DEPARTMENT OF PRIMARY INDUSTRIES

QUEENSLAND

WATER COMMERCIAL

Wivenhoe Dam Design Report
Volume 1 - Text

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WIVENHOE DAM DESIGN REPORT

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WIVENHOE DAM DESIGN REPORT

1.0 INTRODUCTION

1.1 The Project

Wivenhoe Dam is located at AMTD 150.2 km on the Brisbane River and commands 7020 sq km of the river's catchment. The main functions of the dam are to:

(a) meet the increase in water supply demands of the Moreton Region to the end of the 20th century,

(b) provide the lower pool for the 500 MW Wivenhoe Pumped Storage Project; and

(c) provide flood mitigation for the cities of Brisbane and Ipswich.

The Moreton Region is defined as "the catchment area of all streams entering the Pacific Ocean and Moreton Bay from and including the Caboolture River to the New South Wales Border together with the islands adjacent thereto" (Reference 1).

The location and layout of Wivenhoe Dam are shown in Figure 1 (Drawing A2-47529F) and Figure 2 (Drawing A1-50820B). The dimensions of the principal features of the dam are detailed in Appendix A.

1.2 Design Report

This report is submitted as a record of the design of Wivenhoe Dam and the investigations and studies performed to substantiate the design parameters used and decisions made. With appropriate references to data sources and to field and office investigations, it sets out in some detail:

(a) The criteria and assumptions used in design;
(b) The reasoning which led to the type and arrangement of the structures adopted;
(c) The basis of the design of the individual structures;
(d) The design changes made during construction and the reasons for the changes.

The design was carried out by a team of engineers from the Design Division of the Water Resources Commission (formerly the Irrigation and Water Supply Commission) with significant input from other specialist branches within the Commission. The construction contracts were administered by Construction Division of the Commission.

1.3 Some Aspects of Design Report Superseded by Subsequent Studies

Subsequent to the design and construction of Wivenhoe Dam several conclusions and figures quoted in this report have been superseded by further studies. These studies were carried out as part of an overall dam safety review process commissioned by the South East Queensland Water Board. The main areas affected are:- hydrological data including estimates of Probable Maximum Flood, Probable Maximum Precipitation, runoff, routing, and flood levels; wave run up; gate operating procedures; impact of downstream flooding; etc. A list of reports on the Brisbane River and Pine Rivers Flood Study, as at December 1994, is included in Appendix B. A summary of the conclusions and recommendations derived from the flood study is included in the Brisbane River and Pine River Flood Study Initial Draft Report Executive Summary (Reference 2).

The storage volume and submerged area of Wivenhoe Dam have been recalculated. Details are shown on Drawings A3 - 110404 and A3 - 110405, copies of which have been forwarded to the South East Queensland Water Board under separate cover.

Where appropriate, data updated since the design of the dam have been included, or referred to, in this report.

2.0 PROJECT HISTORY

2.1 Preliminary Investigations and Designs

The Brisbane River Basin is the major water supply source for the City of Brisbane and many of the nearby local authorities. The major floods which are derived from the basin have a history of causing significant damage to the local communities.

Somerset Dam was constructed on the Stanley River over the period 1936 to 1954 and has been the major regional water supply source. Studies carried out by the Co-ordinator General's Department identified the need to augment the regional water supply capacity and provide additional flood mitigation on the unregulated sub-basin of the Brisbane River itself (Reference 1)

Shortly after the 1974 floods, the Co-ordinator General's Department was declared the Construction Authority for the Wivenhoe Project, which involved the construction of a dam at Wivenhoe for flood mitigation and water supply within the Moreton Region. The dam would also provide a lower pool for a pumped storage hydro-electric scheme. A full report and discussion on the 1974 floods and the mitigating effects of the construction of Wivenhoe Dam are included in the Proceedings of Symposium on the January 1974 floods, Moreton Region, (Reference 3).

The actual responsibilities for the detailed investigation and design and subsequent construction supervision were delegated to the Water Resources Commission (then the Irrigation and Water Supply Commission).

These responsibilities covered the following:

(a) Investigation of the site and material sources;
(b) Preparation of preliminary designs and estimates;
(c) Preparation of final designs including estimates, tender drawings, specifications, and working drawings; and
(d) Establishment of a construction village and supervision of construction.

The preliminary investigations were carried out by the Co-ordinator General's Department between 1968 and 1970. The resulting report recommended the site (AMTD 150.2 on the Brisbane River) and a full supply level (Reference 1). These recommendations were accepted as being fixed for the detailed investigation and design carried out by the Commission.

Preliminary designs were formulated from late 1974 and design proposals were made to an interdepartmental committee which had been formed to review and make final decisions on design proposals submitted by the designing authority (References 4 and 5) . The Committee was chaired by the Chief Designing Engineer of the Water Resources Commission and included representatives from the Water Resources Commission, The Co-ordinator General's Department, The Local Government Department, the Brisbane City Council, the State Electricity Commission and the Esk Shire.

2.2 Contracts

Because of the size of the project and of the necessity to continue foundation and material investigations during the construction period, the dam was constructed under a number of contracts, which were basically delineated by the major activity of the contract. The concurrent design and construction of the Wivenhoe Pumped Storage Scheme placed further constraints on the availability and choice of natural construction materials. Detailed designs and specifications were produced for the contracts listed in Appendix C.

2.3 Designs

The principal permanent works designed, and described in this report, consist of the following:

PERMANENT WORK			DESCRIPTION
(a)	Main Embankment	:	Rockfill embankment across river channel with central impervious clay core.
(b)	Left Embankment	:	Sloping core embankment on the abutment to the left of the spillway.
(c)	Spillway	:	Gated concrete flip spillway in deep excavated approach channel with plunge pool in deeper discharge channel.
(d)	Outlet Works	:	Intake structure, dual penstocks and downstream regulating valves in the left retaining wall of spillway.
(e)	Flood Gates	:	Five radial gates with hydraulic controls and a bulkhead gate.
(f)	Service Facilities	:	Gantry crane and bridge, highway bridge.
(g)	Saddle Dams	:	Two low homogeneous fill embankments for flood storage.

The major detailed design work concluded in 1982. Minor design changes were necessary during the construction of the radial gates and minor design activity continued until 1984. The embankments are constructed of fill which will settle and move as it consolidates. Instruments have been provided to monitor the behaviour of the embankments and spillway. In the short term they monitor the reservoir filling, primary settlement and construction induced pore pressures. Further details on instrumentation are included in Section 28 of this report.

3.0 PROJECT SPECIFICATIONS

3.1 Project Requirements

The Co-ordinator General's Department carried out preliminary investigations to determine the major design parameters (Reference 1). These formed the framework within which the detailed design was formulated and following consideration of the Co-ordinator General's recommendations the design of the dam proceeded on the following basis:

(a) Reservoir Levels

Full Supply Level (FSL)	EL 67
Reservoir Resumption Level	EL 75
Maximum Flood Level	EL 77

(b) Spillway Capacity

(i) 3600 cumecs minimum discharge at FSL with all gates open.

(ii) Flood storage above FSL must be discharged within six days of the peak water level being attained.

(iii) Degree of flood mitigation to be examined during the design.

(c) General

(i) Alignment of dam axis to be determined after site investigation.

(ii) Material investigations to be carried out for the final design selected.

(iii) Hydraulic model testing to be carried out as required for the final design.

(iv) Hydrological investigations to be carried out to more accurately determine yield and design floods.

3.2 Program Requirements

The design and construction schedules were affected by physical and monetary restraints which manifested themselves as the following requirements:

(a) Stage storage was to be possible to the fixed concrete crest (EL 57) by the end of 1982;

(b) The radial gates were to be complete, and storage to full supply level was to be possible by mid 1983. (This was extended by one year to mid 1984 because of a delay in Contract 2112).

(c) The timing of construction of the dam was to be co-ordinated with that of the pumped storage power station to avoid flooding it;

(d) The foundation conditions would not be known until the overburden was removed. The design of the embankments, particularly the cross river embankment, could not be finalised until these conditions were known. It was desirable to call separate contracts for the excavation of the foundations (Contracts 2103 and 2112), which in turn necessitated the inclusion of the diversion channel excavation and coffer dam construction within these contracts.

3.3 Existing Data

In addition to establishing the key design parameters, the Co-ordinator General's Department had accumulated a large amount of relevant data. This was made available to the design authority and included all the data for the original feasibility study (Reference 1), namely:

(a) Preliminary Designs;
(b) Topographic survey data of sites and reservoir;
(c) Geological and foundation reports including bore logs;
(d) Flood and Yield Hydrology Studies; and
(e) Flood damage studies for Brisbane and Ipswich.

4.0 PRELIMINARY DESIGN

4.1 Scope of Studies

The preliminary design studies for Wivenhoe Dam were directed towards the following aims:

- to satisfy functional requirements;
- to find structurally adequate solutions;
- to find the most economical arrangement.

In most cases these studies involved comparison of a number of alternatives. The principal studies carried out were the following:

(a) A comparison of the site at 150.2 km with one about 1 km upstream;

(b) Schemes with different spillway locations;

(c) Alternative ways of handling river diversion;

(d) Number and size of spillway gates;

(e) Type of dissipator;

(f) Outlet works layouts including provision for possible future hydro-electric powerstation;

(g) Fish ladder studies;

(h) Studies for fuse-plug or emergency spillways; and

(i) Embankment design studies.

The preliminary design studies which led to the adoption of the main features of the design are reported in Reference 4.

4.2 Selection of Axis

The Co-ordinator-General's report of June 1971 identified three possible dam axes, designated A, B and C, at the Wivenhoe Dam Site (Reference 1). Axis B was eliminated because of the great depth of alluvium over much of the higher alluvial terrace on the left bank of the Brisbane River.

Whilst the origins of axes A and C were common high on the left bank of the Brisbane River, Axis C was located approximately 1 km downstream of Axis A on the right bank.

After extensive investigations Axis C was selected as the centreline of the dam embankment for the following reasons:-

(a) the foundation level for the impervious core in the river bed section was much deeper for Axis A leading to higher dewatering and cofferdam costs and longer delays should the cofferdams be overtopped;
(b) siting of the spillway on either bank of the river was possible for Axis C, but limited to the left bank on Axis A;
(c) drilling on the right bank indicated that rock on Axis C was more suitable for the location of the spillway and possible diversion and/or outlet tunnel than on Axis A;
(d) construction of the dam was less expensive on Axis C.
(e) a suitable clay borrow area, subsequently designated WX11, existed between Axes A and C on the left bank of the Brisbane River. Aesthetic considerations favoured Axis C as the depleted borrow area would be inundated upstream of Axis C, or exposed to view downstream of Axis A.
(f) slightly more storage was available from a dam constructed to the same level on the downstream Axis C.

Ajustments to the location of Axis C were made after final location of the spillway axis so that:-

- the dam axis crossed the spillway axis at right angles;
- the overall costs of embankment construction were minimised;
- a smooth transition was made with the existing road alignment on the left bank;
- the crest of the dam was aligned with the spur on the right bank; and
- the high clay terrace just upstream of the dam was avoided.

Adjustments to the original C axis to accommodate these factors are shown in Figure 3 (Drawing K1-45754D).

4.3 Alternative Schemes

The nature of the site favoured an embankment dam with a spillway in one abutment. Suitable spillway sites were considered to exist in either abutment. Of the three schemes considered in detail, two had spillways sited on the left bank and one had the spillway located in the right bank. In addition, a dam with a spillway in the main river channel was considered, but the required large volume of concrete in the spillway and flanking connecting sections to the embankment made this scheme uneconomic.

The schemes considered in detail were as follows:

(a) Proposal 1 had a spillway located high on the left abutment and founded at a high level. The outlet works were located in the right bank and consisted of an intake feeding two tunnels, one for the hydro-electric powerstation and one for the river outlet. The tunnels would also act as the river diversion during construction. This scheme resulted in the least volume of concrete of all the schemes examined, but since weak seams existed below the foundation level of the proposed spillway structure, the latter would have to rely on prestressed anchor bars for stability against sliding. The tunnels on their own were considered to have insufficient capacity for river diversion (the average annual flood frequency peak being 7000 m^3/sec), so a temporary diversion channel between the tunnels and the river was added. This scheme was rejected as being too expensive. A layout of the proposal is shown in Figure 4 (Drawing DB1889).

(b) Proposal 2 had a spillway located in the right bank close to the river with high retaining walls on the left side of the spillway to support the embankment over the river section. These retaining walls incorporated the outlet works. Diversion was to be provided by leaving the two spillway monoliths closest to the river low until the embankment was well advanced. A deepened portion of the spillway approach channel would act as the diversion channel during construction and finally as the intake channel.

Proposal 2 was the best scheme examined in regard to diversion during construction, but the existence of a thick shale layer in the right bank lowered the foundation level for the spillway monoliths and raised the volume of concrete. Although more economical than Proposal 1 it was more expensive than the scheme finally adopted. A layout of the proposal is shown in Figure 5 (Drawing DB1888).

(c) The preliminary design from which the final design evolved was called Proposal 5 and consisted of a dam with the spillway in the left abutment in the same position as for Proposal 1 but founded lower (at EL 36) to get below a known weak seam and thus avoid any prestressing. The outlet works were incorporated in the left wall of the spillway with a deep narrow intake channel extended upstream below the general floor level of the spillway approach channel. Diversion of the river during the early stages of embankment construction would be through a deep temporary diversion channel cut into the sandstone on the right bank of the river. This would be supplemented later by the low channel in the spillway excavation as it was progressively excavated to provide fill for the main embankment. The overflow monolith on the left side of the spillway would be left low to enable it to act as a supplementary diversion. It would become the sole diversion during closure of the right bank diversion channel for completion of the main embankment. The dam would be completed by concreting the low spillway monolith. This scheme proved the most economical. A layout of the proposal is shown in Figure 6 (Drawing DB1918).

All of the original schemes examined included hydraulic jump dissipators. Subsequent studies, following advice from Consultant J Barry Cooke, proved a flip bucket spillway with jet landing in an unlined pre-excavated downstream plunge-pool to be more economical.

The probable maximum flood used in developing the preliminary designs was that from the original report which indicated that 6 gates would be an economical number of gates (Reference 1). Accordingly, six gates were adopted for all preliminary designs. When the flood study (Reference 6) became available, further studies indicated that a $1.5 million saving could be achieved by adopting five gates and a higher embankment.

5.0 HYDROLOGY

5.1 Hydrology - General

Design floods for Wivenhoe Dam were derived from a study in September 1977, when the dam was initially approved, and the dam was designed on information from this study (Reference 6). Data from this 1977 study are presented in Section 5.2 of this report.

In May 1983, the 'Wivenhoe Dam Design Flood Study' was completed (Reference 7). Data from the 1983 report were not used in the design of Wivenhoe Dam as the report was published after the design of the dam had been completed. Results of the 1983 flood study are presented in Section 5.3 of this report.

A comparison of the results of the 1977 and 1983 flood studies is presented in Section 5.4 of this report.

In 1993, the floods for Wivenhoe Dam were again reassessed under the Brisbane River and Pine River Flood Study (Reference 2). A summary of the results and conclusions of this study is included in Section 5.5 of this report.

5.2 September 1977 Hydrology Study

5.2.1 September 1977 Hydrology Study - General

A reassessment of the hydrology for the design of Wivehoe Dam was carried out in September, 1977 (Reference 6), and supplemented preliminary work carried out by the Co-ordinator General's Department in June 1971 (Reference 1).

The 1977 report contains the results of flood studies, historical yield analysis, statistical yield analysis, storage behaviour graphs and overflow releases.

5.2.2 Catchment

The catchment area for Wivenhoe Dam, and the location of rainfall, pluviograph and stream gauging stations and the Kirkleagh evaporation station used in the 1977 hydrology study for the dam are shown in Figure 7. Above Mt Crosby Weir, rainfall and runoff records were collected from five subcatchment areas. The subcatchment areas, the mean catchment rainfalls for the period 1920-1969, and the average runoffs are shown in Table I.

TABLE 1 - MT CROSBY CATCHMENT - RAINFALL AND RUNOFF.

Subcatchment	Catchment Area (sq. km.)	Rainfall (mm)	Average Runoff (mm)
Upper Brisbane River	4 714	880	66
Stanley River	1 334	1 228	296
Residual Area to Wivenhoe Dam	971	930	-
Lockyer Creek	2 980	900	33
Residual Area to Mt Crosby	1 209	936	97

5.2.3 Rainfall Analysis

For the analysis,the catchment above Wivenhoe Dam was divided into three sub-areas for unitgraph derivation. Synthetic unitgraphs were derived using the Clarke-Johnson method. The sub-areas were: Upper Brisbane (subcatchment 1); Stanley River (subcatchment 2); and Residual Area to Wivenhoe Dam (subcatchment 3).

Unitgraphs for the Upper Brisbane and Stanley River subcatchments were verified using actual runoff hydrographs together with the rainfall patterns causing the runoffs. No verification could be carried out for the Residual Area to Wivenhoe Dam subcatchment as no hydrographs of actual runoff existed.

The unitgraphs were used with rainfall of chosen frequencies to estimate flood hydrographs of the same frequencies.

Rainfall frequency duration relationships were obtained using data provided by the Bureau of Meteorology (1975). Regression weighting methods were chosen for the analysis of the raw rainfall data. Various combinations of rainfall stations were used in the analysis, and the combination giving the best representation of catchment storm rainfall was chosen for each subcatchment. Annual maximum rainfalls were calculated for each subcatchment and a frequency analysis of the data was carried out. A Log - Normal Distribution was fitted to the data.

From the analysis, the rainfall values with annual exceedence probabilities of 0.1%, 0.2%, 1.0% and 2.0% were extracted for each subcatchment.

The Brisbane Bureau of Meteorology was responsible for estimating a probable maximum storm. The analysis was carried out on the basis of maximizing the moisture content of major in-situ storms. After careful consideration of the data, the maximized 1893 storm was selected for the analysis.

5.2.4 Flood Frequency Analysis

The probable maximum rainfall for each subcatchment was used to provide scaling factors for the analysis. Depth duration curves were drawn from the 24, 48 and 72 hour values with 0.1%, 0.2%, 1.0% and 2% exceedence probabilities.

Rainfall patterns used were (a) Brisbane Early, (b) Brisbane Late, (c) Average Brisbane, (d) Modified Sydney, (e) Clermont 1916, and (f) Constant, and the analysis was carried out using Water Resources program WT19 (Reference 8). The greatest peak for all subcatchments occurred using the Brisbane Late pattern, and the peak occurred at 33 hours.

The analysis took into account the operation of Somerset Dam with its flood compartment of 493 000 ML. Under normal methods of operation at that time, the Somerset Dam discharge was limited to a maximum of 1 133 cumecs until the flood peak from the Upper Brisbane passed the junction with the Stanley.

The final design hydrographs for the three subcatchments were based on a storm duration of 33 hours, the Brisbane Late Pattern, the estimated unitgraphs, and storm rainfall depths from the depth duration curves. The analyses were carried out for 0.1%, 0.2%, 1.0% and 2.0% exceedence probabilities. A typical example for one of the subcatchments is shown in Figure 8. The design floods for the subcatchments were combined to give design floods at the dam site. These are shown in Table 2.

TABLE 2 - SOMERSET DAM AND WIVENHOE DAM DESIGN FLOODS (m^3/s)
1977 STUDY

Annual Exceedence Probability	Somerset Dam Inflow	Wivenhoe Dam Natural Catchment (No dam)	Wivenhoe Dam Inflow
P.M.F.	9000	13600	15000
0.1%	8100	10500	12400
0.2%	8000	8300	10500
1%	5600	6200	8400
2%	4800	5400	7200

Design floods of 5% and 10% probabilities were obtained from stream flow records near Wivenhoe Dam. From the analysis, the peak discharge for the floods was extracted. Using the shape of the 2% flood hydrograph at Wivenhoe, the 5% and 10% flood hydrographs were deduced both with and without Wivenhoe Dam in existence. These hydrographs are shown in Figure 9.

The pattern and depths of rainfall over the catchment were required to determine the Probable Maximum Flood (PMF). The largest historical storm occurred in 1893 at a time when pluviographs were unknown and the only periodic rainfall record available was from Yandina. For the first analysis, the Yandina pattern was applied to the three subcatchments and the resulting hyetographs and the unitgraphs gave the first estimates of the PMF.

A final report from the Bureau of Meteorology indicated that the Yandina pattern could only be applied to subcatchment 2, and that the "smooth" pattern, obtained from plotting 24 hour totals only, should be applied to the other two subcatchments. The hyetographs were recalculated, and using the unit hydrographs, probable maximum floods for each of the three subcatchments were obtained. The final hydrographs for the probable maximum flood for the subcatchments are shown in Figure 10, 11 and 12.

The catchments were assumed to be in a saturated condition at the start of the storm, thus no initial loss was used. After consultation with the Bureau of Meteorology and the Brisbane City Council, a continuing loss of 1 mm/hour was adopted. This loss rate was also obtained on an analysis of the 1893 flood, used previously to verify the unitgraphs.

The hydrographs from each subcatchment were routed to Wivenhoe Dam. The Stanley River hydrograph was routed through Somerset Dam according to Brisbane City Council flood operation rules applicable at that time. This outflow hydrograph was combined with the hydrograph from the Upper Brisbane and the resultant hydrograph was combined with the inflow hydrograph from the residual area, with dam, and this was routed through Wivenhoe Dam.

The adopted PMF used in the design of Wivenhoe Dam is that shown in Figure 13. It had a peak of 15 000 m^3/s and a volume of 4 210 000 ML, equivalent to 600 mm of runoff over the catchment area.

Similarly, the Stanley River hydrograph was routed through Somerset Dam, and the outflow hydrograph was combined with the hydrograph from the Upper Brisbane. The resultant hydrograph was combined with the inflow hydrograph from the residual area, without dam, to give the natural flow PMF at Wivenhoe Dam site. This hydrograph was used for flood mitigation studies and is also shown in Figure 13.

5.2.5 Yield Studies - Historical Analysis

In the historical yield analysis five storages were considered, viz:-
Perseverence Dam, Cressbrook Creek Dam, Somerset Dam, Wivenhoe Dam and Mt Crosby Weir. Three storages existed in the Wivenhoe Dam catchment area, viz: Perseverance Dam, Cressbrook Creek Dam and Somerset Dam.

Records of inflows to each of the storages considered in the combined storage analysis were required for as long a period as possible.

The earliest records of daily river levels were taken at Mt Crosby extending back to 1894, and several rainfall stations in the Brisbane River catchment had records back to the 1880's.

Analysis of rainfall records showed that 1893/94, 1895/96 and 1897/98 were years of high runoff, so the year of 1897/98 was chosen as the first year of the yield analysis, with all storages in the catchment assumed full in September, 1897.

In order to check the consistency of the flow records, a water balance check was carried out for the following stations:- Watt's Bridge, Silverton, Lyons Bridge, Savages Crossing and Middle Creek.

Monthly residual runoff was calculated for the middle reaches. A monthly listing of the runoff depths for the lower catchment, the upper catchment, and the residual area were produced. Inconsistencies such as negative residual runoff occurred. However, in most cases, the nett negative residual flow was small enough to be accounted for by stream bed losses, or irrigation requirements.

For the water balance check, no serious discrepancies were encountered, except in a few isolated months. No changes were made to the discharge records, in order to reduce subjectivity. The average annual runoff depths obtained from these comparisons are listed in Table 3.

TABLE 3 - AVERAGE ANNUAL RUNOFFS (1977).

Catchment Outlet	RUNOFF (mm)	
	1919 - 1972	1962 - 1972
Watt's Bridge	69	80
Silverton	309	358
Lyons Bridge	52	58
Savages Crossing	97	120
(a) Savages Crossing - Lyons Bridge	114	142
(b) Silverton + Watt's Bridge	123	143
Residual (a) - (b)	97*	153*
Middle Creek	-	137*
Middle Creek - (b)	-	142*
(a) - Middle Creek	-	206*

*Excludes "Negative" Monthly Runoff

Monthly evaporation units were calculated for each of the storages. The average annual EU values are shown in Table 4. An evaporation unit (EU) is a data value used in reservoir storage behaviour analyses. It is usually calculated on a monthly basis and is the loss or gain expressed as a depth over the surface area of a reservoir caused by evaporation, seepage, rainfall and runoff. It is calculated from the following formula:-

$$EU = K \times E - P + R + S$$

where E = Monthly pan or tank evaporation (mm)
K = Factor for converting pan to lake evaporation
P = Monthly precipitation at the reservoir (mm)
R = Monthly runoff depth at the reservoir site (mm)
S = Monthly seepage depth from the reservoir (mm).)

TABLE 4 - ANNUAL EVAPORATION (1977).

Dam	EU (mm)
Wivenhoe	829
Somerset	829
Cressbrook	893
Perseverance	893

The safe yields available from the five storages including Mt Crosby weir were calculated using Water Resources multiple - storage analysis program WT29 (Reference 9). The drafts from the system were the water supply requirements for Brisbane and Toowoomba.

The capacities and dead storages for the five storages are shown in Table 5.

TABLE 5 - DAM CAPACITIES AND DEAD STORAGES.

No.	Storage Name	Full Capacity ML	Dead Storage ML
1	Mt Crosby	2 590	200
2	Wivenhoe	1 150 000*	12 300
3	Somerset	369 000	2 500
4	Cressbrook	78 300	7 830
5	Perseverance	30 300	3 030

* 3 other capacities were tried.

The results of the yield analyses for different storage capacities and methods of operation of Wivenhoe Dam and Somerset Dam are shown in Table 6. For all cases the critical period is the same, and the month of minimum storage for each is April 1903. The safe yield is the demand that could be supplied on an annual basis without failure over the period of analysis. In this case the period analysed was 76 years from 1896 to 1972.

TABLE 6 - SAFE YIELDS AT MT CROSBY (1977).

No.	Wivenhoe Capacity ML	Safe Yield at Mt Crosby ML/Year	Critical Period	Method of Operation
1	0	174 000	6/1898 - 4/1903	Wivenhoe Dam Drawn Down First
2	863 000	339 000	5/1898 - 4/1903	
3	1 150 000	388 000	5/1898 - 4/1903	
4	1 480 000	446 000	5/1898 - 4/1903	
5	1 150 000	396 000	4/1899 - 4/1903	Somerset Dam Drawn Down First

A comparison of historical yields estimated by the Water Resources Commission and the Co-ordinator General's Department is shown in Table 7.

TABLE 7 - COMPARISON OF HISTORICAL SAFE YIELDS.

Wivenhoe Storage Capacity ML	I.W.S.C. Historical Safe Yield ML/Year	C.O.G. Historical Safe Yield ML/Year
1 150 000 (EL 67)	388 000	391 400 Brisbane W/S
1 480 000 (EL 70)	446 000	412 200 Brisbane W/S

The lower yield calculated by the C.O.G. for the 1 480 000 ML storage is probably due to the higher evaporation loss as the C.O.G. made no allowance for reduction of the evaporation loss by precipitation.

5.2.6 Yield Studies - Statistical Analysis

The statistical analysis was carried out using a three storage system, involving less computation than the five storage system used for the historical yield analysis. The storages were Wivenhoe, Somerset and Mt Crosby. Perseverence and Cressbrook Creek dams were fully committed to Toowoomba's water supply, so streamflows from these catchment areas were not included. Simulation analyses using historical data showed that there was very little difference in yields using either the three or five dams systems.

Safe yields estimated by analysing critical historical records are of limited reliability since they depend unduly on the period of record. Therefore, in order to obtain the likelihood of achieving various yields from the storage over its design life, a comprehensive range of flows, derived from the statistical properties of the historical sample, was needed.

The procedure for assessing yields by statistical methods is to consider the historical flow data to be a sample of an infinite population of flows. It is thus possible to deduce the statistical properties of the parent population, and then to inductively generate additional samples. The Water Resources streamflow generation program WT89 was used in the analysis (Reference 10).

The results of the statistical analysis and a comparison with the historical yields for Wivenhoe Dam, together with degrees of confidence and with permissible failure percentages are shown in Table 8. Permissible failure is a percentage figure representing the proportion of time that the system would not be able to deliver its stated yield. Failure = 100% minus reliability.

TABLE 8 - YIELDS FOR WIVENHOE DAM (ML/YR) (1977)

CONFIDENCE	PERMISSIBLE FAILURE			
	0%	1%	3%	5%
Capacity of Wivenhoe - 1 150 000 ML (FSL 67.0)				
90%	432 600	461 200	501 200	551 600
95%	385 300	402 200	444 600	506 800
99%	328 400	344 400	387 400	436 700
Historical	386 200	455 100	481 900	505 500
Capacity of Wivenhoe - 1 320 000 ML (FSL 68.5)				
90%	446 200	479 700	514 100	570 700
95%	401 600	418 800	461 300	516 600
99%	341 300	359 800	402 200	452 000
Historical	417 000	465 700	493 200	516 600

5.3 May 1983 Hydrology Study

5.3.1 May 1983 Hydrology Study - General

After the adoption of new methods of determining Probable Maximum Precipitation (PMP) by the Bureau of Meteorology it was decided to recalculate the floods for Somerset Dam and Wivenhoe Dam. Different methods and a more complete data set were used in the reassessment. The method used involved calibrating the rainfall runoff model, calculating rainfall depths of various probabilities and calculating floods from the rainfalls. Full details are given in Reference 7.

5.3.2 Calibration Data

A runoff routing model, implemented as computer programs WT87 and WT87c, was used to relate rainfall to runoff (Reference 11). This model is a conceptual representation of the catchment storage effects and provides for the routing of rainfall excess to produce a hydrograph of surface runoff. The model is distributed, therefore allowing rainfall and loss input to vary over the catchment.

Two model parameters must be estimated by calibration using recorded data or by the use of regional formulae before the model can be used to calculate design floods.

Data were available for seven major floods in the catchment since 1965 for one or more of the gauging stations in the catchment. Corresponding daily rainfall and pluviograph data were also available.

Rainfall data were available for the seven storms from almost all the nineteen rainfall stations used in the analysis.

Even though there were eight pluviographs in the catchment, data were only available from a few of them for any particular storm.

For the distribution of rainfall and pluviograph stations the catchment was divided into 95 subareas. The large number of subareas was because there was considerably more data available for the Wivenhoe Dam catchment, than for almost any other catchment in Queensland.

For the purposes of model calibration the catchment was considered as two subcatchments, the catchment above Somerset Dam, and the remainder. Calibration of the model for the catchment above Somerset Dam was done by using the inflow records from the Brisbane City Council. For the remaining catchment, the model was calibrated on all of the available gauging station flood records.

5.3.3 Rainfall Analysis

The primary approach adopted in this flood study was the use of a rainfall - runoff model and calculated design rainfall depths. The first rainfall analysis was a frequency analysis on annual rainfall series to calculate rainfalls of different frequencies. The second rainfall analysis was the estimation of the Probable Maximum Precipitation which was carried out by the Bureau of Meteorology in 1982.

As a secondary approach, a flood frequency analysis was carried out and is described in Section 5.3.8.

There were sixteen rainfall stations in the catchment that had sufficient data for a frequency analysis. The analysis was carried out using Water Resources computer program WS06 (Reference 12). The annual series of one-day, two-day and three-day maxima, and the 1%, 0.1% and 0.01% annual exceedence probability rainfall depths were extracted. The results are listed in Table 9.

The rainfall values varied significantly over the catchment from the low rainfall area in the west to the high rainfalls in the north-east corner. The spatial distribution was used in the flood study, as it is reflected in recorded rainfall data.

TABLE 9 - RAINFALL FREQUENCY ANALYSIS RAINFALL DEPTH (mm) (1983).

Station	1 Day			2 Day			3 Day		
	1%	0.1%	0.01%	1%	0.1%	0.01%	1%	0.1%	0.01%
Peachester	384	550	738	570	824	1 115	694	1 003	1 360
Mt Mee	428	643	899	601	901	1 256	727	1 099	1 545
Esk	221	303	393	301	419	550	348	487	642
Kilcoy	247	355	479	359	528	726	402	593	816
Mt Kilcoy	288	422	579	403	595	820	483	719	997
Cooyar	158	210	265	186	245	308	209	276	346
Moore	196	264	338	276	379	492	328	459	605
Toogoolawah	201	278	364	266	367	479	301	414	537
Jimna	223	315	418	300	419	552	363	513	680
Coominya	177	235	297	246	335	433	285	396	520
Blackbutt	172	227	285	226	302	384	256	344	438
Haden	159	212	269	190	249	310	219	290	366
Crows Nest	162	215	272	229	311	400	262	359	464
Woodford	349	508	690	531	795	1 108	639	967	1 360
Yarraman	165	229	294	222	302	389	231	307	387
Nanango	142	187	234	186	246	310	203	266	332

5.3.4 Probable Maximum Precipitation (PMP)

The Bureau of Meteorology calculated Probable Maximum Precipitation estimates for the catchment. They considered three different subcatchments:-

1. Upper Brisbane River (4714 km^2), ie the area upstream of the junction of the Stanley and Brisbane rivers;
2. Stanley River (1334 km^2), ie the area upstream of Somerset Dam;
3. Residual area to Wivenhoe Dam (971 km^2), ie the area downstream of Somerset Dam and downstream of the junction of the Stanley and Brisbane rivers.

The method of adjusted United States data was used for the six hour duration storm and the generalised method of areas subject to tropical cyclones for one day rainfall. Twelve hour, two day and three day rainfalls were derived from the one day estimate. General meteorological considerations were used to derive the design rainfalls for four, five, six and seven days.

The Probable Maximum Precipitation depths are listed in Table 10.

TABLE 10-PROBABLE MAXIMUM PRECIPITATION DEPTH (mm) (1983).

Duration	Subcatchment			Total Wivenhoe Dam Catchment
	1	2	3	
6 h	300	400	420	260
12 h	420	560	560	380
1 day	660	840	820	600
2 days	1 080	1 380	1 360	1 000
3 days	1 380	1 760	1 720	1 260
4 days	1 600	2 040	2 000	1 460
5 days	1 660	2 120	2 080	1 520
6 days	1 700	2 160	2 120	1 560
7 days	1 840	2 340	2 320	1 700

5.3.5 Somerset Dam Flood Operation

Somerset Dam is operated as a flood control dam, and floods were routed through the dam using a flood operation policy developed with officers of the Brisbane City Council. The dam operation was different from that used in the September 1977 analysis.

Somerset Dam has a normal full supply level of EL 99 which corresponds to a storage volume of 373 600 ML. An additional flood storage volume of 582 400 ML is provided for flood mitigation purposes (crest level is EL 107.46). The dam is equipped with four regulators, eight sluice gates and eight spillway gates. The discharge from each of these depends on the storage level. A cross section of Somerset Dam is shown in Figure 14.

The maximum discharge possible from normal full supply level to the top of the flood storage is listed in Table 11.

TABLE 11
MAXIMUM POSSIBLE DISCHARGE FROM SOMERSET DAM .

Lake Level (EL)	Total available discharge (m^3/s)
99	1 884
100	1 919
101	2 002
102	2 218
103	2 511
104	2 864
105	3 313
106	3 824
107	4 385
108	5 085

The operating rule for Somerset Dam in 1983 was as follows (assuming 2 hour time steps):

1. No release for 8 time steps (i.e., 16 hours)
2. Open regulators, one for each 2 hours until 4 are open.
3. Open spillway gates, two each 2 hours until 8 are open.
4. Open sluice gates, two each 2 hours until 8 are open.
5. All gates and regulators kept open until the flood storage is emptied.

5.3.6 Somerset Dam Floods

The floods for Somerset Dam were calculated using the rainfall frequency analysis and the PMP supplied by the Bureau of Meteorology in 1982.

The runoff routing model (Reference 11) was used to calculate the floods using the PMP depths for subcatchment 2 from Table 10 and the temporal patterns supplied by the Bureau of Meteorology. Different patterns were used for the five and six day storms to determine the most critical. For the seven day storm, the temporal pattern with the two peaks was used, the seven day storm being defined as 2 two day storms separated by three days of little or no rain. The inflow hydrographs were calculated for the dam at its normal full supply level (EL 99.0), and then routed through the dam using the operating policy described in Section 5.3.5 above. The results are shown in Table 12.

TABLE 12 - SOMERSET DAM FLOODS (FROM RAINFALL FREQUENCY ANALYSIS - 1983).

Annual Exceedence Probability (%)	Duration (day)	Pattern	Flood Volume (ML)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)
1	1		342 000	3 250	2 574
	2		448 000	3 214	2 362
	3		483 000	2 600	2 076
0.1	1		536 000	5 282	3 437
	2		738 000	5 305	3 444
	3		836 000	4 435	3 044
0.01	1		761 000	7 671	4 569
	2		1 077 000	7 779	4 904
	3		1 250 000	6 612	4 417
PMP	1		1 037 000	10 638	6 236*
	2		1 670 000	12 161	8 442*
	3		2 098 000	11 055	7 850*
	4		2 400 000	11 886	8 376*
	5	early peak	2 450 000	12 380	8 571*
	5	late peak	2 450 000	12 457	8 195*
	6	early peak	2 410 000	12 057	8 238*
	6	mid peak	2 410 000	12 565	8 546*
	6	late peak	2 410 000	12 862	7 807*
	7		2 800 000	14 167	8 685*

***Note that floods caused by the PMP overtop the crest of Somerset Dam for all durations.**

In calculating the Wivenhoe floods, the 1%, 0.1% and 0.01 floods for Somerset Dam remained the same but the floods calculated from the PMP were recalculated. In this case the PMP storm was assumed to cover the total Wivenhoe Dam catchment (including the Somerset Dam catchment) with a reduced rainfall depth over the larger area. The total Wivenhoe catchment PMP depths from Table 10 were applied over the Somerset Dam catchment. The PMP floods were recalculated and routed through the Somerset Dam storage as above. The resulting Somerset Dam inflows and outflows for the Wivenhoe Dam PMP over the Somerset Dam catchment are shown in Table 13.

TABLE 13
SOMERSET DAM FLOODS.
(WIVENHOE DAM PMP OVER CATCHMENT - 1983)

Duration (day)	Pattern	Flood Volume (ML)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)
1		717 000	7 202	4 340
2		1 169 000	8 442	5 351*
3		1 432 000	7 548	5 044
4		1 627 000	8 110	5 117*
5	early peak	1 660 000	8 372	5 271*
5	late peak	1 660 000	8 546	4 919
6	early peak	1 630 000	8 340	5 189*
6	mid peak	1 630 000	8 706	5 394*
6	late peak	1 630 000	8 891	5 321*
7		1 950 000	9 881	5 477*

***Flow over crest of Somerset Dam**

5.3.7 Wivenhoe Dam Floods

Floods for Wivenhoe Dam were calculated in a similar manner to the Somerset Dam floods. Two cases were considered:-

(i) assuming Wivenhoe Dam did not exist; and

(ii) assuming Wivenhoe Dam was at Full Supply Level (EL 67) at the start of the flood, and Somerset Dam was operated according to the flood operating rule described in Section 5.3.5 above.

Detailed operating rules for Wivenhoe Dam were not available in May 1983 when the report was prepared, therefore the floods were not routed through the storage. The floods for various durations are listed in Table 14.

TABLE 14 -WIVENHOE DAM FLOODS (1983).

Annual Exceedence Probability (%)	Duration (day)	Pattern	Flood Volume (ML)	Peak Inflow no dam (m^3/s)	Peak Inflow with dam (m^3/s)
1	1		1 095 000	7 569	8 299
	2		1 234 000	8 303	8 723
	3		1 160 000	6 284	6 588
0.1	1		1 700 000	12 113	12 981
	2		2 044 000	12 914	13 406
	3		2 177 000	10 542	10 898
0.01	1		2 384 000	17 202	18 201
	2		3 045 000	19 065	19 611
	3		3 263 000	15 490	15 846
PMP	1		3 790 000	33 565	34 161
	2		6 170 000	42 174	42 973
	3		7 470 000	38 067	38 503
	4		8 560 000	41 433	41 770
	5	early peak	8 720 000	42 988	43 545
	5	late peak	8 720 000	41 905	42 890
	6	early peak	8 560 000	41 228	41 359
	6	mid peak	8 560 000	42 583	43 106
	6	late peak	8 560 000	41 691	43 576
	7		10 260 000	44 471	47 836

5.3.8 Flood Frequency Analysis

It was not possible to do a direct flood frequency analysis on the streamflow data at Wivenhoe Damsite for several reasons. The gauging station at Middle Creek had only a short record of nineteen years. The gauging stations further downstream at Lowood/Vernor/Savages Crossing had a much longer period of data of 72 years from 1909, but there were two problems with the lengthy data set. Firstly it was downstream of the junction with Lockyer Creek, which contributed considerably to the flood flow, and secondly the series contained a significant discontinuity due to the construction of Somerset Dam .

Another lengthy annual series of floods was available from the Brisbane City Council. This series was derived for Wivenhoe Damsite for the period 1887 to 1975, and was extended to 1983 using records from Middle Creek. It assumed that Somerset Dam operated as a flood mitigation storage for the period 1887 to 1982. The flood series was derived from recorded flood levels at various downstream sites for the early periods and was adjusted to transfer floods to Wivenhoe Dam. The series was therefore derived rather than recorded but was considered to be adequate for the analysis. Water Resources computer programs WS06 and WS06C (Reference 12) and WAKEBY (No manual available) were run to fit a range of statistical distributions to the sequence of 95 flood values. The Log-Pearson Type III distribution was chosen as it most closely agreed with the results from the rainfall frequency-model analysis, and the results are plotted in Figure 15.

5.4 Comparison of 1977 and 1983 Studies

The method of analysis used in the May 1983 report, using rainfall frequency, Probable Maximum Precipitation and rainfall runoff modelling is different from that used in the September 1977 report.

A comparison of the 1977 and 1983 results is shown in Table 15.

TABLE 15
WIVENHOE DAM AND SOMERSET DAM FLOODS
COMPARISON OF 1977 AND 1983 RESULTS.

Annual Exceedence Probability %	Somerset Dam Inflow (m^3/s)		Wivenhoe Dam Inflow (m^3/s)			
			NO DAM		WITH DAM	
	1977	1983	1977	1983	1977	1983
1	5 600	3 250	6 200	8 300	8 400	8 700
0.1	8 100	5 300	10 500	12 900	12 400	13 400
0.01	-	7 780	-	19 100	-	19 600
PMF	9 000	14 200	12 500	44 500	15 000	47 800

The table shows that there is little difference between the floods of 1% and 0.1% annual exceedence probability with the 1983 Somerset Dam floods being smaller and the 1983 Wivenhoe Dam floods being larger. The Probable Maximum Flood however, shows a dramatic increase, in particular for Wivenhoe Dam floods. The Probable Maximum Flood for Somerset Dam increased by a more modest amount.

The increase in floods is due to the increase in Probable Maximum Precipitation estimates obtained from the Bureau of Meteorology.

A comparison of the catchment rainfalls is shown in Table 16.

TABLE 16
DESIGN CATCHMENT RAINFALL (mm)
COMPARISON OF 1977 AND 1983 RESULTS.

Annual Exceedence Probability %	1 day		2 day		3 day	
	1977	1983	1977	1983	1977	1983
1	220	217	340	298	360	347
0.1	340	303	470	419	530	493
0.01	440	400	640	556	740	657
PMP	320	600	480	1000	600	1260

5.5 Summary of Results and Conclusions of the 1993 Flood Estimates for Wivenhoe Dam

5.5.1 Brisbane River and Pine River Flood Studies - General

In August 1990, the South East Queensland Water Board (SEQWB) commissioned the Department of Primary Industries, Water Resources Business Group (DPI, WR) to undertake the Brisbane River and Pine River Flood Study as part of an overall safety review of the Board's dams. Under this study a draft report on Design Flood Estimation for Somerset Dam and Wivenhoe Dam was prepared in March 1993 (Refer Appendix B, Volume 8). Summaries of results and conclusions have been extracted from this report and are presented in Sections 5.5.2 and 5.5.3 below.

5.5.2 Summary of Results

The following Section 2.2 extracted from the abovementioned report summarises the result of the study with respect to Wivenhoe Dam.

2.2 Wivenhoe Dam

The critical design storm scenario for Wivenhoe Dam is the design storm centred over the whole of the catchment of Wivenhoe Dam. The 48 hour duration PMF produces the largest outflow from Wivenhoe Dam under existing normal gate operation procedures. The results of the design flood re-assessment for Wivenhoe Dam are presented in Table 2.2.

The duration of the event that produces the peak outflow from Wivenhoe Dam changes from 48 hours to 72 hours for events of more frequent occurrence, because of the use of temporal patterns from Australian Rainfall and Runoff, (1987), with these events. Again, this only affects events up to 1 in 100 years ARI.

Table 2.2
Wivenhoe Dam Design Flood Estimates
Storm Centred over Wivenhoe Dam Catchment

ARI (YEARS)	STORM DURATION (HOURS)	PEAK INFLOW (m^3/s)	PEAK OUTFLOW (m^3/s)	FLOOD VOLUME (ML)	PEAK LAKE LEVEL (m AHD)
10	72	3 630	2 900	861 570	68.18
20	72	4 980	3 330	1 128 590	70.58
50	72	7 240	3 450	1 405 480	72.83
100	72	9 080	6 810	1 860 400	74.48
200	48	11 110	7 640	1 822 840	74.84
500	48	12 580	9 130	2 104 520	75.50
1 000	48	13 820	9 970	2 336 350	75.99
10 000	48	20 770	13 490	3 593 000	78.61
100 000(PMF)	48	30 670	13 490#	5 333 920	81.28*
Note: *Indicates that the embankment crest level, (EL 79.15 m AHD) is overtopped.					

[#13490 should read 25040]

Embankment dams when subject to a continous overtopping flow will normally fail, depending upon the duration of the flow and the likely extent of scouring of the crest. The Imminent Failure Flood, (IFF), for Wivenhoe Dam has therefore been assessed as the flood event which when routed through the storage under the existing storage operating procedure just threatens to overtop the embankment. The embankment crest level, (79.15 m AHD), has been adopted as the critical level in preference to the top of the wave wall, (79.90 m AHD), because the wave wall does not extend over the whole of the embankment.

The estimated magnitude of the rainfall depth associated with the IFF for Wivenhoe Dam is 75% of the PMP. This rainfall depth has an ARI of approximately 14 300 years.

The peak inflow associated with the IFF of Wivenhoe Dam is estimated to be 21 990 m^3/s, whilst the resultant peak outflow from the dam is 14 080 m^3/s. The flood volume for the IFF is estimated to be 3 794 180 ML.

By way of comparison, if the top of the wave wall, (79.90 m AHD), is adopted as the critical level, the magnitude of rainfall depth associated with this flood is 81% of the PMP. This rainfall depth has an ARI of approximately 23 600 years.

The peak inflow associated with this flood is estimated to be 24 060 m^3/s, whilst the resultant peak outflow from the dam is 14 920 m^3/s. The flood volume for this flood is estimated to be 4 162 020 ML.

5.5.3 Conclusions

The following Section 8.3 extracted from the abovementioned report quotes the conclusions reached with respect to Wivenhoe Dam.

8.3 Wivenhoe Dam

The catchment of Wivenhoe Dam in its natural state, is estimated to have a critical duration of around 24 hours. The PMF at this site is estimated to be 39 090 m^3/s, whereas the 100 Year ARI event is estimated to have a peak discharge of over 10 670 m^3/s. Comparing estimates of peak discharge for outflows from Wivenhoe Dam under existing normal gate operation procedures for corresponding events reveals that the critical duration for the PMF event increases to 48 hours and produces a peak outflow from the dam of 25 040 m^3/s, whilst the peak outflow for the 100 Year ARI event is 6 810 m^3/s for a 72 hour duration storm.

The design floods estimated for Wivenhoe Dam are different from the those obtained by Weeks. A comparison between the corresponding 48 hour duration design floods for various ARIs between 100 years and the PMF shows that the flood volumes are all less as is expected because of the decreased rainfalls. The peak inflow has also been reduced substantially for the PMF event, but for higher probability of exceedance events, the peak inflows have increased. The differences in estimates between the two studies are due largely to the differences in design rainfall estimates and temporal patterns and also the differing runoff-routing model parameters.

Based upon the design flood reassessment, the IFF of Wivenhoe Dam, under current normal storage operation procedures, has an ARI of 14 300 years. The rainfall depth associated with the IFF is equivalent to 75% of the PMP. The PMF will overtop the main embankment crest level by 2.13 metres. Embankment dams, like Wivenhoe Dam, are likely to fail under these circumstances.

ANCOLD guidelines recommend that for dams classified in the high incremental flood hazard category, the annual exceedance probability of the Recommended Design Flood, (RDF), should be between the PMF and 1 in 10 000.

The design flood modelling with the temporal patterns provided in the 1987 edition of Australian Rainfall and Runoff illustrated some shortfalls in the storage routing model and the existing storage operation procedures. The multi-peaked hydrographs derived for some of the higher probability of exceedance events caused the storage routing model to make releases from the storages which were not consistent with the existing operating policies. The model was modified so as to overcome these problems, but it became evident during the modelling that there is scope for further investigation into the storage operating procedures and that the storage operation model needs to be refined.

6.0 CONSTRUCTION SEQUENCE

6.1 Construction Activities Governed by Seasonal Conditions

Runoff at Wivenhoe Dam can be in excess of 10,000 m^3/s, with most of the high flows occuring in the summer months, and flows of small magnitude occuring only in late winter and early spring. The riverbed activities could therefore be safely carried out in the drier season, with such works ceasing and being adequately protected against flooding before the start of the next wet season. The longest continuously recorded period of flow in the Brisbane River is for Savages Crossing approximately 20 km downstream of the damsite. The distribution of flood peaks for Savage's Crossing is shown in Figure 16 (Drawing A3-45418).

6.2 Diversion Considerations Influence Construction Sequence

The water supply for the City of Brisbane passing through the damsite from Somerset Dam to Mt Crosby Weir could not be interrupted, and the flow had to be maintained at all times.

Various methods of diversion were examined in conjunction with the proposals outlined in Section 4.3 above. Methods of diversion considered included separate tunnels (for water supply releases and for the proposed Hydro-electric plant); separate conduits in one tunnel, and a cut and cover concrete culvert incorporating two conduits.

These were rejected for economic reasons and because of the frequency and magnitudes of floods in the Brisbane River. The most economical diversion alternative commensurate with adequate diversion capacity was a primary diversion channel on the right bank of the river. With the main cofferdams at EL 43.0 the diversion channel had a capacity of 1850 m^3/s before overtopping occured. This flow had a frequency of 1 in 5 on an annual basis so that it was still necessary to protect the downstream face of the downstream cofferdam with steel mesh to prevent failure on overtopping. Discharge capacity curves for the diversion channel and spillway diversion channel are shown in Figures 17 and 18 (Drawings A3-49893A and A3-53325). Further details of the diversion works are included in Section 11.4. Availability of the diversion channel at the start of the construction period allowed construction to proceed throughout the year, with riverbed activities planned for the seasons of low flow.

6.3 Planned Construction Sequence

A construction program based on the adopted layout, Proposal 5, was prepared.

The program required construction in the following stages:-

(i) Excavate the primary diversion channel on the right bank and advance the cofferdams from the left bank of the river, not closing off until the floor of the diversion channel had been grouted. The cofferdams were integrated with the main embankment as far as possible. Materials for the cofferdams were to be obtained from the spillway area.

(ii) Divert the river through the diversion channel and build up the cofferdams to give a good immunity against overtopping. Excavate the river bed and the left bank alluvial area to rock in the core contact area. After grouting these areas place the core material.

(iii) Excavate the spillway using excavated material for placement in the embankment shells. Excavation of the spillway to proceed in the following order:-
left wall area; areas of spillway structures; outlet channel both upstream and downstream and lastly the general approach and discharge channels.

(iv) Concrete the spillway, leaving monoliths on the left hand side low. This sequence would allow concreting to start early and supplementary diversion capacity through the spillway to be available early.

(v) Close off the primary diversion channel and complete the embankment.

(vi) Raise the left hand spillway monolith; and

(vii) Manufacture and install the radial gates.

6.4 Modified Construction Sequence to Suit Separate Contracts

The order of construction for Proposal 5 above was modified to suit the calling of contracts in the following stages:

(i) Contract No. 2103. Excavation for Diversion Channel and Dam Embankment.

Excavate the left and right abutments of the dam to foundation level; excavate the left bank of the Brisbane River to core foundation level; excavate the diversion channel, and place suitable excavated material in the embankment on the outer left abutment. Sections of the completed excavations to be handed over to the grouting contractor in stages.

(ii) Contract No. 2108 Foundation Grouting.

Foundation grouting was done as sections of the foundation cleanup were completed under Contract No. 2103 and Contract No. 2112. Grouting in the spillway area was done by day labour.

Further aspects on grouting under the embankment and the spillway are described in Sections 11.17 and 17.7 respectively.

(iii) Contract No. 2112 Second Stage Construction.

Upon completion of grouting in the diversion channel, divert the river through the diversion channel; dewater the river section of the dam embankment and construct the main cofferdams; excavate the first stage of the spillway excavation and construct the embankment between the cofferdams; raise the shell section of the embankment on the outer left abutment.

(iv) Contract No. 2120 Third Stage Construction.

Complete excavation of the spillway and construct the concrete spillway leaving a section low for diversion after diversion channel closure; close the diversion channel after the remainder of the embankment has reached EL 74.0 and all concrete in the spillway flip and monoliths has reached EL 48.0 (except for the low diversion section); close off the low section of the concrete spillway after the embankment in the diversion channel section has reached EL 62.0; complete the main embankment including saddle dams, roads, bridges etc; and install mechanical items.

(v) Contract No. 2123. Fabrication, Delivery and Erection of five 12 m x 16.6 m high radial gates, one 12 m x 12.1 m high Bulkhead Gate, Ten Hydraulic winches and associated Control Equipment.

Fabricate and install the equipment listed above. Design aspects of the radial gates are covered in Section 23 of this report.

(vi) Contract No. 2111. Excavation, Processing and Stockpiling of Gravels.

Large quantities of sand filters and concrete aggregates were required for construction of the works, and were required by separate contractors constructing sections (iii) and (iv) above.

To avoid difficulty of access to common borrow areas affected by construction and floods, it was decided to let a separate contract early in the construction period (and prior to contracts being let for sections (iii) and (iv) above) for the excavation, processing and stockpiling of filter materials and aggregates.

The construction sequence is shown diagramatically in Figure 19.

Targets and time constraints were specified in the various contracts to ensure continuity of construction. These targets and constraints and their effects are listed in Appendix D.

7.0 FOUNDATION GEOLOGY

7.1 Foundation Geology - General

The main dam is located wholly on the Helidon Sandstone (also called the Wivenhoe Sandstone - References 13 and 14). The sandstone consists of quartz grains with minor dark chert fragments in a whitish kaolinitic matrix. Structurally, most of the rock foundation consisted of massive undulating layers of sandstone, sometimes crossbedded, which had dips between 2 and 10 degrees and strikes in the general ENE direction. Most of these units were separated by thin layers of shale, shale conglomerate or fine pebble conglomerates containing minor amounts of fossilized plant material (coal).

An exception occured on the right bank where up to 9 m of interbedded shales and fine sandstone were found. The sandstone unit above was fairly weathered and contained many thin layers of clay. A continuation of the shale/fine sandstone unit is thought to have been intersected on the left bank (Reference 14, Clause 6.01, hole WB77). This suggested that the unit was responsible for the incision of the river into the valley floor at the dam site and subsequent control of the alluvial deposition sequences upstream of the dam site. Such a scenario would agree with Warner's hypothesis on the formation of the Brisbane River Valley (Reference 13).

The influence of the local geological features on design considerations is discussed in the appropriate sections. Up to 20m of alluvium/colluvium overburden was found to exist above the foundation rock. These soils were evaluated as part of the foundation and materials investigation.

8.0 PRELIMINARY MATERIALS INVESTIGATIONS

8.1 Preliminary Materials Reconnaissance Prior to 1971

Prior to June 1971 the only sources of material identified for use in the dam embankment were sandstone from the proposed spillway cut and alluvium from an area upstream of the dam axis, subsequently designated Borrow Area WX 11.

Samples of highly weathered sandstone from the site were subjected to classification tests, compaction tests, triaxial tests, and direct shear tests. Moderately weathered sandstone was subjected to direct shear tests only. The tests confirmed that sandstone at the site would be suitable for rolled fill in the pervious portions of the dam embankment.

Tests were also carried out on samples from 6 auger holes drilled into what subsequently became known as WX 11. This material was classified as a low plasticity clay and it was concluded that the material would be suitable for the embankment core, subject to more extensive searches being conducted to locate stronger material within economic haul distances of the dam.

8.2 Further Preliminary Investigations

More extensive materials investigations began in early 1975, and revealed that the following materials were available locally:-

(a) Impermeable soils such as clays and sandy clays were available within 1 km of the dam axis in the following deposits:-

- (i) Highway Flat (subsequently designated WX 10) - a relatively flat alluvial terrace on the left of the Brisbane Valley Highway's southern approach to Wivenhoe Bridge;
- (ii) Wivenhoe Flat (subsequently designated WX 11) - the upstream extension of the alluvial deposit on the left bank of the Brisbane River immediately upstream of the dam axis; and
- (iii) Beutel's Pumpkin Patch (subsequently designated WX 21) - a relatively high sloping terrace forming the right bank of Sheep Station Creek immediately upstream of the western extremity of the dam embankment.

(b) Large deposits of granular material were identified within a distance of 1.5 km downstream of the dam axis. These deposits were:-

- (i) Atkinson's Crossing (subsequently designated WX 14) - a low level flood overflow channel and associated low level terraces on the right bank of the Brisbane River about 500 m downstream of the dam axis; and
- (ii) Roseborough Gravel/Sand deposit (subsequently designated WX 15) - a low level flood overflow channel on the right bank of the Brisbane River about 1.5 km downstream of the dam axis.

  It was recognised that washing and processing would be necessary to produce material suitable for the filters of the dam embankment and for concrete aggregate.

(c) Sandstone rock was available in large quantities in the vicinity of the damsite, and would require quarrying. For economy, sandstone excavated from the spillway would be used in the dam embankment. The preliminary designs assumed this and examined rockfill dams of the central core type when making economic comparisons.

(d) A source of hard durable rock suitable for riprap was not determined in the preliminary investigations. The sandstone at the site was not durable enough to be used for upstream wave protection or as downstream facing on the cross river embankment. It was assumed that excavation for the Wivenhoe Pump Storage Project would produce suitable material from the metamorphic rock of the Neranleigh - Fernvale block across the Great Moreton Fault.

The locations of the borrow areas are shown in Figure 20 (Drawing A3-53372C).

9.0 FOUNDATION INVESTIGATIONS

9.1 Foundation Investigations - General

This section covers investigations, on the site of the proposed dam and spillway, to determine planned foundation levels for design and construction; and also investigates the suitability of materials on the site which may be used in construction, either directly or from stockpiles, or disposed of as unsuitable materials. The investigations also assisted in determining the final locations of the dam and spillway axes and in determining the method of diversion, and location of the diversion channel.

The investigations were generally carried out from July 1975 to July 1977. The stage construction of the dam allowed progressive appraisal and re-appraisal of the foundations and borrow areas. Investigation of the borrow areas is covered in Section 10.

Exploration of the damsite was carried out by diamond, percussion and auger drilling and by trenching. In the spillway area, investigations included boring of a 1.050 m dia inspection shaft, seismic traverses and a ripping trial. Saddle Dam No. 1 was investigated by surface mapping, trenching and diamond drilling; investigation of Saddle Dam No. 2 encompassed surface mapping, diamond drilling and seismic traverses.

Field and laboratory tests were carried out in accordance with Australian Standards - A.S. 89 for earthfill materials; and A.S. 1465 for concrete aggregates. Soils were classified in accordance with the Unified Soils Classification System.

9.2 Dam Site Investigations

9.2.1 Dozer Trenching

Continuous dozer trenches from Distances 250 m to 1450 m on the left bank and from 2000 m to 2300 m on the right bank were excavated along the original C axis of the dam. Trenching was carried out by an International TD25 and a Caterpillar D7, both equipped with a hydraulic blade. The purpose of the stripping was to establish rock levels and stripping depths over the length of the axis and for geological examination of the rock and intepretation of foundation conditions. Further trenching was carried out on the right bank between Distances 1900 m and 2000 m, i.e. in the area of the diversion channel, to determine the profile of the underlying rock surface on the steeply rising right bank adjacent to the river. The locations of the dozer trenches are shown in Figure 3 (Drawing No. K1-45754D).

9.2.2 Percussion Holes

In 1976, nine percussion holes, PH1 to PH9, were drilled on the left bank of the Brisbane River between approximate Distances 1550 m and 1750 m to determine the depth of alluvium on the left bank adjoining the river. Holes varied in depth from 12.9 m to 23.9 m. In 1977 percussion holes PH10 and PH11 were drilled on the left bank of the river for pump out tests to determine the permeability of the gravels overlying the bedrock. Measured transmissivity varied from 50 m^2/day to 60 m^2/day. The location and logs of percussion holes are shown in Figures 3 and 5 of Reference 15. Results of tests on disturbed samples including grading, soil classification, Atterberg Limits, etc are given in Table 2, Sheet 4 of Reference 15.

9.2.3 Power Auger Holes

Using a Gemco drill, 98 power auger holes, GH1 to GH98, were drilled over the length of the dam embankment between distances 200 m and 2400 m, except for the spillway area, to determine the nature of the dam embankment foundations. Standard penetration tests were conducted in 2 holes to determine approximate Relative Densities for unconsolidated sands and silty sands in the deep alluvial section of the foundations adjacent to the river on the left bank. The location and logs of auger holes are shown in Figures 3 to 6 of Reference 15 and results of tests on disturbed and undisturbed samples are given in Table 2, Sheets 1, 2 and 3 of Reference 15.

Following excavation of the dam foundations under Contract No. 2103 further Gemco holes, GH99 to GH132, were drilled to test the suitability of the foundation materials.

In June 1977, four holes, GH99 to GH102, were drilled between Distances 536 m and 581 m on the left abutment. The foundation material was a SC material, derived from the weathering of sandstone, of low to medium plasticity. In October 1977, thirty Gemco holes, GH103 to GH132, were drilled in an area of deep alluvium containing lenses of silt between Distances 1400 m and 1672 m. The locations of these holes are shown in Figure 21 (Drawing A1-56182). The material was classified CL of medium plasticity. Undisturbed samples were taken and tested for liquefaction. The results proved the material suitable and the alluvium was left in situ as shown in Figure 22 (Drawing A1-49868B). Further details and test results are given in Section 11.8.1.

9.2.4 Diamond Drilling on Dam Axis and Spillway

The initial diamond drilling investigations were made at Wivenhoe Dam in 1968. A total of 42 holes were drilled. Of these, 27 holes, WB1 to WB27, were drilled on or close to the alternative axes A and B, and also on the high right bank abutment of the A axis. A total of 17 holes, WB26 - WB 42, were drilled on or close to the C axis. These initial investigations eliminated the B axis because of the great depth of old alluvium over much of the higher alluvial terrace on the left bank. Drilling on the A axis showed deep areas of silty-sandy gravels unsuitable for the foundations of a central core type embankment. In the river section sound rock was about 15 m beneath the surface on the A axis, compared to 6 m on the C axis.

Samples of cores from holes WB29-WB31 and WB34 from the left bank, and from holes WB39, WB40 and WB42 on the right bank near the proposed diversion channel, were subjected to tests to determine unconfined compressive strength, shear strength, modulus of elasticity, water absorption and swelling, so that preliminary design of concrete structures could proceed. The results showed that the intact sandstone on the left bank had adequate strength to support a concrete structure of the dimensions envisaged.

Further diamond drilling investigations were necessary on the proposed C axis and for the location of other features of the dam. Commencing in January 1975 diamond drilling was carried out on the proposed C axis; on the alternative spillway locations on the left and right banks of the river; on the centreline of a possible diversion tunnel on the right bank; and also on the sites of the proposed Saddle Dams 1 and 2.

A further 12 diamond drill holes (WB43 - WB47; WB59-WB61; WB65 and WB77) were drilled on or close to the C axis between January and October 1975 to determine foundation conditions on the left abutment of the dam.

Alternative spillway sites on the right and left banks of the river were investigated. Between July and December 1975, six diamond drill holes, (WB66-WB69 and WB79 - WB80) were drilled on the possible spillway site on the right bank of the river. Between February and August 1976, a series of 18 diamond drill holes was drilled to investigate the spillway area foundations on the left bank. Four holes WB87 - WB90 were drilled to test the foundations for the right hand side training and retaining walls of the final spillway; WB 91 was drilled at the intersection of the dam and spillway axes; a series of holes (WB92 - WB96, WB96A and WB97) was drilled across the spillway to test the foundations for the spillway monoliths; and holes WB98 - WB102 and WB101A and WB102A were drilled on the spillway centreline upstream of the dam to investigate the approach channel to the spillway.

The location of the diamond drill holes, photographs of cores, and logs of diamond drill holes WB26 - WB102A are included in Reference 16, Parts 1,2 and 3. In February 1977 four holes across the spillway (WB93A, WB94A, WB95A and WB96B) were redrilled to obtain cores from shallower depths than had been obtained in the original holes.

In February 1978, after construction had commenced at the site, a further 8 diamond drill holes were drilled, to approximately 15 metres deep, to determine foundation conditions for the right hand side training and retaining walls of the spillway.

9.2.5 Spillway Inspection Shaft

The spillway foundation on the left bank was also investigated by a 1.050 m dia bored shaft. The shaft was located about 21.5 m downstream of the dam axis and about 2.75 m to the left of the spillway axis. The shaft was bored to a depth of 45 m, the bottom of the hole being at approximate EL 21. The shaft was bored during March-April 1976 using a Calweld C250/B bucket rig. Inspections and geological logging were done from a suspended cage. Horizontal cores were taken from the side of the shaft. Details of tests and results are given in Section 14.6.1. A geological report and logs of the shaft are included in Reference 16, Part 3. The logs of the inspection shaft can be compared with those of D.D. Hole WB84 (Reference 16, Part 2) as they were drilled in the same position.

9.2.6 Ripping tests

In 1975, two ripping tests were carried out on the sandstone, one on the right bank and one on the left bank. A D9G bulldozer with a single tyne ripper was used. On the right bank an outcrop of rock upstream of the C axis was tested, and on the left bank a section near the centre of the proposed spillway where rock existed within 2 m of the surface was tested.

The rock proved to be easily rippable and broke up under the action of dozer tracks. When the excavated material was examined many intact lumps were evident in a matrix of completely pulverised sandstone. Specimens were tested in the laboratory for density, permeability and shear. Results indicated that although the rolled sandstone would have adequate shear strength, ($\varnothing = 37°$), it could not be regarded as freedraining.

9.2.7 Investigations of Saddle Dams' Foundations

Along the extension of the left abutment of Axis C, there existed two low areas designated Saddle Dam No. 1 and Saddle Dam No. 2.

The site of Saddle Dam No. 1 was investigated by surface mapping, trenching, and 5 diamond drill holes. DD1-DD3 were drilled in June 1975, and located a shale layer approximately 20 m from the surface. Diamond drill holes DD4 and DD5, drilled in June 1977, confirmed the extension of the shale layer under the left abutment of the saddle dam. Drill Holes DD1-DD3 were pressure tested at various depths using a single packer. Water losses were negligible.

The site of Saddle Dam No. 2 was investigated by surface mapping, diamond drill holes, and seismic traverses. Seven diamond drill holes were drilled on the site. Six of these holes were pressure tested as above. Water losses occured in all holes with losses of up to 8.7 and 18.4 lugeons occuring in holes DD7 and DD1 respectively. Because of the flat nature of the saddle and consequent long seepage paths, the foundations were not grouted. However in periods of high water levels in the reservoir, areas downstream of the saddle dam should be inspected for seepage. Investigations confirmed that the saddle dam was located on part of the Great Moreton Fault System.

Photos and logs of the diamond drill holes at the saddle dams' sites, a geological report and plans and sections of the sites are included in Reference 16, Part 3.

9.2.8 Seismic Traverses

In initial investigations, seismic traverses were used to assist in determining the choice between the A axis and the C axis of the dam embankment. Traverses were run on both the A and C axes and longitudinally on the proposed spillway axis on the left bank. Generally speaking it appeared that there was a close correlation between the base of the weathered rock and the water table. The seismic profiles showed that generally the base of weathering was about 25 m from the surface.

Seismic traverses were also run at the site of Saddle Dam No. 2. The location and profiles are shown in Reference 16, Part 3, Figures 13 and 14.

In 1976 a seismic traverse was conducted along the line of the left hand training and retaining walls of the spillway on the left bank. The location and profiles are shown in Reference 16, Part 3, Figures 1 and 2.

9.3 Conditions Exposed by Foundations Investigations

9.3.1 Soils

The dozer trenching exposed the surface of the sandstone rock at shallow depths except for the cross river section between distances 1450 m and 2000 m and the section high on the left abutment between distances 150 m and 200 m.

The shallow soils overlying the sandstone were found to consist mainly of slopewash and residual soils resulting from the weathering of the underlying rock. The soils derived from the sandstone generally formed sandy clays with fairly sound strength properties. Soils derived from the insitu weathering of shales formed highly plastic erodible clays of doubtful strength. These clays were detected at the upstream end of the spillway and on the right bank in the vicinity of the diversion channel. An elevated portion of the right bank was found to contain what is believed to be an old river channel. It contained uncemented boulder gravels in a sandy clay matrix. This material was removed from the foundation .

The drilling program revealed that the rock surface on the left abutment fell towards the river valley in two major steps which were thought to represent two recessional erosion profiles left by westward migrations of the river channel.

The area between the two rises was covered with up to 8m of overburden soils consisting of two main stratigraphic units. The lowest unit varied from whitish clayey-gravel to a whitish sandy-clay over the whole step. The upper unit consisted of recent sediments of reddish sandy-clay of slope wash origin at the eastern extremity and a brown boulder/clay of mudflow origin at the western extremity. No organic material was found in any of these units.

The valley floor was covered with up to 24m thickness of alluvium. The deep alluvium occurred on the eastern side of the valley and was probably deposited by successive floodings of the river as it eroded its way west. The stratigraphic units going from top to bottom were:

(i) brown medium plastic clays containing some boulders;

(ii) light brown clays becoming siltier to the west ;

(iii) silty and sandy clays with decreasing clay content to the west;

(iv) layered poorly graded sands and silty sands;

(v) well graded sands and sandy gravels containing some partially carbonized tree trunks.

The top unit extended onto the area between the two rises and exhibited fissuring in the vicinity of the initial rise. This was thought to be due to consolidation of the river bed alluvium as the "mud flow" materials built up.

The western side of the river valley contained fresh sediments of sand and gravel with large sandstone blocks occurring against the steeply rising right bank.

Prior to excavation of the core trench no definite stratigraphic correlation was possible between investigation holes. This occurred because some investigation holes had intersected buried erosion gullies which contained highly variable backfill, including some highly plastic clay while other holes were sunk in recently deposited silty sand banks at the river side of the deep alluvium.

9.3.2 Rock

The rock exposed by the dozer trenching revealed that the strikes of the layered sandstone were, in general, in the ENE direction . The dips were variable but generally flat (< 10 degrees). Some of the interbedded joints exposed on the right bank contained clayey materials and required further investigations. No steeply dipping joints were revealed by the investigation. Apart from a shallow bank of semi-cemented conglomerate on the right bank, the foundation rock exposed by trenching was all sandstone.

Diamond drilling was used to investigate the areas covered with deep alluvium and the subsurface behaviour of the thick shale on the right bank.

The drilling on the right bank established the location and the shape of the shale unit but failed to define the limit of weathering. The existence of sandstone outcrops on the steep right bank which were not conforming to the general bedding pattern suggested that the upper sandstone beds were undergoing downhill movement and settling on the weathering shale. The right bank was to contain the diversion channel and it was essential that both sides of the channel consist of sound rock.

During construction, the right bank was stripped of overburden to locate sound rock to form the western extremity of the cofferdams. Numerous sandstone "floaters" were revealed and the weathering face of the shale was exposed. Tension cracking of the sandstone above the weathered shale was near vertical and transverse to the dam axis. Very plastic material near the top of the weathered shale was found to have a low friction angle of 9 degrees. The shale layer was removed from the foundations under the core zone of the embankment.

9.4 Conclusions Drawn from Foundation Investigations

As a result of the site foundation investigations the following conclusions were drawn:-

(a) A wide range of materials to be stripped or excavated from the dam embankment between the spillway on the left bank and the Brisbane River was suitable for placement in the miscellaneous fill Zone 1D of the left bank sloping core embankment. The materials included weathered rock, granular material, sandy clays, and clays of low plasticity obtained from excavation of the deep alluvium on the left bank of the Brisbane River adjoining the river. Non-plastic silts, silty sands and fine sands from the area would be suitable for the filter (Zone 2D) of the cross river and right bank section of the dam.

(b) Weathered to fresh sandstone to be excavated from the diversion channel (excluding the shale layer which would have to be wasted), and from the spillway channel, would be suitable for inclusion in the rolled sandstone Zones 3B and 3E of the cross river and right bank sections of the embankment.

(c) The alluvium on the abutment to the left of the spillway (Distances 0 m to 1150 m) would be a satisfactory foundation for the miscellaneous fill (Zone 1D) of the left bank sloping core embankment.

(d) The presence of the thick shale and fine sandstone layer (up to 9 m in thickness dipping about 5° downstream) and more weathered rock on the right bank showed the right bank to be inferior to the left bank for the spillway. If the spillway was to be sited on the right bank of the river, a large quantity of unsuitable material would have to be removed, and extra material suitable for Zones 3B and 3E of the embankment would have to be found from other sources. Siting of the spillway on the left bank provided more competent foundations and produced suitable material for the embankment. The presence of the shale layer presented difficulties for construction of tunnels for diversion during construction and the permanent outlet works on the right bank.

(e) The presence of the deep alluvium on the left bank of the Brisbane River and the depth to competent rock, requiring large volumes of concrete, made the cross river spillway uneconomical.

(f) The presence of clay seams and weak layers, at upper levels on the proposed left bank spillway site, indicated that the spillway monoliths should be founded below the level of the clay seams at approximate EL 36.0.

(g) The sandstone from required excavations at the dam site was not durable enough for riprap.

(h) The side slopes of the diversion channel would have to be presplit because of the weathered nature of the sandstone, and the exposed shale seam transversing the channel would have to be protected.

10.0 SOURCES OF EMBANKMENT CONSTRUCTION MATERIALS

10.1 Sources of Embankment Construction Materials - General

The Queensland Co-ordinator General's Department commenced initial investigations for sources of construction materials in 1969 (Reference 1). Tests confirmed that sandstone from the spillway excavation would be suitable as rolled fill in the pervious sections of the dam embankment. Suitable core material was found on the right and left banks (the left bank site between the A and C axes ultimately being designated Borrow Area WX11) of the Brisbane River at the dam site. Filter material was also available, although it was thought it may have been possible to dispense with filters if there was sufficient highly weathered sandstone to place next to the clay core.

Investigations by the Water Resources Commission (and its predecessor the Irrigation and Water Supply Commission) were conducted during the period July 1975 to March 1977. This section provides details of these investigations. Further details may be found in References 15, 16 and 17.

Soils were classified in accordance with the Unified Soil Classification System, with the term medium plasticity referring to soils having a liquid limit in the range 35% to 50%.

The location of all borrow areas is shown in Figure 20 (Drawing A3-53372D.)

10.1.1 Earthfill Materials

Three sources of Zone 1A material - clay core material for the main dam embankment - were investigated. These sources were Borrow Areas WX10, WX11 and WX21.

The materials for the fill for the saddle dams were designated Miscellaneous Fill and Miscellaneous Weathered Conglomerate. For Saddle Dam No. 1 the borrow area was WX24, Hahn's Paddock, a gently sloping weathered sandstone area at the base of a high ridge forming a southerly extension of the left abutment of the saddle dam. The borrow areas for the Miscellaneous Fill Zones of Saddle Dam No. 2 were WX27 and WX29. Borrow Area WX27, Spearhead Flat, was an alluvial terrace on the left bank of the Brisbane River about 0.5 km northwest of Saddle Dam No. 2. Borrow Area WX29, Conglomerate Ridge, was an area of deeply weathered conglomerate immediately upstream of Saddle Dam No. 2.

Zone 1D material was a Miscellaneous Fill Zone on the left abutment of the dam between distances 100 m and 1150 m. A wide range of materials was acceptable including weathered rock, granular materials, sandy clays and clays of low plasticity. The source of this material was any approved source but in general was to originate from foundation excavations. The lower portion of the zone consisted of alluvial clays, sands, silts and gravels. The upper portion consisted of weathered sandstone.

10.1.2 Filter Materials

Four different grades of filter were required in the dam embankment. These were:- Zone 2A, a fine sand filter; Zone 2B, a coarse sand filter, +4.75 mm; Zone 2C material, a coarse sand filter, -20 mm; and Zone 2D material, a sandy silt. The primary source of these materials was Borrow Area WX14, Atkinson's Crossing, a low level flood overflow channel on the right bank of the Brisbane River immediately downstream of the diversion channel outlet. A reserve source of material for filters, Zones 2A, 2B and 2C was Borrow Area WX15, Roseborough gravel/sand, a low level flood overflow channel located approximately 0.5 km immediately downstream of Borrow Area WX14. Borrow Area WX14 was directly in the path of any discharges from the proposed right bank diversion channel, and would have been disturbed by floods diverted through the channel once it was constructed. It was therefore decided to let a contract, prior to construction of the channel, to excavate, process and stockpile the alluvial loams, sands, gravels and cobbles contained in these deposits, and the processed materials for Zones 2A, 2B and 2C and for concrete aggregates were made available to contractors, ex stockpile, for $3.50 per cubic metre.

10.1.3 Rockfill Materials

Rockfill materials covered a wide range of materials. Zone 3A material consisted of hard, durable, free draining river gravel. The source of this material was the northern section of Borrow Area WX16, Hay's Crossing, a low level flood overflow channel on the right bank of the Brisbane River approximately 1 km north of Northbrook Bridge. Borrow Area WX31 on Northbrook Creek, near its junction with the Brisbane River was also investigated and approved as a source of Zone 3A material.

Zone 3B, Zone 3C and Zone 3E materials consisted of a mixture of weathered to fresh sandstone rock fragments obtained from required open cut excavation for the spillway.

Zone 3D material was hard durable well graded river run gravel obtained from Borrow Area WX15.

Materials for Zone 4A, upstream riprap, (maximum size 900 mm with not more than 50% less than 500 mm) and Zone 4B, downstream riprap (maximum size 900 mm with not more than 50% less than 300 mm or 100 mm, depending on its location on the embankment) were intended to be won from Quarry No. 6. Quarry No. 6 was situated in the foothills on the left bank of Northbrook Creek approximately 6 km north of the dam site.

10.2 Borrow Areas Investigated for Zone 1A Material for the Main Dam and Miscellaneous Fill Zones for the Saddle Dams.

10.2.1 Borrow Area WX10 Investigations

Borrow Area WX10 was situated only 0.5 km north of the upper left abutment of the dam.

Investigation of the feature was done in two stages. In the first phase, April to June 1975, fourteen percussion holes were drilled. In the second phase during the period April-May 1978, fourteen power auger holes were drilled and eight backhoe trenches excavated. The percussion holes ranged in depth from 6.0 m to 22 m and were terminated either at refusal on sandstone or after encountering obviously unsuitable material. The backhoe trenches ranged in depth from 2.2 m to 4 m, and were terminated either upon encountering sandstone or obviously unsuitable material or at the machine's limit of 4 m. A variety of soils ranging from clays, generally of low to medium plasticity, to clayey and silty sands, with some poorly graded sands and gravels, was found in the borrow area.

10.2.2 Borrow Areas WX11 and WX30 Investigations

Borrow Area WX11 was investigated by twenty percussion drill holes, ten power auger holes, nineteen backhole trenches and one bulldozer trench. These excavations ranged in depth from 1.2 m to 29.3 m. The materials encountered in the investigations were variable and included clays of low to high plasticity and clayey sands of low to medium plasticity.

Borrow Area WX11 was ideally situated to supply material for Zone 1A in the central section of the dam embankment. However during construction it became evident that the expected quantity of CL material was not available from Borrow Area WX11 and the grading of the available material was not completely satisfactory. For a short period Zone 1A material won from WX11 was supplemented by clay won from Splityard Creek Borrow Area WP14, whilst the Commission investigated Borrow Area WX30 as an alternative source. Borrow Area WX30 was situated upstream of Wivenhoe Bridge on the northern side of the river, and was investigated by trenches on a 25m x 25m grid and a trench, 200m long by 5m wide and 4m deep, excavated by scrapers. Material from Borrow Area WX30 met specification requirements and was placed in the dam embankment from July 1980. The location of Borrow Area WX30 is shown in Figure 20 (Drawing A3-53372D).

10.2.3 Borrow Area WX21 Investigations

This feature was investigated by five auger holes and twenty nine backhoe trenches. The excavations ranged in depth from 1.1 m to 3.5 m. The materials encountered in the investigations were variable but consisted mainly of clays of high plasticity and sandy clays of low to medium plasticity.

Because of the borrow area's close proximity to the diversion channel, the material in Borrow Area WX21 was reserved for the Zone 1A material required for the closure of the diversion channel.

10.2.4 Borrow Area WX24 Investigations

Borrow Area WX24 was situated on the eastern side of the Brisbane Valley Highway extending downstream from the site of Saddle Dam No. 1 for a distance of approximately 0.5 km.

The feature was investigated by seventeen backhoe trenches varying from 0.9 m to 2.3 m in depth, the depths of excavation generally reflecting the refusal of the backhoe to penetrate the underlying sandstone. The materials encountered in the investigations consisted of silts and clays of medium to high plasticity, and poorly graded clayey sand of low to medium plasticity.

The borrow area was reserved for the Miscellaneous Fill Zone of Saddle Dam No. 1.

10.2.5 Borrow Area WX27 Investigations

Borrow Area WX27 was situated approximately 0.5 km north west of Saddle Dam No. 2.

The feature was investigated by three power auger holes and five backhoe trenches. The depths of the auger holes varied from 4.1 m to 9.3 m and the depth of the trenches from 1.7 m to 3.8 m.

The materials encountered in the investigations consisted mainly of clays of medium to high plasticity, sandy clay of medium plasticity and some poorly graded gravels.

The borrow area was reserved for the use in the Miscellaneous Clay Fill Zone of Saddle Dam No. 2.

10.2.6 Borrow Area WX29 Investigations

Borrow Area WX29 consisted of a ridge extending in a north-westerly direction from the left abutment of Saddle Dam No. 2.

Several backhoe trenches were excavated in the feature, some being stopped by hard material at depths of less than 1 m. Three trenches were excavated to depths between 1.8 m and 3.0 m and revealed weathered material mainly classified as poorly graded clayey gravels and clayey sands of low plasticity.

The borrow area was reserved for the supply of material for the Miscellaneous Weathered Conglomerate Zone of Saddle Dam No. 2.

10.3 Borrow Areas Investigated for Filter Materials in the Embankment

10.3.1 Borrow Area WX14 Investigations

Borrow Area WX14 was investigated by twelve percussion drill holes and twenty five bulldozer trenches. The depths of the percussion holes varied from 6 m to 12.6 m and with one exception were drilled to refusal on sandstone. The depths of the dozer trenches varied from 1.5 m to 11.1 m and were terminated just below the water table in the coarser grained sediments or in the case of the dozer trenches at the limit of stability of the trench walls in finer grained alluvium.

The materials encountered in the investigations were variable and included poorly graded gravels and sands, silty sands and well graded gravels.

A contract for excavation, processing and stockpiling of gravels from Borrow Area WX14 was let to W. Wall and Sons. Processed materials were sold to dam contractors for use in Zones 2A, 2B and 2C.

10.3.2 Borrow Area WX15 Investigations

Borrow Area WX15 was investigated by six percussion drill holes and thirteen bulldozer trenches. The depths of the percussion drill holes varied from 4.6 m to 9.9 m and were all drilled to refusal on sandstone. The depths of the dozer trenches varied from 2.1 m to 5.1 m and were terminated due to caving.

The materials encountered in the investigations were mainly classified as poorly graded gravels and sands.

The borrow area was an approved source of materials for Zone 3D of the dam embankment and a reserve source of supply for filter materials, Zones 2A, 2B and 2C.

10.4 Borrow Areas Investigated for Rockfill Materials in the Dam Embankment

10.4.1 Borrow Area WX15 Investigations

Borrow Area WX15 was an approved source of rockfill material for Zone 3D of the dam embankment as well as an approved source for filters Zones 2A, 2B and 2C. Investigations in Borrow Area WX15 are described in Section 10.3.2 above.

10.4.2 Borrow Area WX16 Investigations

Borrow Area WX16 was situated on the right bank of the Brisbane River approximately 1 km north-west of the Northbrook Bridge and approximately 6 km by road north of the dam site.

The feature was investigated by four percussion drill holes, one hand auger hole and thirty eight bulldozer trenches. The depths of the percussion holes varied from 5.7 m to 13.8 m with three of the four holes terminating on sandstone; the hand auger hole was drilled to 1.5 m; and the depths of the dozer trenches varied from 0.9 m to 8.2 m, the majority terminating at the water table.

The materials encountered in the investigations were mainly classified as poorly graded gravels and sands. Material from the northern section of WX16 was approved for Zone 3A of the dam embankment.

During construction of the embankment, Zone 3A material from Borrow Area WX16 was exhausted and gravel meeting the specification requirements for Zone 3A material was won from Borrow Area WX31. The location of Borrow Area WX31 is shown in Figure 20 (Drawing A3-53372D).

10.4.3 Investigations for Materials for Zones 3B, 3C and 3E of the Dam Embankment

For Zone 3B, any weathered to fresh sandstone that could be won by ripping was suitable providing that its maximum size could be accommodated in a layer thickness of 1.0 m. For Zone 3C relatively impermeable sandstone with a maximum size of 0.5 m was required for construction of the main cofferdams. For Zone 3E, selected sandstone, of such a grading that would prevent migration of Zone 2B material into Zone 3B or vice versa, was suitable. After compaction, Zone 3E was to be well graded and the fraction passing 19mm was to be not less than 15% by weight and the maximum particle size was not to exceed 300mm.

Materials for Zones 3B, 3C and 3E were to be obtained from excavation of the spillway.

Investigation of the spillway area is covered by Section 9.2 - Dam Site Investigations.

10.5 Borrow Areas Investigated for Riprap

10.5.1 Sites Investigated and Discarded

Difficulty was experienced in locating rock of sufficient durability and size for Zones 4A and 4B of the dam embankment. Sandstone from the required spillway excavation was not durable enough to use as riprap on the upstream face of the dam.

Sites investigated and/or considered were as follows:-

(a) Quarry Site No. 1 was situated on the south bank of the Brisbane River approximately 0.5 km upstream of the Northbrook Bridge. The site was investigated by trenching, drilling, and a trial blast. The conglomerate at this site broke down into sizes too small for use as riprap.

(b) Quarry Site No. 2 was situated on the left bank of Sheepstation Creek on an outcrop of the same conglomerate existing at Site No. 1, and was discarded as a suitable source without further investigations.

(c) Quarry Site No. 3 was situated on a ridge above Saddle Dam No. 2. The only hole drilled intersected fractured quartzite, andesite and greenstone and indicated that the site would not have yielded rock of sufficient size.

(d) Quarry Site No. 4 was situated on the right bank of Northbrook Creek, 11 km from the dam site. The site was investigated by diamond drilling, rotary percussion drilling, and seismic refraction traverses. Because of the deep weathering of the rock and the distance from the dam site, the site was considered unsuitable.

(e) The Tailrace Channel of the Wivenhoe Dam pump storage Hydro-electric Power Station was also considered as a source for riprap. The material would have to be excavated and stockpiled by the contractor for the power station. However this material also proved to be unsuitable.

10.5.2 Quarry No. 6 Investigations

Quarry No. 6 was investigated by eight diamond drill holes, twenty seven air track percussion holes and six shallow dozer trenches. The penetration rates of the percussion holes were recorded as a guide to rock quality.

The quarry was located within beds of the Neranleigh-Fernvale group of metamorphic quartzites and intrusive volcanics (Greenstone). A wide bed or sill of greenstone combined with two adjacent narrow beds of quartzite and greenstone were expected to yield rock of sufficient size and quality for Zones 4A and 4B of the embankment.

Although Quarry No. 6 was the approved source of riprap for the dam, all riprap placed on the dam came from the Split-Yard Creek Quarry.

10.5.3 Split-Yard Creek Quarry

Split-Yard Creek quarry, originally designated Feature WP7, was the required excavation for the spillway for Split-Yard Creek Dam and its extension into the reservoir storage area. It was situated approximately 1 km north of the Split-Yard Creek dam embankment as shown in Figure 20 (Drawing A3-53372D).

Details of the investigations for the quarry are included in the reports, "Report on Sources of Construction Materials for Split-Yard Creek Dam" (Reference 18), and "The Geology of Split-Yard Creek Dam Site (Reference 19).

Over 2.7 million cubic metres of overburden and rock was removed from the quarry during construction of Split-Yard Creek Dam prior to its use for Wivenhoe Dam. Rock for Zones 4A and 4B for Wivenhoe Dam was obtained by extending the existing benches.

11.0 DESIGN OF EMBANKMENTS

11.1 Design Criteria

The feasibility studies (References 1, 6 and 14) and site investigations provided enough information to establish the design criteria which are listed in Table 17. Although the Full Supply Level for the dam is EL 67.0, the embankment stability analysis was performed using a Design Water Level of EL 69.0 to allow for future raising of the dam Full Supply Level.

TABLE 17 - EMBANKMENT DESIGN CRITERIA

<u>Water Levels</u>	
Full Supply Level	EL 67
Design Water Level	EL 69
Maximum Flood Level	EL 77
<u>Design Floods</u>	
Maximum Probable (Maximised 1893)	15000 m^3/s
<u>Discharge</u>	
Maximum	11700 m^3/s
Design	5000 m^3/s
Zero Damage	3000 m^3/s
<u>Design Wind</u>	80 kph (1:10)
<u>Crest Dimensions</u>	
Crest Width	14 m
Minimum Curvature (crest)	1000 m
<u>Slope Stability - Minimum Safety Factors</u>	
Steady Seepage	1.5
Drawdown	1.25
End of Construction	1.25
<u>Construction Materials</u>	
Core	Clays and sandy clays
Filters	Processed sands/gravels
Shell	Sandstone Fill
Riprap	"Greenstone" Rockfill
Other Fill	River Alluviums and Slope Wash Materials

11.2 General Arrangement of Embankment

The general arrangement of the embankment and typical sections are shown in Figure 2 (Drawing A1-50820B) and Figure 22 (Drawing A1-49868B).

11.3 Embankment Construction Specification

The properties of the materials required for the embankment construction were specified for the Contractor, and sources of the materials were nominated. Testing of the soils in the embankments was in accordance with AS A89 1966-1973 , Testing Soils For Engineering Purposes and AS 1289 - 1977, Methods of Testing Soil for Engineering Purposes.

The materials specifications and construction requirements for the various zones in the embankments were as follows:-

(a) Zone 1A

Selected impervious soils classified as SC, SC-CL, CL-SC and CL under the Unified Soil Classification were used for the embankment core zones.

The properties of the material required were:-

(i) acceptable soils to have a liquid limit for a range of percentages of material passing the 75 μm seive as shown in Figure 23;

(ii) the plasticity index not less than 12 percent;

(iii) the grading curve for the material lies within the limits for Zone 1A material and is approximately parallel to the curves defining the grading limits as shown in Figure 23.

The material was to be conditioned to fall within the range of optimum moisture content plus or minus 1 per cent, for standard compaction, and placed in horizontal layers of 150mm in thickness after compaction.

Tamping rollers with a weight of not less than 6 tonnes per metre length of drum were specified for compaction. The required compaction was that in any group of ten consecutive control tests all had to have a dry density in excess of 97% and at least nine needed to exceed 98% of standard maximum dry density.

(b) Zone 1A Contact Material

The material was to be similar to Zone 1A except that more than 55 percent was required to pass the 75μm sieve size and have a liquid limit between 45 and 55 percent. Grading limits are shown in Figure 23.

The clay was to be placed in 150mm thick layers after compaction, and rolled and worked into the foundation by the passage of pueumatic tyred construction machinery. Contact clay was to be placed only in the first 600mm above the rock foundation. Where foundation treatment was required as described in Section 11.8.2, the contact material was placed against the rock foundation immediately after applying the foundation treatment of a thin layer of grout or mortar.

(c) Zone 1D

The miscellaneous fill zone was to support the impermeable sloping core of the left bank embankment.

A wide range of materials was acceptable for Zone 1D, including weathered rock, granular material, sandy clays, and clays of low plasticity. Clays and sandy clays with a liquid limit in excess of 50 percent and organic material were not permitted.

The material was generally obtained from the foundation excavation between Distances 1273 m and 2450 m, and from overburden and weathered portions of the spillway excavation.

As a result of the construction sequence, the lower portion of the left bank embankment consisted of alluvial clays, sands, silts and gravels, and the upper portion consisted of weathered sandstone from the spillway excavation.

Prior to placing the fill material, the foundation was to be scarified, sprayed with water or dried, and compacted by four passes of a vibratory pad foot roller of a weight not less than 10.5 tonnes.

Cohesive material was to be placed in 300mm thick layers, after compaction with the vibratory roller, and compacted to a dry density of not less than 95% of standard maximum dry density with an insitu moisture content within 2% dry and 1% wet of optimum moisture content.

Non-cohesive materials were to be placed in 300 mm thick layers after compaction and compacted by two passes of the vibratory roller.

(d) Zones 2A, 2B, 2C, 2D and 2E

Zones 2A, 2B, 2C and 2D were to be obtained from the Principal's stockpiles, and 2E was to be obtained from a borrow area.

Zone 2A was to consist of washed well graded river sands having a nominal maximum particle size of 4.75mm and grading limits as shown in Figure 24.

Zone 2B was to consist of washed poorly graded river gravels with a nominal minimum particle size of 4.75mm and a nominal maximum aggregate size of 37.5mm, and grading limits as shown in Figure 25.

Zone 2C was to consist of washed river gravels with a nominal maximum particle size of 19mm and grading limits as shown in Figure 26.

Zone 2D was to consist of non plastic silts, silty sands, and fine grained sands.

Zone 2E was to consist of selected semi-pervious soils classified as SM, SP and SC under the Unified Soil Classification System, and grading limits as shown in Figure 27.

All Zone 2 material was to be compacted in 300mm thick layers, after compaction with vibratory rollers having a weight of not less than 4.5 tonnes. Each layer was to be conditioned to the required moisture content and compacted as follows:-

- Zones 2A, 2B, 2C and 2D to a density index of not less than 70%.
- Zone 2E to a density index of not less than 70% except where the material contained excess fines, compaction was to be 95% of standard maximum dry density.

(e) Zone 3A

Embankment Zone 3A material was to consist of hard durable free draining river gravel selected from borrow pits.

The grading limit of the fine material and requirements for blanket drains are shown in Figure 28.

The material was to be throughly wetted with not less than 50 litres of water per cubic metre of gravel, and compacted in 1m thick layers after compaction by vibrating roller weighing not less than 10 tonnes. The number of passes was to be not less than two, and the material was to be compacted to a density index not less than 70 per cent.

(f) Zones 3B, 3C and 3E

Embankment materials for Zones 3B, 3C and 3E were to consist of weathered to fresh sandstone obtained from the spillway excavation.

In general the foundation for the Zone 3 material was alluvial material or sandstone rock.

The transition Zone 3E was required to have a grading which would prevent migration of Zone 2B material into the Zone 3B shell. The Zone 3E after compaction was to be well graded, the fraction passing the 19mm sieve size was to be not less than 15% by weight, and the maximum particle size after compaction was to be not greater than 300mm.

In the coffer dams, Zone 3C was required to be relatively impermeable to exclude flood water from the works area, and the finer material of lowest permeability was to be placed towards the outside faces.

Excavated material not required for Zones 3C or 3E was to be used for Zone 3B. The maximum nominal particle size for Zone 3E was 300 mm, and for Zones 3B and 3C the maximum size was equal to the layer thickness.

Material for Zone 3 was to be conditioned and compacted in layers of 1m for Zones 3B and 0.5m for Zones 3C and 3E, after compaction. Each layer of material was to be compacted with four passes of a vibratory roller with a weight not less than 10 tonnes.

(g) Zone 3D

The Zone 3D material located under the mesh protection was to be hard durable unprocessed river gravel and was to be obtained from borrow areas.

The grading limits of the material are shown in Figure 29.

Zone 3D was to be thoroughly wetted with at least 50 litres of water per cubic metre of gravel, and compacted in layers not exceeding 1m in thickness after compaction. Each layer was to be compacted by two passes of a vibrating roller with a weight not less than 10 tonnes.

(h) Zones 4A and 4B

The materials for the rip-rap Zones 4A and 4B of the embankment were to consist of hard durable rock obtained from a quarry. Zone 4A was upstream riprap; Zone 4B was downstream riprap.

- Zone 4A rip-rap, in the saddle dams and between Distances 100m and 1100m on the embankment, was to have a maximum size of 900mm with not more than 50 percent less than 500mm, and did not require compaction.

- Zone 4A in the groynes and between Distances 1 200m and 2 400m on the embankment was to have a maximum size of 900 mm with not more than 50 percent less than 500 mm and was to be compacted, in layers of 1.5 m maximum depth, by two passes of a vibrating roller with a weight not less than 10 tonnes.

- Zone 4B to the right of the spillway below EL 53.0 was to have a maximum size of 900 mm with not more than 50 percent less than 300 mm. All Zone 4B to the right of the spillway was to be compacted, in layers of 1m maximum depth, by either two passes of a vibrating roller with a weight not less than 4.5 tonne, or by one pass of a vibrating roller having a weight not less than 10 tonne.

- Zone 4B, except that to the right of the spillway below EL 53.0, was to have a maximum size of 900 mm with not more than 50 percent less than 100 mm. Between Distances 100m and 1100m, Zone 4B was to be compacted, in layers of 1m maximum depth, by two passes of a vibrating roller with a weight not less than 4.5 tonne.

11.4 Flood Protection and Diversion Works

Construction of Wivenhoe Dam commenced in March 1977, and the dam was completed in November 1983. As the construction extended over a number of years, the dam was to be constructed in stages during the dry period of each year, and made secure at the beginning of each wet season to minimise the risk of flood damage.

The flow in the river was diverted through the trapezoidal channel with 45 degree batters while foundation treatment and embankment construction was carried out in the river bed. The diversion capacity and the height of the coffer dams were chosen to give the cross river works an 80 percent probability of not being inundated in any one year. A 1:200 model verified the calculated discharge rating of the channel, and the discharge capacity rating curve is shown in Figure 17 (Drawing A3-49893A).

The location of the diversion channel was determined after examining the sandstone surface exposed by stripping the right bank.

During the early stages of diversion, the downstream slope of the upstream coffer dam was protected with "polyfabric" as shown in Figure 30 (Drawing A1-45419A) and Figure 31 (Drawing A1-45420B). The upper edge of the fabric sheet was embedded in the embankment and the bottom edge was taped to the lower sheet.

Hydraulic model studies and calculations indicated that the drag force from the water flowing down the slope for a 1 metre head over the crest was 1kPa, and the stability of the fabric in the flow increased with the smoother conditions underneath.

The sandstone fill and the river bed alluvium were both easily eroded and any overflow on the partly completed embankment could destroy unprotected works. The flood protection system designed to resist overtopping consisted of an armouring of durable rock held down by a grillage of steel mesh and reinforcing anchored into the fill as shown in Figures 31 and 32 (Drawings A1-45420B and A1-49869B). Flexible rockfill mattresses held together by steel mesh were located at the toe of the overflow section to control erosion of alluvium and prevent undermining of the embankment as shown in Figure 33 (Drawing A1-62303B). The overflow was concentrated in a low section of the coffer dam to minimise the extent of the flood protection works necessary.

As a further protection and to prevent "peeling" of the protection works, the crest of the coffer dam was capped with a concrete slab which sat on a 1 m thickness of cement stabilised sandstone. The western batter of the coffer dam was also constructed of cement stabilised sandstone to prevent the eastward erosion of the coffer dams from the diversion channel.

During construction of the downstream riprap at the toe of the embankment, the rock grading was less than desirable in relation to the dimensions of the reinforcement mesh. As a result, backfill concrete was placed over most of the downstream mesh protection to prevent its removal.

The mesh protection prior to embankment closure is shown in Figure 34 (Drawing A1-50779C). Mesh protection over the earth core and filters was removed prior to embankment closure.

11.5 Determination of Final Crest Level

11.5.1 Wave Run Up and Riprap Size

The minimum embankment crest level was fixed at EL 79.0 with a concrete crest barrier to EL 79.7. The freeboard allowance for waves developing in the storage and the rip-rap size required to prevent erosion of the embankment were determined from References 20 and 21.

Wind velocities and return periods were determined by the Bureau of Meteorology based on information at Amberley, as it was the nearest site possessing wind records, and located more or less in the same topographical and climatic regime.

The adopted design wind speed of 80 kph of sufficient duration to develop waves, has an annual exceedence probability of 1:10. The fetches for wave height calculations were 12 km for the Saddle Dams and 4 km for the Main Dam. For the 80 kph wind and 12 km fetch the height of wave run-up was 1.2 m.

When passing the PMF at EL 77.0, and adopting the 1.2 m wave run up for the saddle dams, the crest level required for the main embankment would be EL 78.2.

In the possible but improbable event of complete loss of supervision or power for opening the spillway gates, the 1:500 flood would reach EL 79 with a discharge of 2202 cumecs over the top of the gates. The 1974 flood would also pass safely with discharge over the top of the gates under the same circumstances.

Two saddle dams were required to provide flood storage capacity. As the saddle dams' foundations consisted of erodible materials, a very weathered sandstone and a clay bound conglomerate, the crests of the saddle dams were built one metre higher than the main embankment at EL 80.0, to ensure that they would not be overtopped.

The rip-rap size determined for the wave height generated by the design wind velocity was 500mm nominal maximum dimension.

In the subsequent Brisbane River and Pine River Flood Study (1993), wind set up and wave run up were examined for the full supply level of EL 67.0 and a maximum flood level of EL 80.0. Because a public road crosses the crest of Wivenhoe Dam, a significant wave height corresponding to the average of the top 1% in the wave spectrum was used in the calculations and the wave wall on top of the embankment was ignored. The results of the study including the totals of the wind set up and wave run up during extreme flood events and a summary of possible wind and flood combinations and the resulting reservoir levels are given in Table B7 of Report 24 of Appendix B of this report.

11.5.2 Fuse Plug Studies

Consideration was given to incorporating fuse plugs in the dam embankment and saddle dams. In the event of the spillway gates failing to open, the erosion of a fuse plug would control the release of flood flows thus preventing overtopping of the embankment, and reducing the risk of flood damage in densely populated areas.

Fuse plug embankment sections were designed to scour out when the reservoir level reached a certain critical stage, thus providing an increased outflow capacity.

The Probable Maximum Flood had a peak of 15000 m^3/sec and a volume of 4 209 000 ML. If no fuse plug was used, this flood could be easily passed, even with one gate remaining closed, without overtopping the embankment. With all gates remaining shut the embankment would be overtopped, with the water level reaching EL 80.87. With all the gates operated normally, the maximum water level would not exceed EL 77.0 during the PMF.

Without a fuse plug, the dam could be endangered only if the flood magnitudes were seriously underestimated and/or some or all of the gates failed to operate at the time when very large floods were is progress.

The only possible fuse plug locations were the two saddle dams and the extreme right hand end of the main dam. Examination of cores and dozer trenches indicated that scour at the saddle dams would proceed below full supply level, and in the case of Saddle Dam No. 2 might reach well in excess of 12m below fully supply level. Therefore cutoffs would be required at the saddle dams.

At the right hand end of the main dam, the sandstone was sufficiently resistant to erosion to make a extensive cutoff unnecessary.

Failure of any of the fuse plugs would cause flooding of the Brisbane Valley Highway, and discharge from the saddle dams' fuse plugs would flow over unresumed land.

Trials with various configurations of fuse plugs required to pass the Probable Maximum Flood led to the conclusion that to keep the water level below embankment crest level, if all gates remained closed, a fuse plug of high capacity was needed. A high capacity fuse plug would be best divided into three sections which would fail at different head water levels and thereby minimise the discharge resulting from failure with lesser floods.

The flood levels chosen and the corresponding discharges resulting from failure of the sections were:-

EL 78.0	-	5340 m^3/sec
EL 78.4	-	9480 m^3/sec
EL 78.8	-	14080 m^3/sec

Discharges were calculated using a broad crested weir discharge coefficient.

Cost estimates for the various fuse plug locations found that the right end of the embankment was preferred.

A fuse plug failing at different head levels introduced the risk of breaching one or more parts of the fuse plug for lesser floods if the gates did not operate. In these circumstances, the 1974 flood would have breached the first section and the 1893 flood probably two sections; each case greatly increasing the outflow above that resulting from proper gate control.

The decision not to provide a fuse plug was made in conjunction with a decision to install concrete traffic barriers as discussed in Section 11.5.3 below.

11.5.3 Effects of Installing Concrete Crash Barriers

Investigations were carried out using a crash barrier to effectively raise the crest of the dam. The results of the investigations shown in Table 18 show reservoir levels and discharges for the Probable Maximum Flood for a number of different gate opening conditions, and for the 1:1000 and 1:500 floods if no gates are opened, with and without a 2500m^3/s fuse plug failing at EL 79.0.

With a concrete wall to EL 79.7 extending onto the abutments and no fuse plug, the 1:1000 flood would be passed with only a slight flow over the dam if all gates remained closed, and the Probable Maximum Flood would be passed with three gates remaining closed (two gates operating), provided the most effective use was made of the two operating gates.

It was therefore decided not to provide a fuse plug, but to install a 700mm high concrete crash barrier on the upstream side of the crest, thereby raising the nominal crest level to EL 79.7. Details of the traffic barrier are shown in Figure 35 (Drawing A1-54666A). The crest level of the saddle dams at EL 80.0 was consistent with this strategy.

As well as withstanding traffic loads on the downstream side, the crash barrier has been designed for a static water load on the upstream side.

The crash barrier does not provide a crest at EL 79.7 completely across the dam. A gap of approximately 45 m was left at the extreme end of the left abutment to provide access to the picnic area. The lowest point on the road in this area is EL 79.25. The area in the vicinity of the access road to the picnic area would have to be raised by 450 mm to extend a crest level of EL 79.7 completely across the embankment.

TABLE 18 - FLOOD ROUTING WITH AND WITHOUT FUSE PLUG (1977)

PROBABLE MAXIMUM FLOOD				
No. of Spillway Crest Gates Operating	**No Fuse Plug Concrete Crest Wall 700 High (Top EL 79.7)**		**With Right Bank Fuse Plug + and Concrete Crest Wall 700 High (Top EL 79.7)**	
	Maximum Water Level EL (m)	**Maximum Outflow (m^3/s)**	**Maximum Water Level EL (m)**	**Maximum Outflow (m^3/s)**
5	76.8 (Normal operating rule)	11 700	76.8	11 700
4	78.2 (Normal operating rule)	10 600	78.2	10 600
3	78.03*	8 281	78.03*	8 281
2	79.64*	7 210	79.29* (Fuse Plug has blown)	9 216
1	80.66* (0.96m over crest wall)	8 847 (Bank overflow = 3292)	80.23* (0.53m over crest wall)	9 570 (Bank overflow = 1367)
Nil	81.29 (1.59 m over crest wall)	10 546 (Bank overflow = 7017)	80.87 (1.17m over crest wall)	11 200 (Bank overflow = 4484)
* When 3 gates or less are operating, the normal operating rule is replaced by a rule which forces the operator to open the gates to match outflow to inflow until the gates are fully open.				
<u>1:1000 FLOOD</u>				
Nil	79.8 (0.1m over crest wall)	2 752 (Bank overflow = 111)	79.35	4 640
<u>1:500 FLOOD</u>				
Nil	79.08	2 202	79.04	4 270

+Fuse Plug washes out when WL reaches EL 79.

Note: The normal operating rule for the PMF was that described in Section 14.4.2.

11.6 Embankment Alignment

The alignment of the original 'C' axis was adjusted for the reasons stated in Section 4.2. Figure 2 (Drawing A1-50820B) shows the adopted axis and the adjacent topography. The deep gully located near the eastern end of the left abutment could not be avoided. The gully contained up to 8m of alluvium. The quantity of excavation required for a satisfactory foundation for the Zone 1A material and the quantity of expensive Zone 1A fill material required were minimised by the adoption of a sloping core section on the left embankment. The adjusted alignment permitted the Zone 1A core to be founded on a satisfactory sandstone foundation at close to the highest point in the gully as shown in Figure 22 (Drawing A-49868B).

The positioning of the diversion channel in relation to the dam centreline meant that no weathered shale would occur in the dam foundation to the right of the channel. The left abutment of the diversion channel consisted of a large block of sandstone sitting on weathered shale. On closure this block was excavated and used as fill for the shell. The shale was entirely removed .

Moving of the dam axis downstream would have increased the fill volume of the dam. Moving of the axis upstream would have caused portions of the embankment on the right bank to be founded on the weathered shale (highly plastic clay).

11.7 Embankment Types and Sections

The design of the embankments was formulated in conjunction with the planning for the staged construction. The final design depended on information obtained during construction.

Two embankment types were incorporated in the final design. The main embankment to the right of the spillway and across the river valley consists of a rockfill section with a central core. The embankment to the left of the spillway consists of a miscellaneous rolled fill zone with an upstream sloping core. Typical sections are shown in Figure 22 (Drawing A1-49868B).

The preliminary designs assumed that a central core type section would be used throughout the dam. However, it was estimated that foundation excavation would produce 950 000 cubic metres of material which could be used as inexpensive fill and the left embankment was then designed as a sloping core dam using this miscellaneous fill. The fill volume of the left embankment increased but the overall cost decreased as shown in Table 19.

TABLE 19 - LEFT EMBANKMENT COST COMPARISON - NOVEMBER 1979.

Embankment Type	Volume m^3	Cost
Central Core	641 412	$1 653 867
Sloping Core	724 000	$1 077 207

(Distance 75m to 1100m based on tendered rates)

The section geometry of the main embankment and the sloping core embankment are constant throughout their lengths, except that the sloping core was steepened as it approached the spillway retaining wall so that it contacted the retaining wall in a vertical position as shown in Figures 36 and 37 (Drawings A1-56111E and A1-56110E). This was designed to avoid the sloping core "hanging up" at its point of contact with the retaining wall. Other adjustments to embankment cross sections were made on both sides of the spillway to provide extended surface areas at crest level for works areas. Typical details are shown in Figures 36 and 37.

The filters and core cutoff are founded on rock. The shell zones of the main embankment and the miscellaneous fill zone of the left embankment are founded on varying thicknesses of overburden. The design parameters adopted for the zones are shown in Table 20.

TABLE 20 - EMBANKMENT MATERIALS PROPERTIES.

ZONE	DESCRIPTION	MATERIAL	DENSITY (kg/m^3)		PROPERTIES	
			Dry	Sat.	ϕ'	c'
1A	core	SC,CL	1726	2094	27.0	17.0
1D	misc. fill	sandstone,SC,	2100	N/A	34.8	10.0
2A	filter	CL	1800	2260	35.0	0.0
2B	drain	SW,SP	1800	2260	35.0	0.0
2C	blocking	GP	1800	2260	30.0	0.0
2D	drain	SW	1800	2260	30.0	0.0
3A	shell fill	SM-SP	1830	2080	41.0	0.0
3B,3E	shell fill	GW/GP	2030	2178	37.0	11.6
3C	coffer dam	sandstone	2030	2178	37.0	0.0
4A	rip rap	sandstone	1860	2100	40.0	0.0
4B	face rock	"greenstone"	1830		41.0	0.0
5	disposal	"greenstone" spoil	ignored			

These material properties were adopted after examining the results of tests conducted on samples of the various materials.

11.8 Embankment Foundations

11.8.1 Embankment Foundations - General

The foundation for the core material consisted of sound sandstone which was excavated to obtain a gradually varying profile especially through the stepped sandstone adjacent to the river. The shoulders of shallow subsurface erosion gullies in the sandstone were excavated to remove abrupt changes in the longitudinal profile of the core trench. Areas of the foundation in the form of large elongated blocks with gaps between them which could form possible seepage paths were removed. Drainage of the core trench was arranged to avoid contamination of the gravel underdrains.

The soils beneath the shells varied in depth, strata thickness and material type. Drilling revealed the presence of some low strength, medium plasticity clays in the vicinity of the deep alluvium. The elevation of these clays determined the depth of excavation of the alluvium for the foundation of the shells. On excavation of the core trench, the soil profile of the cross river alluvium was exposed, showing that the suspect soils occurred as fluvial deposits in old erosion gullies and were not typical of the alluvium.

When exposed by excavation, undisturbed samples of every soil type were taken from the batters of the cutoff trench and tested. The results of these tests determined if the soils would be left in place or excavated. Typical properties for the soils left in place beneath the shells are shown in Table 21.

TABLE 21 - FOUNDATION MATERIALS PROPERTIES.

MATERIAL	DESCRIPTION	TYPICAL PROPERTIES	
		ϕ'	c'
slope wash/sandy	(SC)	32.0	0.0
clays	(CH-CL)	23.0	0.0
residual/clays	(ML)	27.0	0.0
alluvium/silty clays	(CL)	27.0	0.0
alluvium/clays	(SP/SW)	35.0	0.0
alluvium/sands	(SM)	33.0	0.0
alluvium/silty sands	(ML)	33.0	0.0
alluvium/silts	(GW/GP)	33.0	0.0
alluvium/gravels			

Excluding the core contact area, the final design attempted to minimise foundation excavation by leaving as much of the foundation overburden in place as possible. Material investigations into the foundation soils and the fill materials indicated that most of the foundation materials were only marginally weaker than the proposed fill. This meant that the embankment batters would not be affected greatly by leaving the natural soils in place beneath the shells. The additional cost of removing all overburden from the foundation is shown in Table 22.

TABLE 22 - IMPACT OF TOTAL FOUNDATION STRIPPING - NOVEMBER 1979.

ITEM	VOLUME m^3	COST
Foundation Excavation	984 275	$1 122 073
Embankment Construction	721 735	$1 303 840
Total Additional Cost		$2 425 913

The decision to minimise foundation excavation was not without problems in that:

(i) The material left in place would be naturally consolidated and could undergo further settlement under the load of the embankment;

(ii) The embankment design could not be finalised until a thorough appraisal of the foundation soils had been made. This was only possible after excavation of the core trench.

The shell foundation material between embankment Distance 1250 m and 2500 m on which the miscellaneous fill zones were placed, consisted of sandstone, gravel, and other suitable alluvium having sufficiently high strength to avoid embankment stability and settlement problems. Soil testing indicated that SC material or cohesionless soils that had a friction angle in excess of 27° did not need to be removed. Undesirable soil types in the cross river alluvium were silt and high plasticity clays (CH), and the foundation was excavated to below this level.

No special treatment was required for the foundations for the miscellaneous fill zones between Distance 0 m and 1150 m, except that the root containing silty material overlying the material of SC classification had to be removed.

In general, silt deposits followed the line of the river banks, and consisted of unconsolidated, cohesionless, fine grained fluvial deposits overlying sand and gravel deposits.

Because of the unconsolidated nature and the thickness of the strata, the presence of silt beneath the shell of the embankment could result in large uneven consolidation movements, and the silt layers were therefore removed.

Clean silt material was stockpiled for use as a blocking filter for the embankment. Silt material contaminated with clay lenses and binder or excessive amounts of sand, was used as fill for the miscellaneous fill zones.

Clay bound soils were found between the silt lenses on the river bank and the steeply rising sandstone at about Distance 1550 m. The soils classified as SC and CL in this area gave acceptable strength results to justify their use as foundation material for the shell, and as fill for the miscellaneous fill embankment. Low strength CH clays had to be removed, but it was not necessary to finish the surface to a plane foundation beneath the shell zones.

Sand and gravel deposits were suitable for foundation material provided they did not contain excessive amounts of silt. Tests were carried out on materials left in situ as mentioned in Section 11.14.

11.8.2 Embankment Foundation Treatment

After removal of loose and shattered material from the foundations of Zones 1A and 2, the foundations had to be cleaned with jets of air and water under high pressure.

Foundation treatment with either slush grout or mortar was required to fill surface irregularities, open fractures and fissures in the rock, which were minor or where the foundation was generally planar. These areas were treated to provide a suitable surface for the placement of the fill material.

Areas of the foundation which were likely to weather rapidly were mortar treated immediately after clean-up. All areas of the foundation beneath Zones 1A and 2 were treated immediately before placing the fill, which was placed while the slush grout and mortar were still plastic.

Dental concrete was used on the foundation beneath Zones 1A and 2 wherever fill material could not be satisfactorily compacted. A regular surface was produced by the dental concrete over cavities and abrupt vertical faces in the foundation rock.

11.8.3 Geological Mapping of Foundations

Prior to placement of fill materials on the embankment foundations, the foundations were geologically mapped. Figure 38 (Drawing A1-63917) lists the individual drawings covering the geological mapping of the foundations. None of the drawings shown on this key plan have been included in this report, but have been supplied to the South East Queensland Water board under separate cover.

11.9 Embankment Stability

The stability of the embankment slopes was evaluated using limit equilibrium analysis on potential slip surfaces. The Simplified Bishops Method of slices was used for circular slip surfaces while more generalised methods, developed in house and similar to the Morgenstern - Price method, were used for non-circular surfaces.

Most of the design was carried out using circular slip surfaces. The generalised procedures were used where potentially weak bedding features, such as the clay filled joints of the right bank and clay layers in the overburden soils, might have influenced the shape of the critical surface.

The analysis of the upstream slope of the main embankment for drawdown showed that non-free draining sandstone fill could not be used in the drawdown affected region. It was replaced by river run gravel of appreciably higher strength. This material was well graded and contained layers of processed gravels (GP) to ensure effective drainage.

The clay filled joints of the right bank appeared to dip about 9 degrees downstream. Despite drilling investigations, their continuity was not established. To guard against uplift forces being concentrated beneath these layers, pressure relief drains were drilled beneath the downstream shell in this area.

The pore pressure grid used for the central core embankment is shown in Table 23, and the grid for the sloping core embankment is shown in Table 24.

TABLE 23
PORE PRESSURE GRID FOR CENTRAL CORE EMBANKMENT

DISTANCE										
EL	**UPSTREAM**						**CENTRE LINE**	**DOWNSTREAM**		
	-183.0	-13.6	-11.0	-8.0	-4.4	-2.0	0.0	2.0	4.4	8.0
79.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
69.00	0.00	0.00	0.00	0.00	0.00	-0.30	-0.60	-0.50	-0.50	0.00
66.00	3.00	3.00	3.00	3.00	3.00	1.90	1.10	0.30	-0.01	-10.00
64.00	5.00	5.00	5.00	5.00	4.80	3.30	2.30	1.30	0.20	-3.00
62.00	7.00	7.00	7.00	7.00	6.40	4.70	3.40	2.10	0.50	-1.80
58.50	10.50	10.50	10.50	10.50	9.10	7.00	5.20	3.50	1.40	-1.80
55.00	14.00	14.00	14.00	14.00	11.60	9.10	7.00	4.90	2.40	-1.50
51.50	17.50	17.50	17.50	17.50	14.00	11.10	8.80	6.40	3.50	-1.00
48.00	21.00	21.000	21.00	21.00	16.30	13.20	10.60	8.00	4.80	0.00
44.50	24.50	24.50	24.50	23.50	18.50	15.20	12.40	9.60	6.20	1.00
41.00	28.00	28.00	28.00	26.00	20.80	17.30	14.30	11.40	7.70	2.20
36.90	32.10	32.10	32.10	28.80	23.40	19.80	16.70	13.60	9.70	3.80
32.80	36.20	36.20	36.20	31.70	26.20	22.50	19.30	16.00	12.00	5.60
26.40	42.60	42.60	40.80	36.50	31.00	27.30	24.00	20.60	16.40	9.60
20.00	49.00	49.00	46.10	42.10	36.80	33.10	29.80	26.50	22.30	15.90
0.00	69.00	69.00	66.10	62.10	56.80	53.10	49.80	46.50	42.30	35.90

TABLE 24

PORE PRESSURE GRID FOR SLOPING CORE EMBANKMENT.

DISTANCE							
EL	UPSTREAM			CENTRE LINE	DOWNSTREAM		
	-155.0	-45.0	-37.0		60.0	100.0	155.0
79.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
69.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
59.50	9.50	9.50	0.00	0.00	0.00	0.00	0.00
56.80	12.20	12.20	2.70	0.00	0.00	0.00	0.00
54.00	15.00	15.00	5.50	2.80	0.00	0.00	0.00
52.30	16.70	16.70	7.20	4.50	0.00	1.70	1.70
30.00	39.00	39.00	29.50	26.80	22.30	24.00	24.00

11.10 Embankment Stability Analysis Results

Results of the stability analysis for the left bank embankment (axis Distance 100m to 1016m) are shown in Table 25, and the failure surface locations are shown in Figure 39 (Drawing A4 -109976).

TABLE 25 - LEFT BANK EMBANKMENT STABILITY ANALYSES RESULTS.

CASE	REQUIRED MIN. FACTOR OF SAFETY	COMPUTED FACTOR OF SAFETY
Steady Seepage U/S Failure	1.5	2.7
Steady Seepage D/S Failure	1.5	1.5
Rapid Drawdown U/S Failure	1.25	1.4

The embankment section analysed was at Distance 550m. For the analysis a possible clay seam with an angle of internal friction Φ' equal to 22° and with no cohesion, was postulated in the foundation alluvium.

Results of the cross-river embankment stability analysis (axis Distance 1223m to 2450m) are shown in Table 26, and the failure surface locations are shown in Figures 40 to 44 (Drawings A4-109971 to A4-109975).

The embankment sections analysed were at Distances 1400m, 1600m, 1700m, and 1800m, and the foundation materials as determined from the bore logs at each section were included in the analysis.

TABLE 26 - CROSS-RIVER EMBANKMENT STABILITY ANALYSES RESULTS.

DISTANCE	CASE	REQUIRED MIN. FACTOR OF SAFETY	COMPUTED FACTOR OF SAFETY
1400	Rapid Drawdown U/S Failure	1.25	1.26
1600	Steady Seepage D/S Failure	1.5	1.58
1700	Steady Seepage D/S Failure	1.5	1.64
1700	End of Construction D/S Failure	1.25	1.41
1800	Steady Seepage D/S Failure	1.5	1.72
1800	End of Construction D/S Failure	1.25	1.48
1800	End of Construction and Earthquake Loading D/S Failure	1.0	1.23
1800	End of Construction U/S Failure	1.25	2.27
1800	End of Construction and Earthquake Loading U/S Failure	1.0	1.10
1800	End of Construction and Earthquake Loading D/S Failure (Along weak clay seam)	1.0	1.02

11.11 Testing During Construction

Quality control and record testing were also carried out during construction to ensure that materials not complying with the specification were not placed in the embankment.

A comparison between the investigation tests' parameters and those determined from record testing, for some of the embankment zones is shown in Table 27.

TABLE 27 - COMPARISON OF INVESTIGATION TESTS AND RECORD TESTING.

ZONE	TEST	INVESTIGATION TESTS			RECORD TESTING			
		ϕ' Deg.	c' kPa	Dry Density kg/m^3	ϕ' Deg.	c' kPa	Placement Density Mean kg/m^3	Placement Density Std. Deviation
1A	Triaxial	27.0	17.0	1726	33.5	11.8	1751	54
1D	Triaxial	34.8	10.0	2100	34.8	34.8	1865	93
3A	Shear	41.0	0.0	1830	40.0	12.5	2035	46
3B	Triaxial	37.0	11.6	2030	38.5	14.0	1942	78
3C	Triaxial	37.0	17.0	2030	38.5	14.0	1942	78

The p-q plot of record triaxial test results, the Mohr circle envelope, and the design envelope line for Zone 1A and Zone 1D, are shown in Figure 45 and Figure 46.

Sandstone p-q plots for the preliminary investigations (Figure 47), the record testing for 3B and 3C for Stages I and II (Figure 48), and the miscellaneous fill (Figure 49), were combined to determine the record testing Mohr circle envelope for Zones 3B and 3C as shown in Figure 50. The design envelope line is also shown in Figure 50.

The points plotted in Figures 45 to 50 are the top of the Mohr Circles given by:

$$p = \frac{\sigma_1^1 + \sigma_3^1}{2}$$

$$q = \frac{\sigma_1^1 - \sigma_3^1}{2}$$

The equation of the line from the regression analysis has a slope of "$\propto$" and an intercept on the q axis of "a".

The Mohr circle envelope is determined from:

$$\phi^1 = \sin^{-1}(\mathrm{Tan} \propto)$$
$$c^1 = a/\cos \phi^1$$

11.12 Seepage Control

The water retaining barrier is formed by the core and a grout curtain extending up to 35m into the foundation.

Seepage through the core will be captured by chimney drains placed against the downstream filters behind the core. This water is removed through a blanket drain beneath the cross-river embankment and through discrete gravel drains beneath the miscellaneous fill of the left embankment. The discrete gravel drains were used to minimise the volume of uniform gravel required.

Most of the seepage in the foundation rock will occur through joints. An extensive grouting program, including pressure testing, was designed to create an impermeable curtain by filling these joints.

The slush grout and mortar, spread over the rock surface immediately before placing the initial layer of core material, served to stabilise the core in contact with the foundation rock. Some shallow sub-parallel joints could otherwise have led to erosion of the core material bypassing the filters.

11.13 Settlement Provisions

During construction and afterwards various settlement processes are expected to occur. The design incorporates provisions aimed at negating any detrimental effects.

It is possible that sections of the core may have transient low stress states during the consolidation period of the dam. This could be caused by differential settlement where construction stages meet, or by the core "hanging" on the more rigid adjacent zones or on the foundation rock at the points of grade change.

Low stress states caused by the arching of the core over transverse steps, joints and channels should not occur. The smaller steps and joints have been backfilled with concrete to the general foundation profile. The larger features such as the diversion channel were backfilled with effectively non-consolidating fill of heavily compacted sandy clay.

As a general protection against hydraulic fracturing, a blocking filter, Zone 2E, was installed. It consists of an erodible zone of silty sands which cannot pass through the downstream filters. In the event of the hydraulic fracture of the core, this material will wash into any pipes formed, plug them and restrict seepage to a safe level.

The chimney zones surrounding the core of the main embankment are non-consolidating relative to the core and the shell and its foundation. They isolate the settlement processes of the core from those of the shell. Settlement on wetting was minimised by placing the shell materials as wet as practicable.

The core of the left embankment lies against the miscellaneous fill zone which sits on unconsolidated soils of slope wash origin. Construction of the miscellaneous fill zone some two years prior to the core should have consolidated the foundation soils such that the core will not be adversely affected by any additional settlement.

11.14 Earthquake Immunity

In part the seismic immunity of Wivenhoe Dam relies upon the absence of materials within the dam foundation which may undergo earthquake-induced strength reductions by liquefaction. The upper strata of the point bar alluvium deposit at the site contained beds of hydraulically deposited fine sands and silts. Most were removed for construction of the miscellaneous fill zone of the left embankment. Special testing was carried out on materials left in situ to ensure that they were not susceptible to collapse under shock loading (seismic loading).

A pseudo-static method of analysis was not carried out on the embankment.

An analysis was performed assuming liquefaction of the alluvial foundation materials after an earthquake. The failure surfaces for the analysis are shown in Figures 43 and 44, and the factors of safety were greater than 1.

11.15 Saddle Dams

Saddle Dam No. 1 and Saddle Dam No. 2 , located on the eastern side of the main dam wall, are required to contain flood surcharge in the storage.

The saddle dams consist of miscellaneous fill embankments with riprap on the upstream side. Details of the saddle dams are shown in Figures 51 and 52 (Drawings A1-53346B and A1-53347B).

11.16 Instrumentation

Flood discharges from Wivenhoe Dam have the potential to flood large densely populated areas. It is therefore important that the embankment be monitored for behaviours which are associated with decreased dam integrity. Either slip failures or hydraulic fracturing can initiate a breach of the embankment.

Most of the instrumentation has been concentrated in two sections of the main embankment namely one founded on the deep compressible alluvium and one on the shallow non-compressible gravels of the river bed.

Forty eight surface settlement points and four inclinometers were installed to monitor displacement which is the main parameter used in detecting slip movements in the embankment. The fifty one piezometers provide pore pressure information from which background displacements due to consolidation can be assessed.

In conjunction with twenty five load cells, the piezometers provide information on the stress state within the core which is important in assessing its immunity against hydraulic fracture. Special installations have been provided in areas in which low stress states can develop, such as the diversion channel base.

An extensometer has been placed at a steeply dipping upstream-downstream shear zone located in the foundation rock of the original river bed. Movement across this zone, indicating relative displacement between the sandstone blocks could rupture the grout curtain, and possibly the adjacent core.

Further information on dam surveillance is given in Section 28.

11.17 Embankment Foundation Grouting

11.17.1 Embankment Foundation Grouting - General

The foundations under the sloping core of the left embankment and under the central core of the main embankment were grouted to reduce seepage through the foundation. The arrangement of the single line curtain up to 35m deep, supplemented by blanket holes 8m deep, is shown in Figure 53 (Drawing A1-45411D). Full details of grouting including grout takes, water losses, uplifts, etc, are shown in the series of Drawings from A1-57686 to A1-57707 and from A1-79265 to A1-79269. These drawings have been supplied to the South East Queensland Water Board under separate cover. The key plan for the foundation grouting Drawings is Figure 54 (Drawing A1-57686).

11.17.2 Water Pressure Testing of Foundations

Water pressure testing carried out during the drilling of the site to investigate the foundation geology, was used as a guide to determine possible grout takes (References 14 and 16). Results of the water testing indicated that water losses were generally low at depth. High water losses appear to coincide with poorly consolidated sandstone, which is a primary feature and is not the result of weathering. High water loss layers were of limited extent laterally, with the highest losses occurring in the river channel.

11.17.3 Grouting Requirements

Plain cement grout (no sand) was specified for grouting the foundations, and all work was required to be performed in the presence of the Supervisor.

Grouting was not permitted to commence in an area until all blasting had been completed within a distance of 60 m, and grouting through concrete was not permitted to commence until the concrete had been in place for a period of three days.

Drilling methods were limited to wet percussion drilling and diamond drilling. "Rod-dope" and grease on drill rods were not permitted.

During pressure testing and grouting of the foundations the following aspects were carefully monitored to give early warning of rock movements due to the grouting operations:

- Drop in pressure at standpipe or packer,
- Increase in rate of water loss or grout take.

In areas of soft rock where standpipes could not be embedded in the rock, a concrete grout cap was placed in a trench excavated in the rock, and the standpipes embedded therein.

11.17.4 Materials and Equipment

Fresh portland cement Type A or Type C and clean water were used to make the grout.

The minimum diameter of the wet percussion drill holes was 32 mm diameter, and the maximum diameter of the diamond drill holes was a commercial standard EX size bit.

After drilling, holes were washed with radial jets of air and clean water under pressure, using a wash out bit.

Standpipes used were black steel pipe threaded at the upper end to suit drilling and grouting equipment, embedded 600 mm in concrete and projecting 150 mm above the surface. After grouting the holes, the pipes were backfilled and burned off flush with the surface.

All grout was mixed in high speed mixers operating at a minimum mixing speed of 1,000 revolutions per minute. After mixing, the grout was discharged to agitators where a pump circulated the grout through the line past the hole and back to the agitator.

The grout pumps used were of the "Mono" type, and the maximum internal diameter of the circulation lines was 25 mm.

11.17.5 Water Testing of Grout Holes

Before grouting commenced, each stage of a hole was water tested by measuring the quantity of clean water taken over time at a pressure of 100 kPa applied at the surface.

The test was carried out over a 15 minute period, and the quantity of water injected was converted to lugeon units. One lugeon unit is defined as a water take of 1 litre per metre of hole per minute at a pressure of 100 kPa.

11.17.6 Grout Curtain

Generally the depth of the grout curtain is equal to the storage depth, and ranges from a maximum of 35 m to a minimum of 15 m at the abutments.

The spacing of the initial holes drilled, the primary holes, is 12 m. Subsequent sequences of holes, designated secondary, tertiary and quarternary etc, halves the distance between the previous sequences. Holes were drilled and grouted in the order of the designated closure sequence.

Initially, primary and secondary holes were drilled the full depth of the curtain. The need for tertiary and higher sequence holes depended on the water test results. Each hole was treated in stages - up to 10 m in the top stage and greater if necessary in deeper stages.

Tertiary holes were required each side of a secondary hole to the depths indicated on the Drawings, when the 0-10 m stage of a secondary hole had a take in excess of 1 lugeon and 3 lugeons in the deeper stages.

Quarternary holes were also required where the top stage of a tertiary hole had a take in excess of 1 lugeon and 3 lugeons in the deeper stage.

Supplementary holes to stage 1 depth, drilled and grouted upstream and downstream of the curtain, were required when any section of the curtain had been drilled to the tertiary sequence, and four or more adjacent holes had a water take in excess of 3 lugeons.

After a significant percentage of the curtain was completed, the results were reviewed and the criteria for drilling additional holes were modified as follows, with the view to making significant savings in cost without reducing the efficiency of the curtain:-

- Tertiary holes were to be drilled in the 0-10 m stage, except when the water take in the deeper stages of a secondary hole exceeded 5 lugeons, tertiary holes were to be carried down each side to the full depth of the curtain;

- Any tertiary hole having a water take in excess of 3 lugeons in the 0-10 m stage and in excess of 8 lugeons in the deeper stages, was to have quarternary holes drilled each side for the full depth of the curtain;

- If a stage of any quarternary hole had a water take in excess of 10 lugeons, quinary holes would be drilled each side, if directed by the Supervisor;

- Where the foundation level was below EL 40.0 and the water take in the upper stage of any hole exceeded 5 lugeons supplementary holes were to be drilled and grouted on the upstream side;

- Where the water take in the upper stage of any curtain hole exceeded 10 lugeons, supplementary blanket holes were to be drilled and grouted both upstream and downstream of the curtain;

- Where the foundation level was between EL 40.0 and EL 60.0, upstream supplementary blanket holes were to be drilled and grouted when the water take in the upper stage of any curtain hole exceeded 10 lugeons, and both upstream and downstream holes were to be drilled and grouted when the water take exceeded 15 lugeons in the upper stage of a curtain hole;

- Supplementary blanket holes were not required for foundation levels above EL 60.0.

Any section of the foundation having joints striking across the grout curtain where surface leaks had occurred during grouting had supplementary blanket holes drilled and grouted. For foundation levels below EL 40.0 blanket holes were provided both upstream and downstream. For foundation levels above EL 40.0, blanket holes were only provided on the upstream side, when directed by the Supervisor.

For details and results of grouting refer to Section 11.17.1.

11.17.7 Pressure Grouting

The method of grouting used was the "upstage method", whereby the hole was drilled to its ultimate depth before grouting was commenced. Packers were used for grouting each stage, commencing at the lowest stage and proceeding upwards.

In zones of badly fractured rock, the "downstage method" was used, whereby drilling of a stage did not commence until grouting of the upper stage had been completed.

The specification required that grout pressures measured at the surface were to be steadily increased to the following maximum pressures:-

Stage	Maximum Pressure
0-10 m	200 kPa
10-20 m	450 kPa
20-35 m	900 kPa
35m +	1000 kPa

However after commencement of grouting, it was realised that these pressures were excessive and curtain grouting was performed at the following pressures:-

Stage	Pressure
0-10 m	100 kPa
10-20 m	225 kPa
20-35 m	450 kPa

Blanket grouting was performed at a pressure of 50 kPa.

Grout proportions used were within the following limits (water : cement by volume):-

- Thinnest grout = 5:1
- Thickest grout = 0.5:1

Grouting commenced with the injection of the thinnest grout, and if grout takes exceeded a appreciable volume the mix was thickened. Grouting of the hole continued until refusal was reached.

12.0 SPILLWAY LOCATION

12.1 Factors Influencing Location

Although the dam was intended partly as a flood retention storage, the large flood discharges that occur in the Brisbane River made the provision of an overflow spillway of substantial capacity, essential. The selection of the spillway site and alignment was also important to the embankment design as the shell material for the embankment was to come from the spillway excavation and strength parameters for the shell could not be determined until its source was fixed. Other considerations such as diversion channel levels and approach channel velocities dictated that the spillway be founded on fresh sandstone.

12.2 Options Examined

12.2.1 Options Examined - General

The economics of locating the spillway on the right bank, within the cross river section, and on the left bank were examined. The large quantities of concrete required for the cross river spillway made it uneconomic.

Suitable sites existed on both banks of the Brisbane River at the damsite, and two preliminary designs on each of the left and right banks were considered in detail. These alternatives were:-

(a) On the left bank:-

  (i) Proposal 1. A high level spillway with diversion and outlet tunnels on the right bank as shown in Figure 4 (Drawing DB 1889).

  (ii) Proposal 5. A similar spillway founded at a lower level incorporating outlets in the left hand spillway wall, with a temporary diversion channel on the right bank as shown in Figure 6 (Drawing DB 1918).

(b) On the right bank:-

  (i) Proposal 2. A spillway with outlets in a separate monlith adjoining the spillway on the left hand side as shown in Figure 5 (Drawing DB 1888). Diversion would be through low spillway monoliths.

  (ii) Also known as Proposal 2, similar to (b) (i) above except that the outlet works were incorporated in the left hand spillway wall, as shown in Figure 55 (Drawing DB 1904).

12.2.2 Spillway in the River Channel

A concrete spillway situated in the cross-river section was examined. This scheme provided ample diversion capacity with good immunity against overtopping of the cofferdams and of the partially constructed embankment.

The spillway was connected to the embankment on either side by rockfill "wrap-around" sections embracing non spillway concrete monoliths. It was found that although the scheme saved a substantial amount of spillway excavation, a saving that was partly offset by the fact that the fill for the embankment shells had to be won elsewhere, it had over 100,000 m^3 of concrete more than the cheapest scheme, costing an extra $5 million and it was therefore not considered further.

12.2.3 Analysis of Right Bank Spillway Proposals

As outlined in Section 9.2.4, diamond drill holes explored both the right and left bank spillway sites, and whilst sound sandstone existed at both sites the right bank foundation was inferior to that of the left bank. A thick shale layer dipping about 5° downstream was located on the right bank site. The seam varied in thickness from 3 m to 9 m and was unsuitable for the foundation of concrete structures from a point of view of sliding stability. Prestressing could have overcome the problem, but this solution was nearly as expensive as founding the spillway below the shale layer. Below the shale layer the rock was inferior to that on the left bank site, weak sandstone layers and clay seams being present. Excavation to solid rock would have increased concrete quantities, thus adding to the cost.

The right bank spillway schemes had the most advantageous diversion capabilities, and gave great immunity against overtopping.

The quantity of excavation from a spillway founded on the right bank was not sufficient for the fill required in the shells of the dam embankment, and extra material would be required from other borrow areas.

12.2.4 Analysis of Left Bank Spillway Proposals

Diamond drilling confirmed that the foundations for the left bank schemes were superior to those on the right bank. As described in Section 4.3, it was proposed to found the spillway in Proposal 1 at a higher level (EL 48.0) than in Proposal 5. As some clay seams and weak layers existed at this higher level, Proposal 1 would have required prestressed anchors to prevent sliding. In assessing the amount of prestressing required, assuming a factor of safety against sliding of 4 and that the clay seams were continuous, the following shear strength parameters were adopted:-

$\varnothing^1 = 23°$ $\qquad C^1 = 0.$

In Proposal 5 the spillway was founded at a lower level (EL 36.0), below the level of the clay seams. The rock at this lower level has the following strength parameters:-

$\varnothing^1 = 40°$ $\qquad C^1 = 689$ kPa.

Using these values to estimate the shear-friction factor of safety against sliding, it was determined that no special measure would be required to ensure stability.

Another advantage of Proposal 5 was the greater immunity against overtopping in the later more critical stages of construction, because of the lower level diversion through the spillway as compared to Proposal 1. This particularly applied during the closure period of the diversion channel.

12.3 Advantages of the Selected Spillway Location

The finally adopted arrangement of the spillway was Proposal 5 - i.e. a spillway on the left bank, without the need of stabilizing anchors, and incorporating all outlets required for the dam (including those required for a possible future hydro-electric power station) in the left hand wall of the spillway.

This arrangement has the following advantages:-

(i) The left bank provided a fresher rock foundation;
(ii) The spillway was founded on more competent rock, thus avoiding the use of prestressing anchors;
(iii) It had comparable diversion capacity to that of other proposals and better provision for river closure;
(iv) Compared to Proposal 1 it required shorter conduits for the proposed power station;
(v) Installation of all outlets for the dam in one location provided cost savings in providing penstock gates etc;
(vi) It had a slight cost advantage over other proposals.

13.0 CONSTRUCTION MATERIALS FOR THE SPILLWAY

13.1 Construction Materials for the Spillway - General

This section provides details of investigations of borrow areas for fine and coarse aggregates, and details of specified requirements for cement, fly ash and concrete used in the spillway and associated structures. Further details may be found in References 15 and 17.

13.2 Aggregates

13.2.1 Aggregates - General

Fine and coarse aggregates were required for the production of mass concrete in the spillway overflow section, the retaining and training walls of the spillway and for structural concrete required for the outlet works, bridges, etc.

13.2.2 Sources of Aggregates Investigated

The following sources were investigated for sources of fine and coarse aggregates.

Borrow Area WX14 - Atkinson's Crossing
Borrow Area WX15 - Roseborough Gravel/Sand deposit
Borrow Area WX16 - Hay's Crossing
Borrow Area WX18 - Logan Creek Sand deposit
Borrow Area WX20 - Macfarlanes Bridge Sand/Gravel deposit

Investigations carried out on Borrow Areas WX14, WX15 and WX16 are described in Sections 10.3 and 10.4 above.
Borrow Area WX18, Logan Creek Sands, consisted of the bed and low level alluvial terraces of Logan Creek, a tributary of the Brisbane River, approximately 11 km north west of the dam site.
Borrow Area WX20, Macfarlanes Bridge Sand/Gravel deposit, consisted of high, intermediate, and low level terraces on the right bank of the Brisbane River approximately 20 km by road north of the dam site.

13.2.3 Borrow Area WX18 Investigations

Borrow Area WX18, Logan Creek Sand Deposit, extended upstream on Logan Creek for a distance of 6 km from its junction with the Brisbane River.

The feature was investigated by thirteen hand auger holes, forty one driven tube samples, and seventeen backhoe trenches. The hand auger holes varied in depth from 0.8 m to 2.7 m; the driven tube samples varied in depth generally from 0.4 m to 3.0 m and the backhoe trenches from 0.4 m to 3.7 m.

The material encountered in the investigations was classified as a poorly graded sand.

13.2.4 Borrow Area WX20 Investigations

Borrow Area WX20, Macfarlanes Bridge Sand/Gravel Deposit, extended downstream from Macfarlanes Bridge for approximately 1.4 km.

The feature was investigated by twenty four hand auger holes and thirty eight bulldozer trenches. The hand auger holes varied in depth from 1.3 m to 7.5 m, and the dozer trenches varied in depth from 1.5 m to 7.0 m.

The bulk of the material encountered in the investigations was classified as poorly graded sands and gravels.

Borrow Area WX20 was approved as a source of fine sand for blending with concrete aggregate to improve concrete workability.

13.2.5 Tests on Concrete Aggregates

Samples of material from Borrow Areas WX14, WX15, WX16, WX18 and WX20 were subject to sieve analysis and to sodium sulphate soundness, bulk density, absorption and Los Angeles Abrasion tests. Results of these tests are given in Tables 8 to 12 of Reference 17. The location of, and logs of drilling and excavations in these areas, are shown in Figures 12 to 23 of Reference 17.

Samples from WX14, WX15 and WX16 were also subjected to Potential Alkali Reactivity tests, which proved the aggregates to be innocuous.

13.3 Cement and Fly Ash

The following cements were available for use in the manufacture of concrete:-

(a) Type C cement (low heat) supplied by special arrangement with Queensland Cement and Lime Co;
(b) Type A cement to be blended on site with various percentages of fly ash;
(c) Type A cement with 25% fly ash replacement supplied as a factory blend;
(d) Type A cement with a specified percentage fly ash replacement supplied as a factory blend by special arrangement with Queensland Cement and Lime Co.

For economy, and for performance both in heat evolution and durability, it was decided to use a fly ash blend. Tests for heat of hydration and strength were conducted on various blends to determine the lower and upper percentage limits of fly ash.

Cement specified for the spillway and associated structures was a blended cement Type FA in accordance with AS 1317 - Blended Cement.

In order to control heat evolution it was specified that the replacement of ordinary portland cement was to be a practical maximum for concrete in mass concrete structures. In concrete sections where the least dimension was greater than 4 m, the proportion of fly ash in the blended cement was to be at least 20 percent. The absolute maximum proportion of fly ash in blended cement was specified as 40 percent in the heart of mass concrete and 35 percent otherwise. The use of fly ash also eliminated the need for fine sand in most concrete mixes. This was also beneficial as fine sand was in short supply.

No minimum or maximum cement content was specified. However cement content determined the maximum placing temperature of concrete as shown in Section 13.7.

Type A cement to AS 1315-1973, supplied by Queensland Cement and Lime Co of Darra, and fly ash from Swanbank power station were used in the concrete. The cement, fly ash and admixtures were weighed separately and added to the batch.

13.4 Admixtures

Air entraining agent and water reducing set retarding agent were specified for use in the concrete.

The percentage of entrained air, determined in accordance with Part 4 of AS 1012, was specified as in Table 28.

TABLE 28 - SPECIFIED PERCENTAGE OF ENTRAINED AIR.

Coarse aggregate (Maximum size)	Total Air (Percentage by volume of concrete)
20 mm	4 ± 1.0
40 mm	3 ± 0.75
75 mm	2 ± 0.50

The trade name of the air entraining agent used was "NVR".

The amount of water reducing set retarding agent was not specified. The trade name of the agent used was "L78".

The target amount of entrained air for the mass concrete was 2%. The reduction to this figure below previously used values was intended to reflect the greater difficulty of entraining air in fly ash concrete and also the doubtful need to obtain higher quantities.

13.5 Reinforcement

Design was based on reinforcing bars conforming to AS 1302 - Steel Reinforcing Bars for Concrete and AS 1304 - Reinforcing Fabric, placed in accordance with the provisions of AS 1480-SAA Concrete Structures Code. Grades of bars specified were 230S, 230 R and 410C.

13.6 Concrete Trial Mixes

A concrete trial mix program was conducted between November 1977 and April 1978. Materials used in this program were:-

Aggregates from Borrow Area WX14, supplemented by fine sands from Borrow Area WX20; Darra ordinary Portland cement; three combinations of Portland cement with fly ash; and Embecon MB-VR air entraining agent.

Design criteria for the trial mixes were:-

(a) Maximum aggregate size - 37.5 mm
(b) Slump - 50 mm to 100 mm
(c) Air Entrainment Agent - 4% to 6%
(d) Water/Cement ratios - 0.45, 0.55, 0.65 and 0.75.

Mix details and the results of the trial mixes are given in Table 14 of Reference 17.

13.7 Specified Concrete Requirements

The Contractor was wholly responsible for the design of the mixes for production of concrete, but had to conform to specified requirements as follows:-

(a) Aggregate size

The specified maximum size of coarse aggregate in unreinforced mass concrete, and in walls, slabs and sections not less than 600 mm thick was 75 mm, provided that the minimum cover to any reinforcement was 75 mm and the minimum spacing of reinforcement was 200 mm. For smaller sections such as in the Plant Building the maximum aggregate size was specified as 20 mm. Otherwise the maximum aggregate size would be 40 mm or as shown on the Drawings.

(b) Slump

Slump, determined in accordance with AS 1012 Part 3, was not to exceed 75 mm.

(c) Strength

The nominated compressive strengths $F^{I}c$ for various sections of the works were as follows:-

(i) 15 MPa for the interior of mass sections;
(ii) 20 MPa for the 1.2 m zone from the face of all mass concrete and for general concrete sections;
(iii) 25 MPa for the gantry crane beams and cantilever in Monolith RH12, the cast in situ walkway on the road bridge, and the girders and deck slabs of the service bridge;
(iv) 30 MPa for the prestressed concrete deck units of the road bridge;
(v) 40 MPa for the anchorage areas for the prestressing tendons and for the radial gate trunnion corbels in the piers and in monoliths LH11 and RH12, and for the cantilevers off the spillway piers for the road bridge.

The design compressive strength was to be measured at 28 days except for the 15 MPa heart concrete (Grade 15H) which was to be measured at 90 days.

Specifying a 90 day strength for mass concrete was intended to give due recognition to high fly-ash mixes. Until a history of 90 day tests for the batch plant was built up, an attempt was made to predict 90 day strengths from 7 and 28 day tests. A formula for prediction based on trial mixes for variable fly ash contents was:-

$$F_{90} = F_{28}\left[1 + 0.842\,(1 + 3\,fa)\left(1 - \frac{F_7}{F_{28}}\right)\right]$$

where

F_{90}	=	expected strength at 90 days;
F_{28}	=	expected strength at 28 days
F_7	=	expected strength at 7 days
fa	=	fly ash fraction of cementitious material.

Concrete trial mixes designed for the 90 day strength specification used the maximum allowable proportion of fly ash but the water cement ratio adopted was lower than specified. Mass concrete trial mixes for facing purposes, of normal Grade 20, used less than the allowable fly ash. Typical batch quanitities for concrete with 75 mm maximum sized aggregate are shown in Table 29. Additives were used in both mixes.

TABLE 29
TRIAL MIXES - TYPICAL BATCH QUANTITIES - kg.

Aggregate Size (mm)	15 MPa 90 days	Grade 20
75	665	680
40	335	350
20	190	180
10	200	170
Coarse Sand	625	570
Fine Sand	Nil	Nil
Cement:-		
Type A	120	180
Fly Ash	80	70
W/C Ratio	0.68	0.55

Details of control, testing and mix details of concrete placed in the spillway are listed in the Post Construction Report - Reference 22.

(d) Water-Cement Ratio

The Water-Cement ratios specified were:-

(i) a maximum of 0.55 for Grade 20 concrete placed in the 1.2 m wide zone on the exposed faces of mass concrete sections;

(ii) a maximum of 0.75 for Grade 15H placed within the interior of mass sections.

(e) Temperature Control

To control temperature rises in placed concrete, lift heights were specified and excessive cement content was discouraged.

The maximum lift height allowed was 1.5 m and successive lifts were required to be placed at not less than 5 days and not greater than 10 day intervals.

While the maximum temperature at placing for all concrete was set at 27°C, the mass concrete was subject to even lower maximum allowable temperatures in cooler weather. This was done to match more nearly the diurnal temperature variations while the concrete, especially that with high fly ash content, had low tensile strength. As shown in Table 30 the maximum allowable temperature was varied from month to month down to 21°C in mid-winter when the average air temperature is 15°C.

TABLE 30 - MAXIMUM PLACING TEMPERATURES FOR CONCRETE

Month	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Temperature	27.0	27.0	26.0	25.0	23.0	21.5	21.0	22.0	23.5	25.0	26.5	27.0

The maximum allowable temperature at placing was further reduced by 1°C for each 10 kg/m^3 equivalent of Type A cement in excess of:-

(i) 220 kg/m^3 for concrete in the 1.2 m wide zone on the exposed faces of mass placements;

(ii) 160 kg/m^3 for concrete within the interior of mass placements.

For Type FA cement, the equivalent of Type A cement was taken as the mass of FA cement less 50% of the mass of the fly ash fraction.

These temperature control requirements necessitated the installation of a chilled water cooling plant on site. The plant had a capacity of 86 litres of water at 3°C per minute. The aggregates were sprinkled with water to cool them when necessary. A liquid nitrogen cooling system was also used to supplement these measures and resulted in a temperature drop of 3°C, thus enabling the contractor to place concrete on day shift in the 1981-82 summer within the specified constraints.

14.0 SPILLWAY DESIGN

14.1 Spillway - General

The general arrangement of the spillway is shown in Figure 56 (Drawing A1-50771C). The principal dimensions of the spillway and gates and relevant elevations are as follows:-

Number and Size of Radial Gates	5-12 m wide x 16.6 m high
Storage Capacity	1 150 000 ML
Flood Storage	1 450 000 ML *
Design Flood Maximum Outflow	11 700 m^3/s *
Total Volume of Spillway Excavation	1 962 766 m^3
Total Volume of Concrete in Spillway	124 984 m^3
Level of Fixed Concrete Crest	EL 57.0
Fully Supply Level for Optimum Operation of Wivenhoe Hydro-Electric Power Station	EL 67.0
Design Full Supply Level	EL 68.5
Maximum Water Level	EL 77.0 *
Embankment Crest Level	EL 79.0
Embankment Crest Level with Concrete Crash Barrier	EL 79.7

(*In 1993 the flood hydrology of Somerset Dam and Wivenhoe Dam was revised as part of the Brisbane River and Pine River Flood Study. The 48 hour Probable Maximum Flood produced the largest outflow from Wivenhoe Dam under existing normal gate operation procedures. Adoption of temporal patterns from Australian Rainfall and Runoff (1987), for events with a Average Recurrence Interval (ARI) of 100 years or less, led to the 72 hour duration storm producing the largest discharge from Wivenhoe Dam for more frequently occuring events. For these revised design flood estimates of peak inflows and outflows, flood volumes and peak lake levels for various return periods see Table 3.1 of Report 24 of Appendix B attached to this report. Levels in the table do not include allowances for wind set up or wave run up. These estimated extreme floods are of such a magnitude that they would cause overtopping of the embankment. The Imminent Failure Flood (IFF) has therefore been assessed as the flood event which, when routed through the storage under existing operational procedures, would just threaten to overtop the embankment. The estimated rainfall depth for the IFF is 75% of the Probable Maximum Precipitation which has an ARI of 14 300 years. For the IFF the peak inflow to Wivenhoe Dam is estimated to be 21 990 m^3/s; the peak outflow is estimated to be 14 080 m^3/s; and the flood volume is estimated to be 3 794 180 ML. However if the IFF is defined as the flood which will just reach the top of the dam wall including the wave wall it has an ARI of 100 000 years. For full details refer to Report 24 of Appendix B attached to this report.)

The excavated channel for the spillway has a total length of approximately 1100 m. It has a low level channel to serve the outlet works, which was also used for diversion during construction, and a higher approach channel serving the other four overflow monoliths. Downstream of the spillway flip, the discharge channel was excavated an additional 11 m where the spillway jet impinges, to form a plunge pool, which is designed to dissipate the energy and control scouring (Reference 23).

Bays on the spillway overflow crest are five in number, each 12 m wide. A high level spillway flip, of uniform radius directs an overflow jet well away from the crest structure. Steel radial gates of 86 t mass are mounted off piers supported on the crest, lifting winches being located behind each gate leaf. Each pier is a constant 3.5 m width for 12 m from its nose and then tapers to 2 m at its downstream end. At the base of each pier an extension is provided to reduce unsteady flow conditions in the spillway flip.

Twin bridges cross the spillway. One carries the highway and is supported on cantilevers off the piers. A second bridge supports a 79 t gantry crane provided to install the bulkhead gate during maintenance of any radial gate.

An intake structure for the outlet works is slotted into the left bank spillway retaining wall just upstream of the spillway crest. Outlet pipes of 1900 mm and 3600 mm diameter, one above the other, connect the intake to the discharge valves adjacent to the spillway flip. The 3600 mm dia pipe was providing as a possible power station penstock as described in Sections 22 and 29.

Training walls, upstream of the crest, direct flow fairly uniformly towards the spillway thereby maximising its performance. A combination of rockfill groynes and mass concrete walls constitute these training walls.

14.2 Geology of Site and Excavation of Spillway

14.2.1 Geology of Site and Excavation of Spillway - General

The damsite lies on the Helidon Sandstone, formerly known as the Wivenhoe Sandstone. This rock is a massive, thickly bedded, fine to coarse grained argillaceous sandstone of varying hardness, commonly showing current bedding. Bedding is approximately horizontal. Shale, claystone and coal are also present in occasional seams and lenses.

Possible spillway sites existed on both abutments but the existence of a thick shale layer, 4 m to 9 m thick, on the right bank and economic advantages favoured the left bank location.

At the spillway site weathering extended to depths of about 25 m, but it was only in the top two or three metres that the rock was completely to highly weathered so that the bulk of the excavated material was suitable for embankment fill in the outer zones of the dam. The moderately weathered zones were generally excavated by ripping with large bulldozers and loaded by scrapers assisted by bulldozers whereas the fresh rock was drilled, shot and loaded into off-highway rear dump trucks. Drilling was fast and economical in this type of rock. Because jointing was predominantly vertical and horizontal, the spillway excavation was designed for vertical drilling in approximately 12m steps with benches of 6 m to 8 m width, except where concrete was to be placed directly against the rock wall, where no benches were provided. Instead, the contractor was allowed to drill slightly off vertical to undercut the required theoretical line of excavation to accommodate his drillhead. On the left wall of the excavation the existence of weak joints, inclined unfavourably, caused some falls which necessitated the installation of prestressed anchors. The permanent near-vertical faces of the spillway cut and excavation for the overflow monoliths, where shaped to receive concrete, were presplit by line blasting with holes at 750 mm centres, before any bulk excavation was done.

At various levels, continuous weak nearly horizontal joints existed, fortunately at very wide spacing. The weak material filling these joints varied in thickness and composition - in thickness from about a few millimetres to one metre, and in composition from clay to a weak sandstone or shale, coal seams and claystone. On the left side of the spillway, the lowest such seam was at about EL 36, and the foundation of the main spillway blocks was taken to this level to avoid this feature.

14.2.2 Geological Mapping of Spillway Foundations and Side Walls

Following excavation of the spillway the foundations and side walls were geologically mapped prior to the placing of concrete. Figure 57 (Drawing A1-71363) lists the individual drawings covering the foundations and side walls of the spillway. None of the drawings listed in this key plan have been included in this report, but have been supplied to the South East Queensland Water Board under separate cover.

14.3 Flood Routing

The dam was assumed full with the reservoir level at EL 67.0 at the beginning of a flood for all flood routing studies except those involving the Probable Maximum Flood, where the reservoir level at the start was taken at EL 68.5 to allow for a possible future increase in full supply level.

As inflows into the dam are unable to be predicted accurately, the gate controller cannot be expected to utilise 100% of the available flood storage. To allow a margin for error, 85 percent of the available storage between EL 67.0 and EL 77.0 was used when routing floods other than the Probable Maximum Flood through the dam.

For the Probable Maximum Flood, the waterlevel was allowed to come to the maximum, with all gates open since it is a very rare flood.

Flood routing studies were divided into two categories, (i) the flood mitigation value of the dam for the benefit-cost analysis, and (ii) the dam safety for which the Probable Maximum Flood was used.

The adopted Probable Maximum Flood (PMF) is shown in Figure 58. It was derived by maximising the 1893 storm in situ (Reference 24). The 1893 storm was responsible for the largest flood on record in the Brisbane River.

The adopted PMF had a peak of 15 000 cumecs and a volume of 4.21 million ML, equivalent to 600 mm of runoff over the catchment area. By comparison, the flood storage of Wivenhoe Dam between the full supply level (FSL) of EL 67 and the maximum water level of EL 77 is 1.4 million ML.

This PMF was used to assess the safety of the dam against overtopping. In addition, inflow hydrographs for various historical floods (e.g. the 1893 and 1974 floods) and for floods synthesized from storm frequency data were developed in order to provide data for a benefit - cost analysis for the flood mitigation component of the dam.

For the flood mitigation benefit-cost studies, the historic and synthesised floods were routed through the dam and the outflow routed down river to Brisbane. The objectives were to limit outflow below a damaging level for Brisbane consistent with the available storage and to empty the dam within a reasonable time , say 5 or 6 days , after the reservoir has reached maximum level.

The results of flood routing for the economic studies are summarised in a report by Grigg (Reference 25). The 1974 flood, which reached 5.45 metres on the Brisbane City Gauge, would have been lowered by 2.6 metres if Wivenhoe Dam had then been in existence. The damage caused by the 1974 flood was estimated at $178 million, and the savings produced by lowering the flood would have been $140 million.

The flood mitigation studies indicated that all major historical floods could be controlled with outflows not exceeding 3 200 cumecs. If no other inflow occurs below the dam a prolonged outflow of this magnitude would cause little or no damage to Brisbane. The dam would then be able to be emptied in a reasonable time after a major flood such as the 1893 flood.

The above data with respect to the PMF applied at the time of the design of Wivenhoe Dam. Since then design flood estimates for Somerset Dam and Wivenhoe Dam have been revised under the Brisbane River and Pine River Flood Studies. Details of the revised flood estimates are included in reports Nos 8 and 24 of Appendix B of this report.

14.4 Spillway Gates

14.4.1 Gate Arrangement

The number and size of gates were determined mainly by economic considerations. Certain operational requirements also influenced the size of the gates and concrete crest level. For instance, one requirement laid down was a large outflow capacity with the reservoir at FSL so that large volumes of water could be released in the early stages of a major flood. A desirable feature was the ability to hold back a substantial flood volume with little or no release so that coincidence of releases with downstream tributary inflow could be avoided. This last requirement made it necessary to have the top of the gates considerably higher than FSL and EL 73 was adopted for the top of the gates. The reservoir volume between the FSL of EL 67 and the top of the gates is 800 000 ML. This height of gates was also checked by routing historical and synthetic floods with various modes of gate operation.

The height of the gates was checked by flood routing calculations. It was found that, if the most likely type of operation was adopted, as set out in Cossins (Reference 3) with respect to the criteria of emptying the dam quickly , the major floods like the 1893 and the 1974 floods could be controlled by a gate with its top at EL 70. However, in order to achieve lower initial outflow levels and to give flexibility of operation, as described earlier, it was decided to have higher gates.

Economic comparisons led to the adoption of 5 radial gates each 12 metres wide by 16.6 high with a fixed concrete crest of EL 57. The savings of this arrangement compared to 6 gates and a crest level of EL 58 was $1.2 million (August 1979). The five gates chosen gave a width of spillway cut providing sufficient excavated material to balance the fill required in the embankment. The design of the gates is covered in Section 23.0.

14.4.2 Gate Operation

The aspects of gate operation in this section is based on hydrological data available at the time of the design of Wivenhoe Dam (Reference 6).

During a major flood, the outflow from Wivenhoe Dam will normally be controlled with the gates partly opened during the whole period of the flood. It is only in the event of a very large flood that the gates will be fully open and then only in the later stages of the flood. The following considerations would apply during a major flood control operation:

(a) Safety of the dam.
(b) Outflows must be kept to non-damaging levels for as long as possible.
(c) Outflows should generally be less than the corresponding natural flood flow.
(d) The rate of increase of outflow should be limited to allow adequate warning downstream.
(e) To avoid coincidence of the outflows from the dam with peaks arriving at Brisbane from tributaries downstream of the dam, it may be desirable to severely cut back the dam outflow for short periods.
(f) The outflow should be high enough so that the reservoir may be emptied in time to receive inflow from a possible subsequent flood event.
(g) The rate of increase and decrease in outflow from the dam should be kept to within limits so as to avoid possible danger to people and damage to river banks downstream.

For the PMF, various modes of operation were considered, and cases with a varying number of gates inoperable (i.e. not able to be opened) were examined. When 3 gates or less out of the 5 were operable, it was assumed that the operating rules would force the operator to open gates to match outflow to inflow until the gates were fully open. It was found that the PMF could be passed with this rule as long as at least 2 gates remained operable, with no freeboard remaining.

Figure 58 shows one method of routing the PMF through the dam with five gates operating. The reservoir level at the start of the flood was at EL 68.5. The gates remained closed until the reservoir reached EL 70.5 when the gates were opened incrementally until the discharge equalled the non damaging outflow of 3200 cumecs where it was held as long as possible. When the water level reached EL 73.5, gate opening resumed until the gates were fully open with the reservoir level at EL 75.5 and a discharge of 10,350 cumecs. As the inflow continued the reservoir reached a maximum level of EL 76.9 where the discharge was 11780 cumecs. (This example does not apply to the current operation of Wivenhoe Dam. Operating procedures are included in Reference 26).

In the unlikely event that the controller kept the gates shut during the PMF and the reservoir level reached the top of the gates, it was found that as long as the subsequent rate of opening was at least 300 cumecs per hour then the dam would not be overtopped. This method of operation would not be recommended for very large floods. Calculations have proved the safety of the design. The aim of the design was to provide as much flexibility as possible for future controllers of the spillway gates.

The current procedures for operating the gates at Wivenhoe Dam are detailed in the Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam (Reference 26).

14.5 Hydraulic Model Studies

14.5.1 Hydraulic Models

Most of the hydraulic data for the spillway design was obtained by testing of two physical hydraulic models (Reference 23). A 'pilot' model of 1 to 200 scale was used to determine the feasibility of a flip bucket spillway and to assess overall hydraulic behaviour of the spillway, the approach flow conditions and discharge back to the river. The section of the river from AMTD 152.0 km to AMTD 148.9 km was reproduced in this model.

A 1 to 80 scale 3-dimensional model of the spillway termed the Main Spillway Model, was tested to determine the spillway structure geometry, hydraulic loading data, gate discharge ratings, proposed sequencing of gate openings for flood discharge, and dissipation performance in the flip bucket plunge pool. Only part of the approach channel, the spillway structure, and part of the discharge channel were reproduced in this model.

14.5.2 Pilot Model

The initial spillway approach arrangement tested was a conventional vertical training wall layout with a radiused transition on the upstream ends. This layout produced a concentration of flow towards the left of the spillway. Large vortices were shed from the end of the right approach wall and considerable vortex motion occurred in front of the two gates on the right. This behaviour would have resulted in poor discharge control and asymmetric flip bucket and plunge pool conditions. The major reason for the asymmetric approach condition was that the higher natural surface levels on the left of the spillway approach channel forced a skewing of flow from the right. Various arrangements of approach groynes and long-radiused training walls were tested to produce a final approach layout as shown in Figure 59 (Drawing A1-50782C). A reasonably uniform approach velocity distribution was obtained across the face of the spillway with this layout. Detailed velocity measurements were taken on the face of the groynes to determine riprap protection requirements. Results of these measurements are shown in Figure 60.

The flip bucket appeared to be satisfactory in the pilot model with a number of improvements which were tested on the main spillway model. The spillway flip was originally the same width as the discharge channel (74.0 m) but convergence of the sidewalls downstream to a width of 68.0 m was necessary to ensure that the overflowing jet from the spillway did not impact on the berms above the discharge channel. The initial piers were parallel sided with square ends. The waves generated from the pier ends were unstable and relatively high, intermittently forcing a high velocity jet to impact on the discharge channel berm. This problem was alleviated by tapering of the piers and was further investigated in the main spillway model.

The chosen flip bucket exit angle of 25° was based on current world practice with a compromise between the distance the jet is projected downstream and the angle of incidence of the jet on the floor of the plunge pool. The resulting scour hole developed a sufficient distance downstream not to endanger the stability of the spillway. The lip level of the flip bucket was some 8.0 m below the tailwater level for the Probable Maximum Flood but the flip bucket was not drowned.

It was virtually impossible to reproduce in the model the scour of the rock material in the discharge channel. If the discharge channel had a non-erodible bed at EL 28.0 supercritical flow was possible with a hydraulic jump being formed at the downstream end. This was considered undesirable as massive scour could develop in the less resistant rock at the downstream end of the discharge channel and uncontrollably progress back upstream. For this reason, a pre-excavated scour hole was experimented with to ensure energy dissipation occurred in the region where the jet impacted the bed of the discharge channel and thus controlled the location of the major scour. This concept was further tested in the main spillway model.

To give some guidance to the range of possible scour depths, a number of approaches were used. Several scour 'formulae' are presented in the literature based on model and prototype measurements but the diversity of the results obtained would indicate some unreliability in predicting stable scour depths. Scour tests were performed in the pilot model using vertical non-erodible sidewalls in the discharge channel with an erodible bed. Granular beds ranging from fine sand to graded mixtures were tested. Loosely packed concrete cubes were also tested in an attempt to reproduce the behaviour of hard but jointed rock where the scour could be caused by the high dynamic pressures of impact penetrating the joints and lifting blocks of rock into the flow.

The maximum bed velocities in the order of 4 m/s were encountered in the discharge channel downstream of the plunge pool. There was little tendency for strong circulation currents in the river between the embankment and the end of the spillway discharge channel.

14.5.3 Main Spillway Model

The approach channel flow distribution of the pilot model was reproduced in the main spillway model. Water surface profiles were measured against the approach walls to determine design loadings. The necessity for the convergence of the spillway side-walls, as shown on the plan of the spillway, Figure 56 (Drawing A1-50771C), was confirmed and the overall behaviour of the two models was similar.

The crest section of the spillway profile consisted of a standard USBR ogee crest shape for a design head of 15.0 m. The radial gates were located downstream of the crest so that the jet trajectory from small gate openings more closely approximated to the ogee shape. A 15.0 m radius was initially used for the flip bucket with a sloping apron and 17.0 m radius to connect to the crest shape. Pressures measured along the spillway profile indicated pressure peaks on the two radii but with subatmospheric pressures on the sloping apron for some flow cases. Also at higher discharges, flow concentric with the circular flip bucket surface was not maintained with consequent poor jet trajectory. The two radii and sloping apron were substituted by a single radius of 41.8 m with the same location and exit angle of the flip and improved performance was obtained.

Water surface profiles were measured for various flows to determine the height of the spillway side walls. Pressures on the crest and flip bucket were measured for various uncontrolled and gate controlled discharges. Figure 61 shows a comparison of water surface profiles and pressures for the gate controlled design discharge of 5000 m^3/s and an uncontrolled discharge of 11 700 m^3/s. Pressures were also measured for the case of the bulkhead gate used as an emergency flow control. A maximum subatmospheric pressure of 3.6 m was developed which gave a reasonable margin against cavitation on a smooth concrete surface. Maximum velocities on the face of the spillway were in the order of 22 m/s.

Waves from square ended piers and tapered piers were generated in the main spillway model similar to those in the pilot model. An extension of the tapered pier by 7.0 m downstream and approximately 5 m high, further improved the uniformity of the jet as well as controlling its spread for single gate operation. Mitre, semi-circular, triangular, parabolic and rounded mitre pier nose shapes were tested. A rounded mitre nose was selected, giving the best compromise between discharge performance and lateral loading on the piers. The lateral water loads on the piers were determined by pressure tappings located in the piers. Further details on the design of the piers are given in Section 18.0.

The pre-excavated plunge pool was considered necessary to initiate the scour hole downstream of the flip bucket in a controlled manner. The basic consideration in the development of the pre-excavated plunge pool shape was that the large scale turbulence should be concentrated away from the unprotected sidewalls of the spillway discharge channel. Various shapes were tested with the shape shown on Figure 56 regarded as a practical solution. The side benches protect the sidewalls from undermining, and sloping the downstream bench faces reduced the possibility of deflecting the jet laterally.

The radial gates were rated over the complete range of operation. A recommended gate opening procedure was developed from the model. The major considerations were to produce symmetrical jet energy dissipation in the plunge pool, keep high velocity jet impact away from the sidewalls for as great a discharge as possible, and to have the jet impact into the greatest tailwater depth. The recommended procedure is to open only the middle gate for small discharges then open adjacent gates for larger discharges with symmetric flow about the middle gate. For discharges greater than 3000 m^3/s all gates would be opened equally. Operating procedures were developed for the emergency case of a gate stuck either open or shut. These procedures were developed in an attempt to produce near-symmetrical conditions for as great a discharge as possible.

14.6 ROCK PROPERTIES OF THE SPILLWAY AREA

14.6.1 Rock Properties of the Spillway Area - General

In addition to the detailed geological investigation, testing of the mechanical properties of the rock was carried out to establish design parameters for both the spillway walls and over-flow monoliths. For this purpose vertical and horizontal cores were available from 75 mm and 100 mm drill holes at the site of the 1050 mm exploratory shaft. Unconfined compression tests of the core, both saturated and dry, indicated a general increase in strength and stiffness of the rock with depth. Also, the strength and stiffness in the plane of bedding was only marginally greater than that normal to the bedding. The strength of saturated samples was in general less than half that for dry samples. Typical compressive strength for saturated samples at about foundation level was 20 MPa with Youngs modulus in the range 5 - 10 GPa.

14.6.2 Spillway Walls

Because the rock was competent and relatively uniform with the bedding planes undisturbed and with no notable set of inclined joints, the cuts in rock below the highly weathered zone were designed as nominally vertical. The rock faces were cut successfully by line blasting with holes at 750 mm centres.

In the vicinity of the spillway retaining walls, intake structure, crest and flip-bucket, rock walls were lined with 1 m width of reinforced concrete anchored into the rock by unstressed anchors. The rear of the walls was drained to relieve excess pressure build-up. The location of grouted anchor bars in walls is shown in Figure 62 (Drawing A1-54537C). All anchor bars were cold worked deformed bars (Grade 410C) to AS 1302-1977. A typical layout of drainage behind vertical spillway walls is shown in Figure 63 (Drawing A1-56196A).

As a check on horizontal movement in the excavated rock face, an inclinometer tube was installed in a vertical drill hole adjacent to the deepest section of cut. Movements due to stress relief were expected and measurement showed that stepped displacement occurred across three identifiable bedding features. The lower two of these defined a zone of weakness in the sandstone which was removed from the foundation for the crest monoliths.

The shear movements across surfaces within the high left-hand rock wall were detected during blasting and excavation of the spillway channel. Creep of the face with time was not significant although a second inclinometer was installed to continue the observations when the initial one was closed off by shear movements at the lower levels. Also during the excavation for the left-hand rock wall, a limited number of fresh joints showed up with a strike roughly parallel to the wall and dipping approximately 60° into the cut. In the vicinity of monoliths LH7 and LH8, and to a lesser extent monoliths LH11 to LH13, where potential failures had been activated in the process of excavation, the overlying wedges of rock were removed. The possibility that similar but undetected joints existed within the wall resulted in a system of prestressed anchors being provided in the rock face. This guaranteed stability both during construction and for the completed structure. Details of the prestressed rock anchors are given in Section 15.3.

To test the frictional resistance which could be expected along an inclined joint, cores were split approximately at the dip angle of the joints and sheared across the irregular fracture. Tests were representative of distinct upper and lower rock strata. According to the relationship ($\tau = \sigma \tan [\varnothing + i(\sigma)]$) peak shear strengths of the fractures over a range of normal stress σ were measured as well as the residual shear angle $\varnothing r$ (see Table 31). The range of shear strengths over a range of stress could thereby be deduced.

TABLE 31 - SHEAR AND FRICTION ANGLES OF SPILLWAY STRATA.

STRATA	(Ør)	FRICTION ANGLE (Ør + i)		
NORMAL STRESS (MPa)		0.5	1.0	1.5
UPPER	27°	58°	52.5°	48.5°
LOWER	31.5°	64.5°	58.5°	55°

14.6.3 Foundations for Crest Monoliths

From investigations, a weak zone beneath the spillway crest was anticipated to determine the foundation level for the monoliths. Advance proof drilling below the expected foundation indicated that the rock there was substantially uniform and relatively massive. To measure the horizontal shear properties of the rock for the foundation, samples of intact core were tested. The shear behaviour for typical samples at confining pressures up to 2 MPa has a lower limit defined by
$\tau = 1.7 + \sigma \tan 56°$ (MPa).

Excavation of rock for the embedment of the steel outlet pipes in the left hand monolith made available a face below the general foundation for inspection. It indicated that lenses of siltstone included in the sandstone and thin films of coal on bedding planes were more extensive than expected.

Additional tests were therefore made of samples which contained varying proportions of siltstone and coal on the shear surfaces. The results of these tests suggested that the shear capacity of the foundation would not be seriously impaired by the likely incidence of coal and siltstone.

15.0 SPILLWAY TRAINING AND RETAINING WALLS

15.1 Spillway Training and Retaining Walls - General

Upstream of the spillway crest , rock groynes merging with gravity concrete walls were designed to provide uniform approach conditions across the width of the spillway as shown in Figure 59 (Drawing A1-50782C.)

In regions where velocities of the spillway flow could be resisted by rock protection, rock groynes were used to train the flow. The groynes were constructed of rolled sandstone with riprap protection. Model studies to determine the approach flow conditions upstream of the spillway crest were carried out as discussed in Section 14.5. Velocity measurements taken on the pilot model indicated the need to concrete the riprap on the sides facing the approach channel. However specially selected oversize Zone 4A riprap was placed on these surfaces and concrete was not required. Water loadings on the training walls for various floods were measured in the main spillway model.

Closer to the spillway crest, where velocities are higher, the rock groynes give way to concrete training walls. Figure 60 shows the water velocities approaching the spillway for the Probable Maximum Flood.

The mass concrete walls also perform the function of containing the embankment fill adjacent to the spillway.

In the preliminary design, reinforced concrete counterforts, and mass concrete were considered for the spillway training/retaining walls, the latter being adopted mainly because of the ease of providing contraction joints in the complex area of the inlet structure.

15.2 Gravity Walls Located Above Vertical Cuts

The left hand side training wall upstream of the spillway crest consists of 11 monoliths LH1 to LH11. Monoliths LH1 and LH2 were designed as mass concrete walls founded on a level foundation. Monoliths LH3 to LH5 were designed as mass concrete walls founded on top of a vertical cut approximately 6m high. Monoliths LH6 to LH8 were designed as mass concrete walls standing on top of progressively increasing vertical cuts up to approximately 31 m high. However following the removal of potential slip failure material as described elsewhere, the design of Monoliths LH6 to LH8 was modified as shown typically in Figure 64 (Drawing A1-54833A). The area was also stabilised by the installation of grouted anchor bars and by drilling pressure relief holes 75 mm dia 2.0 m deep on a 2 m x 2 m pattern. Details are shown in Figure 65 (Drawing A1-56265A).

Monoliths LH9 and LH10, incorporating the inlet structure for the outlet works, are mass concrete retaining walls standing on vertical cuts up to about 40 m high. Monolith LH11, adjoining the underground control structure, is a concrete wall placed against a vertical cut 39 m deep.

Generally, side walls downstream of the spillway crest are concrete walls anchored to the vertical cuts with grouted anchor bars.

Because of their location, monoliths LH6 to LH10 were at risk of overturning about a point below the general foundation level of the wall. Uplift pressures in the rock joints were also a major destabilising force. A less likely mode of failure was sliding along an inclined joint. However the inclined joints parallel to the walls of the excavation were of a limited extent and the risk of sliding failure was minimal. Vertical and horizontal joints were much more frequent with an increased risk of toppling failure.

In monoliths LH11 to LH13 downstream of the spillway axis, the side walls are vertical or near vertical and there was little danger of sliding or toppling failure. As there were no retaining wall loads or earthfill loads and the uplift loads were not extreme, stability of the walls in this area was required for the construction period only.

The training/retaining walls on the right hand side of the spillway, Monoliths RH1 to RH11, were designed as mass concrete walls founded on level rock foundations or at the top of vertical walls no higher than 5 metres and stability problems did not exist.

The walls of the spillway were supported by grouted anchor bars as shown in Figure 62 (Drawing A1-54537C), and also by prestressed rock anchors as described in Section 15.3 below.

15.3 Prestressed Rock Anchors

The existence of further undetected inclined cracks as mentioned in Section 14.6.2 could not be overlooked. As any movement of the embankment retaining walls could endanger the safety of the whole dam, a decision was made to stabilize the vertical rock face behind monoliths LH7 to LH13 on the left hand side of the spillway.

The maximum height of the vertical rock wall was 40m and in the design of the prestressed anchors it was assumed that an inclined crack could occur from any level on the vertical face and at any inclination.

Direct shear testing of cores from a hole adjacent to the left bank vertical wall was carried out to determine shear strengths to be used in the design of the prestressed anchorage system. Details are given in Section 14.6.

In designing the prestressed anchorage system a load factor approach was adopted. In general the critical failure surfaces extend from the bottom of the vertical rock face to the heel of the mass gravity retaining wall. The spacing and location of cables were arranged so that all cables were of the same length and size, being 17 m long including an anchorage length of 6 m except when shale was present in the drill hole where the length was increased to 18 m, and the anchorage to 7 m.

Prestressing strands were 12.5 mm dia Supa 7, with 25 strands per tendon. The ultimate tensile strength of the tendon was 4600 kN. All tendons were stressed to 80% of the ultimate tensile strength i.e. to 3680 kN. A total of 211 anchors were installed.

The design and location of the rock anchors are shown in Figures 66 and 67 (Drawings A1-56087A and A1-56088A.)

15.4 Design Parameters for Mass Concrete Spillway Walls

The design parameters described in this section apply to the less complex gravity training and retaining walls such as monoliths LH1 to LH8 and RH1 to RH11. Monoliths LH6 to LH8 were modified following the removal of potential slip failure material in the area. A typical analysis of a more complex retaining wall monolith such as RH12 is detailed in the following Section 16.

Loads acting on the gravity walls are:-

- **Water pressure.**

  Water pressure acts on both sides of the walls. On the streamside water can be flowing and hence lower than the static water pressure on the back of the wall.

- **Earth and rockfill pressures from the retained fill.**

  In the spillway area these pressures act on the embankment side only, whilst in the groyne area the pressure acts on both sides of the wall. Differential loads, varying during construction, act on monoliths LH1 to LH4 and RH1 to RH4 where the walls merge with the groynes.

Three different water levels were used in checking the stability of the walls. These were:-
- Water level below the foundation level;
- Adopted Full Supply Level of EL 68.5; and
- Probable maximum flood level of EL 77.0.

Because of the sloping zones in the embankment, the earth and rockfill loads can vary across a monolith. This produces torsional as well as unidirectional shear friction under the base. In determining the embankment loads on a monolith the centreline cross-section was usually chosen. In some cases where the additional torsional base friction was of concern, the end having the higher embankment load was chosen for design of the monolith.

Coulomb's trial wedge method was used in lieu of Rankine's method for determining active and passive forces acting on the monoliths for the following reasons:-
- The retaining walls have sloping back faces;
- In some cases the embankment fills slopes up and away from the wall whilst in others it slopes down and away from the wall;
- The embankment fill against the wall consists of zones of materials having different angles of internal friction; and
- The water level in the embankment varies.

The following values and assumptions were adopted in the design of the spillway walls:-

• Soil density	$2.2 t/m^3$
• Angles of internal friction:-	
Riprap	45°
Well graded river gravel	40°
Rolled sandstone	37°
Clay core	25°;

- For determining the active forces an angle of wall friction of 10° was used; for passive forces an angle of wall friction of 0° was used;
- Active earth forces were calculated in lieu of "earth at rest" forces;
- Coefficient of base friction varied between Tan 28° and Tan 37°;
- Maximum bearing pressure of 1 MPa was used.

In checking the stability of the monoliths the following were determined:-
- Factor of Safety against sliding;
- Factor of Safety on the coefficient of base friction;
- Factor of Safety against overturning;
- Location of the resultant vertical forces on the base; and
- Pressures under the base.

It was found that the Factor of Safety on the coefficient of base friction was a better indicator of the actual stability against sliding than the Factor of Safety against sliding.

The monoliths were considered to be safe if one or both of the following conditions were met:-

- Factor of Safety against overturning of at least 2.0;
- Resultant of vertical forces under the base was within the middle third.

Waterstops were provided between monoliths to prevent removal of fines from the embankment.

16.0 TYPICAL STABILITY ANALYSIS - MONOLITH RH12

16.1 Stability of Monolith RH12

16.1.1 Monolith RH12 - General

The general arrangement of Monolith RH12 is shown in the following figures:-

Figure 68	(Drawing A1-54815B)
Figure 69	(Drawing A1-54816D)
Figure 70	(Drawing A1-54817C)
Figure 71	(Drawing A1-54818B)
Figure 72	(Drawing A1-54820G)

Monolith RH12 forms part of the retaining wall system on the right abutment of the spillway. It performs the following functions:

(a) It retains part of the right embankment fill including the impervious core;

(b) It forms the right abutment of the spillway and functions as part of the spillway training wall;

(c) It supports a radial gate trunnion corbel;

(d) It houses the bulkhead gate chamber;

(e) It provides access between the top and bottom galleries via a shaft;

(f) It supports a beam which carries one of the crane rails, the other crane rail being supported by the downstream wall of the gate chamber.

In order to provide a positive contact with the clay core, the back of the bulkhead gate chamber was provided with cantilevered wing walls and a cutoff. These were shaped and sloped to provide positive contact on settlement of the core.

The bulkhead gate chamber was designed integrally with the retaining wall downstream and the whole presents a monolith 28.5 m wide. The integral action of this assembly is aided by the longitudinal prestressed tendons installed in the streamside face of the monolith to take the radial gate load , as shown in Figure 68 (Drawing. A1-54815B). On the back face, liberal amounts of reinforcement were provided at the junction of the chamber and retaining wall to ensure integral action.

Galleries were provided both longitudinally under the bulkhead gate chamber and upstream-down stream in the retaining wall. Drain holes 12 m deep were drilled into the rock from these galleries to provide uplift relief under the monolith.

16.1.2 Stability Analysis

As described above, the monolith has been treated as a single block for the purpose of the stability calculations. Bending has been considered about both horizontal axes. Torsion about the vertical axis was ignored.

In calculating earth and rockfill pressures, Active Pressure calculated by the Rankine Method was adopted and wall friction was ignored. The following parameters were used:

Impervious core: Saturated Density = 20.54 kN/m^3;
Angle of Friction = 27° .

Sandstone Rockfill: Saturated Density = 18 kN/m^3;
Angle of Friction = 37 °.

The following loads act on the structure:

(i) The self weight of the monolith (concrete unit weight taken as 23.5 kN/m^3).

(ii) Weight of sandstone rockfill acting down on the sloping back face of retaining wall (unit weight of rockfill taken as 18 kN/m^3).

(iii) Rockfill pressure and water load on the upstream face. A surcharge load of 20 kPa was added to the rockfill pressure. It is to be noted that the existence of Monolith RH11 immediately upstream relieves Monolith RH12 of much of this rockfill load.

(iv) Earth pressure from the impervious core and water pressure below the phreatic line on the embankment wall of the bulkhead gate chamber. The earth pressure includes a contribution from the assumed 20 kPa surcharge. Earth pressures below the phreatic line were calculated using buoyant weight.

(v) Sandstone rockfill load acting towards the river on the retaining wall section downstream of the gate chamber. This has a small stabilizing component acting upstream which was not taken into account. Again, 20 kPa surcharge was considered as acting.

(vi) Water pressure behind the gate acting against the wall on the spillway side.

(vii) Rockfill pressure acting in the upstream direction.

(viii) Radial gate load acting on the trunnion corbel acting mainly in the downstream direction.

(ix) Uplift acting underneath the structure. Provision was made for 2/3rds reduction of uplift pressure at the line of drains, i.e., the pressure assumed was full headwater pressure at the upstream face, 1/3rd of this at the line of foundation drains, and zero at the downstream end, where there is no tailwater.

Any buttressing effect from the downstream Monolith RH13 was neglected.

Each load was calculated as the resultant of a three-dimensional solid stress figure with the load acting through the centroid of the figure.

The section analysed was the foundation section at EL 53. Two loading conditions were considered:

Unusual Load Case: PMF water loading with the gate load corresponding to a partly open position, i.e., with the top of the gate level with the upstream water level of EL 77, and the bottom of the gate at EL 61.4.

Extreme Load Case: Upstream water level at EL 79 with gate stuck shut.

Under these load cases, it was considered that some tension (up to 0.4 MPa) would be acceptable on the embankment end of the monolith.

The Factor of Safety against sliding was assessed firstly by calculating the Sliding Factor (SF):

$$SF = \frac{\text{Total horizontal force}}{\text{Total vertical force}}$$

Since the Sliding Factor proved to be very favourable, it was not considered necessary to calculate the Shear Friction Factor of Safety.

For the overturning forces, the location of the resultant was calculated with respect to each axis and a factor of safety against overturning about the edge parallel to the river was calculated as well as the consequent stresses.

Figure 73 shows the results of both analyses, including effective vertical stresses at the corners, location of resultant and sliding factors. These values are for a section at EL 53.

The stability of the portion of Monolith RH12 downstream of the gate chamber (which is shaped like a conventional retaining wall of decreasing height),was also checked as if it was entirely independent, and acted on only by the rockfill active pressure behind it. Although this is an unrealistic case, it would tend to indicate the degree of strain between the two very dissimilar elements of the monolith. For instance, if there is a great deal of difference between the factors of safety for overturning or the location of the resultant then it would be expected that a great deal of strain would result at the junction of the two elements. The following results were obtained:

Factor of Safety for overturning about the toe	= 3.41
Sliding Factor	= 0.23
Location of Resultant in from toe	= 4.33 m.

The location of the resultant from the toe compares favourably with the 5.44 m obtained from the three dimensional analysis.

The above results show that this portion of the monolith would be stable even if acting independently.

16.1.3 Components of Monolith RH12

Concrete in Monolith RH12 is Grade 20 generally but in special areas mentioned below it was varied to suit the load conditions.

Bulkhead Gate Chamber Walls: These are designed for 'at rest' earth or rockfill pressure ($K_o = 1 - \text{Sin } \phi$) and water pressure where applicable. The design conforms to AS1480 - The Use of Reinforced Concrete in Buildings. Concrete is Grade 25.

Gantry Track Beam and Cantilever: The cantilever which supports one end of the gantry track beam is cantilevered off the end wall of the bulkhead gate chamber. The other end of the track beam is supported on the retaining wall of Monolith RH12 as shown in Figure 69 (Drawing. A1-54816D). All supports are continuous so that the cantilever sustains some torsion. Concrete is Grade 25.

The load on the track beam is imposed by the gantry wheels which are arranged in two bogeys 10m apart, each with 4 wheels 1m apart. One bogey has 1300 kN total load while the other has 800 kN total load.

The track beam is buried in the embankment fill up to its top surface. While this fill cannot provide support owing to its settlement, it will prevent any catastrophic collapse of the track beam in the event of an overload or defective materials. Therefore, it was decided to design the beam and cantilever to conform to the less stringent AS 1480 - The Use of Reinforced Concrete in Buildings rather than the more conservative NAASRA code for bridges.

The cantilever supporting the track beam also supports a continuation of the track beam by means of a corbel, as shown in Figure 69 (Drawing A1-54816D), and Figure 74 (Drawing A1-56240A). The other track beam shown on this drawing is supported off a corbel attached directly to the endwall of the bulkhead gate chamber. These track beams have a smaller span than the track beam attached to the monolith, but are designed to the same standard. The loads are the same as for the track beam described above for the downstream beam but are higher for the upstream beam, being 1600 kN in both bogeys. The inequality of loading on the beams is due to eccentricity of the gate load and to wind load.

Gate Trunnion Corbel and Longitudinal Prestressing: The corbel which supports the radial gate trunnion is prestressed to the wall by 15 - 35mm dia. Super Grade bars to AS1313 having a UTS of 1041 kN , a jacking load of 750 kN and a final effective prestress of 600 kN each. Grade 40 concrete was specified in the corbels and immediate surrounding area as shown in Figure 75 (Drawing A1-56282A).

The trunnion corbel transfers the load from the gate to the monolith wall near its downstream end and in order to ensure that the wall face does not crack it was decided to prestress the wall over its whole length. This has been achieved by the use of 4 tendons with a required final prestressing force of 6060 kN per tendon. Allowing for losses due to concrete shrinkage and creep, steel relaxation and draw-in losses, resulted in an initial jacking force of 7500 kN.

In calculating the stresses caused by the prestress and gate load at the critical section of the wall (immediately U/S of the corbel), the prestress force was assumed to spread at 35 degrees. This gave a section which was analysed as a prestressed beam. The following results were obtained at this critical section:

Initial Prestress Only:	Front Face:	= 3.374 MPa
	Back Face:	= -0.963 "

(There is negligible dead load acting on the section).

Final Prestress Only:	Front Face:	= 2.530 MPa
	Back Face:	= -0.722 "

Gate Load + Final Prestress:

Unusual Gate Load (Reservoir EL77):	Front Face:	= -0.013 MPa
	Back Face:	= 0.946 "
Extreme Gate Load (Reservoir EL79.5):	Front Face:	= -0.513 MPa
	Back Face:	= 1.274 "

Design of the bursting steel in the anchor zones was mainly by the method given in the NAASRA code. LH11, which has a similar prestress arrangement was stressed first, and showed some cracks parallel to the prestressing strands at the dead end anchor, so the bursting steel in the anchor zones of RH12 was increased.

17.0 SPILLWAY OVERFLOW MONOLITHS

17.1 Pier Arrangement

After closure of the main diversion channel, any flood was to be passed through a low monolith on the left hand side of the spillway, where final closure of the dam was made. Therefore the adjacent gate pier had to be built in advance of its left hand side crest section. As a result, monoliths, with the gate piers located centrally, were impractical. The resulting arrangement produced two monoliths each carrying two piers and three monoliths of overflow crest only. This resulted in non-uniform loading such that the gate loads from the equivalent of two bays applied to each monolith incorporating the piers. Details of the two types of spillway monoliths are shown in Figure 76 (Drawing A1-56213A) and Figure 77 (Drawing A1-56212A).

When the gates are controlling the outflow, the pier monoliths are generally less stable than the intervening monoliths while when free overflow occurs the situation is reversed due to the extra weight from the piers. Each monolith is designed to be stable in its own right but to increase the load sharing tendency between monoliths and the ultimate factor of safety, shear keys were provided at the contraction joints between monoliths. No provision was made to grout the keyed joints.

17.2 Load Cases

The possible variations in gate operation, under manual control, for any given flood made it difficult to define the maximum loading on the monoliths for design. Whereas at maximum flood level in the reservoir (EL 77.0), the gates would be expected to be clear of the flow, one or more gates may be lowered either intentionally or by accident. Even for reasonable means of gate operation, partial gate openings were conceivable for very high reservoir levels. So that maximum flexibility of gate operation was provided, the normal design flood loading provided for both the water level and the top of all gates to be at EL 76.0. This coincided with the maximum intentional flood surcharge (approximately 85% of that available) and produced a controlled outflow of 2450 m^3/s, well below the "non-damaging" flow of 3200 m^3/s referred to previously. In extreme floods producing reservoir levels above EL 76.0, the gates needed to be raised progressively to limit the rise in level to EL 77.0 maximum. More severe loading conditions were analysed with commensurate safety factors being allowed.

Catastrophic conditions of gate failure combined with the Probable Maximum Flood led to the consideration of a supplementary emergency spillway through a fuse plug but a barrier wall of 700 mm height added to the crest proved to be most effective. With the barrier wall on the dam crest, the spillway can pass the Maximum Probable Flood (no freeboard) with three gates stuck shut or the 1 in 1000 year flood with all gates stuck shut. This then provides the most extreme loading on the spillway overflow monoliths for which minimum factors of safety were required.

17.3 Foundation Pressures

17.3.1 Foundation Pressures - General

The foundation pressure plays a major role in the stability of the crest monoliths. Initially it was intended to have only one gallery through the crest from which both foundation grouting and drainage at the heel of the base would be carried out. The reduction in both horizontal thrust and vertical uplift to be gained by lessening the foundation pressures under the sloping foundation upstream led to the provision of a second gallery at a higher level (Figure 78). Foundation grouting and a first line of drains from the upper gallery produce a pressure drop along the sloping foundation. To guarantee that full reservoir head did not penetrate beyond the grout curtain at the foundation interface, a special grout cap slab was provided over the top of the grout holes. It was anchored into rock and sealed at the joint with the monolith by waterstops , as shown in Figure 79 (Drawing A1-56249A)

Downstream of the single grout curtain, the emphasis was on providing adequate drainage to prevent excessive pressure build-up. The second line of primary drains is that from the lower gallery through the heel of the base. A subsidiary line of drains from the lower gallery protects the sloping foundation. Seepage from the drains into the galleries is cleared by a pump operating with a stand-by power supply. Further details of installation of the grout curtain and pressure relief drains are given in Sections 17.7 and 17.8.

17.3.2 Pressure Assumptions

Due to possible imperfections in the grout curtain and inefficiencies of the drains, it is difficult to quantify the pressure gradient in the foundations. The normal assumption for a single line of drains is to allow a pressure drop, at the line of drains, of 50% to 75% of the difference between headwater and tailwater with uniform gradients upstream and downstream. In this case, the pressure reduction was related to the outlet level of the drains. The headwater level was the upstream reference for the pressure at the first line of drains and this was in turn the reference for the second line of drains. A reduction of 50% of the difference between the upstream reference and the drain outlet was assumed in each case. Full headwater pressure was adopted to the grout curtain with linear variations to each drain as shown in Figure 78. Although an apron slab is provided on the approach channel upstream of the crest to increase the seepage path, it does not influence the uplift assumptions. At the downstream toe of the base, a foundation drain is provided opening into the underdrains for the flip-bucket. An apparent excess of 10% of the head drop across the base was applied at the toe to compensate for the non-linear head distribution across the base.

17.3.3 Tailwater Levels

The under-drains for the flip bucket open to the tailwater in the discharge channel. For flows less than 5000 m^3/s the tailwater level is less than EL 40.0, the foundation level for the flip-bucket. For these flows, the foundation pressure at the toe of the monoliths is fixed at EL 40.0. At higher flows, the water level under the flip was measured from the hydraulic model and applied at the toe of the monolith. The forward momentum of the jet in the discharge channel tends to reduce the water level at the flip below river levels. Although the river level at the highest discharge exceeds EL 50, the maximum water level under the flip is EL 45.0.

17.3.4 Instrumentation

It is important to measure the behaviour of the foundation in service to ensure that the pressures do not exceed those allowed for. Two lines of pressure measuring devices (hydraulic piezometers) were installed in the foundation under the crest. The pattern of pressure distribution will be of prime interest as well as any changes with time. Details of piezometers installed in the spillway are given in Section 28. In addition, measuring weirs on the gallery drains will allow the quantity of seepage to be measured.

17.4 Stability of Crest Monoliths

17.4.1 Stability of Crest Monoliths - General

Overturning and sliding stability of the crest monoliths on the foundation were assessed by classical methods from the shear friction factor SSF, sliding factor SF and location of the resultant force along the base.

$$SSF = \frac{V \tan \varnothing + CA}{H}$$

$$SF = \frac{H}{V}$$

V	= Vertical forces net of uplift (MN)
H	= Horizontal forces (MN)
A	= Area of plane of contact (m^2)
tan ∅	= Co-efficient of friction
C	= Cohesion of rock (MPa)

Stability against overturning measured by the ratio x/B (x = distance from the heel to the resultant; B = base width) is an alternative to limiting tensions at the heel calculated by simple theory of compression and bending, viz. the "middle third" rule for no tension, $x/B < 0.667$.

A combination of factors caused the monolith adjacent to the diversion channel (Monolith S2) to be the most critical for stability for the following reasons:- (i) The general foundation was lowest at that point due to the geological conditions. (ii) The monolith is one including the piers. (iii) The monolith is partly within the diversion channel.

17.4.2 Stability Factors of Monolith S2 at Foundation Level EL 36.0

Table 32 gives the stability factors of this monolith for a range of loading conditions at a foundation level of EL 36.0. For the severity of the loading, the foundation conditions and the material properties, these factors are judged to represent an adequate margin of safety.

TABLE 32 - STABILITY FACTORS FOR SPILLWAY CREST MONOLITH S2.

HWL (EL)	Gates	TWL (EL)	Flow m^3/s	Shear* Friction Factor	Sliding Factor	Resultant x/B
76.0	HWL	40.0	2450	5.45	.551	.662
76.0	Closed	40.0	600	4.04	.656	.731
77.0	Open	45.0	11700	8.52	.360	.539
77.0	HWL	40.0	5000	5.05	.538	.707
77.0	Closed	40.0	1710	3.46	.699	.768
79.7	3 Closed	42.0	6700	2.07	.815	.878

* for C = 1 MPa; ∅ = 40°.

Trial mixes with the proposed construction materials allowed a check of the concrete density which was slightly in excess of the 2.3 t/m^3 used in the analysis.

A significant factor in the stability analyses for gates closed or partially open was the pressure applied on the upper surface of the crest. Location of the radial gates to suit the crest shape exposed the upstream 8 m of the surface to reservoir pressure. The crest pressures in the hydraulic model allowed the load on the crest upstream of the gates to be evaluated for partial gate openings. Also, the circular profile for the flip began well within the crest monolith and produced significant crest pressure downstream. In the analysis, these were taken into account from measurements on the model.

Loads from the flip bucket and the rock underlying it, but above the founding level of the crest monoliths, provided additional stability to the crest monoliths against ultimate failure. However, no account was taken of this in the assessment of primary stability. The buttressing action of the downstream block infilling the diversion channel was capitalised on by grouting the contraction joint with the crest block and providing continuity.

17.4.3 Stability Factors at EL 50.5

Monoliths S2 and S4: These monoliths have gate piers at each end. The vertical cantilever steel which reinforces the piers is terminated at EL 50.5 so that below this level the stability depends on gravity action only. A conventional gravity calculation was therefore performed for the pier and associated mass concrete crest at this level. A general arrangement of the monolith S4 is shown in Figure 76 (Drawing A1-56213A). Reinforcement details are shown in Figure 80 (Drawing A1-56200B).

The concrete unit weight was assumed to be 23.5 kN/m^3. The section analysed lies just above the upper gallery and is intercepted by the cored uplift-relief drains, so that the usual uplift assumption of 2/3 reduction at the line of drains, which was adopted, is conservative.

The two load cases considered were:

The Unusual Load case: PMF with gates partly open, i.e., with waterlevel to top of gates at EL 76, and the bottom of gates at EL 60.

The Extreme Load case: Gates stuck shut with upstream water level at EL 79.7.

The resultant was located at a point x from the upstream face and the ratio x/B was calculated where B is the total width of the section measured from the upstream face to the downstream edge of the pier ignoring the extension at the downstream end of the pier. The Sliding Factor (SF) was also calculated. The following results were obtained:

Unusual Load: x/B = 0.655 and SF = 0.63

Extreme Load: x/B = 0.857 and SF = 0.97

The values obtained for the Unusual Load case are considered acceptable. The values for the Extreme Load case are also considered acceptable, but the Shear Friction Factor of Safety (SFFS) was also calculated using an angle of friction of 45 ° and a cohesion for the concrete of 1500 kPa. The result was a SFFS of 1.97, which is sufficient for this extreme case. The compressive concrete stress at the downstream edge of the piers was also calculated assuming that the concrete was unable to take tension. This resulted in a compressive stress of 3278 kPa, again acceptable for this extreme case.

17.4.4 Seismic Stability of Spillway Crest Monolith

The earthquake effects were computed in two ways:

(i) A pseudo-static analysis with a horizontal acceleration of 0.05g applied at the centre of gravity of each component of the structure. The water load due to earthquake was obtained by the Zangar Method, which assumes that the water is incompressible (Reference 27). The reservoir was assumed to be at FSL. A conventional stability analysis including the earthquake loads was then performed on the foundation section at EL 36 and gave the following results:

Location of Resultant x/B = 0.49 where B is the base length and x is measured from the upstream edge. This is so favourable that it was not worthwhile calculating the foundation stresses.

The Sliding Factor H/V = 0.39. Again this is very safe.

(ii) Simplified method of earthquake analysis taking earthquake response of the structure into account (Reference 28). The fundamental period of the structure was calculated using the formula, which was developed by finite element analysis of non-spillway dam sections and therefore would be only very approximate when applied to a monolith with two gate piers. In the absence of an Australian response spectrum, the response spectrum used for assessing the structure response was that given in the reference, which is applicable to Californian conditions where earthquakes are due to tectonic boundaries. Australian earthquakes are of a different kind and have different response spectra, but it is believed that the adoption of a Californian spectrum is conservative.

The adopted peak ground acceleration was 0.05g as before. The loads were computed for 10 sections and the total loads were used in a conventional gravity analysis to locate the resultant. It was found that the results were almost identical to those from the pseudo-static method:

Location of resultant x/B = 0.5m

Sliding Factor = 0.39

These results were considered acceptable.

17.5 Flip Bucket

Design of the flip bucket provided for the pressures which could penetrate to the foundation level. Entry of fluctuating pressures from the flow was to be supressed by waterstops at all contraction joints. Entry past the seals or seepage through the foundation was meant to be collected by drains against the foundations. To provide for a nominal excess pressure build-up under the slabs, anchor bars were grouted into rock. The anchors were designed on a 2 m square pattern for 32 mm diameter cold-worked bars, Grade 410C, embedded 4.5 m into rock.

On the 2:1 slope below the flip leading down to the plunge pool, a concrete slab was provided to protect the foundation of the flip-bucket from erosion mainly from small discharges before the jet starts to spring clear. This slab has nominal grouted anchor bars with the addition of a closely spaced row of deep anchors at the downstream end to guard against regression of scour from the plunge pool. The 10 m of deep anchorage into rock extends approximately to the lowest excavated level of the plunge pool.

17.6 Shear Keys

The spillway overflow monoliths are of two types. Monoliths S2 and S4 are each 19m wide and are integral with the piers which support the road and service bridges and the gate raising mechanisms. Monoliths S1, S3 and S5 are each 12m wide and are simple overflow sections. Each monolith is stable in its own right but to increase the load sharing between monoliths and the ultimate factors of safety, shear keys were provided at the contraction joints between the spillway overflow monoliths and also between overflow monolith S1 and the adjoining monolith LH11, and between overflow monolith S5 and the adjoining monolith RH12. These contraction joints were not grouted. Typical details are shown in Figure 76 (Drawing A1-56213A).

To enhance the stability of the diversion closure monolith S1, as well as being keyed into the adjoining monoliths S2 and LH11, it was also keyed into the downstream flip slab FIA. The keyed contraction joint between Monoliths S1 and FIA was grouted, the grout take being 1786 kg of cement. A 240 mm central bulb type PVC waterstop was installed around the keyed area. Details of this joint are shown in Figure 81 (Drawing A1-56210B)

17.7 Spillway Grouting

17.7.1 Spillway Foundations - Pressure Assumptions

The pressure assumptions used in the design of the crest overflow monoliths are detailed in Section 17.3.1 and 17.3.2 above. The rationale for the construction of two galleries is also described in Section 17.3.1. This section and Section 17.8 describe the actual arrangement of the grout and drainage holes and the results of water loss and grout takes.

17.7.2 Spillway Curtain Grouting - Location

A grout curtain to limit seepage under the dam has been provided over the whole length of the dam. Most of the grouting under the embankment was done under Contract No. 2108 and is described in Section 11.17. The following sections under the spillway and adjoining the vertical spillway cuts were grouted by the Commission's day labour forces:-

(a) On the left hand side of the spillway:-
   (i) the section within 20 metres of the vertical rock face of LH10. This section joins up with grouting under the left abutment as shown in Drawing A1-79268;
   (ii) the floor under the 3600 mm dia penstock in Monolith LH11.

   The arrangement of the curtain grouting in (i) and (ii) above is shown in Drawing A1-56195B.

(b) Under the spillway monliths:-
   The section under the spillway monolith S1, which was grouted from the lower gallery, and the section under monoliths S2 to S5 which was grouted from the upper gallery are shown in Drawing A1-56216B.

(c) On the right hand side of the spillway:-
   The section adjoining spillway monolith RH12 was grouted from the stairs between the lower and upper galleries (EL 48 and EL 55 respectively), and from the upper gallery. Details are shown in Drawings A1-56219A and A1-54817C. This section linked up with the grout curtain from Distances 1240m to 1354m as shown in Drawing A1-57696. This embankment section was also completed by day labour forces at reduced pressures because of its proximity to the vertical right hand wall of the spillway excavation.

The drawings listed in (a), (b) and (c) above show the location of grout holes in the spillway area. Results of grouting, including water losses and grout takes are shown on Drawings A1-57708, A1-57709, A1-57710A, A1-79268 and A1-79269. Copies of these drawings have not been included in this report, but have been supplied to the South East Queensland Water Board under separate cover.

17.7.3 Grout Cap

The grout curtain across the spillway was drilled and grouted from the galleries and through a grout cap which extended from under monolith RH12 on the right side of the spillway to under monolith S2 on the left side of the spillway. The grout cap was reinforced and anchored to the excavated surface with two C20 anchor bars, at 750 mm centres and 3.5 m long, grouted into the rock. Except for a small section 6.5 m long at EL 44.0 under monolith S2, the grout cap was placed in one continuous section without contraction joints, and 190 mm wide central bulb type PVC waterseals were placed continously on each side of the cap to make a watertight connection with the overflow monolith concrete. The spillway monolith concrete was not otherwise bonded to the cap and this allows the concrete monolith to deflect away from the rock under load without full headwater pressure penetrating into the concrete rock interface. 50mm UPVC caps were located in the grout cap and 50mm grout tubes led to the gallery floor.

Details of the grout cap are shown in Figures 79 and 82 (Drawings A1-56249A and A1-56221A).

17.7.4 Spillway Curtain Grouting - General

Except for a section grouted from the lower gallery in Monoliths S1 and S2, the single row of grout curtain holes was drilled and grouted from the upper gallery, and was inclined upstream beneath the bed of the spillway approach channel. To increase the seepage path an apron slab was added upstream of the overflow monoliths and this apron was anchored, as shown in Figure 83 (Drawing A1-54538C), by four rows of C24 anchor bars grouted into the rock up to 8.5 m deep. The grout holes in the section under Monoliths S2 and S1 were inclined progressively downstream.

The grout curtain consisted of two groups of holes. In addition to the deep holes drilled normal to the axis, other holes were drilled at 45° to the axis (in plan view). The 45° holes were provided to intersect shallow sub-vertical joints striking normal to the axis. The primary holes, both vertical to and inclined to the axis, were spaced at 6 m centres. Secondary grout holes were spaced midway between the primaries, and tertiary holes were spaced midway between the primaries and secondaries. The spacing of adjacent vertical holes is therefore 1.5 m; as is the spacing between adjacent inclined holes. Spacing between adjacent grout holes irrespective of their orientation is 750 mm.

The drain holes drilled from the lower gallery were used as proving holes to determine if grouting was required to a deeper level. These drain holes were drilled 25 m deep i.e. to approximate EL 15.0. Only two of the holes lost water under testing, with the maximum loss being 0.9 lugeons. If the loss from a proving hole had exceeded 5 lugeons overall or 2.5 lugeons from a stage of a proving hole, it was intended to review the depth of grouting. The proving holes were backfilled with fine graded sand which was removed after completion of the grouting. The grout curtain was completed prior to the drilling of any other drain holes.

The specified criteria for grouting a hole were:-

"Grouting of a stage of any hole will not be necessary if the water take in that hole is 3 lugeons or less. Only if water takes greater than 5 lugeons are encountered in the secondary grout holes either inclined or normal to the axis, will consideration be given to drilling of tertiary holes."

In spite of the low water losses, all holes including secondary and tertiary in both the holes normal to the axis and the inclined holes were drilled and grouted.

The primary, secondary and tertiary short grout holes at 45° to the dam axis were drilled and grouted in a single 13 m stage prior to drilling the deeper holes normal to the axis. The primary, secondary and tertiary holes normal to the axis were grouted by the upstage method in three stages. These 24m long holes reached approximately EL 31.

On water testing, the majority of the holes were watertight and except for a few isolated holes the grout takes were low. Results of water losses and grout takes are shown in Drawings A1-57708, A1-57709 and A1-57710A, which have not been included in this report, but have been supplied to the South East Queensland Water Board under separate cover.

Grouting pressures measured in the gallery for the various stages were:-

(a) For holes inclined at 45° to the dam axis:- 200 kPa.
(b) For the deeper holes normal to the dam axis:-

Top stage:	250 kPa
Intermediate stage:	300 kPa
Bottom stage:	450 kPa.

Grouting of a hole started with a 3:1 mix (Water:Cement by volume), and the thickest grout used was 1:1. Holes were grouted to refusal, with refusal being defined as a rate of take of 0.05 litre/minute/metre length of hole/100 kPa applied pressure. Measurements for refusal were taken over ten minutes.

17.8 Spillway Drainage

To minimise uplift pressures, pressure relief drains were drilled from both the upper and lower galleries at the upstream face of the spillway overflow monoliths and also from Monolith RH 12. A foundation drainage collection system was installed under the spillway flip blocks. Details of the drainage system are shown in Figures 84 and 85 (Drawings A1-56216B and A1-54547A).

Three principal lines of drains were installed, one from the upper gallery inclined downstream of the grout curtain and two from the lower gallery. The orientation of these rows is shown in Figure 86 (Drawing A1-74872)

The line of holes, drilled from the upper gallery consisted of two groups of different depths. The group 'A' holes were drilled to rock through 100 mm dia starter pipes installed at 3.0 m centres. The group 'B' holes were drilled, through 100 mm dia UPVC pipes at 3.0 m centres installed between the rock face and the gallery, into rock to the depths shown in Figure 86 (Drawing A1-74872), and water tested.

The two lines of drainage holes, drilled from the lower gallery, consisted of three groups of different depths. Groups 'C' and 'D' were drilled in one line. Group 'C' holes were drilled to rock through 100 mm dia starter pipes installed at 3.0 m centres. Subsequent to a review of flows and weeps from Group 'C' holes, they were drilled a further 7m into rock. Group 'D' holes were drilled 4.5 m into rock through formed holes consisting of 100 mm dia UPVC pipe at 3.0 m centres installed between the rock face and the lower gallery. Another line of holes ,Group 'E', was drilled from the lower gallery to intercept the upstream heel of the overflow monoliths. These holes were drilled down to EL 15.0 and water tested as reported in Section 17.7.4.

The results of water testing of Groups 'B' and 'E' holes; and of holes drilled from Monolith RH12 are also shown in Figure 86 (Drawing A1-74872).

All drainage holes were drilled normal to the dam axis, with 60 mm dia diamond drill core bits.

During drilling and grouting, the galleries were drained to a sump in the lower gallery in Monolith S2 and waste was drained by a temporary 225 mm dia UPVC pipe discharging downstream of the apron slabs. Following completion of grouting and installation of drains, the sump was backfilled and two V notch measuring weirs installed to measure seepage. Leakage from the drains is led to the sump at EL 23.5 in Monolith LH14.

17.9 Waterstops

In addition to construction of the grout curtain and pressure relief drains, waterstops were also utilised to control seepage and uplift pressures in the spillway area.

Two rows of 240 mm wide centre bulb type extruded P.V.C. waterstops, complying with BS 2571 Class 3 Compound type G4, were installed between spillway overflow monoliths from a concrete cut-off in the upstream toe around the upstream perimeter of the monoliths to approximately 1m downstream of the radial gate slot. A single waterstop then continued following the downstream profile to the transverse contraction joint between the overflow monoliths and the flip slabs. A typical detail is shown in Figure 76 (Drawing A1-56213A). Waterstops were also installed in the longitudinal and transverse joints in the upstream apron slabs and in the flip slabs downstream of the overflow monoliths. As mentioned in Section 17.7.3 waterstops also sealed the grout cap slab to the spillway monliths.

18.0 PIER DESIGN

18.1 Pier - General

Each pier is 3.5 m maximum width by 31 m long and carries the highway bridge and a service bridge, both of prestressed concrete. The piers have seal faces for the radial gates, and seal faces for a vertical lift bulkhead gate passing between the bridges. Hoists for the radial gates are located on downstream ledges cantilevered off the piers and on platforms cantilevered off Monoliths LH11 and RH12.

Post tensioning within the piers was used for three distinct purposes: (i) the cantilever supports for the road bridge; (ii) the trunnion corbels supporting the gates; and (iii) anchorage of the trunnion corbels into the piers.

The combination of the prestress with the stresses due to the live loads makes conditions within a pier very complex. The shape of each pier is complicated, due to its variable thickness and level of support. Analysis of the piers was carried out by finite element methods.

The critical transverse water loading on the piers is caused by differential openings of the radial gates, and hydraulic model studies were used to determine the water loads on the piers (Reference 23).

Details of the piers are shown in Figures 87 and 88 (Drawings A1-56289A and A1-56309B).

18.2 Geometry of Pier

The basic pier section determined for the five gated spillway was 24 m long and 3.5 m wide with a mitred nose and a square downstream end. The final geometry of the pier section was determined by model studies.

Behaviour of flow through the spillway was tested in two models - a 'pilot' model at a scale of 1:200 and the Main Spillway Model at a scale of 1:80. The main spillway model was constructed to permit easy substitution of various pier nose shapes and to also allow setting of the radial gates to any desired opening. One pier was instrumented with pressure tappings to allow pier pressures to be measured and this pier could be moved to different positions on the spillway as desired.

In the original pilot model with the 24 m long by 3.5 m wide piers, the training walls on the side of the spillway were parallel to each other and 74 m apart, the same width as the spillway discharge channel. With this configuration and the design outflow of 5000 m^3/s, the jet from the flip bucket spread and was not contained within the spillway discharge channel, but landed on the EL 40.5 berm. The width of the spillway flip was then tapered from 74 m at the downstream end of the piers to 68 m at the downstream edge of the flip bucket in order to contain the jet within the spillway discharge channel. For discharges of the order of 3200 m^3/s controlled by the gates, large pier end waves were generated with wave heights in excess of the water depth. (The 3200 m^3/s flow was regarded as the non damaging outflow which corresponded to the likely outflow which would have occured for the larger historical floods of 1893 and 1974). The waves did not strike the spillway training walls but intermittently impacted onto the EL 40.5 berm. This problem was eliminated by tapering the piers from the 3.5 m width at a point 12 m from the nose of the pier to a width of 2 m at the downstream end as shown in Figure 89 (Drawing A3-54791). This shape was further investigated in the Main Spillway Model.

In the Main Spillway Model with the constant width piers, pier end waves similar to those in the 'pilot' model were generated and struck the EL 40.5 berm in the spillway discharge channel. The tapered piers improved the situation considerably. A pier extension of 7 m length and approximately 5 m high was added to reduce the pier end waves and to improve the flow distribution downstream of the piers giving the pier an overall length of 31m. The pier extension also permitted greater flows to be discharged for single gate operation because they restricted the spread of the flow.

In the Main Spillway Model, the piers were constructed to allow interchanging of pier noses. Several pier nose shapes were tested including Mitre, Semi-circular, Triangular, Parabolic and Rounded Mitre. The configuration of the pier and various shapes tested are shown in Figure 89. The best pier nose shape was determined in relation to the effect on the uncontrolled crest discharge curve, the effects on uneven gate operation and the amount of air drawn down the bulkhead gate slot.

With increasing flows, separation and contraction effects became more dominant and the effective width of the spillway reduced. This was partially due to the bulkhead gate slot which induced separation from a fixed point and hindered reattachment of flow to the side of the pier thereby reducing the effective spillway width. In this way the pier noses which provided a gradual transition into the main spillway flow caused less contraction of flow and their headwaters, relative to the triangular pier nose, were reduced.

The degree of separation was also indicated by the amount of air drawn down the bulkhead gate slot under uncontrolled discharge conditions. The smoother transition provided by the rounded mitre and semicircular shapes virtually eliminated this drawdown of air at all flows below about 8000 m^3/s. With the triangular shape this limit was below the design discharge of 5000 m^3/s.

With one gate shut or with largely different discharges through two adjacent gates, sharp pier nose faces produced separation from the pier nose face. Blunt pier noses allowed the flow to remain attached to the face and so reduced the transverse loadings on the pier to a minimum. The rounded mitre shape was adopted because it produced a reasonable contraction behaviour along with minimum pier loading.

Further factors influencing the final geometric shape of the piers are detailed in Reference 23.

The finally adopted shape of the piers is shown in Figure 87 (Drawing A1-56289A).

18.3 Pier Hydraulic Loads

When a gate on one side of a pier is stuck shut with an adjacent one open, the flow becomes detached from the pier at its nose, and almost full hydrostatic load is applied from one side of the pier over the upstream 8 m length. This load over a 20 m height produced very severe loading on a pier and heavy reinforcement and anchorage into the crest were required. Pressure tappings in the hydraulic model provided the loadings on the piers for structural analysis. The greatest loading was produced for the passage of the maximum flood (10,600 m^3/sec) with one gate shut as shown in Figures 90 and 91. (Drawings A3-54812 and A3-54796.)

18.4 Pier Design

For analysis the pier was considered to be a cantilevered flat plate of varying thickness. The in plane membrane stiffness handled most of the hydraulic loads on the gates and pier while the bending stiffness handled the out of plane and non-symetric hydraulic and seismic loadings. The finite element program SAP IV was used for the analysis using the thin plate element with 5 degrees of freedom per node.

Where allowable flexural tensions in some areas were exceeded during loading, the cases were rerun using cracked section properties in such areas to model the effects of cracking on stress and moment redistribution.

The high density of discretization possible with the thin plate element enabled accurate modelling of:

- the variable thickness of the pier using step thicknesses,
- the curvilinear shape of the spillway crest, and
- stresses and moments in areas of high stress and moment gradients.

The use of 3D elements would have necessitated the use of coarser element meshes.

For the design of the pier stem different load cases were analysed as shown in Table 33.

TABLE 33
ANALYSIS OF PIER STEM FOR VARIOUS LOAD CASES.

Load Case	Load	Analysis
1	Gate shut, water over gates at EL 77.0.	Stability against downstream sliding and overturning.
2	Normal working loads, water level EL 67.0.	Stress distribution from prestress and working loads.
3	One gate open and one gate shut, water level EL 77.0, nonsymmetrical flow around pier and no dead weight considered.	Stress distribution from loads.
4	One gate open and one gate shut, water level EL 77.0, nonsymmetrical flow around pier, prestress and dead loads included.	Stress distribution from loads.
5	Dead loads only.	Check finite element model.
6	Dead loads and earthquake horizontal acceleration of 0.05 g.	Stress distribution and moments from loads.
7	Dead loads and earthquake horizontal acceleration of 0.10g.	Stress distribution from loads.

Load case 4 was the worst load case analysed producing the highest moment at the base of the pier stem at the spillway crest.

Design moments (unfactored) for the pier at the spillway crest are shown in Table 34.

TABLE 34
DESIGN MOMENTS (UNFACTORED) FOR PIER AT SPILLWAY CREST.

Distance from nose of pier m	Moment kNm/m
0 - 12	10,000
12 - 17	5,000
17 - 24	2,500
24 - 31	1,500

The design of the pier stem was in accordance with AS 1480-1974 Concrete Structures Code.

Specification requirements were:-

- Concrete
  - Strength - Grade 20
    Grade 40 was used for dead end prestressing anchors as shown in Figure 87 (Drawing A1- 56289A)
  - Minimum cover - 100mm
  - Maximum coarse aggregate - 75mm in Grade 20; 40mm in Grade 40.
- Reinforcement to AS 1302 - 1977 Cold worked bars Grade 410 (Mpa)

18.5 Road Bridge Cantilever Supports

The road bridge supports are cantilevered upstream from the top of the spillway piers. They are of tapered rectangular section with recesses on each side to support the precast beams of the traffic bridge. Details of the cantilevers are shown in Figure 88 (Drawing A1-56309B). A 75mm diameter conduit passes through the cantilever to the upstream end for future lighting for the road bridge.

The cantilevers are of prestressed concrete designed in accordance with the NAASRA Bridge Design Specification 1976. They are loaded by their own dead load, the dead load of the bridge beams and the prescribed T44 truck load or alternatively the T44 load, Abnormal Vehicle load.

Loads on the cantilever were supported by sixteen - 38mm diameter prestress bars in two layers. The top layer of eight bars was discontinued at two points as required by the moment diagram.

The following maximum stresses were obtained at the support section of the cantilever (assumed to be 0.9 m downstream of the leading edge of the pier support):

Initial Prestress+Dead Load:	Top	3.71 MPa
(Cantilever only)	Bottom	0.01 "
Final Prestress+Dead Load:	Top	0.66 MPa
(Bridge Deck added)	Bottom	2.39 "
Final Prestress+DL+T44 Truck Load:	Top	-1.05 MPa
	Bottom	3.98 "
Final Prestress+DL+Abnormal Vehicle:	Top	-1.95 MPa
	Bottom	4.81 "

Nominal reinforcement consists of 16mm bars at 300 mm centres both ways except in the tension zones above where 16 mm bars at 207 mm centres were provided, and at the anchor zones where bursting steel was provided.

The bridge beam seat was designed as a corbel cantilevered from the main cantilever. The thickness of this assumed corbel increases towards the pier support. In general the nominal 16mm bars at 300 mm centres were sufficient for this corbel but near the end of the cantilever where bursting stresses would also exist an extra layer of 16 mm bars at 200 mm centres was added. It was noted during construction that shrinkage cracks formed in the inside corner of the recess formed for the bridge beam support at the support end of the cantilevers. This occurred in the first two cantilevers constructed, those off piers P3 and P4. It was therefore decided to double the amount of steel at this re-entrant corner to 16mm bars at 150 mm centres in the cantilevers off the other two piers, P1 and P2.

Vertical prestressing was also provided to fix the cantilever to the top of the piers. This consisted of twelve 38 mm prestress bars. The cantilever overlaps the top of the piers by 3475 mm so that concrete stresses calculated on a horizontal section at the junction of the bottom of the cantilever and the top of the pier were quite moderate. Dead ends for the vertical prestressing were embedded in Grade 40 concrete in the body of the pier.

The road bridge support cantilever specifications were as follows:-

- Concrete

Strength	-	Grade 40
Strength at transfer	-	30 MPa
Minimum cover	-	40mm
Maximum coarse aggregate	-	40mm

- Reinforcement to AS 1302 - 1977

Minimum yield stress 'C' bars	-	410 MPa

- Prestressing bar

Type	-	38mm nominal diameter super grade to AS 1313 - 1972
Nominal area	-	$1140mm^2$
Minimum breaking load	-	1230 kN
Modulus of elasticity	-	160 to 180 x 10^3 MPa
Load at lock off	-	900 kN
Prestress force after 13% losses	-	783 kN

18.6 Gate Winch Corbels

The wire rope winches for raising and lowering the radial gates are supported on concrete corbels near the top of the piers and training walls on each side of the gates as shown in Figure 92 (Drawing K1 60160.) The gate lifting points are located on the downstream side near the bottom of the skin plate.

A winch cannot be operated independently of the other winch attached to the gate. When the hydraulic motors driving the winch are not operational, the gate is held in position by the winch brakes. One winch can support the total weight of the gate. The pressure rating of the hydraulic system will not allow the gate to be lifted with one winch. Overload of the winch corbel if the gate becomes jammed, is prevented by pressure relief valves in the hydraulic system driving the winch motor. During operation of the winches the hydraulic pressure keeps the brake open. If the pressure relief valve reduces the pressure in the system, the spring loaded brake is activated and stops movement of the gate.

The total weight of a gate is 90t, and the design load cases for the winches were:-

(a) Hydraulic pump normal operating pressure = 150 Bars ,which is equivalent to a rope tension of 156 kN;

(b) Hydraulic pump maximum operating pressure = 210 Bars, which is equivalent to a rope tension of 218.4 kN. If a gate jams the load cannot rise above this value. This results in a critical load condition; and

(c) If a rope fails the pump will shut down and the load will be supported from one side only, friction neglected. The maximum load occurs when the gate is just open. Under the fully open condition the rope loads are significantly reduced.

A load factor of 1.8 was applied to all these loads. Table 35 shows the loads per unit rope tension at the centroid of the platform section.

TABLE 35
GATE WINCH CORBEL LOADS PER UNIT ROPE TENSION AT CENTROID OF THE PLATFORM SECTION

Load for UNIT rope tension at centroid of platform section	Gate in shut position kN	Gate in raised Position kN
F_x*	1.1433	3.6577
F_y	0.0952	0.1957
F_z	2.9191	1.1745

* where the top of the pier is in the 'xy' plane with the 'z' axis vertical.

Table 36 shows the moments per unit rope tension at the centroid of the platform section.

TABLE 36
GATE WINCH CORBEL MOMENTS PER UNIT ROPE TENSION AT CENTROID OF THE PLATFORM SECTION

Moment for UNIT rope tension at centroid of platform section	Gate in shut position kNm	Gate in raised Position kNm
M_x*	3.8884	1.8921
M_y	6.1038	7.2035
M_z	1.4633	4.6919

* where the top of the pier is in the 'xy' plane with the 'z' axis vertical.

The moment due to the self weight of the concrete corbel was 85 kNm; and the moment due to the mass of a winch ≈ 7 tonne at 0.6 m was 41.4 kNm.

The corbel section was designed for combined bending, shear and torsion with a concrete strength of 40 MPa.

Anchors embedded in each end of the winch corbel allow inspection of the winch using the inspection platform as shown in Figure 93 (Drawing A1-75038.) The maximum load carrying capacity of the inspection platform is two persons.

The design of the winch corbel was in accordance with AS 1480-1974 Concrete Structures Code.

Specification requirements were:-

- Concrete

Strength	-	Grade 40
Minimum cover	-	50mm (upstream face 40mm)
Maximum coarse aggregate	-	40mm

- Reinforcement to AS 1302 - 1977

Minimum yield stress 'C' bars	-	410 MPa

During the construction of the corbel, the contractor decided to drill the holes for the winch holding down bolts, rather than forming the holes. The drilling damaged one of the 2041 DL bars which was repaired. Details are shown in Figure 94 (Drawing A1-79264).

18.7 Trunnion Corbels

The radial gate trunnion corbels support the arms of the radial gates. The water load on the gate is transferred through the gate arms to the trunnion bearings which are supported by the trunnion pedestals, bolted to the trunnion corbels on the downstream end of the pier. Details of the trunnion corbel are shown in Figures 87 and 88 (Drawings A1-56289A and A1-56309B).

The gate loads are resisted by thirty 38mm diameter prestress bars passing through the corbel. The corbel is tied to the pier with nine anchors, each made up of fifty 12.5mm strands terminating at dead ends near the nose of the pier. Anchors vary in length from 17m to 23.5m, and the dead ends are embedded in Grade 40 concrete.

Analysis of the corbel was carried out by finite element methods, using shell elements with 5 degrees of freedom per node.

The corbels were designed in accordance with AS1480-1974 Concrete Structures Code and AS1481-1974 Prestressed Concrete Code.

Gate loads used for the trunnion corbel design are shown in Table 37.

TABLE 37
DESIGN LOADS FOR TRUNNION CORBEL.

Load Case	Gate Load	Water Level
Normal	8250 kN	Top of gate
Extreme	11980 kN	EL 77.0
Ultimate	13840 kN	EL 79.0 (gate overtopped)

Specification requirements were:-

- Concrete

Strength	-	Grade 40
Strength at transfer	-	40 MPa anchorage and stressing ends 20 MPa elsewhere
Minimum cover	-	50mm
Maximum coarse aggregate	-	40mm

- Reinforcement to AS 1302 - 1977

Minimum yield strength 'C' bars	-	410 MPa

- Prestressing tendon

Strand type	-	12.5mm nominal diameter 7 wire super grade stress relieved low relaxation to AS 1311 - 1972
Number of strands	-	50 per tendon
Nominal area	-	4935mm^2 per tendon
Modulus of Elasticity	-	180 to 195 x 10^3 MPa
Minimum breaking load	-	9200 kN per tendon
Load at lock off	-	6900 kN per tendon
Prestress force after 13% loss	-	6000 kN per tendon

- Prestressing bar

Type	-	38mm nominal diameter super grade to AS 1313-1972
Nominal area	-	1140mm^2
Minimum breaking load	-	1230 kN
Modulus of elasticity	-	160 to 180 x 10^3 MPa
Load at lock off	-	900 kN
Prestress force after 17% loss	-	750 kN

19.0 BRIDGES ACROSS SPILLWAY

19.1 Road Bridge

The road bridge carries the two-lane Brisbane Valley Highway and consists of five spans over the 12m wide spillway gate openings. Support for the bridge spans are the left and right bank training walls and stressed cantilevers on the upstream side of the spillway piers. Telecom cables are housed in two 100 mm dia conduits embedded in the upstream walkway. Details of the bridge are shown in Figures 95 and 96 (Drawings A1-50767D and A1-50762C).

The minimum width required by Queensland Transport (Main Roads Department) was 8.5 m between the kerbs, and an additional 700 mm on each side for emergency walkways. Queensland Transport directed that the bridge design loading be the T44 Standard Vehicle Loading in accordance with the NAASRA Bridge Design Specification.

The design for the bridge was in accordance with the following:-

AS 1250-1975 Steel Structures Code
AS1480-1974 Concrete Structures Code
AS1481-1974 Prestressed Concrete Code
NAASRA Bridge Design Specification 1976

The bridge was constructed using prestressed concrete deck units (based on the Queensland Transport standard drawings) separated by a 20 mm mortar gap, and stressed transversly with prestressing bars. Each bridge span has a free end and a fixed end. At the fixed end each unit is held in position by a mortared 24mm diameter anchor bolt. The free end is supported on 300 x 130 x 25 thick neoprene bearing pads with a Shore Durometer Hardness of 70, and upward vertical movement is limited by 24mm diameter anchor bolts mortared into the support.

Each deck unit was prestressed with eighteen 12.5 mm strands some of which were debonded towards the end of the beam to reduce stressing loads. Void formers were placed within the beams to reduce the selfweight.

Specification requirements for the road bridge were are as follows:-

- Concrete

Strength	-	Grade 45
Strength at transfer	-	30 MPa
Minimum cover	-	40mm
Maximum coarse aggregate	-	20mm

- Reinforcement to AS 1302 - 1977

Minimum yield stress 'R' bars	-	230 MPa
Minimum yield stress 'S' bars	-	230 MPa
Minimum yield stress 'C' bars	-	410 MPa

- Prestressing strand

Type	-	12.5mm nominal diameter 7 wire super grade stress relieved low relaxation to AS 1311 -1972.
Nominal area	-	98.7mm^2
Minimum breaking load	-	184 kN
Modulus of elasticity	-	180 to 195 x 10^3 MPa
Load at concrete placement	-	130 kN

- Transverse stressing bar

Type	-	29mm nominal diameter regular grade to AS 1313 - 1972
Nominal area	-	660mm^2
Minimum breaking load	-	680 kN
Modulus of elasticity	-	160 to 180 x 10^3 MPa
Load at lock off	-	350 kN

19.2 Service Bridge

The service bridge spans each of the five 12m wide spillway gate openings, providing access for the track mounted gantry crane and maintenance traffic, as well as supporting a service walkway under the deck. The general arrangement of the bridge is shown in Figure 97 (Drawing A1-50829D).

Over the piers and between the bridge spans, the profile of the girder and deck of the service bridge was designed in reinforced concrete, and attached to the top of the piers with starter bars as shown in Figure 98 (Drawing A1-56256A). At pier No. 2, a cable for the electricity supply for the gantry crane passes through a cable guide hole in the deck.

The service walkway consists of a grid mesh floor supported on steel RHS beams as shown in Figure 99 (Drawing A1-56299B). The design loading for the walkway was 5 kPa.

The service bridge carries the following pipes and cables:-

(a) a 38mm dia air line to the downstream wall of the bulkhead gate storage pit at EL 66.2, with an offtake to the top of monolith RH12 at EL 79.8 and offtakes to each of the radial gate winch platforms on Piers 1 to 4;

(b) a 50mm dia water pipe supplying treated water to two hosecocks in Monolith RH12 adjacent to the air supply points described in (a) above;

(c) Hydraulic pipes for the operation of the radial gate winches as shown in Figure 100 (Drawing K1-60177);

(d) Electrical cables carrying the main power supply across the dam to the right bank permanent office and housing area, sewage pump station, piezometer terminal structures etc. Cables on the service bridge walkway also supply power to the radial gate control consoles and radial gate winches on the piers, and to the bulkhead gate gantry crane. Electrical cableways in the spillway area are shown in Figure 101 (Drawing K1-51418E)

The concrete design for the bridge was in accordance with the following:-

AS 1250 - 1975 Steel Structures Code
AS 1480 - 1974 Concrete Structures Code
AS 1481 - 1974 Prestressed Concrete Code
NAASRA Bridge Design Specification 1976

Each span consists of two prestressed rectangular girders with a cast insitu reinforced concrete deck, supported on elastrometic bridge bearings on levelling pads of "Monolith" non shrink grout. Details of the prestressed girders are shown in Figure 102 (Drawing A1-84960).

The 584 mm x 584 mm x 60 mm thick bearings are based on the "Advance Type V10-57-4", and consist of a natural rubber block with embedded mild steel plates. The bearings with Shore Durometer Hardness of 55 were compression tested to 3000 kN and shear tested to 125 kN in accordance with AS 1523.

Each bridge span has a free end and a fixed end. The fixed end has two 38mm diameter dowel bars embedded in the supporting piers and training wall, and extending up through the bearing into rubber dowel caps set in the lower face of the girder to allow for misalignment and rotation. The free end has two 20 mm diameter dowel pins fixed in the bottom face of the girder, and entering oversized holes in the upper steel plate in the bearing.

The crane rails for the 79 tonne gantry crane are set flush with the deck and positioned centrally over each girder as shown in Figure 103 (Drawing A1-54750B).

Because of access requirements along the service walkway under the deck, the use of solid concrete diaphragms was not practical, and therefore structural steel bracing was used as shown in Figure 104 (Drawing A2-50807W). **This bracing is essential for providing lateral stability to the bridge girders and must not be removed.**

The girders and cast insitu deck are designed for the following load cases:-

- Deck - T44 Standard Vehicle Loading in accordance with the NAASRA Bridge Design Specification. Single lane with 30% impact factor, or twin lane without impact.

- Girder - Gantry crane design loading of 1660 kN on each leg spaced 10.2m apart. Bogies on each leg are spaced 2300mm apart with wheels on each bogie 1000mm apart. The loading is made up of the dead load of the gantry crane supporting the bulkhead gate and with a live load wind velocity of 20 m/sec.

Each precast girder was prestressed with thirty two 12.7mm strands some of which were debonded towards the ends of the girder to reduce the stressing loads. Void formers were positioned within the girder to reduce the selfweight. After positioning the girders the reinforced concrete deck was cast insitu.

Specification requirements for the service bridge construction were:-

- Reinforcement to AS 1302-1977

Minimum yield stress 'S' bars	-	230 MPa
Minimum yield stress 'C' bars	-	410 MPa

- Deck

Concrete strength	-	Grade 25
Minimum cover to reinforcement	-	30mm
Maximum coarse aggregate	-	20mm

- Girders

Concrete		
Strength	-	Grade 40
Strength at transfer	-	25 MPa
Minimum cover	-	30mm
Maximum coarse aggregate	-	20mm
Prestressing strand		
Type	-	12.7mm nominal diameter 7 wire super grade low relaxation to AS 1311-1972
Nominal area	-	100.1mm^2
Minimum breaking load	-	184 kN
Modulus of elasticity	-	180 to 205 x 10^3 MPa
Load at concrete placement	-	147 kN

20.0 INLET STRUCTURE

20.1 Inlet Structure - General

The inlet structure is incorporated in Monoliths LH9 and LH10, details of which are shown in the following figures:-

(a) Monolith LH9
- Figure 105 (Drawing A1-60888A)
- Figure 106 (Drawing A1-60889A)
- Figure 107 (Drawing A1-59942C)
- Figure 108 (Drawing A1-54613B)

(b) Monolith LH10
- Figure 109 (Drawing A1-60903A)
- Figure 110 (Drawing A1-60904B)
- Figure 111 (Drawing A1-56120C)
- Figure 112 (Drawing A1-54614E)

The function of the Inlet Structure is to provide an inlet to the penstock and river outlets and to allow withdrawal of water from selected levels between the Minimum Operating level of EL 33.2 and the potential full supply level of EL 68.5, and to provide protection for the outlet valves and a possible future hydro-electric turbine by the installation of trashracks.

The provision of a free standing inlet tower, which is common in earth and rockfill dams, was avoided by recessing the inlet structure into the spillway training and retaining walls of Monoliths LH9 and LH10.

Four bays of screened inlets in the plane of the spillway wall are located in Monolith 9 extending from EL 33.2 to the top of the structure at EL 79.0. Transverse and longitudinal beams 600 mm x 600 mm supporting the trashracks slot columns are provided at 6.3 m centres vertically. Sixty individual trashracks 3.04 m high by 3.064 m wide with a mesh opening of 88 mm are installed in two slots. One spare trashrack is stored on the storage floor at EL 71.0. A second slot, in front of the trashrack slot, is provided in the columns for the trashrack rake and for stoplog installation if needed. The general arrangement of the columns, beams and trashracks is shown in Figure 106.

Four precast concrete waterproof hatch covers are installed over each bay forming the roof of Monolith LH9 at EL 79.15, which is at the same level as the top of the embankment and accessible to vehicles. Guard rails are installed on the streamside edge of Monolith LH9. Details are shown in Figure 105.

Crane rails for the 3 tonne gantry crane are also installed on the roof of Monolith LH9. The gantry crane which services Monoliths LH9 and LH10 is designed for:

(i) installation and removal of trashracks, and handling the trashrack rake;
(ii) lifting hatch covers;
(iii) removal and installation of equipment for maintenance purposes and general lifting duties spanning Monoliths LH9 and LH10.

A local walkway at EL 76.0 overlooks the inlet structure behind the trashracks and the selective baulk slots. A local control panel for operating the baulk hoists is located on the walkway. The storage floor, for storage and maintenance of trashracks and baulks is constructed at EL 71.0.

In Monolith LH10 the streamside walls are supported by three horizontal rows of transverse beams at 6.3 m centres vertically, except that the downstream beam at EL 39.33 has been omitted to improve stream flow into the penstock and river outlet, as shown in Figure 112 (Drawing A1-54614E). Selective baulks, as described in Section 21, are installed in Monolith LH10.

20.2 Design Considerations for Monoliths LH9 and LH10

Because of joints in the rock dipping into the spillway in the vicinity of Monoliths LH7 and LH13, rock was removed from unstable areas and prestressed rock anchors installed as shown in Figures 66 and 67 (Drawings A1-56087A and A1-56088A). The removal of rock resulted in increased concrete volumes above those allowed for in the initial design. In the design of the concrete monoliths it was assumed that the rock would help support the concrete structures. Hence the rock had not only to support its own weight but also had to offer restraint for the adjoining concrete structure. This restraint was offered by grouted anchor bars as shown in Figures 113 and 62 (Drawings A1-50788D and A1-54537C). In order to enable free drainage of the rock and to reduce the water loads between the concrete structures and the adjoining rock, weep holes were provided through the concrete at 2 m centres both vertically and horizontally.

The inlet structure components were designed to collapse under the following differential heads:-

Component	Differential Head
Collapsible baulk - Tension pin failure	1.0 m
Trashracks in Monolith LH9	1.5 m
Non collapsible baulks	2.0 m
Monolith LH9	3.0 m
Monolith LH10	3.0 m

Monolith LH9 and LH10 were designed by the "Equivalent Frame Method" of Section 21 of the Concrete Code - AS 1480. This method consisted of analysing horizontal and vertical frames. To simplify the design of Monolith LH9, horizontal props were placed at the intersection of each horizontal and vertical trashrack supporting member, and these props extended to the rear wall at 6.33 m vertical intervals.

In addition to hydrostatic loads, Monoliths LH9 and LH10 were designed for the following loadings:-

(a) Embankment loads.

The crest of the left hand embankment terminates against monoliths LH9 to LH11. As the vertical rock wall of the spillway excavation extends to approximately EL 62.0 only and the crest of the embankment is at EL 79.0 embankment loads are exerted on Monoliths LH9 and LH10 above EL 62. These monoliths were originally designed for an embankment with a sloping core where the core material (Zone 1A) would have terminated across several monoliths. As this could lead to cut off problems it was decided to gradually steepen up and thicken the clay core so that it would terminate wholly against Monolith LH10. To resist these embankment loads, gravity walls were designed for the top of the monoliths above the vertical rock wall level. These gravity walls were cast monolithically with the rest of the monolith.

(b) Gantry Crane Loads

The safe working load of the gantry crane is 3 tonne. The gantry crane rails extend over the length of LH9 and for 9.115 m over LH10. The crane loads and configuration taken into account in the design were:-

(i) the gantry crane was supported on four wheels.
(ii) the dynamic load per wheel was not to exceed 15.0 tonne.
(iii) the lateral load per wheel was not to exceed 7.0 tonnes.
(iv) the longitudinal force on each rail was not to exceed 7.0 tonnes.
(v) 86 kg/m rail section was to be used.

(c) Traffic Loads on the Roof

The precast concrete waterproof hatch covers forming the roof of Monoliths LH9 and LH10 at EL 79.15 were designed to the Standard Abnormal Vehicle Loading of the NAASRA Bridge Design Specification with an impact factor of 10% and a reduced load factor as applicable for this loading.

(d) Loads on Storage Floor at EL 71.0

The storage floor at EL 71.0 was designed for a 70kPa live load resulting from stacking the baulks and trashracks.

(e) Log Impact

Log impact forces were considered. When water is flowing over the spillway it is unlikely that a log will strike the trashracks or vertical columns of the inlet structure. When water is below spillway crest level and flow normal to the trashracks, the trashracks can withstand impact from a log of 4800 kg mass when the trashracks are clogged and approach velocity reduced.

(f) Walkways

The walkways were designed for a 4 kPa live load.

20.3 Trashracks

The trashracks are the conventional type with 12 mm vertical bars at 100 mm centres (giving an opening of 88 mm) supported by a RHS steel frame. The spacing of the bars was determined by the following factors:-

(i) head loss across the trashracks;
(ii) protection of the future hydro-electric power station turbine;
(iii) the lifting capacity of the gantry crane; and
(iv) vibration from vortex shedding and from the future turbine.

The trashracks were designed for a head of 1.5 m. Fishing gear was provided for raising and lowering the trashracks, and a cleaning rake which cleans on the way down was provided. Details of the trashracks and fishing gear are shown in Figures 114 and 115 (Drawings A1-50774 and A1-50772).

21.0 WATER QUALITY PROVISIONS

21.1 Water Quality Provisions - General

The selection and utilization of the best quality water available is ensured by the provision of water quality sampling offtakes and selective baulks in the inlet structure.

21.2 Water Quality Sampling

To enable the quality of water at different levels in the reservoir to be determined, eleven water sampling offtake points were installed in Monolith LH9 between EL 40.0 and EL 65.0. These offtakes are fitted with strainers and are located just upstream of the trashracks as shown in Figure 106 (Drawing A1-60889A). Because the water released back into the river for the Brisbane water supply will be naturally aerated with consequent improvement in quality, it was considered that offtakes at 3 m intervals would be adequate, and the offtakes beteen EL 40.0 and EL 55.0 were therefore spaced at 3 m intervals. As the better quality water was expected to be at the top of the reservoir the upper six offtakes between EL 55.0 and EL 65.0 were spaced at 2 m intervals to ensure that the optimum quality water could be selected for release. The lowest intake shown at EL 37.0 is connected to a pressure switch which prevents start up of any pump which is above water level at the time.

Each of the sampling intakes is connected by a flexible coupling to a 100mm dia Class 12, rigid PVC pipe which contains a three phase 4 inch multistage submersible pump as shown in Figures 116 and 117 (Drawings A0-70655 and A0-71687). The pumps are all connected to a single 38 mm dia stainless steel tube which delivers the sampled water to the water quality control room via the route shown in Figure 117.

Because of the possibility in the future of supplying water to other towns, water can also be sampled from connections to the 3600 mm dia penstock and the 1905 mm river outlet in Monolith LH11. These 38 mm stainless steel tubes are connected to pumps in the lower gallery which deliver the sampled water to the water quality control room. The controls for the sampling pumps are situated in the Water Quality Control room.

21.3 Selective Baulks

In Monolith LH10, intermediate between the trashracks and the entrances to the penstock and river outlet, provision is made for the installation of selective baulks between EL 33.175 and EL 71.0 as shown in Figures 110 and 112 (Drawings A1-60904B and A1-54614E).

There are six individual baulks and their positioning within their individual vertical slots enables water of selected quality to be drawn off from any interval over the operating range of the storage. If sufficient waterway has not been provided between the baulks, a microprocessor in the Water Quality Control Panel prevents opening of the outlet valves.

The set comprises five standard baulks as shown in Figure 118 (Drawing A1-50800C) and one collapsible baulk as shown in Figure 119 (Drawing A1-50824E). Each baulk measures 6.28 m wide by 6.3 m high and is permanently attached to and is raised and lowered by its individual winch driven by a 3HP “Cyclo Drive” geared brake motor assembly. The winches are mounted at EL 78.05, underneath the crane rails for the 3 tonne gantry crane, the gantry crane rails being supported by steel columns and beams in this area. Each baulk slot is covered by individual precast concrete waterproof hatch covers at EL 79.15.

A standard baulk consists of an 8mm skinplate supported over horizontal members of a steel frame as shown in Figure 118. A baulk comprises two horizontal sections hinged in the middle and was designed for a differential head of 2 m. The lowest baulk is designed as a collapsible baulk, consisting of a RHS frame supporting two vertical doors held closed by a tension pin. The tension pin is designed to fail at a load of 30.53 kN which corresponds to a head differential of 1 m.

When baulks are not in use or are being serviced they are located on the storage floor at EL 71.0.

A standard baulk consists of an 8mm skinplate supported over horizontal members of a steel frame as shown in Figure 118. A baulk comprises two horizontal sections hinged in the middle and was designed for a differential head of 2 m. The lowest baulk is designed as a collapsible baulk, consisting of a RHS frame supporting two vertical doors held closed by a tension pin. The tension pin is designed to fail at a load of 30.53 kN which corresponds to a head differential of 1 m.

When baulks are not in use or are being serviced they are located on the storage floor at EL 71.0.

22.0 OUTLET WORKS

22.1 Outlet Works - General

The outlet works extend over 4 monoliths LH11 to LH14 with the entrances to the penstock and river outlet being in Monolith LH11 and the regulating valves in Monolith LH14. At the entrance to the outlet works in Monolith LH11 a 3.6 m dia penstock with a large capacity intake was installed to meet the demands of a proposed 30MW hydro-electric turbine, and a 1.905m dia river outlet was installed directly above the penstock so that one fixed wheel bulkhead gate could command either outlet (but not both outlets) to provide for emergency closure or dewatering. Entrances to these outlets were installed at the end of the chamber recessed into the spillway wall for the intake structure, and the outlets were then turned under the spillway crest and concreted into a trench in the spillway diversion channel. Following closure of the main diversion channel on the right bank of the Brisbane River floods could be diverted through the low spillway crest block S1 over the concrete encased outlets. During construction of the spillway crest block S1, flows were diverted through the 3.6 m dia penstock.

The arrangement of the outlet pipes is shown in Figure 120 (Drawing A1-56160D) and the diversion stage is shown in Figures 121 and 122 (Drawings A1-54697A and A1-54698A).

22.2 Penstock and Outlet Pipes

Two independent outlet systems have been provided, as shown in Figure 120. The lower outlet consists of a 3.6 m dia. penstock for the possible future hydro-electric station. The penstock has been sealed off at its downstream end with a semi-ellipsoidal dome, and the recess housing the dome has been filled with 5MPa concrete. A 1.5 m offtake from this penstock provides one of the river outlets and is controlled by a 1.5 m dia. stainless steel regulating valve.

The upper outlet, consisting of a 1.9 m dia. pipe, is located vertically above the 3.6 m dia. penstock. This arrangement had the advantage that only one penstock gate and one set of gate guides were required. This outlet is also controlled by a 1.5 m dia. regulating valve. Each regulating valve discharges into a stainless steel lined dispersion chamber. Additional offtakes are provided for town water supplies and possible local urban development.

The penstock and outlet pipe wall thicknesses were determined for normal and extreme load cases, assuming that no support was provided by the surrounding concrete.

The maximum permissible stress for the normal load case was 0.66 F_y, and because of the presence of the concrete the maximum permissible stress for the extreme load case was 0.85 F_y.

Actual stresses in the pipe wall were determined by combining the circumferential, axial, and temperature stress effects.

Details of the load cases are as shown in Table 38.

TABLE 38 - DESIGN LOAD CASES FOR PENSTOCK AND OUTLET PIPE

Load Case	Design Pressure	Penstock	Outlet Pipe
Normal	890 kPa	Static Head + water hammer from turbine close down in 4.5 sec.	Static Head + water hammer from valve closure in 1.1 sec.
Extreme	1780 kPa	Static Head + water hammer from turbine close down in 1.5 sec.	Static head +water hammer from valve closure in 0.7 sec.

Because of the high design velocity for the 3.6 m penstock (10.0 m/s) it was necessary to line the entrance rather than form it in concrete. A mild steel transition ~~section~~ was designed to take the flow from the 5 m by 3 m rectangular entrance to the 3.6 m dia. penstock. The geometry of the transition is such as to give a smooth transition and to accommodate most of the flow from the top of the entrance. Because of the flat sections in the transition, substantial stiffeners and anchors to take the external forces into the surrounding concrete were required. Grout holes were provided in the invert. Details are shown in Figure 123 (Drawing A1-50760E)

The penstock intake transition was designed for an external pressure of 470 kPa (Static head for PMF - EL 77.0) decreasing to 345 kPa in four equal increments at each circumferential stiffening ring.

Each outlet pipe was provided with an air vent, 750 mm dia. for the 3.6 m penstock and 375 mm dia. for the 1.9 m outlet pipe. These vents are vented to the atmosphere at EL 79.0 on the roof of Monolith LH11. The size of the air vent pipes was determined using the method given in Reference 29.

The internal surfaces of the outlet pipes were coated with coal tar epoxy to a minimum thickness of 500 microns.

22.3 Fixed Wheel Penstock Gate

22.3.1 Fixed Wheel Penstock Gate - General

A fixed wheel penstock gate 5.25 m by 4.1 m wide was provided to close off either the 1.9 m dia. outlet or the 3.6 m dia. penstock. Simultaneous closure of both outlets was not considered to be essential.

The fixed wheel penstock gate has been designed as a guard gate to enable inspection and maintenance work to be carried out on the outlet and penstock pipes, and for emergency closure of the penstock pipe. It was not intended that the gate be used when flood storage was above full supply level.

The design operating conditions are:

- Normal operation - Maximum water level in the storage at Full Supply Level, EL 67.0;
- Emergency operation - Closure of the penstock with full turbine flow.

The gate is articulated in 3 sections, each 1.75 m high joined by hinges consisting of over-lapping plates with 100 mm diameter stainless steel pins. If the gate was in one section it would have required extremely close tolerances in setting the wheels and gate track to ensure uniform loads on the track from each wheel.

The gate wheels are 500 mm dia. with 120 mm treads hardened to 255 B.H.N. to a depth of 10 mm. The design wheel load, resulting from hydrostatic pressure, is 510 kN, when the storage is at Full Supply Level, EL 67.0.

When the gate is to be used to close off the 1.9 m outlet pipe only, two protruding arms are positioned on the gate to engage supports set in the concrete face above the outlet.

Figure 124 (Drawing A1-56136) shows details of the assembled gate.

22.3.2 Fixed Wheel Penstock Gate Model Studies

Flow patterns in the intake area and around the bottom of the gate were investigated in a laboratory flume. A flat-bottomed gate shape was initially investigated, and as predicted, unstable eddies formed at the bottom of the gate. The predicted frequency of vortex shedding varied from 1.5 to 10 Hz depending on the gate opening and as the natural frequency of vibration of the gate suspension system was calculated at 1.0 to 2.0 Hz, self-controlling vibration of the gate was predicted.

Various gate shapes were investigated and a 45° bottom shape was adopted as the most satisfactory to avoid separation. The separation was accentuated by the direction of flow into the intake, most of which approaches the intake vertically.

Flow nets were drawn from the flow pattern observations to enable calculations of pressures on the top and bottom of the gate to determine the hydraulic downpull on the gate. A maximum downpull of 629 kN was calculated for the case of closure against maximum turbine flow.

Downpull and vibration of the gate were further investigated in a 1:40 model of part of the intake and penstock. Both the original gate shape and the improved shape were tested and downpull measurements agreed reasonably well with the two-dimensional flow pattern calculations.

Both model gates were operated over the whole range of positions to investigate vibration possibilities. The model suspension system had an appropriately scaled stiffness to reasonably reproduce prototype behaviour. Violent vertical vibrations occurred with the original gate shape at large openings as predicted by the two-dimensional flow pattern and mathematical analysis. The improved gate shape did not vibrate in the vertical direction.

With turbine flow through the penstock and the bulkhead gate passing over the 1.9 m dia. outlet, horizontal vibration of each gate occurred with the gate lifting off its track. Detailed measurements of velocities on the front of the gate during lowering were carried out in conjunction with pressure readings at the back of the gate. The maximum horizontal force on the gate for turbine flow was calculated at 60 kN. Further flow visualisation studies indicated that the horizontal load was generated by separation of the flow from the top and bottom edges of the gate. Pressure tappings were installed in the gate and subsequent measurements substantiated this.

The horizontal vibration was greatly reduced in the model by reducing the clearance of the wheels in the slots to an equivalent of 5 mm in the prototype. This was achieved in the fixed wheel gate by providing two rails, one upstream and one downstream of the wheels with clearances limited to this 5 mm. Details of the track and seal plates are shown in Figure 125 (Drawing A1-54540E). In the model, once the horizontal force developed, the gate generally ran on the upstream rail instead of the downstream rail.

22.3.3 Storage of Gate

When not in use the gate is stored on the storage frame in the gate storage chamber at EL 71.0 in Monolith LH11, as shown in Figure 126 (Drawing A0-51450A). The bulkhead gate sections are lifted by the lifting frames shown in Figure 127 (Drawing A1-62314), and the 80 tonne hydraulically operated hoist arrangement is also shown in Figure 126.

22.4 Regulating Valves

22.4.1 Regulating Valves - General

The arrangement of the regulating valves and dispersion chambers is shown in Figure 128 (Drawing K1-50201), and the general arrangement of a valve is shown in Figure 129 (Drawing K1-69223). Figure 128 may also be referred to for an overall view of the outlet valve arrangement.

The two stainless steel 1500 mm nominal bore cone dispersion regulating valves are installed at EL 31.50 and are designed to close tight under a maximum static head of 75m. The weight of each valve is 4.97 tonnes.

The valves are operated by a pair of hydraulic cylinders controlled by a single motor driven hydraulic pump. The valves can be operated locally from the valve control cubicle shown in Figure 128 or remotely from the underground control complex or from the area office. Each location has a position indicating station and control can be taken over at any of the three locations.

Waterproof hatches, 2.3m square, were provided on the floor at EL 34.0 for access for installation, removal or maintenance of the valves. A smaller waterproof hatch, 1500mm x 650mm provides access to the air chamber between the valves. The 6 tonne monorail hoist arrangement for handling the valves is shown in Figure 130 (Drawing K1-51408B).

22.4.2 Inspection and Testing of Valves

The valves were fabricated and tested in accordance with AS1210 - Unfired Pressure Vessels for Class 1 Pressure Vessels.

Castings for the valves were ultrasonically examined and where faults were suspected, the suspected areas were radiographed in accordance with ASTM E446-72 or ASTM E186-186-73 depending on the thickness of the casting. After machining all stainless steel components were passivated.

The assembled valves were hydrostatically tested with the sleeve in the closed position at 1800 kPa for 15 minutes and the pressure then lowered to 1500 kPa until all welds were inspected. Working pressure is 730 kPa.

22.5 Dispersion Chambers

To dissipate the energy of flow from the outlet valves, dispersion chambers were provided immediately downstream of the valves.

The dispersion chambers are lined with stainless steel liners 4.88 m long by 3.824 m wide and 3.8 m high. The liners were fabricated from 12 mm thick stainless steel plate conforming to ASTM A240 type 316L and anchored into the concrete with mild steel anchor bars. The liners were fabricated in accordance with AS 1250 and welding conformed to AS 1554.

Stainless steel step irons are welded to the inside of the liners, giving access to and from the maintenance platform at EL 31.0 adjoining the outlet valves. Two rows, each of eleven vibration ports 150 mm dia, were provided in the invert for consolidation of concrete under the liners, and three rows, each of 11 screwed holes of 38 mm dia, were also provided in the invert for grouting the interface between the liner and the concrete invert. The vibration ports were sealed with welded stainless steel closure discs, and the grout holes were sealed with stainless steel plugs screwed into the grout holes, welded up and ground flush. Each liner is drained with a 60.3 mm O.D. stainless steel tube.

Details of the liners are shown in Figure 131 (Drawing A1-54654).

23.0 RADIAL GATES

23.1 Radial Gates - General

The design of the Wivenhoe Radial Gates is fully covered in the "Report on Wivenhoe Dam - Design of Crest Control Structures" prepared by Senior Engineer P.H. Allen of the Engineering Services Group of the Water Resources of the Department of Primary Industries (Reference 30).

Where possible this section will give a broad outline and general summary of the design of the gates and their components and alternatives considered in the design. Where it is not possible to precis a section of the report without detracting from its integrity or logic the section has been reproduced in full. Lengthy tabulations have not been reproduced but are referred to in the original report.

23.2 Arrangement of Gates

The radial gates are required for the purpose of storing water above the spillway crest level and for controlling the discharge of flood waters over the spillway at Wivenhoe Dam. The gates will normally be in the closed position resting on their seal plates. The gates were designed for the FSL of 68.5 m AHD although the FSL for the efficient operation of Wivenhoe Dam Hydro-Electric Power Station is EL 67.0 m.

There are five gates each 12 m wide by 16.5 m high set between concrete piers except that gates 1 and 5 are set between a concrete pier and adjoining spillway retaining wall. The gates are numbered 1 to 5 from the left hand side of the spillway. An isometric view of the spillway and gates is shown in Figure 132 (Drawing K1-56530A), and a general arrangement of a spillway gate is shown in Figure 133 (Drawing A1-61555D).

The gates will be raised and lowered using hydraulically operated winches located on concrete bases cantilevered off the piers and adjacent retaining walls.

23.3 Determination of Principal Dimensions

The spillway was shaped as an "Ogee" section using "Design of Small Dams" (Reference 27), with the crest level at EL 57.0 and an uncontrolled head of 15 m. The radial gate was then located so that the lower nappe of the profile of the jet issuing under the gate at the design discharge of 5000 m^3/s and a head of 20 m matched the ogee profile as closely as possible. This resulted in the crest seal plate being located 4.0 m downstream of the spillway crest.

The gate trunnion was then located using a radius of 16 m and the requirement that the trunnion be out of water during the passage of the Probable Maximum Flood. The 16 m radius was chosen as being approximately equal to the design height of the gate, based on the practice of the United States Corps of Engineers.

23.4 Model Studies

As described in Section 14.5 model studies on the spillway were conducted by the Queensland Water Resources Commission in 1979. These studies confirmed the location of the trunnions and confirmed that pressures measured along the spillway profile were within acceptable levels. The model studies were also used to produce discharge rating curves for flows under and over the gates. Flow profiles over the top of the closed gates were also recorded to allow log impact to be taken into account in the design of the gate arms. Recommended gate operating rules were also developed in the model studies. These were aimed at keeping the flow in the flip bucket and downstream dissipation pool balanced and free from flow concentrations which could erode deep scour holes in the plunge pool. Full details of model studies are recorded in Reference 23.

23.5 Configuration of a Gate

Different configurations of the overall gate structure were considered during the preliminary design. These arrangements included:

(a) a conventional three girder radial arm design;

(b) a conventional four girder radial arm design; and

(c) a design based on horizontal box girders with the upstream flange of the box acting as the skinplate.

Both (b) and (c) produced designs that were heavier and more complex to fabricate and assemble than (a) and therefore a design based on (a) was selected for final design.

A single arm gate was not considered due to the extreme height of the gate and the great variation in location of the resultant water loads produced by the necessity to provide for the overtopping of the gate.

In laying out the radial gates it was decided to opt for lifting the gate from the downstream side using multiple wire ropes. In doing this, the advantages of lifting the gate from the upstream side were compared with the advantages of downstream lifting.

Advantages of downstream lifting included:-

- The reduction of maintenance problems on ropes and lifting gear intermittently immersed in water in a sub-tropical climate;
- The possibility of failure of an underwater connection does not occur and the repair of a failure may not require the use of bulkhead gate;
- For debris to foul the lifting ropes overtopping of the gate has to occur.

The disadvantages of downstream lifting are:-

- The lifting gear cannot be incorporated into the spillway service bridge and it must be placed on a special winch platform inside the arc of the skinplate radius. The torsional loads on this winch platform are high and a heavily reinforced section is required;

- With the reduced radius of lifting, increased rope forces result requiring higher capacity ropes and hoisting equipment; and

- The attachment of the anchorage point requires special treatment rather than being transferred directly into the skinplate as in the case of upstream lifting.

The design itself was relatively conventional and followed the general design philosophy enunciated in the US Corps of Engineers' Manual, on the Design of Spillway Tainter Gates (Reference 31).

The principal elements of the gate structure are the skinplate assembly, the horizontal girders, the radial girders or end frames and the trunnion hubs, bearings, pins and pedestals.

Where possible, interaction between these components was investigated to determine combined effects.

Generally the magnitude of loadings was such that proprietary sections were inadequate and built up sections were required. Overall costs were generally reduced by minimising welding and fabrication while accepting marginally greater material costs.

Gate vibration was not considered a problem as cases reported in the literature required either the tailwaters to be built up behind the gate or the centre of curvature to be offset from the trunnion pin centreline before vibrations became a problem (References 32 and 33). Here with a free discharging flip bucket downstream of the gate and the trunnion pin and centre of curvature coinciding, this problem does not occur.

Lateral vibrations are damped by the side seals and load bearing stiffeners beneath the horizontal girders.

23.6 Loading Conditions and Design Stresses

Three separate loadings were considered in the overall design of the radial gates.

Condition 1 - Water to EL 73.0, Gate Shut

EL 73.0 corresponds to the top of the gates with the gate shut. Even though it is 4.5 metres above the design Full Supply Level and will only be reached during flood mitigation operations it was considered frequent enough to act as the normal design case.

Condition 2 - Water to EL 77.0, Gate Shut

EL 77.0 is the maximum expected top water level in the dam. It may be produced by the gate controlled discharge of the spillway design flood of 5000 m^3/sec or by the uncontrolled discharge of the Probable Maximum Flood discharge of 11,700 m^3/sec. With one gate shut and inoperable, the controllers of the dam would still try to limit the rise in the reservoir to EL 77.0 or lower.

With up to 4.0 metres of overtopping on a shut gate, this loading condition was considered an emergency case and increased allowable stresses were permitted. This loading case could be compared to wind loading allowances in building design.

Condition 3 - Water to EL 79.0, Gate Shut

EL 79.0 is the nominal embankment crest level of the dam and as such it was never to be exceeded. This water level may be reached during the passage of the Probable Maximum Flood with one gate shut. When the safety of the dam is considered it would be preferable for the gate to fail rather than the embankment, so stresses up to the yield point were permitted in this case in the expectation that gate failure will occur before the embankment is breached.

In general, the loadings produced by the above loading conditions were added to the dead loads and multiplied by factors to reduce them to a common base for the determination of design cases and to enable standard code stresses to be used in the design of components.

The figures generally adopted were:

(i) EL 73.0 - Dead load (X_d) + Live load (X_l)

(ii) EL 77.0 - $0.75*(X_d + X_l)$

(iii) EL 79.0 - $0.625*(X_d + X_l)$

An additional factor of 1.20 was added as a factor for the importance of the structure and as a form of corrosion allowance. This is consistent with the US Corps of Engineers' approach (Reference 31) of adopting allowable stresses of 0.50 F_y as against the AS 1250 approach of typically allowing 0.60 F_y.

Therefore the above loads were increased to:

(a) EL 73.0 - $1.20*(X_d + X_l)$

(b) EL 77.0 - $0.90*(X_d + X_l)$

(c) EL 79.0 - $0.75*(X_d + X_l)$

and the allowable stresses of AS 1250-1972 were applied to the design.

Further details of loading conditions and design stresses are given in Section 2.3 of Reference 30.

23.7 Skinplate Design

The internal radius of the skinplate was set at 16 metres using the criteria of the radius of curvature of the gate being approximately equal to the height of the gate. This left the only variables in the skinplate design as the spacing of the ribs and the location of the horizontal girders.

The number and spacing of the ribs were examined on the basis of cost. Costs of 18 and 22 ribs were considered. The cost comparisons were inconclusive and the choice of the 18 rib configuration was influenced by the cost of lifting gear favouring the lighter gate. Any reduction below 18 ribs was not beneficial as it would increase the skinplate thickness from 12 mm to 20 mm.

Two finite element models, using the SAP IV finite element package, were set up to analyse the action of the skinplate and the horizontal girders for the three conditions of water loadings to EL 73 m, EL 77 m and EL 79 m combined with support at the anchorage points or the crests. The arrangement of the models are shown in Figures 134 and 135. The general purpose of the analysis was to look at the combined action of the skinplate and horizontal girders. As the loadings and deflections in the gate were expected to be symmetrical about the centreline of the gate only one half of the gate was modelled in each case. Other data to come out of the model included the horizontal girder loadings and the effective width of skinplate supported by the arms.

The horizontal girder spacings were laid out in conjunction with a two dimensional analysis which optimised the total weight of the skinplate for a given number of ribs.

Envelopes of bending moments about the trunnion axis, varying linearly from the bottom to the top of the gate, were determined for the three loading conditions of water levels at EL 73 m, 77 m and 79 m and for the two support criteria, i.e. supported on the crest or supported by the winches. These bending moment envelopes included allowances for pressure relief during overtopping, and when combined with the bending moments due to hydrostatic loading in the opposite direction, they produced the design loadings for the skinplate.

Full details of these analyses and a comparison of the combined action of the skinplate and horizontal girders at Coolmunda Dam and Wivenhoe Dam are given in Sections 2.4.2 and 2.4.3 of Reference 30.

In the case of the Wivenhoe gates no attempt was made to model any slip that might occur between the ribs and girders as the joints at Wivenhoe were to be packed with epoxy grout after erection to effectively fix the joint against slippage.

Each skinplate anchorage point provides sufficient capacity to hold the full weight of the skinplate should cable failure occur on the other side of the gate, as the winches were designed to be capable of only holding the gate and not raising it from one side only. By placing the anchorage point between the second and third vertical ribs from the skinplate edge the maximum angle between the cable and the sheaves is limited to less than 3° except for the gate in the fully raised position.

Further details of the anchorage points and sheaves are given in Section 2.4.4 of Reference 30.

23.8 Horizontal Girder Design

23.8.1 Horizontal Girder Design - General

The horizontal girders were designed as simply supported members, at the arm girder connections, that were uniformly loaded on the upstream skinplate connection. To reduce the maximum moment and shear force requirements the arm girder connection was positioned at the 1/5 th point of the girders. The typical arrangement of the middle horizontal girder is shown in Figure 136 (Drawing A1-60908A). The top and bottom girders were similar.

The high axial loads in the radial arms were transferred through to the girders and skinplate sections by bolted connections and the addition of stiffeners to the horizontal girders as a continuation of the radial arm flanges, terminating behind the fourth and fifth vertical ribs from the edge of the skinplate. To cater for this continuation of the flanges of the radial arm girders, local widening of the girder flanges was provided. Transverse butt welds were used to join the locally widened plates to the adjoining flanges but these were kept away from the highly stressed regions adjacent to the connection by positioning them close to the point of contraflexure.

23.8.2 Design Loadings

Design unit loads were based on the boundary element loadings produced in the finite elements models analysing the left half of the gate, and the critical design load case was determined using equations (a) to (c) of Section 23.6. These loads plus moments and shears calculated for the three load conditions of water level at EL 73, EL 77 and EL 79 are tabulated in Section 2.5.3 of Reference 30. The final design was determined using the design loads in this tabulation and the allowable stresses from AS 1250-1975.

23.8.3 Web Openings

The only complication to the design was to provide web openings of sufficient size to permit human access through the girders. Calculations to determine if web reinforcement was required after cutting the holes was carried out based on a paper "Design of Beams with Web Openings" (Reference 34). Full details of these calculations are given in Section 2.5.6 of Reference 30. These calculations showed that no reinforcement was necessary around the holes cut in the horizontal girders.

Further details on assumptions and calculations made in the design of the horizontal girders is given in Section 2.5 of Reference 30.

23.9 Radial Arm and Bracing Design

23.9.1 Geometry

The geometry of the gate arms was fixed by the location of the fifth points of the horizontal girders and the need to supply satisfactory clearance to the winch platform. A 250 mm offset of the bearing centreline from the untapered face of the pier produced a plan clearance of approximately 250 mm between the lower downstream corner of the winch platform concrete and the gate handrails. Details of the radial girders are shown in Figure 137 (Drawing A1-60902A).

The major axes of the radial girder were placed in a vertical plane and it was braced for rotation about the trunnion axis by the bracing system shown in Figure 133 (Drawing A1-61555D).

23.9.2 Design Loadings

Design loadings for the arm and bracing systems were determined for eleven load cases as listed in Section 2.6.2 of Reference 30. The critical loads, determined using a two dimensional SAP IV analysis of a half gate section, varied slightly with the chosen section geometry as a result of the relative ratios of the bending moments to axial loads. The design was based on Section 8.2 of AS 1250-1975 using the design loads determined by equations (a) to (c) of Section 23.6 of this report.

23.9.3 Effective Lengths

The effective lengths of the arm units about their minor axes were assumed to be the spans between the braces, and this was later checked using an "s and c" function type analysis as described by Horne and Merchant (Reference 35). This analysis showed that even for bracing consisting of 200 UB 25s, the smallest commercially available universal beam, there was sufficient stiffness to ensure Le = 0.82 span. Thus the assumption was conservative but not overly detrimental as the critical length dimesion was the effective length in the plane of the frame which controlled the design.

With the bearing end pinned and the other end effectively fixed to the horizontal girder and sidesway prevented by the side seals and rollers the effective lengths in the plane of the frame were assumed to be 0.85 L.

The effectiveness of the 'fixed' joint to the horizontal girder was checked as a byproduct of tolerance calculations, to determine the degree of fixity of the bolted joint. The SAP IV finite element program was applied to the middle arm to girder connection, and the bolt forces calculated and used to determine the stiffness of the joint. Figure E1 of Appendix E of AS 1250 was then used to determine the effective length of the arm units as 0.77 L.

23.9.4 Alternative Designs

Checks were carried out into the possibility of using box girders for the radial arms. Although they showed a slight weight advantage over the I section arms they were not adopted because:-

(a) even with a mass reduction of approximately 0.5 tonne per frame, the weight of each arm unit was not within the capacity of the bulkhead gate gantry crane to lift in one piece; and
(b) it was more difficult to provide satisfactory arm-girder and arm-trunnion connections and tolerance requirement would need to be tighter.

The possiblity of using higher strength steel in the arms was considered, and whilst a weight saving would have been achieved, the fabrication costs would have been higher, and the higher strength steel could not be justified on the basis of weight reduction alone.

23.9.5 Log Impact

With the design of the gate for possible overtopping it was necessary to design some impact resistance into the top horizontal girder. The spillway hydraulic studies produced upper and lower nappe profiles for water levels at EL 77.0 and EL 79.0. Whilst EL 79.0 is the worst loading case for the top radial arm it is an ultimate load case and as such it was considered unrealistic to add log impact to the load. EL 77.0 was therefore the loading condition to which log impact was added.

At a water level of EL 77.0 with the gate shut, the water impacts on the downstream half of the top radial arm as shown in Figure 138 (Drawing A4-56847). The zone of impact moves upstream with water level decreasing, but then the loadings on the arms are also reducing and the reserve capacity of the arm is greater. It was therefore only necessary to reinforce the downstream portion of the arms. With the arms being I sections the worst effect of log impact would be denting of the flanges which could cause buckling. A 16 mm cover plate was used to close the top section of the I beam and stiffen the impact area.

23.9.6 Radial Arm - Horizontal Girder Joints

The joints between the radial arms and the horizontal girders were specified to be tight. The fabricators were unable to meet this requirement. The effect of the gap was to:-

(a) overstress the epoxy grout between the joint by, in effect, making some parts of the joint stiffer than others; and
(b) the joint was more flexible than originally designed and this had the potential effect of increasing the effective length of the radial arm. They were originally designed for 0.85 L which implied fixity at the radial arm to horizontal girder joint.

A 'Basic' programme was written to analyse the revised joint, and all joints proved satifactory, with no reduction in effective length being necessary. Details of the analysis are given in Section 2.6.7 of Reference 30.

Further design considerations and details regarding the radial arm and bracing are given in Section 2.6 of Reference 30.

23.10 Epoxy Grouts as Packers

The Wivenhoe gates broke with tradition with the specification of epoxy grouts for use as a packing material both between the flanges of the skinplate ribs and the horizontal girders and the horizontal girder flanges and radial arm base plates.

(a) Girder - Rib Joints

Past practice for these joints has been to "shop fit" steel packer plates to suit each joint and matchmark them for site erection. When it came to site erection, deformations due to transportation and temperature variations meant that these shop fits were no longer the same and difficulty was experienced in actually fitting the skinplate with minimal gaps.

The introduction of an epoxy grout having a flowable consistency such as "Nordbak Backing Material" could fill these gaps and have sufficient bearing capacity to support the loadings and should solve the problem as well as speeding up site erection.

The required bearing capacities are listed in Table 4.2 of Reference 30.

(b) Arm - Girder Joints

The use of epoxy as a packer in this case was the alternative to machining bearing plates. Prior to 1975 the requirement for a full contact splice compression joint was a maximum gap of 0.50 mm with 60% of the bearing surface having less than 0.25 mm. Although this was relaxed in 1975 to a maximum gap of 1.0 mm, welding distortions could be of the order of 4 to 5 mm so that a machined base plate would still be required even if they were not machined to pre 1975 tolerances.

The design loadings for each joint after the combination of the loading case using equations (a) to (c) of Section 23.6 of this report are given in Table 4.3 of Reference 30.

(c) General

The above bearing stresses were based on a 30° dispersion to determine the stiff bearing portion of the connection. This approach was consistent with the web crushing angle of dispersion of AS 1250-1975, Rule 5.11.2. If the more conservative 45° distribution was adopted the stresses would be increased by approximately 45%.

The use of epoxy also has the added advantage that it not only insures a full contact joint but it also seals the joint against corrosion.

Tests on the epoxy "Nordbak" gave a compressive strength of 105 MPa which provided a factor of safety of better than 2.0 for the above applications. Its long term stability was illustrated by creep tests. The test results on a 5 mm thick test piece stressed to 22% of capacity showed the equivalent of a 6% creep strain over a 100 year period.

Therefore creep should not be a problem here since even under flood conditions with water to the top of the gate, the bottom girder is only stressed to about 25% of capacity.

(d) As Constructed

Two separate epoxy fillers were eventually used during construction.

Araldite K75 was used where joints were over a millimetre wide and Loctite Adhesive 290 Adhesive/Sealant was used for smaller gaps. A report on the properties of these materials is presented in Appendix A of Reference 30.

23.11 Trunnion Pedestals

The arrangement of the trunnion pedestals is shown in Figures 139 and 140 (Drawings A1-60915C and A1-60916B). Details of the trunnion pedestal are shown in Figure 141 (Drawing A1-61560D).

The design load cases that the trunnion pedestals were designed to accommodate were:

(a) Ultimate EL 79.0 water loading with gate shut;

(b) No water load with gate in fully raised position;

(c) The top of the gate at EL 77.0 with the water level at EL 77.0 (the maximum design case for flood control).

In load case (a) the trunnion pedestal was designed as a beam on an elastic foundation using formulae from Roark and Young (Reference 36) and an allowable bending stress in the plate of 230 MPa for the 100 mm thick steel. These calculations resulted in a plate 1050 mm by 1000 mm.

In the initial trials no side stiffeners were used and the base plate was modelled as a beam with cantilevered ends. When the lateral stiffeners were added it was treated as a combination of two cases:-

(i) A plate with no side stiffeners with a concentrated load at the centreline of the arms;

(ii) A plate with a uniform load over the length of the stiffeners.

A second approach properly modelled the actual conditions in the central region of the plate with the first being more applicable to the plate edges where there was a tendency for the lifting of the corners. The model applicability is shown in Figure 142 and the resultant deflection curves for both cases are plotted in Figure 143.

A three dimensional finite element model would have been required to analyse the plate accurately. However both models produced similar maximum compressive stresses in the supporting grout pad and this was taken as being sufficient. Using these models the probable maximum load of 14000 kN produced a bearing stress of 40-45 MPa and this was used as the criteria for the strength of the non-shrink grout required.

Construction loads would only be a fraction of these ultimate loads and if a 'Monolith' non-shrink plastic grout was used with a 3 day strength of 47.0 MPa, work could proceed on the pedestals within 24 hours when the strength should have reached an ample 25 MPa.

The 90 mm thickness of the grout pad reduced the bearing stress concentrations that could occur in the corbel and allowed good adjustability to be built into the trunnion pedestals.

The lateral forces in the pedestal approximated 16% of the axial loads and they dictated the use of 50 mm side stiffeners. These lateral stiffeners also produced greater lateral dimensional stability during stress relieving.

The bolting arrangement for the base plate of the trunnion pedestal was determined through load case (c). Under this condition, with a 45 degree stress distribution through the base plate, the capacities required by the bolts were:-

Shear	:	50.6 kN;
Axial	:	70.6 kN.

Using Grade 4.6 bolts to AS 1111-1972 the minimum bolt size capable of handling these loads was an M39 bolt. However the next preferred bolt size, an M42, was adopted.

Provision was made in the Specification for the on-site determination of the location of these holding down bolts which were installed under a previous contract (No. 2120) and these locations were provided to the Contractor to enable him to drill the trunnion pedestal base plate accordingly. This eliminated any major bolt installation errors and insured trouble free site installation.

Even with this adjustment provision built into the Specification it was thought necessary to provide a tolerance on the bolt holes of plus 8 mm on the diameter. This allowed for any lack of straightness of the bolts or holes and surveying inaccuracies.

To finally locate the trunnion pedestal the bolts were threaded on both sides of the expected position of the 100 mm thick base plate and provision was made each side for the use of adjusting nuts to provide positive position control in relation to the trunnion corbel. Temporary propping was required in the vertical direction prior to the filling of the bolt cavity with an epoxy grout. A 8 mm grout injection hole was added to the bolt for this purpose and the nut on the downstream side was vented to allow the exhausting of air.

To ensure that the capacity of the epoxy was not less than the shear capacity required of the holding down bolts, a bearing capacity of 20.6 MPa was required for the epoxy.

The bearing stress permitted on the pedestal arms was again governed by the ultimate loading condition and the situation is somewhat akin to the allowable bearing stresses for bolts bearing on plates.

It was assumed, for the purposes of design, that since the action of the bearing on the arm would place the top of the arms in tension, Table 9.4.2 of AS 1250-1975 could be used and an allowable bearing stress of 1.35 F_y was adopted.

Therefore the minimum thickness of the gate arms for bearing was 71.0 mm for a 320 mm diameter pin. This figure did not include the additional factor of safety of 1.20 introduced in Section 23.6 to act as a corrosion allowance and factor for the importance of the structure. When this was included the minimum thickness was increased to 84.5 mm and it was this figure that was adopted for design. By using the next preferred plate size of 90 mm, 5.5 mm could be machined off the thickness in the event of severe distortions interfering with the fit of the bearing and retaining rings.

It was envisaged that the pedestal units would be electro-slag welded, possibly in pairs back to back to reduce distortions, and stress relieving was specified to reduce the residual stresses and improve the grain structure.

23.12 Bearing Design And Trunnion Pin Assembly

23.12.1 Design Loadings and Operational Conditions

Details of the final bearing arrangement are shown in Figure 140 (Drawings A1-60916B).

The design operating conditions were fixed at the following:-

(a) Maximum gate opening = 62.65° which corresponds to the bottom of the gate at EL 73.0 and is necessary to permit the unhindered passage of the Probable Maximum Flood;

(b) Maximum rotational velocity under heavily loaded conditions equals 0.000391 radians/sec. which is fixed by the maximum hoist speed of 0.3 m/min lifting at a radius of 13 metres;

(c) Operational temperature range = -10°C to +70°C and;

(d) The bearing is to be capable of maintenance free operation.

The requirement of maintenance free operation eliminated the use of steel on steel bearings and dictated the use of reinforced PTFE (polytetrafluoroethylene) for the bearing surfaces. The cavity between the bearing housing and the pin was filled with a lithuim based grease as an aid to corrosion protection.

A specification of loads and intervals of use was prepared for the manufacturer's life calculation and is detailed in Table 2.12 of Section 2.8 of Reference 30. The six design load cases that the bearings were designed to accommodate are also listed in Section 2.8 of Reference 30.

23.12.2 Selection of Bearing Type and Arrangement

Three basic bearing and trunnion pin arrangements were considered. These are listed below and are shown in Figure 144.

1. A cylindrical sleeve type bearing.

2. A spherical plain bearing with the axis of the trunnion pin aligned with the spillway axis.

3. A spherical plain bearing with the axis of the trunnion pin aligned in the plane of the end frames.

Arrangement 2 was selected on the following basis:-

(a) Comparison of Arrangement 1 and Arrangement 2.

- Arrangement 1 has the disadvantage that extreme accuracy is required in setting up the bearing during fabrication and erection.
- Arrangement 2 provides an allowable angle of tilt which can be taken advantage of through a widening of the tolerances and added flexibility during erection.
- There is a cost differential in favour of Arrangement 2.
- The mass of the trunnion hubs and pedestals are lower for Arrangement 2 implying a cost advantage in terms of material and erection costs.

(b) Comparison of Arrangement 2 and Arrangement 3.

- In Arrangement 3 the bearing is aligned with the principal loading direction, i.e. along the arms and no transverse loadings are transferred to the trunnion pedestals, allowing the elimination of side stiffeners on the pedestal and allowing an arrangement of the trunnion corbel concrete to give extra capacity in the regions of greatest shear.
- Arrangement 3 has the disadvantage that the available range of standard fittings is limited and it is more difficult to install in the field.
- Arrangement 3 requires a higher strength hub forging which could be achieved by increasing the carbon content of the forging which may have caused additional welding problems.
- For Arrangement 3 the span of the trunnion pin is increased because of the necessity to provide additional clearances to cope with the different axes of rotation.

A full comparison of the three arrangements is given in Table 2.13 of Section 2.8 of Reference 30.

23.12.3 Trunnion Tolerances and Fits

To permit shop and field assembly an interference fit was specified for the bearing housing and a clearance fit was specified for the trunnion pin and retaining rings. Therefore the trunnion had to be assembled first by press fitting the bearing into the hub and then, on site assembly, positioning the elements and sliding the pin through. As the mass of the pin was approximately 0.43 tonne a 5 mm x 45° chamfer was provided to facilitate site assembly.

The tolerance on the gap between the pedestal arms was critical since the bearing relied on uniform bearing of the retaining ring on the arm to transmit the transverse loads.

23.12.4 Trunnion Pin

The need to have a complementary material to that of the inner ring of the bearing so that a corrosion cell would not be set up meant that a stainless steel pin was required. The pin was checked for shear and bending as a simply supported beam. The strength requirements and deflections at ultimate loads for the pins of each arrangement are shown in Table 2.13 of Section 2.8 of Reference 30.

It was decided to take the lateral loads of the bearing directly into the trunnion pedestal via the retaining rings rather than by fixing one end with bolts. This offered greater simplicity and virtually eliminated the strict tolerance requirements on the length of the pin demanded by its alternative.

The pin is kept in place by loose fitting keeper plates in specially cut grooves at each end of the trunnion pin.

24.0 SPILLWAY CREST BULKHEAD GATE

24.1 Spillway Crest Bulkhead Gate - General

The design of the Spillway Crest Bulkhead Gate is fully covered in the "Report on Wivenhoe Dam - Design of Crest Control Structures" prepared by Senior Engineer P.H. Allen of the Engineering Services Group of the Water Resources Section of the Department of Primary Industries (Reference 30). This section, extracted from that report, gives details of the design of the various components of the spillway crest bulkhead gate.

The spillway crest bulkhead gate can be installed in slots constructed in the piers and side walls of the spillway in front of the radial gates. The bulk head gate is one unit 12.350 m wide by 12.150 m high and weighs approximately 55 tonnes. The arrangement of the bulkhead gate is shown in Figure 145. (Drawing A1-61575D) and the installation details are shown in Figure 146 (Drawing A1-62297A).

The bulkhead gate will permit maintenance of the radial gates without loss from the storage and can also act as a flow controller should any single radial gate malfunction and remain open. The advantages of this capability include:-

(i) Uniform flow can be maintained in the flip bucket and thus minimise erosion in the downstream pool;

(ii) the storage above the fixed crest level of EL 57.0 will not be lost in the event of a gate being stuck open. The volume of water between the crest level and the top of the bulkhead gate at EL 69.0 represents approximately 66% of the usable storage above EL 33.20, the sill level of the intake structure.

The sealing of the bulkhead gate will be accomplished by rubber seals attached to the sides and base of the gate on the upstream side and bearing against stainless steel seal plates in the sides of the piers and spillway walls and the crest of the dam as shown in Figure 146. The seating frame and gate guides are fabricated from 316L stainless steel to AS 1449-1978, and the wheel track is fabricated from stainless steel to ASTM A564-630.

A 150 mm dia valve and handwheel is installed in the bulkhead gate to permit flooding between the bulkhead and radial gate, to minimise additional loading on the crane hoisting cables.

The bulkhead gate will be handled and positioned by the 79 tonne bulkhead gantry crane. When not in use the bulkhead gate will be stored in the bulkhead gate chamber in Monolith RH12 on the right hand side of the spillway.

24.2 Design Loadings And Operational Conditions

The bulkhead gate has been designed to support a water loading to EL 69.0 which corresponds to the top of the gate with the gate in the installed position. This allows a 0.5 metre surcharge over the design full supply level of EL 68.5 and gives some flexibility in its use should it be down with the onset of a flood.

Although the gate has been designed to be operational under flood conditions this only extends to the raising and lowering of the gate under flow. The gate has not been designed for overtopping. However there should be some reserve capacity in the gate for overtopping to occur prior to gate failure.

24.3 Hydraulic Model Studies

A situation not investigated at the preliminary design stage was that of the radial gate malfunctioning in the nearly closed position with the bulkhead gate then being required to act as a control to close off the flow.

Under these conditions it is possible for the bottom of the gate to be drowned and the configuration adopted for the preliminary design could induce vertical gate vibration.

A study carried out by the QWRC's Hydraulics Laboratory at Rocklea confirmed the possibility of self controlled vibrations at small gate openings and recommended improvements to the shape of the bottom of the gate (Reference 37).

Schematically the situation is illustrated in Figures 5, 7 and 8 of the report on Wivenhoe Dam Spillway Bulkhead Gate - Study of possible Flow Induced Vibration (Reference 37).

The recommendations made in the report included:

(a) The bottom shape of the gate be amended from the original design to a 60 degree flared shape with perforations in the flare and the bottom girder.

(These shape provisions were largely complied with although the perforations to the bottom girder could not be satisfactorily included in the final design due to the high shear forces and the already restricted geometry due to the 60 degree flare. Some holes were included. However these only represented a small percentage of the area of the lower flange).

(b) In addition to the gate dead weight and frictional resistance a further 100 kN be allowed for in the lifting force which is to be made up of 50 kN in extra downpull if the seal is moved to the upstream face and 50 kN to allow for exciting forces on the gate bottom.

(c) The bulkhead gate should not be raised with head on the upstream side and a closed radial gate downstream without filling the compartment between the two gates first.

A bypass line was recommended and this was included in the final design to facilitate the filling of this compartment. Figure 147 presents a graph of the approximate filling times of this compartment for different headwaters using the 150 mm gate valve provided.

Figure 148 plots the minimum tailwaters required in the space between the gates so that in the event of downpull and vibration occurring the crane is not loaded beyond its design load. From this graph it can be seen that when the headwater is below about EL 65.5 no tailwater is required behind the gate. However, it would be advisable in all cases to fill the compartment downstream of the gate to within say 0.5 or 1 metre of the upstream water level prior to lifting whenever possible to minimise the additional loads placed on the lifting cables.

(d) The radial gate should be raised as high as the spillway operating procedures allow before using the bulkhead as an emergency control gate.

(e) If the bulkhead gate vibrates for any reason during lowering the downward motion should be continued. The gate should not be stopped and raised again, otherwise the design load will be exceeded.

24.4 Horizontal Girder

A two dimensional analysis, modelling the skinplate as a continuous member, was used to determine the skinplate moments and the distribution of loading per girder. Using this analysis the weight of the bulkhead gate was optimised by spacing the horizontal girders to enable the use of a constant skinplate thickness. This approach resulted in varying horizontal girder loadings which were accounted for by varying the web and flange dimensions.

The alternative optimisation approach was to equalise the girder loadings and vary the skinplate thicknesses to suit. This resulted in increased spacings at the top of the gate and correspondingly increased plate thicknesses giving a heavier gate overall.

The allowable stresses adopted for the gate were the same as those adopted for the radial gate with a water loading to EL 73.0.

Table 3.1 in Section 3.3 of Reference 30 gives a summary of the sections, loads and stresses calculated in the design of the skinplate and horizontal girders.

24.5 Wheel Locations And Design Loadings

The analysis of the wheel loadings was carried out using two methods. The first of these was a two dimensional frame type analysis and the other was a three dimensional analysis using the SAP IV finite element program.

Coarse positioning was carried out using the two dimensional (2D) program with the only restrictions being the need to provide adequate clearances to the horizontal girders which explains why some of the intermediate wheels are more highly loaded than others.

Final determination of the wheel loadings and the effect of wheel track tolerances were done using the three dimensional (3D) analysis which more correctly modelled the geometry of the gate.

Nine wheels a side were chosen because:

(a) A larger number of wheels would have reduced the minimum spacing between the bottom two wheels and thus provided too great a restriction on the diameter of the wheels to carry the loads.

(b) Nine wheels conveniently fitted into the arrangement of the horizontal girders with a minimum of compromise.

(c) A lesser number of wheels would have meant increases in the wheel loadings requiring an increase in the width of the bulkhead gate slot to accommodate the required larger wheels.

The top and bottom wheels are significantly more heavily loaded than the intermediate wheels and this was deliberate to a certain extent to take into account the effect of gate rail and wheel position tolerances. Their effects can be seen in the normal and overload loadings of Table 3.2 of Reference 30.

The design loadings for the "overload" conditions were determined using the three dimensional analysis and giving the tracks under each of wheels 7, 8 and 9 a 1mm displacement through the use of boundary elements.

The tolerances selected were the same as those specified and attained for the fixed wheel gates at Beardmore Dam, i.e.

- gate wheels were to be aligned to within ±0.125 mm before installation; and
- the face of track crown was to be in a true plane within ±0.4 mm in any 3 m length ±1.0 mm in the overall length

The other reason for the high "normal" loading on the bottom wheel was that the spillway crest proved to be a constraint on its location as the wheel could not have been moved lower in the gate without touching the crest with the gate in the fully down position. The wheel, in its present position, has a 25 mm clearance to the crest which means the wheel could not have been lowered without providing a dishing of the bulkhead gate slot which could fill with debris and prevent complete closure of the gate.

Had a smaller bottom wheel been used to bring the wheel closer to the crest and reduce the loading, it would have fouled the 60 degree flare introduced to assist the vibration characteristics of the gate.

Following the application of tolerances and preset as detailed in Section 3.4 of Reference 30, the following design wheel loadings were adopted for the wheels, bearings, axles and tracks for the bulkhead gate:-

Normal ... 512 kN
Overload .. 736 kN

24.6 Design Of Wheel And Track Assembly

24.6.1 Wheel Design

The stress in the wheel tread where the wheel contacts the track was the controlling factor in the design. However the diameter of the wheel was determined by practical limits such as the available wheel space.

The maximum wheel diameter was set by the minimum of:

(a) Minimum spacing between wheels = 713 mm

(b) Maximum available slot space = 689 mm

The preliminary design was based on a 650 mm diameter wheel. However in the final design this was raised to 670 mm to improve the available factors of safety. This design produced a 19 mm clearance to the bulkhead gate skinplate.

The following basic design assumptions as to material properties, design factors of safety and allowable stresses were taken from Skinner (Reference 38), and were based on the ASTM specifications for an A57 steel.

- F.S. for "normal" wheel loads = 3.0
  F.S. for "overload" wheel loads = 2.0
  (factors calculated on critical stress on projected area)

- Critical stress = (B.H.N.*0.1689 - 15.17)MPa

- B.H.N. of wearing surfaces : Wheels = 255
  Track = 305

- Maximum allowable shearing stress in wheel = 90,000 psi or 621 MPa (Skinner notes that this is conservative)

- Depth of rim hardening = 1.5 to 2 times depth to max. stress

A 817M40 to BS970:Part 2,1970 was substituted for the ASTM A57 of Skinner and this is the same material specified for the Wivenhoe penstock gate wheels.

Two separate references were used to assess the load carrying capacity of the wheels. Both were essentially formulae for crossed cylinders however Skinner's method relies on interpolation of coefficients using a relatively small scale figure and it produces variable results (Reference 38). Roark's formulae rely on tabulated coefficients which can be plotted to a larger scale for interpolation and produce more consistent results (Reference 36). A comparison of the results produced from the two sources is shown in Table 3.4 of Reference 30.

The radius of curvature of the track was set at 2000 mm as being the largest radius that could be practically fabricated. Past designs have used a 1500 mm radius and the calculations were done for this radius and the 650 mm wheel diameter as a comparison.

Skinners' factors of safety operate on the "critical stress" and do not seem to properly take into account the effect of variations in track radius. It seems more logical to apply these factors to the actual stresses at the points of contact and this was done after initial sizing of the wheels using Skinner's critical stress.

Based on the average capacities for the selected configuration a factor of safety of 2.854 on normal loads and 1.985 on the overload condition applied.

These are slightly lower than Skinner's desirable minimums and to raise them closer to acceptability the rim hardness was increased to a minimum of 260. If it is assumed that the allowable local contact stress is proportional to B.H.N. the increased factors of safety become 2.91 on normal and 2.02 on overload.

These factors were considered adequate for the application as unlike Skinner's penstock gates, this gate will only rarely be operated as an emergency control gate and even then under normal operating conditions there will be some freeboard.

24.6.2 Wheel Axles and Supports

The wheel axles and supports were designed as simply supported cantilevered beams with circular cross section using the maximum lateral wheel displacements and the "overload" condition. The minimum diameter was set at 180 mm, the diameter of the bearing, and the diameter of the axle at the vertical rib was set at 200 mm.

The required material properties were determined using the ASME formula for axles subject to bending and torsion, as described in Section 3.5.2 of Reference 30.

After discussions with Water Resources mechanical engineers on the difficulty of providing corrosion protection to the areas beneath the bearings and the areas adjacent to the disc springs, it was decided to substitute a stainless steel for the axle material. An EN57 stainless steel was selected. This steel has typical properties in a 150 mm section of:-

Ultimate Tensile Strength	= 849 MPa
Yield Strength	= 680 MPa.
Izod Impact Strength	= 20.3 Nm

Sufficient bearing area was provided at the vertical rib by attaching an additional 20 mm plate to locally stiffen the rib. The plate thickness was determined using the ultimate capacity of the wheel and an allowable bearing stress of F_y in the rib. This was done on the basis that an axle failure would be preferable to a structural failure of the gate.

Locking nuts were provided at the end remote from the wheel to prevent pull-out of the wheel and to resist the lateral forces exerted by the wheel if it drifts off-line while going down the rail.

24.6.3 Disc Springs

Once heavy loads are placed on the gate wheels it becomes very difficult to alter their direction of movement. Therefore disc springs, arranged similarly to those of the North Pine Dam bulkhead gate, were provided to keep the wheels centred in the unloaded condition and to provide a ±12 mm float should the wheels begin to move off-line. When the float is exceeded the full lateral loading is taken by the axle or by the keeper plate at the end of the axle. The keeper plate is held to the axle by 4 M20 studs with an ultimate tensile capacity of 392 kN or 53% of the expected maximum loading in the "overload" condition.

Each set of springs is arranged in pairs so that only 6 mm deflection is required of each spring to give the full float. Each spring set was given a 2 mm preset to provide an additional centering force at close to centre. Above this preset only one set of springs operate.

Formulae from Roark (Reference 36) were used to determine the stiffness of the springs such that in this configuration a deflection of 4.55 mm was sufficient to support the wheel weight of 2 kN.

24.6.4 Bearings

The selected bearing for the main wheels is a standard "off the shelf" Garlock DU series MB180 100DU which is a PTFE, maintenance free, cylindrical sleeve type bearing with a nominal diameter of 180 mm and length of 100 mm.

The design loadings used for the selection were:

"Normal" = 512 Kn
"Overload" = 736 Kn

which correspond to the wheel design loadings. The velocity of gate movement under load was taken as 1.5 m/min. The gantry crane is capable of lifting the gate at 3.0 m/min. but for increased bearing life, it is recommended that this higher speed be only used for operation under relatively balanced loading conditions.

Under these conditions the manufacturers' "life" calculations produced a working life in excess of 650 hours. This is equivalent to raising and lowering the fully loaded gate some 2440 times which, since it is only to operate loaded under emergency conditions, should be sufficient.

24.6.5 Wheel Track And Support Frames

The wheel track and support frame were designed as an infinite beam on an elastic foundation using formulae from Roark (Reference 36). All exposed surfaces were specified to be of stainless steel for improved corrosion resistance.

Composite action between the rail and support frame would have required M24 bolts at 80 mm centres and it was therefore decided to make the connection flexible enough to permit non-composite action. Fortunately sufficient depth of support frame had been provided in the preliminary design stage and it was not overstressed by the loss in effective section.

An elastic analysis was carried out to determine the longitudinal displacements in the support frame for non-composite action. This showed a relative displacement of 0.0354 mm was necessary to ensure that no large shear forces would be transferred across the connecting bolts.

This displacement was permitted by using M16 bolts in oversize holes with nuts welded to the underside of the support frame. The bolts therefore span the thickness of the plate support frame allowing the necessary displacement without overstressing the bolts. With this arrangement the bolts now act only as hold down bolts and a spacing of 300 mm centres is sufficient.

With no composite action the following stresses result with the passage of a wheel over the beam:

$M_{max} = 132.5$ kNm
$\sigma_{\text{steel flange}} = 162$ MPa $= 0.65\ F_y$
$\sigma_{\text{stainless flange}} = 108$ MPa $= 0.635\ F_y$

The supporting cleat system was arranged to permit full adjustability of the frame to allow the attainment of the necessary tolerances.

Extra shear reinforcement consisting of S24 bars at 150 mm centres and designed in accordance with AS 1480 was provided beneath the support frame to cater for the high concentrated loadings induced by the fully loaded gate. The maximum height of the reinforcement was selected at EL 72.5 on the basis of controlling floods to EL 74.5.

With no additional shear reinforcement other than the ferrules and cogged bars provided in the preliminary design the gate track could support a wheel loading equivalent to 3.6 m of water which corresponds to the bottom wheel at EL 71.9 with a headwater of EL 74.5.

With the gate lip just touching the flow (EL 72.1) the bottom wheel is at EL 72.5

Under full control with water lapping the top of the gate at EL 74.5, the top wheel will be at EL 71.9.

Since the wheel track has been freed from composite action it really only has to satisfy two conditions:

(a) That its wearing surface has a minimum B.H.N. of 310 and,

(b) That its composition is corrosion resistant and compatible to that of the abutting support frame.

Stainless steel to ASTM A564-630 in condition H.620 with a minimum B.H.N.of 311 should be sufficient for the purpose as no welding is involved.

To eliminate the possibility of a wheel becoming stuck in a gap between rails it is essential that the track units butt closely together with no offsets at the joints.

24.7 Bulkhead Gate Guides And Axle Keeper Plates

To provide positive positioning of the bulkhead gate as it enters the slot, guides were installed in the top section of the slot. These guides provide a 200 mm lead-in for easier insertion of the gate with the gantry crane.

The bulkhead gate is held by the gantry crane from its lifting points and by a restraining arm which fits into a fabricated "slot" that was site welded to the flanges of the horizontal girders after erection. Contact with the restraining arm ceases when the bottom of the gate reaches EL 72.6.

At this point 5 wheels touch the rail and an additional restraint is necessary to prevent the gate blowing over in high winds.

The bulkhead gate is progressively righted with each wheel that touches the rail by the keeper plates of the bottom wheel riding down the gate guides. With the halfway point of the gate dropping below the start of the rail the gate again becomes stable under wind loadings with a 10 mm clearance between the guides and the axle keeper plates.

The guides were designed for the gate being lifted into position with winds of up to 64 km/hr (40 mph) from the upstream direction. Permissible deflections of the top of the gate as each wheel contacts the start of the rail as well as the force exerted on the guide are listed in Table 3.5 of Reference 30.

It was assumed that the crane could restrict the movement of the top of the gate to less than 1040 mm for two wheels in contact and that the design case occured when wheel No.7 contacted the rail. As the gate drops further the maximum force on the guide drops to zero.

The safe shear capacity of the studs in the keeper plate is 100.4 kN which should be more than adequate to cater for the expected load of 55 kN.

To ensure that a minimum of three wheels are controlled by the guides at any one time until the seals come into contact with the top of the seal seating frame at EL 69.0, the bottom of the guide was extended down to EL 72.2. A 100 mm lead-in was provided for lifting in the reverse direction.

A lead-in of 60 mm was provided on the seal seating to ensure that the side seal of the bulkhead gate does not catch on the edge of the seating frame.

25.0 RADIAL GATES AND BULKHEAD GATE - MISCELLANEOUS

25.1 Rubber Seals

Neoprene was selected in preference to other rubber compounds for the seals for the bulkhead and radial gates on the basis of:

(i) Its excellent resistance to oxidation and sunlight;

(ii) Its good moulding and cohesive properties;

(iii) Its higher colder rebound efficiency and its higher compression modulus for a given durometer than butyl rubber.

Butyl rubber could have been used if it proved more economical but the bottom seal sections would be required to have been redesigned and thickened.

The design procedure for the crest seals followed that contained in Kent (Reference 39), using the provision that deflections were to be limited to 20% to reduce the effects of permanent set and drift in the compound over long periods of time. The crest seals in both cases were designed to accommodate the selfweight of the gate with deflections sufficient to cater for any deviations from the plane of the crest seal plates. Arrangements of the seals for the radial and bulkhead gates are shown in Figures 149 and 150 (Drawings A1-61549 and A1-61576B).

A summary of the crest seal calculations is given in Table 4.1 of Reference 30.

The bulkhead gate can accept slightly higher bearing pressures since it will generally be in the unloaded condition in its storage bay and its periods of loading will be relatively short.

The side seals for the radial gate have the same section as those used for Coolmunda Dam although for Wivenhoe Dam the PTFE strips were moulded onto the seals rather than being glued on after moulding. The PTFE reduces the coefficient of friction to 0.15 or lower although a figure of 0.2 was used in the design. The shape of this seal, as shown in Figure 149, allows positive sealing onto the side seal plate set in the pier.

The bulkhead gate side seal is of the music note type which relies on its flexibility and water pressure to seal against a seating frame set at the front edge of the bulkhead gate slot as shown in Figure 150.

In both sets of side seals a 3 mm preset was specified although it is not expected that this preset will remain effective for long as usage will introduce a permanent set in the opposite direction.

In all side seals 3 to 6 mm gaps in the PTFE were specified at the joints between moulded sections to eliminate the need for bonding the PTFE to itself at the bonding of these joints with rubber cement. The PTFE was also stopped 20 to 30 mm short of the bottom of the gate to allow expansion of the seal under load.

To ensure that the seal plates and seals could be bolted directly onto the skinplate segments, provision was made for drilling of the bolt holes in the seal plates and seals in conjunction with the bolt holes in the skinplate.

Lateral adjustment was built into the side seal plates of the radial gate through the use of slotted holes.

25.2 Weldments

All welds were designed in accordance with AS 1554, Part 1-1980 with full penetration butt welds only being specified in areas where it was felt that they were necessary. Stress relieving was specified for the hub forging welds and the trunnion pedestal welds to ensure that there were no significant stress concentrations and that the grain structure was not modified to any great extent by the welding processes.

A discussion session was held with consultants to the Rocklea workshops and some welding procedure and design changes were made as a result.

(i) Low hydrogen electrodes were not considered necessary since no high strength steels were being used and no low service temperatures would be met.

(ii) They suggested the use of partial penetration butt welds for the field welds joining the skinplate segments. Partial penetration butt welds would cause less shrinkage than full butt welds and there would be a substantial reduction in the time required for field erection of the gate since they would be quicker to lay down. The substitution was made possible by positioning the welds closer to the contraflexure points in the skinplate so that the required strength did not become a problem.

The process had the added advantage that the same size welds could be used for the full length of the skinplate. i.e. no change in weld size was necessary for the 12 or 16 mm skinplate. Therefore lateral shrinkage of the welds should have been relatively uniform.

(iii) The welding procedure for the trunnion plates was rearranged with the web plates being welded to the forging first to reduce the stress concentrations resulting from weld shrinkage. The detail for the side plate welds was also altered to a fillet backing run followed by back gouging and rewelding.

(iv) A lower carbon content forging was also recommended to reduce possible welding problems and this necessitated a bigger bearing to reduce the bearing stress on the hub forging.

The following schedule is a summary of the major structural welds. All fillet welds were designed for E41 electrodes and where possible a 1 mm corrosion allowance was added to the thickness.

(a) Full penetration butt welds were provided for:

  (i) All flange to flange connections in horizontal girders and skinplate segments;
  (ii) All web to web connections in the skinplate segments;
  (iii) Hub unit and trunnion pedestal welds;
  (iv) Gate guides.

(b) The skinplate field welds were specified as partial penetration butt welds welded both sides with a 60° included angle.

For a 4 mm weld on both sides of the skinplate the welds are only stressed to 33% or less of the 0.33 F_{uw} allowable in both the 12 and 16 mm plate. A 1 mm corrosion allowance was added to the upstream weld and 20% of the weld was to be subjected to dye penetrant testing to ensure that the upstream surface was free from cracks which would allow water to penetrate and promote a corrosion cell in the unfused central section of the weld.

(c) Fillet Welds

Details of fillet welds are shown in Table 39.

TABLE 39 - DETAILS OF FILLET WELDS.

Location	Force/mm (kN/mm)	Reqd. size (mm)	Adopted (mm)
Skinplate segments			
- flange to web	0.272	2.9	5 *
- web to skinplate	0.349	3.7	6 *
Horizontal Girders Flange to Web			
Top	0.492	5.2	7
Middle	0.559	5.9	7
Bottom	0.553	5.8	7
Radial Arms - flange to web	-	6 *	7
Anchorage Point 25 mm Plate	0.394	4.1	6 *
Bulkhead Gate horizontal girders	<0.15	<2	6 *
Lifting Points - connection to top web	0.551	5.8	7

Note: An '*' indicates that this is the minimum fillet weld for the given plate size.

(d) Seal welds have been specified at the join of the top arm cover plate to the web for the radial gate and for the field splices between the section of the bulkhead gate skinplate to seal out corrosion.

26.0 GANTRY CRANES

26.1 Gantry Cranes - General

Two gantry cranes were designed for use in the spillway area at Wivenhoe Dam, the locations of which are shown in Figure 132. (Drawing K1-56530A)

The 79 tonne gantry crane traverses along the embankment and across the spillway via the service bridge so that the hook coverage ranges from Distance 1094 m on the left bank to approximate Distance 1253 on the right bank. It therefore commands coverage of the left bank Underground Control Complex, the five spillway gate bays, and the bulkhead gate storage pit on the right bank of the spillway.

The smaller crane, a 3.2 tonne capacity crane services the inlet structure and traverses Monolith LH9 and part of Monolith LH10, on the left hand side of the spillway upstream of the service bridge.

26.2 79 Tonne Gantry Crane

The 79 tonne gantry crane was designed to lift, lower and travel with the spillway bulkhead gate to position it in any of the five bulkhead gate slots which are located just upstream of the radial gates, and to return it to storage in the bulkhead gate chamber in Monolith RH12 on the right hand side of the spillway. The crane was also designed for use in the installation and maintenance of the radial gates, the radial gates' hoisting units, and the penstock gate.

The crane is an electro-hydraulically operated double cantilever type crane with a maximum safe working load of 79 tonne.

Contract No. 2122 for the manufacture, inspection, supply, delivery, erection, testing and commissioning of the crane was let to Evans Deakin Industries of Brisbane on 28 July 1981 and was completed in September 1985. The crane was designed by officers of the Queensland Water Resources Commission.

A general arrangement of the crane is shown in Figures 151 and 152 (Drawings A0-60000A and A0 60001B).

26.2.1 Description of Hoists

Two hoisting units were provided on the crane. Each hoist is powered by a Hagglunds Series 62-16300 hydraulic motor with brake. Each hoist drum is grooved left and right hand thereby handling two falls of the eight rope falls to each hoist block.

Both hoists are controlled electro-hydraulically from the crane operators control cabin.

26.2.2 Lifts

The two hoists on the crane can be operated mechanically coupled or separately. When a single hoist unit is selected the SWL is limited over the downstream cantilever section of the bridge to 7 m downstream of the centreline of the service bridge, i.e. the centreline of the radial gate winch platform. The SWL of a single hoist in this location is 10 tonne. Single hoist operation is available anywhere in the area commanded by the crane.

Dual hoist operation is restricted however and is not available over the downstream cantilever section of the bridge. Where dual hoist operation is available the full SWL of 79 tonnes is allowable with equal block loads. Hook travel for dual hoist operation is limited to 3 m downstream and 5 m upstream of the centreline of the service bridge - i.e. the centreline of the penstock gate.

26.2.3 Speeds

Hoist operation block speeds are nominally as follows:

(a) Dual hoist.

Low Speed	1.5 m/min
High Speed	3.0 m/min

(b) Single hoist.

Low Speed	3 m/min
Medium Speed	6 m/min
High Speed	12 m/min

26.2.4 Limits

Upper and lower hoist limits automatically limit block travel.

26.2.5 Braking

Fail safe braking is incorporated in each hoisting unit and each brake is hydraulically powered off.

Direct emergency application of each brake is available to the crane operator via the electro-hydraulic system.

26.2.6 Travel Motions of Crane

The crane's longitudinal travel motion is powered by four Hagglunds hydraulic brake motors Series 2165, two motors providing tractive effort to each rail.

The longitudinal drives are electro-hydraulically controlled from the control cabin.

Longitudinal travel speeds are as follows:

Creep speed	1 m/min
Medium speed	15 m/min
High speed	30 m/min

The longitudinal drive is arranged such that the hydraulic motors driving on both rails are powered by power packs that can, in an emergency, be separated to drive the motors on one rail only.

Driving on one rail only requires the presence of maintenance staff to release the brakes on the non driving motors and to park the crane.

The hook coverage on longitudinal travel is 91.6 m on the left bank and 67 metres on the right bank both measured from the spillway centreline.

26.2.7 Parking and Storm Anchorage of Crane

The crane can be safely parked at either end of the dam crest to accommodate out of service wind loads.

The parking procedure is as follows:

As the crane enters into a parking area it trips a limit switch which automatically sets the long travel speed to creep speed. The crane can continue long travel into the parking area at this speed until both buffers contact the end stops at which point travel limits shut down the long travel drives. The buffers will then automatically align the crane on the end stops by fully extending. In this position an audible alarm is activiated which stays on until the storm anchor pins are inserted or the crane is driven away from the end stops.

The crane can only be storm anchored provided the buffers are fully extended and in contact with the end stops.

To storm anchor the main crane structure the crane operator presses a push button which activates the insertion of the storm anchor pins. When the pins are fully inserted the alarm is deactivated and the main structure is parked.

26.2.8 Travel Motion of Crab

Crab cross travel is powered by two Staffa hydraulic brake motors - series B80 motors with F100 brake units.

Cross travel is electro-hydraulically controlled by the crane operator from the control cabin.

Crab cross travel speeds are as follows:

Low Speed	2 m/min
High Speed	4 m/min

Travel limits automatically restrict the extent of crab cross travel.

26.2.9 Parking and Storm Anchorage of Crab

To storm anchor the crab the crane operator must turn a selector switch to crab 'park' mode. In this mode the operator must traverse the crab towards its park position. A position limit switch will stop the crab and activate rams which will accurately locate the crab to enable storm anchor pins to be set. This limit switch also activates an audible alarm. When the crab is correctly located the operator activates the insertion of the storm anchor pins. When the pins are correctly set the alarm is deactivated.

26.2.10 Crane Rails Supports

The crane runs on 86 kg/m BHP rails. The layout of the rails is shown in Figure 103 (Drawing A1-54750B). On the left bank the rails are installed on reinforced concrete beams which are supported on 600 mm dia bored piles at 3.5 m centres, and the walls of the Underground Control Structure which are also supported on 600 mm dia bored piles at 4.235 m centres. Details are shown in Figures 153 and 154 (Drawings A1-54718B and A1-54663B). Further details on the underground control structure and support of the crane rails are given in Section 27.

The rails cross the spillway supported on the prestressed concrete beams of the service bridge. On the right bank the rails are supported by reinforced concrete beams as described in Section 16.1.3.

26.3 3.2 Tonne Gantry Crane

26.3.1 3.2 Tonne Gantry Crane - General

The 3.2 tonne gantry crane was designed to raise and lower the trashracks and the trashrack fishing gear, hatch covers on Monoliths LH9 and LH10, the selective baulks and the baulk hoists, and for general lifting duties in the intake structure area.

The crane was designed by officers of the Queensland Water Resources Commission and fabricated in the Commission's workshop at Rocklea.

The general arrangement of the crane is shown in Figures 155 and 156 (Drawings A0-62000 and A0-62001A).

The crane meets the Class 2 requirements of AS 1418. It was fabricated from Grade 250 steel conforming to AS 1204 and protected with a primary coat of Dimet "Zincilate" followed by a top coat of Dimet "Armourdor 920".

26.3.2 Operating Parameters

The crane operates within the following parameters:-

Lifting Capacity SWL:-	3.2 tonne
Lifting range:-	51.9 m
Lifting speed:-	0.1 m/s
Longitudinal travel:-	19.96 m
Longitudinal travel speed:-	0.2 m/s
Crab traverse:-	10.26 m
Crab traverse speed:-	0.2 m/s

26.3.3 Crane Rail Supports

The crane runs on 86 kg/m BHP rails supported on a reinforced concrete beam on the streamside and the concrete wall on the bankside of Monoliths LH9 and LH10, except for the area above the selective baulk installation. Here the crane rails are supported by fabricated steel beams as shown in Figure 157 (Drawing K1-51441). The design loadings are given in Section 20.2.

27.0 UNDERGROUND CONTROL STRUCTURE AND SUPPORT OF GANTRY CRANE RAILS

27.1 Underground Control Structure and Support of Gantry Crane Rails

The underground control structure is located on the embankment adjacent to spillway retaining wall Monolith LH11. The structure houses the transformer room, switch room, store room, diesel alternator room, and the ventilation plant room. Details are shown in Figure 154 (Drawing A1-54663B).

The gantry crane travels over the top of the underground control structure. Because of possible track alignment problems which could be caused by settlement of the embankment, it was decided to support the control structure and track beams on 600 dia bored piles.

The crane rails were positioned over the side walls of the structure so that crane loads were transferred directly through the walls to the bored piles.

Contraction joints were located along the structure and beams to allow for shrinkage and temperature movement.

The design for the control structure and track beams was in accordance with the following:-

AS 1480 - 1974 Concrete Structures Code
AS 1250 - 1975 Steel Structures Code
NAASRA Bridge Design Specification 1976

Details of the 1600 kW gantry crane design loading is given in Section 16.1.3.

The roof and hatch covers were designed for a T44 Standard Vehicle Load in accordance with NAASRA Bridge Design Specification.

Soil properties to determine soil loads were $\varnothing = 37°$ and $\delta = 1890$ Kg/m^3.

Floor live loads were as follows:-

Store	13 kPa
Transformer Room, Switch Room, Store Room, Diesel Alternator Room, and Ventilation Plant Room	5 kPa
Gallery	4 kPa
Stairway	2.5 kPa

The crane rail buffer as shown in Figure 153 (Drawing A1-54718B) was designed for a 205 kW load acting 750 mm above the rails.

28.0 DAM SURVEILLANCE

28.1 Dam Surveillance - General

Measuring devices were installed in the foundations of the dam and spillway and in the embankment to enable monitoring of the behaviour of the dam to ensure its safety. The measuring devices included:-

Surface Movement Points;
Piezometers;
Total Pressure Cells;
Inclinometers and Electric Settlement Gauges;
Hydrostatic Settlement Gauges;
An Extensometer; and
Drainage systems.

The location of the devices in the dam embankment are shown in Figures 158, 159 and 160 (Drawings A1-53374W, A1-53375B and A1-50780D). Devices were installed in the embankment at two critical sections to record the settlement behaviour at those points. Distance 1600m was chosen where the outer shells of the embankment are founded on deep alluvium, and Distance 1800m is a section near the old river bed where the dam is close to its maximum cross-section, and where the outer shells are founded on alluvium and river gravel. The devices installed at Distance 1600m are terminated at Piezometer Terminal Structure No. 1 located at Distance 1705m, and devices installed at Distance 1800m are terminated at Piezometer Terminal Structure No. 2 located at Distance 1845m. Both structures are situated at the downstream side of the dam at EL 28.0.

Piezometers were also installed in the foundations of the spillway overflow monoliths as shown in Figure 161 (Drawing A1-56230A). The spillway piezometers are terminated on a board mounted on the upstream face of the lower gallery in Monolith S3.

28.2 Surface Movement Points

The purpose of the Surface Movement Points is to monitor settlement of the dam as a whole. The points were installed on the crest of the dam and also on the upstream and downstream faces of the dam.

Forty eight surface movement points were installed on the dam at the locations shown in Figures 158 and 159. Thirty two points were installed on the crest of the dam at intervals between 50m and 100m; four were installed on the upstream face at EL 69.0 and the remaining twelve on the downstream face at various elevations between EL 29.0 and EL 69.0.

A movement point consists of a standard brass target retaining bolt with brass cap set in a 100mm NB galvanised MS pipe which is bolted to a rock of 750mm minimum diameter or set in a block of concrete approximately 600 mm cube. A survey target is placed on the bolt and its position and elevation recorded.

Surface Movements Points are required to be read annually.

28.3 Piezometers

The purpose of the piezometers in the embankment is to register hydraulic changes at the buried tips indicating variations in pore pressures and in strength of the core material. Changes in pore pressures under the spillway overflow monoliths will give an indication of the effectiveness of the grout curtain and drainage holes.

Fifty one piezometers, consisting of forty seven hydraulic and four electric piezometers, were installed in the foundations of the embankment and in the earthfill Zone 1A and filter Zones 2A or 2B at the locations shown in Figures 158, 159, and 160. In addition fourteen piezometers were installed in the spillway foundations as shown in Figure 161.

The hydraulic piezometer tips are filled with water and transmit pore pressures via twin nylon tubes to a hydraulic pressure gauge and to pressure transducers at the terminal structures. The four electric piezometers in the dam foundations were equipped with pressure transducers and the results transmitted electrically to the terminal structures. The piezometers fitted with transducers were connected electronically to a data logger which recorded the pressures daily or in the event of a flood was able to record pressures every two hours. The results in the logger could be downloaded via telephone or modem onto a computer and could be either plotted, printed out or stored on disc. However following unsatisfactory performance up to 1990, the data logger was removed and not replaced. The hydraulic piezometers were thereafter read manually.

Piezometers are required to be read monthly.

28.4 Total Pressure Cells

The purpose of the total pressure cells is to record stresses in any desired plane in Zone 1A material , Zone 2A and 2B material in the embankment, and in the backfill Zone 1A material in the diversion channel.

Twenty five total pressure cells were installed in the dam at the locations shown in Figures 158, 159, and 160.

Total pressure cells consist of "flat jack" pressure cells filled with oil. The oil pressure, equal to the pressure of the surrounding soil, is transmitted via a tube to an electric pressure transducer and then by electric cable to the terminal structures. The cells may be placed at any orientation to record pressure in any plane normal to the cell.

Twenty two of the cells were placed in earthfill Zone 1A material and filter Zone 2 materials at the two main instrumented cross-sections at Distances 1600m and 1800m. Cells TP23, TP24 and TP25 were placed in Zone 1A material in the diversion channel backfill.

Total pressure cells are required to be read quarterly.

28.5 Inclinometer Installations and Electric Settlement Gauges

The purpose of the combined inclinometer installations and electric settlement gauges is to record and monitor horizontal movement and vertical settlement in the dam embankment and the alluvial foundations.

Four inclinometer and electric settlement gauges were installed in the dam at the locations shown in Figures 158 and 159. Installations I1 to I3 were installed at distance 1600 and I4 at distance 1800. Installations I2 and I3 were installed through the alluvial foundations of the outer downstream shell of the embankment to record any settlement of the alluvium. Within the alluvium the installations are encased in 150 mm bore casing.

The inclinometer installations consist of vertical aluminium access tubes buried over their full height in the dam embankment. Movements away from the vertical are recorded by an inclinometer lowered down the tube. To function as settlement gauges the aluminium tubes have magnetic cross arms and rings attached. Their position is recorded by a torpedo sensitive to magnetic fields, enabling settlement of the fill or alluvium to be recorded.

The electric settlement gauges are required to be read monthly and the inclinometers are required to be read quarterly.

28.6 Hydrostatic Settlement Gauges

The purpose of the hydrostatic settlement gauges is to monitor settlement of the alluvium under the outer shell of the embankment.

Two hydrostatic settlement gauges were installed in the dam at the locations shown in Figure 159. The gauges were placed 42m upstream and downstream of the dam axis at Elevation 32.0 at Distance 1600m where the shells are founded on alluvium up to 15m deep.

A gauge consists of an overflow weir in an enclosed box, the weir being connected to an external measuring point by inflow and return lines which circulate water through the weir. The level of water in the weir is read directly by reading the water level in the supply line.

The hydrostatic settlement gauges are required to be read quarterly.

28.7 Extensometer

During cleanup of the core foundations in the vicinity of the old riverbed at approximate distance 1867m a fault striking at right angles to the dam axis and dipping approximately 45° towards the left bank was exposed. The fault consisted of a 300mm gouge beneath a 5m thick zone of fractured sandstone as shown in Figure 162.

Further movement of the fault could cause a potential hazard by creating a flow path under the core of the dam. To monitor future movement an extensometer was installed across the fault as shown in Figure 162. The extensometer was installed in a 75mm dia hole drilled at approximately 30° to the horizontal. The extensometer consisted of a bottom anchor, a stainless steel wire in a nylon protective sheath, a support assembly and measuring head. Movement is related to the change in resistance of the wire and is accurate to within 0.5mm. Leads from the extensometer terminate in Piezometer Terminal Structure No. 2.

The core foundation in the vincinity of the fault was blanket grouted, anchor bars were installed, and the area of the fault zone backfilled with concrete to the levels shown in Figure 163 (Drawing A1-57707).

28.8 Embankment Drainage Systems

In the sloping core embankment on the left abutment of the dam, filter drains were installed at 50m intervals as shown in Figures 164 and 165 (Drawings A1-45403C and A2-79207). Drainage of the right abutment is via pressure relief holes drilled into the foundations at 25m intervals as shown in Figure 166 (Drawing A1-54651B) and filter blankets placed over the shell foundations downstream of the core. Seepage is collected from the downstream toe between distance 1750m and the diversion channel by a series of vertical drains and two 600mm dia concrete pipes connecting to the filter blanket as shown in Figures 167 and 168 (Drawings A1-79257 and A1-79259). The concrete pipes discharge via flap gates into a V notch measuring weir.

Flows from the weir should be recorded monthly and any increase in flow from any source or change in colour should be reported.

29.0 PROVISION FOR FUTURE HYDRO-ELECTRIC TURBINE

29.1 Capacity of the Plant

The original report by the Co-ordinator General's Department suggested that a hydro-electric power plant, similar to the one at Somerset Dam but of a larger capacity, be installed at Wivenhoe Dam (Reference 1). The Interdepartmental Committee on Design considered various capacities from 5MW to 30MW.

The size of the plant would be governed by releases required for losses (evaporation and irrigation) between Wivenhoe Dam and Mt Crosby Weir, water requirements for Brisbane's daily water demand at the weir, and flow over the weir required by the Clean Water Act. It was intended that operation of the turbine would take advantage of the "free" energy available in these necessary daily releases. Over and above these daily releases, "additional" water would be available from the dam when storage was above Full Supply Level and the spillway was operating. It was envisaged that the plant could operate at rated output for 24 hours a day during such periods.

After consultation with the Snowy Mountains Engineering Corporation a 30MW plant was recommended and costing and design was based on a 30MW plant.

29.2 Outlet Works Arrangement for Hydro-Electric Power Station

It was determined that a 3.6 m dia inlet penstock was required for a 30MW power plant. As it was proposed to build this power station at some indefinite future date, the penstock was blanked off with a semi-ellipsoidal dome welded to the downstream end. Because of future maintenance requirements for the power plant and the necessity to close off the flow in the 3.6 m dia penstock, a 1.905 m dia branched offtake from the penstock was provided to maintain releases to the river during such maintenance.

The most economical arrangement for the powerstation penstock was to install it in the left hand wall of the spillway training and retaining wall, immediately beneath the separate 1.905 river outlet, so that one fixed wheel gate, guides etc, would serve both the river outlet and the powerstation penstock.

The arrangement of the outlet works is shown in Figure 120. Further information on the outlets is included in Section 22 of this report.

29.3 Effects on Downstream Sites

A study, consisting mainly of hydraulic calculations, was undertaken to assess the effects of suddenly releasing a full load flow through the turbine on the river downstream of the dam. A thorough site inspection, of the river downstream of Wivenhoe Dam to Fernvale Bridge, was carried out, with particular attention being paid to existing picnic areas and swimming holes.

In this section of the river the three areas used for swimming by the public are:-

(a) an area near the Lowood flood warning gauge (designated Swimming Hole No. 1) about 9 km downstream of the dam;
(b) an area just upstream of the twin bridges on the Wivenhoe Pocket Road (designated Swimming Hole No. 2), about 14 km downstream of the dam; and
(c) the Fernvale Bridge picnic area (designated Swimming Hole No. 3), about 15 km downstream of the dam.

If a 30 MW plant was installed, flow through the turbine would vary from zero to full flow of 100 m^3/s, assuming the turbine operated for peak load supply. The worst case would be for one peak per day, when the mean demand of approximately 1050 ML would pass through the turbine in 2.9 hours. For calculation purposes a flow of 100 m^3/s for 2.9 hours with instantaneous increase and decrease of flow was adopted.

Pools downstream of the dam act as reservoirs when there is a sudden increase in flow so there is much attenuation of the hydrograph as it proceeds downstream.

At a point 5½ km downstream of the dam there is a constriction in the river channel which forms a suitable weir site for pondage to regulate the turbine outflow. This weir would have a crest level of EL 25.5 m (approximately 3 m high from existing bed level to crest), and would consist of dumped rocks large enough to remain in place during large floods, or alternatively would consist of smaller rocks held in place by a mesh of reinforcing bars. If the natural porosity of the rockfill was such that the weir could empty within the time cycle of operation, there would be no need for a concrete outlet culvert. If the culvert was installed the estimated cost (1976) would be approximately $100,000 including provision of bank protection. Mr J Barry Cooke, the American consulting engineer, advised that rockfill weirs without culverts had proved satisfactory in the United States. The cost of such a weir without an outlet culvert would be about $70,000 (1976).

The inflow hydrograph resulting from the turbine outflow was routed down the river using reservoir routing and the Muskingum method both with and without the pondage weir. Table 40 shows the results, the figures in brackets representing values without a weir.

TABLE 40 -EFFECTS OF PEAK LOAD RELEASES DOWNSTREAM OF WIVENHOE DAM.

Swimming Hole Number	Initial Rate of Rise Metres/hour	Maximum Rate of Rise Metres/hour	Maximum Rise metre	Maximum Flow m^3/s
1	0.37 (0.6)	0.70 (1.1)	1.2 (2.0)	47 (94)
2	0.15 (0.3)	0.15 (0.4)	0.5 (1.0)	34 (87)
3	0.15 (0.3)	0.15 (0.4)	0.5 (1.0)	33 (84)

The effects of a flow of 100 m^3/s through the turbine for 2.9 hours on Mt Crosby Weir was also calculated. It was found that storage in the river channel between Wivenhoe Dam and Mt Crosby Weir attenuated the hydrograph so that its peak at Mt Crosby was 21 m^3/s and the base was spread out over 24 hours. To avoid spill, Mt Crosby Weir would have to be drawn down by 300 mm, assuming uniform drawoff, or alternatively the weir crest would have to be raised by 300 mm.

29.4 Further Reassessment Required

If it is decided in the future to proceed with the installation of the turbine, the situation should be reassessed as the winning of gravels from Borrow Areas WX 14 and WX 15 for construction of Wivenhoe Dam has created two pools downstream of the dam - one immediately downstream of the tailrace channel and one about 800 metres downstream. These pools would further attenuate the hydrograph and construction of a weir about 1½ km downstream of Wivenhoe Dam may be all that is necessary to minimise risks at downstream swimming holes and picnic areas.

29.5 Further Details on Hydro-Electric Plant

Further details on the proposed 30MW Hydro-Electric power plant are included in the "Interim Report on the Proposed Hydro-Electric Power Station at Wivenhoe Dam on the Brisbane River" (Reference 5).

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APPENDIX A

APPENDIX A

DIMENSIONS OF PRINCIPAL FEATURES

RESERVOIR

Full supply level (FSL)	EL 67
Full supply level for design purposes (Potential FSL)	EL 68.5
Minimum operating level for pumped storage	EL 49
Minimum operating level for water supply	EL 34
Probable Maximum Flood inflow peak	15 000 m^3/s
Probable Maximum Flood level	EL 77
Maximum reservoir level for an 1893 type flood	EL 73
Reservoir resumption level	EL 75
Storage at FSL	1 150 000 ML
Temporary flood storage (EL 67 to EL 77)	1 450 000 ML
Reservoir surface area at FSL	10 820 ha
Catchment area	7 020 km^2

EMBANKMENT

MAIN EMBANKMENT

Original river bed level	EL 23
Lowest foundation level	EL 20
Embankment Crest level - Nominal	EL 79
Crest level including Crash Barrier	EL 79.7
Maximum embankment height	59 m
Crest length	1 210 m
Earth and rockfill	3 014 348 m^3

LEFT EMBANKMENT

Lowest foundation level	EL 60
Embankment Crest level	EL 79
Crest level including Crash Barrier	EL 79.7
Crest length	1 050 m
Earth and rockfill	984 003 m^3

GROYNES

Rockfill	115 467 m^3

SADDLE DAMS (NO.1 AND NO. 2)

Maximum height above foundations	10 m
Crest level	EL 80
Combined crest length	430 m
Earth and rockfill	61 368m^3

TOTAL EARTH AND ROCKFILL QUANTITIES

Embankment - Earth and Rockfill	3 998 351 m^3
Groynes - Rockfill	115 467 m^3
Saddle Dam No. 1 - Earth and Rockfill	29 845 m^3
Saddle Dam No. 2 - Earth and Rockfill	31 523 m^3
TOTAL EARTH AND ROCKFILL	**4 175 186m^3**

CREST LENGTH

Total Crest length	2.3 km

SPILLWAY

Number and size of radial spillway gates	5 gates each 12 m wide x 16.6 m high
Level of fixed concrete crest	EL 57
Width of gate piers	3.5 m
Length of approach channel	520 m
Bed level of approach channel	EL 48.0
Bed level of low level approach channel	EL 32.0
Length of discharge channel	400 m
Bed level of outlet channel	EL 26.0
Width of spillway excavation at bed level	74 m
Level of flip bucket lip	EL 45
Lowest level of plunge pool floor	EL 17
Spillway discharge for Probable Maximum Flood	11 700 m^3/s
Spillway discharge for design flood	5 000m^3/s
Spillway discharge at FSL all gates open	3 600 m^3/s
Tailwater level for Probable Maximum Flood	EL 53
Volume of concrete in spillway	124 984 m^3
Volume of spillway excavation	1 962 766 m^3

INLET AND OUTLET WORKS

Height of intake structure	47 m
Outlet penstocks	one 3.6 m dia. and one 1.9 m dia.
Length of each penstock	Approx. 60 m
River outlets	2 x 1 500 mm dia. fixed cone dispersion valves
Guard gate	One winch operated wheeled gate 5.25 m x 4.1 m
Selective withdrawal moveable baulks	6 winch operated baulks 6.3 m x 6.3 m
Intake sill level	EL 33
Width of intake channel	15 m
Invert level of intake channel	EL 32

CONCRETE QUANTITY

Total volume of concrete	130 160 m^3

EXCAVATION QUANTITIES

Embankment excavation	870 149 m^3
Groyne excavation	23 941 m^3
Diversion channel excavation	306 350 m^3
Spillway excavation	1 962 766 m^3

APPENDIX B

20 **BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT**
Final Draft Report on Somerset Dam - Dam Failure Modes.
June 1994 *(Aqua)*

21a **BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT** VOLUME I
Final Draft Report on Somerset Dam
- Dam Failure Analysis.
July 1994 *(Gold)*

b BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT VOLUME II
Appendix A Flood Height Profiles and Inundation Maps.
July 1994 *(Gold)*

c BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT VOLUME III
Appendix B Time Series Plots.
July 1994 *(Gold)*

d BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT VOLUME IV
Appendix C Somerset Dam to Wivenhoe Dam Hydraulic Model Rubicon Data Files.
July 1994 *(Gold)*

22 **BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT**
Final Draft Report on Wivenhoe Dam - Dam Failure Modes.
July 1994 *(Grey)*

23a **BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT** VOLUME I
Final Draft Report on Wivenhoe Dam Hydraulic Model Calibration.
October 1994 *(Tan)*

b BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT VOLUME II
Appendix A Volume of A3 Figures
October 1994 *(Tan)*

c BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT VOLUME III
Appendix B Derivation of Wivenhoe Dam Discharges
October 1994 *(Tan)*

d BRISBANE RIVER SYSTEM HYDRAULIC MODEL REPORT VOLUME IV
Appendix C Cross-sectional Data
October 1994 *(Tan)*

APPENDIX C

APPENDIX C

MAJOR CIVIL CONTRACTS

Designs and specifications were produced for the major civil contracts listed below.

Contract No.	Contract	Contractor	Commencement Date	Completion Date
2103	Excavation for Diversion Channel and Dam Embankment	Thiess Bros Pty Ltd	March 1977	September 1977
2108	Foundation Grouting	SIF Enterprise Bachy	July 1977	April 1979
2111	Excavation, Processing and Stockpiling of Gravels	W. Wall & Sons	October 1977	16 March 1982
2112	Second Stage Construction	Thiess Bros Pty Ltd	January 1978	February 1979
2120	Third Stage Construction	Wivenhoe Constructions (A Thiess-Codelfa Cogefar Joint Venture)	October 1979	November 1983
2122	Manufacture, Inspection, Supply, Delivery, Erection, Testing and Commissioning of one 79 tonne S.W.L. Bulkhead Gate Gantry Crane for Wivenhoe Dam.	Evans Deakin Industries	February 1981	September 1986
2123	Fabrication, Delivery and Erection of Five Radial Gates, One Bulkhead Gate, Ten Hydraulic Winches and Associated Control Equipment	Samsung Heavy Industries Co Ltd	August 1982	September 1985

APPENDIX D

APPENDIX D

TIME CONSTRAINTS

The principal time constraints imposed on the Contractors by the Specifications were as listed below. Contract titles are listed in Appendix B.

CONTRACT NO.	CONTRACT REQUIREMENT	REMARKS
2103	Excavation of diversion channel and left bank to be complete by 30 September 1977.	A delay in completion would mean that the grouting contractor would have insufficient time to grout the low-lying foundations before the 1977/78 wet season.
2108	Grouting of the foundations for the sloping core embankment and diversion channel to be complete by 11 March 1978.	A delay would have held up diversion of the river and a start on the second stage construction. (Contract 2112)
	Grouting of the river bed to be complete by 30 June 1978.	The river bed had to be covered by core and filter materials to a safe height for the subsequent 1978/79 wet season by the contractor for Contract 2112.
2112	Excavation in the main river bed to be complete by 20 May 1978.	To enable grouting of foundation to be completed under Contract No. 2108 and to enable fill to be placed to a safe height under Contract 2112 prior to the 1978/79 wet season.
	Main coffer dams to be completed by 1 August 1978 and contract to be essentially complete by 30 November 1978.	Completion required prior to the 1978/79 wet season, to enable Third Stage construction to commence in February 1979. (However a flood occured in April 1978 and delayed work. Contract No. 2112 was not completed until February 1979 and this caused possession of site date for Third Stage construction to be deferred from 5.2.79 to 3.9.79, and planned completion of Third stage construction to be deferred from 30.6.82 to 30.6.83).
2120	Embankment over original river bed not to be raised above its second stage level of EL 42 before 30 April 1980.	This was necessary to enable sufficient capacity for large floods to pass over the protected embankment if the diversion capacity of the right bank diversion channel was exceeded.

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CONTRACT NO.	CONTRACT REQUIREMENT	REMARKS
2120 (continued)	Excavation of the 15 m wide spillway diversion channel to El 32, and concreting of walls in spillway to a safe height (min El 40) to be complete by 30 November 1980.	This made extra diversion capacity available allowing fill to be placed over the river bed section.
	Construction of the dam embankment between the spillway and the right bank diversion channel to at least El 56 by 30 November 1980.	This was necessary to enable immunity from overtopping of the main embankment during the 1980/81 wet season except where protected by steel mesh. (Six weeks were lost in March-April 1981 due to industrial action and a consequent Industrial Agreement limited working hours to 48 hrs/week).
	Closure of the right bank diversion channel was required no earlier than 1 April 1982 and no later than 1 May 1982.	Closure was required to be as early as possible after the wet season to enable time for raising the main embankment before the 82/83 wet season.
	Works to be completed prior to closure of the Diversion Channel: All other earthworks to El 74; the spillway side and training walls; all concrete in the spillway flip and monoliths to at least El 48 except for the final stage concrete in the spillway diversion; penstocks and river outlets painted.	
	Embankment over the right abutment and diversion channel to be constructed to at least El 68 by 1 September 1982.	This was designed to provide reasonable immunity against overtopping during the 82-83 wet season.
	Closure of the spillway diversion not to commence until the embankment over the right abutment and diversion channel has reached El 62.	This enables reasonable immunity against overtopping.
	Closure of the spillway diversion and placement of all spillway concrete was specified for completion by 1 December 1982. This target was not achieved. The spillway concrete was completed on 22 August 1983.	Failure to achieve this target could mean failure to store runoff during the 1982/83 wet season up to El 57.
	All works under Contract No. 2120 were to be completed by 30 June 1983.	Contract No. 2120 was not completed until November 1983.

CONTRACT NO.	CONTRACT REQUIREMENT	REMARKS
2120 (continued)		(Floods occured in May and June 1983. The June 1983 flood rose to El 59.78 and overtopped the incomplete spillway. The outlet valves were installed on 25.8.83 and storage commenced. The last concrete was placed on 31.8.83, and Contract No. 2120 was completed on 16 November 1983.)
2122	Erection, testing and commissioning of 79 tonne gantry crane to be complete by 22 December 1982. This target was not achieved.	The crane was to be used to erect the bulkhead gate for the radial gate contractor. The Certificate of Final Completion was issued on 1 September 1986.
2123	Two radial gates (Bays S2 and S4) to be installed and operable by 2 January 1984 and all gates installed and operable by 30 June 1984.	To enable full storage and flood mitigation capacity. The Certificate of Practical Completion was issued on 30 September 1985.