Department of Transport and Works

POWER
AND
WATER
AUTHORITY

DEPARTMENT OF TRANSPORT AND WORKS

on behalf of

POWER AND WATER AUTHORITY

**Preliminary Report on**

# Ludmilla To Larrakeyah Rising Main

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CLIENT REVIEW ISSUE

**Gutteridge Haskins & Davey Pty Ltd**

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# 1. INTRODUCTION

The GHD report on Darwin sewerage strategies (PAWA - Darwin Urban Area - Wastewater Collection, Treatment, Disposal and Beneficial Re-Use Systems - Final Report - June 1996) proposed an effluent rising main from the Ludmilla WWTP to the Larrakeyah Outfall to convey effluent to Larrakeyah for discharge and to allow reuse for irrigation during dry weather.

The purpose of this Report is to identify the route and preliminary grading of the proposed rising main; assess requirements for pumping of effluent at the Ludmilla Plant and for control of odours; and to prepare capital and operational costs for the pumping and rising main system.

The report would therefore allow the feasibility of and funding availability for the proposed system to be assessed by PAWA before more detailed consideration is given to effluent reuse requirements/potential.

# 2. PRELIMINARY DESIGN OF RISING MAIN

## *2.1* General Route Options

A number of route options were investigated for the route of the proposed Ludmilla Rising Main from the Ludmilla WWTP to the Larrakeyah Outfall (refer Figure 1 attached).

In general, the preferred route options follow the alignment of the existing 600 dia rising main from the Ludmilla WWTP, along Dick Ward Drive, East Point Road, Gilruth Avenue, Lambell Terrace and Schultze Street to the Larrakeyah Outfall.

Additional routes were considered from the Ludmilla Plant along the general alignment of Dick Ward Drive, Playford Street, Parap Road, Stuart Highway, Packard Place/Street to the Larrakeyah Outfall, but were rejected due to the impracticality of construction along this general alignment. This general route option involves heavily trafficked streets in commercial areas and offers no advantages with respect to pipeline length, operation, or constructibility compared to the other general route option considered.

Comments regarding the specific route options associated with the preferred general route are discussed below.

## 2.2 Specific Route Options

### 2.2.1 Ludmilla WWTP to Playford Street

The route proposed runs parallel to the existing 600 dia rising main from the Ludmilla WWTP to Dick Ward Drive, then along or parallel to the bikeway on the northern side of the road as far as Playford Street. There are no water and sewerage services along the northern side of Dick Ward Drive and the bikeway alignment is clear of trees and other obstructions. The southern side of the Drive beside the Racecourse contains a water main and the 750 dia Trunk sewer from the Parap area and is not considered to be feasible in comparison to the northern side for construction of the proposed rising main.

### 2.2.2 Playford Street to Lampe Street

Two broad route options were considered for the route of the proposed rising main from the intersection of Dick Ward Drive and Playford Street to the intersection of East Point Road and Lampe Street, namely a route along Dick Ward Drive, Ross Smith Avenue and East Point Road versus a route via various residential streets.

Several routes via residential streets are available, including:

- Playford Street, Parsons Street, Ross Smith Avenue, Conigrave Street and Lampe Street;
- Playford Street, Parsons Street, Ross Smith Avenue, Giles Street and Lampe Street;
- Playford Street, Brogan Street, Ross Smith Avenue, Goldsmith Street, Holtze Street, Green Street and Lampe Street.

The 750 dia Parap trunk sewer is located along the northern side of Parsons Street and down the centre of Ross Smith Avenue while a water main is located along the southern side of Parsons Street. Power, water, telephone and sewers are located along either side of Conigrave Street. There are significant trees on both sides of that road together with an elevated rock outcropping on the northern side of the street near Holtze Street.

The remaining streets are narrower than Conigrave Street and have water, power and telephone services and occasional trees on either side.

In all cases, it is concluded that it would be necessary to construct the main down the centre of the road in the sealed pavement to avoid existing services, trees and other obstructions.

Location of the main along these route options would result in significant community impact and disturbance; disruption of access to properties; disruption to services to properties; and additional costs associated with construction in a relatively confined area and road restoration over the full length of the section of main in question. It is concluded that the potential savings in pipeline length of from 100 to 200m do not justify the significant community impacts and additional costs as noted above. Location of the proposed rising main along the residential streets in this area is therefore not recommended.

The proposed route over this section of main is to continue along or parallel to the bikeway along the northern side of Dick Ward Drive and Ross Smith Avenue up to the park area opposite the intersection of Ross Smith Avenue with Allen Street. The main is then proposed to cross over Ross Smith Avenue to the park land area to the north and west of the old Fannie Bay Gaol (around the outer edges of the parking area), thence across East Point Road to run in the western road shoulder to Lampe street.

This route option avoids the majority of services and residences along Dick Ward Drive and Ross Smith Avenue, which are located generally on the southern side of the roads. This route also avoids the shopping area and residences located on the northern side of Ross Smith Avenue from the park area to the intersection with East Point Road. The route along the western side of East Point Road as proposed minimises the impacts on residences along the road and also results in the least disturbance to traffic.

### 2.2.3 Lampe Street to Liveris Drive

From Lampe Street to south of Conacher Street, the proposed route is along the road shoulder along the western side of East Point Road. There are no major services along this alignment and the route is generally clear of obstructions and would minimise traffic disruption.

South of Conacher Street, past the intersections of Goyder Road and Salonika Street to the intersection of Atkins Drive and Gilruth Avenue, the route is at its most elevated, passing over a rocky ridge adjacent to the Darwin High School. The western side of East Point Road and Gilruth Avenue is in a cutting up to the edge of the road pavement in this location and construction of the main along the edge of the road would be impractical. The main is therefore proposed to divert from the western road shoulder/bikeway alignment south of Conacher Street in the vicinity of the bikeway deviation towards the Darwin High School, to follow the general alignment of the existing power line across the ridge to Atkins Drive.

This alignment is feasible for construction of the main and avoids the significant disruption to traffic that would result if the main were to be constructed through the cutting. Continued location of the main on the western side of the East Point Road and Gilruth Avenue also avoids disruption to the more major intersections with Goyder Road and Salonika Street.

On the southern side of the ridge adjacent to Darwin High School, the route is proposed to be located at the western edge of Gilruth Avenue, along the eastern edge of Atkins Drive, until the commencement of the bikeway near the Gardens Road intersection. The route is then proposed to follow south along the western shoulder of Gilruth Avenue until the Darwin Tennis Association Courts adjacent to the MGM Grand Casino entrance at Liveris Drive. From the Tennis Courts to south of Liveris Drive, the route is proposed to be located within the western road shoulder of Gilruth Avenue. This location avoids the obstructions along the restricted bikeway alignment in this area and would still be expected to minimise impacts on traffic flow along Gilruth Avenue.

The main would be required to be located so as to avoid the existing 225 dia water main along the western boundary of Gilruth Avenue south of Gardens Road.

### 2.2.4 Liveris Drive to Smith Street

South of Liveris Drive, the main is proposed to be located off the western edge of Gilruth Avenue, until the rocky outcrop which extends from the eastern end of Temira Crescent west towards Gilruth Avenue. Between this outcrop and Smith Street, Gilruth Avenue is constructed in an embankment.

Two options exist for location of the main in this area. The first option involves location of the main immediately adjacent to the western edge of Gilruth Avenue, along the toe of the embankment. This alignment would result in a localised low point in the main, as the ground falls south of the outcrop before rising up to Smiths Street, and is not preferred.

The preferred option would follow around the southern side of the rocky outcrop towards the north-eastern corner of Burnett Place and thence south along the eastern edge of the disused pavement of Burnett Place towards Smith Street. This alignment minimises construction disturbance to the treed area adjacent to Gilruth Avenue and maintains the pipeline at a more constant grading. This alignment also avoids as far as possible the existing 225 dia water main along the western boundary of Gilruth Avenue.

### 2.2.5 Smith Street to Temira Crescent/Malabar Street

At the Gilruth Avenue/Kahlin Avenue/Lambell Terrace/Smith Street intersection, the main is proposed to run to the north and west of the main Gilruth Avenue/Smiths Street intersection, thus minimising traffic disturbance. After crossing the southern end of Kahlin Avenue, the main is proposed to be located along the north-western boundary of Lambell Terrace up to Schultze Street.

This side of Lambell Terrace is clear of major services and is largely utilised for parking bays. Adequate room is available for construction of the main with little impact on the residences along the other side of the road.

From Lambell Terrace to Temira Crescent/Malabar Street, the main is proposed to be located along the western boundary of Schultze Street. There are no major services located along the western footpath of Schultze Street and the route is clear of significant trees or other obstructions. While this route will impact on the residences on the western side of the road, particularly with respect to property access during construction, there is little feasible alternative available.

### 2.2.6 Temira Crescent/Malabar Street to Larrakeyah Outfall

There are two alternative route options for this section of the main.

The first option involves location of the main along Marella Street, Packard Street and Murray Street to the connection to the existing Larrakeyah Outfall in Larrakeyah Terrace.

Power and water services are located on the eastern side of Marella Street and there are a number of significant trees located on the western side of the road. It would therefore be necessary to construct the main within the central pavement area of the road. Construction of the main within Marella Street, while feasible, would result in significant community impact and disturbance; disruption of access to properties; disruption to services to properties; and additional costs associated with construction in a relatively confined area and road restoration over the full length of Marella Street.

From Marella Street to Murray Street, the main could be located on the northern side of Packard Street, which is free of major services and is generally utilised for parking for the Larrakeyah Primary School. Construction of the main on this alignment is feasible but would impact to some degree on access to the primary school.

Murray Street is similar to Marella Street in that there are water and sewerage services on either side of the road, with many palms and trees on both sides of the road. The main would therefore be required to be located within the central pavement area of the road, with similar significant community impacts as for Marella Street. The main would also be required to avoid the existing 450 dia stormwater main located down the centre of Murray Street.

The preferred option for location of the main in this section involves location of the main along Temira Crescent, through the Larrakeyah Primary School grounds to the Larrakeyah Military Area and thence to the end of Larrakeyah Terrace and connection to the existing Outfall.

From the end of Schultze Street, the main is proposed to angle across Temira Crescent to the Larrakeyah Primary School grounds, along the western edge of the school playing fields area adjacent to the trees along the western boundary, and thence between the rear of the

properties fronting Temira Crescent and the school buildings to the Larrakeyah Barracks grounds.

Within the Barracks, it is proposed to locate the main adjacent to the western edge of the service road located along the eastern boundary of the Military Area. This route avoids the Gatehouse and the existing sewerage vent chamber and manhole to the south of the site. It is then proposed to route the main between the existing trees at the end of Larrakeyah Terrace to the connection to the existing Outfall.

The preferred route avoids significant community impacts and construction constraints within Marella, Packard and Murray Streets and, subject to the necessary approvals from Defence Department and the Department of Education, is recommended.

### 2.3 Route Selection/Preliminary Design

The preferred route was determined on the basis of visual inspection together with consideration of information of all known existing services. The route was inspected with Darwin City Council Officers who indicated their general concurrence with the route as proposed. Discussions were also held with Australian Army representatives with respect to the location of the main within the Larrakeyah Barracks area and no objection was made to the route as proposed.

Following preliminary discussions with PAWA Officers, the proposed route was defined in the field by flagging/paint marks. A setout and profile survey was then undertaken along the proposed route, with marks placed at bends or otherwise at intervals of approximately 100m. Services were located from visible evidence or by reference to available drawings.

Setout plans and longitudinal sections of the proposed main, showing a preliminary grading for the main, are attached (Drawings 19277F10-18).

## 3. SYSTEM HEAD CURVES

Based on the survey and preliminary grading of the main as established above, system head curves were derived for a number of alternative rising main sizes, namely 450, 500 and 600 diameter.

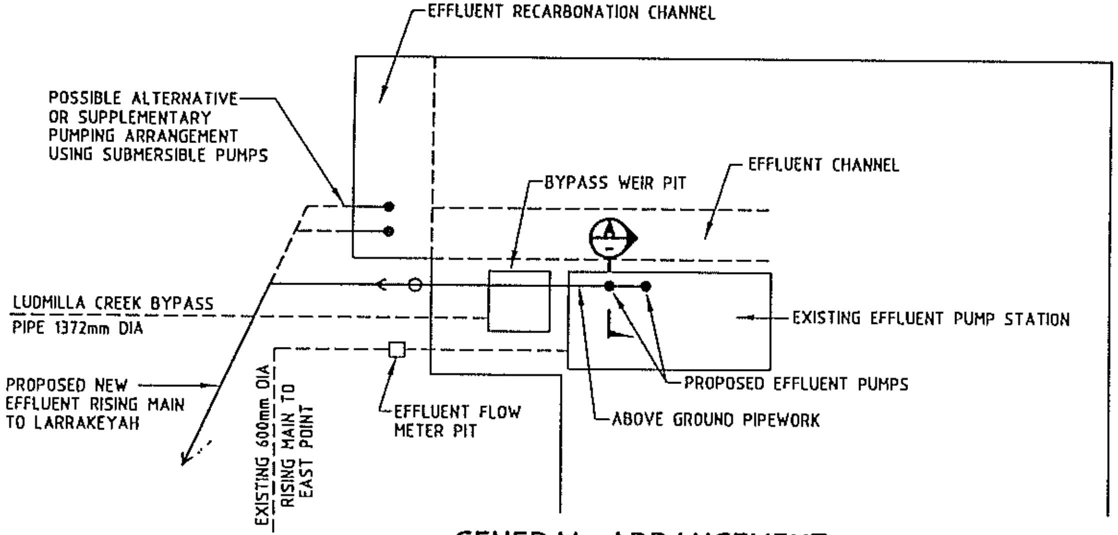
These curves were derived taking into account the rising main profile (as schematically indicated in Figure 2) and the estimated performance of the Larrakeyah Outfall under both High Water and Low Water conditions.

The relevant calculations for derivation of the system head curves are presented in Appendix A. The system head curves for each rising main size option are presented in Figure 3.

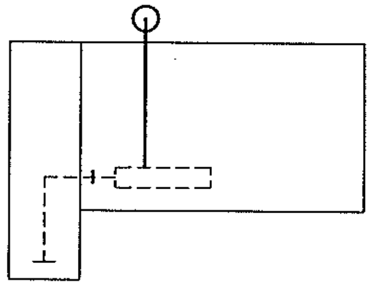
## 4. PROPOSED EFFLUENT PUMPING STATION

### 4.1 Location Of Pump Station

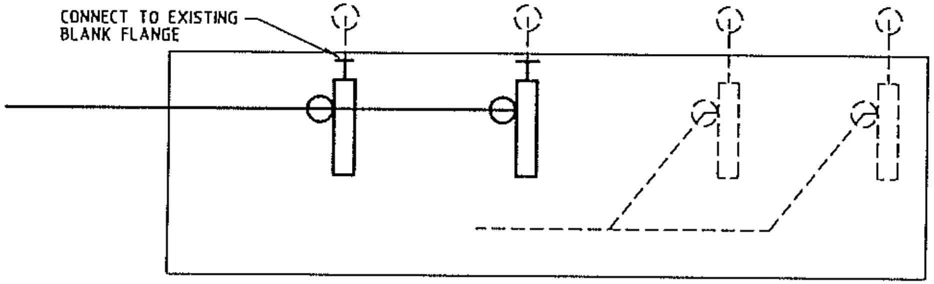
Based on advice received from Ian Wallis (Consulting Environmental Engineers Pty Ltd) regarding possible options for future augmentation of the Ludmilla WWTP, the new effluent pumps are proposed to be installed within the existing effluent pump station at the plant.



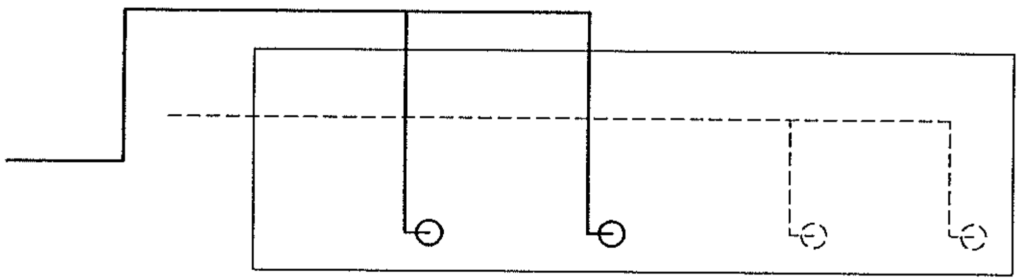

GENERAL ARRANGEMENT



SECTION A




PLAN



SECTION

DEPARTMENT OF TRANSPORT AND WORKS
LUDMILLA WASTEWATER TREATMENT PLANT
**PROPOSED EFFLUENT PUMPING ARRANGEMENT TO LARRAKEYAH OUTFALL**

NOT TO SCALE

GHD

19277F01

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FIGURE 2
Ludmilla Effluent Rising Main - Ludmila WWTP to Larrakeyah Outfall
Rising Main Profile

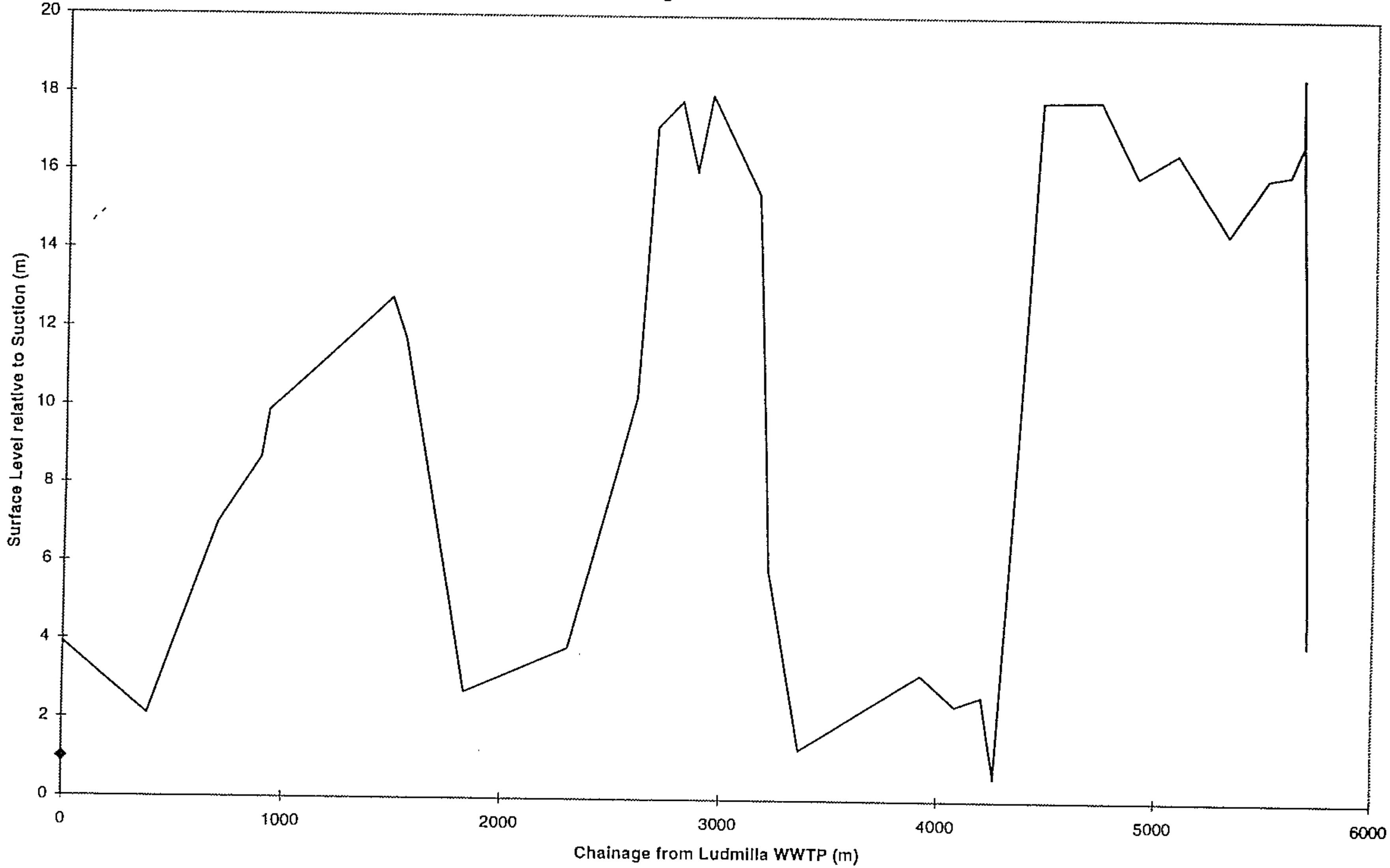
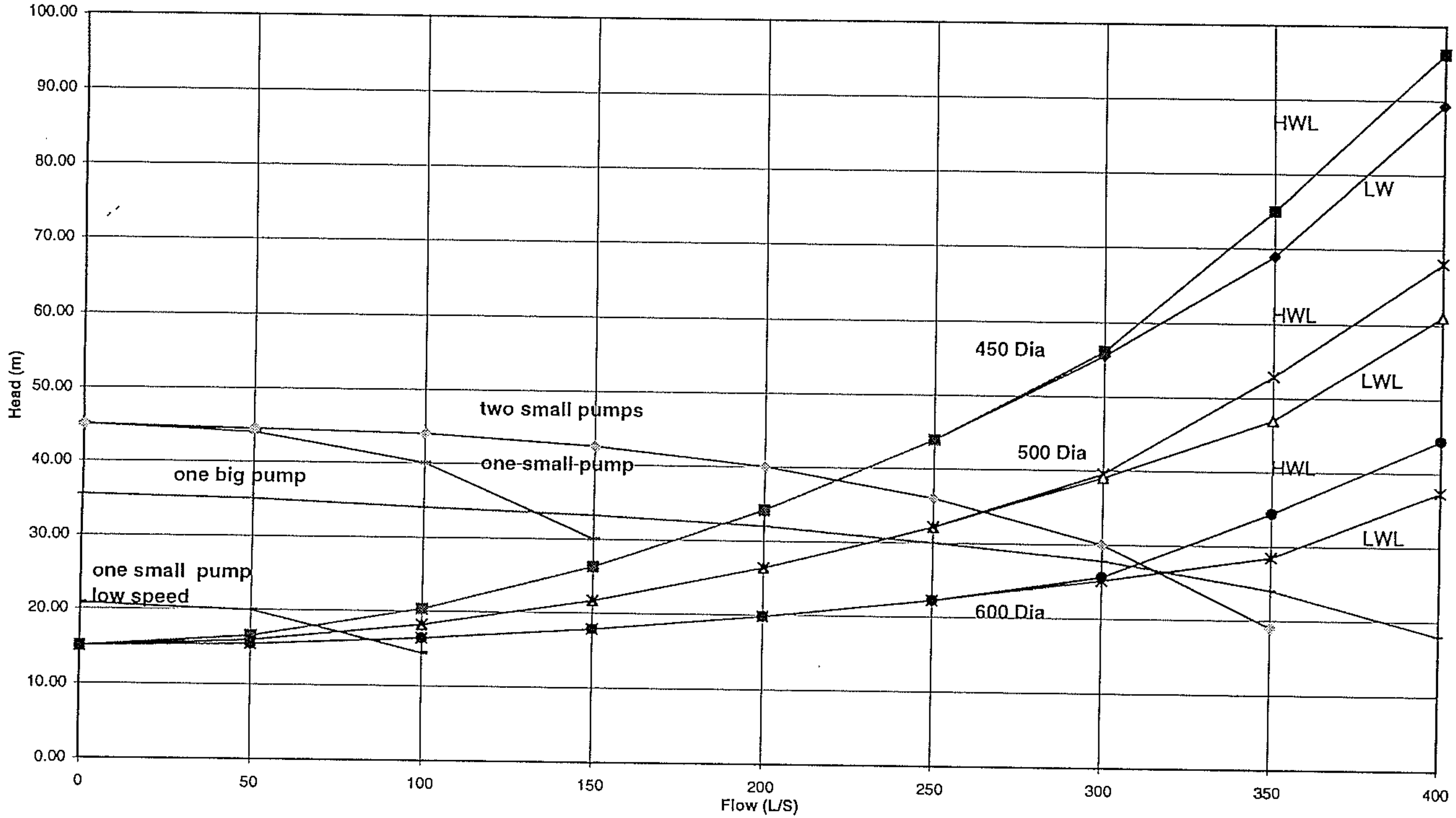


FIGURE 3 - Ludmilla Rising Main System Curves




System Head Curves Chart 1 System Head Curves Chart 1 15-10-97

If the CASS treatment system is retained, then effluent will continue to be discharged via the existing effluent recarbonation/outlet channel. If the augmentation involves a biological secondary treatment process such as extended aeration, etc., then the effluent will likely still discharge to the existing recarbonation channels, although at a slightly lower water level. The existing recarbonation channel has a floor level of 1.52 AHD and an operating water level range from 4.02 to 4.5m AHD (5.32mAHD in the reactor). The overflow has a minimum setting of 4.166m AHD. The existing pump well floor is at 2.62m AHD with a pump suction connection at about 3.10 m AHD and a bottom suction inlet at 1.72m AHD. The pumps could pump down to approximately 2.75 m AHD with adequate submergence of the suction however a minimum level of approximately 3.45m AHD would be desirable to ensure the existing and proposed pumps remain primed without the need for foot valves. It is assumed that the hydraulic design of the future system can accommodate this requirement

It is therefore considered appropriate at this time to provide for pumping of effluent via the new Ludmilla Rising Main from the existing effluent channel. Space exists within the existing effluent pump station for the provision of a further two effluent pumps, and this arrangement is preferred at this stage.

The new pumps are to be connected to the existing suction pipework stubs within the dry well and discharge via vertical risers to a new overhead discharge main. This main would be supported off the western wall of the pump dry well, span across the top of the existing bypass weir pit clear of the access cover and bend underground immediately clear of the access pit. The main would then run above the existing 1370 dia Ludmilla Bypass Line until clear of the existing 600 dia stub/tee on the East Point Rising Main, before deflecting clear of the bypass line.

Alternatively, submersible effluent pumps could be located within the existing recarbonation channel, which would involve some rearrangement of the existing gas pipework. Pumping of the recarbonated effluent at this point is not considered to be desirable due to the potential for release of gas in the rising main.

The proposed effluent pumping arrangement is shown schematically on Drawing 19277F01.

As discussed below, in order to cover the range of flows required with just two pumps it is necessary to adopt two smaller pumps and rely on both pumps to achieve the maximum flow. Should one fail it would be necessary to resort to the existing outfall or to overflow to Ludmilla Creek. Neither would cause major problems however should it be considered necessary to provide full flow standby pumping capacity the two main pumps could each be sized to suit the full flow with a third smaller pump to meet low flows. The third pump would not fit in the pump dry well but could be installed as a submersible unit sufficiently downstream in the recarbonation channel to avoid the bubble zone.

## 4.2 Design Flows

The design pump flows are summarised in Appendix B. Based on a design population of 40 000 EP as specified in the brief, the design Average Dry Weather Flow (ADWF) is 12 ML/d or 140 L/s. The design maximum pump rate specified in the brief is a flow of 27ML/day or 312.5 L/s. This is a fairly arbitrary figure which is about 1.37 times design peak dry weather flow (PDWF) for 40,000 EP. Of more relevance is that it may also corresponds to the probable capacity of the existing outfall under the maximum safe surcharge pressure.

It is noted that these flows represents the expected plant load immediately after diversion of the CBD and makes no allowance for ongoing growth. It is expected that the load on the Ludmilla plant will ultimately increase to approximately 50,000 EP. At that time the ADWF will be 15 ML/day and the Peak DWF approximately 280 L/s hence the design capacity will continue to meet all dry weather flows.

Pump curves for a typical pump with appropriate duty for both DN500 and DN600 rising main options for single pump and dual pump options are included in Appendix B. Variable speed drives are proposed for the pumps to allow the pumps to 'follow the flow'. It is considered unlikely on the basis of preliminary information obtained from pump suppliers and the pump curve information shown that a single pump can be obtained with the capacity to operate over the full duty range required. The efficiency of the unit for the single pump option is excellent at high order flows but drops to unacceptable levels (<70%) for flows less than around 125 L/s. This would correspond to the entire dry season operating range.

It is therefore more likely that two identical pumps can be selected to meet the flow of 140 L/s (ADWF) operating alone and the design maximum flow of 312.5 L/s (PWWF) operating in parallel. The pump curve shown in Appendix B indicates that this option can achieve better than 80% efficiency for flows between approximately 70L/s (single pump at reduced speed) and 300L/s (two pumps). This covers the full range of most probable flows.

The design flows and all economic analyses are based on a continuous flow treatment system with typical diurnal flow distributions. All effluent is assumed to be pumped to the outfall. This may not be the case under some possible scenarios:

- It is understood that consideration is being given to an intermittent decant treatment system which could discharge in slugs requiring all pumping to take place at a higher than average rate. This would potentially favour larger rather than smaller main sizes.

- Introduction of effluent re-use would change the flow conditions in a couple of ways.

  A proportion of the flow would be extracted from the rising main at various points along the main resulting in a lower pump head for the same pump rate. This would favour a smaller main.

  In order to meet re-use demands it would be likely that on-site storage at Ludmilla would be required to retain peak discharge effluent flows for delivery at late night irrigation times when normal plant flow would be reduced. This again will change the diurnal flow patterns and the pump rates required. The storage require may at least partially comprise the settlement basins of an intermittent decant system should this option be adopted.

## 4.3 System Operation

Effluent will be pumped via the new pumps in the Effluent Pump Station to Larrakeyah Outfall, with the pumps being controlled in the first instance by high and low level controls in the effluent channel.

Initially, while gravity discharge of sewage from the Darwin CBD area continues via the Larrakeyah Outfall, the rate of flow of the new effluent pumps will also be controlled so as to maintain a maximum level in the existing discharge manhole downstream of the Larrakeyah Macerator Station. This system will ensure that the available reserve capacity of the outfall

is fully utilised under all tidal and flow conditions and that the capacity of the outfall is not exceeded under peak flow conditions.

Where the output of the new effluent pumps is required to be reduced to suit the available capacity of the Larrakeyah Outfall and the level in the effluent channel continues to rise, then the existing effluent pumps will be operated to discharge the balance of the effluent via the existing East Point Outfall. Where the capacity of both pumps is exceeded under peak flow conditions, then effluent will be bypassed to Ludmilla Creek as at present.

In this way, priority will be given to discharge of effluent via the Larrakeyah Outfall, followed by discharge of effluent via East Point and finally via Ludmilla Creek.

In the future, with the pumped diversion of sewage from the Darwin CBD area to the Ludmilla WWTP for treatment, the initial gravity discharge of effluent via the Larrakeyah Outfall will need to be upgraded in order to achieve the required design outfall capacity of 27 ML/d.

This will require the outfall to operate under a positive head at the discharge manhole downstream of the Larrakeyah Macerator Station, requiring the station to be isolated from the system and the existing manholes to be sealed or bypassed. At this time, the outfall control of the effluent pumps would be abandoned.

It is understood that the proposed pumped diversion of local catchment flows at Larrakeyah will be required to discharge to the outfall at times when the Waterfront pump station would otherwise be overloaded. This will require an actuated diversion valve and connection from the pump discharge to the rising main. Emergency overflow will also be required from the proposed diversion pump station. This will not be possible to the outfall but will require an independent discharge down the cliff face..

# 5. ODOUR CONTROL

## 5.1 Requirement

At minimum flow (less than 50 L/s say), retention of effluent within the rising main will vary from about 5 hours for a 450 dia main to about 10 hours for a 600 dia main. However minimum flow will not exist for this period of time, so it is reasonable to consider retention for average flow conditions. This will be a maximum of between 3 to 5 hours for a 600 dia main for 12 and 8 ML/d respectively. This period is not expected to be excessive for a good quality effluent and should not result in significant odour problems arising for a secondary treated effluent.

For the case where primary effluent is continued to be discharged, the effluent is commonly recarbonated before discharge and odours from the effluent at the plant at present are not unreasonable. Again, it is considered that the level of retention is such that significant odours should not occur provided exposure to the atmosphere is limited and gas release is adequately controlled.

The potential release of odour would occur as follows>

At the existing macerator station, effluent will be discharged under gravity flow conditions only for the interim period when the outfall is required to operate under gravity flow to accept local raw sewerage discharge in conjunction with pumped effluent. Even then, there will be minimal exposure to the atmosphere and existing odour control measures in place at

the outfall should remain adequate. Based on the proposed program for diversion of raw sewage from the CBD from the Larrakeyah Outfall to Ludmilla WWTP and the likely program for construction of the effluent rising main, then the period during which this situation exists should only be a number of months at most. Once the local gravity flow is diverted, the main will be converted to a closed pipe and there will be no direct exposure to the atmosphere. However the end section of main will continue to operate under gravity flow conditions for flow rates up to around 280 L/s.

For a default grading at minimum depths, the main would operate under gravity from the high points in the ground profile at CH 5500 and/or CH 4588. At various times either of these points can change from above to below the hydraulic grade line or visa versa. This would require significant volumes of air to be expelled or drawn in via automatic air release valves. Expelled air would potentially be odorous and it is recommended that measures be taken in the grading of the line to minimise the potential for this occurrence.

Due to the profile of the proposed rising main, odour control measures such as oxygenation are unlikely to be feasible. The low level of odour compounds likely to be present would also make such measures of little benefit and not cost-effective. The only potential for release of odour is at air-release valves and these are discussed below.

## 5.2 Air Release Requirements

Referring to the pipe profile there are a number of high points in the rising main which will require provision of air/gas release. These are as follows

CH 1452 - East Point Road at Fannie Bay Gaol.

> This will remain under pressure except when the main is drained for service. Release of gas from solution would normally be inhibited by the pressure and a manual air release for maintenance service only is proposed.

CH 2800-2940 - Gilruth Av at Darwin High School

> These points are level with the highest point in the main at CH 4588. The pipe can be graded a little deeper to ensure that it will normally be held full by the high point at CH4588 and hence will not normally require significant air inflow/outflow. As the points are not under significant pressure, some gas may be evolved from the flow which will need to be released. An automatic air inlet/release will be required to admit air to avoid negative pressures and to release any accumulated air/gas on pump start. The sites are well away from any existing or proposed occupied buildings but an elevated vent would be recommended to disperse exhausted air. This may be stand alone stack or preferably a small vent pipe attached to the adjacent power poles. Release of gas would be occasional small volumes only and odour would be comparable with a domestic sewer vent.

CH 4588 - Lambell Terrace at Mitchell St.

> With a normal grading of the main, this point would be the hydraulic grade control at low flows and would drain downstream, refilling as flows increase. The site is adjacent to residential development and the former university site and release of any significant volumes of air at this point would be undesirable. On that basis it is recommended that the pipe grading be designed to ensure that this point remains below the hydraulic grade. While the pipe may be lowered slightly (to say IL 17.5) at

CH 4588 this will also require the artificial elevation of main at an appropriate point downstream. This will have a fairly minor adverse effect on pump heads for a small range of flows but the extra operating cost would be trivial. An automatic air release will still be required to vent small volumes of accumulated gas however this would be vented to a small stack (DN50) attached to a suitable power or light pole or to an adjoining gravity sewer.

CH 5500-5662

As discussed above it is recommended that an artificial high point be established in this area to prevent the creation of draining high points at less suitable upstream locations. The most appropriate location for the high point would be immediately prior to the drop to the macerator station at CH5662 where the ground level is 18.15. The high point would involve a elevated loop of pipe with an IL of not less than say, RL18.5 or 0.35 above ground to ensure the main at CH 4588 remains full. An automatic air release would be provided on top of the pipe loop. The AV could be vented to the existing DN750 vented sewer system from discharge manhole of the Kahlin Oval pump station. This system connects to the existing scrubber at the macerator station or alternatively shares the vent stack from the discharge manhole for the Larrakeyah Base pump station. Given the proximity of housing with some sensitivity to odour a connection to the scrubber system may be desirable. It is noted that the proposed pump station to be constructed for the diversion of gravity flow will also require treatment of vented air from wet wells etc.

Exhausting or intake of small amounts of air would occur regularly in response to increase or decrease in discharge flows. As flows vary, this would effect the surcharge level in the downstream outfall pipe. The effluent would be highly agitated by the velocity in the steep gravity section hence the air exhausted would potentially be fairly odorous. The potential for odour would be minimised by adopting the smallest feasible rising main and hence the lowest detention time.

# 6. ESTIMATES

Preliminary budget estimates have been prepared for construction of the rising main based on typical costs for similar works as obtained from the Department of Transport and Works. Budget capital cost estimates, inclusive of allowances for engineering and contingencies, have been prepared for three rising main sizing options, namely 450, 500 and 600 diameter, as detailed in Appendix C.

Annual power costs for operation of pumping equipment for each rising main size option have also been assessed as detailed in Appendix D. Annual power costs have been assessed on the basis of pump operation at average day load (ADWF) for 7 months of the year (dry season operation) and at peak load (PWWF) for 5 months of the year (wet season operation).

The total estimated costs for each rising main option as detailed in Appendices C & D are summarised as follows:

| Rising Main Diameter (mm) | Total Capital Cost | Annual Pumping Cost ($) |
|---|---|---|
| 450 | 3 650 000 | 163 030 |
| 500 | 3 900 000 | 115 130 |
| 600 | 4 500 000 | 76 660 |

# 7. ECONOMIC ANALYSIS

A present worth analysis has been undertaken for each rising main size option to assess the most economic main diameter, taking into account capital costs, annual operating costs and replacement costs for electrical and mechanical equipment.

The economic analysis has been carried out for a 50 year operating period, which is considered to represent the minimum operating life of the rising main. Pumping equipment is assumed to require replacement after 25 years. In general, only costs not common to each option have been considered in the analysis. The analysis has been undertaken for varying discount rates from 5 to 10 %. It is understood that a discount rate of approximately 7.5% is currently adopted by PAWA for evaluation of power costs.

The results of the economic analysis for each rising main option are presented in Appendix E and the Net Present Values (NPV's) for the options for each discount rate are summarised below.

| Rising Main Diameter (mm) | 450 | 500 | 600 |
|---|---|---|---|
| NPV @ 5% discount rate | 6 745 000 | 6 072 000 | 5 877 000 |
| NPV @ 7% discount rate | 5 995 000 | 5 538 000 | 5 519 000 |
| NPV @ 10% discount rate | 5 339 000 | 5 072 000 | 5 205 000 |

Given the orders of accuracy involved in the above economic analysis, it is concluded that selection of either the 500 and 600 diameter rising main options will result in the least overall life-cycle costs and both are of identical total cost at the normally adopted discount rate of 7.5%. The final choice must then be made on other grounds such as:

- The 600 diameter option provides flexibility to accommodate a larger proportion of the higher wet season flows. It is noted that current wet season flows at the plant are in excess of 100ML/d or 1200L/s. Higher flows would however be possible only if the existing outfall can tolerate higher flows. The initial section of outfall pipe is old style Class 4.5 uPVC which with age and temperature derating may not have a current safe working pressure much in excess of around 20m or the equivalent of approximately 320 L/s flow. Higher flows are hence unlikely to be possible without upgrading the outfall. The current peak design capacity is more than adequate for peak dry weather flows in excess of 50,000EP hence additional capacity is of marginal benefit.

- The larger main would be slightly favoured if selection of intermittent decant operation of the future plant required slug discharge of effluent at higher than average flow rates.

- The economic analysis was done on the basis of 12ML/day average dry weather flows (40,000 EP). Growth to 50,000 EP would slightly favour the larger main.

- The limitations set by budget allocation and the perceived benefits in deferral of capital expenditure would favour the smaller main.

- Minimisation of detention time and hence the reduction of potential for odour generation would also favour the smaller main. This would be of particular priority if a low grade CASS or similar treatment process is adopted for the future development of Ludmilla Plant

- Flow velocities in the DN600 would be somewhat below desirable slime control values even at peak flow and would barely meet self-cleansing limits at peak dry weather flows. This may exacerbate odour production and sludge accumulation and hence would also favour the smaller main.

- Introduction of re-use would result in lower average flows being discharged to the outfall and hence would favour the smaller main.

# 8. CONCLUSIONS

The proposed rising main route is as detailed on Drawings 19277F10-18 attached.

Effluent is proposed to be pumped into the rising main via pumps with variable speed drives (VSD's) located within the existing effluent pump station at the Ludmilla Plant, with suction connection to the existing effluent channel. Two identical pumps are expected to be required, with each pump capable of meeting the average dry weather flow of 140 L/s (one duty/one standby) and both pumps operating in parallel to meet the peak wet weather flow of 312.5 L/s.

Given the estimated detention period for effluent within the main of less than 5 hours, even for a 600 dia main, specific odour control procedures are not considered to be necessary, particularly when the Ludmilla Plant process is upgraded to provide a secondary effluent standard suitable for effluent irrigation. Some consideration of air release points is nevertheless required and the grading should include an artificial high point at CH5662 to ensure other high points upstream are not prone to fill and drain with pump flow fluctuations. Manual air release valves only would be provided at minor high points. Automatic air release valves will still be required at major high points and where required would preferably be vented to an elevated stack or to adjacent sewers. Significant release of gas is not expected to occur at intermediate points except when draining for maintenance. Venting of the point at CH5662 may be connected to the existing vented inlet pipe from the Kahlin Bay rising main discharge.

On the basis of an economic analysis of varying rising main sizing options for varying discount rates from 5 to 10%, it is concluded that within the accuracy of the analysis, either a 500 diameter or 600 diameter rising main would result in the least overall life cycle cost for the project. The final choice will then depend on other factors as listed above and which again are inconclusive until a decision is made on the process to be adopted for treatment at Ludmilla. At this time the benefits of lower capital cost, lower odour potential and the reasonable probability that re-use will be adopted would outweigh the doubtful advantage of

higher possibility flow rates and the DN500 is recommended. This could be amended if the selection of an intermittent plant process significantly increases the required pump rates.

The preliminary budget estimate of cost for implementation of a 500 dia rising main from Ludmilla WWTP to the Larrakeyah Outfall, complete with all associated pumping equipment, is $3 900 000.

## 9. RECOMMENDATIONS

It is recommended that copies of Drawings 19277F10-18 be forwarded immediately to Darwin City Council, Australian Army and the Department of Education for approval in principle of the proposed rising main route.

It is further recommended that the Power and Water Authority review the findings of this Report in the light of funding availability, to determine the feasibility of the Ludmilla rising main as proposed, prior to proceeding with more detailed investigations with respect to effluent re-use requirements/potential.

# Appendix A

## System Head Curve Calculations

**Larrakeyah Outfall Capacity**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 370 | 370 | 370 | 370 | 370 | 370 | 370 | 370 | 370 |
| L (m) = | 800 | 800 | 800 | 800 | 800 | 800 | 800 | 800 | 800 |
| C = | 120 | 120 | 120 | 120 | 120 | 120 | 120 | 120 | 120 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| **H (m) =** | **0.00** | **0.60** | **2.15** | **4.56** | **7.77** | **11.74** | **16.44** | **21.87** | **28.00** |
| Available Head at MH - **HWL** | 7.75 | | | | | | | | 7.75 |
| Available Head at MH - **LWL** | 13.75 | | | | | | | | 13.75 |
| Head relative to MH - **HWL** | -7.75 | -7.15 | -5.60 | -3.19 | 0.02 | 3.99 | 8.69 | 14.12 | 20.25 |
| Head relative to MH - **LWL** | -13.75 | -13.15 | -11.60 | -9.19 | -5.98 | -2.01 | 2.69 | 8.12 | 14.25 |

**Ludmilla 600 Effluent Rising Main (to high point - 0 to 5662.14)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 621 | 621 | 621 | 621 | 621 | 621 | 621 | 621 | 621 |
| L (m) = | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 |
| C = | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) = | 0.00 | 0.37 | 1.33 | 2.81 | 4.78 | 7.22 | 10.11 | 13.45 | 17.22 |

**Ludmilla 600 Effluent Rising Main (to end - 0 to 5683)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 621 | 621 | 621 | 621 | 621 | 621 | 621 | 621 | 621 |
| L (m) = | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 |
| C = | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) = | 0.00 | 0.37 | 1.33 | 2.82 | 4.79 | 7.24 | 10.15 | 13.50 | 17.28 |

**Ludmilla 600 Effluent Rising Main System Curve**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 621 | 621 | 621 | 621 | 621 | 621 | 621 | 621 | 621 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) =15+Hf(0-5662.14) | 15.00 | 15.37 | 16.33 | 17.81 | 19.78 | 22.22 | 25.11 | 28.45 | 32.22 |
| H (m) =8.56+Hf(0-5683) | 8.56 | 8.93 | 9.89 | 11.38 | 13.35 | 15.80 | 18.71 | 22.06 | 25.84 |
| H (m) =LWL + Hf(0-5683) | -8.00 | -7.03 | -4.52 | -0.62 | 4.56 | 10.98 | 18.60 | 27.37 | 37.28 |
| H (m) =HWL + Hf(0-5683) | -1.00 | -0.03 | 2.48 | 6.38 | 11.56 | 17.98 | 25.60 | 34.37 | 44.28 |

**Final System Curve - 600 Main**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 | |
| V (m/s) = | 0.00 | 0.17 | 0.33 | 0.50 | 0.66 | 0.83 | 0.99 | 1.16 | 1.32 | |
| Retention (Hrs) = | n/a | 9.59 | 4.80 | 3.20 | 2.40 | 1.92 | 1.60 | 1.37 | 1.20 | |
| H (m) = | 15.00 | 15.37 | 16.33 | 17.81 | 19.78 | 22.22 | 25.11 | 28.45 | 37.28 | outfall@LWL |
| H (m) = | 15.00 | 15.37 | 16.33 | 17.81 | 19.78 | 22.22 | 25.60 | 34.37 | 44.28 | outfall@HWL |

**Ludmilla 500 Effluent Rising Main (to high point - 0 to 5662.14)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 520 | 520 | 520 | 520 | 520 | 520 | 520 | 520 | 520 |
| L (m) = | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 |
| C = | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) = | 0.00 | 0.87 | 3.15 | 6.66 | 11.34 | 17.13 | 24.01 | 31.93 | 40.88 |

**Ludmilla 500 Effluent Rising Main (to end - 0 to 5683)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 520 | 520 | 520 | 520 | 520 | 520 | 520 | 520 | 520 |
| L (m) = | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 |
| C = | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) = | 0.00 | 0.88 | 3.16 | 6.68 | 11.38 | 17.20 | 24.09 | 32.05 | 41.03 |

**Ludmilla 500 Effluent Rising Main System Curve**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 520 | 520 | 520 | 520 | 520 | 520 | 520 | 520 | 520 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) =15+Hf(0-5662.14) | 15.00 | 15.87 | 18.15 | 21.66 | 26.34 | 32.13 | 39.01 | 46.93 | 55.88 |
| H (m) =8.56+Hf(0-5683) | 8.56 | 9.44 | 11.72 | 15.24 | 19.94 | 25.76 | 32.65 | 40.61 | 49.59 |
| H (m) =LWL + Hf(0-5683) | -8.00 | -6.53 | -2.69 | 3.25 | 11.15 | 20.93 | 32.54 | 45.92 | 61.03 |
| H (m) =HWL + Hf(0-5683) | -1.00 | 0.47 | 4.31 | 10.25 | 18.15 | 27.93 | 39.54 | 52.92 | 68.03 |

**Final System Curve - 500 Main**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 | |
| V (m/s) = | 0.00 | 0.24 | 0.47 | 0.71 | 0.94 | 1.18 | 1.41 | 1.65 | 1.88 | |
| Retention (Hrs) = | n/a | 6.73 | 3.36 | 2.24 | 1.68 | 1.35 | 1.12 | 0.96 | 0.84 | |
| H (m) = | 15.00 | 15.87 | 18.15 | 21.66 | 26.34 | 32.13 | 39.01 | 46.93 | 61.03 | **outfall@LWL** |
| H (m) = | 15.00 | 15.87 | 18.15 | 21.66 | 26.34 | 32.13 | 39.54 | 52.92 | 68.03 | **outfall@HWL** |

**Ludmilla 450 Effluent Rising Main (to high point - 0 to 5662.14)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 467 | 467 | 467 | 467 | 467 | 467 | 467 | 467 | 467 |
| L (m) = | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 | 5662.14 |
| C = | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) = | 0.00 | 1.47 | 5.31 | 11.24 | 19.14 | 28.92 | 40.52 | 53.89 | 69.00 |

**Ludmilla 450 Effluent Rising Main (to end - 0 to 5683)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 467 | 467 | 467 | 467 | 467 | 467 | 467 | 467 | 467 |
| L (m) = | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 | 5683 |
| C = | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 | 115 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| H (m) = | 0.00 | 1.48 | 5.33 | 11.28 | 19.21 | 29.03 | 40.67 | 54.09 | 69.25 |

**Ludmilla 450 Effluent Rising Main System Curve**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| D (mm) = | 467 | 467 | 467 | 467 | 467 | 467 | 467 | 467 | 467 |
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
| | | | | | | | | | |
| H (m) =15+Hf(0-5662.14) | 15.00 | 16.47 | 20.31 | 26.24 | 34.14 | 43.92 | 55.52 | 68.89 | 84.00 |
| H (m) =8.56+Hf(0-5683) | 8.56 | 10.04 | 13.89 | 19.84 | 27.77 | 37.59 | 49.23 | 62.65 | 77.81 |
| H (m) =LWL + Hf(0-5683) | -8.00 | -5.92 | -0.52 | 7.84 | 18.98 | 32.76 | 49.12 | 67.96 | 89.25 |
| H (m) =HWL + Hf(0-5683) | -1.00 | 1.08 | 6.48 | 14.84 | 25.98 | 39.76 | 56.12 | 74.96 | 96.25 |

**Final System Curve - 450 Main**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Q (L/s) = | 0 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 | |
| V (m/s) = | 0.00 | 0.29 | 0.58 | 0.88 | 1.17 | 1.46 | 1.75 | 2.04 | 2.34 | |
| Retention (Hrs) = | n/a | 5.42 | 2.71 | 1.81 | 1.36 | 1.08 | 0.90 | 0.77 | 0.68 | |
| H (m) = | 15.00 | 16.47 | 20.31 | 26.24 | 34.14 | 43.92 | 55.52 | 68.89 | 89.25 | outfall@LWL |
| H (m) = | 15.00 | 16.47 | 20.31 | 26.24 | 34.14 | 43.92 | 56.12 | 74.96 | 96.25 | outfall@HWL |

Appendix B

Design Pump Flows

## Design Pump Flows

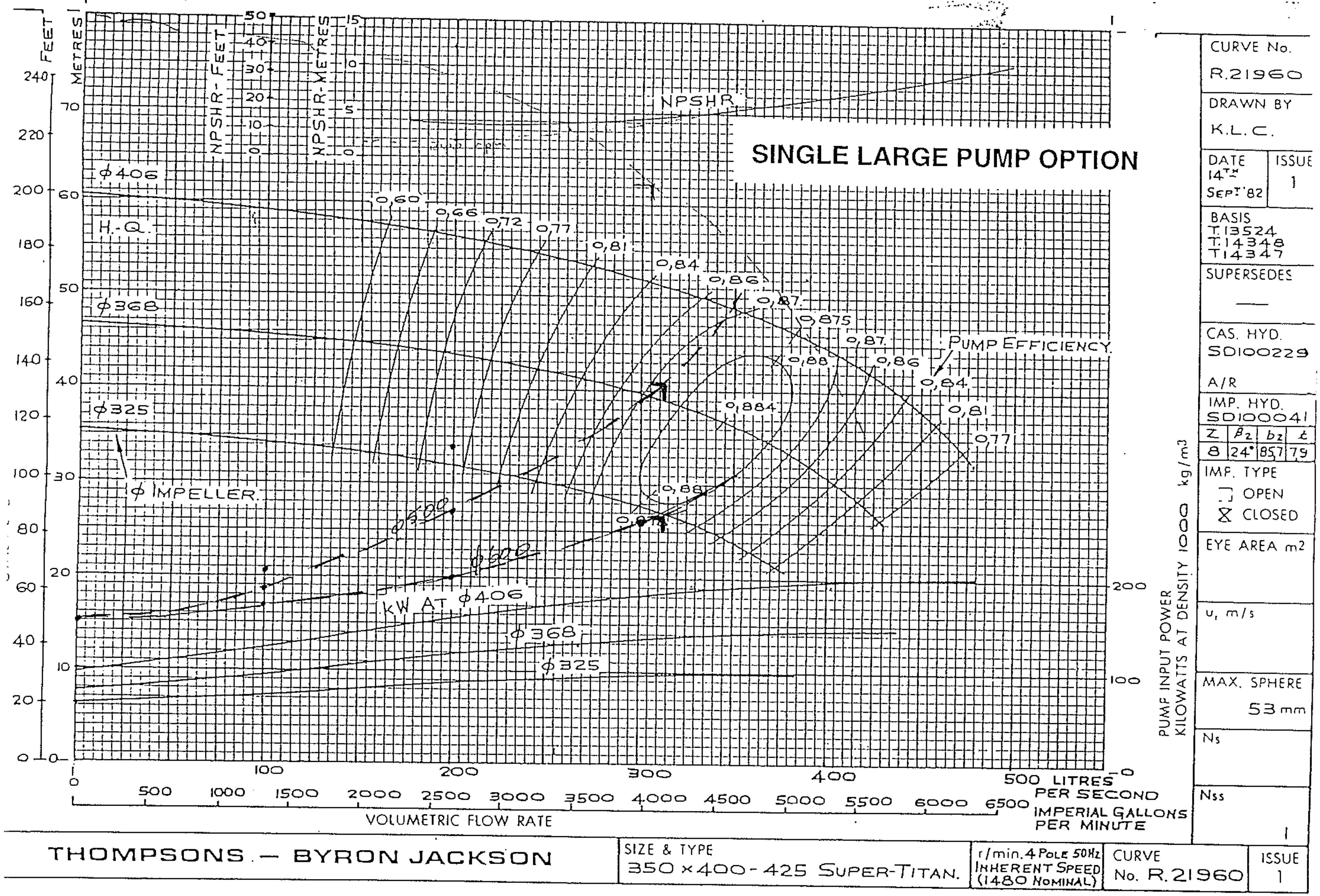
| | | |
|---|---|---|
| Existing Population (EP) = | 26700 | |
| Exist'g Min. Dry Weather Flow (MDWF)= | 3 ML/d = | 32.45 L/s |
| Exist'g Average Dry Weather Flow (ADWF)= | 8 ML/d = | 92.71 L/s |
| Exist'g Peak Dry Weather Flow (PDWF) = | 14 ML/d = | 159.32 L/s |
| Design Population (EP) = | 40000 | |
| Design Min. Dry Weather Flow (MDWF)= | 4 ML/d = | 51.39 L/s |
| Design Average Dry Weather Flow (ADWF)= | 12 ML/d = | 138.89 L/s |
| Design Peak Dry Weather Flow (PDWF) = | 20 ML/d = | 228.70 L/s |
| Design Peak Wet Weather Flow (PWWF) = | 27 ML/d = | 312.50 L/s |

**Single Pump Duty Range** 35-50 L/s min to 312.5 L/s max
Peak Effic 90 to 140 L/s

**Low Perf. Pump Duty Range** 35 L/s min to 230 L/s max
Peak Effic 90 to 140 L/s

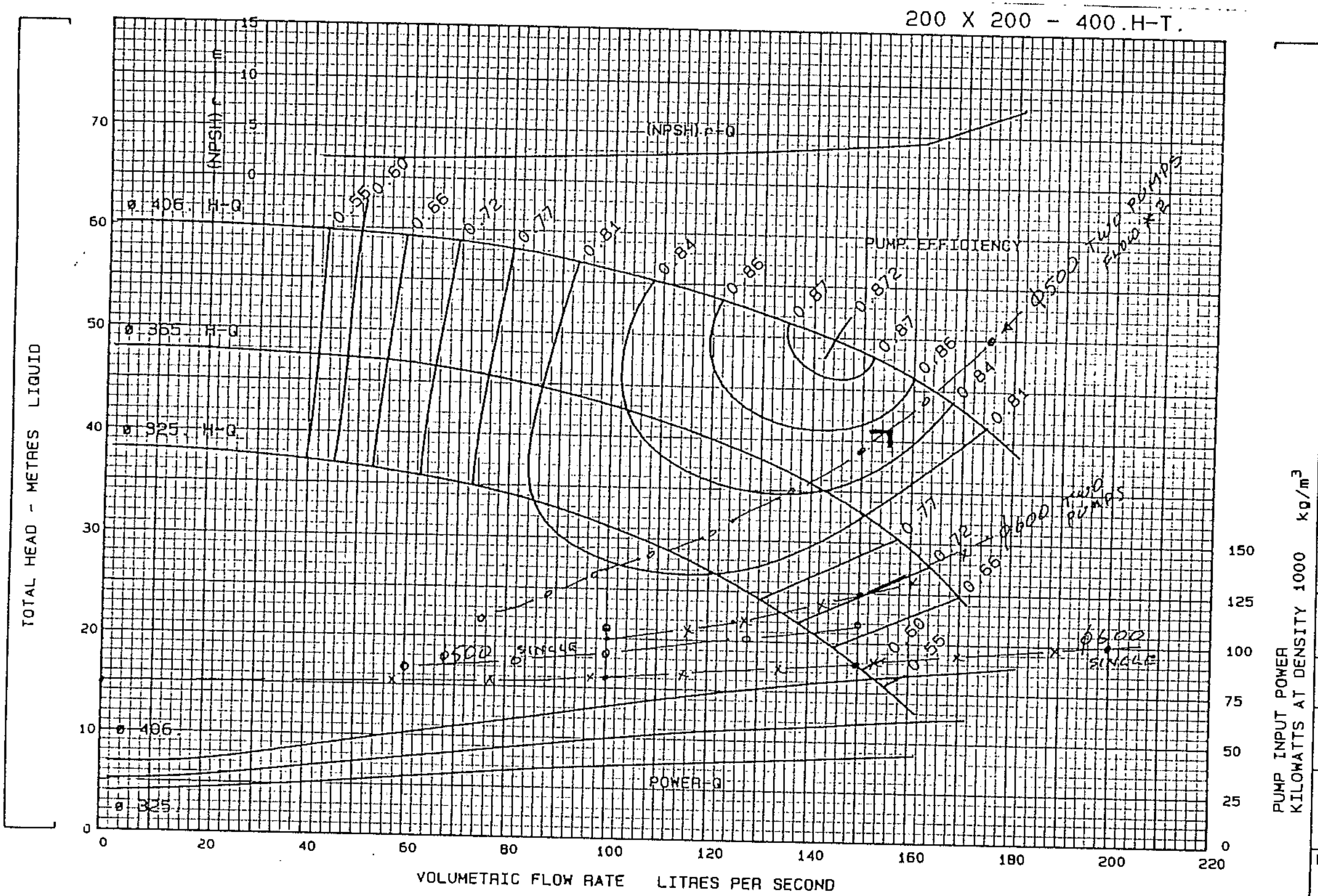
**High Perf. Pump Duty Range** 230 L/s min to 312.5 L/s max
Peak Effic 312.5 L/s

# SINGLE LARGE PUMP OPTION



# DUAL SMALL PUMP OPTION

200 X 200 - 400.H-T.




| | |
|---|---|
| CURVE No. | R24087 |
| DRAWN BY | K.L.C |
| DATE | 7 DEC88 |
| ISSUE | 2 |
| BASIS | E402 |
| SUPERSEDES | R24087 ISS 1 |
| CAS. HYD. | SD100324 |
| A/R | |
| IMP. HYD. | SD100323 |
| IMP. TYPE | ☐ OPEN ☒ CLOSED |
| MAX. SPHERE | 30 mm |
| Driver kW. | |
| Proposal No | |

**Thompsons Kelly&Lewis**

| SIZE and TYPE | r/min. | CURVE | ISSUE |
|---|---|---|---|
| 200 X 200 - 400.H-T. | 4Pole 50Hz Inherent Speed (1480Nominal) | No. R24087 | 2 |

Data refer to clean cold water.

No liability is accepted for any direct or consequential loss arising out of the use of this performance curve.

## Preliminary Budget Estimate - 10 September 1997

| Item | Description | Quantity | Unit | Rate $ | Dia 500 Amount $ |
|---|---|---|---|---|---|
| | **Effluent Rising Main** | | | | |
| 1.0 | Clearing & grubbing | 5700 | m | 1.00 | 5700.00 |
| 2.0 | Excavate Trenches | 5700 | m | 22.95 | 130815.00 |
| | Extra in medium/soft rock | 2100 | m | 25.00 | 52500.00 |
| | Dispose of excess spoil | 5700 | m | 4.50 | 25650.00 |
| 3.0 | Prepare base of trench | 5700 | m | 4.00 | 22800.00 |
| 4.0 | Fencing of excavation | 5700 | m | 3.00 | 17100.00 |
| 5.0 | Bedding | | | | |
| | - type 1 | 3950 | m | 25.20 | 99540.00 |
| | - type 2 | 1750 | m | 36.00 | 63000.00 |
| 6.0 | Backfill | | | | |
| | - in pavements | 3250 | m | 21.60 | 70200.00 |
| | - o/s pavements | 2450 | m | 13.50 | 33075.00 |
| 7.0 | Reinstatement | | | | |
| | - undeveloped areas | 1500 | m | 5.00 | 7500.00 |
| | - grassed areas | 1000 | m | 15.00 | 15000.00 |
| | - unsealed pavements | 700 | m | 25.00 | 17500.00 |
| | - bitumen sealing | 2500 | m | 40.00 | 100000.00 |
| | - existing services | 4700 | m | 14.00 | 65800.00 |
| 8.0 | Supply, lay & joint DN600 pipe | 5700 | m | 282.97 | 1612926.87 |
| 9.0 | Major Road Crossings | 90 | m | 440.00 | 39600.00 |
| 10.0 | U/G marking tape | 5700 | m | 1.50 | 8550.00 |
| 11.0 | Traffic Control | | Item | | 50000.00 |
| 12.0 | Drain/Creek Crossings | 3 | No. | 10000 | 30000.00 |
| 13.0 | Connection to Larrakeyah Outfall | | Item | | 50000.00 |
| 14.0 | Establishment & Miscellaneous | | Item | | 503451.37 |
| | **Total** | | | | $3,020,708.24 |
| | | | | **Adopt** | **$3,000,000.00** |
| | Pipeline Cost per metre (incl. eng. & cont.) = | | | | **$624** |
| | **Ludmilla Pumping Station** | | | | |
| 1.0 | Preliminaries | | Item | | 20000.00 |
| 2.0 | Relocate existing re-use filters | | Item | | 5000.00 |
| 3.0 | Conventional Dry well Pumps | 2 | No. | 37500.00 | 75000.00 |
| 4.0 | VSD drives | 2 | No. | 31250.00 | 62500.00 |
| 5.0 | Electrical power & Controls | | Item | | 62500.00 |
| 6.0 | Pipework and valves | | Item | | 70000.00 |
| 7.0 | Flow meter | | Item | | 12500.00 |
| | **Total** | | | | $307,500.00 |
| | | | | **Adopt** | **$330,000.00** |
| | TOTAL CONSTRUCTION BUDGET ESTIMATE | | | | **$3,330,000.00** |
| | Design and supervision (7%) | | | | **$233,100.00** |
| | Contingencies (10%) | | | | **$356,310.00** |
| | TOTAL | | | | **$3,919,410.00** |
| | | | | **Adopt** | **$3,900,000.00** |

## Preliminary Budget Estimate - 10 September 1997

| Item | Description | Quantity | Unit | Rate $ | Dia 600 Amount $ |
|---|---|---|---|---|---|
| | **Effluent Rising Main** | | | | |
| 1.0 | Clearing & grubbing | 5700 | m | 1.00 | 5700.00 |
| 2.0 | Excavate Trenches | 5700 | m | 25.50 | 145350.00 |
| | Extra in medium/soft rock | 2100 | m | 25.00 | 52500.00 |
| | Dispose of excess spoil | 5700 | m | 5.00 | 28500.00 |
| 3.0 | Prepare base of trench | 5700 | m | 4.00 | 22800.00 |
| 4.0 | Fencing of excavation | 5700 | m | 3.00 | 17100.00 |
| 5.0 | Bedding | | | | |
| | - type 1 | 3950 | m | 28.00 | 110600.00 |
| | - type 2 | 1750 | m | 40.00 | 70000.00 |
| 6.0 | Backfill | | | | |
| | - in pavements | 3250 | m | 24.00 | 78000.00 |
| | - o/s pavements | 2450 | m | 15.00 | 36750.00 |
| 7.0 | Reinstatement | | | | |
| | - undeveloped areas | 1500 | m | 5.00 | 7500.00 |
| | - grassed areas | 1000 | m | 15.00 | 15000.00 |
| | - unsealed pavements | 700 | m | 25.00 | 17500.00 |
| | - bitumen sealing | 2500 | m | 40.00 | 100000.00 |
| | - existing services | 4700 | m | 14.00 | 65800.00 |
| 8.0 | Supply, lay & joint DN600 pipe | 5700 | m | 350.00 | 1995000.00 |
| 9.0 | Major Road Crossings | 90 | m | 440.00 | 39600.00 |
| 10.0 | U/G marking tape | 5700 | m | 1.50 | 8550.00 |
| 11.0 | Traffic Control | | Item | | 50000.00 |
| 12.0 | Drain/Creek Crossings | 3 | No. | 10000 | 30000.00 |
| 13.0 | Connection to Larrakeyah Outfall | | Item | | 50000.00 |
| 14.0 | Establishment & Miscellaneous | | Item | | 589250.00 |
| | **Total** | | | | $3,535,500.00 |
| | | | | **Adopt** | **$3,500,000.00** |
| | Pipeline Cost per metre (incl. eng. & cont.) = | | | | **$730** |
| | **Ludmilla Pumping Station** | | | | |
| 1.0 | Preliminaries | | Item | | 20000.00 |
| 2.0 | Relocate existing re-use filters | | Item | | 5000.00 |
| 3.0 | Conventional Dry well Pumps | 2 | No. | 30000.00 | 60000.00 |
| 4.0 | VSD drives | 2 | No. | 25000.00 | 50000.00 |
| 5.0 | Electrical power & Controls | | Item | | 50000.00 |
| 6.0 | Pipework and valves | | Item | | 75000.00 |
| 7.0 | Flow meter | | Item | | 15000.00 |
| | **Total** | | | | $275,000.00 |
| | | | | **Adopt** | **$300,000.00** |
| | TOTAL CONSTRUCTION BUDGET ESTIMATE | | | | **$3,800,000.00** |
| | Design and supervision (7%) | | | | $266,000.00 |
| | Contingencies (10%) | | | | $406,600.00 |
| | TOTAL | | | | $4,472,600.00 |
| | | | | **Adopt** | **$4,500,000.00** |

Appendix D

## Annual Power Costs

## Annual Power Costs

Power costs = Peak kW/yr x $28/mth + 8c/kWh ( sum of day rate and night rate peak power monthly cost plus kW/hr cost)

Long Term Avge Load = 12 ML/d = 140 L/s        Assume pump Avge Load for 7 months & Peak Load for 5 months of year.
Peak Design Load = 27 ML/d = 312.5 L/s

Power cost for each pipe size option = .98*312.5*Hpk*28*12/Effic. + .98*140*Hav*24*365*(7/12)*.08/Effic. + .98*312.5*Hpk*24*365*(5/12)*.08/Effic.

| | | | | |
|---|---|---|---|---|
| D = (mm) | 450 | 500 | 600 | |
| H(312.5) = (m) | 60.55 | 41.80 | 26.79 | |
| H(140) = (m) | 24.89 | 20.86 | 17.47 | |
| Pump Efficiency | 80 | 80 | 80 | - TKL 200x200-400 HT, Actual effic. 83-87% @ 140 L/s |
| Annual Power Cost = ($) | 163027 | 115127 | 76663 | |

Appendix E

# Economic Analysis of Rising Main Options

**Present Worth Analysis**

| | | | | | | |
|---|---|---|---|---|---|---|
| D= (mm) | **450** | Cost ($/m) = | 570 | **Rate = (%)** | **5** | |
| Pipeline Cost | Annual Power Cost | | Pump Cost | NPW Power | NPW Pump | **Total Cost** |
| $3,249,000 | $163,027 | for | $283,102 | $3,125,016 | $370,883 | **$6,744,899** |
| | 50 | years | Replace Pumps in (yrs) = | | 25 | |

| | | | | | | |
|---|---|---|---|---|---|---|
| D= (mm) | **450** | Cost ($/m) = | 570 | **Rate = (%)** | **7** | |
| Pipeline Cost | Annual Power Cost | | Pump Cost | NPW Power | NPW Pump | **Total Cost** |
| $3,249,000 | $163,027 | for | $283,102 | $2,407,384 | $338,914 | **$5,995,299** |
| | 50 | years | Replace Pumps in (yrs) = | | 25 | |

| | | | | | | |
|---|---|---|---|---|---|---|
| D= (mm) | **450** | Cost ($/m) = | 570 | **Rate = (%)** | 10 | |
| Pipeline Cost | Annual Power Cost | | Pump Cost | NPW Power | NPW Pump | **Total Cost** |
| $3,249,000 | $163,027 | for | $283,102 | $1,778,019 | $311,844 | **$5,338,863** |
| | 50 | years | Replace Pumps in (yrs) = | | 25 | |

**Present Worth Analysis**

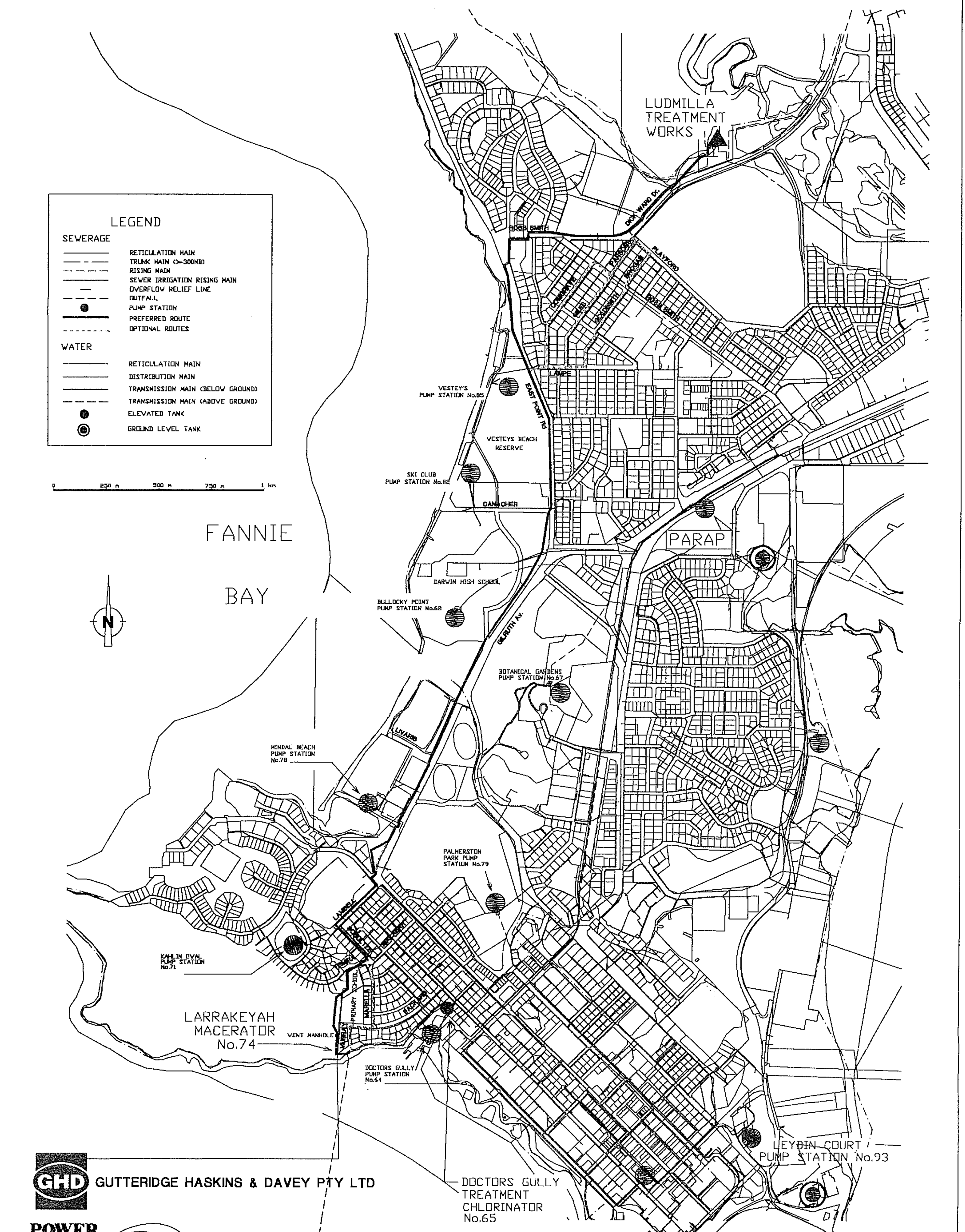
| | | | | | | |
|---|---|---|---|---|---|---|
| D= (mm) | **500** | Cost ($/m) = | 624 | **Rate = (%)** | **5** | |
| Pipeline Cost | Annual Power Cost | | Pump Cost | NPW Power | NPW Pump | **Total Cost** |
| $3,556,800 | $115,127 | for | $235,226 | $2,206,829 | $308,162 | **$6,071,791** |
| | 50 | years | Replace Pumps in (yrs) = | | 25 | |

| | | | | | | |
|---|---|---|---|---|---|---|
| D= (mm) | **500** | Cost ($/m) = | 624 | **Rate = (%)** | **7** | |
| Pipeline Cost | Annual Power Cost | | Pump Cost | NPW Power | NPW Pump | **Total Cost** |
| $3,556,800 | $115,127 | for | $235,226 | $1,700,051 | $281,600 | **$5,538,451** |
| | 50 | years | Replace Pumps in (yrs) = | | 25 | |

| | | | | | | |
|---|---|---|---|---|---|---|
| D= (mm) | **500** | Cost ($/m) = | 624 | **Rate = (%)** | **10** | |
| Pipeline Cost | Annual Power Cost | | Pump Cost | NPW Power | NPW Pump | **Total Cost** |
| $3,556,800 | $115,127 | for | $235,226 | $1,255,604 | $259,108 | **$5,071,512** |
| | 50 | years | Replace Pumps in (yrs) = | | 25 | |

**SUMMARY**

| Pipe Dia (mm) | 450 | 500 | 600 |
|---|---|---|---|
| NPV (Discount rate 5%) | $6,744,899 | $6,071,791 | $5,877,249 |
| NPV (Discount rate 7%) | $5,995,299 | $5,538,451 | $5,518,518 |
| NPV (Discount rate 10%) | $5,338,863 | $5,071,512 | $5,204,552 |




GHD GUTTERIDGE HASKINS & DAVEY PTY LTD

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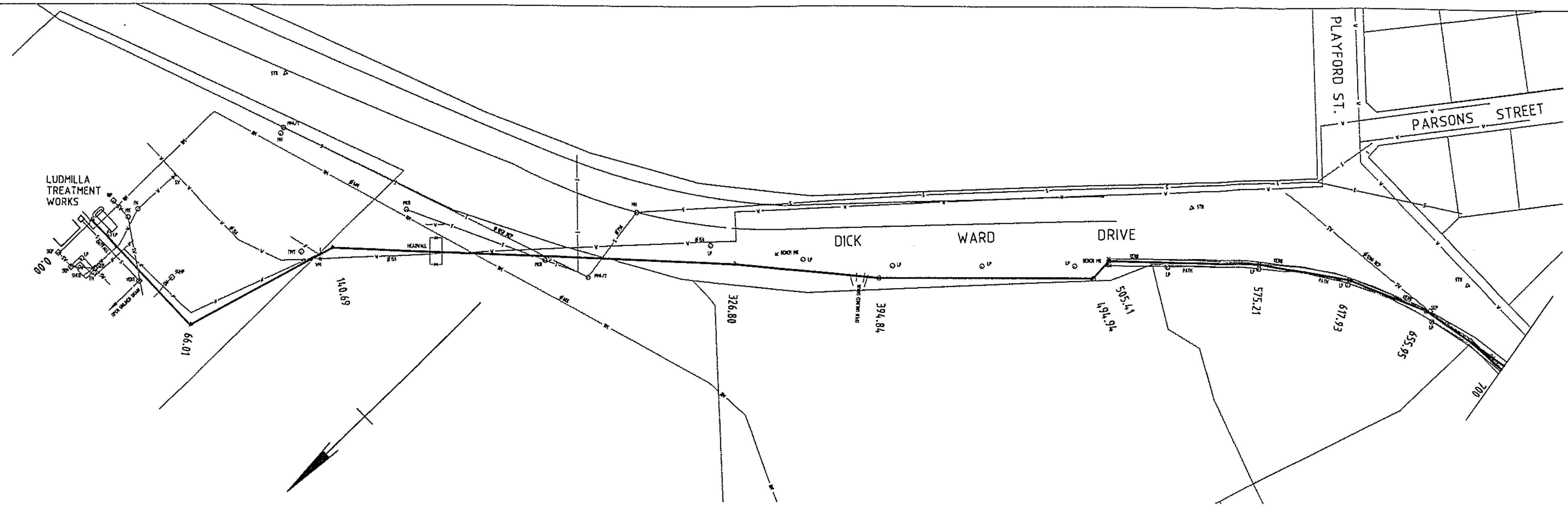
NETWORK SERVICES

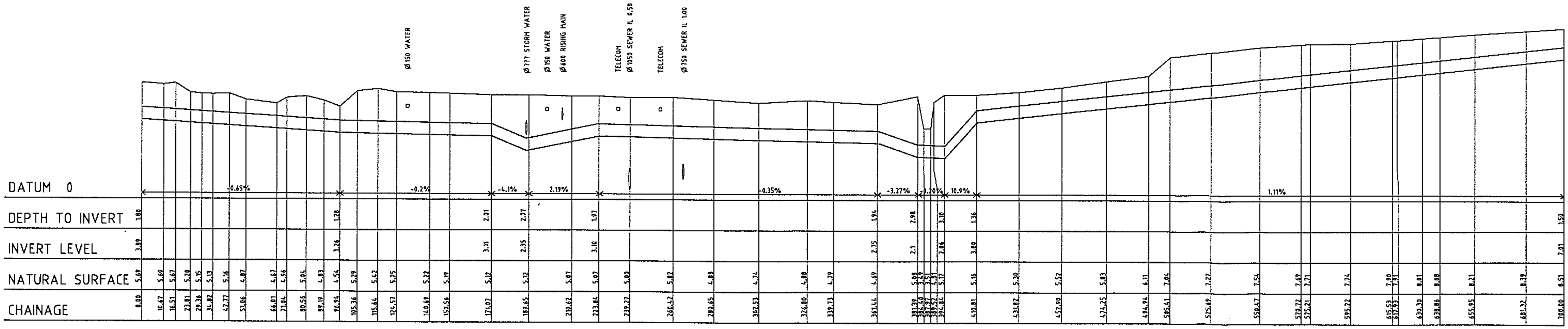
## LUDMILLA TO LARRAKEYAH EFFLUENT RISING MAIN

**Figure 1**

LTLERM.DGN

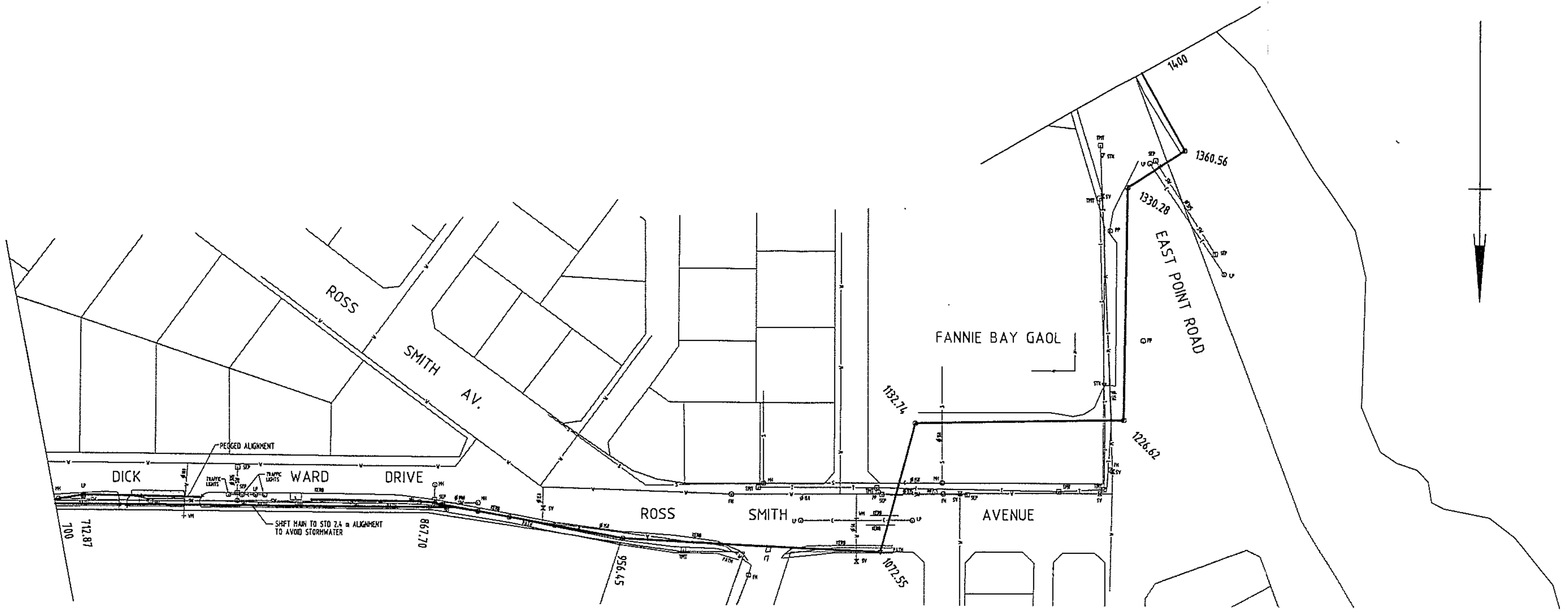



PLAN
1:1000




LONGITUDINAL SECTION CH 00 - 700
SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F10.DWG

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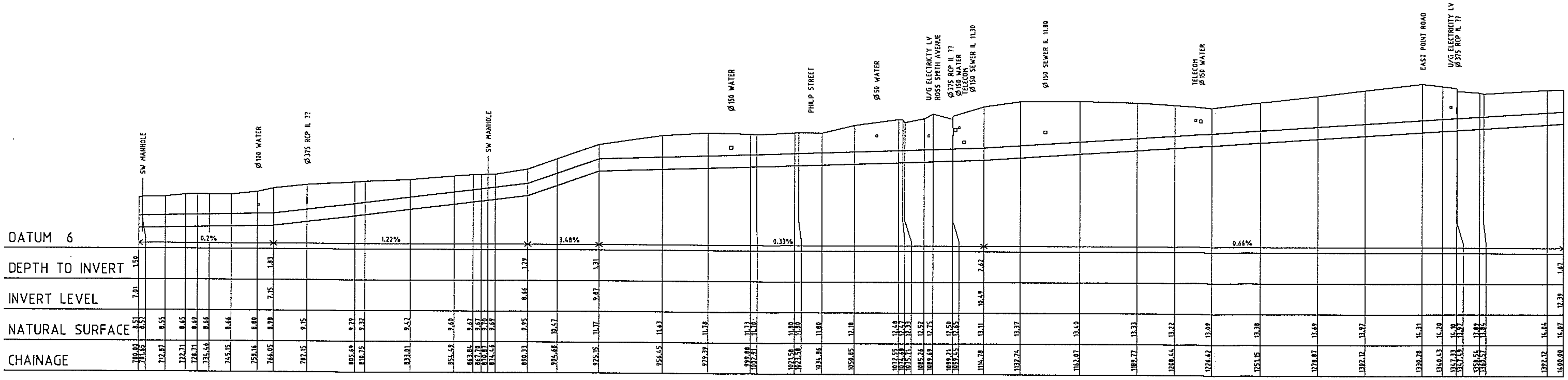
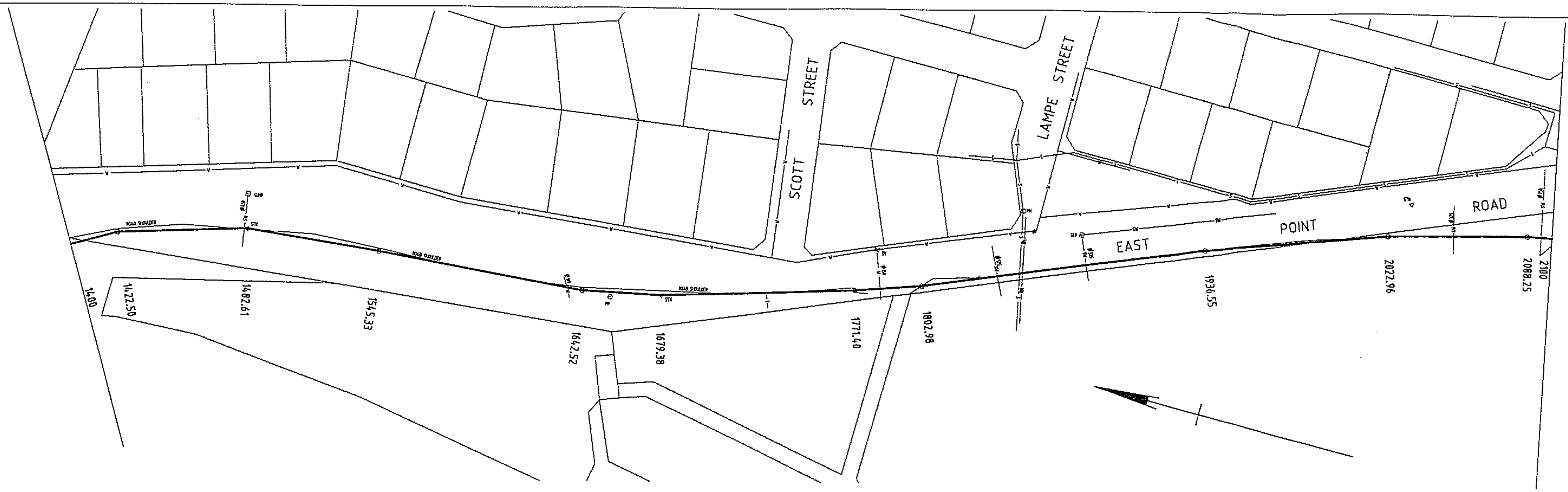



LONGITUDINAL SECTION CH 700 - 1400
SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F11.DWG




PLAN
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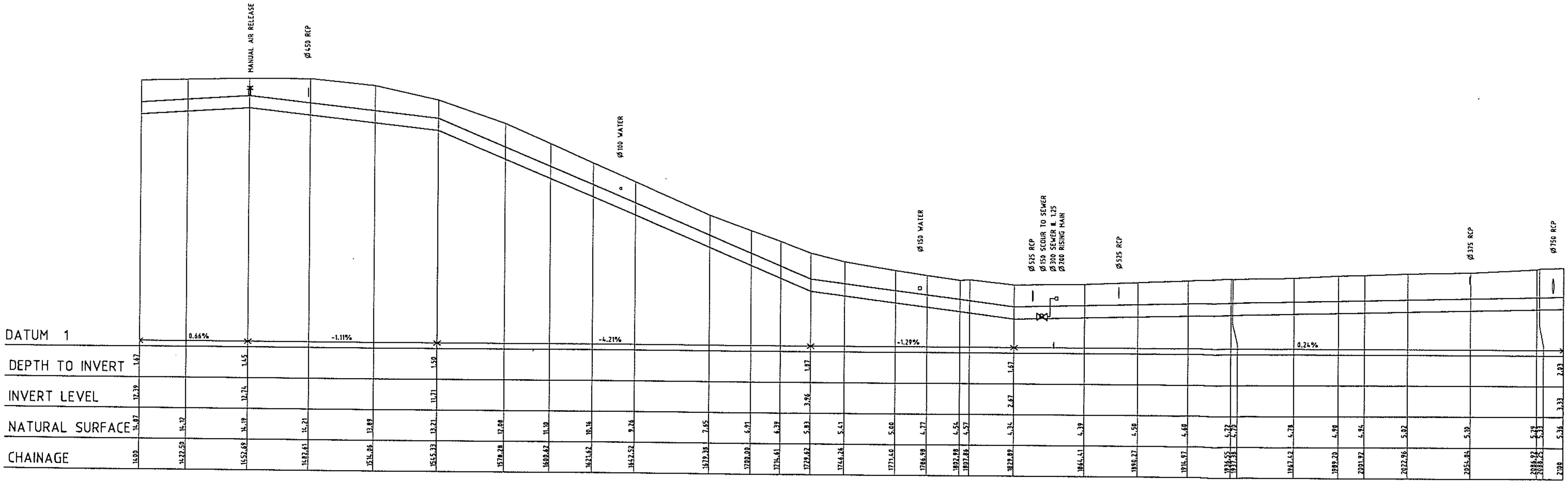
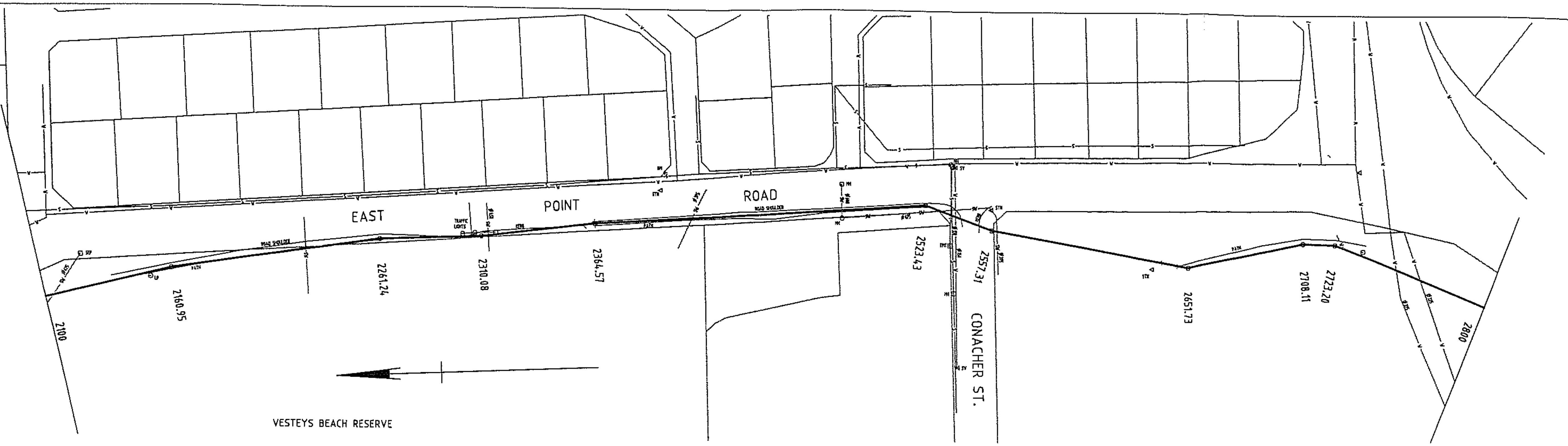



LONGITUDINAL SECTION CH 1400 - 2100

SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F12.DWG




PLAN
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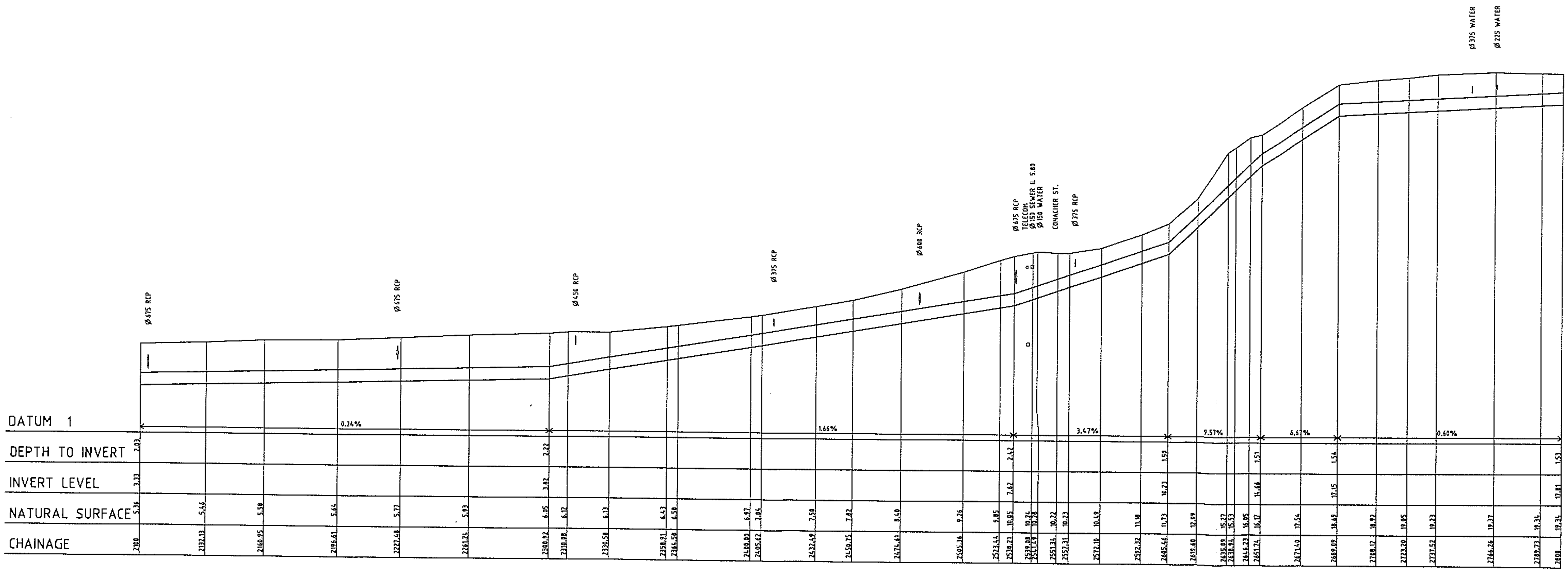
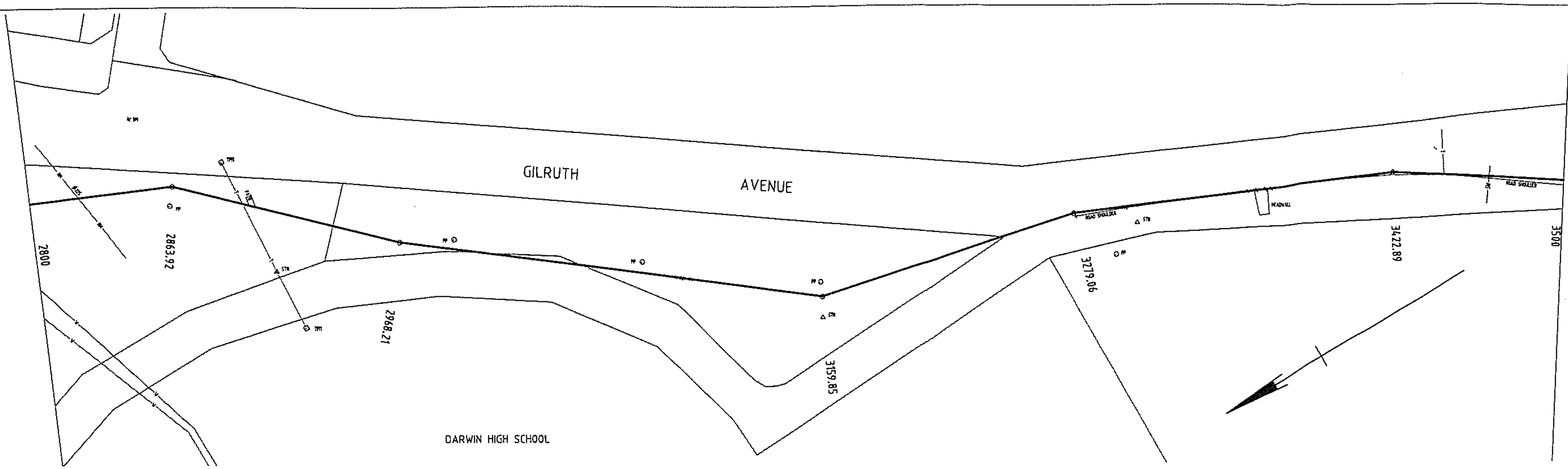



LONGITUDINAL SECTION CH 2100 - 2800
SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F13.DWG




PLAN
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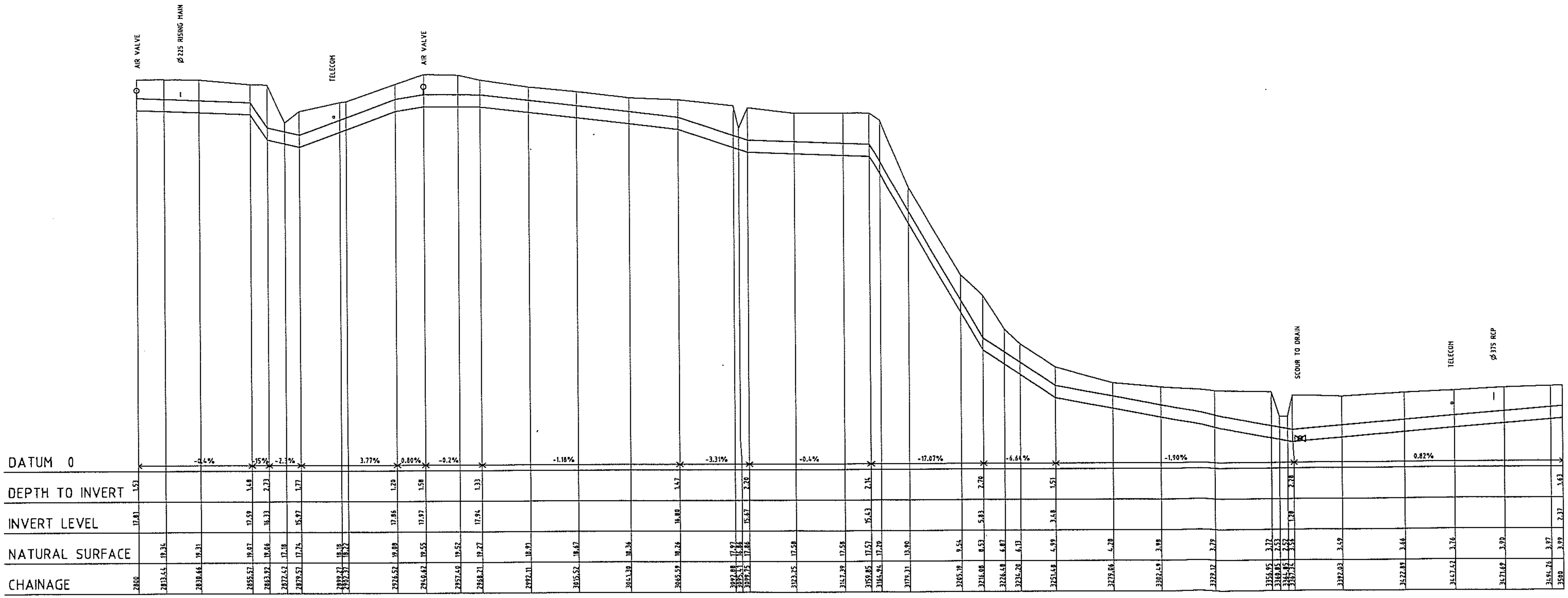
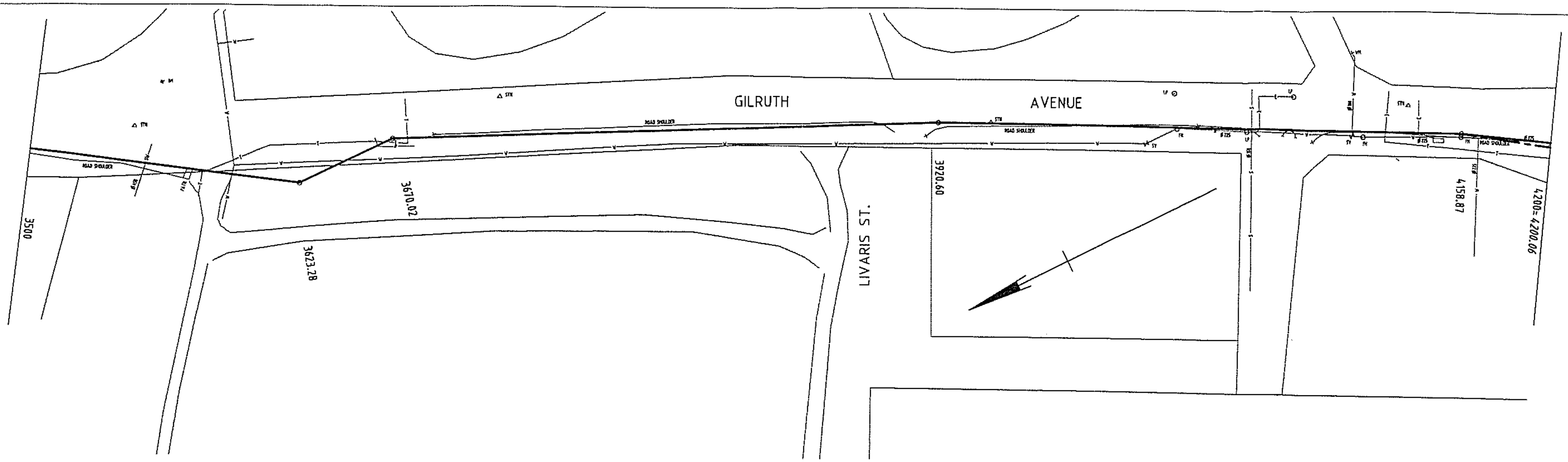


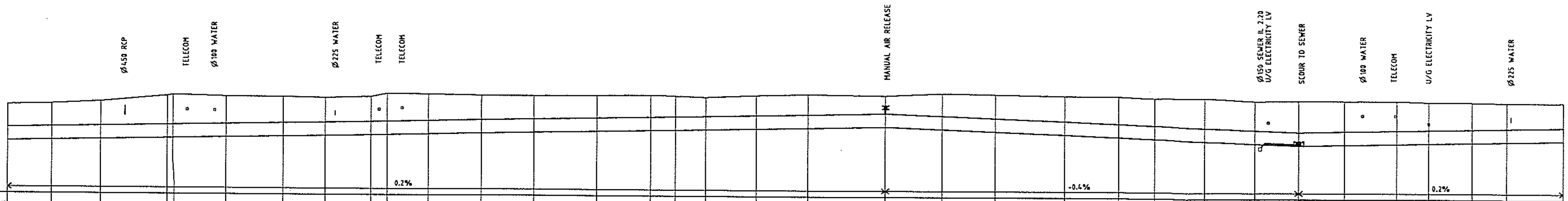

LONGITUDINAL SECTION CH 2800 - 3500

SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F14.DWG




PLAN
1:1000

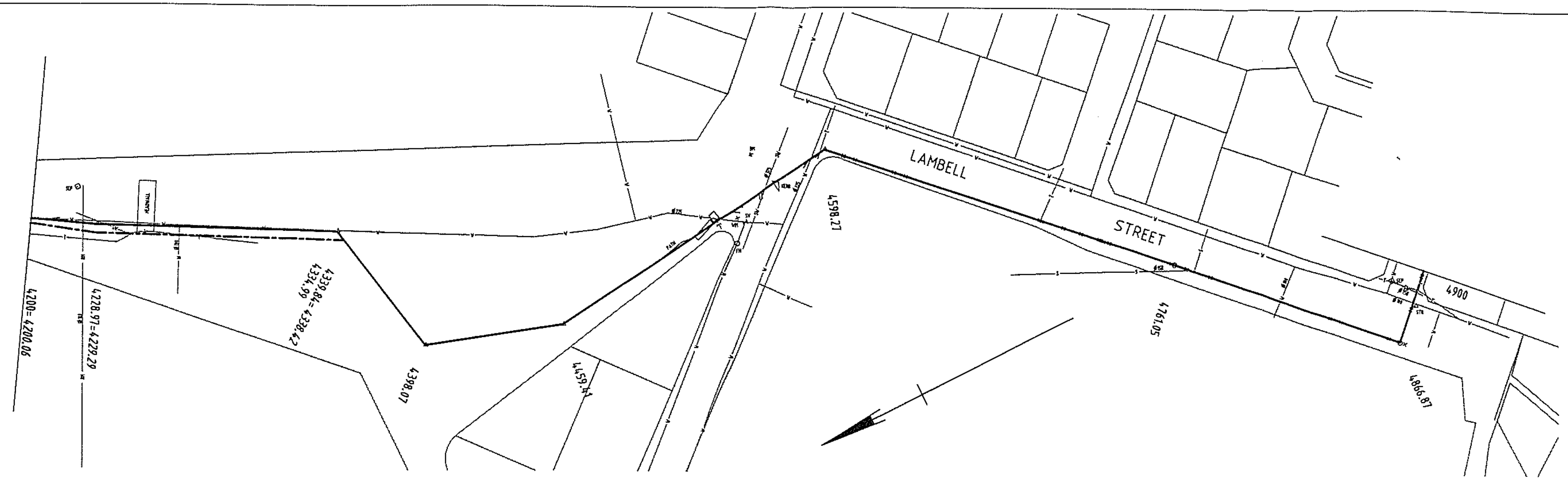



| CHAINAGE | NATURAL SURFACE | INVERT LEVEL | DEPTH TO INVERT |
|---|---|---|---|
| 3500 | 3.99 | 2.37 | 1.63 |
| 3519.83 | 4.04 | | |
| 3544.81 | 4.14 | | |
| 3571.72 | 4.42 | | |
| 3574.24 | 4.46 | | |
| 3597.79 | 4.40 | | |
| 3623.28 | 4.39 | | |
| 3642.09 | 4.36 | | |
| 3662.57 | 4.42 | | |
| 3670.02 | 4.56 | | |
| 3688.74 | 4.54 | | |
| 3712.63 | 4.49 | | |
| 3736.35 | 4.48 | | |
| 3764.96 | 4.51 | | |
| 3789.16 | 4.53 | | |
| 3800.00 | 4.50 | | |
| 3813.89 | 4.45 | | |
| 3836.84 | 4.54 | | |
| 3860.29 | 4.60 | | |
| 3895.04 | 4.55 | 3.16 | 1.40 |
| 3920.60 | 4.67 | | |
| 3944.46 | 4.63 | | |
| 3975.26 | 4.57 | | |
| 3999.60 | 4.57 | | |
| 4015.80 | 4.50 | | |
| 4038.66 | 4.52 | | |
| 4060.91 | 4.40 | | |
| 4080.85 | 4.42 | 2.41 | 2.01 |
| 4101.27 | 4.42 | | |
| 4124.89 | 4.43 | | |
| 4139.35 | 4.38 | | |
| 4158.87 | 4.38 | | |
| 4174.67 | 4.35 | | |
| 4200 | 4.35 | 2.65 | 1.70 |

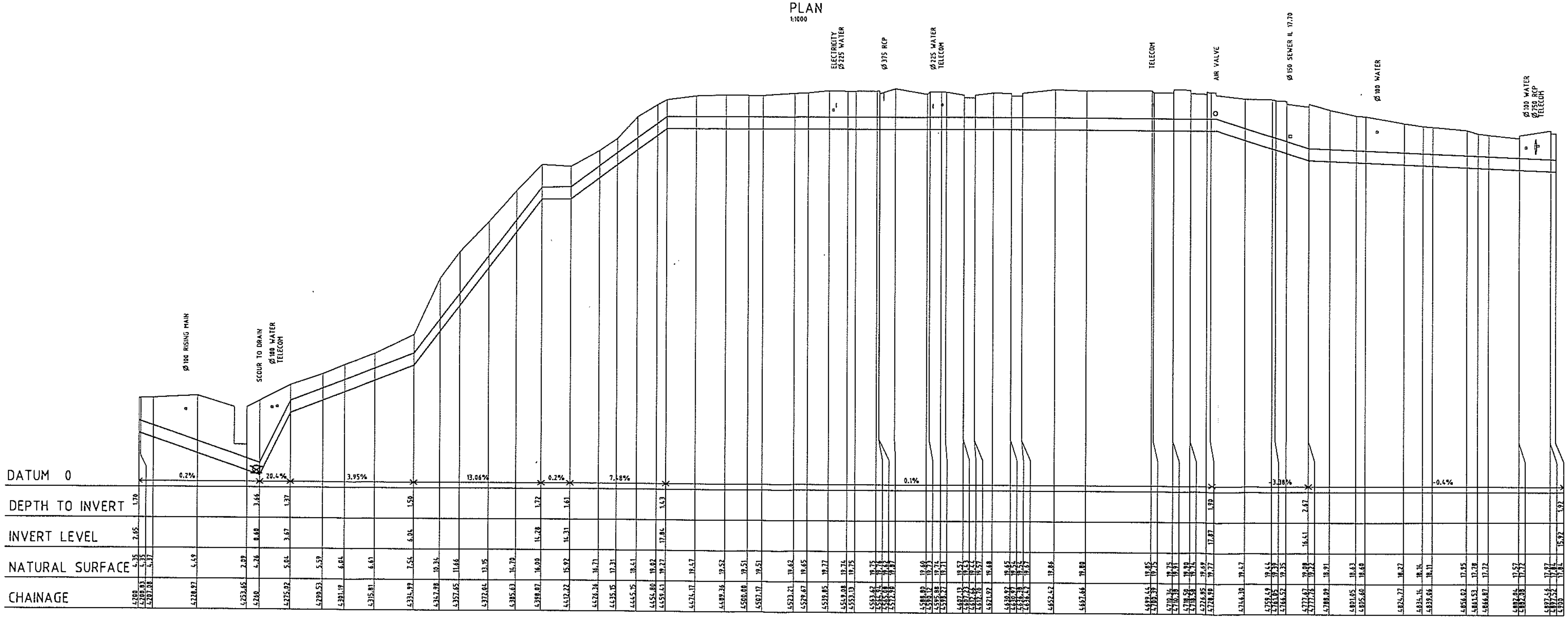
LONGITUDINAL SECTION CH 3500 - 4200

SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F15.DWG



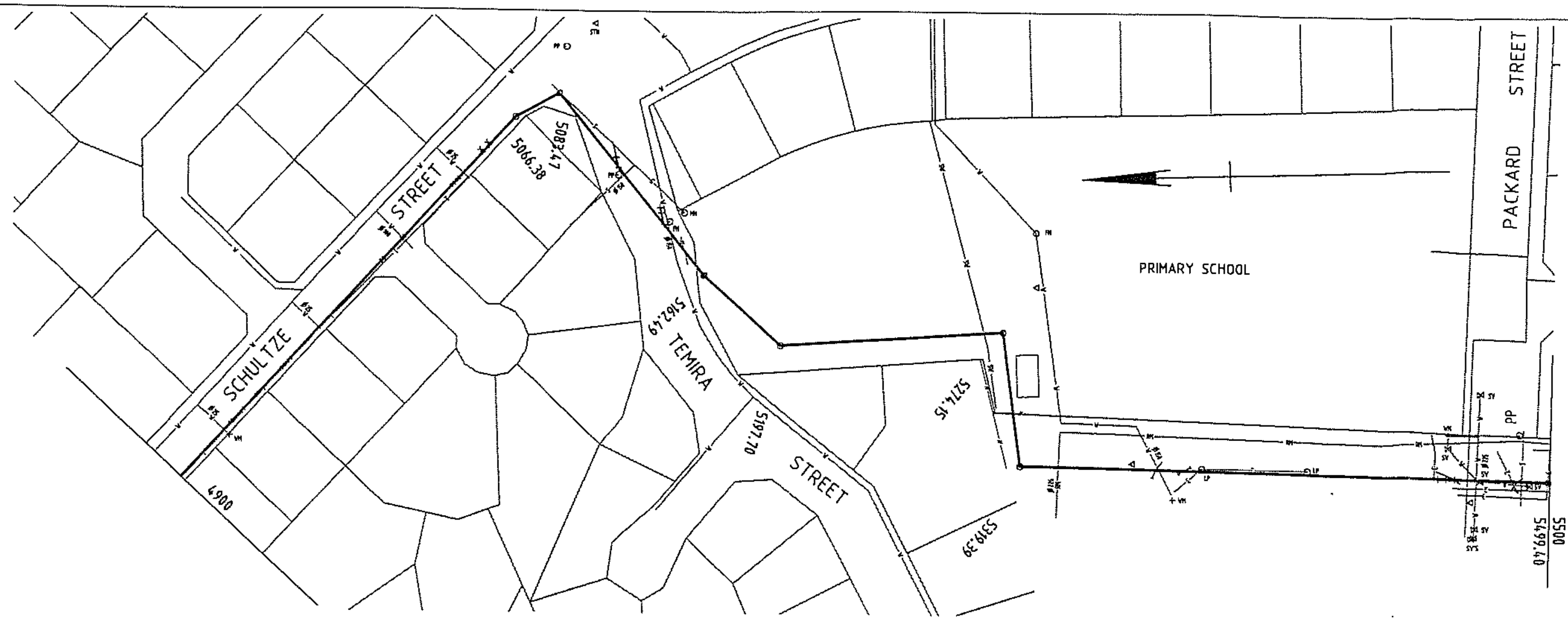

PLAN
1:1000



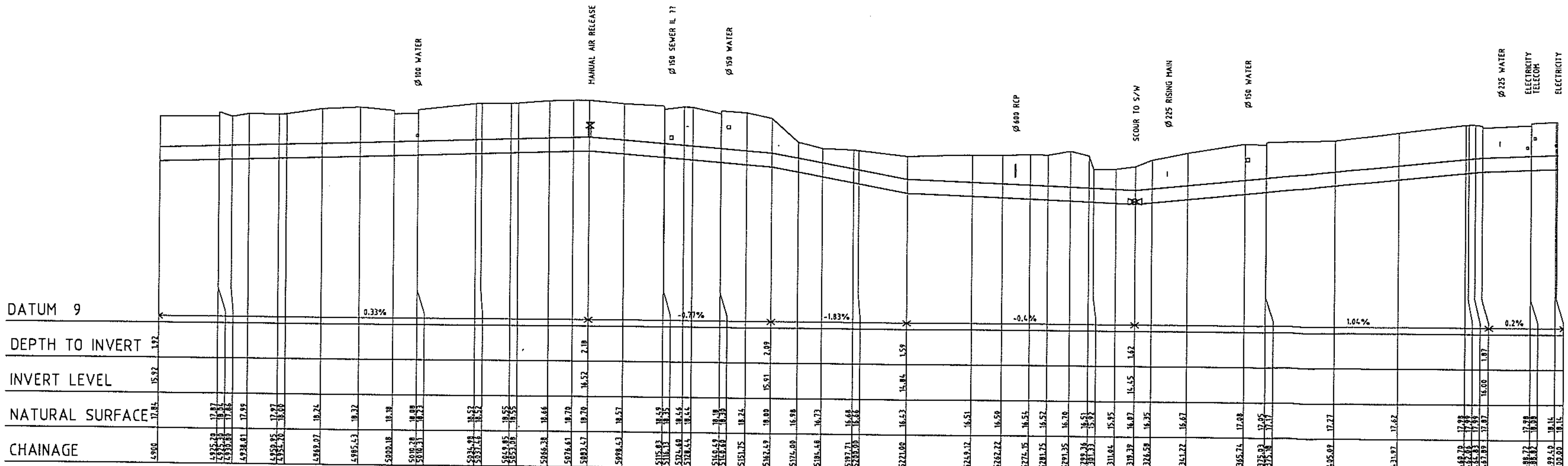

LONGITUDINAL SECTION CH 4200 - 4900

SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F16.DWG



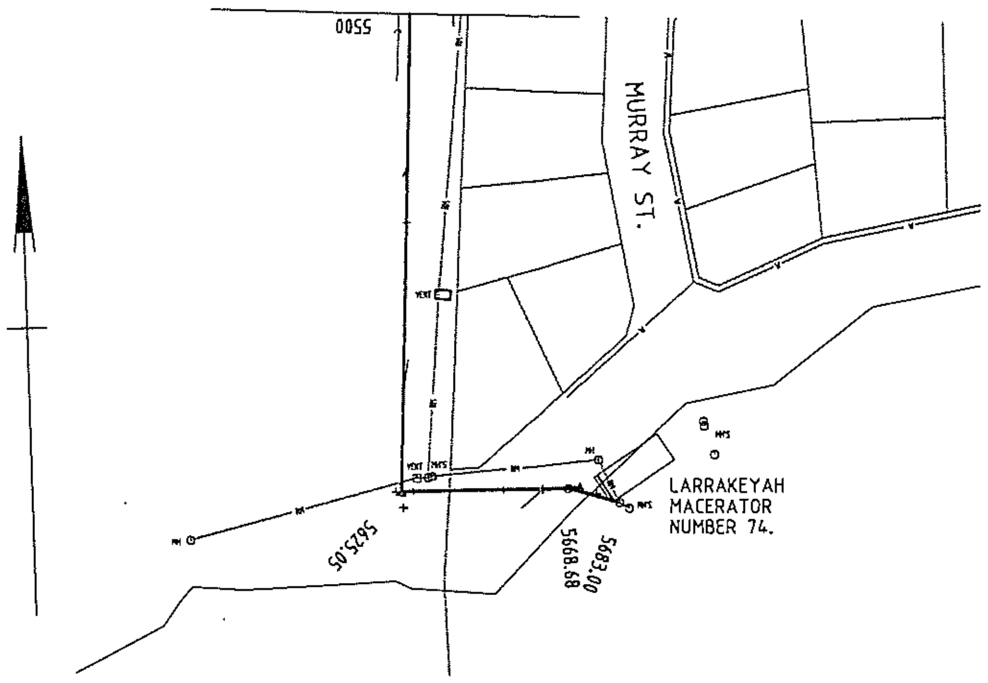

PLAN
1:1000




LONGITUDINAL SECTION CH 4900 - 5500

SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F17.DWG




PLAN
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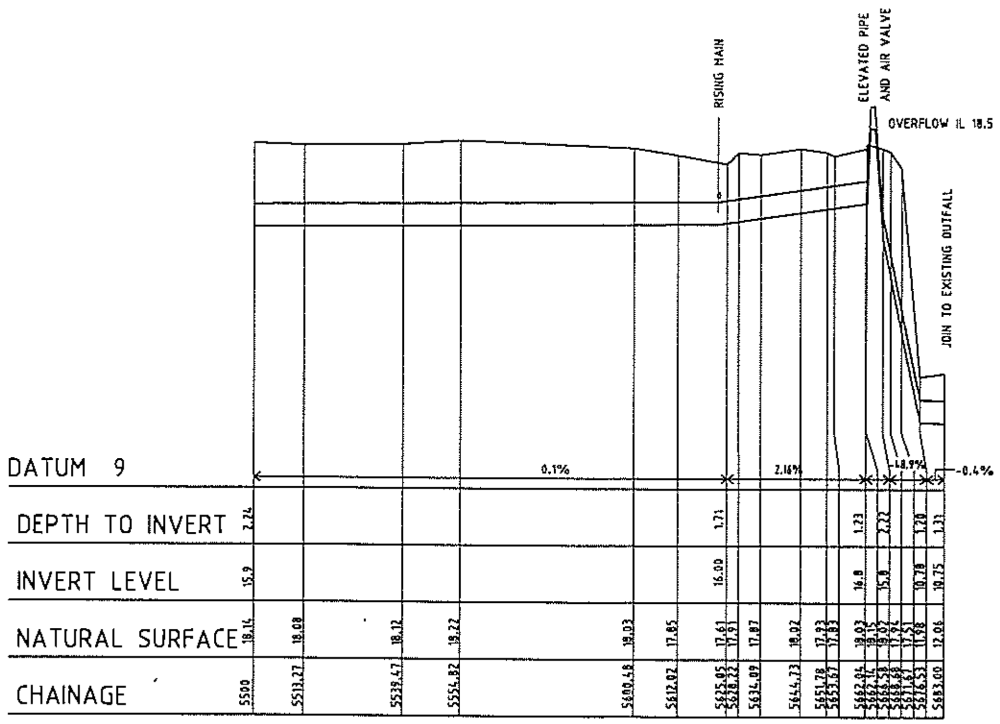


LONGITUDINAL SECTION CH 5500 - 5683.80
SCALES HOR 1 : 1000 VER 1 : 100

POWER AND WATER AUTHORITY
LUDMILLA TO LARRAKEYAH EFFLUENT
RISING MAIN
19277F18.DWG