide the pre-dam regime. Though a photo taken closer to 1957 would have been more preferable, this was not available.

Copies of each photo were obtained at a common scale of 1:2000. Changes were compared by plotting the bank positions from each photograph on a plastic overlay. The bank edge was well defined in all photographs and fences, houses etc, provided good positional control for the overlay. A minor difficulty was experienced with the surveys of 1952 and 1987 which were at a slightly different (3-5%) scale to the other photos. However, scaling from known features ensured accurate plotting of the bank edge from these prints.

6.3 Discussion of Results

The pattern of erosion along the (right) bank at site 6 between 1945 and 1987 is shown on Figure 6.1.

It is notable that a marked change in the pattern of bank retreat is evident in the period 1972-87 - the extent of erosion being markedly more restricted than observed in the preceding periods. (Figure 6.1). As the pattern of erosion between 1966 and 1972, predominantly a period of post-dam flow regime, was virtually identical to that occurring in the previous (pre-dam regime) periods, it seems unlikely that the change in pattern observed between 1972 and 1987 can be primarily attributed to the operation of the dam. More probably, the change reflects the relatively low frequency of large flows during this period (Figure 6.2).

The rate of erosion shows a similar pattern with a marked reduction in the rate of bank retreat being evident between 1972 and 1987 (Figure 6.2). The rate of erosion in the post-dam period of 1966-72 ($1360 \text{ m}^2/\text{yr}$) is within the range ($889\text{--}2171 \text{ m}^2/\text{yr}$) observed in the preceding periods - and is, in fact, virtually identical to the average rate ($1316 \text{ m}^2/\text{yr}$) observed over the total 1945-66 period of pre-dam regime. Thus, the rate of erosion appears to be primarily related to the occurrence of large flows rather than the presence or absence of the dam.

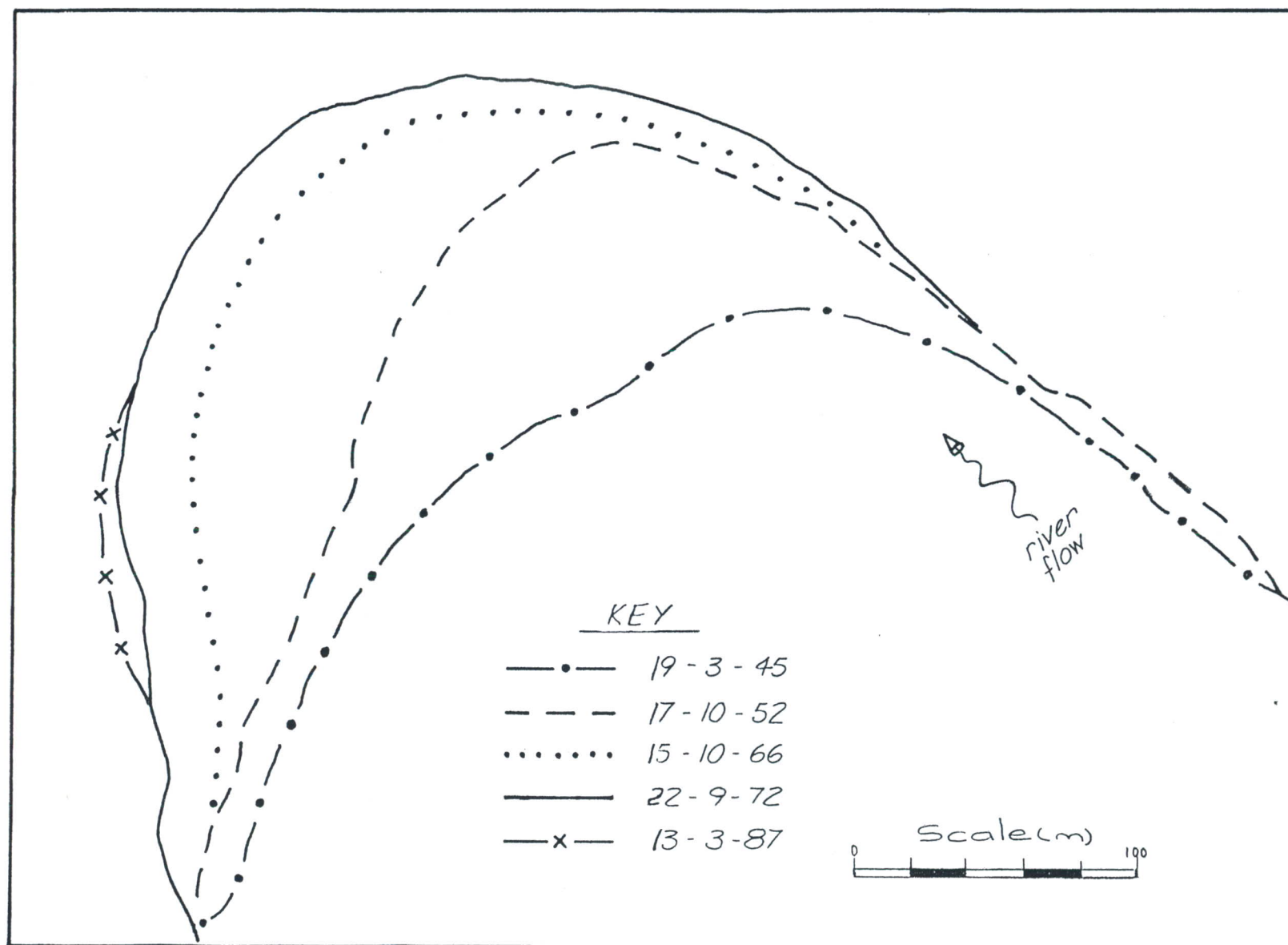


Figure 6.1 : Pattern of bank retreat of site 6, 1945-1987

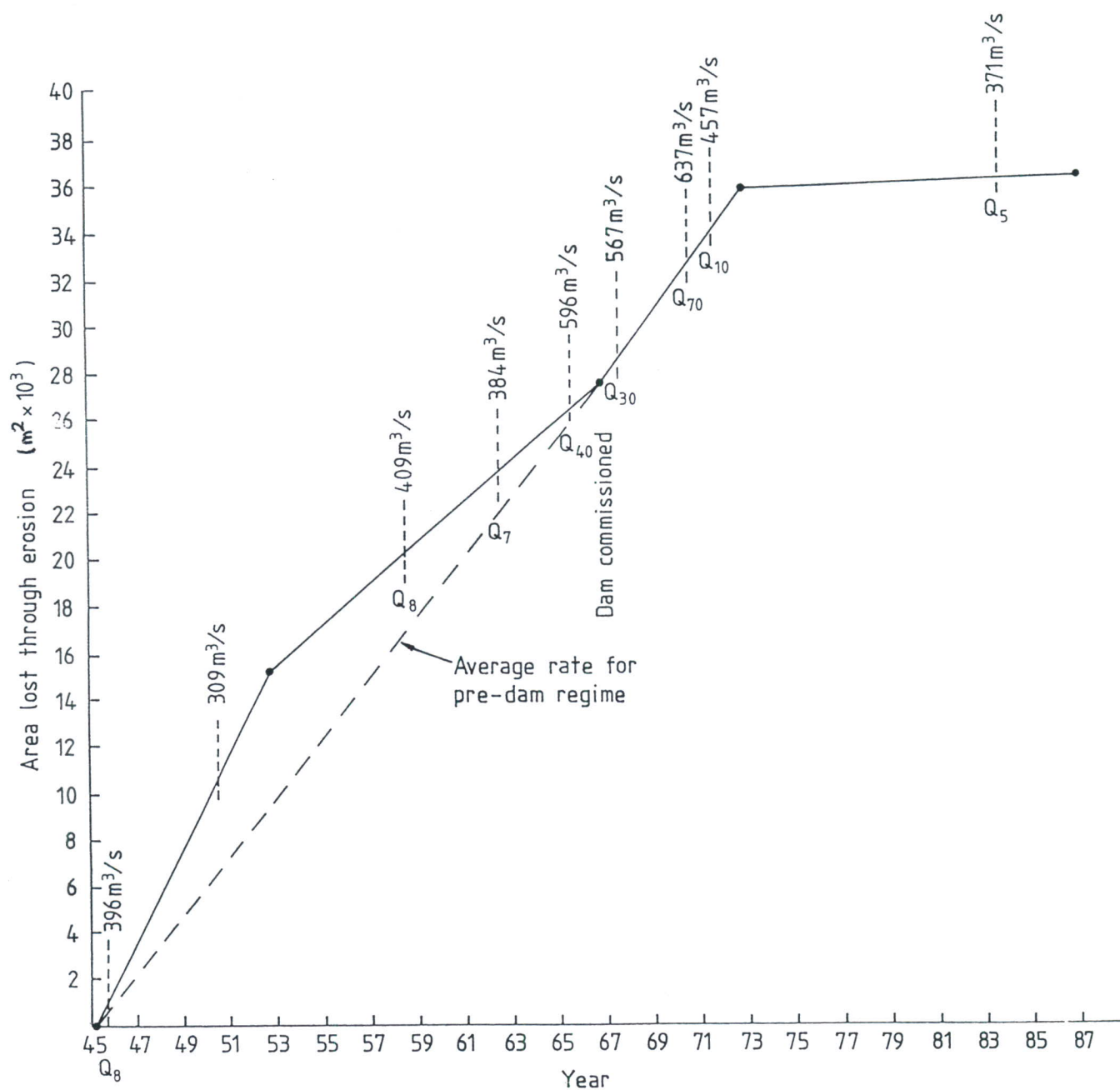


Figure 6.2 : Rate of retreat along right bank at site 6 during period 1945-87

An apparent anomaly in this explanation is the rapid rate of bank retreat ($2171 \text{ m}^2/\text{yr}$) noted between 1945 and 1952 - a period with only two significant floods, only one of which was in excess of Q_5 (Figure 6.2).

However, a major flood occurred in 1944 immediately preceding this period - the peak flow being estimated at $27700 \text{ ft}^3/\text{s}$ (about $699 \text{ m}^3/\text{s}$) in Table 4-VIII, Appendix 4-1 of BOPCC (1968). This peak flow is close to the 100 year return period discharge estimated in a recent interim report (BOPCC, 1987) - though that report noted that further work is required before the estimated design flows could be confirmed. It is probable that this peak flow in 1944 would have resulted in widespread scour of the banks and removal of protective vegetation, leaving the banks more vulnerable to erosion by subsequent flows - particularly the flood which occurred in the immediately following year (Figure 6.2). The 1972 photo, taken 2 years after a similar major (about Q_{70}) flow, provides considerable evidence that major changes occur during such peak flows. The significant changes indicated in the vicinity of the site include development of a mid channel shoal (incipient point bar) just upstream of the site, significant erosion of the upstream left hand bank (only minor erosion had occurred in this region between 1945 and 1966), a marked change in the shape of the point bar at the site and a large increase in the rate of erosion (about 50 percent higher) over the immediately preceding period (Figure 6.2). By contrast, the 1982 photo taken after a prolonged period without major floods, shows considerable vegetation cover developed around the base of much of the bank (not noted in any of the other photos) and on shoals in the channel.

Thus, the evidence from the aerial photographic analysis suggests that erosion at the site is primarily related to the frequency and size of major (probably $>Q_5$) floods. The relatively minimal erosion noted between 1972 and 1983, together with the development of stabilising vegetation around much of the base of the bank during this period, suggests that the normal power station operation has little, if any, detrimental effect. In fact, as the dam is operated to reduce the size of some peak flows, it is more probable that any overall effect of the dam at this site has been to reduce the rate of bank retreat. However, the relatively high rate of erosion noted between 1966 and 1972 does suggest that any such positive effect is also minimal.

ACKNOWLEDGEMENTS

The assistance of the Bay of Plenty Catchment Commission, Electricorp and the Rotorua Water and Soil Survey Unit (DSIR) in the provision of data used in this study is acknowledged. Special thanks are due to Mr Don Roberts of the Bay of Plenty Catchment Commission who accompanied us on our field inspection of the river and provided much useful background information on the instability problems. Mr Stan Lodge of the Rotorua Water and Soil Survey is also due special mention for his skilled jet boat navigation of the river, including the difficult Kokohinau section, which enabled a full inspection of the lower river to be conducted.

REFERENCES

- Bay of Plenty Catchment Commission 1968. "Rangitaiki River Major Scheme, Volumes 1-3."
- Bay of Plenty Catchment Commission 1980. Untitled internal report on major erosion problems in Lower Rangitaiki River - compiled by L.T. Craig, Hydrologist.
- Bay of Plenty Catchment Commission and Regional Water Board 1987. "Post Earthquake Flood Evaluation of the Lower Rangitaiki River" - Interim Report.
- Callender, R A and Duder, J N 1979 "Reservoir Sedimentation in the Rangitaiki River". New Zealand Engineering Volume 34(9), 15 September, pp 208 - 215.
- Lambe, T W and Whitman, R V. "Soil Mechanics". John Wiley and Sons, New York, 1969.
- Pizzuto, J E 1984. "Bank Erodibility of Shallow Sandbed Streams" Earth Surface processes and Landforms, Volume 9, pp 113-124.
- Simons, D.B. and Li, R M 1982. "River Erosion on Regulated Rivers." In Gravel Bed Rivers (edited by R.D. Hey, J.C. Bathurst and C.R. Thorne) John Wiley & Sons, pp 717 - 747.
- Springer, F M, Ullrich, C R, and Hagerty, D J 1985. "Streambank Stability". Journal of the Geotechnical Engineering Division, ASCE, Volume 111, pp 624-640.
- Thorne, C R 1982 "Processes and Mechanisms of River Bank Erosion" In Gravel Bed Rivers (edited by R D Hey, J C Bathurst and C R Thorne). John Wiley & Sons, pp 227 - 259.
- Ullrich, C R, Hagerty, D J and Holmberg, R W 1986. "Surficial Failures of Alluvial Stream Banks". Canadian Geotechnical Journal Volume 23, pp 304-316.

APPENDIX A

CORRESPONDENCE



Ministry of Works and Development

District Office

Dey Street,

Private Bag, Hamilton

Telephone 62 899

Telex NZ 2777

Inquiries to Mr D N Jennings

Date 10 July 1987

Ref 96/154000

Your Ref

Regional Manager
Electricorp
P O Box 445
HAMILTON

ATTENTION : Mr D Pattenden

Dear Sir

MATAHINA POWER STATION OPERATION RANGITAIKI RIVER BANK STABILITY APPRAISAL

Further to our meeting of 1 July I have given this subject further consideration as requested. I now present below a two phase proposal:

Phase I:

Objective

To review existing data related to the operation of Matahina Power Station and the associated impact on the stability of the Rangitaiki River, particularly with respect to the twice daily peak flow in 1980 and subsequent single daily peak operation, and establish the geotechnical constraints on Matahina operations.

Discussion

The twice daily peak operating mode in 1980 resulted in increased erosion and instability of the Rangitaiki River banks associated with rapid drawdown effects. Because of this the operating mode was modified to a single daily peak discharge. From the performance of the river banks under these different operating modes some interpretation of whether or not more severe operational level fluctuations can be tolerated can be made. A site inspection would be necessary followed by an office study of data and bank stability. Discussions with BOPCC and MWD Rotorua staff would be held in conjunction with the site inspection to identify any operational suggestions and any concerns they may have. A report outlining possible operations, with assessed acceptable drawdown rates for the river, would be produced. This report would postulate acceptable drawdown rates and propose a test programme (phase II) to confirm its recommendations.

| | | |
|---------------|---|-------------------|
| Estimate | Data collection 3 m days @ \$400 | = \$ 1,200 |
| | Site inspection 2 m x 2 days @ \$100 | = 2,000 |
| | Office study & report 10 m days @ \$400 | = 4,000 |
| | 3 m days @ \$600 | = 1,800 |
| | Disbursements (LMV, computer hire, etc) | = 1,000 |
| | Contingencies | = 1,000 |
| Total Phase I | | <u>= \$11,000</u> |

Personnel The phase I study would be carried out by Mr D N Jennings with assistance from Mr M R Edwards.

Phase II:

Objective To provide criteria defining the operating river level fluctuations permissible such that river bank instability is not induced.

Discussion This phase would follow an economic assessment by Electricorp of the merits of a more severe operating regime (following on from phase I). An investigation of soil conditions in areas of previous instability (suggest 3 sites) would be undertaken to establish strength and permeability parameters. Monitoring of groundwater profiles would be undertaken. Some surveying of bank profiles may be required. Following data collection an indepth groundwater and stability study would be undertaken. A report presenting the findings would be produced. A more specific estimate for this work would be prepared following phase I.

| | | |
|---------------------------|---|-----------------|
| Estimate (Preliminary) | Site investigations (drilling & sampling, piezometers & monitoring, logging of materials) | = \$15,000 |
| | Laboratory testing (triaxials & permeability tests) | = 5,000 |
| | Office study, analysis and reporting | = 8,000 |
| | Contingency | = 2,000 |
| | Total Phase II | <u>\$30,000</u> |

Personnel The phase II study would be carried out under the direction of Mr D N Jennings with various MWD laboratory and technical service inputs as required.

I appreciate that this proposal is rather brief in content but it does serve as a basis for your consideration of the economic merits of such a

study. I would be happy to provide you with further details on request.

Yours faithfully

A handwritten signature in dark ink, appearing to read 'D. N. Jennings', with a long, sweeping horizontal stroke extending to the right.

David N. Jennings
District Materials & Investigation Engineer
for District Commissioner of Works



electricorp

ELECTRICITY CORPORATION OF NEW ZEALAND LIMITED

File 2/100

17 July 1987

District Commissioner of Works
Ministry of Works & Development
Private Bag
HAMILTON

Attention : Mr D Jennings



Dear Sir

MATAHINA POWER STATION
RANGITAIKI RIVER BANK STABILITY APPRAISAL

Thank you for the proposals outlined in your letter dated 10 July 1987. We have re-appraised our position regarding the peak operation of Matahina Power Station and it is unlikely that we will proceed with evaluation of two peak daily operation.

However we consider that the phase I section of the investigation you propose will give a good evaluation of our current operating practice and the effects on the river downstream.

Would you therefore proceed with this part of the evaluation and forward a report on your findings in due course.

Yours faithfully

ELECTRICITY CORPORATION OF NZ LTD


D R Pattenden

GROUP ENGINEERING MANAGER

DRP:CML



Ministry of Works and Development

District Office

Dev Street,

Private Bag, Hamilton

Telephone 62 899

Telex NZ 2777

Inquiries to Mr D N Jennings

Date 4 September 1987

Ref 96/154000

Your Ref 2/100

Regional Manager
Electricity Corporation
P O Box 445
HAMILTON

ATTENTION : Mr D Pattenden

Dear Sir

MATAHINA POWER STATION OPERATION RANGITAIKI RIVER BANK STABILITY APPRAISAL

Following your approval to proceed with the Phase I investigations (your letter of 17 July 1987) we have completed the field inspection/survey of riverbank stability over the full length of the Rangitaiki River downstream of the Matahina Power Station. This survey proved more time consuming and extensive than originally anticipated but it has provided much useful information on the existing features of the river.

This inspection has found that most of the existing erosion/instability features are on the outside of bends which indicates that basal scour is the primary mechanism of instability (rather than initiation through power station operations). Also recent earthquake instability was observed and in the lower reaches tidal fluctuations appear to be contributing to river bank instability. While it appears unlikely that the power station operation has initiated individual instability features the bank characteristics at many sites are such that regular water level fluctuations would aggravate bank erosion. It is considered that this aggravation would be through a complex interaction of soil piping and in some cases a component of rapid drawdown. Observations in the upper reaches suggest that rapid drawdown is not a significant feature.

As phase 2 is unlikely to proceed, we believe it would be useful to extend the scope of Phase 1 as follows to provide a more comprehensive assessment of the impact of existing operations:

- Aerial photograph analysis to examine the rates of erosion, at 3 typical sites, before and after dam construction. This analysis will establish whether or not power station operation has significantly aggravated bank erosion and provide an estimate of the degree of impact.
- Laboratory testing of bank materials, particularly to establish permeability parameters. This will enable a more rational assessment of the effect of the existing regime as well as providing better data with which to assess the impact of the 1980 regime.

These extra investigations would increase the cost of the Phase 1 investigations by \$3500 to a total of \$14,500. Would you please advise whether or not you would like the scope of the study enhanced as suggested above. I would be happy to discuss this proposition further with you if required.

Yours faithfully

A handwritten signature in dark ink, appearing to be 'DNJ' or similar, written over the typed name.

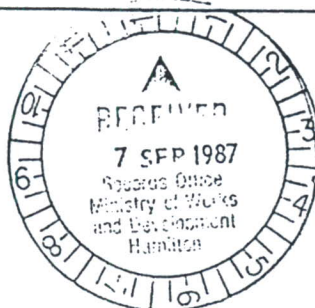
D N Jennings
for District Commissioner of Works



electricorp

ELECTRICITY CORPORATION OF NEW ZEALAND LIMITED

96/154000



File: 2/100

4th September 1987

District Commissioner of Works
Ministry of Works and Development
Private Bag
HAMILTON

Attention: Mr. Jennings

Dear Sir,

Matahina Operation - Rangataiki River Bank Stability Appraisal

Thank you for your letter reference 96/154000 of 4.9.87 giving some progress information on the study of erosion in the Rangataiki River. As suggested by you I agree that the additional investigation related to Aerial photographs and soil testing of bank materials should be carried out.

You are therefore authorised to spend the additional \$3,500 on this project bringing the total cost to \$14,500.

Yours faithfully

ELECTRICITY CORPORATION OF NZ LTD

D R Pattenden
D R Pattenden
for Hydro Group Manger

M Edwards *M.R.E.*

(P)

DRP:NGL/0143M

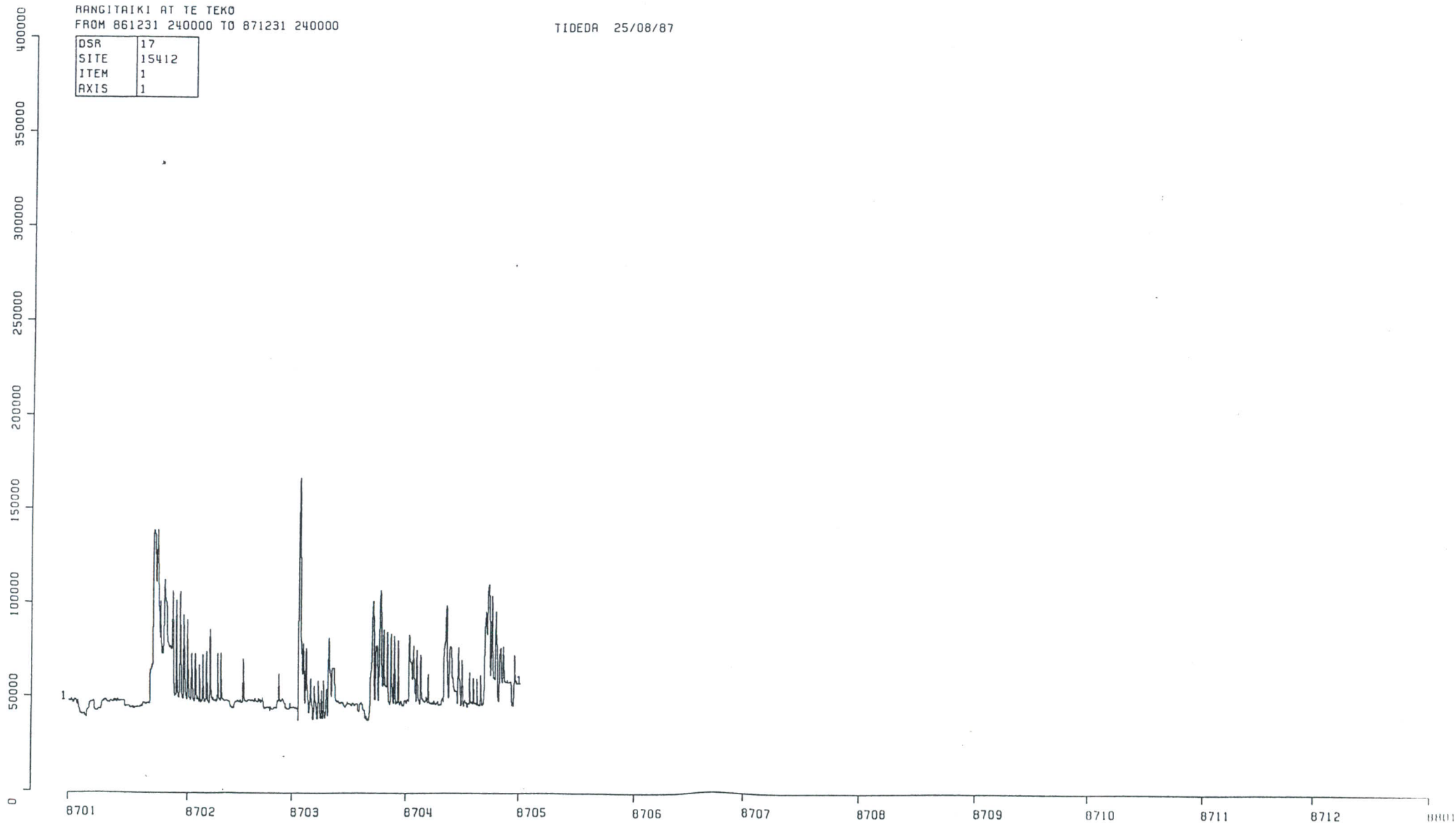
APPENDIX B

STAGE-TIME PLOTS FOR THE RANGITAIKI
RIVER AT TE TEKŌ : 1966-87

RANGITAIKI AT TE TEKO
FROM 861231 240000 TO 871231 240000

TIDEDR 25/08/87

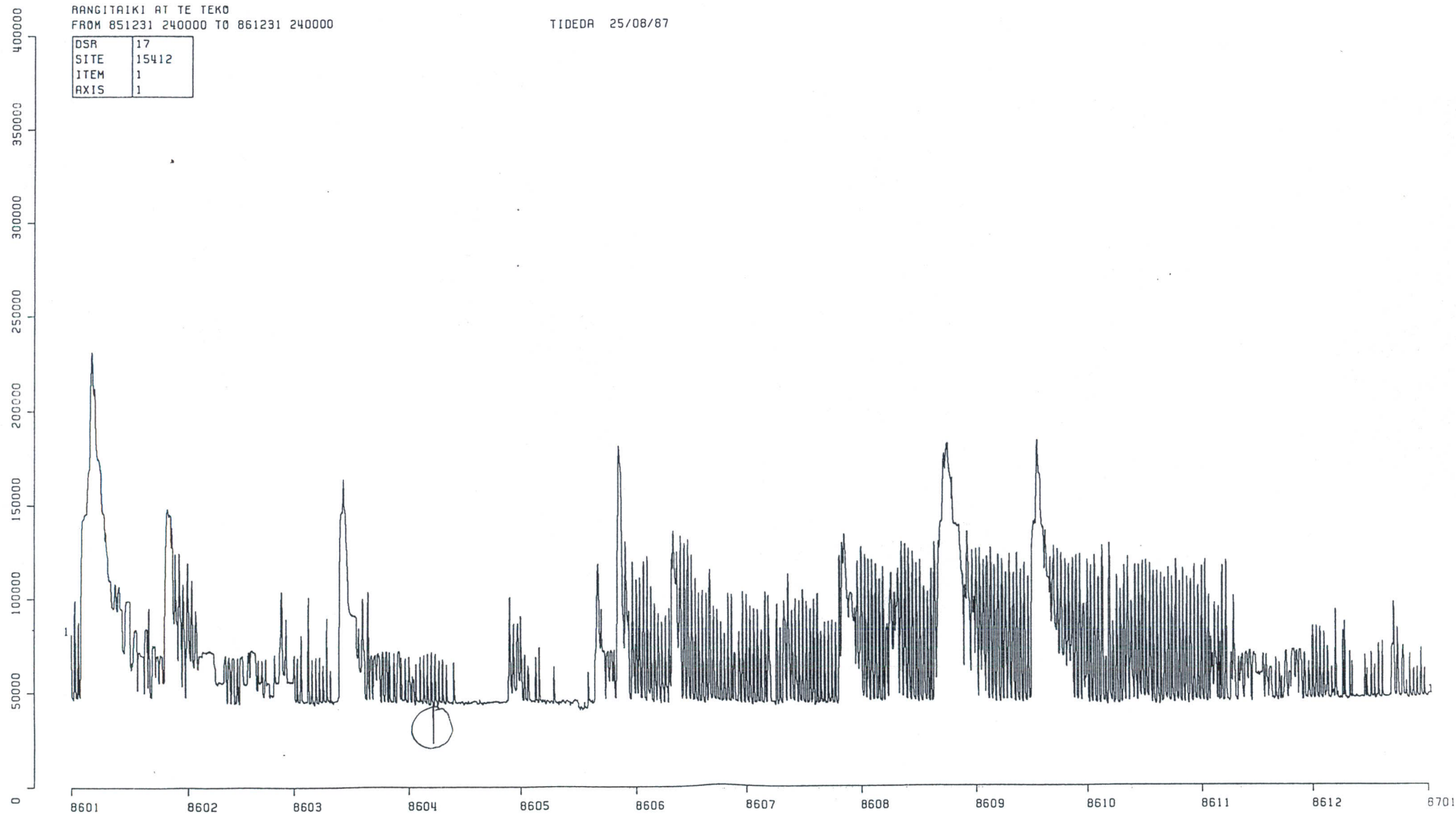
| | |
|------|-------|
| DSR | 17 |
| SITE | 15412 |
| ITEM | 1 |
| AXIS | 1 |



RANGITAIKI AT TE TEKŌ
FROM 851231 240000 TO 861231 240000

TIDE0A 25/08/87

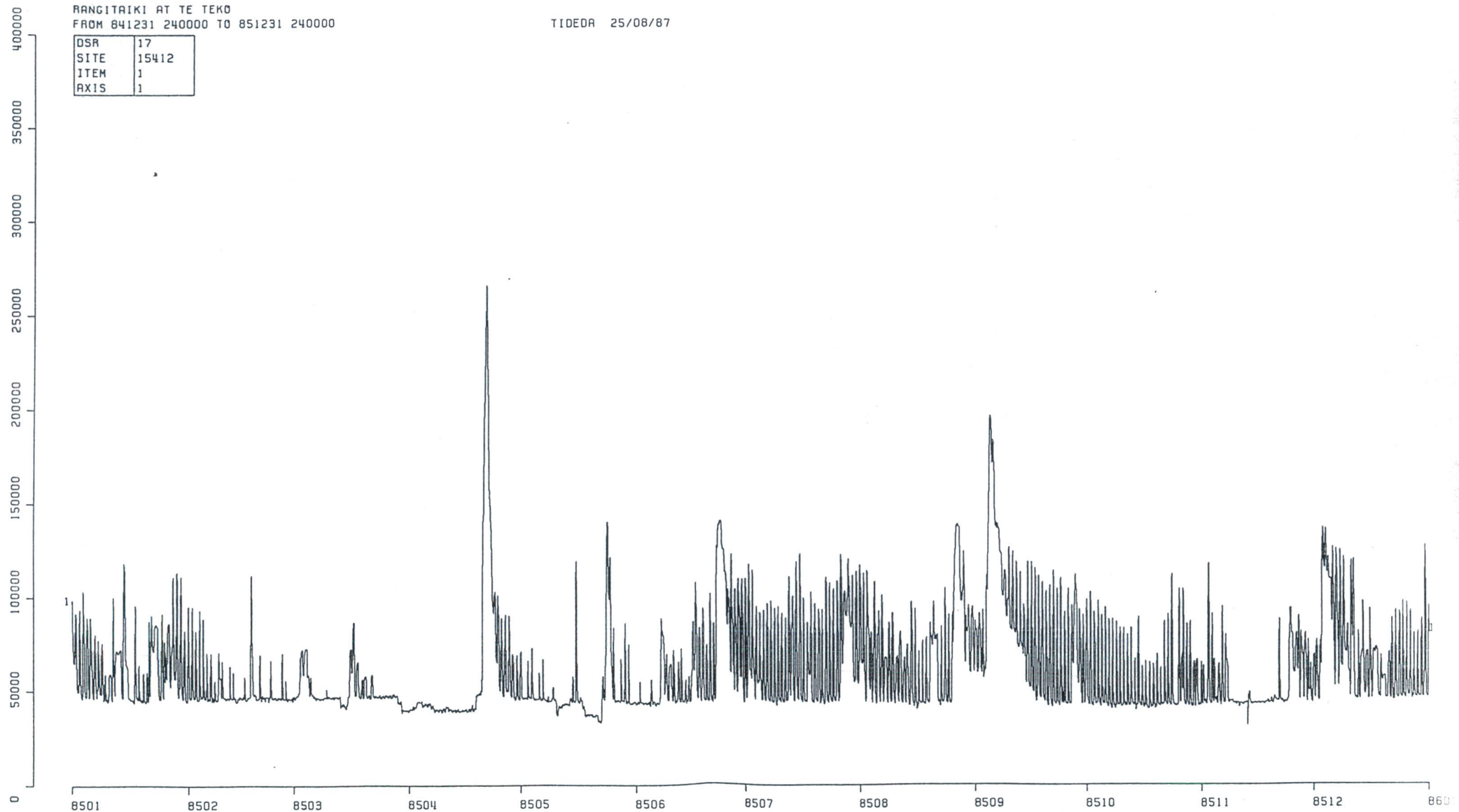
| | |
|------|-------|
| DSR | 17 |
| SITE | 15412 |
| ITEM | 1 |
| AXIS | 1 |

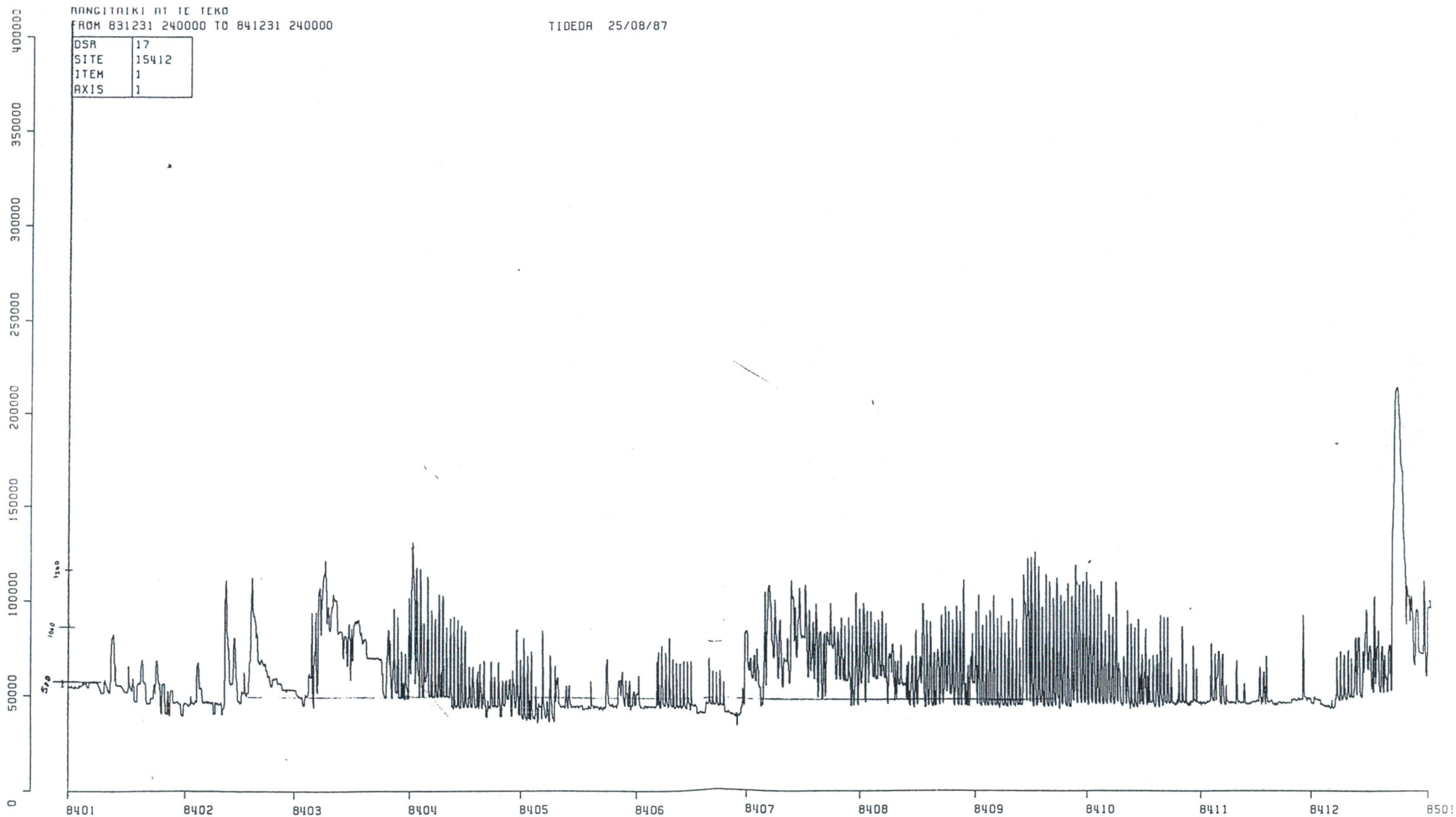


RANGITAIKI AT TE TEKO
FROM 841231 240000 TO 851231 240000

TIDEDR 25/08/87

| | |
|------|-------|
| DSR | 17 |
| SITE | 15412 |
| ITEM | 1 |
| AXIS | 1 |

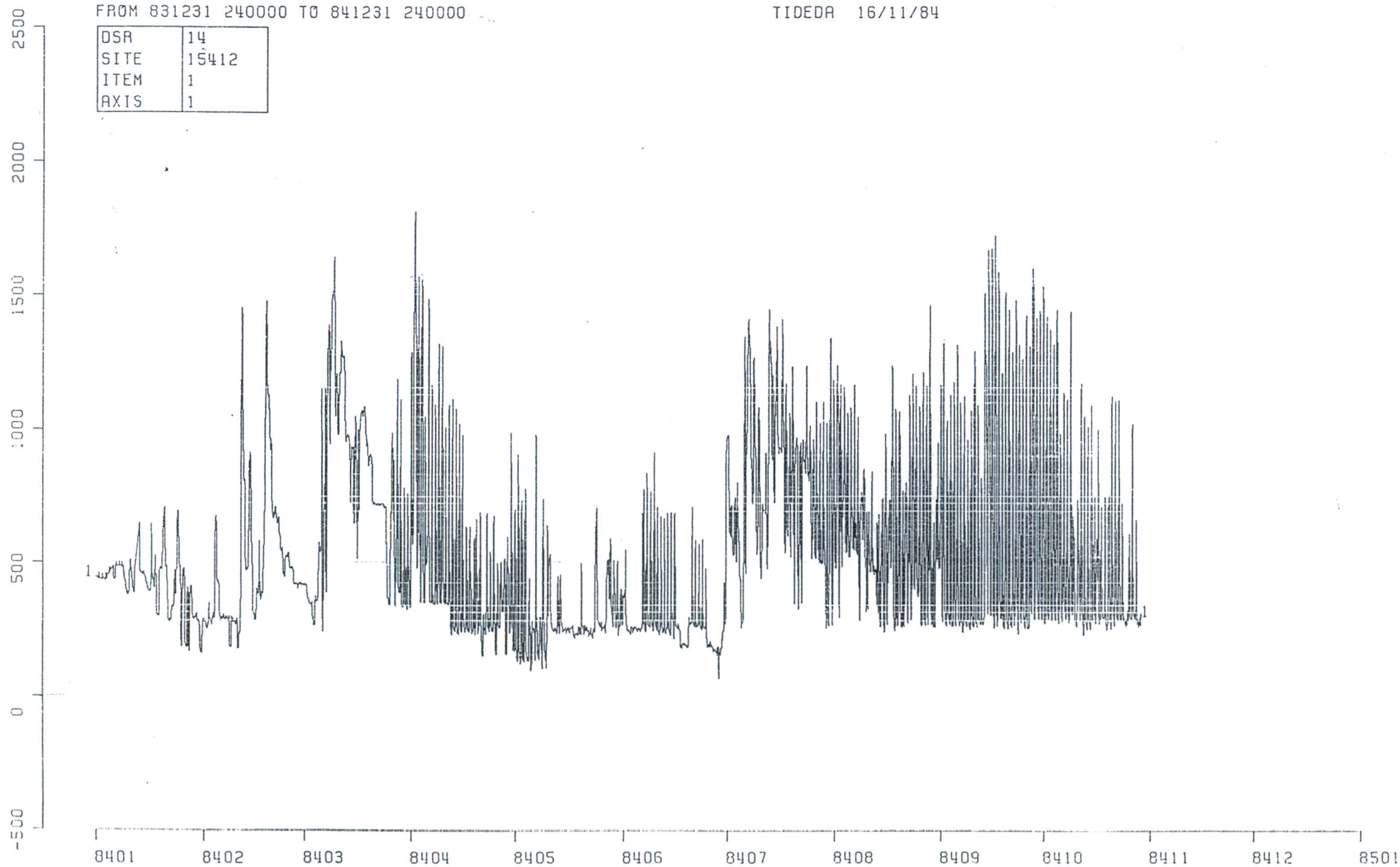




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TIDEDR 16/11/84

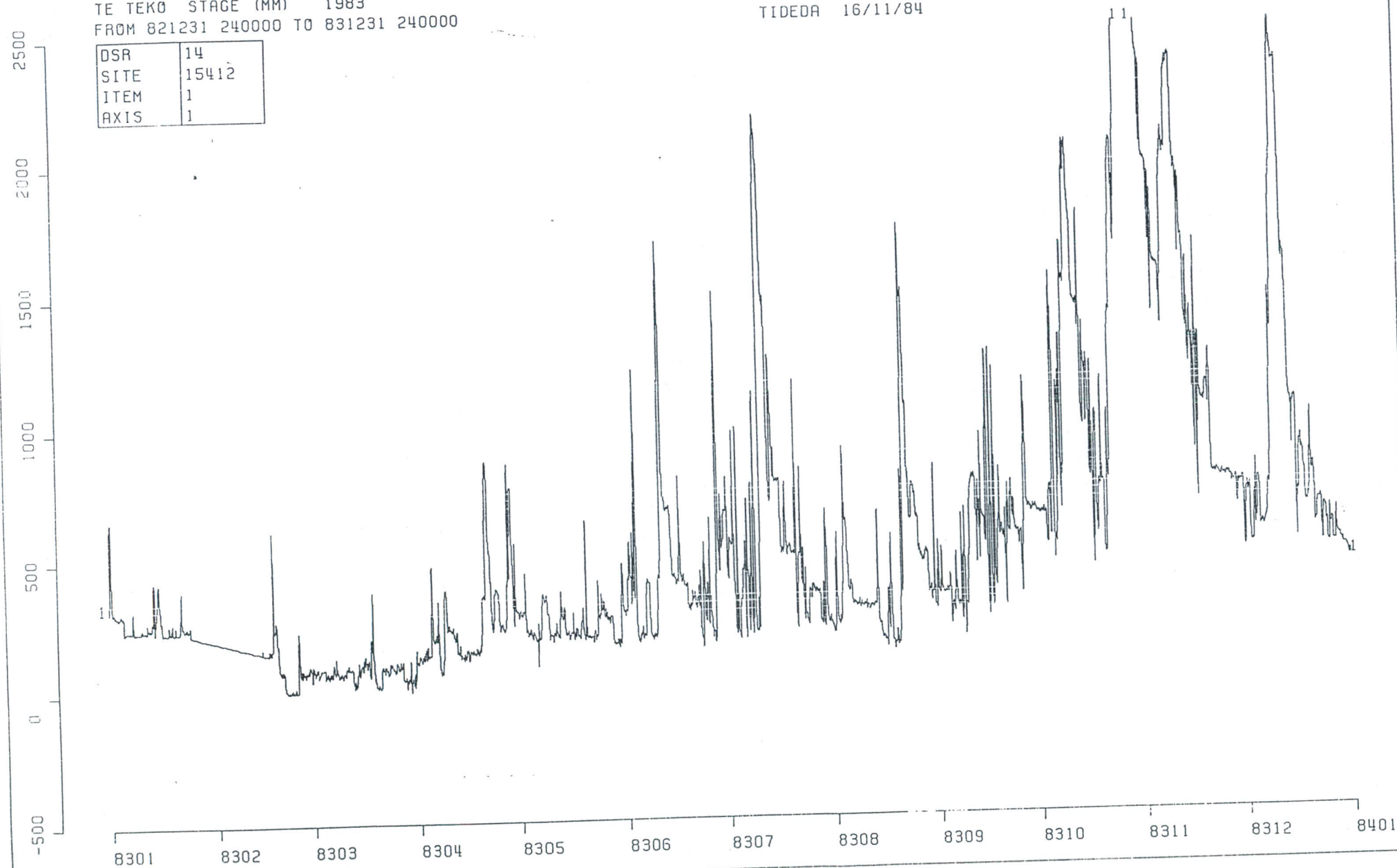
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TIDE0A 16/11/84

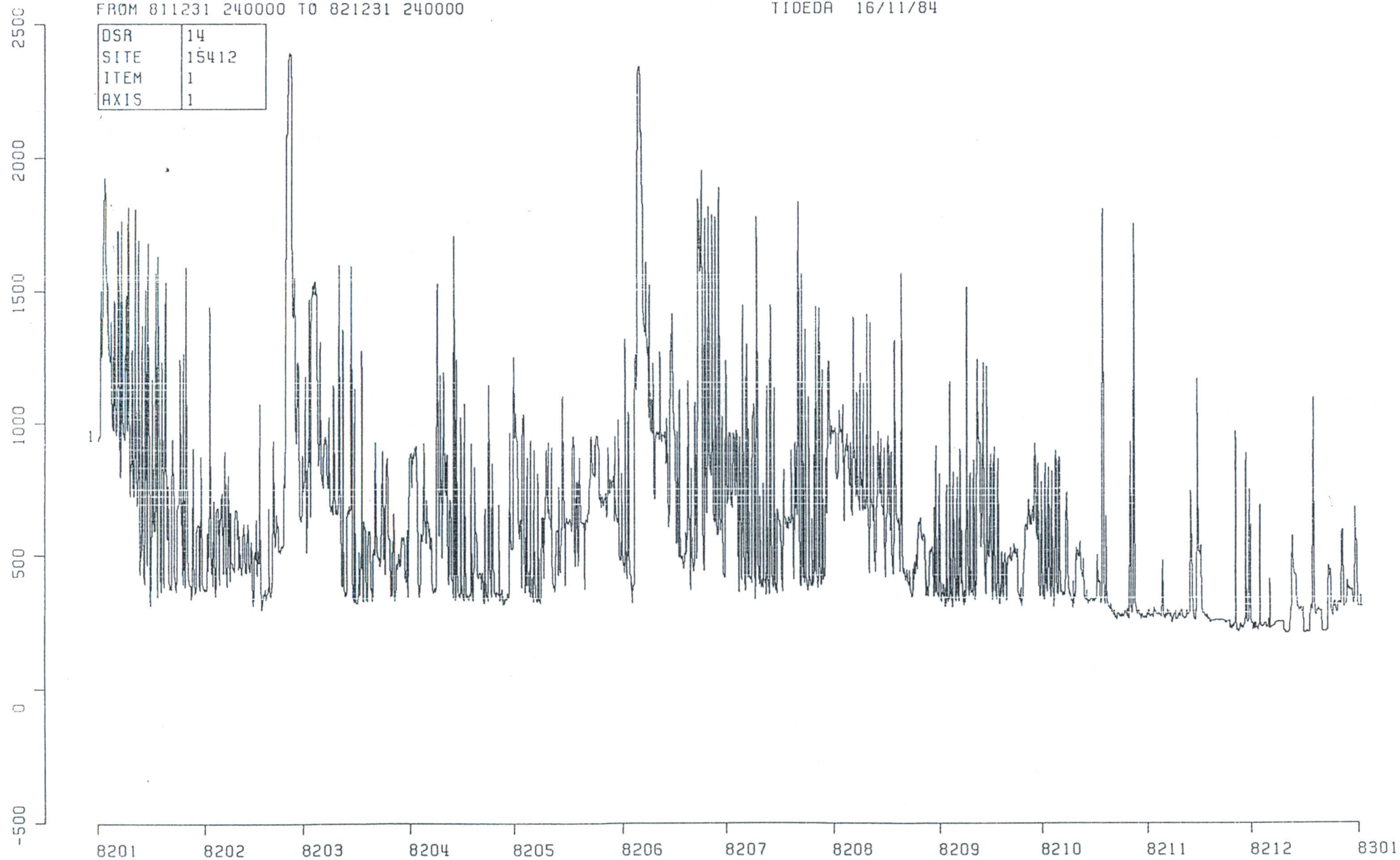
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TIDEDA 16/11/84

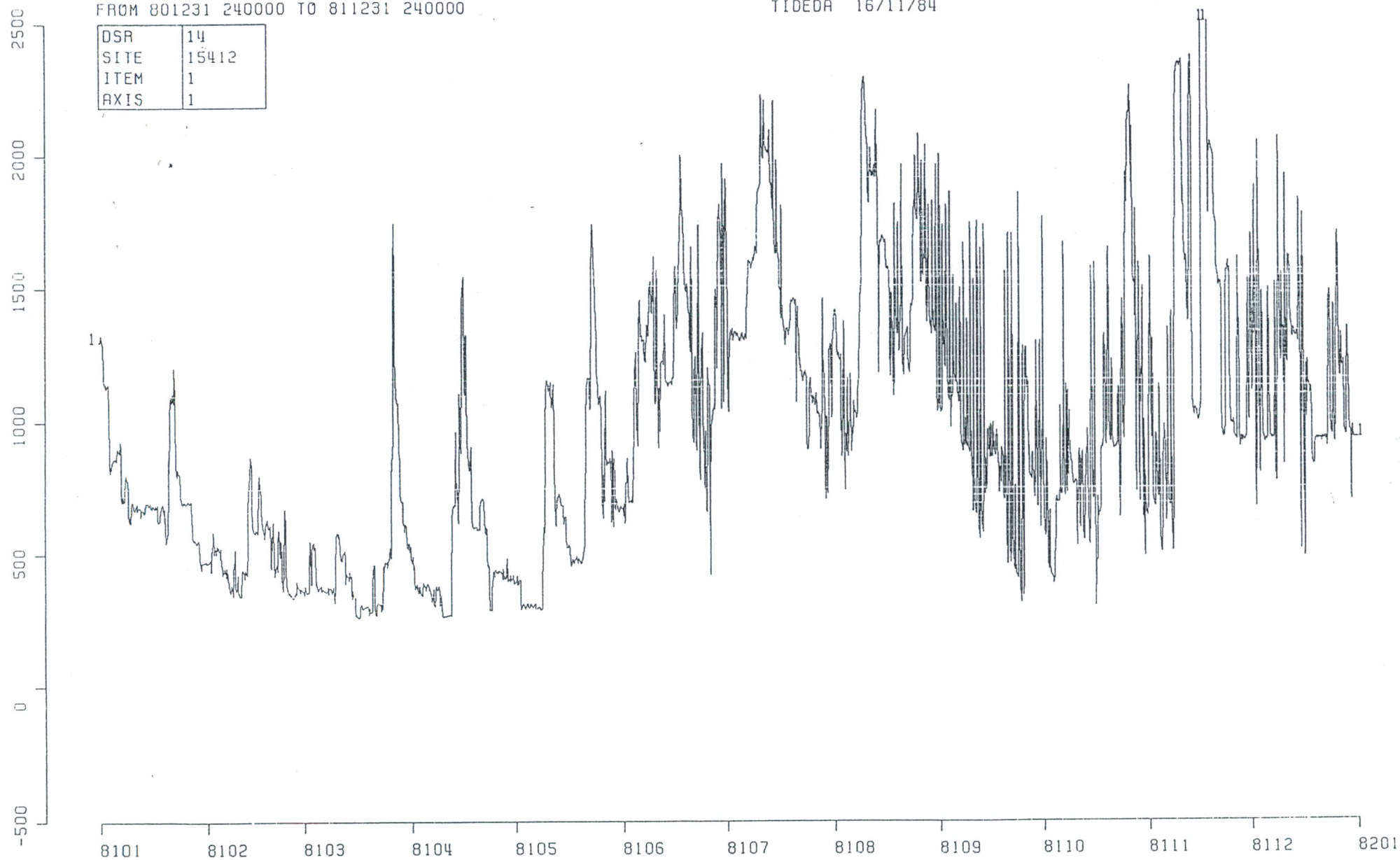
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TIDEDR 16/11/84

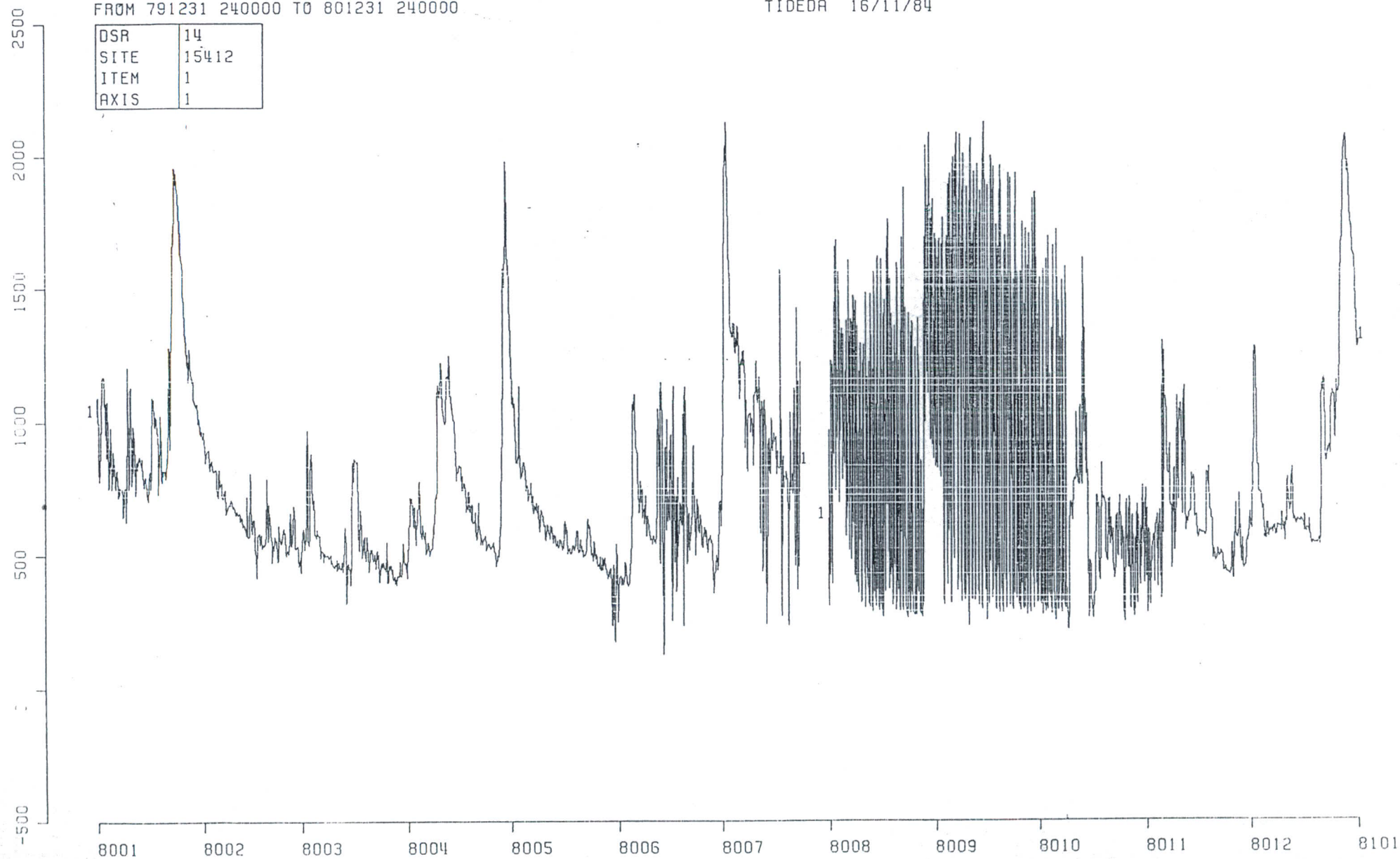
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TIDEDR 16/11/84

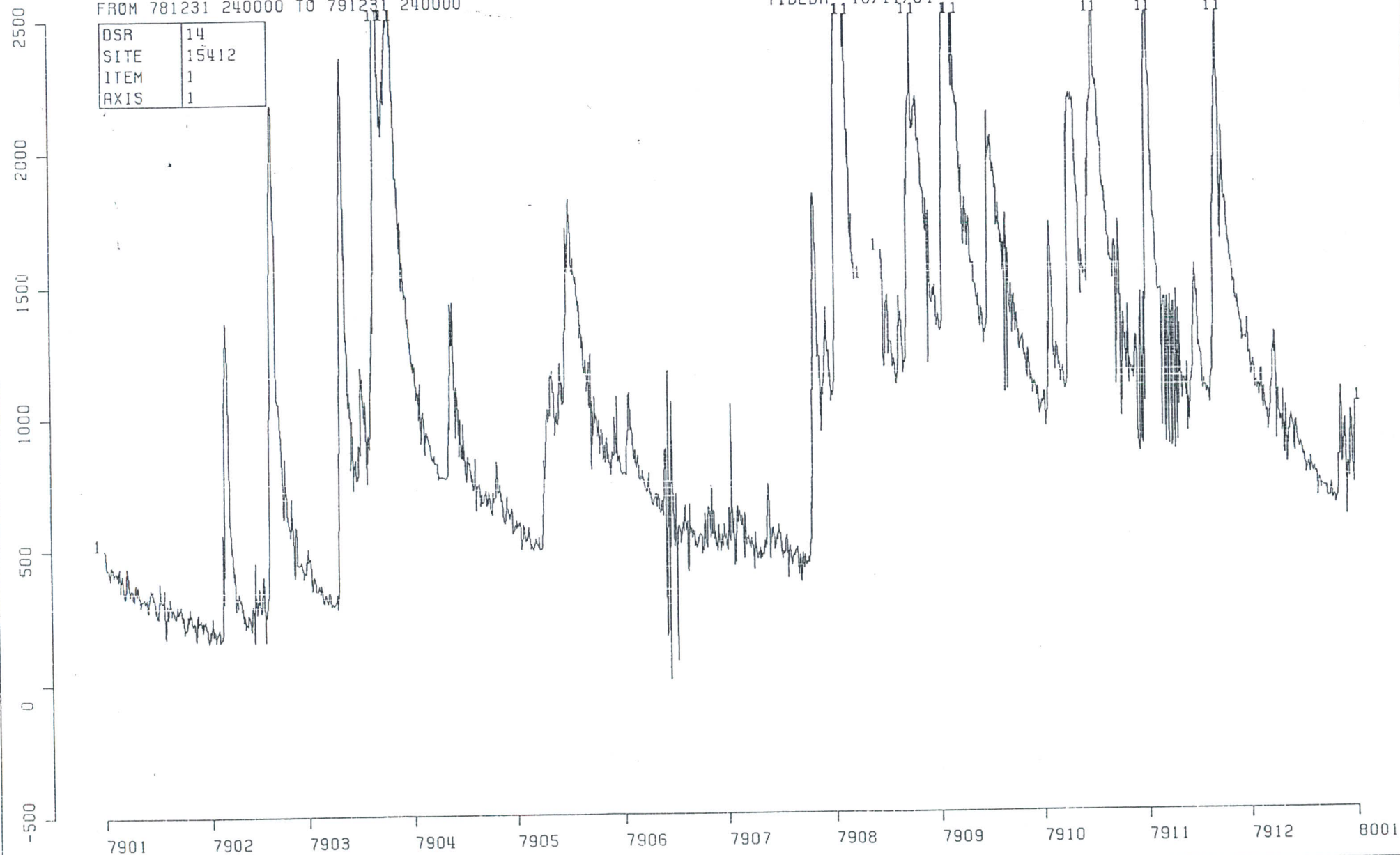
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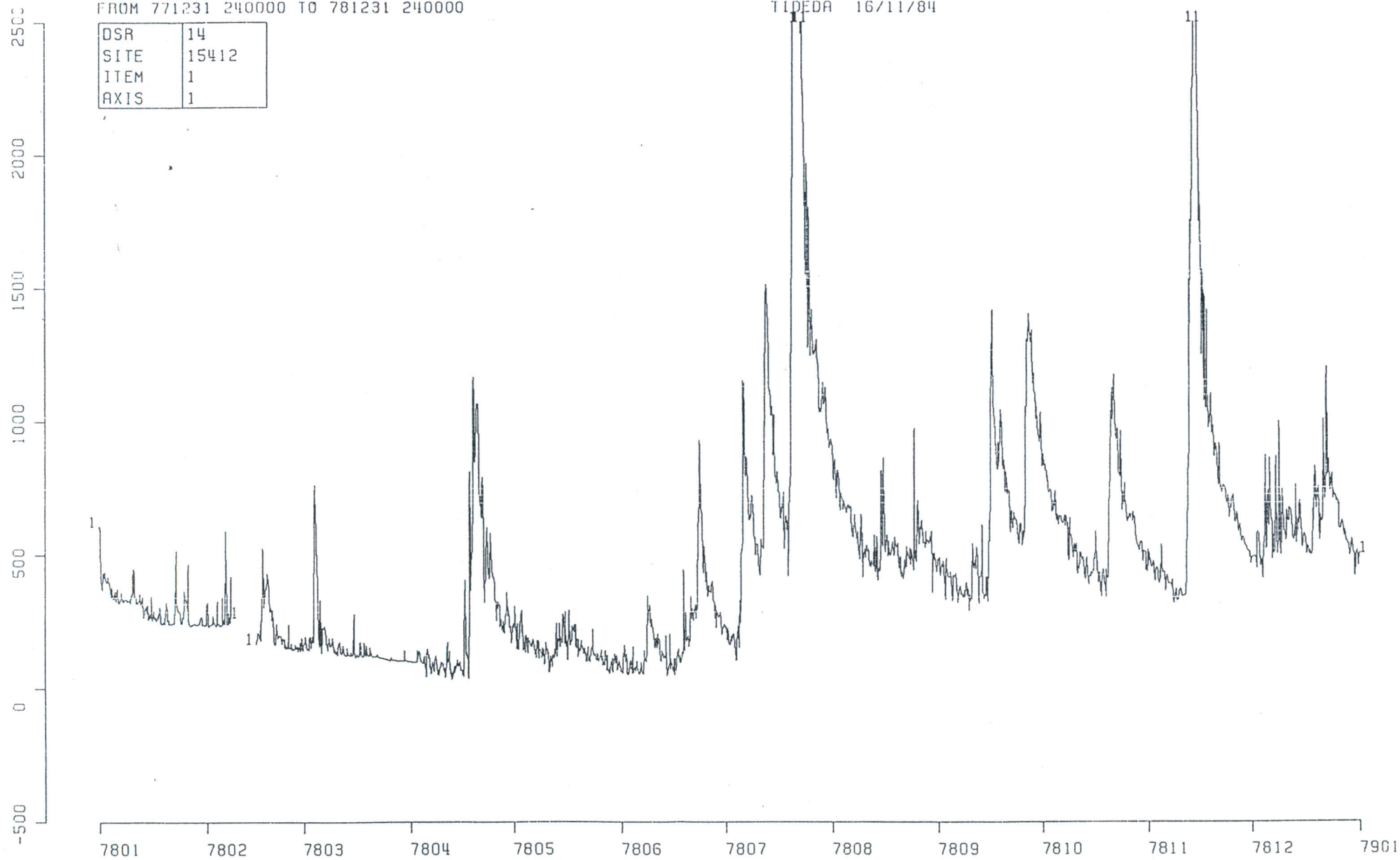
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TIDEDA 16/11/84

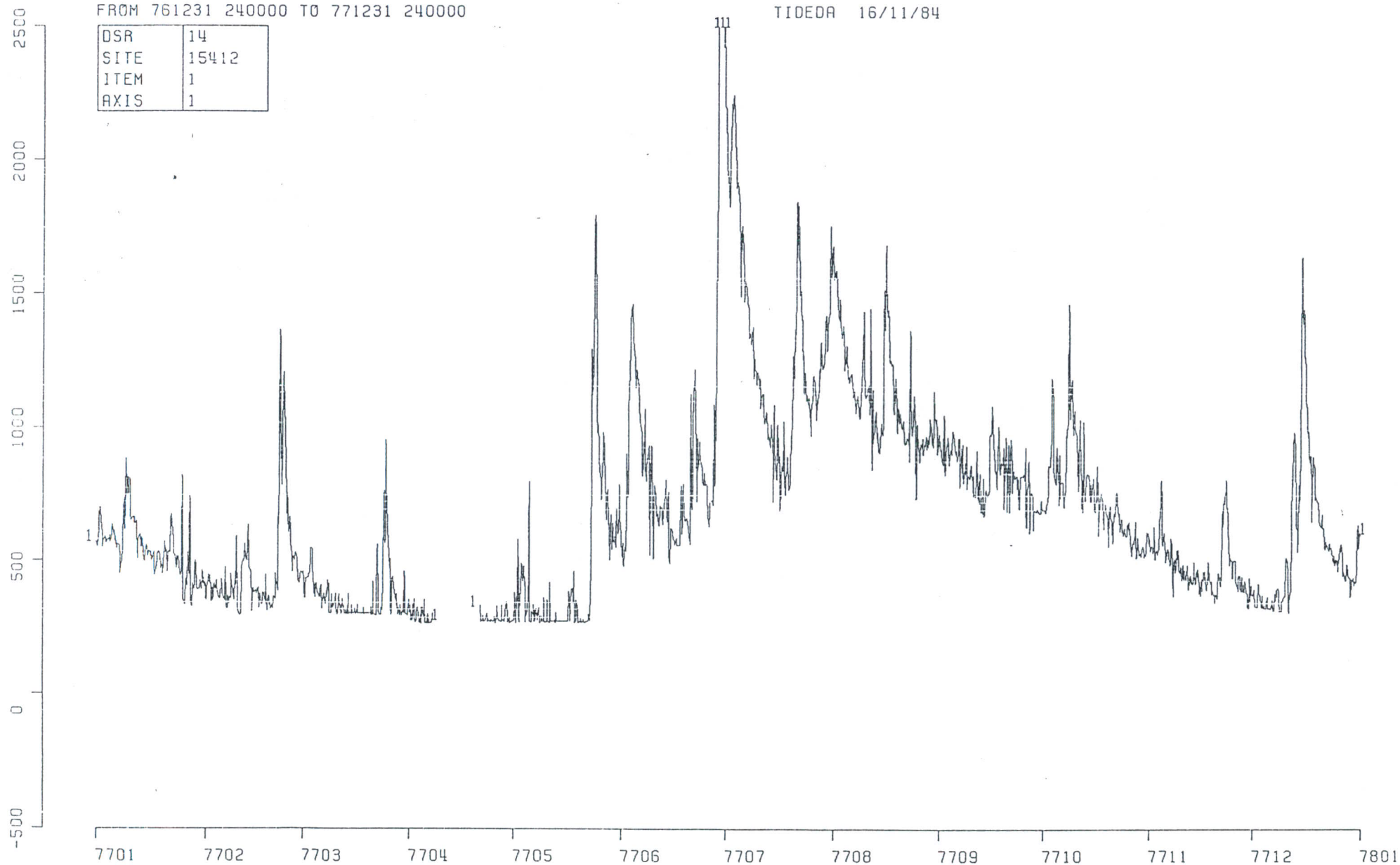
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TE TEK0 STAGE (MM) 1977
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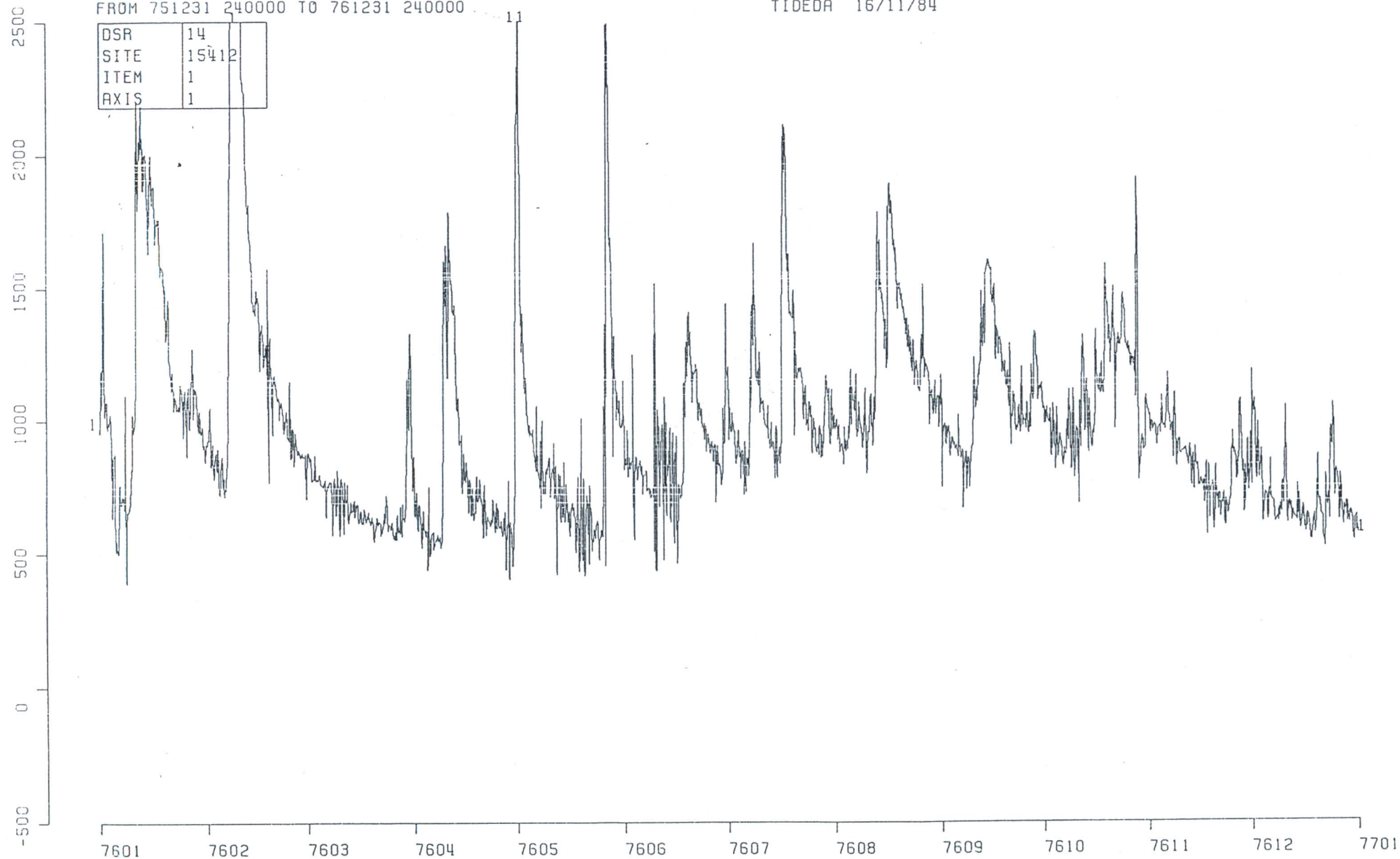
TIDE0A 16/11/84



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TIDE0A 16/11/84

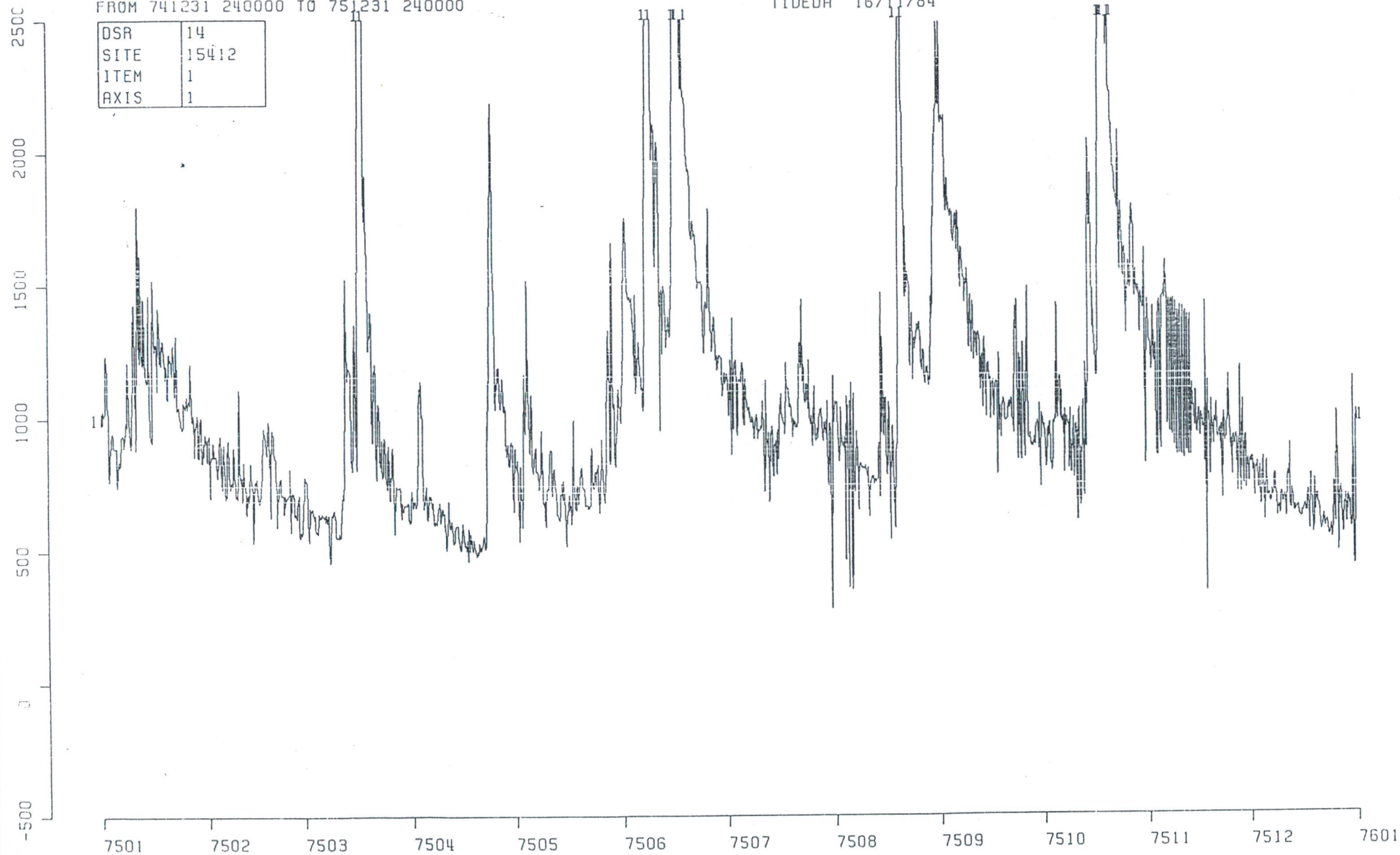
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TIDEDR 16/1/84

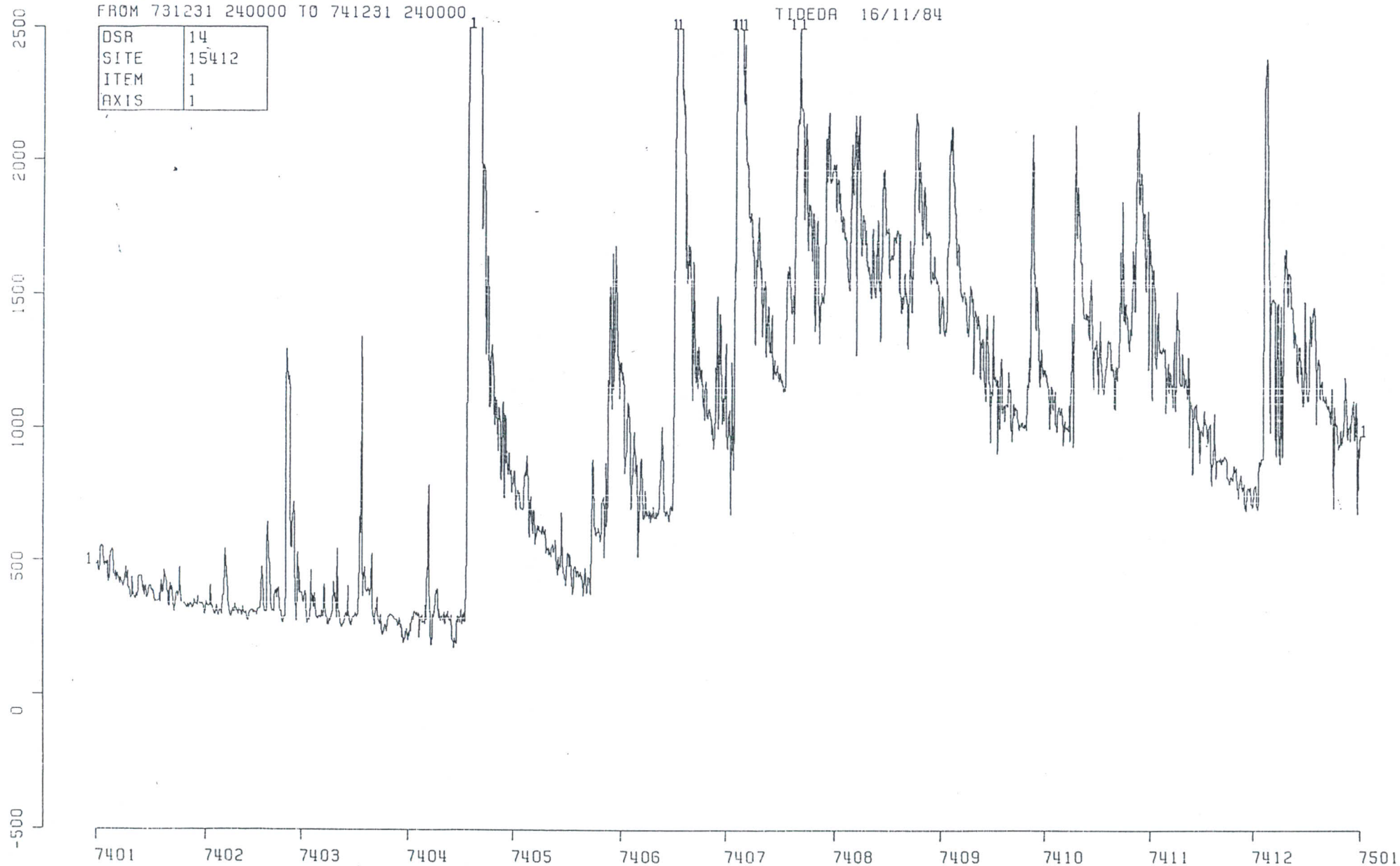
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| AXIS | 1 |



TE TEK0 STAGE (MM) 1974
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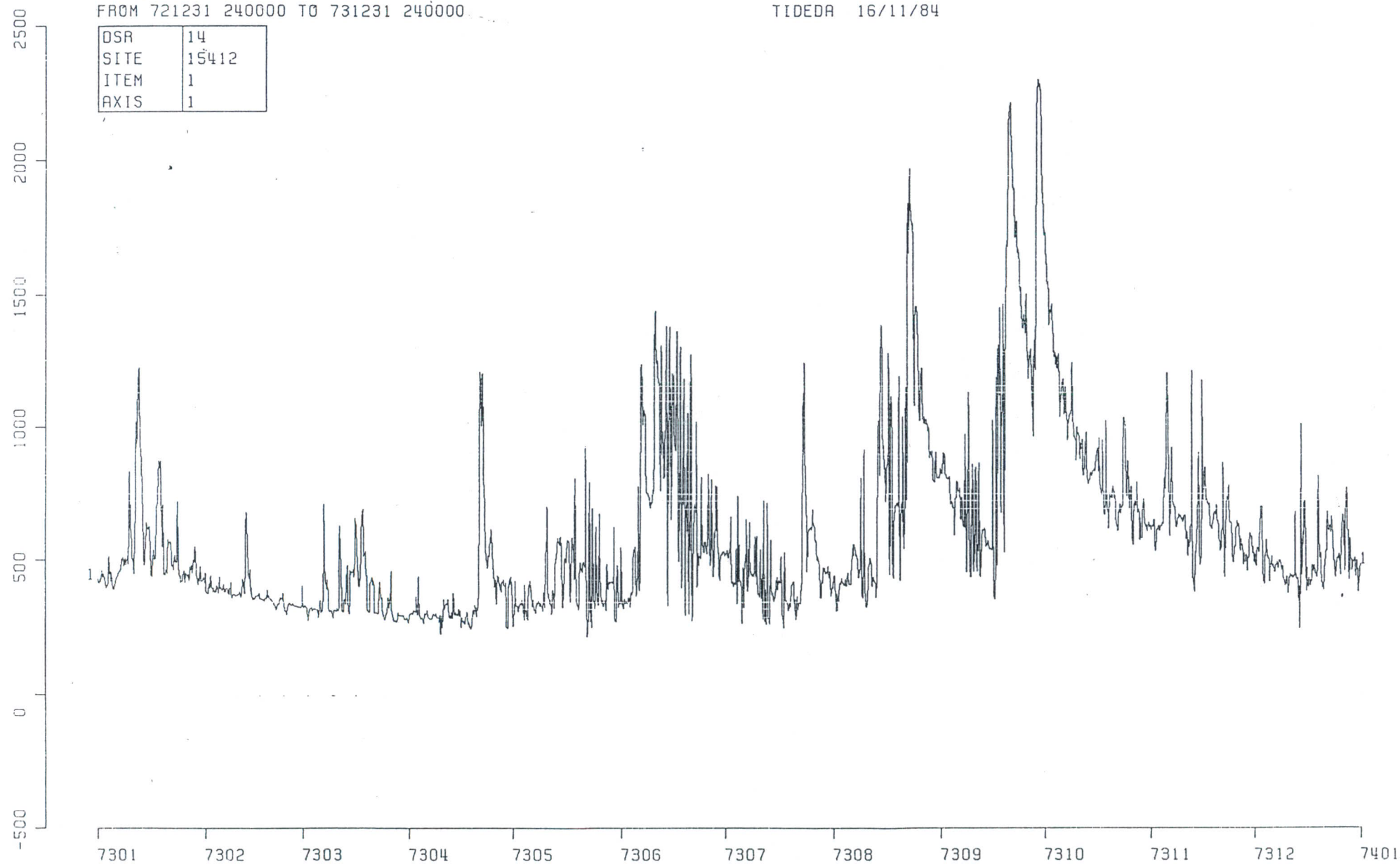
TIDEDR 16/11/84



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TIDEDA 16/11/84

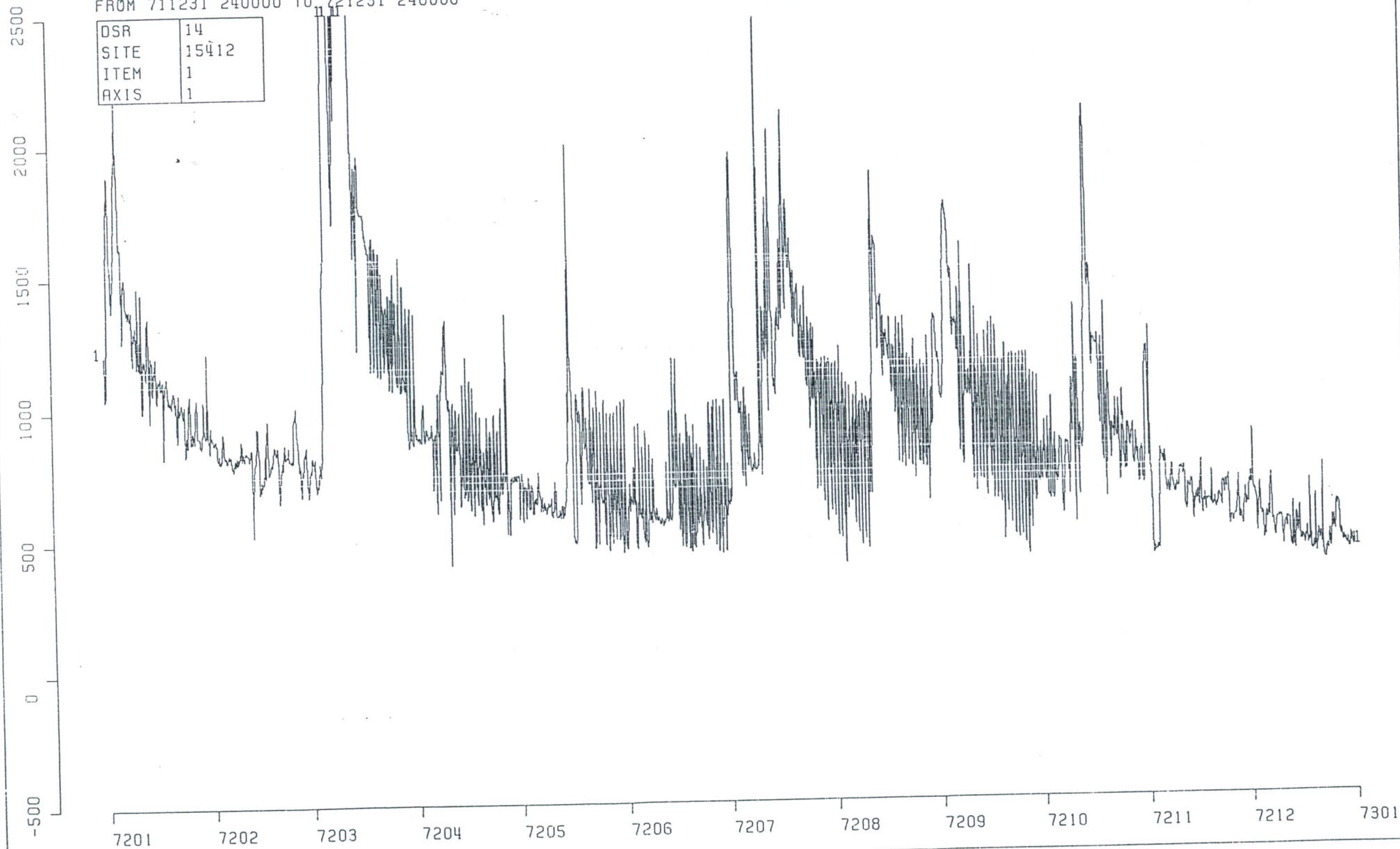
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TIDEDA 16/11/84

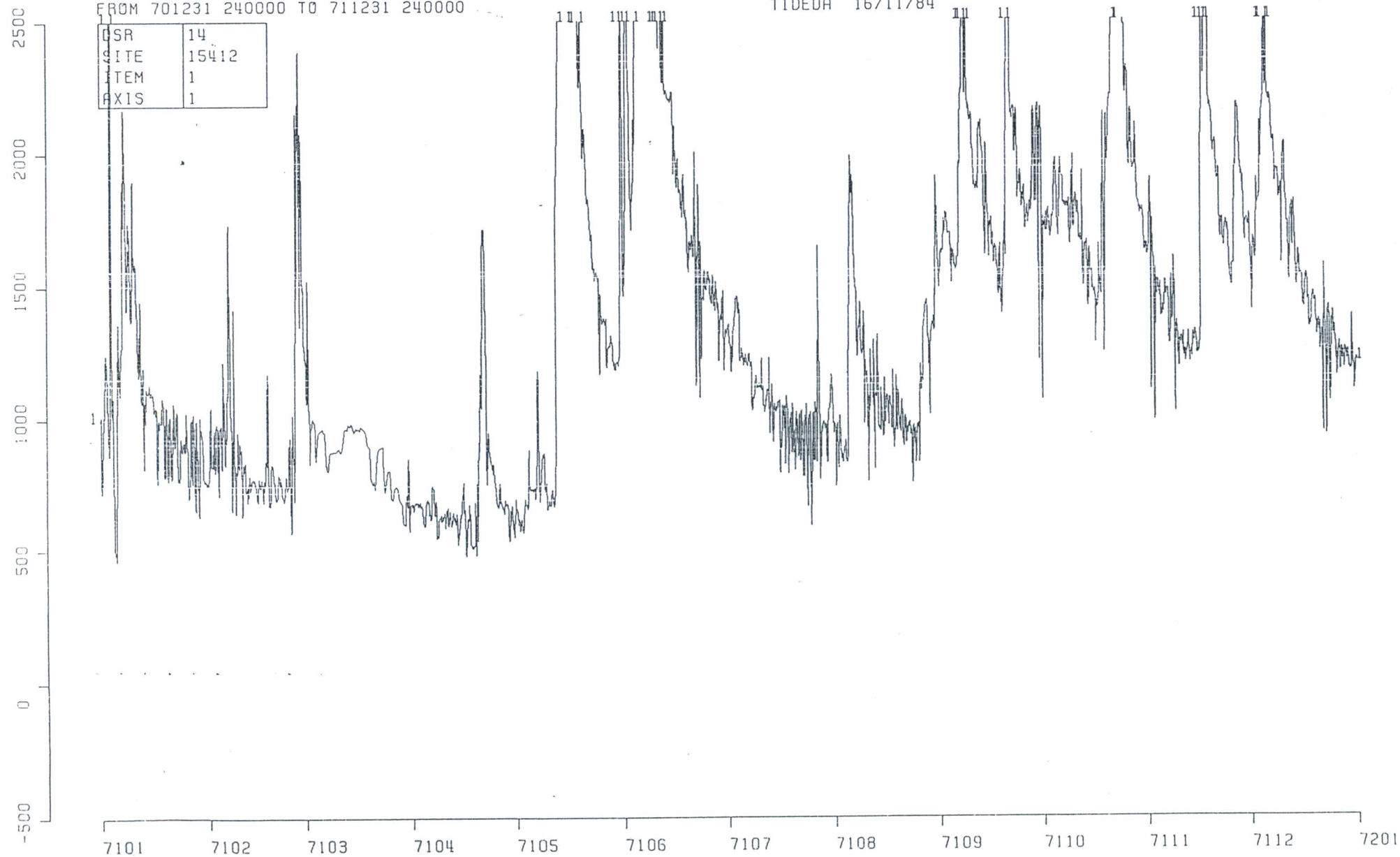
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TIDEDA 16/11/84

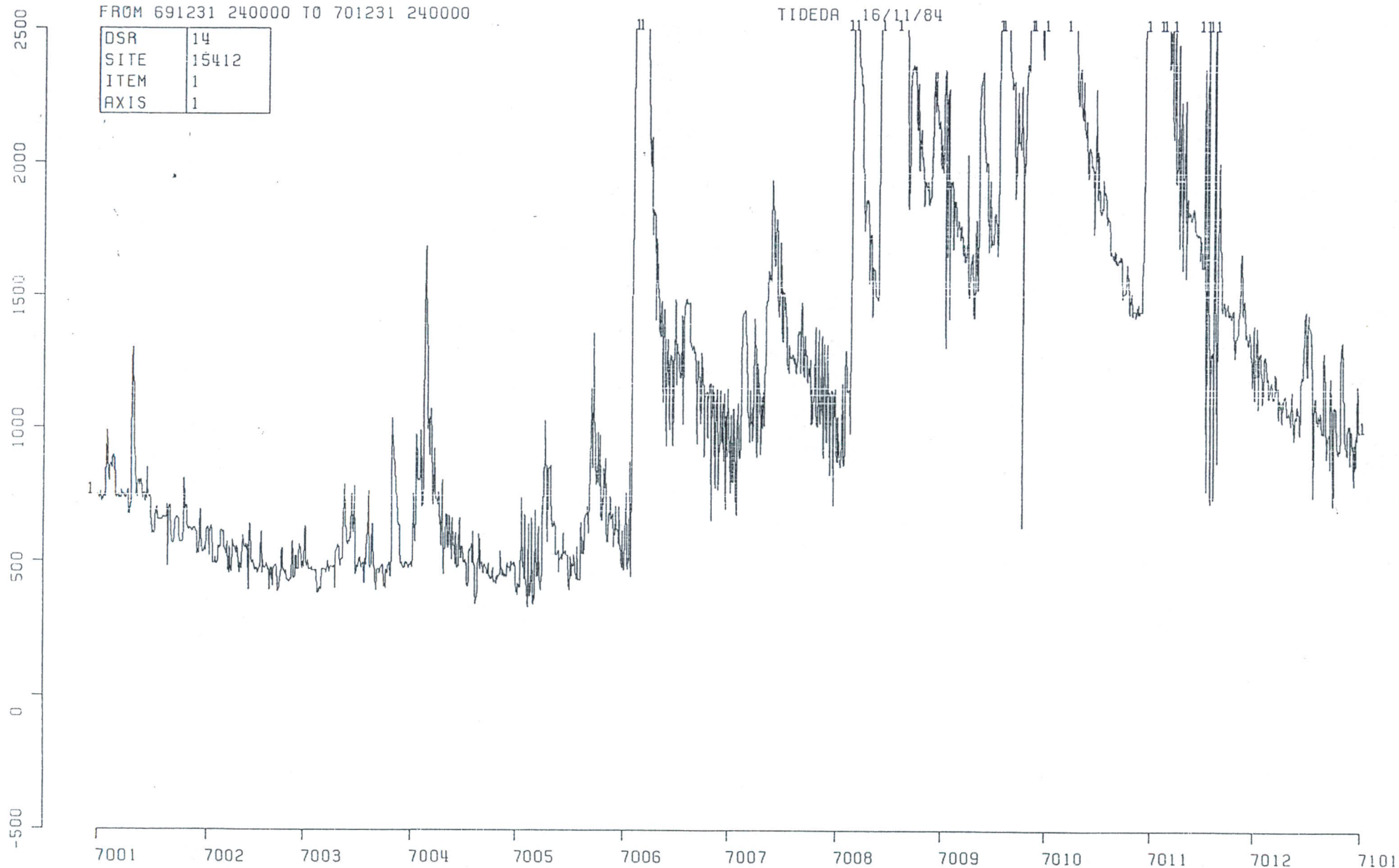
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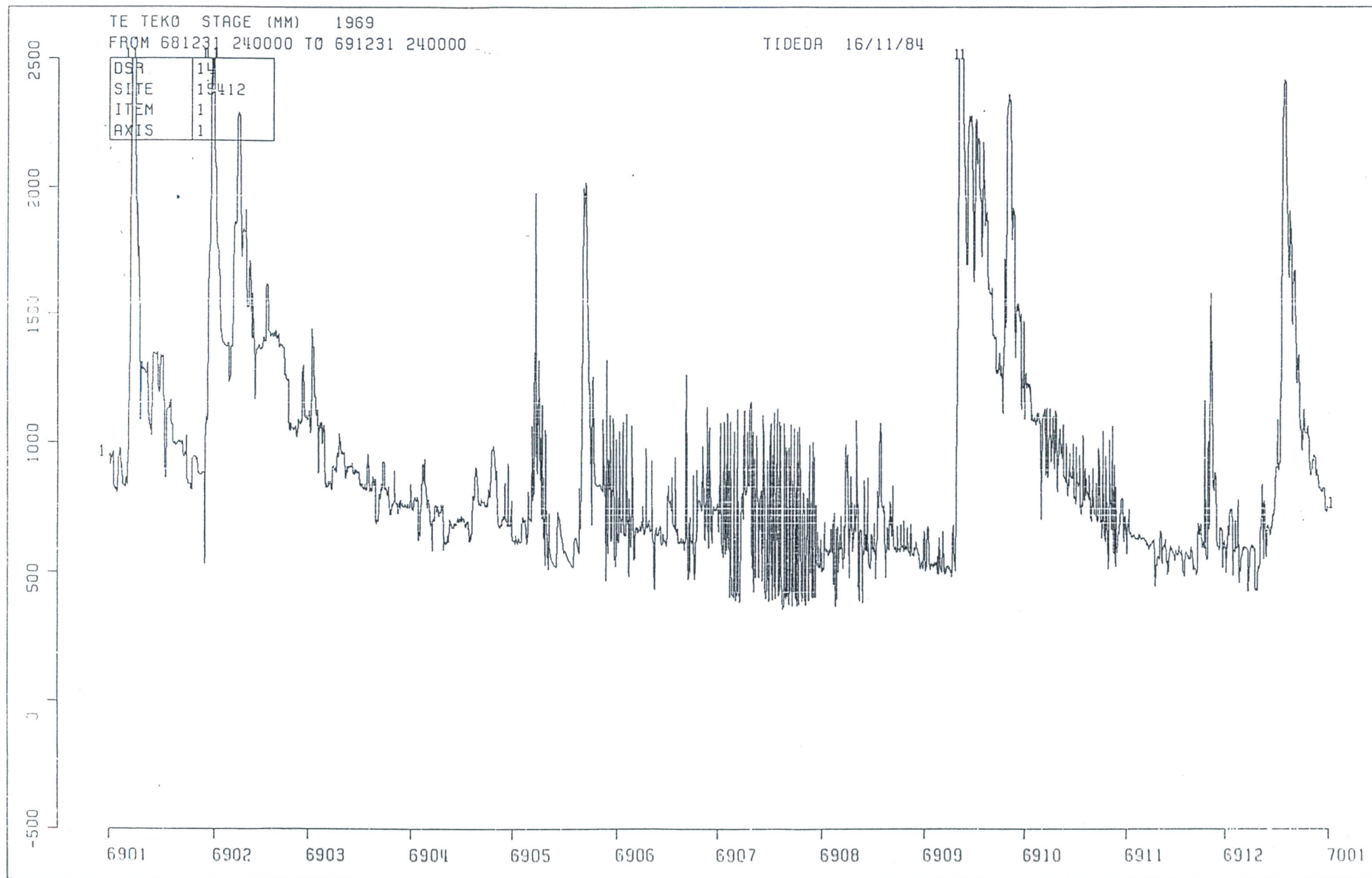


TE TEK0 STAGE (MM) 1970
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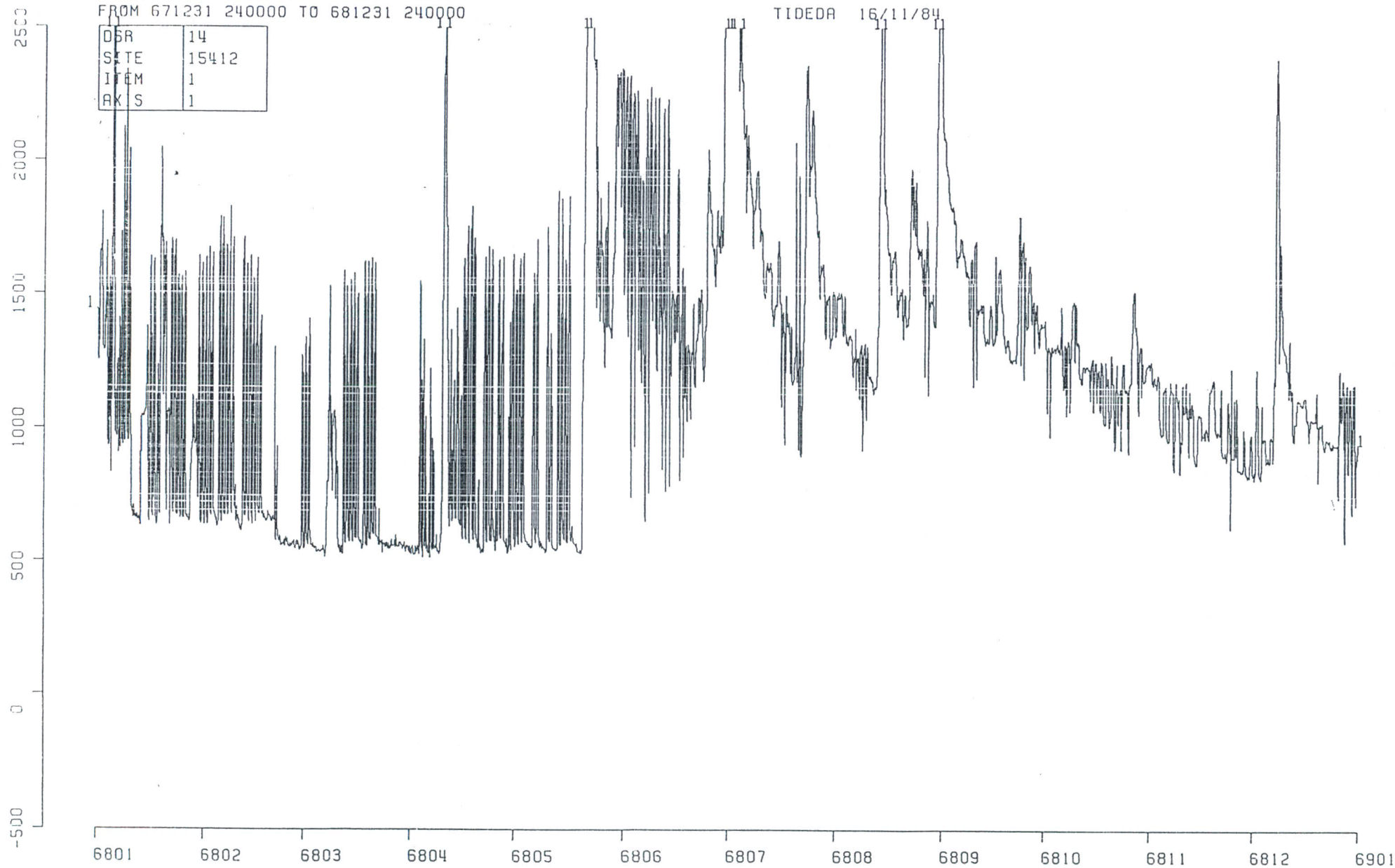




TE TEK0 STAGE (MM) 1968
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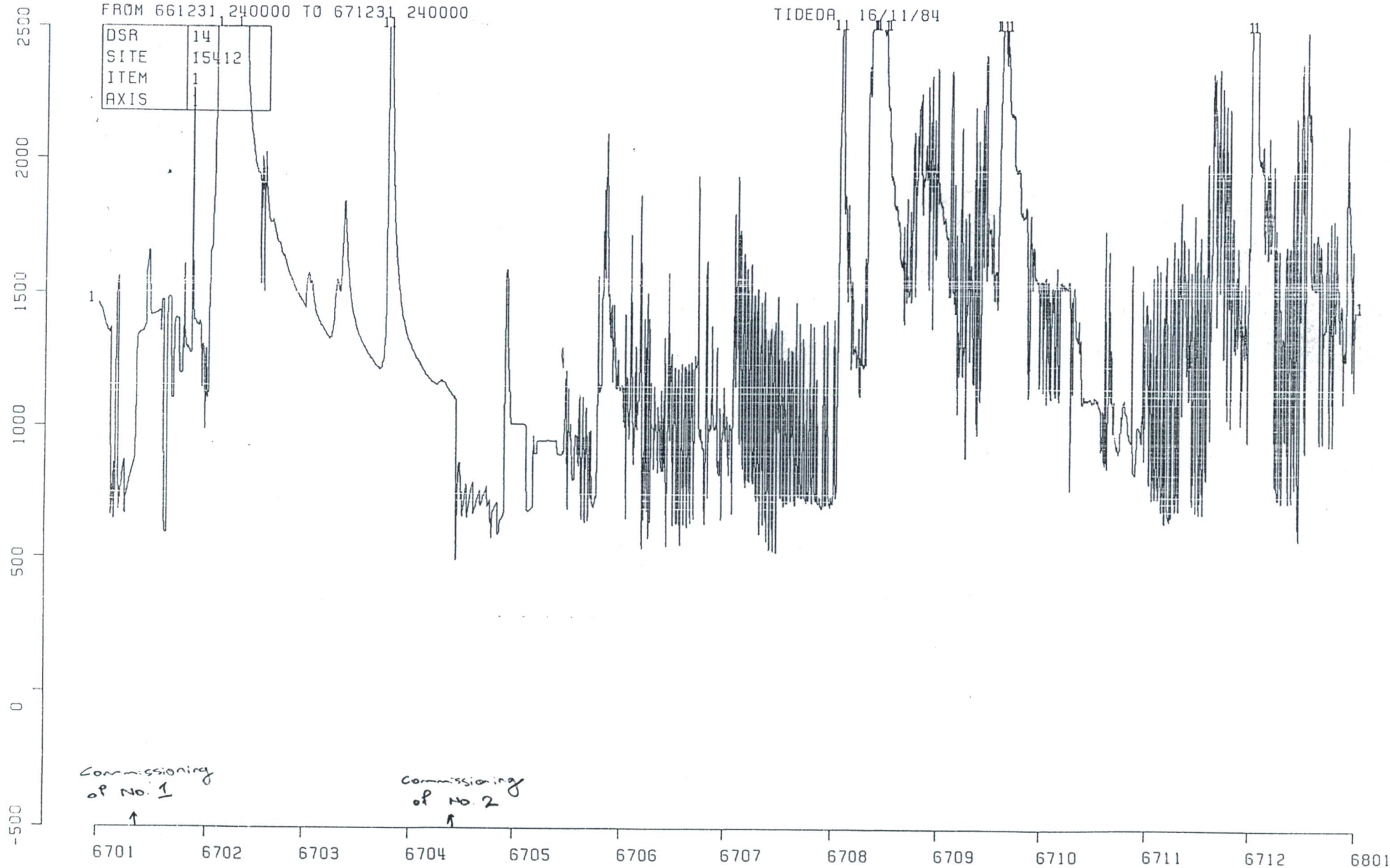
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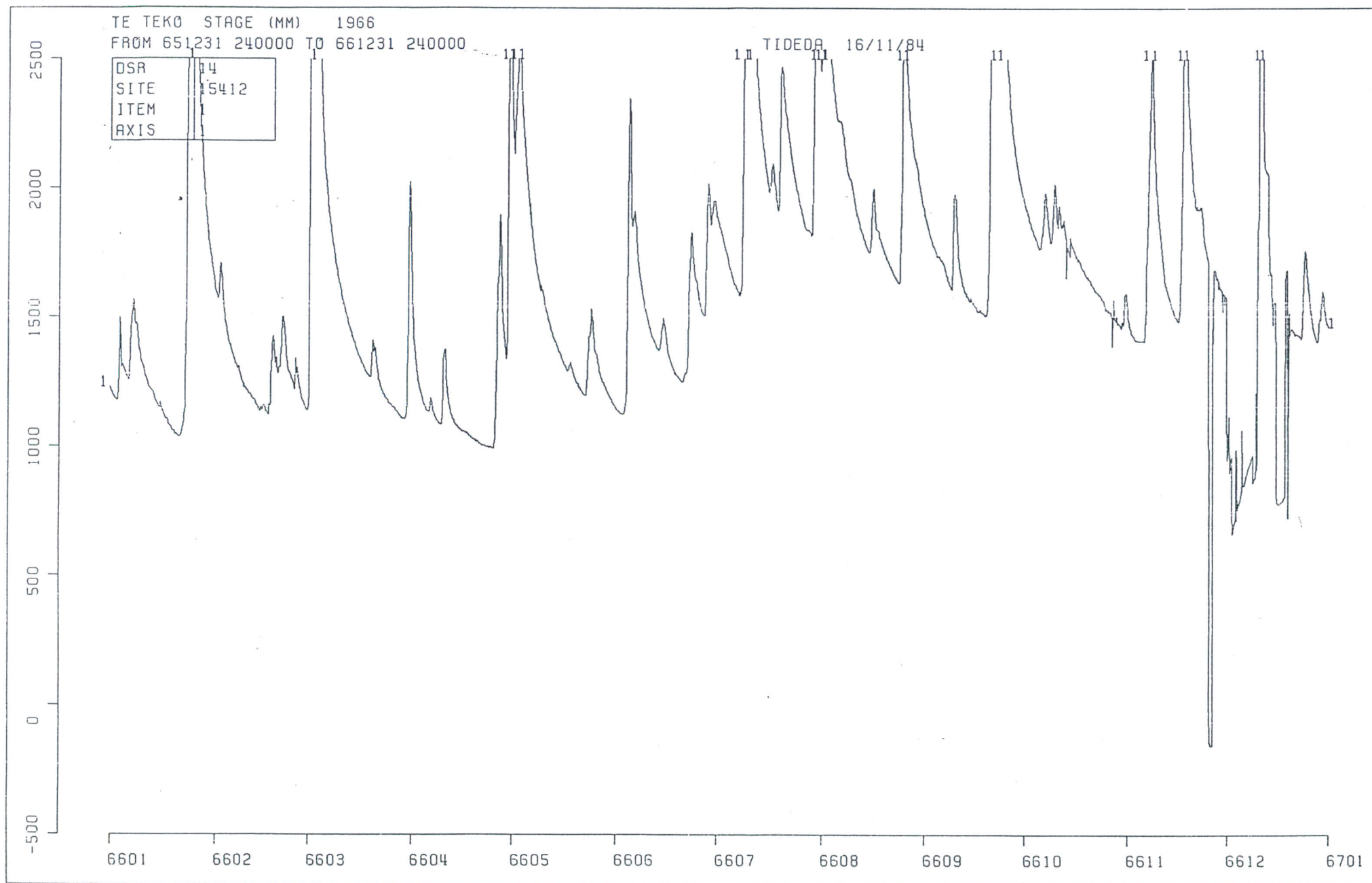


TE TEK0 STAGE (MM) 1967
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TIDE0A 16/11/84

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| AXIS | |

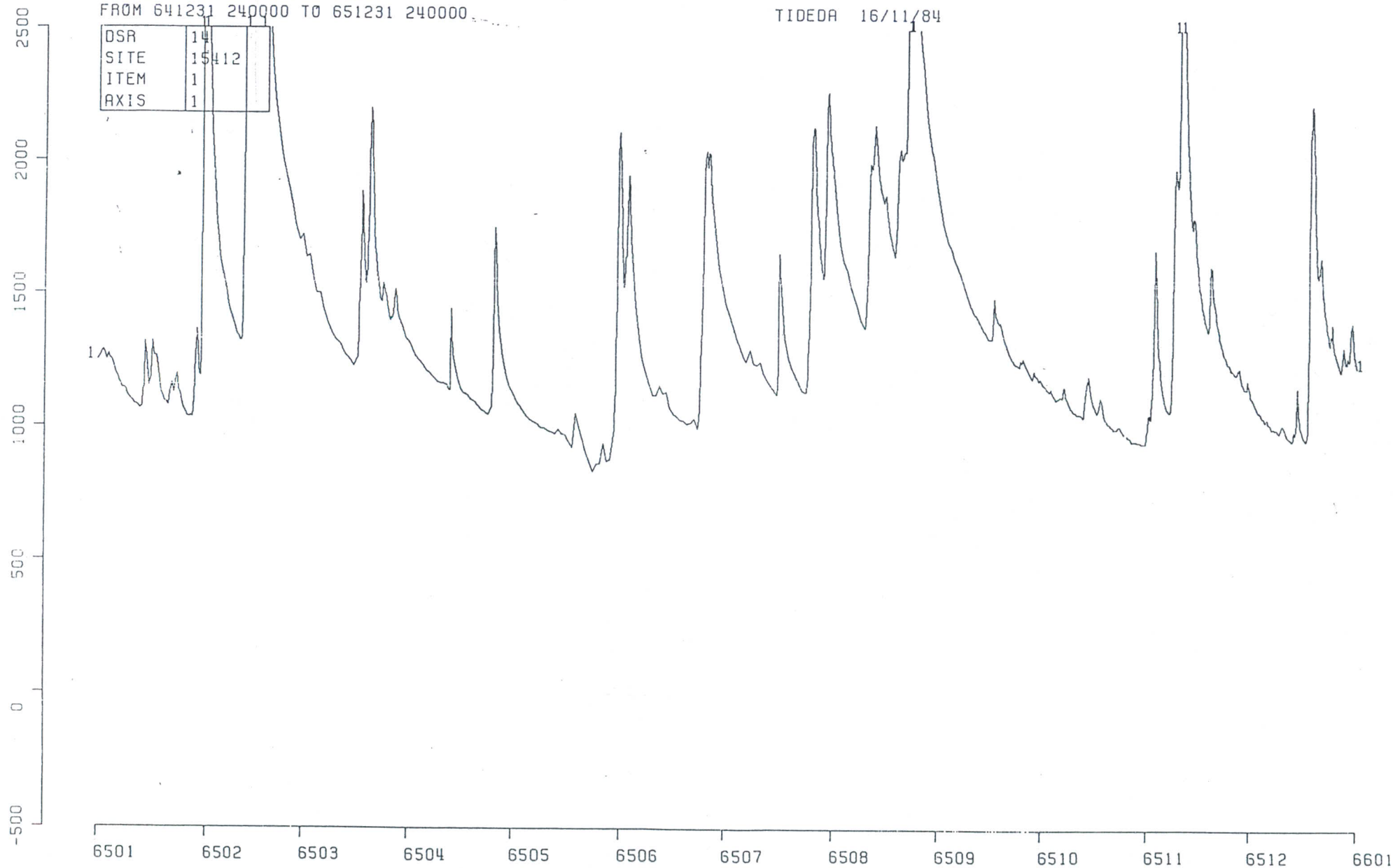




TE TEK0 STAGE (MM) 1965
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TIDEDA 16/11/84

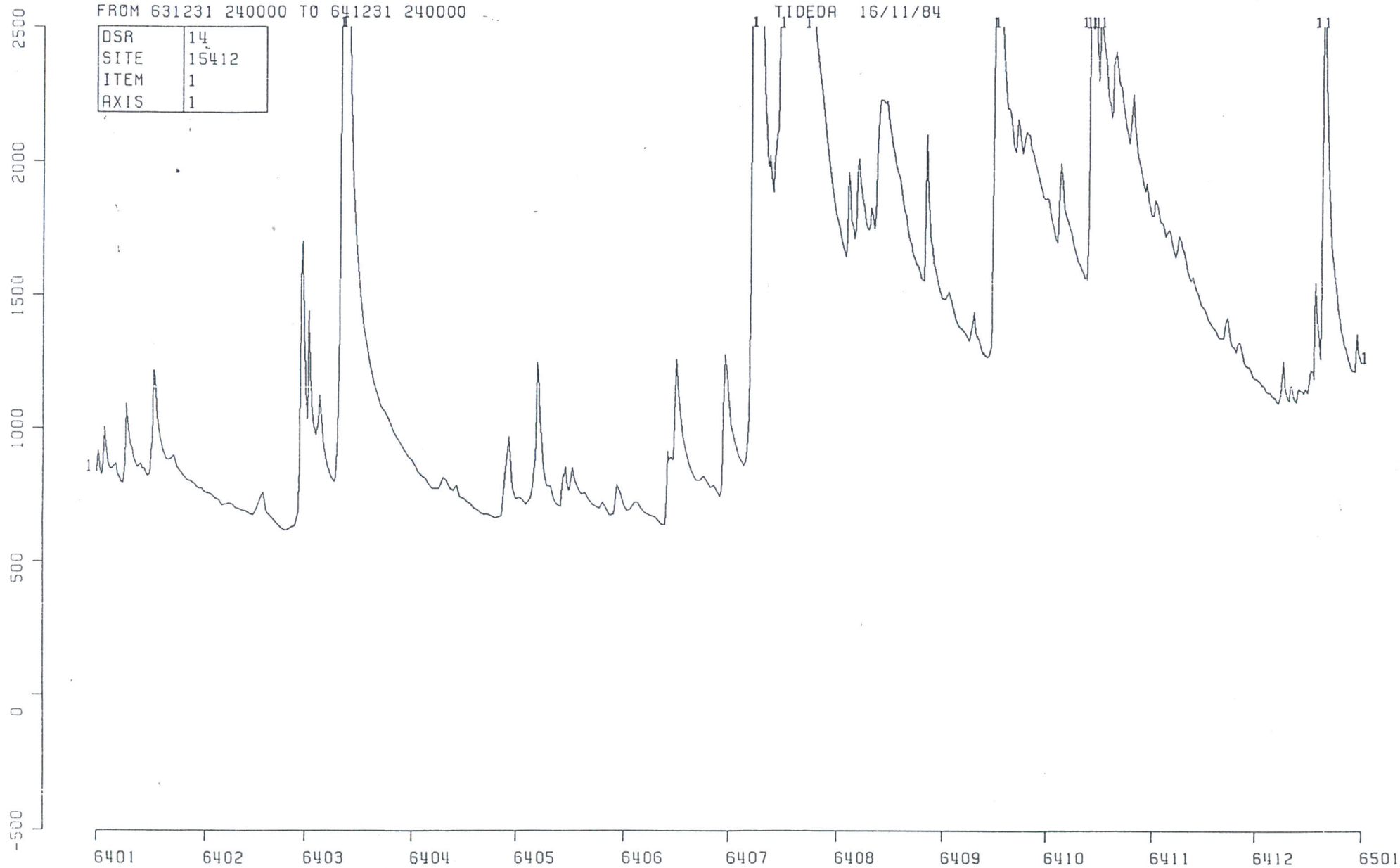
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TE TEK0 STAGE (MM) 1964
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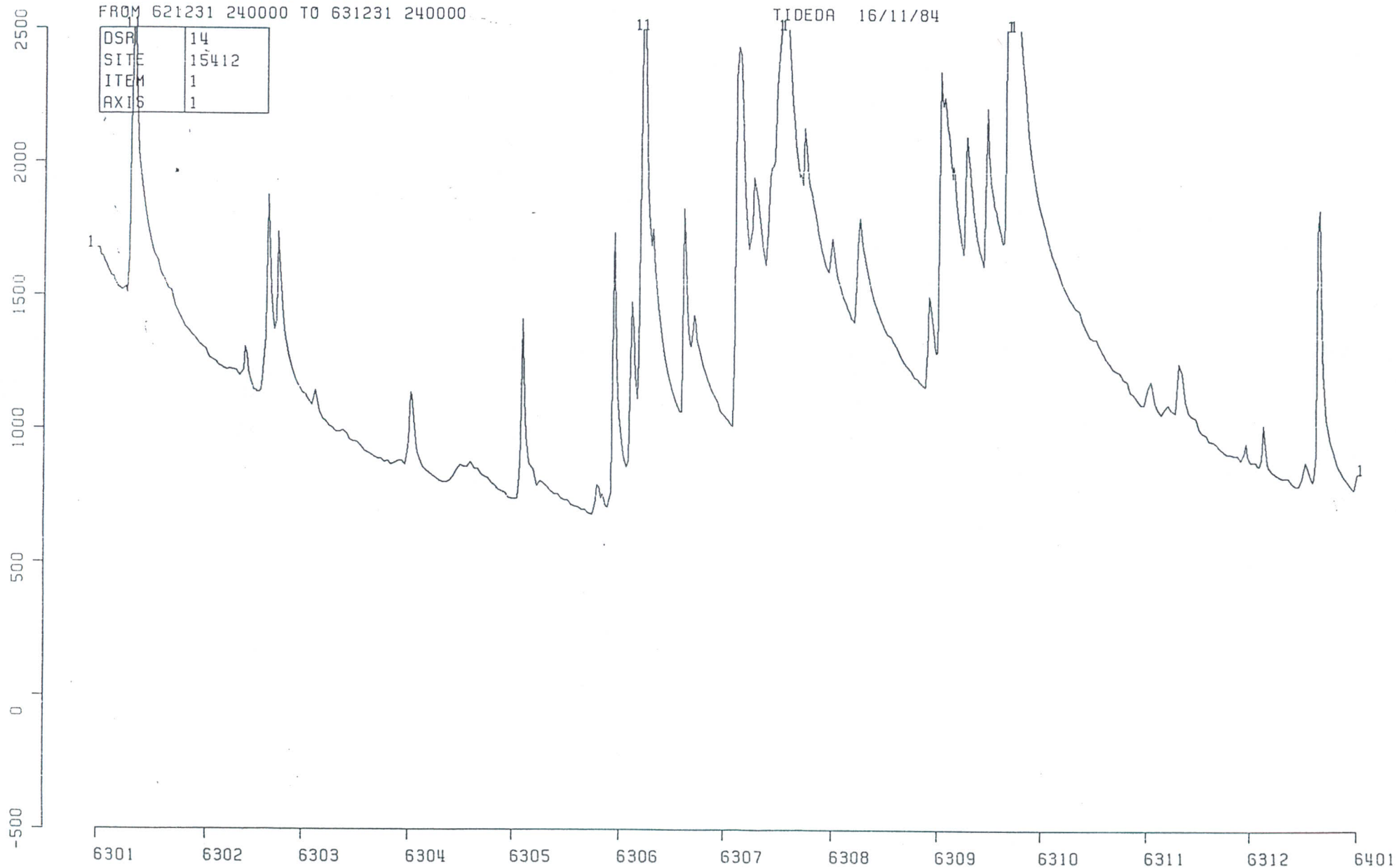
TIDEA 16/11/84



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TIDEA 16/11/84

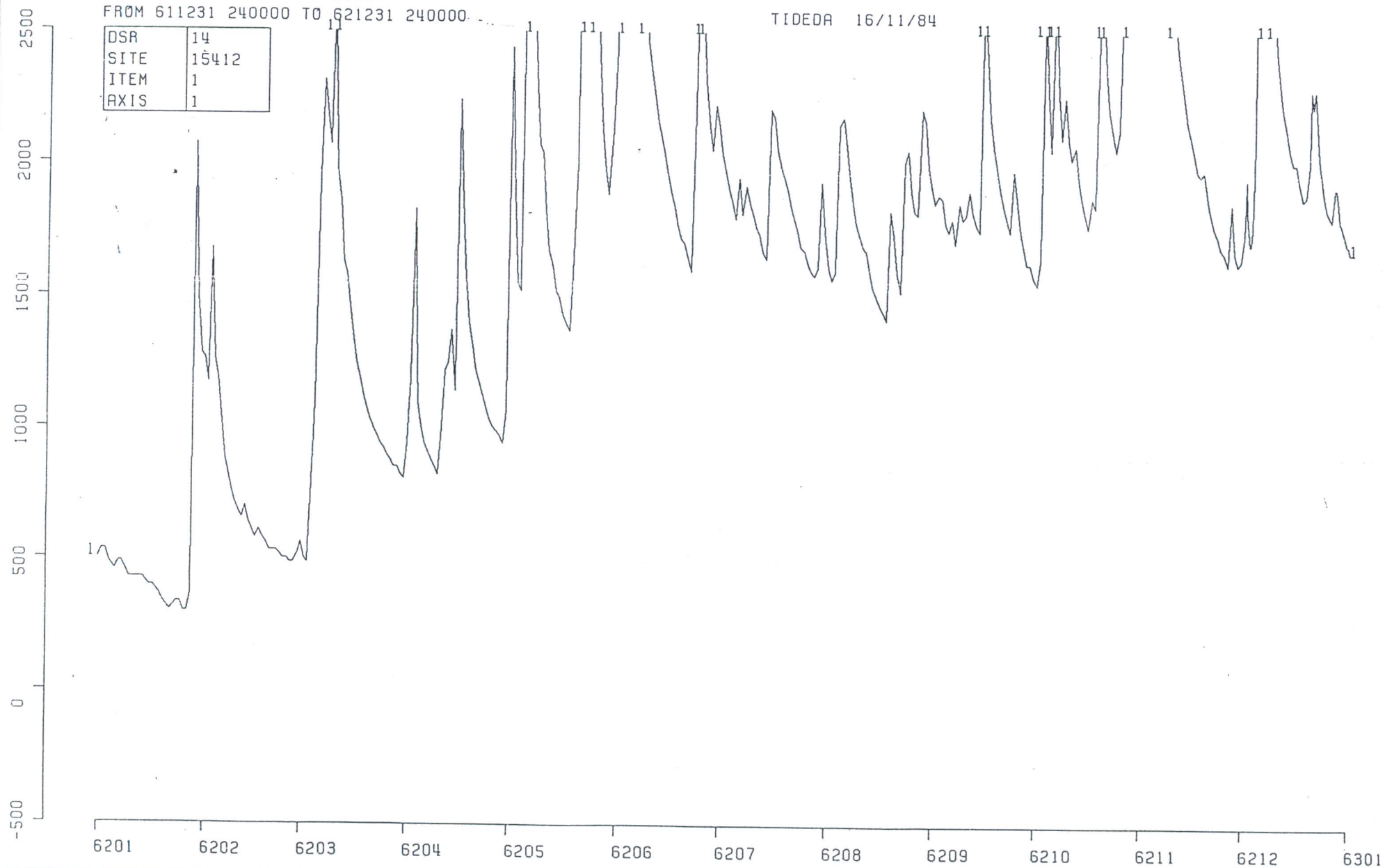
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TE TEK0 STAGE (MM) 1962
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TIDEDR 16/11/84

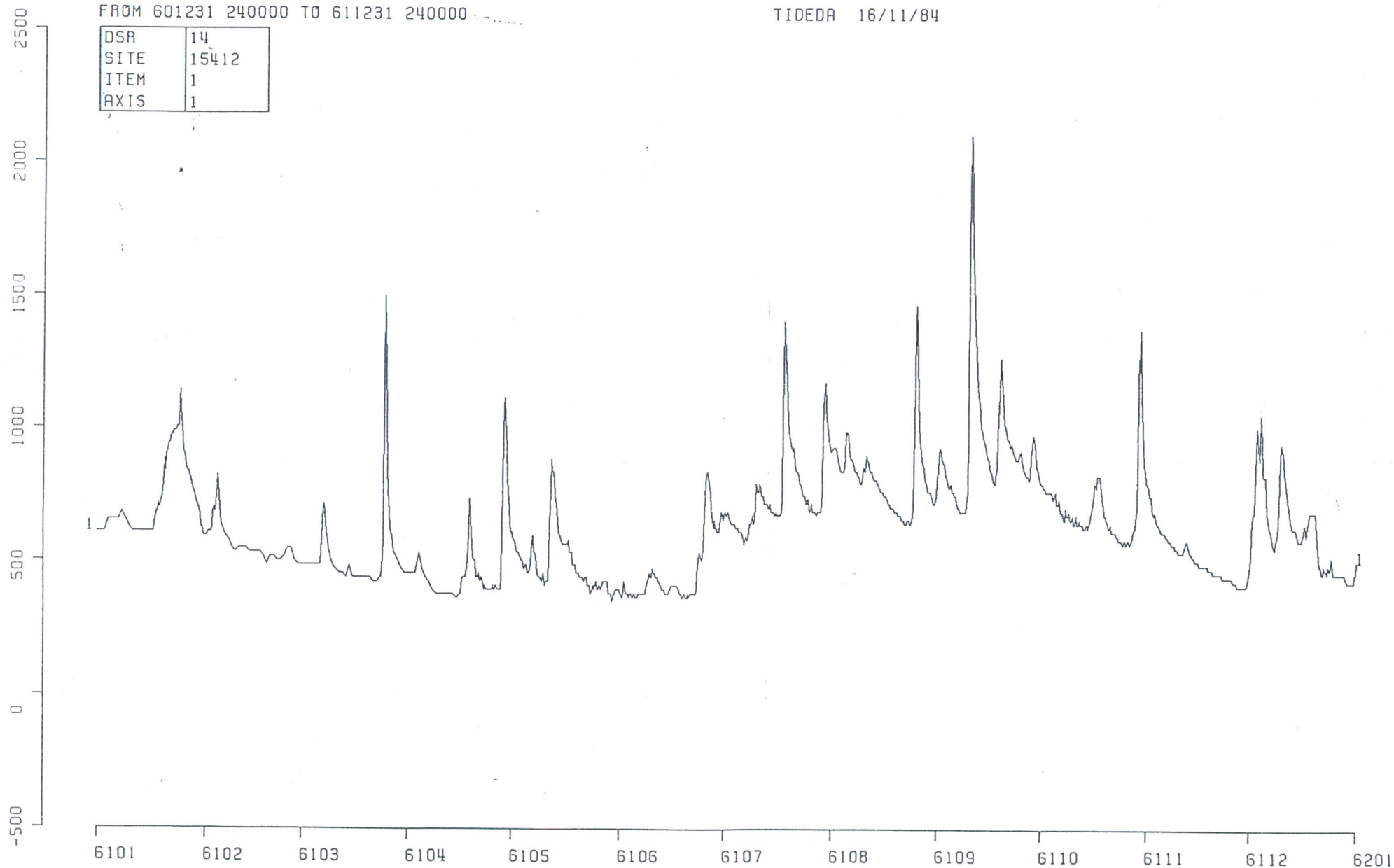
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TE TEK0 STAGE (MM) 1961
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TIDEDR 16/11/84

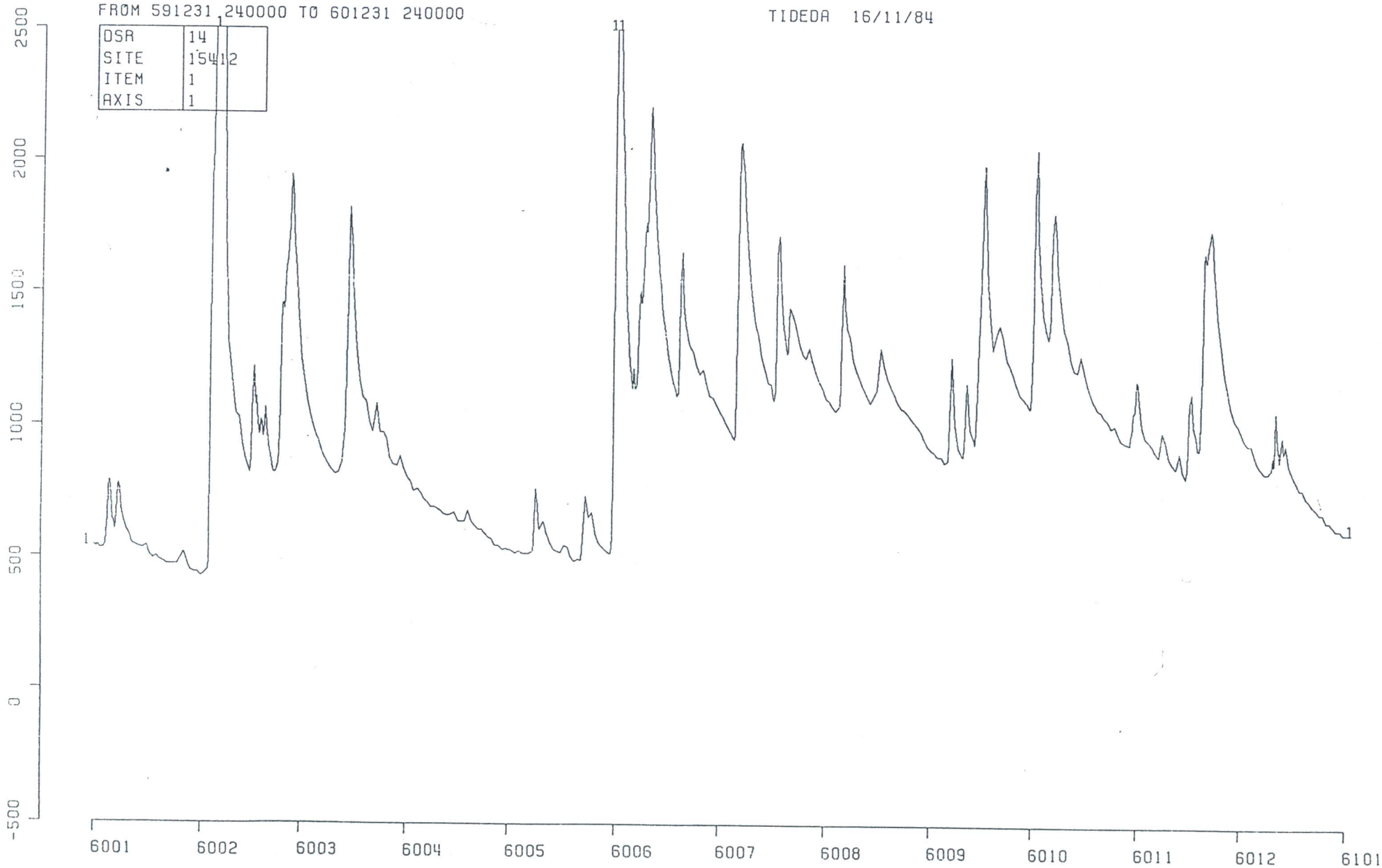
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| DSR | 14 |
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| ITEM | 1 |
| AXIS | 1 |



TE TEK0 STAGE (MM) 1960
FROM 591231 240000 TO 601231 240000

TIDEDR 16/11/84

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|------|-------|
| DSR | 14 |
| SITE | 15412 |
| ITEM | 1 |
| AXIS | 1 |



APPENDIX C

SITE OBSERVATIONS AND DISCUSSION

SITE 1

1.0 LOCATION

Distance 34.5 km [All locations expressed in terms of approximate distance from mouth of Rangitaiki River]

2.0 DESCRIPTION

Right hand bank on outside of bend. Bank is generally stable but two isolated breaks, one of 60m length and another of 30m (Figure C1) were noted. The bank consists of alluvial silts and silty sands overlying cohesionless sands and gravels. The bank ranges in height from 3-3.5m. A depth profile measured adjacent to the sites indicates that depths increase rapidly.

3.0 DISCUSSION

The geomorphic setting of the sites on the outside of a curve in the river and the measured depth profile both suggest that the instability was initiated by erosion of the basal sands and gravels leading to oversteepening and slumping of the overlying bank. At the time of observation the relatively free draining basal sands and gravels extended to 1.0 metre above water level. The water level at the Te Teko site was about 1.1m RL at this time. Therefore, as water level fluctuations associated with power generation rarely exceed 2.0m RL (chapter 3), it is unlikely that drawdown effects associated with power generation will significantly affect erosion at this site.

However, it is possible that erosion at one site is accentuated by eddying and turbulence in the lee of local willow trees.



Figure C1. Site 1.

SITE 2

1.0 LOCATION

Distance 34.1 km.

2.0 DESCRIPTION

Left bank on outside of a particularly sharp bend. A large amount of recent slump debris is evident at the base of the bank. The bank is generally 8-9m high and composed of interbedded pumiceous sands and gravelly sands - appears to be Taupo Pumice material, perhaps redeposited. The major slump (Figure C2) appears to be a recent feature but older erosion scarps are also evident further upstream, suggesting that the site has been actively eroding for some time. The sharp angle of the bend and the slump debris at the base of the bank (Figure C2) results in marked redirection of flow towards the right bank. This has led to some erosion of the right bank immediately downstream. In particular a large slump, of about 20m length, is evident where flow impinges on the 3.5-4m high alluvial bank.

3.0 DISCUSSION

Location of the instability on the outside of a sharp bend suggests basal erosion as the initiatory mechanism. The recent nature of the slump evident in figure 2 suggest that the erosion may also have been aggravated by the recent earthquake. Erosion of the right bank is also clearly related to basal scour - associated with redirection of flow across the channel.



Figure C2. Site 2.

SITE 3

1.0 LOCATION

Distance 33.2 km.

2.0 DESCRIPTION

Left hand bank of 4m height on outside of a bend. Erosion scarp extends over a total length of about 150m and is active (Figure C3). Measurements indicate that depth increases sharply with distance from the bank. The bank is composed of silty sands (about 1.5m thickness) overlying cohesionless gravels - the underlying gravels resting at an angle of about 35°.

3.0 DISCUSSION

The presence of cohesionless materials at the base of the bank, together with location of the site on the outside of a bend with medium to high velocities and evidence of bed scour, suggests basal erosion is the principal reason for bank instability. The presence of highly permeable sands and gravels (ie rapidly draining) up to a height of 2.5m above water level, suggests water level fluctuations associated with station operation are unlikely to significantly aggravate the existing erosion.



Figure C3. Site 3.

SITE 4

1.0 LOCATION

Distance 31.8 km.

2.0 DESCRIPTION

Steep, actively eroding, left hand bank on outside of a bend extending over a distance of 200m (Figure C4). Bank is 10-12m height and is composed of interbedded pumiceous materials which appear to be airfall deposits of Taupo Pumice. These materials overly a relatively impermeable layer, generally within 0-2m of water level at time of visit. This layer appeared to be a weathered ignimbrite surface - which is also consistent with the location of the bank, adjacent to the base of local hills composed of Matahina Ignimbrite. Velocities through the reach are medium to swift and depths increase sharply with distance from the bank.

3.0 DISCUSSION

The location of the instability on the outside of a bend, together with the depth profile measured adjacent to the bank, suggests basal scour is the primary mechanism responsible for bank erosion. An anomalous factor in this explanation is the weathered ignimbrite at the base of the bank which would not be expected to be readily erodible. However, the measured depth profile and the absence of a bench adjacent to the base of the bank, does suggest that the material has been eroded or undermined. Nonetheless, it is probably that this ignimbrite does limit erosion at the site.



Figure C4. Site 4.

SITE 5

1.0 LOCATION

Distance 31.2 km.

2.0 DESCRIPTION

Moderately steep actively eroding left hand bank on outside of bend. Instability extends over a distance of 200m. The bank ranges in height from 2.0-2.2m (Figure C5) and is comprised of interbedded sands and silts overlying fine gravels/coarse sands. The coarse basal layer lies beneath water level over much of the length of the bank. Medium to swift velocities occur along the base of the bank and a measured depth profile indicated that depths increase quite sharply with distance from the base of the bank.

3.0 DISCUSSION

The location of the site on the outside of a meander bend, the evidence of basal erosion in the depth profile and the erodible basal material all suggest that undermining by erosion is the primary reason for the bank instability. As most of the subaerial materials are of relatively low permeability (Figure C5) water level fluctuations could possibly aggravate the erosion by draw-down effects.

However, the sharp increase in depths adjacent to the bank and the absence of a bench suggest that the rate of bank retreat is controlled by basal erosion (see chapter 4) and thus, that draw-down effects do not accentuate the rate of erosion.



Figure C5. Site 5.

SITE 6

1.0 LOCATION

Distance 30.7km

2.0 DESCRIPTION

Steep, actively eroding, right hand bank over a distance of 800m. The bank is high, ranging from 10.0m to 12.0m, and comprised of reworked Taupo Pumice underlain by a relatively impermeable paleosol located above river level (Figure C6). Further coarse cohesionless sands underly the Paleosol (Figure C7). The upstream 250m is partially protected at low to medium flows by incipient point bar development upon which willows are becoming established. River velocities along the downstream end of the reach are medium to swift with moderate depths adjacent to the bank. Several acres of pastoral land have been lost as a result of erosion at this site over recent years (site analysed in detail in chapter 6).

3.0 DISCUSSION

The location of the site on the outside of a bend, the presence of readily erodible cohesionless materials at, or near, river level, and the moderate depths adjacent to the bank all suggest that bank retreat is controlled by basal erosion. Water level fluctuations could induce high seepage pressures immediately above the relatively impermeable paleosol layer accentuating bank instability. However, the absence of a bench above the paleosol suggests that such effects do not increase bank retreat above that rate occurring due to basal erosion.



Figure C6. Site 6



Figure C7. Site 6

SITE 7

1.0 LOCATION

Distance 29 km.

2.0 DESCRIPTION

Right hand bank on outside of bend, with active erosion over a 20m length. Previously active over a further 40m upstream (BOPCC, 1980) but this region is now vegetated and appears to have stabilised. The bank is 2.5-3m in height composed of sandy silts (2m) overlying medium sized gravel material which extends to about 1m above water level (Figure C8). A relatively impermeable paleosol occurs, being just below water level at the time of observation, underlain in turn by cohesionless sands. Depth measurements at the base of the slope indicated a slight bench (about 1m wide) associated with the paleosol, with depths thereafter dropping sharply into the channel. Velocities in the reach are medium to swift.

3.0 DISCUSSION

The location of the site on the outer bank of a meander bend suggests that erosion is primarily responsible for the bank instability. The depth measurements indicating active scour adjacent to the bank provide further confirmation. However, the bench above the paleosol indicates that erosion of the lower bank is to some extent restricted by this feature. Thus, it appears that the rate of bank retreat at the site is controlled by erosive removal of the cohesionless sediments overlying the paleosol. Water level fluctuations lowering the water surface below the top of the paleosol could also aggravate this process by generating seepage pressure (leading to piping) in the sediments overlying the paleosol.

At the time of the field visit to this site, the water level at the Te Teko gauge was at about 1.0m R.L. (gauge datum). As water levels below this stage occur frequently during power generated fluctuations (Appendix B), it is probable that the water surface at site 7 frequently drops below the level of the paleosol. Hence, seepage pressures could be a secondary mechanism affecting the rate of bank retreat at the site.



FIGURE 8 : SITE 7

SITE 8

1.0 LOCATION

Distance 28.5 km.

2.0 DESCRIPTION

Right hand bank in a straight reach, but with notable point bar development on left bank which diverts flow into the bank. Bank eroding over a distance of 100m. The 2.5m high bank is comprised of sandy silts (about 1.2m) underlain by interbedded sands and silty sands. A marked break in slope occurs between these two layers due to the lower containing cohesionless materials (Figure C9). Gravel material underlies these layers (being below water level at the time of the photo shown in figure C9). Velocities in the reach are medium to swift. A depth profile adjacent to the bank indicated that depths were less than those noted at upstream sites. This is probably related to the fact that the point bar development occurs on a relatively straight reach between two meander bends.

3.0 DISCUSSION

The bank retreat is primarily related to the erosion of basal sediments by flow diverted into the right bank by the point bar development. An earlier survey conducted by the BOPCC in 1980 noted active erosion of the upstream left bank and commented that the "bottom end" of this break directed current into the right bank (BOPCC, 1980). It is thus possible that this earlier erosion, which has now stabilised, was responsible for development of the point bar now observed in this relatively straight reach. Water level fluctuations could generate seepage pressure in the sandy layers interbedded with less permeable materials at the base of the bank. However, the absence of any notable benching suggests the rate of bank retreat is primarily controlled by the erosive removal of material from the base of the bank. Thus, there is no evidence that draw-down effects play a significant role at this site under the current operating regime.



Figure C9. Site 8.

SITE 9

1.0 LOCATION

Distance 28.2 km.

2.0 DESCRIPTION

Left bank on outside of bend. Bank previously active over a distance of 100m, but now appears to be stabilised by willow protection placed at the base of the bank (Figure C10). Bank is 7-8m high and composed of pumiceous alluvium. Bank stratigraphy was not able to be determined as bank face covered with debris. A low bench established during placement of the willow protection (Mr D. Roberts, BOPCC, pers. comm) has subsequently been removed by erosion. Medium velocities occur along the base of the bank.

3.0 DISCUSSION

The setting of the bank, located on the outer edge of a meander bend, suggests the bank instability was initiated by basal scour. The basal scour now appears to have been restricted by the willow protection - though it is possible that this protection may be undermined, renewing the erosion, during a major flood. As the bank stratigraphy was not able to be established the influence of water level fluctuations on any future bank erosion cannot be commented on.



Figure C10. Site 9

SITE 10

1.0 LOCATION

Distance 27.8 km.

2.0 DESCRIPTION

Left hand bank on outside of bend. Bank previously active over 200-230m (BOPCC, 1980). However, rip rap rock protection has now been placed over 130m of upstream bank. A small break occurs at the upstream end of the bank protection but the most significant instability occurs at the downstream end where flow cuts into the unprotected bank (Figure C11). The bank is 2.5-3m high and is composed of interlayered sand and silts. Gravel occurs at the base in places but a puggy, dark grey, fine sandy silt was evident at water level in most places (Figure C12). It is possible that this layer is in turn underlain by gravels as these were noted at the base of a profile just upstream. Velocities in the reach are medium to swift.

3.0 DISCUSSION

Flow impinges on the base of the bank and thus basal scour appears the most likely reason for the bank instability. However, as most material above water level at the time of visit (about 1-1.1m RL at Te Teko) was relatively cohesive (either silts or silty sands), it is possible that draw-down effects associated with water level fluctuations above this level could aggravate bank retreat at the site.



FIGURE C11 : SITE 10

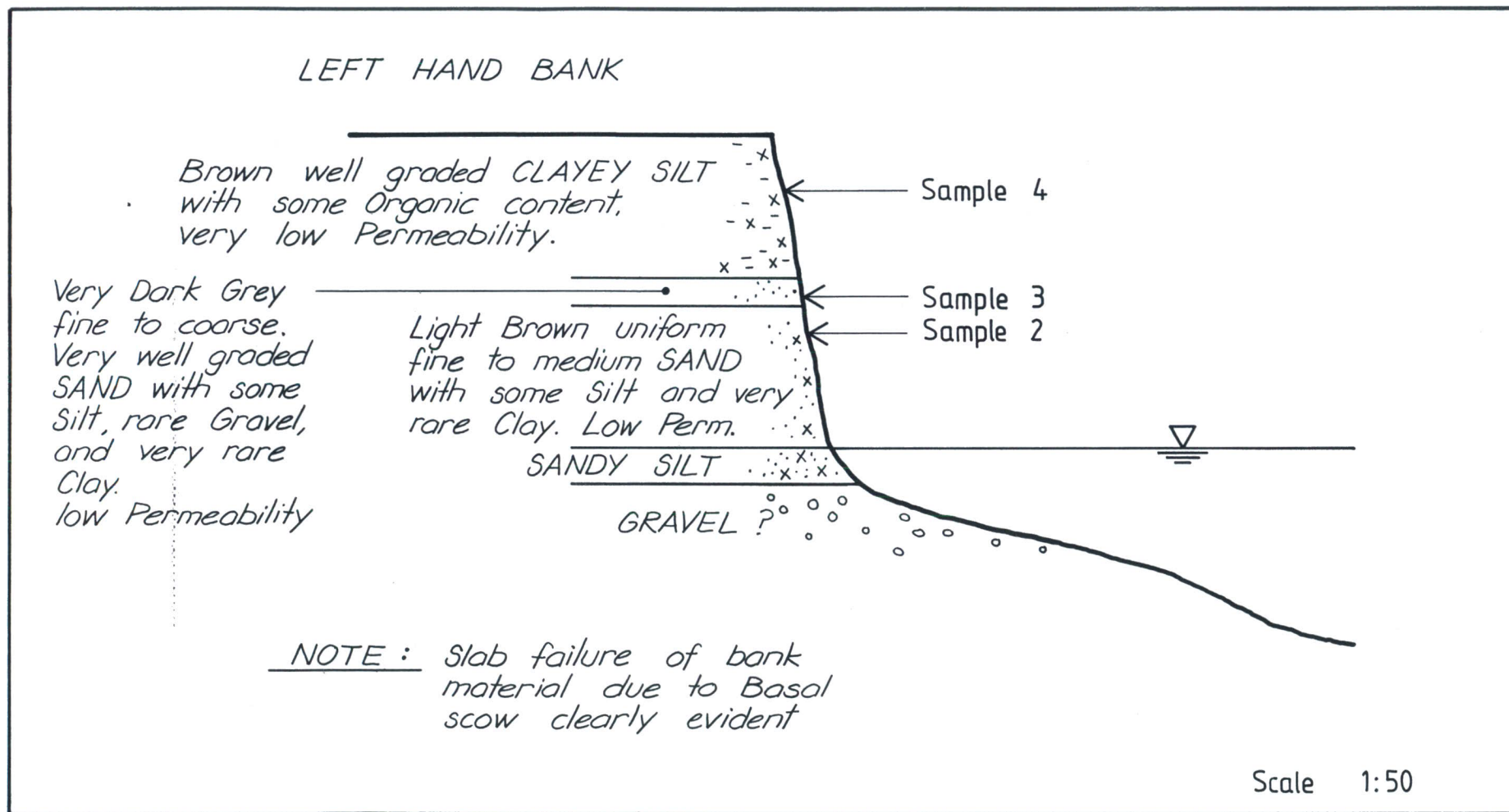


FIGURE C12

Site 10

Bank Stratigraphy

SITE 11

1.0 LOCATION

Distance 27 km.

2.0 DESCRIPTION

Right bank on outside of a bend with medium to swift velocities adjacent to the base of the bank. Erosion extends over a distance of 250m with bank heights ranging from 2-3m. At the upstream end the banks, 3m high, are composed of older material similar in appearance to Taupo Pumice alluvium (Figure C13). In this region of less active erosion a layer of silty sand of 1.0m thickness overlies 2.5m of coarse cohesionless sand. Under this layer is a firmly compacted, moderately indurated, finely stratified silt with very low permeability (evident in figure 13 just above layer with wood debris). This material is underlain by a thin layer of cohesionless sand and a paleosol (evident at water level in figure C13). The major, and more active, region of instability occurs in a bank of 2-2.5m height composed of more recent interbedded, moderately cohesive, silty sands and silts with cohesionless gravelly coarse sands in lower regions (Figure C14). A break in slope is evident in figure 14 between the interbedded sands and silts and the underlying coarser cohesionless materials. Velocities adjacent to the bank are medium to swift.

3.0 DISCUSSION

The location of the bank on the outer edge of meander bend, the swift velocities adjacent to the bank, the readily erodible cohesionless materials at the base of the areas of more active erosion and sharply increasing depths adjacent to the bank all suggest that basal scour is the primary mechanism responsible for bank instability. The presence of free draining gravels throughout most of the lower region of the area of most active erosion (ie, the downstream end of the site) suggests that drawdown effects associated with water level fluctuations are unlikely to play a significant role. The absence of any notable bench further confirms this. Thus, it appears that the rate of bank retreat is controlled by the rate of erosive removal of the basal sediments.



FIGURE C13 : SITE 11



FIGURE C14 : SITE 11

SITE 12

1.0 LOCATION

Distance 26.8 km.

2.0 DESCRIPTION

Left hand bank on outside of bend. Bank has been rip-rapped over a distance of about 100m but there is a 30m zone of bank instability outside the scope of the bank protection (Figure C15). There is also the occasional break in the rip-rap area but overall this protection appears to be performing well. Bank is of 3m height and consists of interlayered sands and silts overlying gravels. Velocities through the reach are swift.

3.0 DISCUSSION

The bank lies on the outside of a bend with swift velocities impinging against the readily erodible cohesionless base. Thus, fluvial entrainment appears to be the primary cause of the bank instability.

SITE 13

1.0 LOCATION

Distance 25.4 km.

2.0 DESCRIPTION

Right hand bank on outside of meander bend. Bank ranges in height from 2.5 to 5.5m (Figures C16 and C17) and the instability extends over a total distance of 300m. The bank is composed of interbedded, generally cohesive, silts and sandy silts overlying, generally cohesionless, sands and silty sands. A break in slope is evident at the boundary between these materials (Figure C17). Velocity adjacency to the bank is medium. There is evidence of a slight bench of 2-2.5m width just below water level suggesting a slightly more cohesive material underlies the sands. Thereafter depth increases sharply suggesting this material is in turn underlain by more readily erodible sediment.

3.0 DISCUSSION

The location of the bank on the outside of a bend with readily erodible materials at and above water level suggests basal erosion is the primary mechanism responsible for bank instability. The bench observed just below water level indicates less readily erodible (and probably less permeable) materials. The sharp increase in depth beyond the narrow bench suggests this material is in turn being removed by undermining due to scour of more readily erodible materials lower in the bank. Water level fluctuations extending below the bench may induce removal of material by piping at the interface between the less permeable bench layer and the overlying sands. Thus, it is possible that draw-down effects could slightly aggravate bank instability at this site. However, more probably, the rate of bank retreat is controlled predominantly by the removal of loose sediments, above the bench, by lateral corrasion.



Figure C16. Site 13.



Figure C17. Site 13.

SITE 14

1.0 LOCATION

Newey's Bend 25.2 km.

2.0 DESCRIPTION

Left hand bank on outside of a meander bend. Erosion extends over a distance of 100m. Some rock protection has been placed at the upstream end of the site and appears to be effective. Bend is sharp and results in the medium to swift velocity flow directly impinging on the eroding bank (Figure C18). Bank is 2.5-3.0m high and consists of relatively cohesive interlayered sands and silts over cohesionless fine-medium sized gravels (Figure C19). The direct impinging of flow against the left bank results in a strong across channel deflection into the right bank just downstream of the bend, resulting in erosion of this bank over a length of 60-70m.

3.0 DISCUSSION

The location of the eroding area together with the field evidence of medium swift velocities impinging on the erodible base of the bank all suggest that basal erosion is the primary mechanism responsible for both initiating, and controlling the rate of, bank retreat. Clear evidence of undermining of the upper bank by erosive removal of lower bank materials is evident in figure C19.



Figure C18. Site 14.



Figure C19. Site 14.

SITE 15

1.0 LOCATION

Distance 24.9 km.

2.0 DESCRIPTION

Right bank on outside of bend-actively eroding over a distance of 30m. Some rock protection has previously been placed immediately upstream of the site, protecting the region commented on in the BOPCC report of 1980. The present break has thus clearly developed since 1980. The bank height ranges from 4-5m and is composed largely of relatively cohesive interbedded sands and silts. Debris at the base of the bank prevented the stratigraphy near and below water level from being ascertained. However, a relatively impermeable pug layer was observed about 1m above water level and it is possible this layer extended below water level (the top of the pug is evident about halfway down the spade shown in figure C20). At the time of the site visit, a perched water table seeped out of the bank through a layer of cohesionless sands above this pug. A number of old tree stumps are exposed in the cutting (Figure C20). Swift velocities are evident against the base of the bank with considerable eddying and turbulence evident. Depths increase sharply adjacent to the bank.

3.0 DISCUSSION

The location of the bank and the adjacent depth profile both suggest basal scour is the primary mechanism initiating and controlling bank retreat at this site. Seepage of ground water above the pug layer may also contribute to the instability but the absence of a bench in this region suggests the effect of basal scour is predominant. There is however some evidence in the photos of the BOPCC report that this seepage influenced bank stability - a slump of the upper bank being evident, suggesting failure above the pug layer.



FIGURE C20 : SITE 15

SITE 16

1.0 LOCATION

Te Teko Gauging Station. Distance 24.5 km.

2.0 DESCRIPTION

Left hand bank on outside of approach to a minor bend. The bank is failing over a distance of 80m. Bank height ranges from 1.5-2m (Figure C21).

Bank stratigraphy consists of alternating layers of cohesive silts and silty sands over relatively cohesionless sands (Figure C22). Depths generally increase quite readily away from the bank though a narrow bench was evident in places. Velocities through the reach are swift. This erosion has developed since the BOPCC report of 1980, though that report did note that continued erosion of the upstream site (our site 15) could result in erosion of this bank.

3.0 DISCUSSION

Benches are evident at the site immediately above the cohesive silt layer (figure C21). This suggests that bank instability is induced by removal of the cohesionless sands undermining the silts. The sand may be removed by either erosion or piping. The location of the site and the relatively swift nearshore velocities suggests that erosion is probably the predominant mechanism. However, draw-down related mechanisms may play a contributory role.



Figure C21. Site 16.

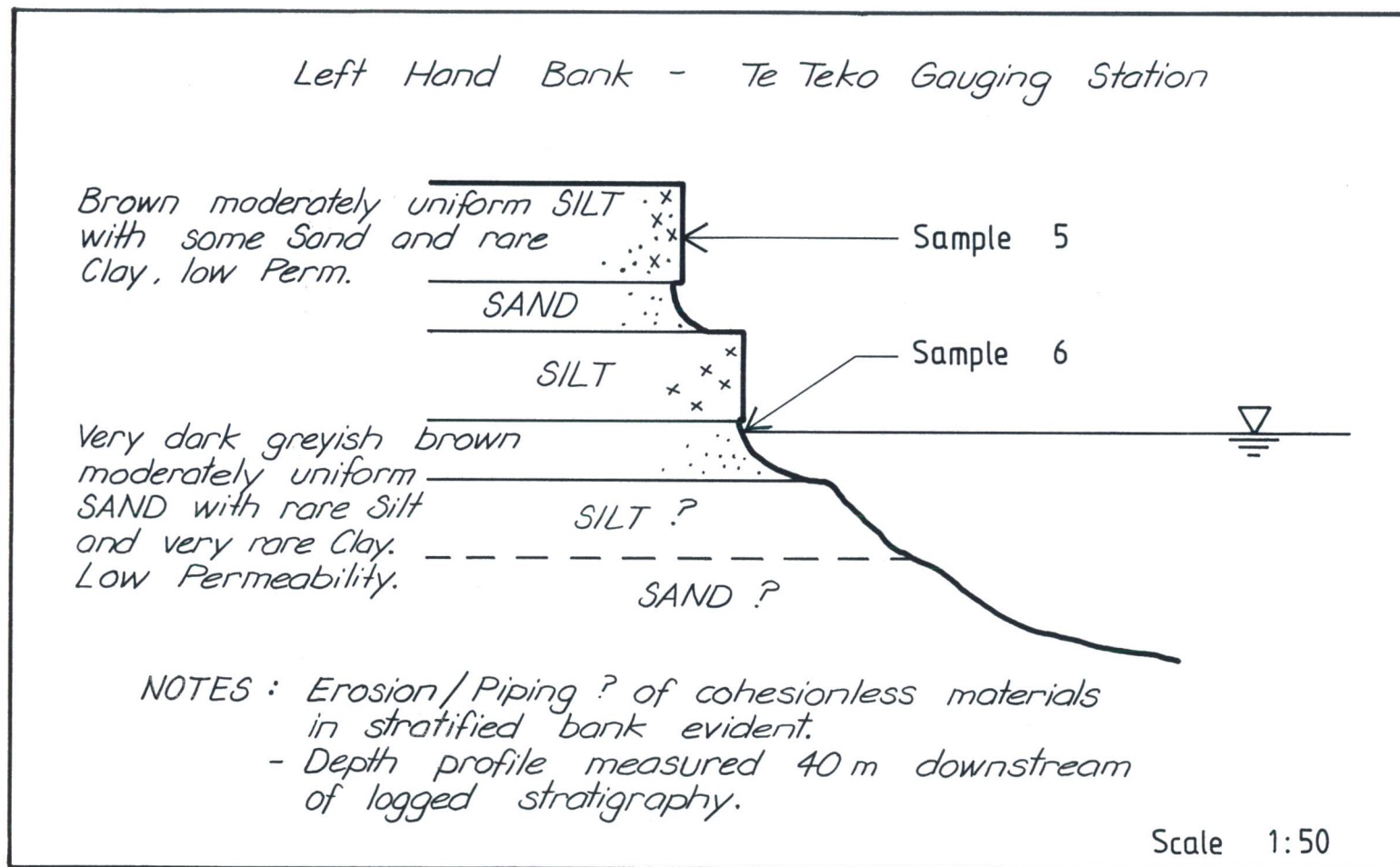


FIGURE C 22

Site 16

Bank Stratigraphy

SITE 17

1.0 LOCATION

Distance 23.6 km.

2.0 DESCRIPTION

Left bank on outside of a meander loop. Bank is actively eroding over a distance of 200m. Bank height ranges from 1.5-2.5m and the bank is composed of relatively cohesive sandy silts (about 1.2m thickness) overlying interbedded, and generally cohesionless, sands and gravelly coarse sands. A sharp break in slope is evident between the silts and the underlying cohesionless material (Figure C23). Velocities through the reach are medium and depths increase markedly adjacent to the bank.

3.0 DISCUSSION

The location of the site, the depth profile adjacent to the bank and the presence of erodible cohesionless materials at the base all suggest that erosive removal of the basal sands is the primary cause of bank instability. A bench previously cut at the base of this bank associated with an attempt at bank protection has also been eroded away (D Roberts, BOPCC, pers. comm.) further supporting the contention that basal erosion is the primary mechanism. Removal of the underlying sands leading to undermining and block failure of the overlying silts is also evident in figure C23.



Figure C23. Site 17.

SITE 18

1.0 LOCATION

Distance 17.1 km.

2.0 DESCRIPTION

Extensive slumping (Figure C24) along left hand side bank on inside of a meander bend. Instability occurs right around the inside of the bend a distance of about 300m, and merges with left hand bank erosion at site 19. Bank heights range from 1.5-2.5m. The bank is composed of relatively cohesive interbedded sands and silts overlying cohesionless coarse pumiceous sands (Figure C25). Velocities are medium at the upstream end of the reach but increase to be relatively swift at the downstream end. The fault scarp of 2 March 1987 earthquake lies only 500-600m downstream and velocities have almost certainly increased over the entire reach since this event.

3.0 DISCUSSION

Several factors appear to play a role at this site. Over much of the upstream region the damage appears to be primarily related to the recent earthquake with numerous tension cracks evident in this area (Figure C26). However the more notable erosion scarp at the downstream end (Figure C24) appears also to be related to basal erosion. Flow velocities are swift in this region and the cohesionless materials in the lower bank (Figure C25) are readily erodible. In addition a tendency for flow to be deflected across the channel towards the left bank is evident in aerial photographs taken on 3.7.87. Turbulence in the lee of the numerous tree stumps in this region may also act to aggravate bank stability. These stumps also divert flow towards the banks.

The presence of a relatively coarse free draining basal layer up to 0.6m above water level at the time of our observation (at the same time water levels at the Te Teko gauge were 1-1.1m R.L.) suggests that draw down effects associated with power generated fluctuations are unlikely to a significant influence at this site.



FIGURE C24 : SITE 18



FIGURE C25 : BANK STRATIGRAPHY AT SITE 18



FIGURE C26 : SITE 18

SITE 19

1.0 LOCATION

Distance 16.5 km (Zinks).

2.0 DESCRIPTION

Isolated areas of bank instability on both right and left hand banks in a straight reach of about 500m length. The reach is intersected by the fault scarp of the March 1987 earthquake at the downstream end - with a drop in level of about 2 m (BOPCC, 1987) having occurred on the downstream side of the fault. The most extensive erosion occurs in the vicinity of the fault, particularly over a length of about 100m on the right bank where previous rock protection has been undermined (Figure C27). Slumping is also evident over 30-40m of the left bank in this region (Figure C28). Breaks in rock protection are also evident in isolated areas over the entire 500m reach, particularly at the upstream end of the left bank. Banks are generally 1.5-3m in height with extensive areas of rip rap placed subsequent to the 1980 BOPCC report. Velocities through the reach are very swift, particularly at the downstream end in the vicinity of the fault scarp. The drop in level on the downstream side of the fault has significantly increased the hydraulic gradient, and hence the velocities, in this region - with a small waterfall actually being evident immediately after the quake (Mr D Roberts, BOPCC, pers. comm.). An extensive number of tree stumps from a previously buried forest occur in the reach. Dating of the cambium layer from one of this stumps, undertaken by the University of Waikato Radiocarbon Laboratory, indicated that the material was probably originally buried about 1800 BP. (Dr A G Hogg, Director, Radiocarbon Laboratory, pers. comm.).

3.0 DISCUSSION

A number of factors are apparently involved in initiating bank instability in this region. The site was noted as undergoing erosion in the BOPCC report of 1980, with the most notable erosion at that time also occurring on the downstream right bank. It appears that flow impinged on the right bank in this region at that time. Thus basal scour was probably the primary mechanism. However rock protection was subsequently placed and the present problems appear to have originated largely as a consequence of the recent earthquake. This has resulted in increased velocities adjacent to the banks, particularly at the downstream end. Bed degradation has occurred as a consequence of the increased velocities, undermining the existing rock protection. This latter effect is particularly evident in figure C28 where recently undermined rock protection, at the point of incipient failure, is arrowed. Erosion, particularly as a consequence of the increased velocities and turbulence associated with the numerous stumps in this region can be expected to aggravate the erosion of any exposed banks.



Figure C27. Site 19.



Figure C28. Site 19.

SITE 20

1.0 LOCATION

Distance 15.7 km.

2.0 DESCRIPTION

Right hand bank on inside of bend. The instability extends over a distance of 100m. The bank is composed of sands containing some silt in the upper regions (Figure C29). Lower bank materials were unable to be ascertained due to the presence of recent slump debris. However the BOPCC report of 1980 noted a "heavier silt" at the toe. Bank heights range from 2-2.5m with a platform, up to 5m width, evident just below water level (Figure C30). Water velocities through the reach are slow and have almost certainly been decreased by the lowering of this region in the recent earthquake.

3.0 DISCUSSION

Erosion has occurred at this site for some time - being reported in the 1980 BOPCC report. In addition both old and new scarps are evident at the site (Figure C31). However the presence of numerous recent tension cracks along the top of the erosion scarps (Figure C30) suggests that the extensive recent scarps could be the result of seismic damage. The presence of large slump deposits, remaining at the foot of even old erosion scarps (Figure C31), suggests that basal erosion is relatively insignificant at the site. This is consistent with the location on the inside of the bend and the presence of a wide shallow platform at the base of the bank. As available evidence indicates that the bank is largely composed of relatively impermeable materials, particularly near the base, it is probable that the original erosion was largely initiated by weakening processes - possibly draw-down effects. However as the slump deposits are not rapidly removed, hence providing some toe support, it is unlikely that bank retreat was ever rapid. Photographic evidence contained in the BOPCC report of 1981 provides some confirmation for this view indicating almost complete vegetation (grasses) cover of the bank.

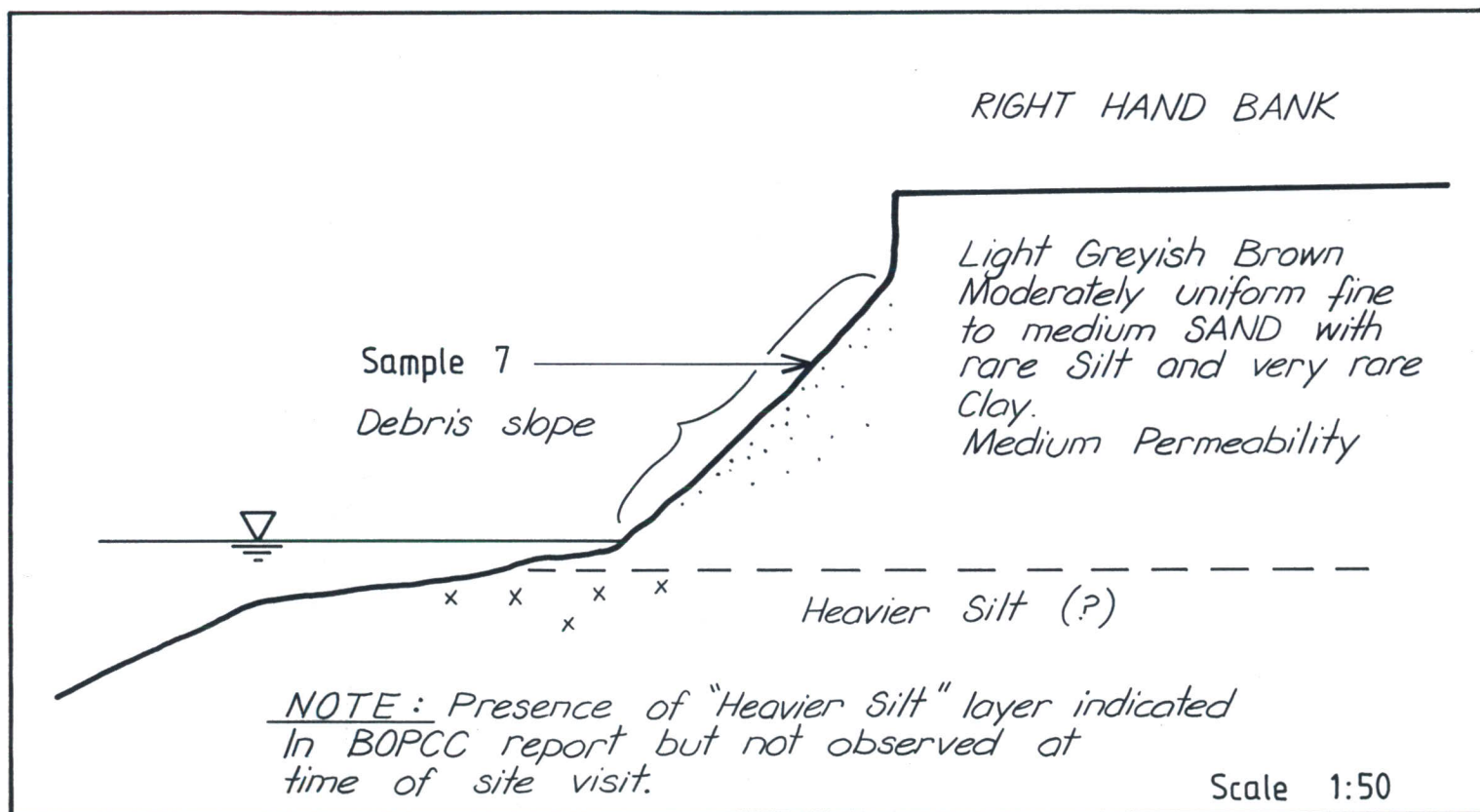


FIGURE C29

Site 20 Bank Stratigraphy



Figure C30. Site 20.



Figure C31. Site 20.

SITE 21

1.0 LOCATION

Distance 14.5 km.

2.0 DESCRIPTION

Left hand bank, 2-2.5m high, on outside of bend. A series of active and previously active slumps are evident (Figure C32) over a distance of about 200m. The bank is composed of moderately cohesive sands (1.0m) overlying, largely cohesionless, interbedded sands and silty sands (0.7m) with a layer of peat (about 0.3m thick) near water level (water level was 1.1m RL at Te Teko at the time of observation). A sharp break in bank slope is evident between the upper layer and the underlying, largely cohesionless, sands (Figure C32). Velocities through the reach are medium. A measured depth profile indicated that depths increase relatively sharply adjacent to the bank.

3.0 DISCUSSION

The location of the bank on the outside of a bend approach together with the measured depth profile suggests basal scour is the primary reason for bank instability. Although the peat layer would be relatively resistant to erosion active undermining of this layer was evident - with large slabs of peat noted lying on the channel bottom. Thus, clearly more erodible materials underlay this layer.

The restriction to drainage through the lower profile provided by the peat suggests that the erosion could be aggravated by water level fluctuations. In particular seepage out of the bank above the peat following draw down could remove sandy material from this area - undermining the overlying bank. However if this mechanism was very significant then the overlying bank would retreat more rapidly than the area of bank incorporating the peat and underlying materials. This did not appear to be the case in the field with only a relatively narrow (generally less than 1m) peat shelf being evident. Hence it appears that the rate of bank retreat is controlled by the basal erosion.



Figure C32. Site 21.

SITE 22

1.0 LOCATION

Distance 13.8 km.

2.0 DESCRIPTION

Right hand bank on approach to, and outside of, bend. Isolated patches of minor slumping are evident over a distance of at least 700m (Figure C33). Bank is typically 1.5-3m in height, with height generally decreasing from the upstream end. Bank appears to be less active than evident in photos taken during BOPCC inspection of 1980. However part of the more active region evident in these earlier photos has subsequently been rip rapped (Figure C34). Bank materials were not examined in detail but were described in the earlier BOPCC report as a fairly cohesive light sandy loam. Velocities through the reach are low to medium.

3.0 DISCUSSION

The location, on the outside of a bend, suggest that basal scour could be the primary mechanism for bank instability. In particular this appears to be the case for the instability towards the upstream end of the reach. However the isolated breaks towards the downstream end (Figure C33) could be related to other factors, particularly the recent earthquake. Historically the site has only had limited activity, the BOPCC report of 1980 noting only limited stress on this section. With the reduction in river gradient effected by the recent earthquake it is possible that any basal erosion will be even more limited in the future.

The BOPCC report of 1980 speculated that some recent breaks observed at that time were attributable to the operating regime of September/October 1980. However, they also noted that aggravation of the erosion by stock appeared to be severe. Hence at least three mechanisms - basal scour, stock damage and possible draw down effects - have been implicated at this site. As the rate of erosion has never been severe it is difficult to isolate the relative contribution of each mechanism historically. However, as noted above, any effect of basal erosion could be significantly reduced in the future. Hence water level fluctuations (due to tidal effects or power generation) could be the more significant factors in any future erosion. However there is presently no evidence to suggest that significant erosion can be expected.



Figure C33. Site 22.



Figure C34. Site 22.

SITE 23

1.0 LOCATION

Distance 13.1 km.

2.0 DESCRIPTION

Left bank on the outside of a relatively sharp bend. Instability occurs over a length of 150m (Figure C35), much of the damage directly affecting the stopbank (Figure C36). This area has been significantly affected by the March 1987 earthquake, with water level now apparently about 1.0m higher relative to the banks (Mr D Roberts, BOPCC, pers. comm). Consequently some areas of previous rock protection are partially, and in some cases completely, submerged. Bank materials were not logged but appear to be relatively cohesive silty materials, though cohesionless materials may exist lower in the bank (ie, now submerged). Water level measurements surveyed by the BOPCC in June 1987 indicate a very low water level gradient in this region - 1:27157 between 1.505km and 15.74km (BOPCC plan R595). Consequently velocities in the reach are now relatively low.

3.0 DISCUSSION

The location of the site on the outer bank of a meander bend suggests that basal scour has historically been the principal reason for bank instability.

A midstream island previously existing at the site, but removed subsequent to 1980, may also have contributed by directing flow into the bank. The recent change in regime is likely to significantly reduce the importance of basal erosion in the future - though some uncompacted fill (reinstated earthquake damage) was removed in a recent small fresh (Figure C36).

The BOPCC report of 1980 noted that the operating regime of September/October 1980 resulted in active block slumping at this site. Photos taken at that time indicate banks of relatively less permeable silty materials (possibly silty sands), which could have been susceptible to drawdown effects. The failure types evident in these photos also suggest, not conclusively, that draw down effects may have aggravated erosion at the site.

As the recent earthquake has increased water level relative to the banks - water level fluctuations now influence materials higher in the bank profile. As such materials tend typically to be less permeable materials (having a higher percentage of silts and fine sands than lower bank materials) the bank may be more susceptible to draw down effects in the future.



Figure C35. Site 23.



Figure C36. Site 23.

SITE 24

1.0 LOCATION

Distance 9.3 km.

2.0 DESCRIPTION

Right bank on outside of bend. Bank is presently active over a distance of about 60m (Figure C37). Rock protection has been placed over a further distance of 100m immediately upstream. The bank is composed of relatively cohesive silts and silty sands with coarse cohesionless sands at the base of the 2-2.8m high bank (Figure C38). Measurements indicate that depth increases sharply adjacent to the base of the bank. Velocities through the reach are fairly low. The reach is now subject to tidal influence but tidal water level fluctuations are generally minimal (200-250mm, BOPCC Plan R595).

3.0 DISCUSSION

The location of the site on the outside bank of a bend in the river together with the measured depth profile suggests basal scour has been the primary mechanism initiating bank instability. The presence of coarse cohesionless sands at the base of the bank is also consistent with this conclusion. As velocities through the reach are now significantly decreased, as a consequence of the March 1987 earthquake, basal scour may be less marked in the future.

Although water level has risen relative to the banks due to the March earthquake, coarse relatively free draining sediments were evident up to 0.8m above water level (1.1m RL at Te Teko at the time (near high tide) of the site visit (Figure C38). Consequently typical power generated water level fluctuations, which generally do not exceed 1.8m RL at Te Teko (chapter 3), are unlikely to saturate the lower permeability material higher in the bank. Therefore draw-down effects associated with power station operation are not normally likely to significantly influence bank stability. Water level fluctuations associated with flood flows may however have some influence.



FIGURE C37 : SITE 24



FIGURE C38 : SITE 24

SITE 25

1.0 LOCATION

Distance 8.6 km.

2.0 DESCRIPTION

Right bank on outside of a meander bend. Bank previously active over a considerable length but now extensively protected by rip rap. However isolated breaks occur, particularly upstream of the rip rap (Figure C39). The bank is composed of reasonably consolidated silts and silty sands. Reach is now tidal with tidal fluctuations typically about 300mm (BOPCC plan R595).

3.0 DISCUSSION

The location of the bank and the nature of slump faces evident in earlier photographs (BOPCC report of 1980) suggests the bank instability was originally initiated by basal scour. The decrease in river gradient following the March 1987 earthquake suggests that basal scour could be less important in the future. However given the relatively impermeable nature of the bank sediments it is also possible that water level fluctuations influence bank stability, although the relative roles of power and tidally generated fluctuations would be difficult to assess. If basal scour is significantly reduced in the future then any slump deposits due to draw down effects will probably tend to accumulate and stabilise the bank.



Figure C39. Site 25.

SITE 26

1.0 LOCATION

Distance 7.0 km.

2.0 DESCRIPTION

Left hand bank on what appears to be a straight reach - though aerial photographs suggest that flow does impinge on this bank in the vicinity of the site. The bank is active over a reach of about 100m (Figure C40). The bank is about 1.5m high and composed of relatively cohesive sandy silts. The reach is now tidal with tidal water level fluctuations typically about 0.5m (BOPCC plan R595). Velocities are tide dependent, but generally very low, during normal flows.

3.0 DISCUSSION

The aerial photographic evidence noted above suggests that basal scour may have played a role in initiating the instability. However the nature of the bank materials is such that water level fluctuations may also have been important. The recent nature of much of the slumping evident in figure C40 may indicate that the March 1987 earthquake has also aggravated the problem.



Figure C40. Site 26.

SITE 27

1.0 LOCATION

Distance 4.5 km.

2.0 DESCRIPTION

Right hand bank on inside of bend and along straight reach - extending over a distance of about 1km. The damage occurs largely along the stopbank and the exposed bank materials are largely relatively cohesive silts and silty sands. The reach is tidal with water level fluctuations normally about 600mm (BOPCC Plan R595). Velocities are tidally dependent and generally low at normal flows.

3.0 DISCUSSION

Damage may be entirely attributable to stock movement on the steep stopbank face in combination with saturated loading. However the site could also be a draw-down failure - the debris of which has remained at the base of the bank and is now vegetated (Figure C41). The persistence of slump materials at the base of the bank also suggests that basal erosion is not normally significant. These slump deposits also act to stabilise the toe of the bank and restrict further instability. Unless these deposits are eroded or undermined by flood scour then further bank retreat is unlikely. Nonetheless, the location of the damage on a stopbank necessitates all possible action to restrict further instability and stock, which have clearly contributed to the damage, should be fenced out of this region.

If the evident damage was initiated by draw-down related processes then water level fluctuations associated with power generation, tidal action and/or floods could have been the initiating process.

Given the location of the site on the inside of a bend and the persistence of the apparent slump deposits (Figure C41) it does not seem that basal scour is an important mechanism at the site. Nonetheless, this mechanism cannot be entirely ruled out on the basis of the existing data. For instance, the evident damage might be a part rotational slump initiated by a combination of flood scour in the channel and saturated loading of the bank. Without more detailed study it would be difficult to confidently assess the mechanisms primarily responsible for the instability noted in this reach.



Figure C41. Site 27.

SITE 28

1.0 LOCATION

Distance 2.4 km.

2.0 DESCRIPTION

Left hand bank on outside of curve in the river. Damage extends over a distance of about 400m. The bank ranges in height from 0.5-2m, in some places being cut into the stopbank and in other places the low terrace fronting the stopbank (Figure C42). The low terrace is composed of fine cohesive materials (Figures C43 and C44) while the stopbank is composed of coarser and more friable materials (Figure C45). The reach is tidal and velocities are generally low at normal flows.

3.0 DISCUSSION

The location of the site on the outside of a bend suggests basal erosion at high flows could be an important mechanism. However, velocities are such that basal erosion is not likely to be significant at normal flows. The low terrace could also be influenced by draw-down related effects given the relatively puggy and impermeable materials composing this feature. It would be expected that this bank is regularly saturated over its full height by water level fluctuations, as tidal fluctuations alone generally exceed 1.0m at this site (BOPCC Plan R595). Moreover the materials of this bank appear to be relatively resistant to erosion. Clearly, however, unless the slump deposits were removed by erosion the bank would probably soon stabilise.



Figure C42. Site 28.

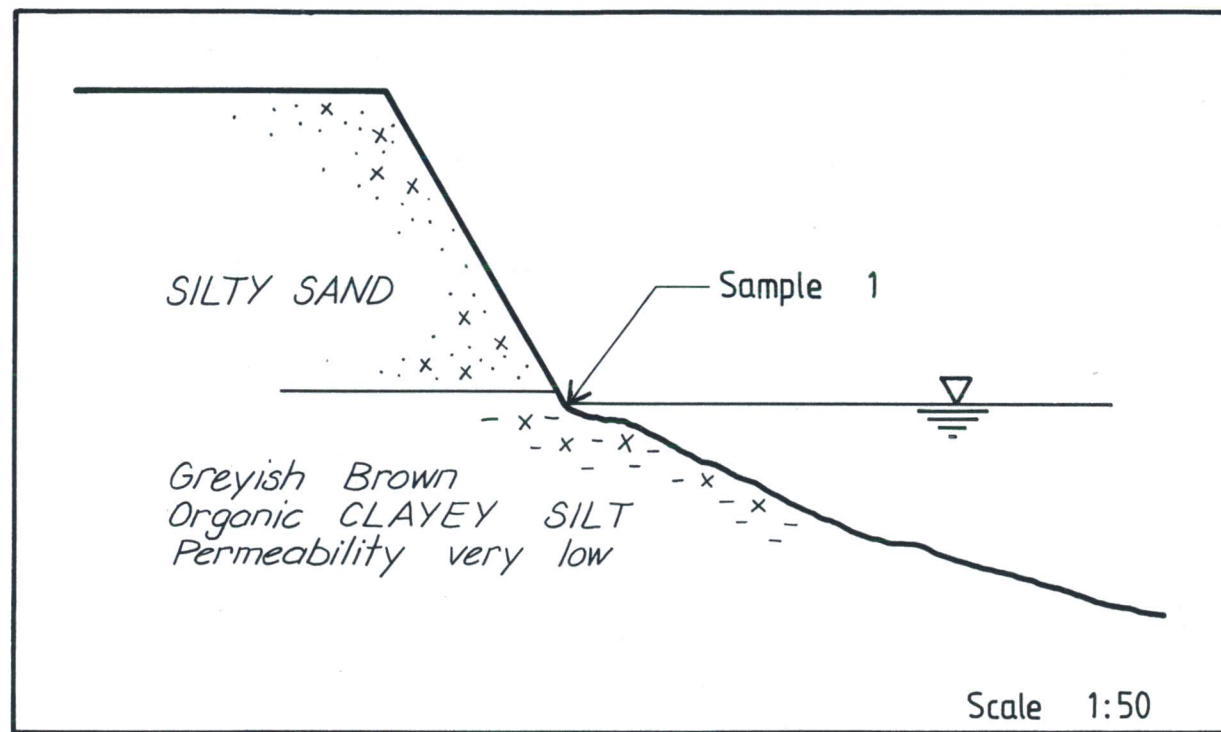


FIGURE 43

Site 28

Bank Stratigraphy



FIGURE C44 : SITE 28



FIGURE C45 : SITE 28

SITE 29

1.0 LOCATION

Distance < 1.8 km.

2.0 DESCRIPTION

Left and right hand side banks between Thornton Bridge and the mouth of the river. Bank is generally 0.5-1.0m in height (Figure C46) and composed of relatively cohesive silts and silty sands with cohesionless sands also evident in the lower profile at many sites (Figure C46). A platform is evident in front of the bank over most of the region at low tide (Figure C46). The reach is tidal with water level fluctuations normally 1.2-1.3m (BOPCC plan R595).

3.0 DISCUSSION

A variety of mechanisms could be active in this reach. As the bank is regularly saturated by tidal fluctuations, which are of similar scale to the bank height, draw-down effects could conceivably play an important role. To some extent this is supported by the fact that erosion is common along both banks throughout this region. Wave lap erosion could also be important, particularly with winds from a northeast or southwest direction which would blow directly along the 2km reach. Such small wind generated waves lapping against the bank could be expected to be particularly effective in regions where the bank is underlain by cohesionless sands. Current erosion could also play a role during freshes and/or floods. The fact that debris does not simply accumulate at the base of the bank, stabilising the region, suggests removal of this material by river flows. Nonetheless, the clumps of slump debris over the platform fronting the banks (Figure C46) does suggest that current erosion is minimal at normal flows.



Figure C46. Site 29.

APPENDIX D

GRADING CURVES AND TRIAXIAL TEST RESULTS

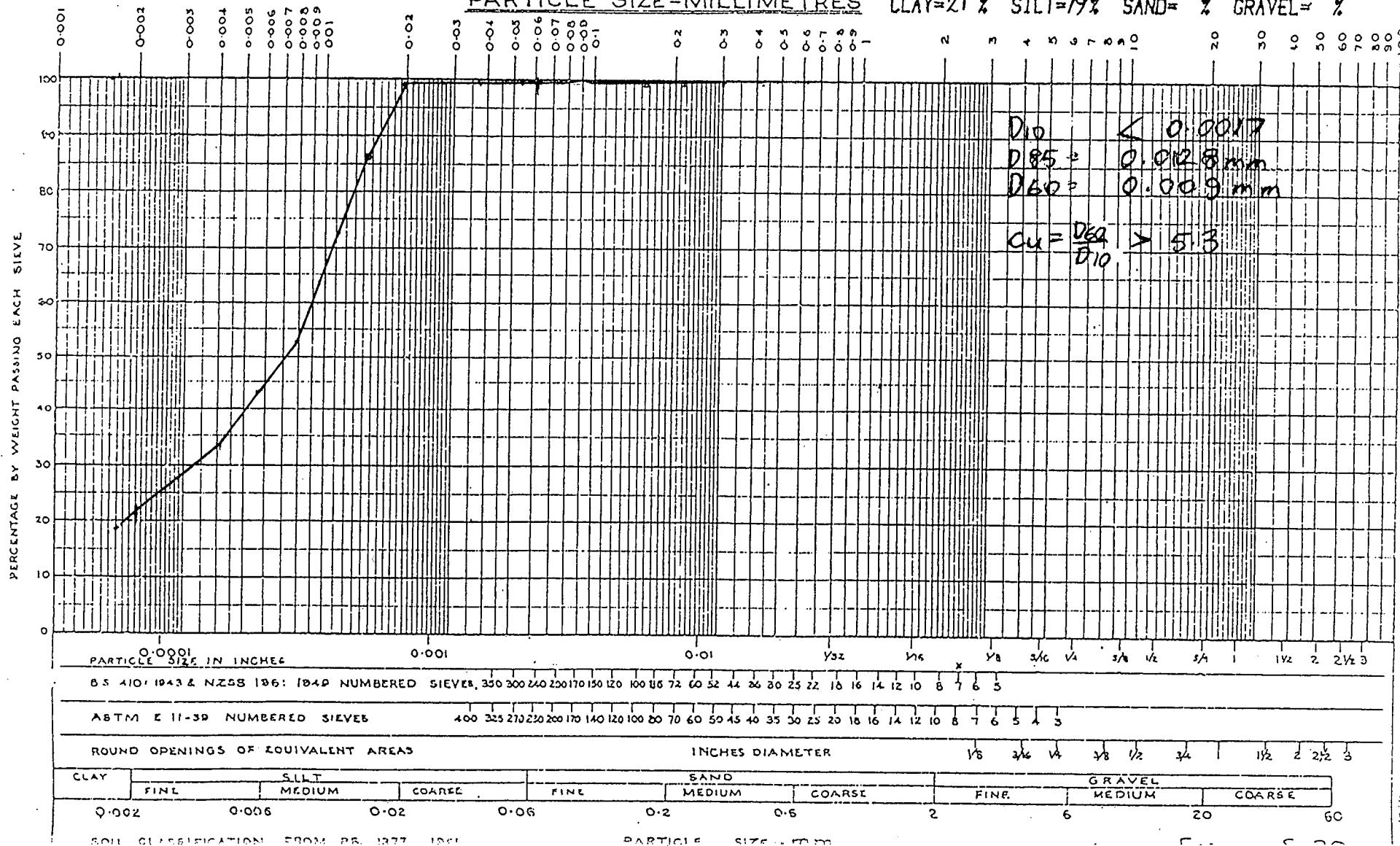
Job No. 87/315

Sample No.1.....

Tested by: S.W.LIM Date: 9/87

Checked by: M.R.E. Date: 1/88

PARTICLE SIZE-MILLIMETRES CLAY=21% SILT=79% SAND= % GRAVEL= %



History of Sample Natural/~~Air Dried~~/~~Oven Dried~~/
~~Unknown~~.

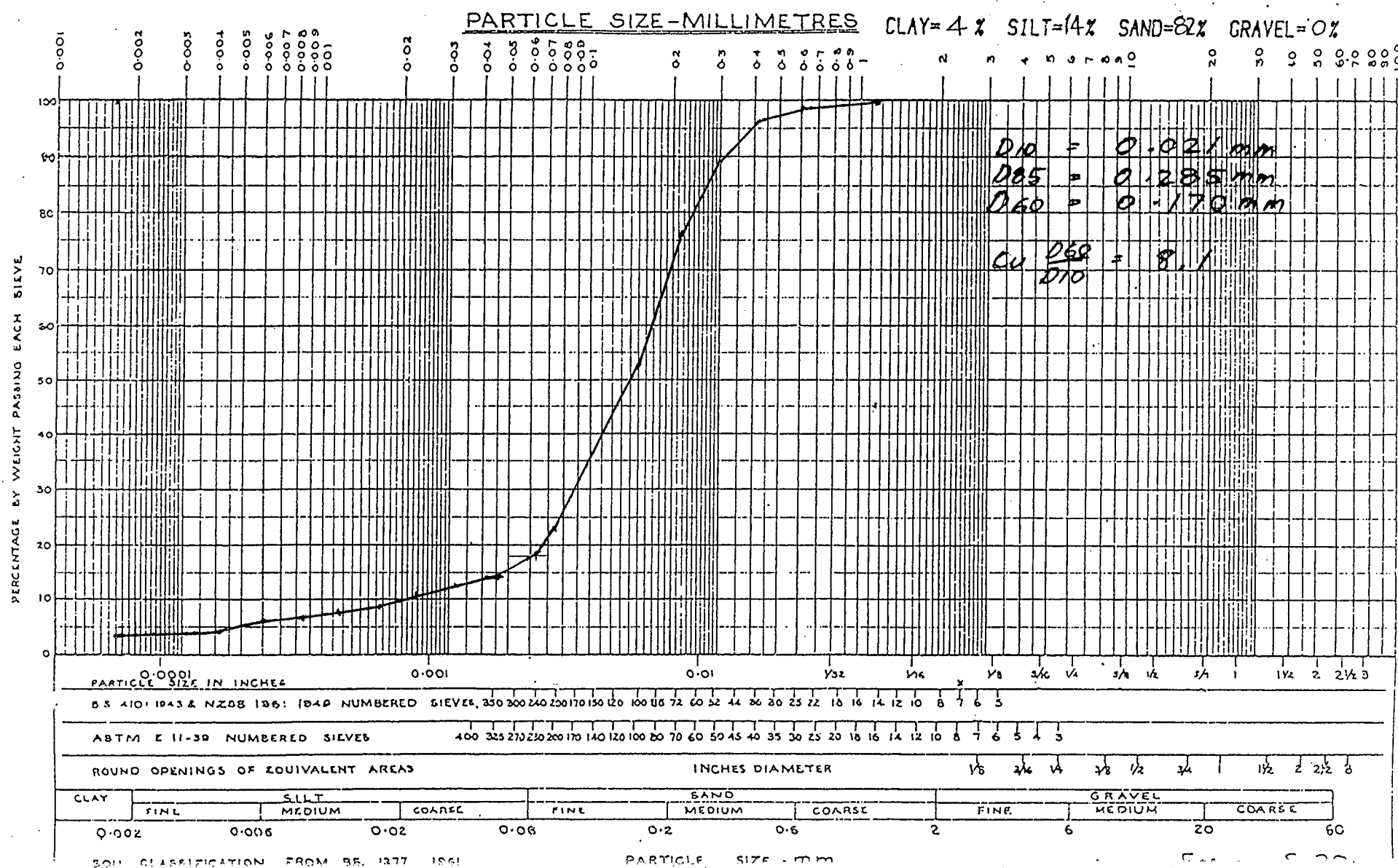
ph of Suspension'

Dispersant used. Sodium Hexametaphosphate/....

Tested by: S.W.LIM Date 9/87

Checked by: Jw..... Date: 12/11/11

10'-SAND 1.0m



Job ... RANGITAHI RIVER ...

History of Sample Natural/Air Dried/Oven Dried/
Unknown.

Location ... SITE 10 ...

ph of Suspension ...

Bore ... Depth ...

Dispersant used. Sodium Hexametaphosphate/....

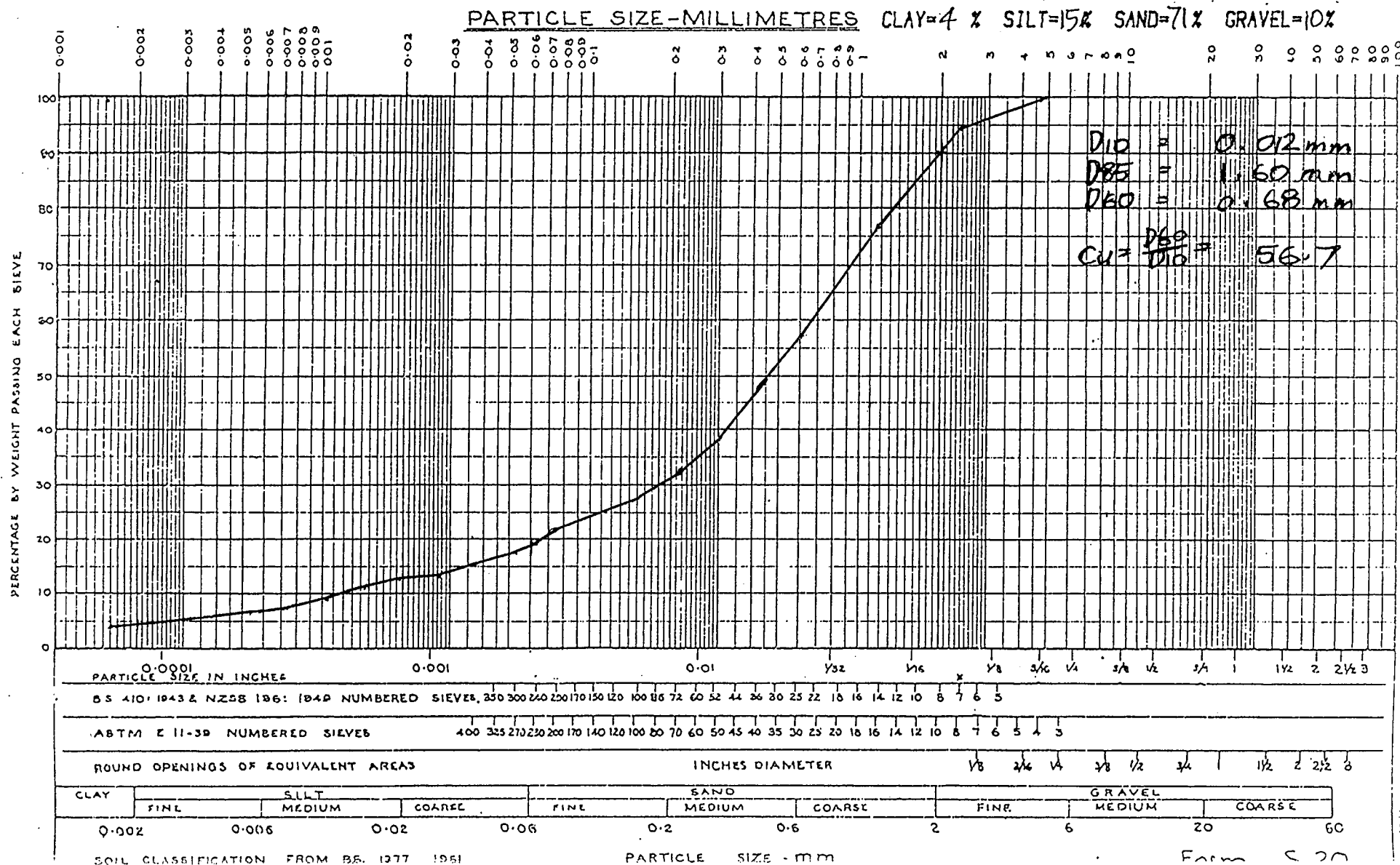
Job No. 87/315

Sample No. 3

Tested by: S.W. LIM Date: 9/87

Checked by: M.R.E. Date: 1/88

10' - BLACK SAND 1.2m





Job ... RANGITAHI RIVER ...

Location ... SITE 10 ...

Bore ... Depth ...

History of Sample Natural/~~Air Dried~~/~~Oven Dried~~/
~~Unknown~~.

ph of Suspension ...

Dispersant used. Sodium Hexametaphosphate/....

Job No. 87/315

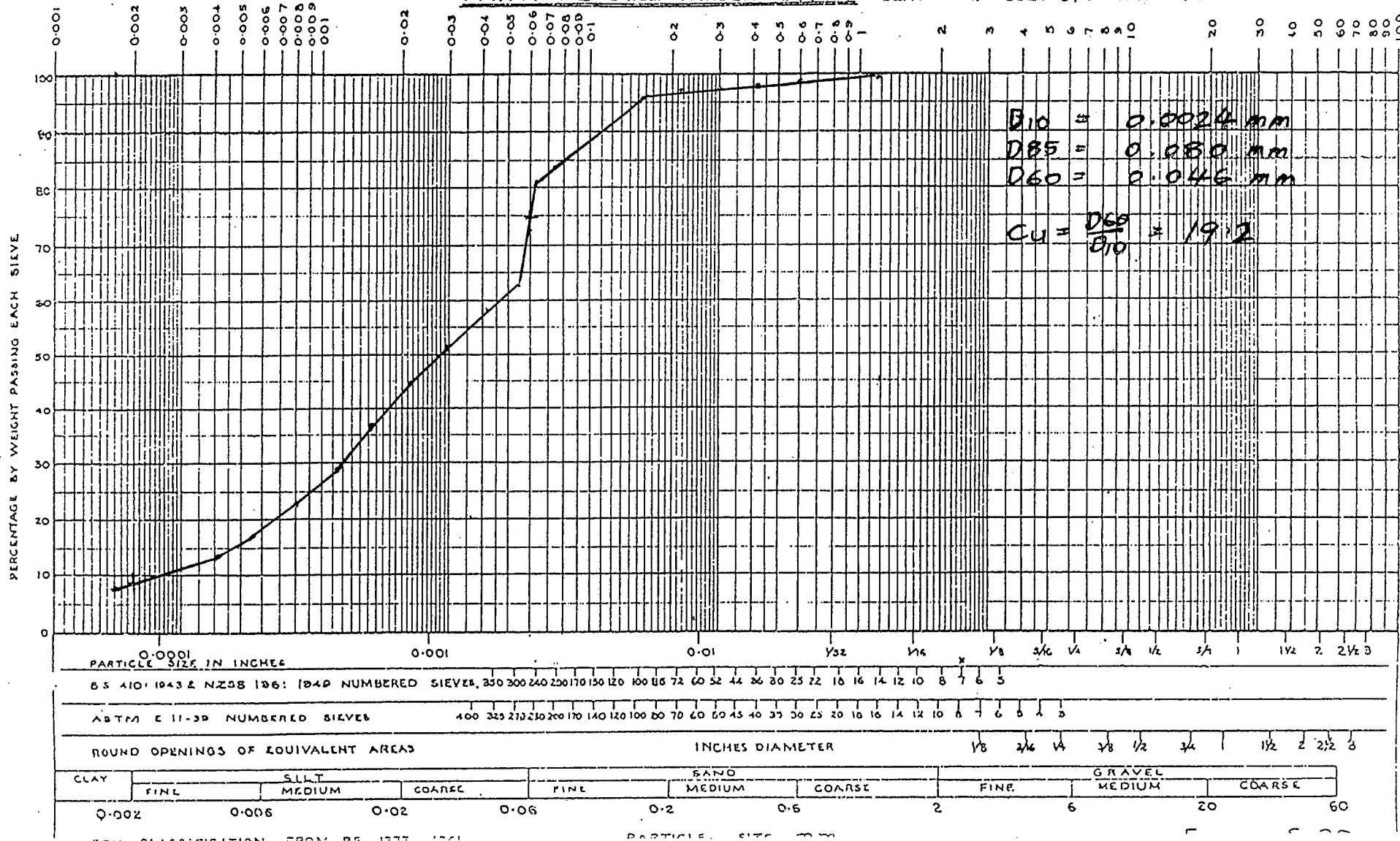
Sample No. 4

Tested by: S.W. LIM Date: 9/87

Checked by: Jw Date: 9/87

10' SANDY SILT 2m (WATER LEVEL)

PARTICLE SIZE-MILLIMETRES CLAY=8% SILT=67% SAND=25% GRAVEL=0%



Job ... RANGITAIKI RIVER ...

History of Sample Natural / ~~Air Dried~~ / ~~Oven Dried~~ / ~~Unknown~~.

Location ... SITE 16 ...

ph of Suspension

Bore Depth

Dispersant used. Sodium Hexametaphosphate /

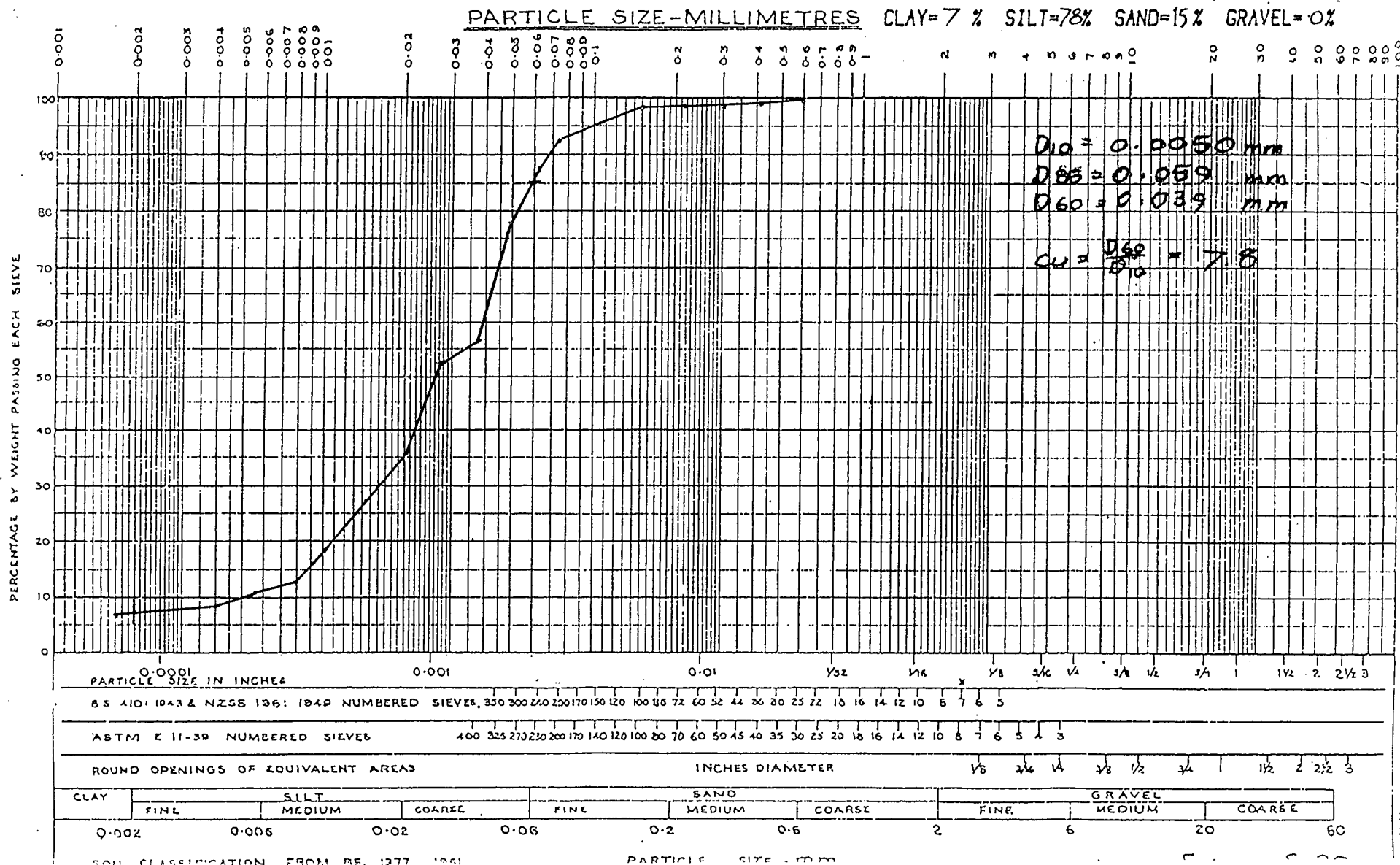
Job No. 87/315

Sample No. 5

Tested by: S.W. LIM Date: 9/87

Checked by: J.W. Date: 9/87

16' SILT 1.4m ABOVE WATER LEVEL



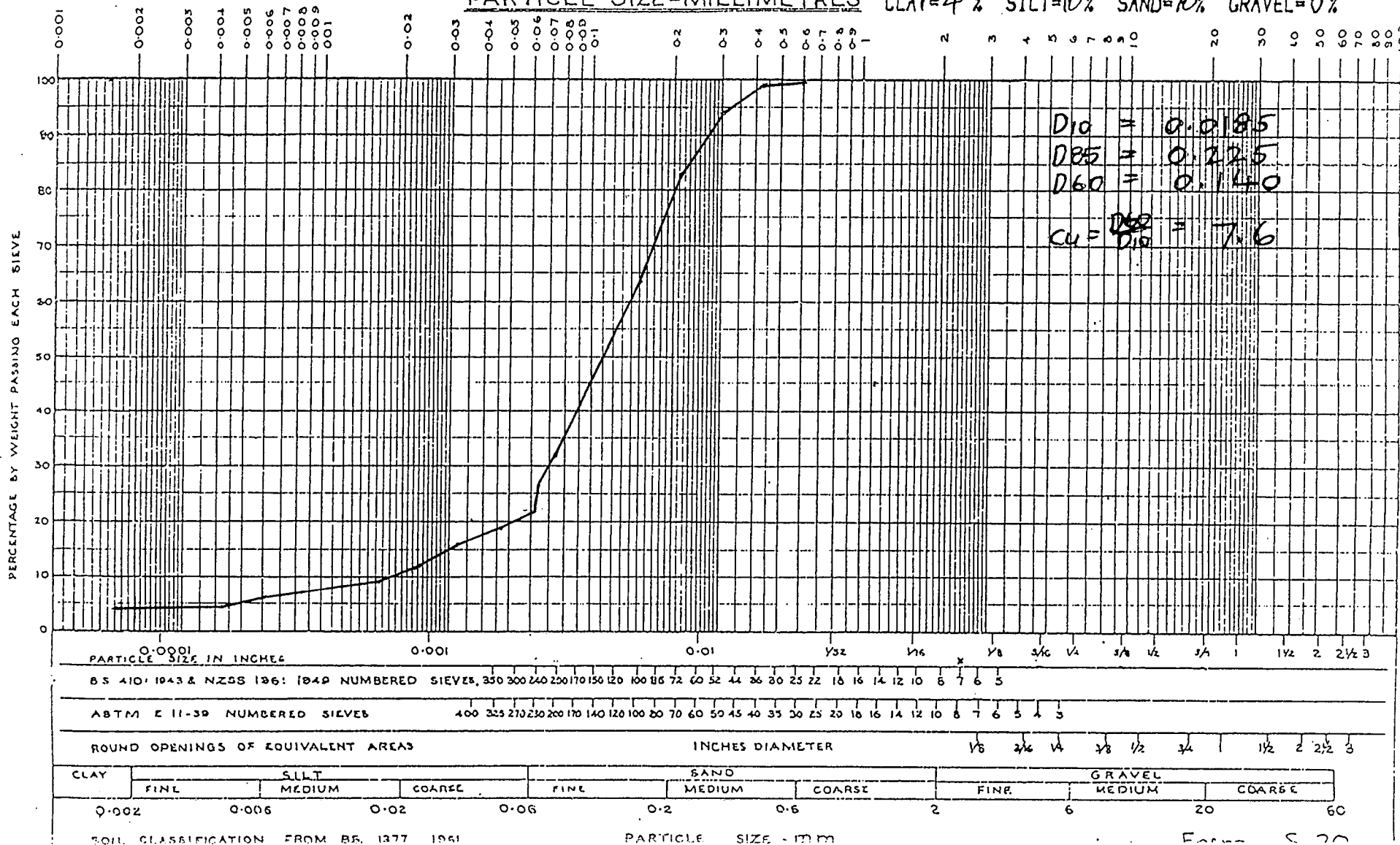
Job No. 87/315

Sample No. . . . 6

Tested by: S.W.LIM Date: 9/87

Checked by: M.R.E. Date: 1/88

PARTICLE SIZE-MILLIMETRES CLAY=4% SILT=18% SAND=78% GRAVEL=0%



Job ... RANGITAIKI RIVER

Location ... SITE 20

Bore ... Depth ...

History of Sample Natural/Air Dried/Oven Dried/Unknown.

ph of Suspension

Dispersant used. Sodium Hexametaphosphate/

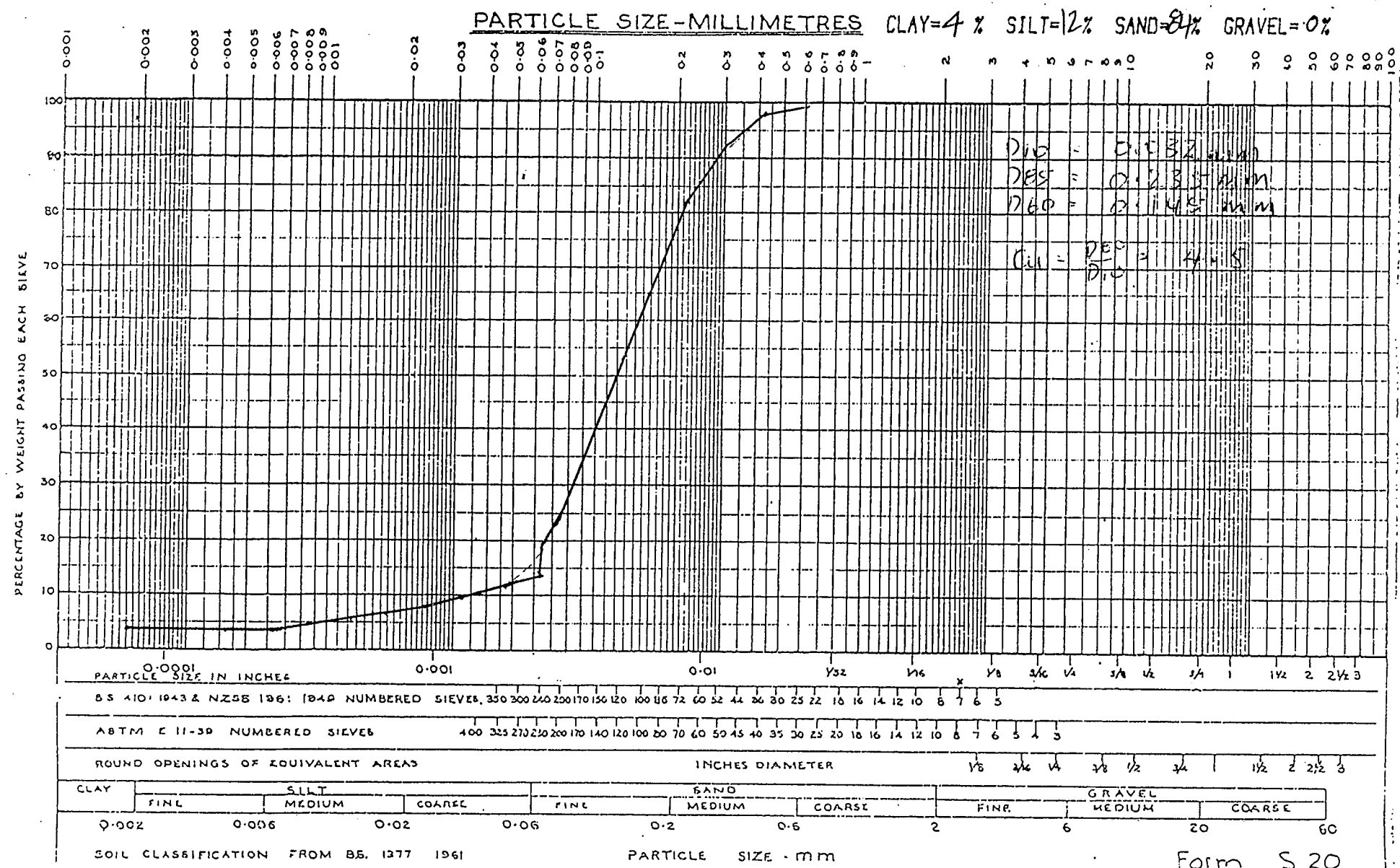
Job No. 87/315

Sample No. 7

Tested by: S.W. LIM Date: 9/87

Checked by: M.R.E. Date: 1/88

20' -SAMPLE TAKEN BANK MID HEIGHT



MINISTRY of WORKS & DEVELOPMENT
HAMILTON DISTRICT LABORATORY

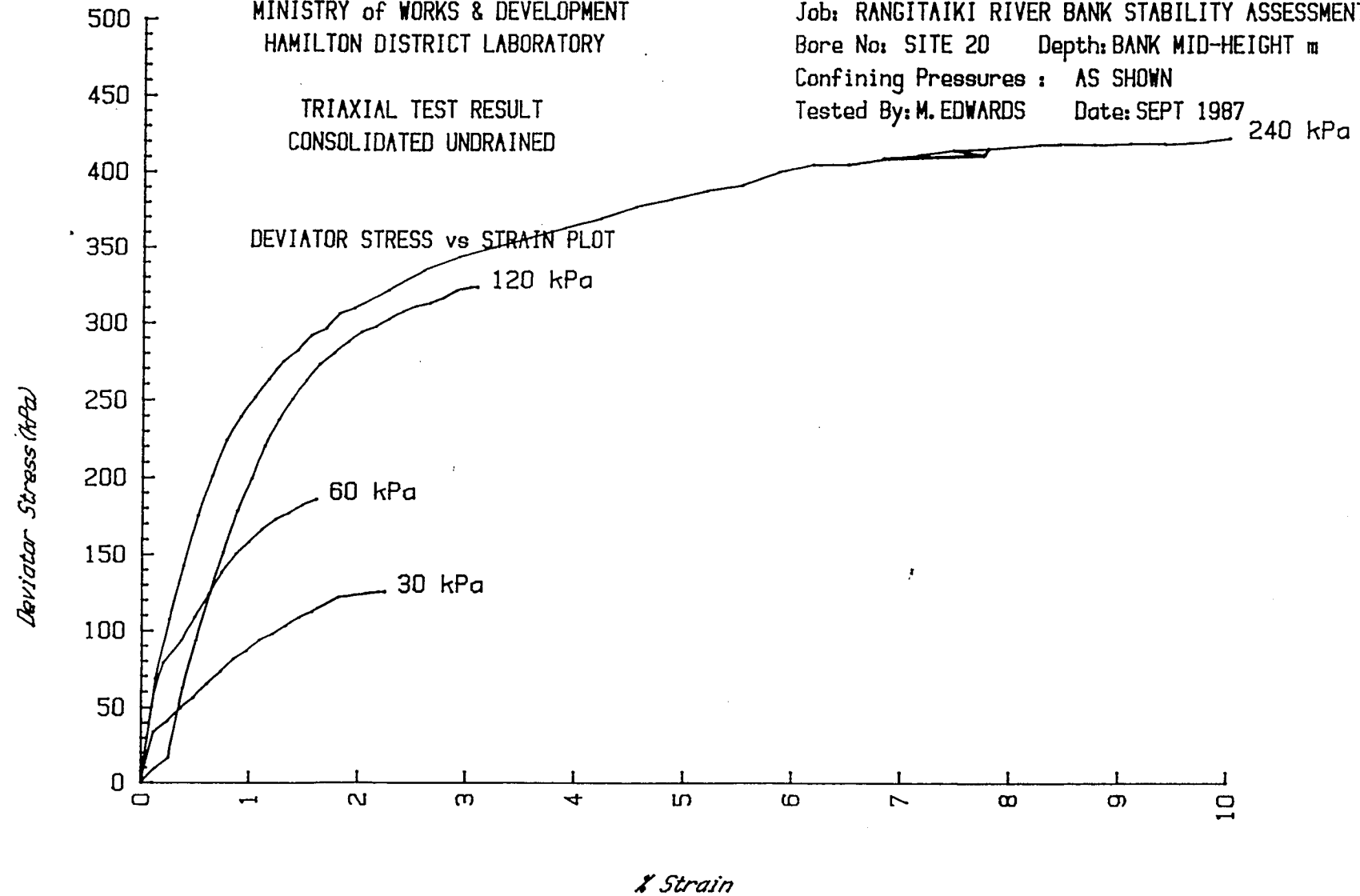
Job: RANGITAIKI RIVER BANK STABILITY ASSESSMENT 87/3

Bore No: SITE 20 Depth: BANK MID-HEIGHT m

Confining Pressures : AS SHOWN

Tested By: M. EDWARDS Date: SEPT 1987

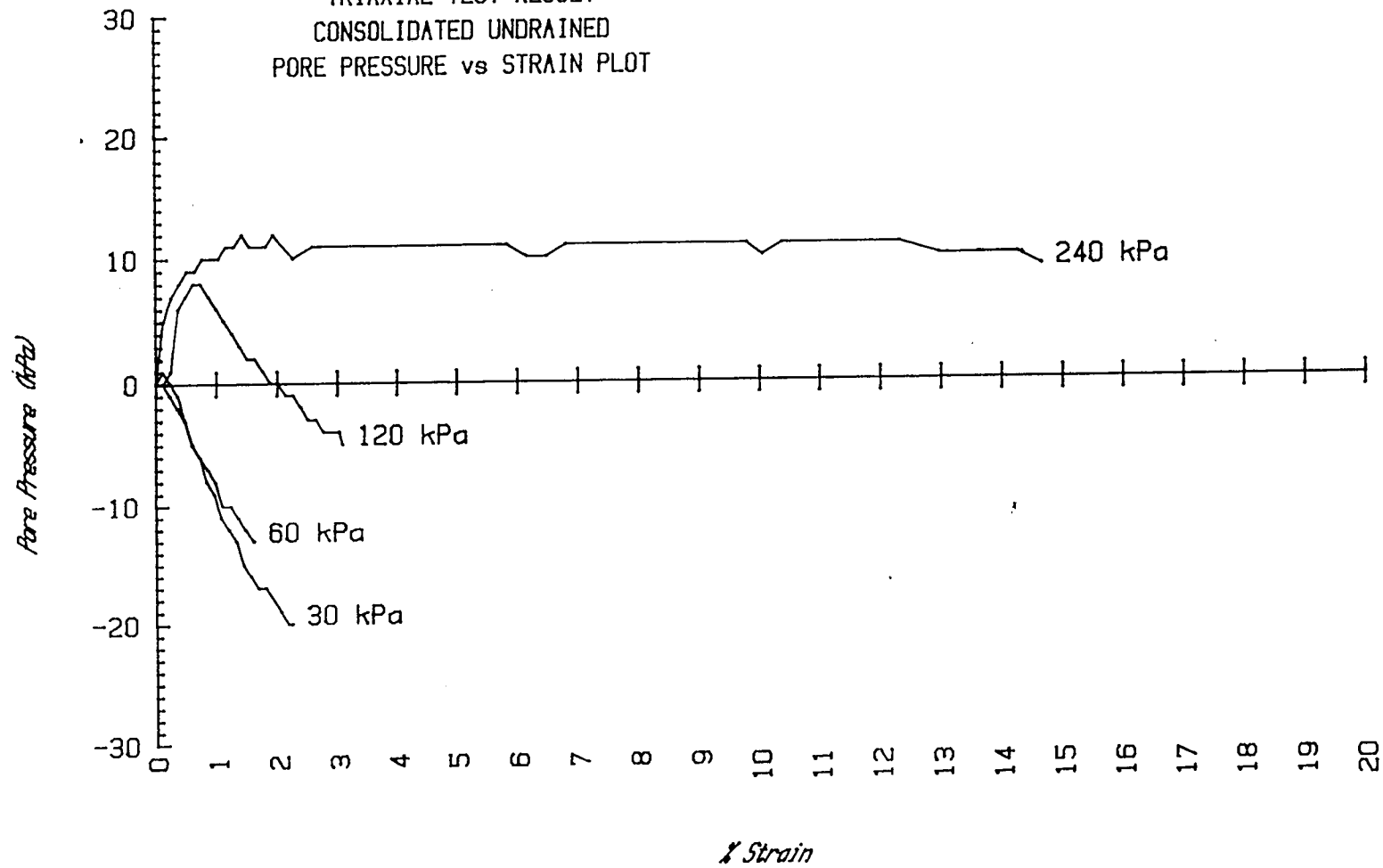
TRIAXIAL TEST RESULT
CONSOLIDATED UNDRAINED



MINISTRY of WORKS & DEVELOPMENT
HAMILTON DISTRICT LABORATORY

Job: RANGITAIKI RIVER BANK STABILITY ASSESSMENT 87/3
Bore No: SITE 20 Depth: BANK MID-HEIGHT m
Confining Pressures : AS SHOWN

TRIAXIAL TEST RESULT
CONSOLIDATED UNDRAINED
PORE PRESSURE vs STRAIN PLOT



MINISTRY of WORKS & DEVELOPMENT
HAMILTON DISTRICT LABORATORY

Job: RANGITAIKI RIVER BANK STABILITY ASSESSMENT 87/315

Bore No: Depth: m

Sample No: SITE 20 - BANK MID-HEIGHT

Tested By: M.R. EDWARDS Date: SEPT 1987

TRIAXIAL TEST RESULT
CONSOLIDATED UNDRAINED

$c' = 0$ kPa

$\phi' = 35$ deg

$c = 15$ kPa

$\phi = 32$ deg

