EHP

The Instutution of Engineers, Australia

Australian Historic Engineering Plaquing Programme

Commemorative Plaque Nomination

Historic Engineering Marker

Canning Dam

June 1998

Western Australia Division Engineering Heritage Panel

CANNING DAM PLAQUE NOMINATION

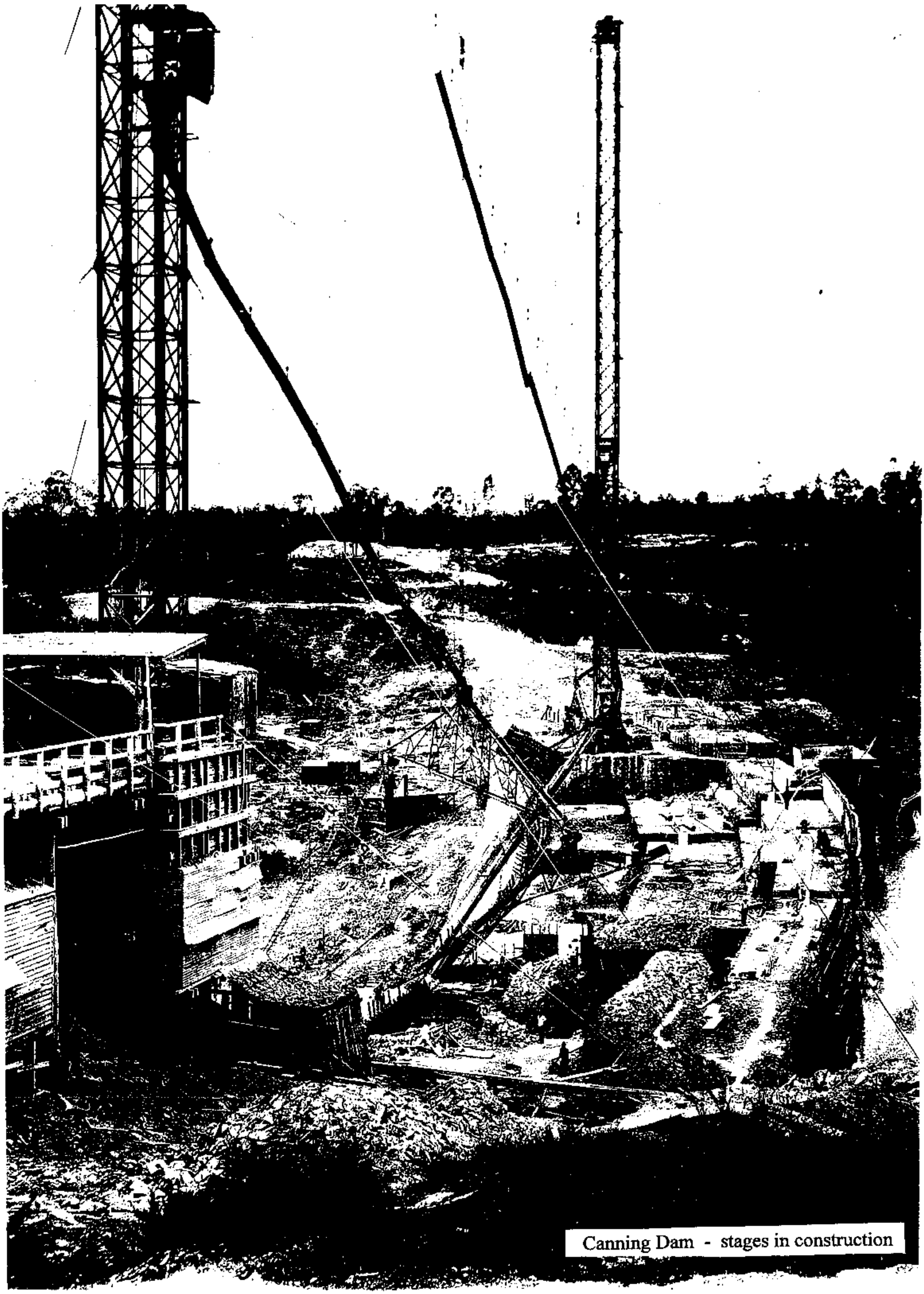
Additional Supporting Information

Physical Condition- The dam was designed as a high mass concrete gravity dam following principles generally adopted for dam construction at the period.

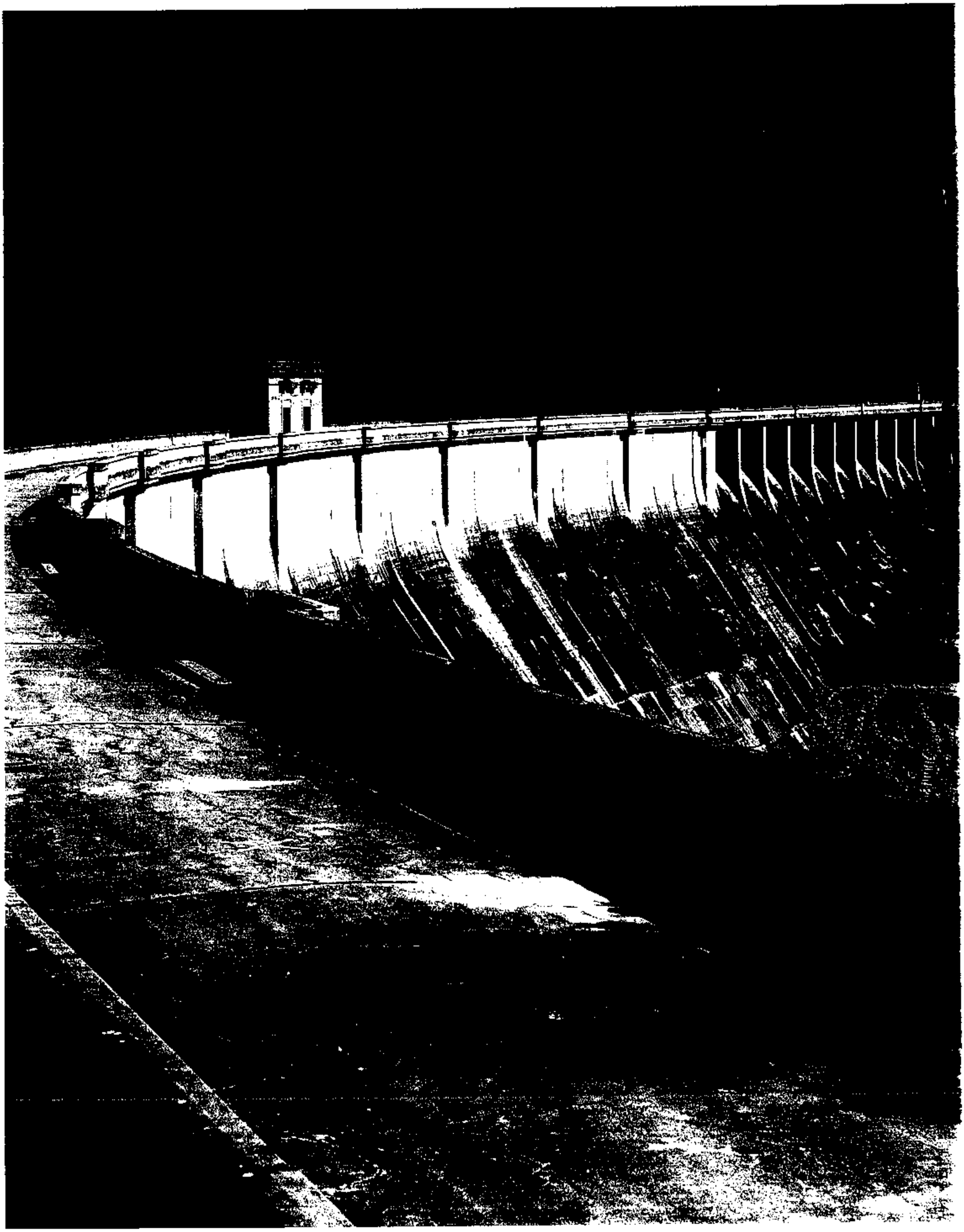
The dam remains substantially as originally constructed and is still used for its original purpose of providing the water supply for Perth. The condition of the structure however, has been affected by problems of movement and alkali/aggregate reaction appearing at the top section of the dam wall.

Future Care and Maintenance of the Work- Canning Dam is a major component of the water supply system for Perth and the metropolitan region. It currently supplies approximately 15% of total metropolitan supply requirements. The dam is scheduled for major work to strengthen and repair the wall, commencing in early 1999. A conservation plan has been prepared. Care and maintenance of the dam is assured into the foreseeable future.

Technological/Scientific Value-	See attachment No 1, page 133 para .4, page 134 para 5.4
Historical Value-	See Attachment No 1, page 132 para 5.3, page 133 para 5.3 and Attachment No 2, three pages
Social Value-	See Attachment No 1, page 134 para 5.5
Landscape or Townscape Value-	See Attachment No 1, page 131 para 5.2
Rarity-	See Attachment No 1, page 134 para 5.6, page 135 para 5.6
Representatives-	See Attachment No 1, page 135 para 5.7
Contribution to the Nation or Region-	See Attachment No 2, three pages
Contribution to Engineering-	See Attachment No 3, pages 68 and 69
Persons Associated with the Work-	See Attachment No 4, pages 63-65
Integrity and Authenticity-	See Attachment No 1, page 135 para 5.8
Comparable Works-	See Attachment No 5, pages 65-67 para 2.7



Canning Dam - stages in construction



Commemorative Plaque Nomination Form

Date: JUNE 24 1998

To:
Commemorative Plaque Sub-Committee
The Institution of Engineers, Australia
Engineering House
11 National Circuit
BARTON ACT 2600

From: W.A. DIVISION ENGINEERING HERITAGE PANEL
Nominating Body

The following work is nominated for a:-

- National Engineering Landmark
- Historic Engineering Marker

*(delete as appropriate)

Name of work: CANNING DAM

Location, including address and map grid reference if a fixed work: ON CANNING RIVER APPROX 33 KM SOUTHEAST OF PERTH.

Owner: WATER CORPORATION OF WESTERN AUSTRALIA

The owner has been advised of the nomination of the work and has indicated (attach a copy of letter if available): AGREEMENT WITH PROPOSAL

Access to site: CROYDEN ROAD OFF BROOKTON HIGHWAY

Future care and maintenance of the work: ASSURED (SEE ATTACHED PAGES)

Name of sponsor: N/A.

For a NEL, is an information plaque required? N/A.

H.E. HUNT
Chairperson of Nominating Committee

W. LARK
a/Chairperson of Division Heritage Committee/Panel

ADDITIONAL SUPPORTING INFORMATION

Name of work: CANNING DAM

Year of construction or manufacture: 1933 to 1941

Period of operation: C 1939 To PRESENT

Physical condition: SEE ATTACHED PAGES.

Engineering Heritage Significance:-

Technological/scientific value

Historical value

Social value

Landscape or townscape value

Rarity

Representativeness

Contribution to the nation or region

Contribution of engineering

Persons associated with the work

Integrity

Authenticity

Comparable works(a) in Australia

(b) overseas

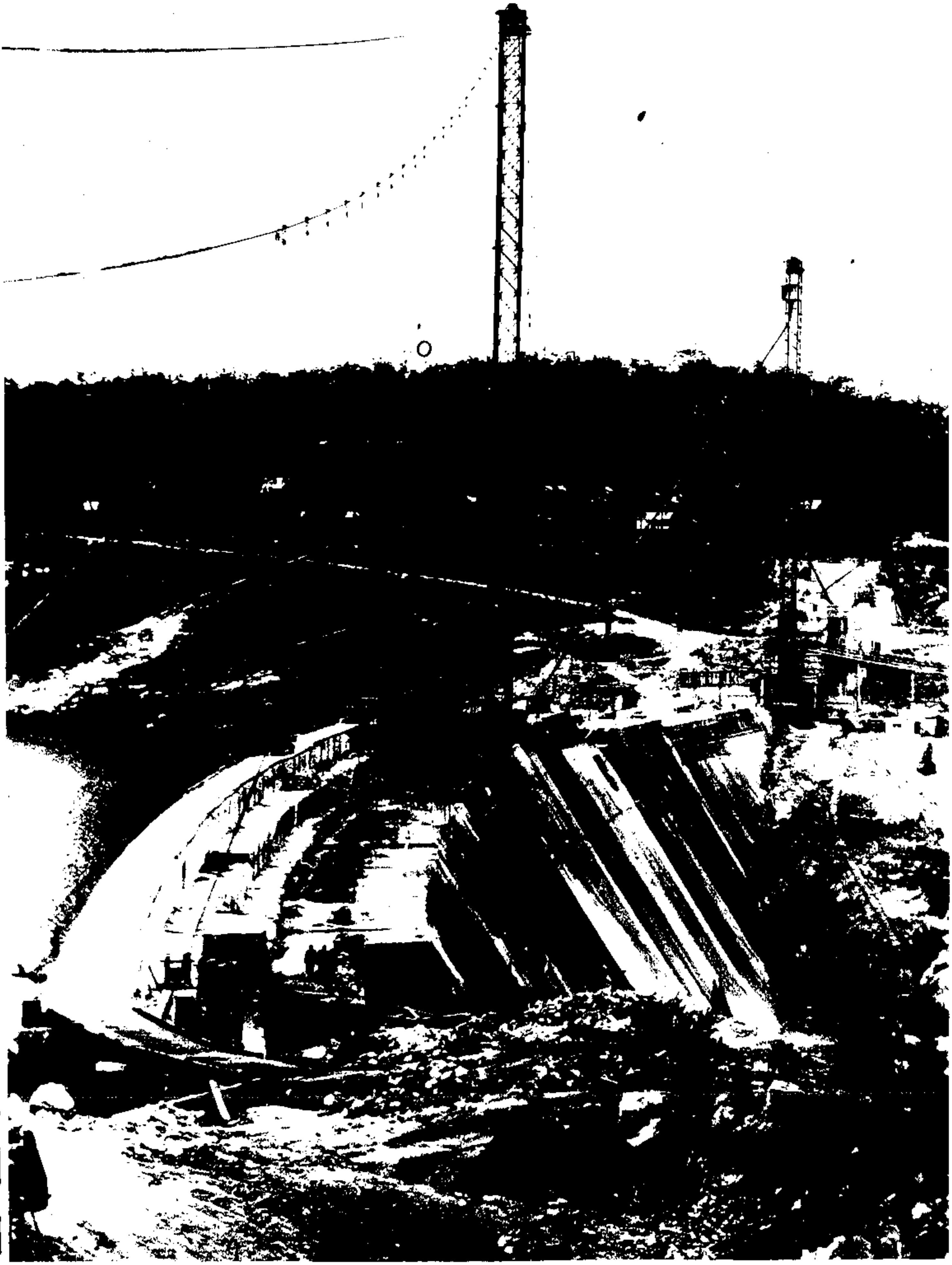
SEE ATTACHED PAGES

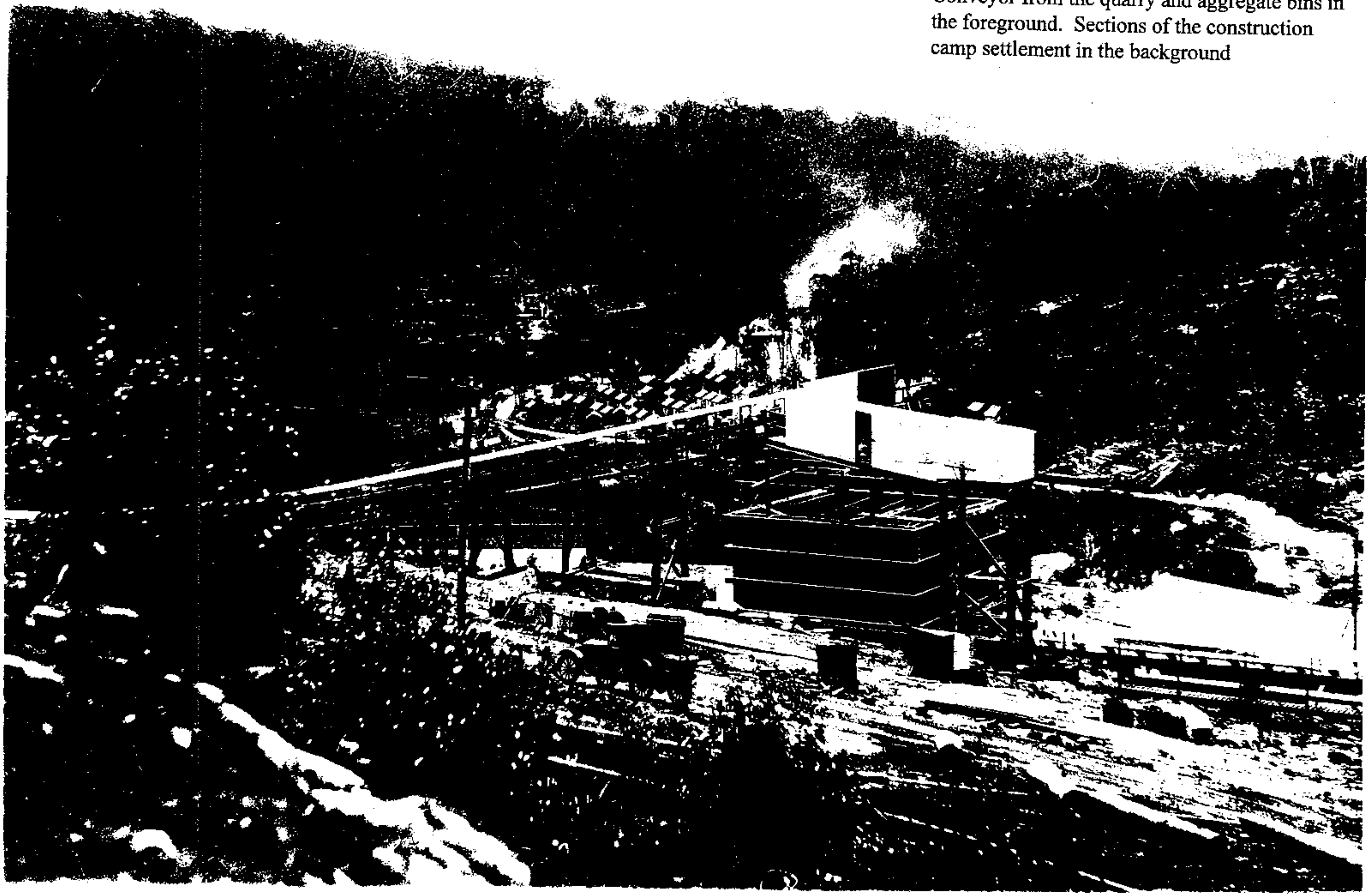
Statement of significance, its location in the supporting documentation

Citation (70 words is optimum)

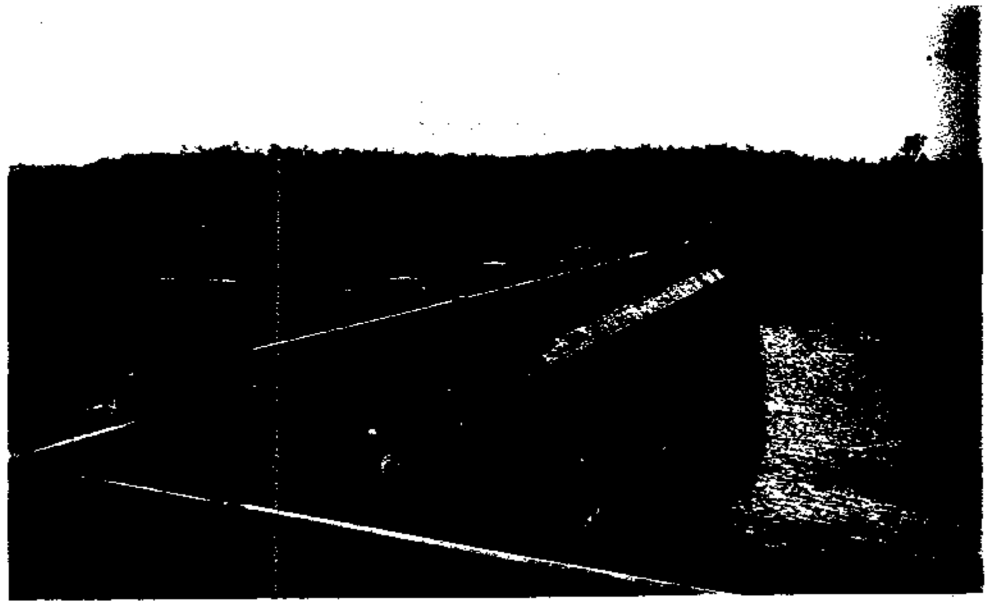
Attachments to submission (if any): 8 ATTACHMENTS

Proposed location of plaque (if not at site): To BE DECIDED (BUT AT SITE)





Conveyor from the quarry and aggregate bins in the foreground. Sections of the construction camp settlement in the background



Canning Dam opening ceremony 1940

The spillway last flowed in 1974

ATTACHMENTS

<u>List of Attachments</u>

ACKNOWLEDGEMENT

Attachments numbered 1 to 8 were taken from the

'Canning Dam Conservation Plan

Assessment of Significance and Conservation Policy'

prepared for the Water Corporation May 1998 by

HERITAGE AND CONSERVATION PROFESSIONALS

First Floor Rear, Murray Mews, 329 Murray Street Perth WA 6000

Postal Adress :- P.O. Box 7576 Cloisters Square, Perth, 6850

5.0 ASSESSMENT OF SIGNIFICANCE

5.1 PREAMBLE

The *Burra Charter,* (Australia ICOMOS, 1992) defines 'cultural significance' to mean 'aesthetic, historic, scientific or social value for past, present or future generations...The places that are likely to be of significance are those which help an understanding of the past or enrich the present, and which will be of value to future generations'. It is stressed that the terms 'aesthetic', 'historic', 'scientific' and 'social' are not mutually exclusive and that, for example, 'architectural style has both historic and aesthetic aspects'.[139]

The significance of Canning Dam is assessed in accordance with requirements established in *Criteria for entry into the Register of Heritage Places.* (1996, Heritage Council of WA.) (Appendix B) This employs the terms outlined by the *Burra Charter* and the degree of significance is determined by the level of rarity and representativeness. Significance is also affected by the condition, integrity and authenticity of the place.

5.2 AESTHETIC SIGNIFICANCE

Criterion 1. *It is significant in exhibiting particular aesthetic characteristics valued by the community.*

Importance to a community for aesthetic characteristics. (Criterion 1.1)

Canning Dam including the dam structure and environs is of aesthetic significance to the community as a destination for outings and a venue for picnics and passive recreation. The place has been valued by the community for its aesthetic qualities from the time of its construction to the present.

Importance for its creative, design or artistic excellence, innovation or achievement. (Criterion 1.2)

Canning Dam is of aesthetic significance for its innovative structural and hydraulic design. The elegant simplicity of the curved concrete dam wall and the use of Inter-War Stripped Classical detailing on the gate house, piers and balustrade give the structure a visual quality which belies the achievement of its engineering design. The structural design of the dam is considered today to represent a high degree of technical excellence.

Importance for its contribution to the aesthetic values of the setting demonstrated by a landmark quality or having impact on important vistas or otherwise contributing to the identified aesthetic qualities of the cultural environs or the natural landscape within which it is located. (Criterion 1.3)

Canning Dam contributes to the aesthetic value of its setting because it has created a distinct and comparatively rare landscape comprising the juxtaposition of the elegant engineered form of the concrete dam wall, a large open lake of impounded water with its rugged, largely natural landscape of the enclosing valley and the steep-sided valley landscape downstream of the dam structure.

The public roads, including McNess Drive from Araluen to south of the dam contribute to the aesthetic qualities of the place by facilitating access to and views of

139 *Ibid.*

it while at the same time, providing a valued landscape experience by virtue of their small, human scale, close integration with the natural landform and enclosure by the surrounding forest.

In the case of an historic precinct, importance for the aesthetic character created by the individual components which collectively form a significant streetscape, townscape or cultural environment. (Criterion 1.4)

5.3 HISTORIC SIGNIFICANCE

Criterion 2. *It is significant in the evolution or pattern of the history of Western Australia.*

Importance for the density or diversity of cultural features illustrating the human occupation and evolution of the locality, region or the State. (Criterion 2.1)

The construction of Canning Dam is of particular significance in the context of the evolution of history in Western Australia because it solved the problem of Perth's water supply. It ended the period of unsatisfactory supply in both the quantity and quality of water provided to the city. It was the main source of water from 1940 to 1961 and remains essential to the city's water supply service today.

Canning Dam is of historic significance as a place of scenic and passive recreation from the time of its construction to the present. The overall site layout, including the dam wall, roads and quarry, remain largely in their original form and as such illustrate the construction and original development of the site primarily as a water supply facility but also as a tourist destination.

The former Engineer's house and garden and the few other surviving remnants of the former settlements have the potential to illustrate aspects of human occupation associated with the development of the place.

Importance for close association with an individual or individuals whose life, works or activities have been significant within the history of the nation, State or region. (Criterion 2.3)

Canning Dam is of historic significance for associations with those prominent engineers closely associated with its design and development. In particular:

Russell John Dumas who designed Canning Dam and directed the majority of the construction works.

Donald Campbell Munro ~~resident~~ senior assistant engineer on Canning Dam.

Victor Cranston Munt hydraulic engineer under Dumas and resident engineer on Canning Dam.

Thomas Cowley Hodgson who recommended the site where Canning Dam was eventually constructed.

Frederick Washington Lawson who prepared the Hills Scheme for the supply of water for Perth.

Canning Dam is also of historic significance for associations with the prominent Western Australian benefactor Sir Charles McNess whose financial assistance permitted the upgrading of public access to the place by the State Gardens Board.

Importance as an example of technical, creative, design or artistic excellence, innovation or achievement in a particular period. (Criterion 2.4)

The design and construction of Canning Dam are of historic significance for their innovation and technical achievement. The design of Canning Dam was considered to be at the forefront of concrete gravity dam design at the period, based on the latest information from both Britain and the USA as well as from elsewhere in Australia. The design of the dam incorporated a number of innovative features including:

- The use of weight batching of concrete materials and the use of bulk cement
- Internal pressure release drains connected to inspection galleries
- Water scabbling and sandblasting of horizontal construction joints and keyed vertical construction joints.
- Compaction of concrete by internal vibrators
- Allowance for hydraulic uplift in the design
- Foundation cut-off trench and pipe and stone drain to minimise seepage at concrete-rock interface
- Use of conveyors for aggregate movement from quarry to batching plant

  The most advanced use of concrete placement by tower and chute in Australia

5.4 SCIENTIFIC SIGNIFICANCE

Criterion 3A *It has demonstrable potential to yield information that will contribute to an understanding of the natural or cultural history of Western Australia.*

Importance for information contributing to a wider understanding of natural or cultural history by virtue of its use as a research site, teaching site, type locality, reference or benchmark site. (Criterion 3.1)

Canning Dam can be considered a benchmark site in the history of the provision of Perth's water supply and in the history of concrete gravity dam construction in Western Australia.

Remnant indigenous landforms and vegetation on the site area have some scientific significance as an example of pre-development landscape of the site and its surrounding area.

Importance for its potential to yield information contributing to a wider understanding of the history of human occupation of the locality, region or the State. (Criterion 3.2)

The archaeological and documentary evidence relating to the site indicates the Canning Dam site has potential to yield information on:

- the first use of new concrete batching technology in Western Australia.
- the effect the ready supply of cheap labour had on building techniques utilised during the building of the dam.
- the material culture and social display of dam workers and their families from different social backgrounds living together in a relatively isolated community.
- the social and living conditions experienced by subsistence workers during the depression.

Criterion 3B *It is significant in demonstrating a high degree of technical innovation or achievement.*

Importance for its technical innovation or achievement. (Criterion 3.3)

Canning Dam is of scientific value for its contribution to the understanding of dam construction in the 1930s. This value is increased because extensive physical evidence of the construction process remains extant.

5.5 SOCIAL SIGNIFICANCE

Criterion 4 *It is significant through association with a community or cultural group in Western Australia for social, cultural, educational or spiritual reasons.*

Importance as a place highly valued by a community or cultural group for reasons of social, cultural, religious, spiritual or educational associations. (Criterion 4.1)

Canning Dam is of social significance for those associated with its design and construction and particularly those who lived in townsite that developed in association with the project from 1933 to 1940.

Canning Dam is of social significance for associations with the sustenance work programme in the 1930s. The project was planned to be labour intensive and as a result a large number of people in Perth had associations with it.

The landscape and contextual setting of Canning Dam is of social significance to the generations of Western Australians who have enjoyed it as a scenic destination for day trips and as a place for passive recreation.

Importance in contributing to a community's sense of place. (Criterion 4.2)

Canning Dam contributes to a sense of place for those closely associated with the community that developed around its construction and the friendships that endured in the wider community beyond that period.

5.6 RARITY

Criterion 5 *It demonstrates rare, uncommon or endangered aspects of the cultural heritage of Western Australia.*

Importance for rare, endangered or uncommon structures, landscapes or phenomena. (Criterion 5.1)

Canning Dam is significant as an uncommon example of the construction of a large scale engineering project and associated access roads and site works with minimal disturbance to the surrounding landscape downstream of it.

Whilst there are a number of other dams in Western Australia the size and complexity of Canning Dam sets it apart.

Importance in demonstrating a distinctive way of life, custom, process, land-use, function or design no longer practiced in, or in danger of being lost from, or of exceptional interest to, the locality, region or the State. (Criterion 5.2)

Canning Dam represents the use of design principles and a method of construction considered advanced in its time but no longer practiced today.

Canning Dam represents a large scale engineering project constructed in the period before World War II involving the establishment of a townsite and associated community of workers involved in the construction process. This represents a way of life no longer practiced on major engineering projects in Australia.

5.7 REPRESENTATIVENESS

Criterion 6 *It is significant in demonstrating the characteristics of a class of cultural places or environments in the State.*

Importance in demonstrating the principal characteristics of a range of landscapes or environments, the attributes of which identify it as being characteristic of its class. (Criterion 6.1)

Canning Dam is a representative example of a concrete gravity dam constructed in the inter-war period. In the Western Australian context however its size and design excellence distinguish it from other dams in its class.

Importance in demonstrating the principal characteristic of the range of human activities (including way of life, philosophy, custom, process, land-use, function, design or technique) in the environment of the locality, region or the State. (Criterion 6.2)

5.8 CONDITION, INTEGRITY AND AUTHENTICITY

The dam remains substantially as originally constructed and is still used for its original purpose of providing the water supply for Perth. The condition of the structure however has been effected by problems of movement and of alkali/aggregate reaction appearing in the top section of the dam wall.

Attachment No 2

CANNING DAM

History of Development

The first reticulated water supply for Perth was provided by the City of Perth Waterworks Company. An elaborate opening in October 1891 was quickly overshadowed by the angry protests of ratepayers and members of the Perth City Council. This hostility between the Company and its customers erupted due to the company's handling of reticulation and house connections: these manifestations were symptoms of deeper problems including :-

- The population was increasing rapidly due to gold discoveries, the scheme was outgrown and inadequate by the time it was opened;
- The scheme was for Perth only; supplies to Fremantle, Claremont and Midland Junction were provided and managed by the PWD some 15 years later;
- Initially the Government saw the provision and management of the water services as in the province of local government as was the case in England. It was reluctant to become involved directly;
- Many were excluded from the services. Typhoid was rampant and waste disposal was primitive;
- The legislation was inadequate particularly as to health measures;
- Assertions by the Premier and others to the effect that the Company was on a bonanza, were mischievous and completely untrue.

By September 1896 the Government had no alternative but to acquire the waterworks and to set up a Board of four, with Edward Keane as Chairman.

1896 was a drought year and many of the people of Perth were to suffer mercilessly, with sections of the reticulation on the city margins being without water for weeks. The newly established Metropolitan Water Works Board was active. Contrary to the expectations of the Government Geologist, it established an artesian bore on its office land in Wellington Street. In its 1897 Annual Report, the Board referred to the artesian supply as an expedient- 'horrid bore-water characteristically ill smelling'. A system of bores and pumping might overcome the difficulty for a year or two but the suffering public had to see the light of a new dawn-

> "By building the proposed new reservoir on the Upper Canning the difficulty would be solved we can safely say, for our time or until the city could well afford to increase it...."

By its energies and example, the Board had done much to mitigate impending disaster, but unfortunately it had also spent prodigally and unwisely. It was dumped unceremoniously and replaced by the Traylen Board which survived from 1898 to 1904. Thereafter, successive Boards were administered by the Minister for Works or the Engineer-in-Chief until 1910 when it was administer as a department for many years. In his term as Minister, Mr Wilson MLA, did his best to replace "the horrid bores" with the Upper Canning Dam but he was denied when funds were unavailable.

Significant events in the evolution of a municipal water supply from an artesian water (of high mineral content, high temperature and often ill-smelling) to a supply ex-Canning Dam include:-

- Engineer-in-Chief, C Y O'Connor recommended the continuance of the use of artesian water since the community could not afford the expense of a good quality supply; likewise, he demonstrated that sewage disposal to the ocean was prohibitive financially and this led to the adoption of a two pan disposal system, on the recommendation of Traylen.

- Palmer, the next Engineer-in-Chief, appears (in common with many of his contemporaries) to have been overawed by the emerging theories of water pollution. He favoured the use of artesian water which was drawn from aquifers free from human pollution. By way of antithesis, he was a champion of the septic tank, as used at Burswood some years later.

- Palmer contributed usefully by envisioning a water supply for the whole metropolitan area with a population growth from 58,000 to 100,000.

- The next contributor was J Davies MICE MIMechE, the Under Secretary for Works NSW, who reported in 1905. He raised his sights and planned for a population of 200,000 and with a per capita consumption of 50 GPHD. He proposed staged development up to 4 MGD of artesian water and thereafter, a storage on the Canning to provide a further 5 MGD.

- It is noteworthy that Davis observed the daily per capita consumption in Perth was one third that of the other Australian Capital Cities at that time.

- Two of the briefing papers handed to Davis are revealing. The President of the Central Board of Health advocated artesian water be used for the reasons given by Palmer above. The Government Analyst preferred upland surface water for its lower dissolved solids but was nevertheless adamant that it should be treated by filtration.

- In August 1904, a deputation of Perth City Councillors met the minister deploring the want of a sewerage scheme, complaining of the unpalatable bore water and advocating a diversion of Mundaring water to the city.

- In July 1906, Oldham opposed the use of Mundaring water and spoke of some improvement in the quality of the new bores at Leederville.

- The Australia wide drought of 1914 brought a new urgency to water supply planning. Lawson was uneasy that so much reliance was placed on the artesian source when its provenance was unknown. Post war, in an environment of emergency he had to look to expedients. The Government imported Ritchie from Victoria who, in a whirlwind 14 day visit unhelpfully recommended for an earth fill, rock fill dam on Kangaroo Gully, a tributary of the Canning River.

- Lawson proposed Churchman Brook Dam as a first step in a Canning development. Parr built Churchman and supply problems were alleviated by wet winters.

- Dumas built Canning (1934-1941) as an employment- water supply project in the Great Depression using sustenance labour.

- Thus 45 years had elapsed since Keane had made his brash proposal. For the first time, Perth had a water supply befitting a capital city.

Conclusion

Canning Dam was essentially a provision of social significance, nevertheless, it was a structure of importance in the engineering hierarchy. Sydney had its dams built in cyclopean masonry and Adelaide built its Mount Bold Reservoir at about the time of Canning. Mount Bold was technically the superior on account of its low cement content; Canning used concrete placement by chutes which was not best practice, but the dam design was otherwise of high standard for its time.

2.8 SUMMARY OF TECHNICAL INNOVATIONS USED AT CANNING DAM

Several innovative design concepts and construction methods which were new to Australia were introduced on the Canning Dam project, while others which were used on the Wellington dam, (a smaller concrete gravity dam built by the PWD which in 1933 was nearing completion) were improved upon at the Canning dam site.

- Bulk handling of cement was used for the first time in Australia with significant savings over the use of bagged cement. Discussion by Dumas and Munt in the published paper of the problems initially encountered and how they were overcome helped those using bulk handling later to avoid similar problems.[126] Cement was also hauled in bulk by rail for the first time in Australia. It was railed from the cement works at Rivervale to Armadale where the containers were transhipped to road trucks for cartage to the dam site.[127]
- Concrete materials (aggregate, sand and cement) were measured by weight before mixing (that is, were batched by weight) giving much greater control over the concrete quality then the old system of batching by volume. This is believed to have been the first time batching by weight was used in Western Australia.[128]
- The dam wall consists of a series of monoliths made up of concrete blocks each consisting of one day's concrete pour. The blocks were arranged so that no horizontal or vertical joints were continuous between blocks. To obtain a good key for the next lift of concrete a method of spraying water onto the horizontal surfaces of the blocks while the concrete was still green to remove the surface laitance and to expose the aggregate was devised. In some cases, when the next pour was delayed sand blasting was also used to ensure that all grout was fully removed from the exposed aggregate before the next pour.[129]
- At the vertical contraction joints between monoliths, faces of the monoliths were provided with vertical recesses to act as keys_during the pouring of blocks in the adjacent monolith.[130]
- In all dams care has to be taken to prevent water seepage between the foundation rock and the structure of the dam. At Canning in addition to cutting back the granite foundation to solid unfractured rock, a cut-off trench was sited near the upstream face of the dam, down stream of which a rock-

[126] Dumas & Munt,'Construction of the Canning Dam', *JIEAust* Vol. 8, 1936, p.1.

[127] J.A. Ellis, 'Railway Progress in Western Australia 1927-1935', *JIEAust* Vol. 8, 1936, p.24.

[128] W.H.R. Nimmo, "Historical Review of Dams", *ANCOLD*, 1966, p.35. No documentary evidence has been found to clearly prove that weight batching was first used in WA at the Canning dam. As an internationally recognised new technique it may have been used experimentally by the PWD or others in WA before 1934. However, it does not appear to have been used on any major project in WA before the Canning dam. On the Wellington dam concrete was batched by volume. There is a strong professional tradition that it was introduced at Canning and no evidence has been found to the contrary.

[129] V.C. Munt, 'A Decade of Dam Construction', PWDWA, 1940, p.76. The method described in Dumas & Munt (1936, p.10) of scabbling the surface of old concrete with pneumatic tools before placing new concrete on it, was abandoned in 1937 in favour of Munt's procedure.

[130] Another of the 'new features' mentioned by Nimmo *op.cit.*1966, p.35.

filled drain containing open-jointed pipes was provided to cut off any seepage between the rock face and the concrete of the dam.[131]

- The inclusion of an internal drainage system was considered innovative at the period. Near vertical tubular (200 mm) cut off drains were provided at 5ft (1.52m) intervals along the dam to relieve internal seepage through the concrete.[132]
- From March 1938 immersion vibrators were used for the compaction and placement of concrete at the dam, the first Western Australian construction site on which they were used.[133]
- Conveyor belts of the length used to convey the crushed aggregate from the crushing plant to the batching plant (285 m or 935 ft) had seldom if ever been used on construction sites previously in Australia (certainly not in Western Australia) and were considered by Dumas and Munt as being one of the outstanding features of the plant used on the site.[134]
- The use of the concept of hydraulic uplift was used in the design of Canning Dam. The previously constructed Wellington Dam made use of this concept in its design. In Canning Dam the quality of the granite foundations meant that the uplift force was assumed to act over only half of the area of the base of the dam.[135]
- The system of chuting concrete using two high towers to support the chutes, which had been devised at Wellington dam, was repeated at Canning on a much larger scale.

131 Dumas,*op.cit.* 1933, p.290. Nimmo *op.cit.*1966 pp.35-36 notes that the cut-off drain was a 'new feature'but one was also provided at the Wellington dam (designed by B.S. Crimp, the Hydraulic Engineer, and Dumas).

132 Nimmo, *op.cit.*1966 p.35.

133 B. Mahon, *Canning Dam, Dam Data Review Report,* Part 3 – Construction, MWA, 1984, p.29. This volume contains a discussion of the materials and methods used in the dam construction where these affected its long term integrity. Included as appendices are Dumas & Munt's paper (1936), Munt's *A Decade of Dam Construction* (1940), a further lengthy report by Munt and Munro (115 pp) 'Canning Dam. Commenced 18th September 1933 – Completed 17th September 1940'(no date), and the final version of the Specification for the works.

134 Dumas and Munt, *op.cit.* p.5.

135 Doherty *op.cit.* p.13.

2.6 PROMINENT PEOPLE ASSOCIATED WITH CANNING DAM

The following people were associated with the construction of Canning Dam.

2.6.1 Russell John Dumas

Russell John Dumas (1887-1975) was born in South Australia and educated at the University of Adelaide. He graduated in 1909 and joined the South Australian Engineer-in-Chief's Department. After war service in France he returned to the department in 1919 and worked on the construction of the River Murray locks and weirs. In 1925 he joined the Metropolitan Water Supply Sewerage & Drainage Department in Western Australia, and worked on the construction of the Churchman Brook Dam. He transferred to the PWD and worked on the design of the Drakesbrook and Wellington Dams, and the raising of Harvey Weir, under B.S. Crimp, the PWD's Hydraulic Engineer. He was responsible for the design of the Canning Dam and for supervising some of the early work on the site. On the resignation of John Parr in 1934 he became Engineer for Metropolitan Water Supply and Sewerage and directed the remainder of the construction of the Canning Dam. In 1941 he became Director of Buildings and Works (equivalent to the earlier Engineer-in-Chief's position) and also Chief Hydraulic Engineer. After an investigation of the potential of the north-west for closer settlement in 1941-2, he recommended irrigation based on two potential dam sites on the Ord River. He was appointed Chairman of the North-West Development Committee and was a strong advocate for the development of the Ord River Irrigation Scheme. During the war he served as liaison officer between the Allied Works Council and the state government departments. He also served as Regional Controller of Electricity and in 1946 became first Chairman of the State Electricity Commission. After the war he directed the completion of the Stirling Dam (for the Harvey Irrigation Scheme) and also the raising of the Mundaring Weir and Wellington Dam, the headworks for the Comprehensive Agricultural Areas Water Supply Scheme, for which he was largely responsible for obtaining federal funding.

Dumas realised that before extensive industrialisation could occur in Western Australia, the manufacture of certain basic industrial commodities had to be carried out in the state. In 1951-52 he was largely responsible for negotiating the establishment at Kwinana of the Anglo-Iranian Oil Company's oil refinery, BHP's steel rolling mill and the Cockburn Cement works. Because of his crucial role in establishing these industries, his services as Director of Buildings and Works were extended for nearly two years after his retirement age, and he was also given the additional title of Co-ordinator of Works and Industrial Development. He retired in December 1953 but remained influential as an advisor to the Brand-Court government.[108]

2.6.2 Donald Campbell Munro

Donald Campbell Munro was born in Fremantle in 1909 and was educated at Fremantle Boys' School, Perth Modern School and the University of Western Australia. He was an engineering cadet with the Public Works Department and

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108 L.Layman, 'Dumas, Sir Russell John', *ADB* Vol 14; Le Page; D.A. Cumming, 'Biographies of Western Australian Engineers', unpublished first draft, 1991

during university vacations worked on a survey for the Lake Grace - Hyden railway, the raising of Harvey Dam and the building of Drakesbrook Dam. He graduated in 1932 and joined the Hydraulic Engineering Department of the PWD, working under Thomas Munt on the construction of the Wellington Dam and the Canning Dam. In 1934, he transferred to the Metropolitan Water Supply, Sewerage and Drainage Department when supervision of the Canning Dam came under that department. He became Resident Engineer at Canning Dam from 1938 to its completion in 1940. He rejoined the PWD and worked under Munt on the Samson Brook Dam and Stirling Dam, before becoming Resident Engineer on the latter in 1941.

After serving from 1942 to 1945 in the Royal Australian Engineers in which he became a Major, he returned to the PWD as Principal Assistant Hydraulic Engineer under Munt and was project engineer for raising Mundaring Weir which commenced in 1946. He became Hydraulic Engineer in 1955, Deputy Chief Engineer in 1962 and Chief Engineer of the PWD in 1965. Munro worked on the first proposals for the Ord River Scheme in 1949, and was project engineer for the Diversion Dam (1961-63). He successfully advocated an earth and rock-fill dam for the main Ord Dam, as opposed to the concrete dam proposed by Dumas. When he succeeded John Parker as Director of Engineering in 1969 he became the first Western Australian born engineer to hold the post.(1969-72). Parker had acted as unofficial coordinator of development (of the Pilbara in particular). Under the Tonkin government this role was formalised by combining the infrastructure development role of the PWD with the former Department of Industrial Development to form a Department of Development and Decentralisation, of which Munro became the first permanent head in 1972. One of its important roles was to negotiate agreements with large scale mineral developers. Munro was also a member of the Metropolitan Regional Planning Authority, the Metropolitan Water Board and the State Electricity Commission (1969-1979, Chairman 1974-75).[109]

2.6.3 Victor Cranston Munt

Victor Cranston Munt (1903-1953) graduated from the University of Western Australia in 1926. He was Resident Engineer for the Drakesbrook Dam, Wellington Dam, Canning Dam, Samson Brook Dam and Stirling Dam, from which he gained unrivalled experience in the construction of concrete and earth-fill dams. He was Hydraulic Engineer under Dumas (who as well as being Director of Works retained the title of Chief Hydraulic Engineer) and was responsible for designing the raising of Mundaring Weir. He was engaged on plans for the raising of Wellington Dam when he died in 1953.[110]

2.6.4 Thomas Cowley Hodgson

Thomas Cowley Hodgson (1858-1939) was born in Victoria and was educated at the the Ballarat School of Mines. He worked for twelve years as a consulting engineer to country shires and in Melbourne before joining the Public Works Department in WA as engineer for water supply and sewerage under O'Connor in 1895. In 1896, he investigated possible sources of water supply for Perth, and recommended the site

[109] Le Page, *Building a State. The Story of the Public Works Department of Western Australia 1829-1985.* Water Authority of WA 1986. And D.C. Munro Battye Library Oral History OH 2670.

[110] Le Page, *op cit.* 1986; Cumming *op cit.*, 1991.

that was eventually adopted for the Canning dam. He did an extensive investigation of sites for the headworks of the Goldfields Water Supply Scheme and identified the site for Mundaring Weir. He designed a number of critical elements of the Goldfields Scheme, notably the pumping system. In 1900 he became, in effect, the construction manager for the scheme. He resigned from the department in 1902.[111]

2.6.5 Frederick Washington Lawson

Frederick Washington Lawson (1861-1924) was born in Launceston and educated in Melbourne. He joined the NSW Public Works Department in 1896, working on sewerage and water supply dam construction. He came to Western Australia in 1906 to work for the PWD on sewerage works. When the Metropolitan Water Supply Sewerage and Drainage Department was formed, Lawson became head of its engineering division in 1912. His 1915 report recommended a dam on the Canning near Araluen in preference to Hodgson's site because of the former's increased impounding capacity. He served in the Royal Australian Engineers in the war, reaching the rank of major, and was awarded the DSO for organising water supplies for the Australian troops in France. Faced with a water supply crisis in 1920, he proposed using water from the Canning at Nicholson Road, but this scheme was abandoned by the government. In 1923 he prepared a comprehensive programme for the short and long term development of water supplies for Perth which became known as the 'Hills Scheme', and which formed the basis for water supply development in Perth for the next 25 years. [112]

2.7 CANNING DAM IN THE CONTEXT OF OTHER DAMS IN WESTERN AUSTRALIA

2.7.1 Western Australian Dam Construction 1923-1941

During the 18 years from 1923 to 1941 six major dam works were constructed in the Darling Ranges. These were built to satisfy one of two different requirements. The first was to provide water supplies for Perth and the second to prevent winter flooding of the coastal plain by rivers discharging from the Darling escarpment and to release the impounded water during summer for use in irrigated agriculture.

Dam [113]	*Type*	*Use*	*Constructed*	*Height*
Churchman Brook	earth	water supply	1924-29	26m
Drakesbrook	earth	irrigation	1930	16m
Raising Harvey Dam	concrete	irrigation	1930-31	12.2 - 18.3m
Wellington Dam	concrete	irrigation	1931-34	19 m
Canning Dam	concrete	water supply	1933-40	66m
Samson Brook Dam	earth	irrigation	1938-41	34m

111 Le Page, *op cit.* 1986; Cumming *op cit.*, 1991.

112 H.E. Hunt, Australian *Heritage Engineering Record. Perth's Early Water Supplies;* Institution of Engineers Australia, WA Division, 1984. And Le Page, *op cit.* 1986; Cumming *op cit.*, 1991.

113 S.-J. Hunt, with F.B. Morony (ed.), *Water, The Abiding Challenge,* MWB, 1980, p.69; Irrigation dams: J.S.H. Le Page, *Building a State,* WAWA, 1986, pp.444-455; Canning dam: R.J. Dumas & V.C. Munt, 'The Construction of the Canning Dam', *JIEAust* Vol.8, 193, p.1. (Churchman Brook Dam)

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111 Le Page, *op cit*. 1986; Cumming *op cit*., 1991.

112 H.E. Hunt, Australian *Heritage Engineering Record. Perth's Early Water Supplies;* Institution of Engineers Australia, WA Division, 1984. And Le Page, *op cit*. 1986; Cumming *op cit*., 1991.

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Before this period of continuous dam construction three important dams had been built in Western Australia. The first of these was the Victoria Reservoir, a 25 m high concrete dam on Munday Brook providing the first surface water supplies to Perth. When completed in 1891 it was the second highest concrete gravity dam in Australia The highest was the Beetaloo Dam in South Australia, completed the previous year which was 37m high.[114] The second important dam was the Niagara dam, built in 1898 on the goldfields north of Menzies, which was a concrete dam over 10m high.[115] Although it was never fully utilised because of the discovery of good underground supplies not far away at Kookynie, its construction was nevertheless a remarkable achievement. The third notable dam was Mundaring Weir (1898-1902), the height of which (30.5m above stream bed level) also made it the second highest mass concrete dam in Australia after Beetaloo.[116] Mundaring Weir was constructed as the source of water for the Goldfields Water Supply Scheme.

The theory of soil mechanics and its practical application to the building of earth fill dams was only in its infancy when the Churchman Brook dam was built, so its height of 26m was ambitious for the Water Supply Department's first major earth dam. The original concrete core had to be replaced by a clay core, which became the conventional way to construct an earth dam. Russell Dumas was recruited from South Australia as an expert on dams (although his experience had been on the River Murray weirs rather than earth dams) and became Resident Engineer for the latter part of its construction.[117] He then transferred to the Public Works Department and was involved in the design of the irrigation dams and later the Canning Dam.

The remaining five dams built during the 1930s were all built using unemployed workers registered under the sustenance programme. The first of these, the Drakesbrook Dam near Waroona, a 16m high earth dam was built in seven months using horses and drays as the only construction equipment in order to maximise the number of people employed.[118]

The first stage of the Harvey Dam, a 40ft (12.2 m) high concrete weir had been built in 1915-16. The second stage which raised the dam to 60ft (18.3m) was built in 1930-31 to help satisfy the demand for more irrigated farmland. The original wall was treated as an impervious core and the additional height and width were built in concrete with the down stream face strengthened by adding a rock fill batter.

Work on the Wellington Dam, a 19m high concrete dam which was the headworks for the Collie River irrigation works, began immediately after completion of the Drakesbrook Dam. The design of the dam, consisting of a series of mass concrete monoliths, tightly keyed into each other, was very similar to the much larger Canning Dam, and a number of construction methods used on the Canning Dam were first trialed on the Wellington Dam. One important one was the method of placing concrete by towers and chutes which is discussed in Section 2.5. On the

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114 L.Doherty, "Gravity Dams in Australia", Adelaide, 1997 (draft only), pp.5-7.

115 Le Page, *op.cit.* 1986. p.280.

116 Ibid., p.499.

117 Ibid., p.468.

118 Irrigation dams: Le Page, op.cit. 1986. pp.446-55.

Canning Dam both towers were over 50% higher than the larger of the two at Wellington Dam indicating the difference in scale between the two projects.[119]

The Samson Brook Dam, a 34m high earth dam was built to extend irrigation to the Waroona area. The project commenced in 1938.

2.7.2 Post-war Irrigation and Water Supply Dam Construction

Two large earth dams completed the headworks for the irrigation areas and made a major addition to metropolitan water supplies after the war.

The Stirling Dam, which had been planned as the major dam for the Harvey irrigation area, was commenced in January 1940 but construction was abandoned in January 1942 because of World War II. Work recommenced in February 1946 and was completed in 1947. At this time it was the highest earth dam in Australia with a height of 150ft (45.7 m).[120]

By the end of the war due to advancements in the knowledge of soil and rock mechanics, and improved methods of earth-moving and compaction, earth and rock fill became more economical to use than concrete for the construction of large dams. When the MWSS&D Board embarked on the construction of Serpentine Dam, the organisation's largest project since the Canning Dam, in the 1950s an earth fill dam 171ft (52.1m) high proved the most economical.[121] Work commenced in November 1957 and was completed in October 1961.

The other major post-war dam construction works involved the raising of Mundaring Weir and Wellington Dam as headworks for the Comprehensive Agricultural Areas Water Supply Scheme.

In the 1980s the pendulum began to swing back in favour of concrete dams with the advent of the use of roller-compacted concrete and methods of designing dams to incorporate post-tensioning in their initial construction.[122]

2.7.3 Canning Dam in Comparison with Other Dams

When built (1940) the Canning Dam was the longest concrete gravity dam in Australia, and also the second highest, after the Burrinjuck Dam in NSW (built in 1927 and 79m high to the Canning's66 m).[123] Two other higher gravity dams, the Avon (72m) and the Nepean (81m) which are both in NSW, are masonry dams.[124] Of the 90 large concrete and masonry gravity dams in Australia in 1997, the Canning Dam is still the fifth highest and sixth longest.[125] It is the largest concrete dam in the state in terms of length of crest and volume of concrete.

119 V.C. Munt, 'A Decade of Dam Construction 1930-1940', PWD, c.Nov.1940, pp.39-44 (Wellington), 73-75 (Canning). Le Page cites this source as an UWA Engineering School thesis but this seems unlkely as part of the Canning Dam section was written by D.C. Munro.

120 Le Page, op.cit. 1986. p.455.

121 Hunt & Morony *op.cit.* 1980, p.88.

122 Doherty, *op.ci.t* p.2.

123 Doherty, p.8.

124 Ibid., p.12.

125 According to Doherty's tabulations.

6.0 STATEMENT OF SIGNIFICANCE

Canning Dam is significant because:

it is a benchmark site in the history of the provision of Perth's water supply and in the history of concrete gravity dam construction in Western Australia;

it solved the problem of Perth's water supply, was the main source of water from 1940 to 1961 and remains essential to the city's water supply service today;

of its innovative structural and hydraulic design considered to be at the forefront of concrete gravity dam design at the period;

of the construction techniques used to build the dam;

of the visual qualities of the curved concrete dam wall and the use of Inter-War Stripped Classical detailing on the gate house, piers and balustrade;

of its contribution to the aesthetic value of its setting creating a distinct landscape comprising the juxtaposition of the concrete dam wall, the large open lake of impounded water with its rugged, largely natural enclosing landscape;

of associations with the sustenance work programme in the 1930s;

it represents a large scale engineering project constructed in the period before World War II involving the establishment of a townsite for workers;

of social associations with those associated with the townsite that developed in association with the project from 1933 to 1940;

it has always been valued by the community as a destination for outings and a venue for picnics and passive recreation;

of its potential to illustrate aspects of human occupation associated with the development of the place;

of historic associations with those prominent engineers closely associated with its design and development. In particular:
- Russell John Dumas
- Donald Campbell Munro
- Victor Cranston Munt
- Thomas Cowley Hodgson
- Frederick Washington Lawson; and

because of historic associations with the prominent Western Australian benefactor Sir Charles McNess.

Whilst there are a number of other concrete dams in Western Australia the size, complexity and technical achievement of Canning Dam set it apart.

Attachment No 7

Location Plan

Site Plan

Drawings

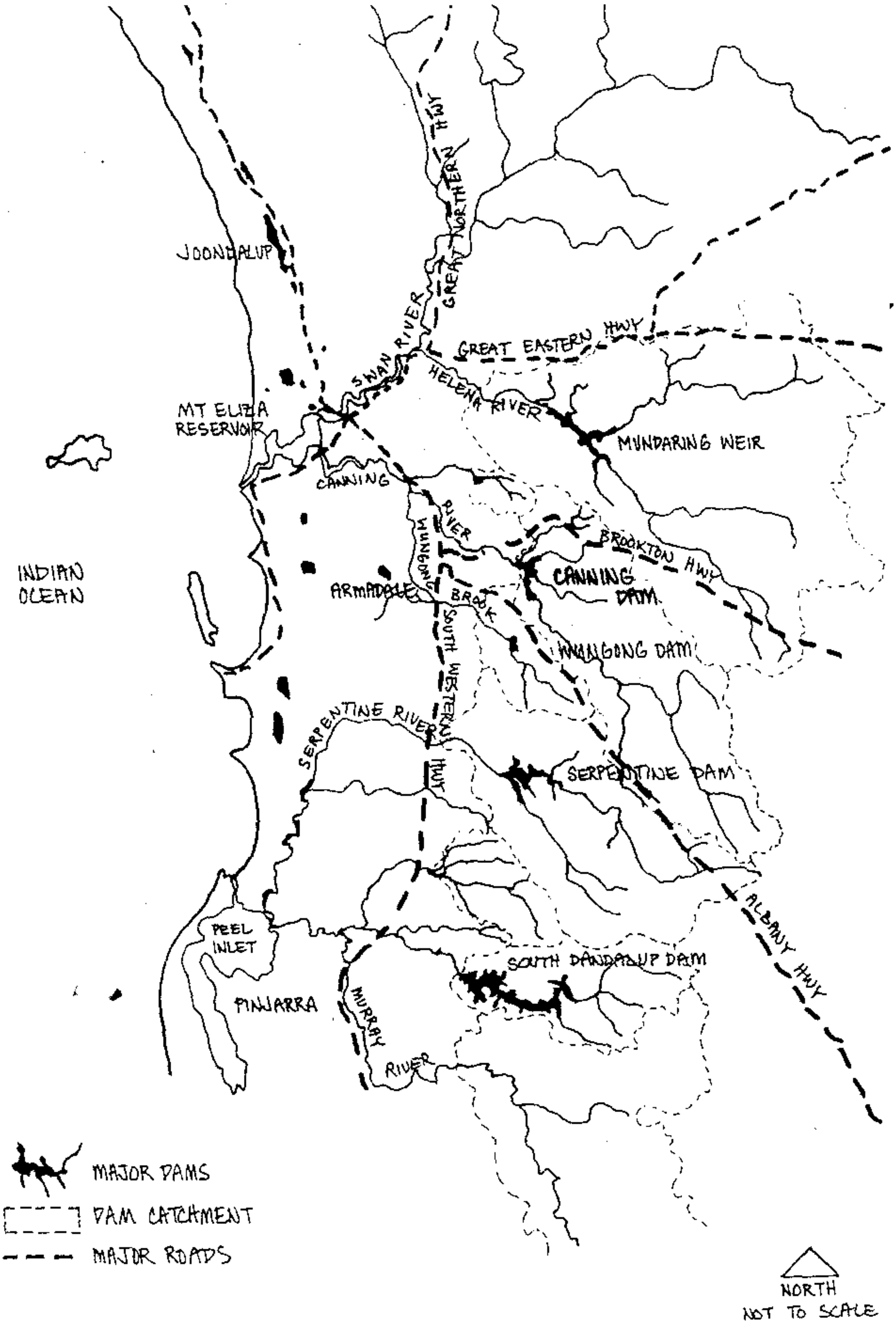


Fig 2 Locality Plan

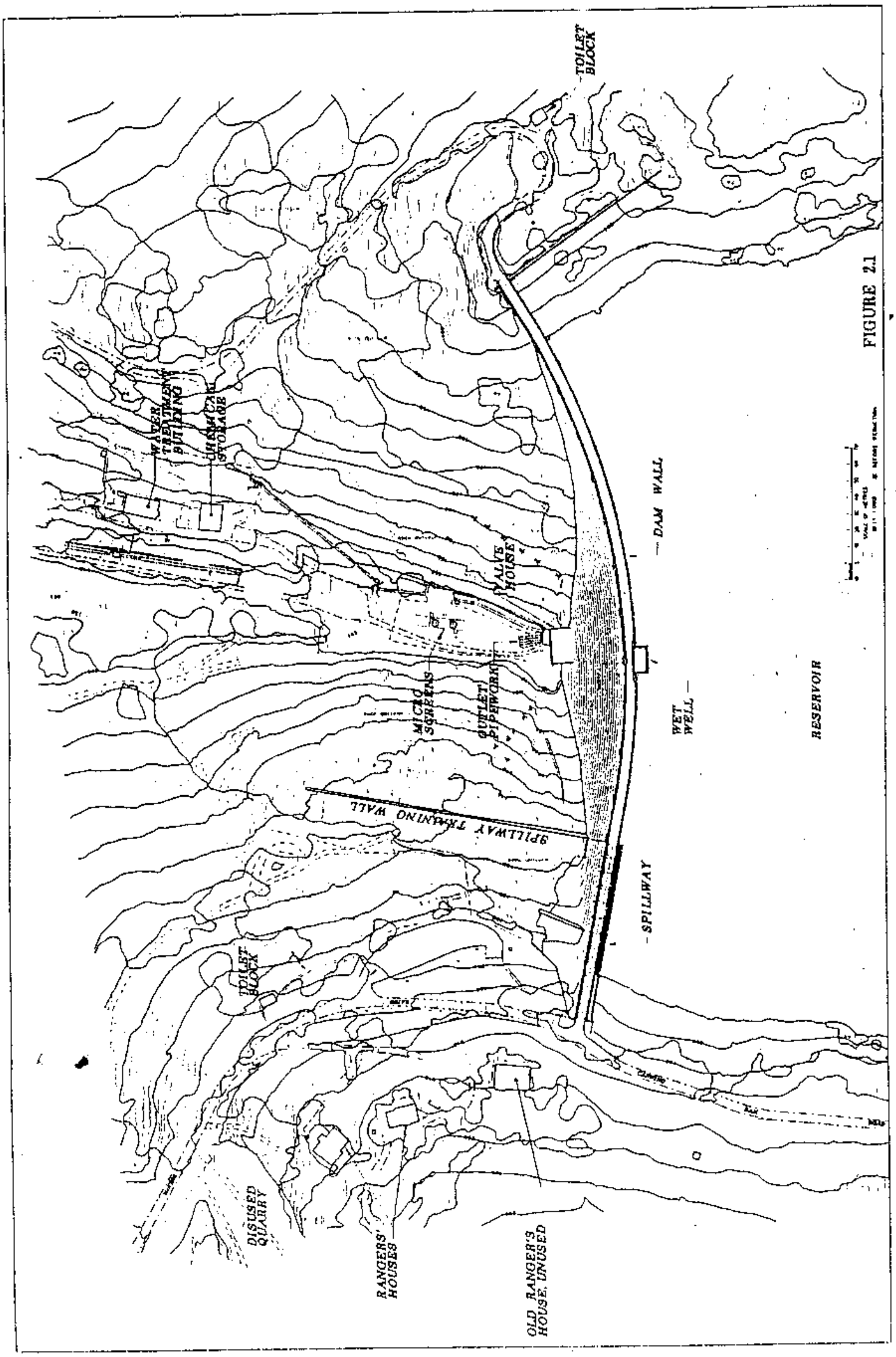


Fig 22 Canning Dam - site arrangement plan view
Extract from Geo-Eng Dwg EA82-01-01
Courtesy *Water Corporation*

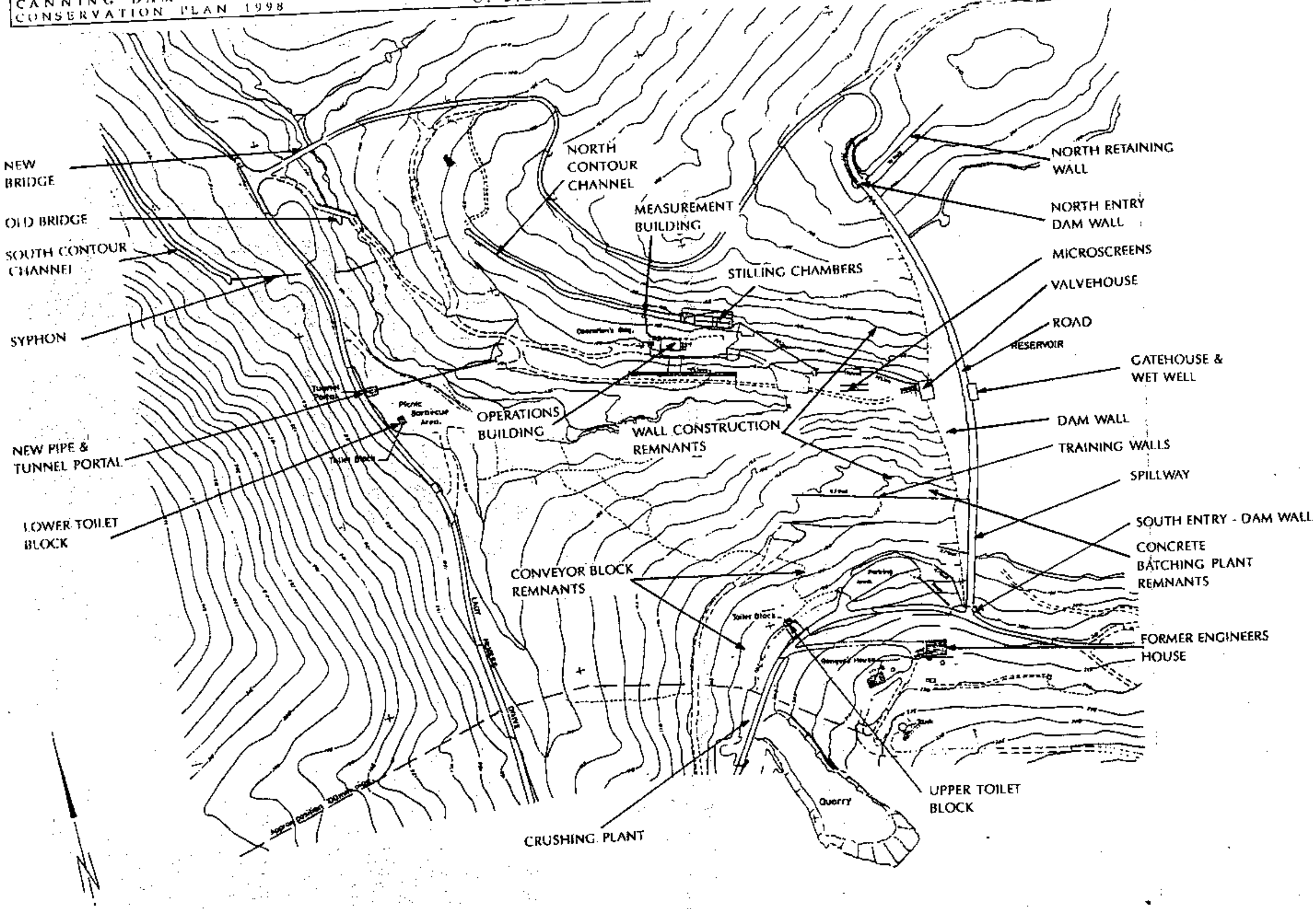
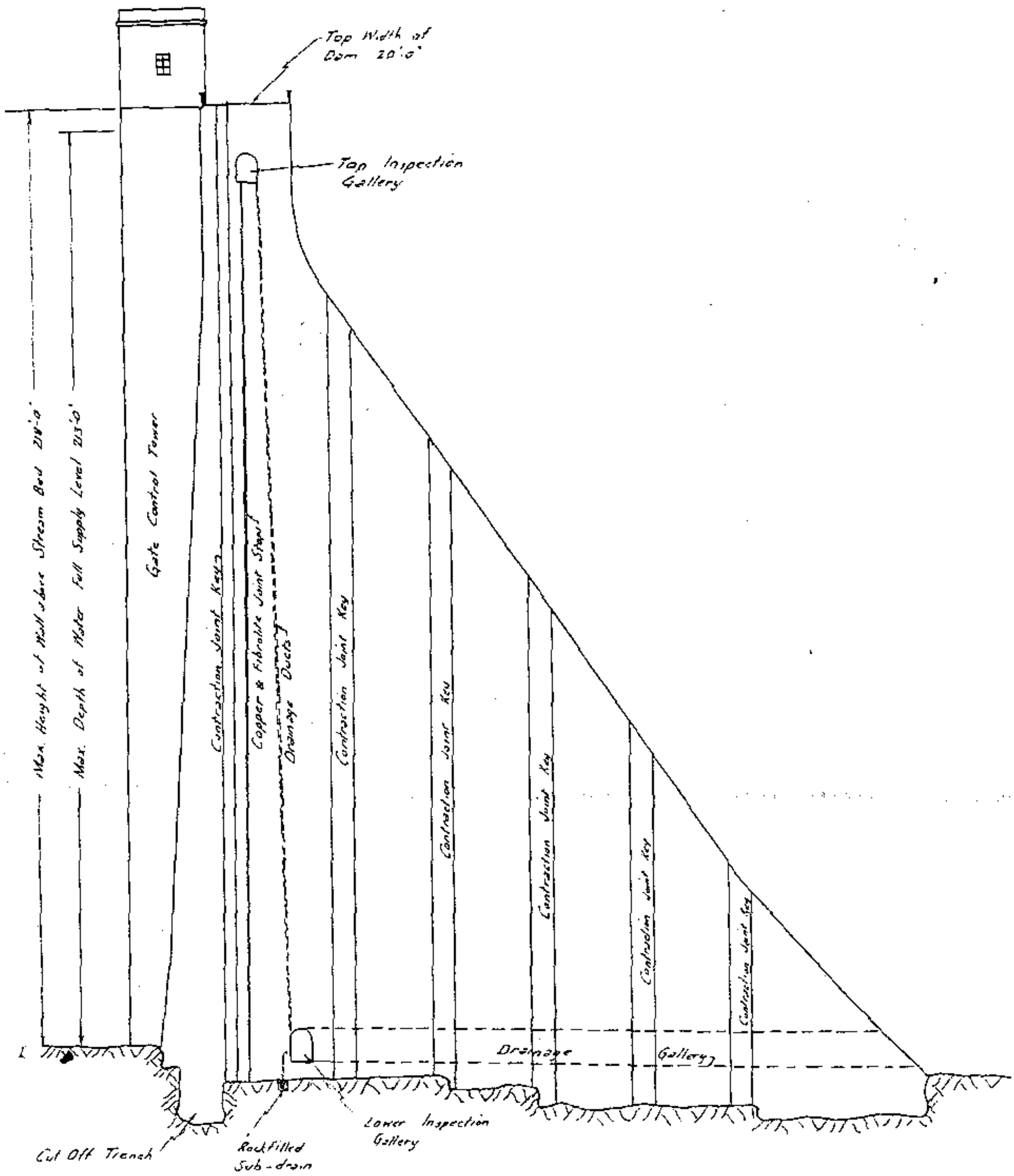


Fig 24 Site plan identifying elements



Canning Dam

Typical Cross Section.

Scale: 30 Feet to an Inch.

Fig 6 Canning Dam - Section showing dam wall, inspection galleries etc.
Courtesy *Water Corporation*

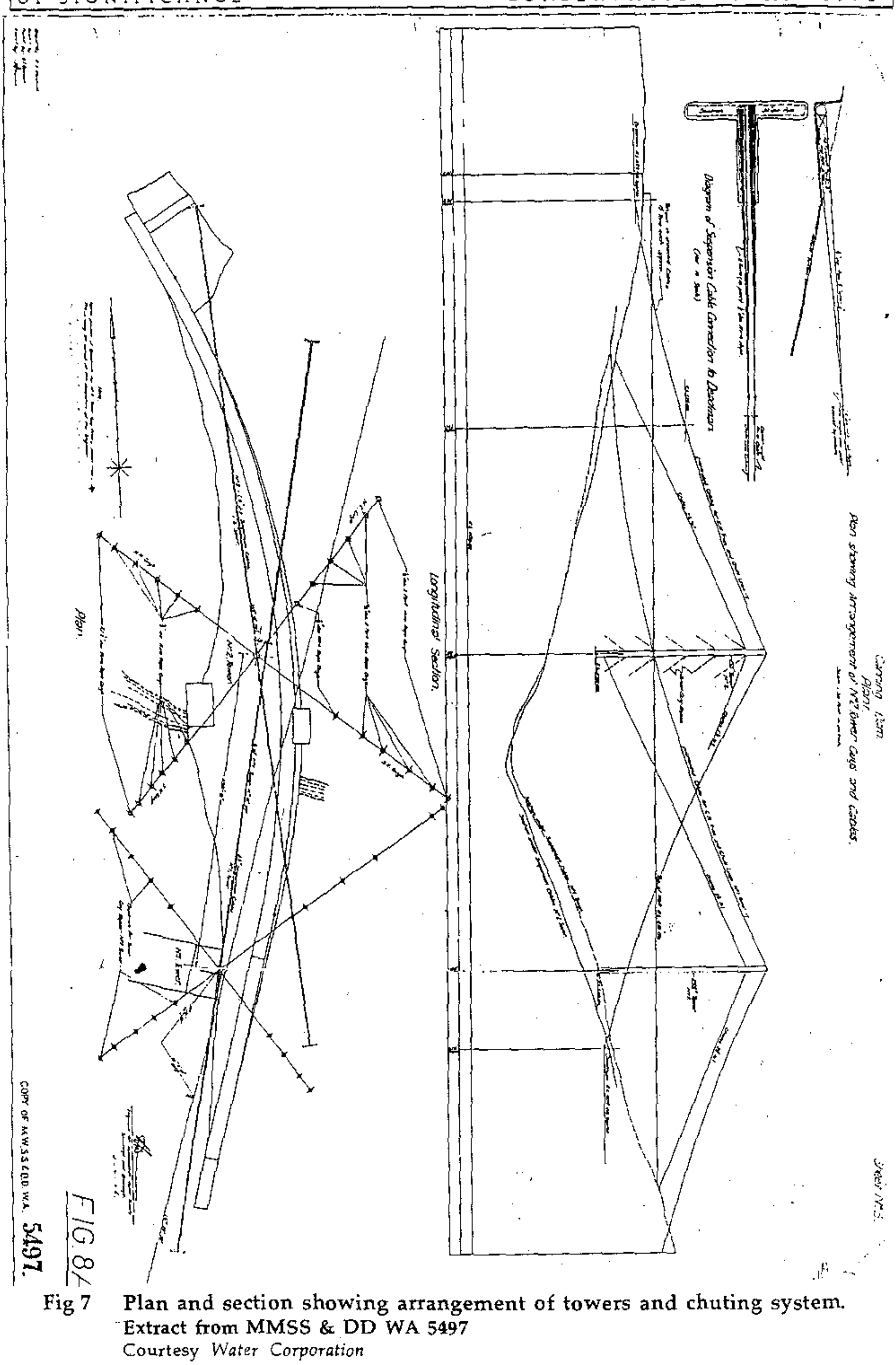


Fig 7 **Plan and section showing arrangement of towers and chuting system. Extract from MMSS & DD WA 5497**
Courtesy *Water Corporation*

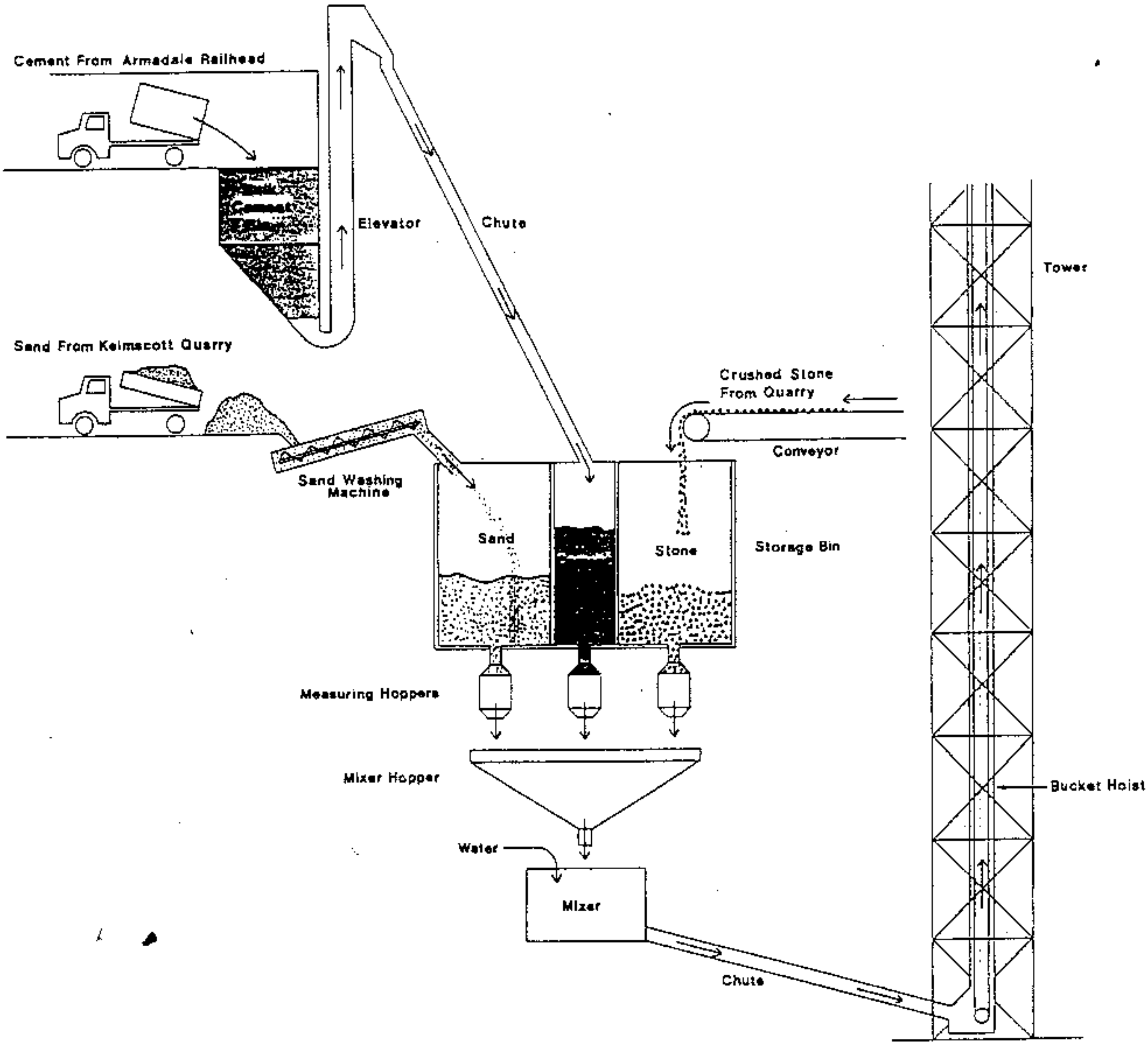


Fig 9 Figure showing process of mixing concrete in the early stages of construction. (Later sand washing was abandoned, the cement handling altered and the crushed stone added in two sizes)
From exhibition material prepared for Canning Dam bi-centennial celebrations 1983.
Courtesy *Water Corporation*

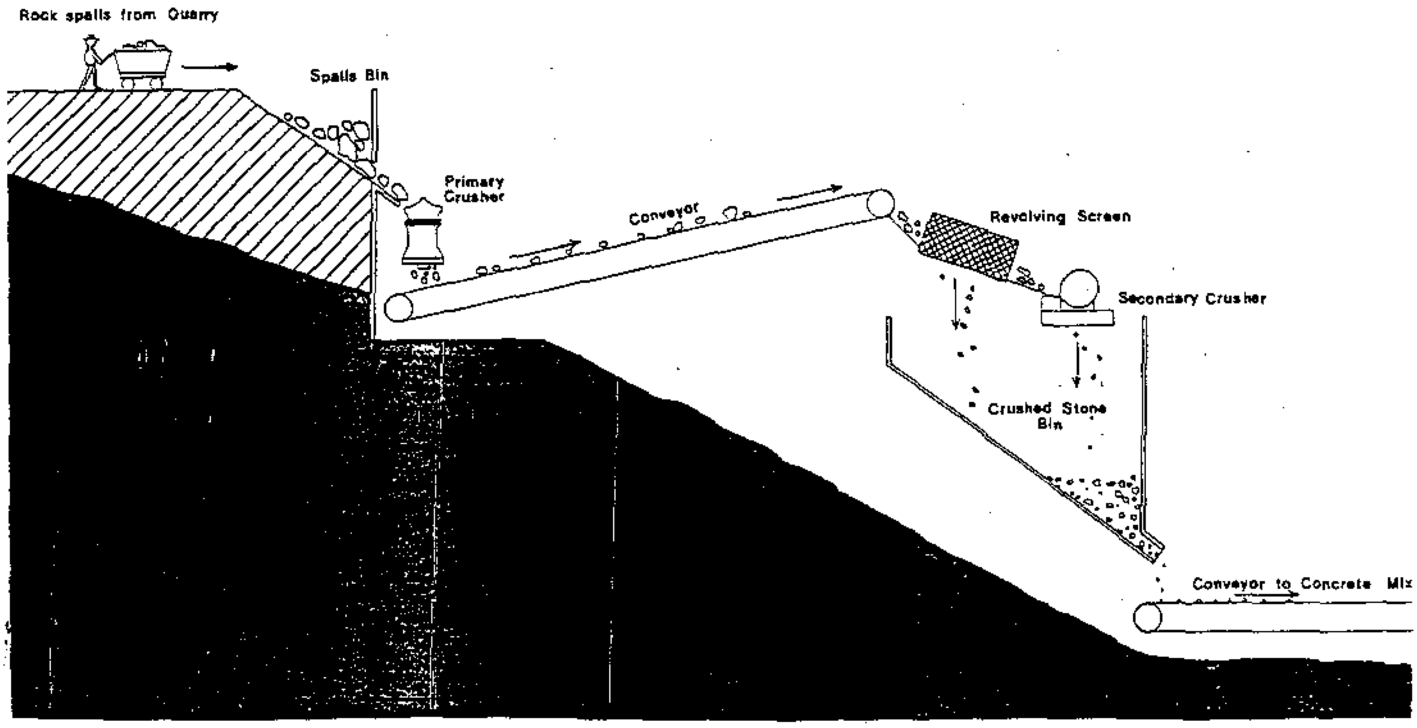


Fig 10 Figure showing process of crushing quarried stone for concrete use.
From exhibition material prepared for Canning Dam bi-centennial celebrations 1983.
Courtesy *Water Corporation*

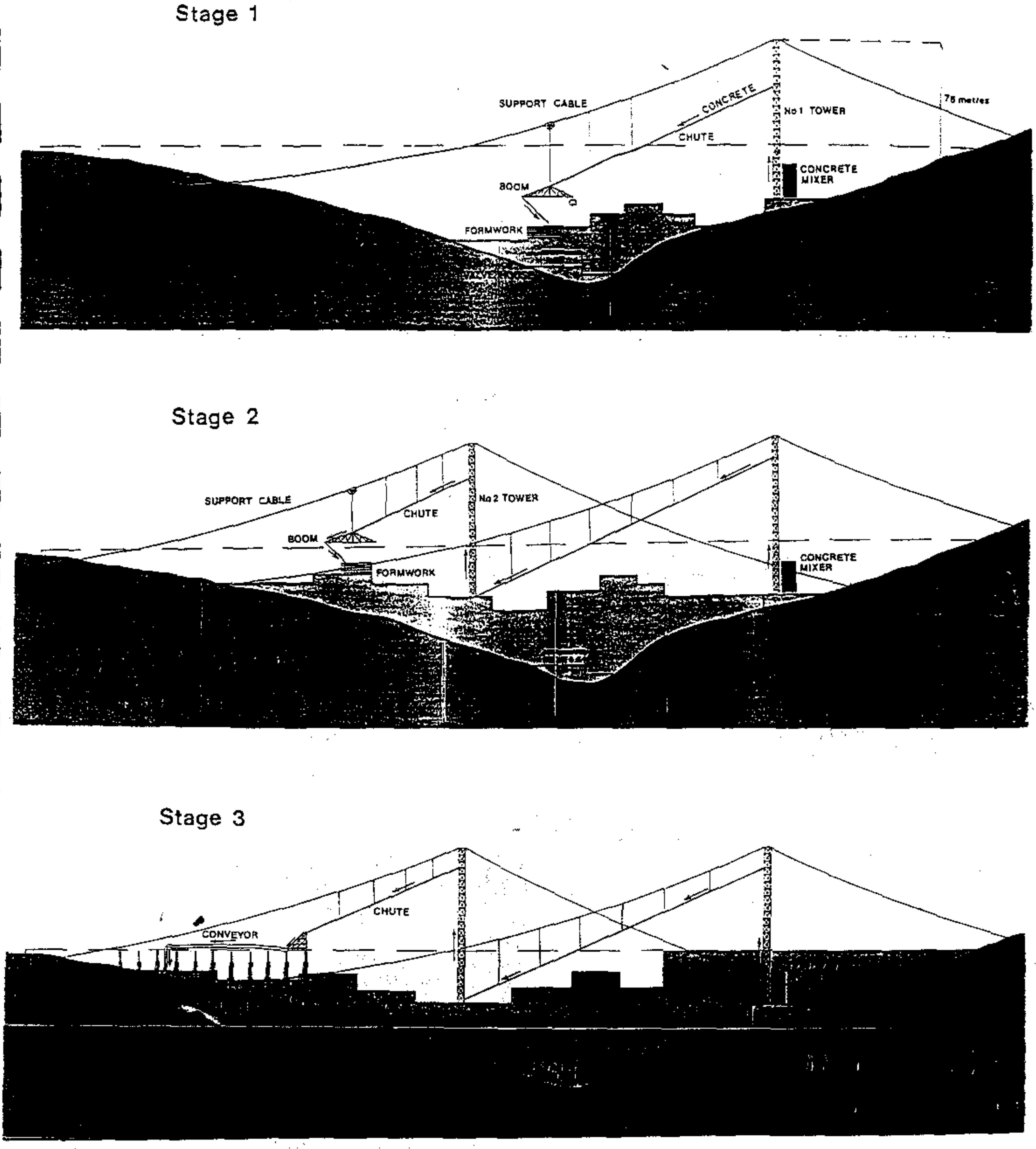


Fig 11 Stages of construction of dam monoliths showing towers and chuting system.
From exhibition material prepared for Canning Dam bi-centennial celebrations 1983.
Courtesy *Water Corporation*

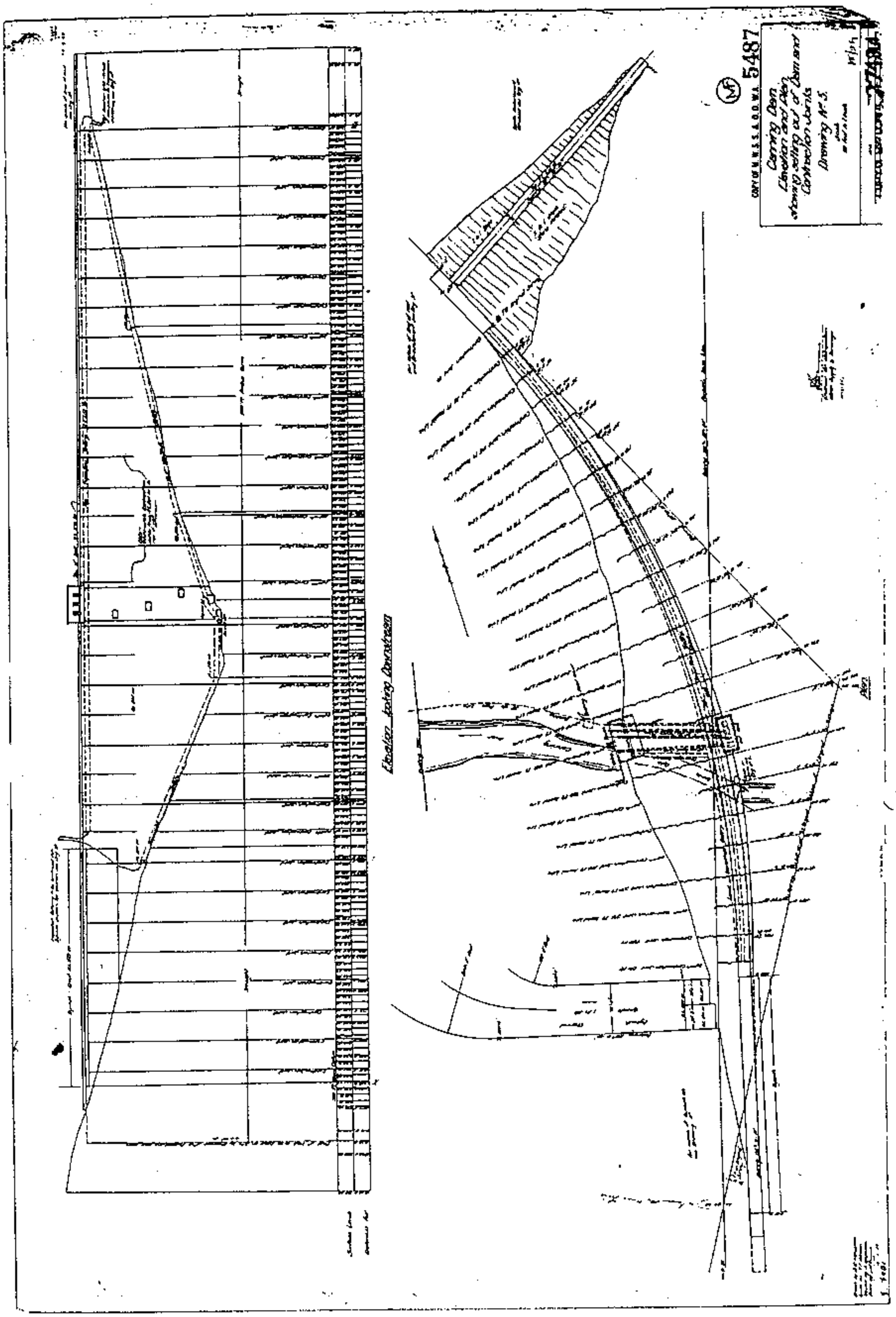


Fig 12 Canning Dam, Elevation and Plan.
Example of MWSS & DD WA Documentation for the construction of Canning Dam 1933-40.
Courtesy *Water Corporation*

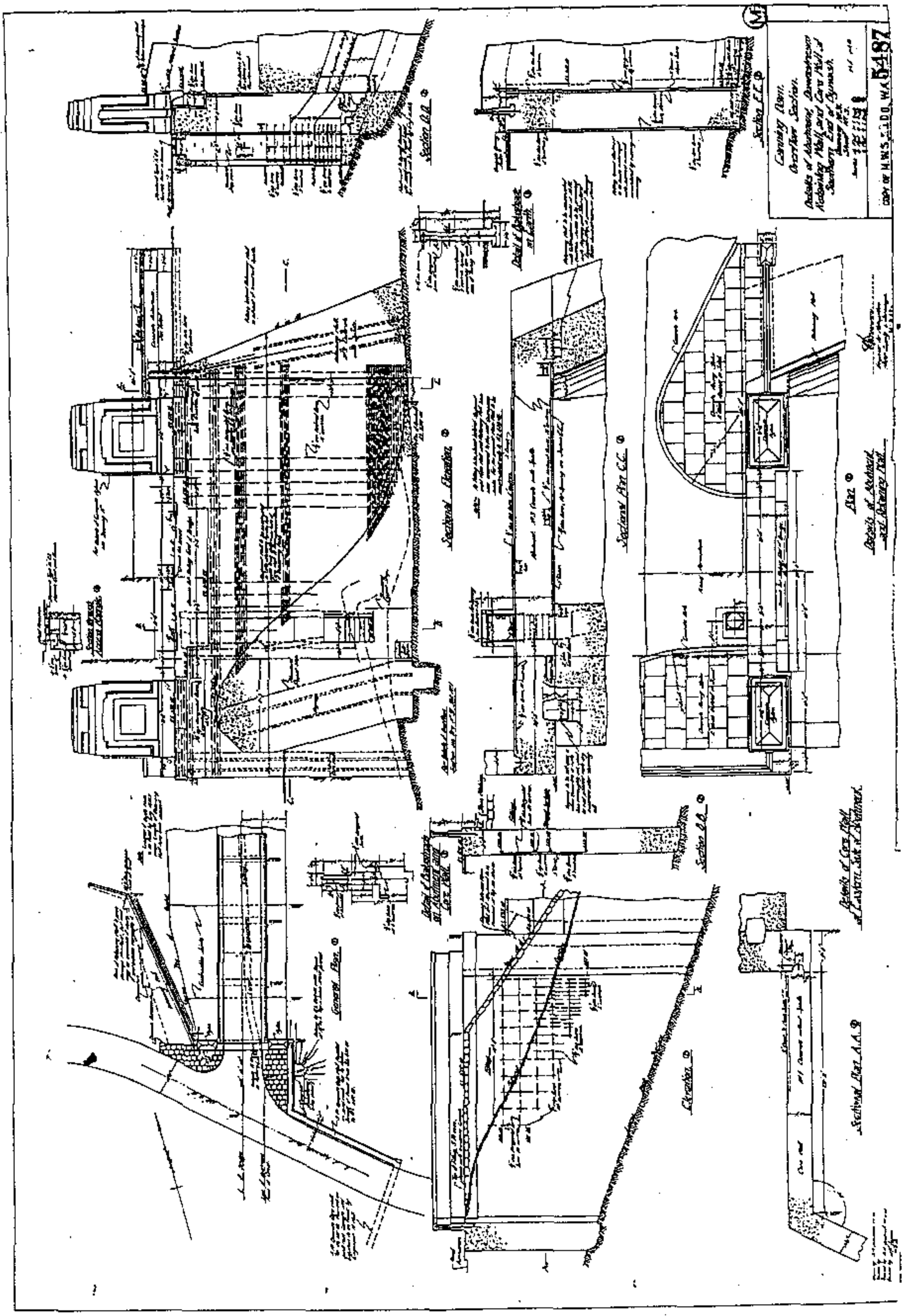


Fig 13 Canning Dam, Overflow Section.
Example of MWSS & DD WA Documentation for the construction of Canning Dam 1933-40.
Courtesy *Water Corporation*

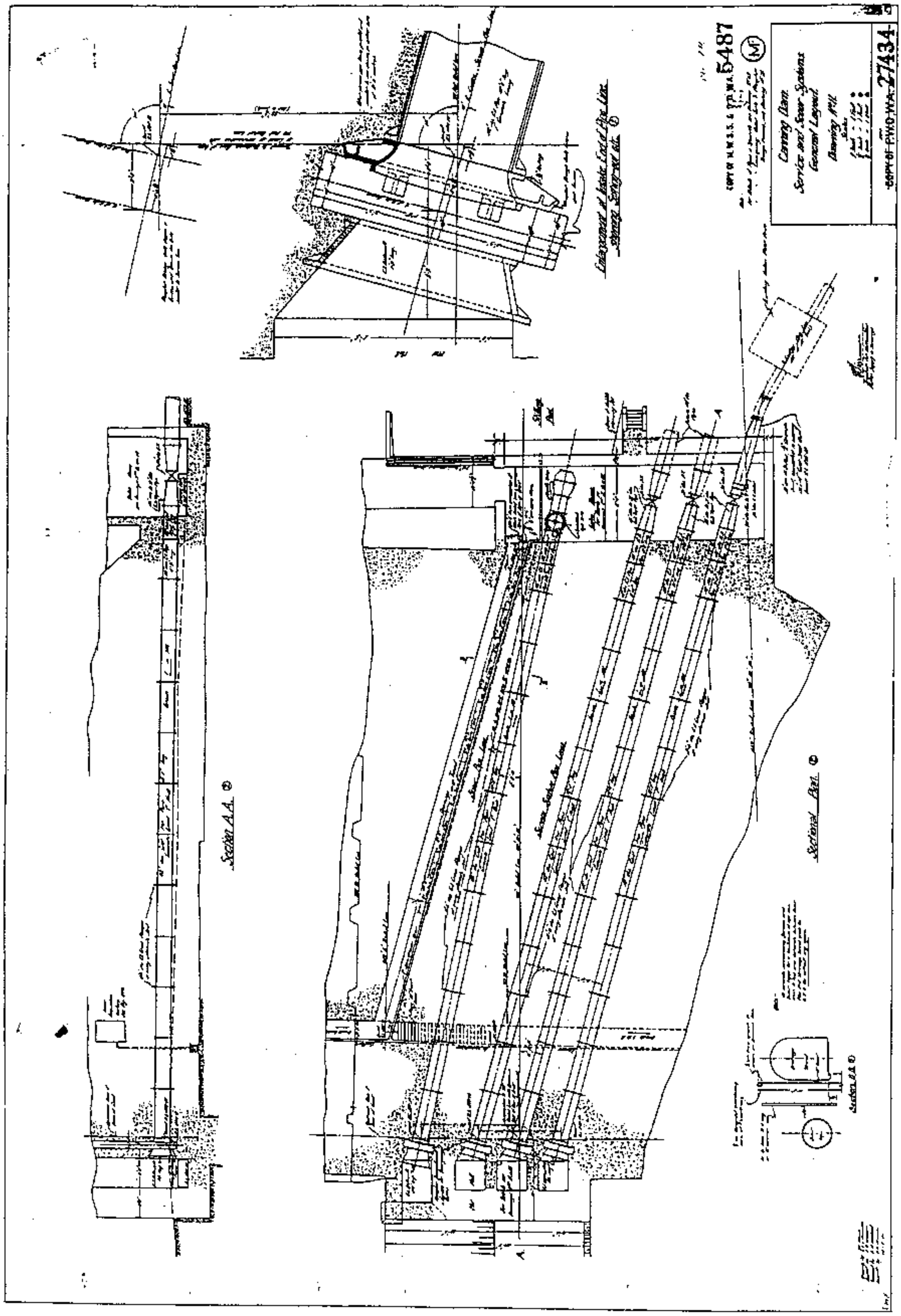


Fig 16 Canning Dam, Service and Scour Systems General Layout.
Example of MWSS & DD WA Documentation for the construction of Canning Dam 1933-40.
Courtesy *Water Corporation*

Attachment No 8

Reference Material

W. H. R. Nimmo 'Historical Review of Dam ANCOLD 1966

WARREN DAM,[36,77] 84 feet high, completed in 1916 on South Para River South Australia is of interest only because of its unusual cross section (Figure 15) with curved back face, and front face battered 45 on 1.

WELLINGTON DAM,[77,99] completed in 1933, near Collie in Western Australia is a curved gravity dam 748 feet long. Although its ogee spillway section was only 68 feet high it was designed for 50% uplift. There is a system of drains leading to a gallery. The dam is of further interest because it was later raised, 1944 and 1960, to about twice its original height and has the unusual section shown in Figure 16.

The New South Wales portion of HUME DAM[51] is a gravity structure approximately 1000 feet long. A non-spillway section at the north end contains four irrigation outlets discharging through needle valves into a stilling pool. There are also three hydro-electric outlets. The spillway section, 725 feet long, with crest at R.L.614, discharges into a dissipator pool, approximately 50 feet wide, retained by a 25 foot high baffle wall. It was originally designed for a storage of 2 million acre feet retained at R.L. 626 by 29 steel gates operating between piers which support the roadway over the dam at R.L. 642, that is 162 feet above the foundation. A flood surcharge of 9 feet was allowed for above the storage level. For economic and other reasons, construction ceased in 1936 with crest level at R.L. 606 and a capacity of 1¼ million acre feet and for a long period the dam was operated at that level without gates. Later, because of the growing demand, it was decided to increase the storage capacity to 2½ million acre feet by increasing the size of the gates and raising the storage level to R.L. 636. The dam has recently been completed with the spillway crest at R.L.612 with provision for a flood surcharge to R.L. 636. To ensure stability with the raised storage level and allow for uplift, the dam has been post-tensioned, this being the first application of that technique to a dam in Australia.

CANNING DAM,[33,77] completed in 1940, 245 feet high on the Canning River, Western Australia, although it did not establish any new record for height, embodied several new features. Uplift (50%) was allowed for; there are both upper and lower galleries connected to near vertical drains; contraction joints are keyed; bulk cement was used; crusher run aggregate was weigh batched. The concrete which was chuted into place, had a water-cement ratio of 0.88 to 0.94. A rock-filled drain located immediately behind the cut-off trench and connected to the lower gallery is intended to intercept water seeping along the concrete-rock contact and springs were piped up to the gallery but there does not appear to be any deep drainage of the granite foundations. The dam is curved in plan but to a much larger radius than that of the dam in Table 4.

As a result of model tests, the problem of controlling the discharge of flood water over a long spillway into a narrow gorge and at the same time dissipating its energy, was accomplished at PINE TIER DAM,[105] completed in 1953 on Nive River Tasmania. An ogee spillway, 400 feet long, has a central 200 feet notch which is 6 feet deeper than the end sections. This discharges onto sloping aprons which direct the flow into that passing over the central notch, the resulting turbulence dissipating the energy. Modified cement with low heat of hydration was used in concrete.

A similar trapezoidal type of spillway was adopted for the 110 feet high GUTHEGA DAM,[27] completed in 1955, on Snowy River N.S.W., though in that case the ogee spillway was all at one level.

TINAROO FALLS DAM,[12] completed in 1958, (155 feet) on the Barron River Queensland, created the first large storage reservoir in tropical Australia. It is a gravity dam having a vertical upstream face and a downstream face sloping 0.78 to 1. The ogee spillway, 200 feet by 12 feet designed to cope with a flood of 70% Myer rating, discharges into a pool retained by a large baffle wall, the energy of the overfalling water being dissipated by turbulence, the slope at the stream being too steep to permit the development of an hydraulic jump.

There was a spectacular increase in height from the raised BURRINJUCK DAM, (260 feet) to the highest gravity dam in Australia, WARRAGAMBA,[67] completed in 1960 and 450 feet high. The design and construction of this dam which would be a high one in any country, has been adequately dealt with by Nicol[67] and can only be briefly referred to here. An outflow of 450,000 cusecs resulting from an inflow of 700,000 cusecs can be passed by the outlets and the spillway which is controlled by a drum gate, 90 feet by 25 feet, and four radial gates, each 40 feet by 43 feet 9 inches. The spillway discharges into an hydraulic jump dissipator. Temperature in the large mass of concrete was controlled by the use of low heat cement: the addition of ice to the mix: and circulation of chilled water in cooling coils embedded in the concrete.

SOMERSET DAM,[66,71,77,88] on the Stanley River Queensland was commenced in 1935, but stopped during the war. It was finally completed in 1959. It is a straight gravity dam, 1000 feet long and, exclusive of cut-off wall, is 163 feet high above the foundations. Designed for the dual function of water supply and flood mitigation

TRANSACTIONS OF THE INSTITUTION.

The Design of the Canning Dam.

By RUSSELL JOHN DUMAS, M.E.

*Member.**

Summary.—This paper is a description of the design of a large dam to augment the water supply to the City of Perth, Western Australia. The theoretical design is outlined under the general headings of selection of cross-section, design of theoretical section, contraction joints, water control, bywash, and detail design, etc. Sample calculations and comparative tables, useful for general dam design purposes, are included in Appendices.

INTRODUCTION.

Selection of Site.—In selecting a reservoir site for the water supply of a city, the following features are to be studied:—

- Quality of water from catchment;
- Freedom of catchment from sources of contamination;
- Proximity to city;
- Elevation;
- Catchment area;
- Rainfall;
- Class of country comprising catchment and approximate percentage of run off;
- Daily discharge records of all streams;
- Impounding basin capacity and geological structure;
- Suitability of foundations for masonry, earth or rock-fill dam;
- Proximity of suitable materials for construction; and
- Natural formations facilitating economic construction and getting rid of surplus or flood waters.

Location of Canning Dam.—At a locality approximately 5 miles from the city of Perth (pipe line measurement) in the Canning River gorge is an extremely favourable site. The water flowing in the stream is of a highly potable quality, coming off an almost unsettled catchment.

The reduced level of the bed of the river at this point is approximately 460 feet above datum level. The principal city service reservoirs are at reduced level of 236 feet. Thus ample head for economic pipe lines from the reservoir is available.

The catchment area comprises 282 square miles of almost wholly unalienated land, heavily timbered, and controlled by the Forestry Department for re-afforestation purposes.

A great extent of the catchment area surface consists of porous ironstone gravel, and this, combined with the heavy timber, renders the run-off from the catchment a comparatively slow one.

The total annual discharge of the Canning River at the site during each of the last 14 years is given in Table I.

The impounding or storage basin immediately above the dam site opens out well with a large capacity, the gorge being at the confluence of three tributaries: Death Adder Creek, Canning River and Canning South Branch.

The floor and sides of the basin consist of basic granite and diorite formation overlain with varying thicknesses of more or less decomposed rock.

The gorge, which is approximately 300 yards in length, has been formed by erosion and weathering of a diorite dike approximately 20 feet in width which runs its full length. The sides and floor of the gorge are of granite rising to a height of over 200 ft. on either side.

*Abridgment of paper No. 392, which originated in the Perth Division of Institution.

The junctions of the diorite and granite walls are good, having a thickness of only 4 in. of semi-decomposed rock between.

On either side of the gorge the granite formations provide ample supplies of rock for crushing for concrete, etc.

On the flats immediately west of the Darling Range foothills in the Kelmscott district and approximately 10 miles from the dam site are large deposits of sand of good quality for concrete construction purposes.

An ample supply of spring water exceeding 250,000 gallons daily is available from sources one-half mile distant at an elevation 50 feet above the top of the dam.

Locally-manufactured cement is obtainable from the cement works at Rivervale, 24 miles distant from the dam, or imported cement via Fremantle.

Thus all materials necessary for construction are readily accessible.

TABLE I.

Year	Discharge, gallons	Year	Discharge, gallons.
1915	22,120,673,000	1923	23,550,000,000
1916		1924	13,429,819,000
1917	34,728,936,000	1925	3,416,557,000
1918	17,307,731,000	1926	32,581,942,000
1919	4,342,816,000	1927	19,244,739,000
1920	15,439,483,000	1928	23,355,251,000
1921	6,856,973,000	1929	6,797,645,000
1922	4,249,623,000	1930	15,912,890,000

The average of these discharges is 15,348,000,000 gallons, and based on the accepted average annual rainfall of 39 inches over the catchment area of 282 square miles, the average run-off is 9.6%.

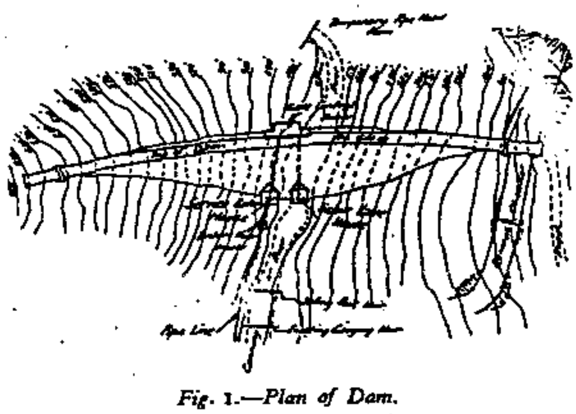


Fig. 1.—Plan of Dam.

SPECIFICATIONS FOR DESIGN OF WALL.

The various factors governing the design of the wall and the specifications adopted are set out briefly below. In Appendix I some of the particular problems involved are dealt with in greater detail.

The height of the top of the wall is R.L. 653.00.

(1) *Type of foundations.*—Diamond drills, trial holes, costeens, etc., supported by the Government Geologist's

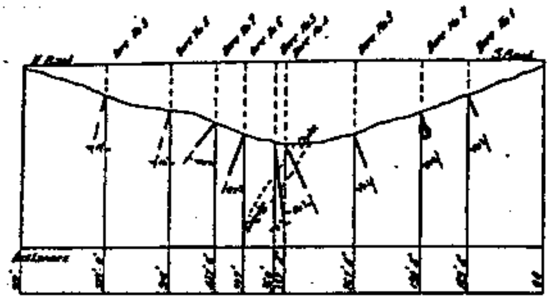


Fig. 2.—Section showing Diamond Drill Borings.

report, showed that the foundations throughout the whole length of the wall would be founded on solid homogeneous granite in an undisturbed and unbroken state. The foundations are practically ideal and do not require special treatment, but are subject to inspection on opening up.

(2) *Uplift to be provided for.*—Uplift of 50% of the full hydrostatic pressure at upstream heel of dam diminishing uniformly to nil at downstream toe acting over 50% of the base is provided for. Provision will be made for measuring the uplift pressures.

(3) *Drainage system.*—Upper and lower inspection galleries with connecting drainage tubes, tubes to consist

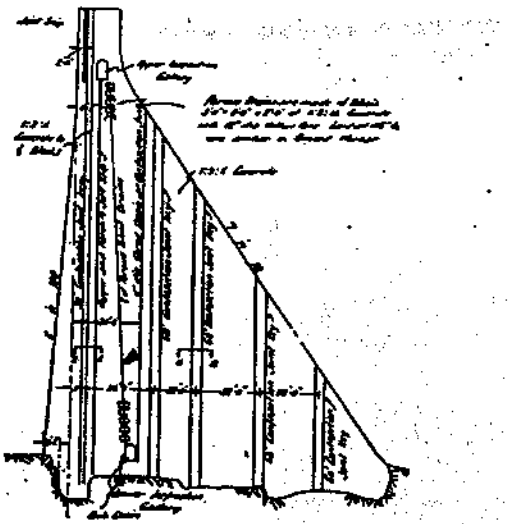


Fig. 3.—Section of Dam along a Contraction Joint.

of 3 ft. x 3 ft. porous concrete blocks with 12 in. core spaced 11 ft. 3 in. centres horizontally—not staggered. Tubes to be placed with centres 11 ft. from upstream face at junction with upper inspection gallery, and built on a batter of 0.048 to 1 to connect with lower inspection gallery, at lowest level of which a drainage gallery discharges the leakage water at the toe of the downstream face. The reasons for the adoption of the abovementioned systems are given in Appendix I.

In the case of the Canning dam, built on granite rock, the water under pressure will make its way beneath the dam by seepage through joints in the rock or by seepage along the junction of the rock and the concrete.

To prevent the water working along the open joints, in the rock, a cut-off trench is to be provided under the upstream heel of the dam, and such grouting under pressure is to be done as may be found necessary after foundations are opened up. In addition, the specification provides for all spring water to be taken up in pipes and dealt with as instructed.

Still further to ensure complete drainage of foundations, a longitudinal drain just downstream from the cut-off trench is provided.

A rectangular cut-off trench is to be provided. The upstream vertical face of the cut-off trench is to be kept 3 ft. 6 in. downstream of the upstream heel of the dam. This procedure follows average practice in other dams.

A rock-filled drain, 2 ft x 2 ft. in section, with 6 in. diameter open jointed pipes, is to be placed under the upstream edge of the lower inspection gallery, and connected to the gallery by 4 in. tubes every 50 ft. discharging under pressure. The 2 ft. x 2 ft. stone drain is to be divided into 50 ft. sections by concrete stops across the drain.

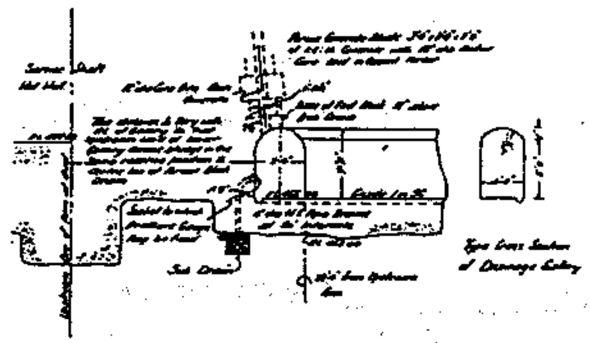


Fig. 4.—Details of Inspection Drainage Galleries and Adit.

(4) *Weight per cubic Foot of Material.*—The weight adopted was 145 lb. per c. ft. inclusive of all plums, etc. (See Appendix I for method of determining.)

(5) *Treatment of Flood Waters.*—Floodwaters not to overflow the dam, but a bypass weir to be provided with channel cut in the solid rock to discharge down the hillside well below the dam.

(6) *Width of Top of Dam.*—15 feet.

(7) *Upstream and Downstream Slopes.*—The upstream and downstream slopes were finally decided after theoretical section had been designed. The decision was made to make slopes as uniform as possible for practical reasons of construction. A comparison was made of the slopes in other dams and it was found that an average slope of 7 in 10 was suitable.

(8) *Limiting toe pressure.*—It was decided to conform to Bouvier's theorem and Creager's recommendation, viz., compute for inclined pressure

$$P_i = P_v \sec^2 \theta$$

where θ = angle made by resultant of forces with the vertical. Maximum allowable compressive stress = 14 tons per sq. ft., after consideration of test figures and results from other parts of the world. (No limit was adopted in the first design of the theoretical section.)

(9) *Resistance to Sliding.*—Limiting coefficient of friction to be 0.7, i.e. tan θ not to exceed 0.7. This gives

factor of safety against sliding of 2. In the Canning dam, the foundations are about 5 feet below the surface of the granite rock, which provides an additional factor of safety.

(10) *Resistance to Shear.*—If resistance to sliding is provided for, there is no necessity to consider failure by shearing.

(11) *Wind pressure is neglected.*—Wind pressure can only have an effect when blowing upstream while the reservoir is empty. A calculation made shows that with 30 lb. wind pressure per sq. ft. on a 200 ft. high wall, the resultant pressure is moved in position only about 3 in. upstream. The "practical" section to be adopted will amply allow for this.

(12) *Wave Pressure is neglected.*—

Stevenson's formula $h = 1.5\ F + (2.5 - \sqrt[4]{F})$.

h = height in ft. from trough to crest

F = fetch in miles in straight length.

In deep water, waves do not break and very little impact occurs.

(13) *Tension in the Concrete.*—Tension in the concrete of the wall or at junction of concrete and rock at either heel or toe, will not be permitted in designing the section.

(14) *Vertical Component of Water Pressure.*—The vertical pressure on an upstream face batter is to be allowed for.

Theoretical Cross Section.

Where a straight line stress distribution is assumed on a horizontal cross section of a dam wall it can be shown that the resultant of forces should act through either extremity of the middle third depending on whether the reservoir is full or empty, if no tension in the concrete is permitted and maximum economy is sought.

In the preliminary design, or first trial, no limit was placed on the toe pressures, and minimum widths were determined on planes at intervals of ten feet downwards from R.L. 621.00 to R.L. 451.00.

A sample calculation is shown in Appendix II, from which the general method adopted can be seen.

Using these methods, and modifying where necessary to keep the resultant just within the middle third, a theoretical cross section was developed.

Thus a design was completed of a theoretically economic section based on the determined conditions as to uplift, weight per c. ft. of concrete, etc., but without any limitation as to toe pressures.

This section was plotted and lines of pressure coinciding with extremities of middle thirds drawn in. Using the information and section thus obtained, variations were made to obtain a practical section for construction, and by comparison ascertain the excess of material employed over the minimum theoretical section.

A schedule of upstream and downstream slopes of dams in various parts of the world was consulted. From this schedule it was found that of 5 Australian dams the downstream slope of one is 7.1 in 10; in two it is 7 in 10, and two are 6.7 in 10. The American irrigation and power supply dams have a downstream slope of 6.6 to 6.7 in 10, but the American city water supply dams are given a greater factor of safety, and the downstream face is accordingly flattened.

In the case of the Canning dam, the extra amount of concrete required if the downstream slope is increased from 6.7 in 10 to 7 in 10 is approximately 1,600 cubic yards, involving an extra cost of approximately £5,000, which is only 0.5% of the total estimated cost of the work.

Records of the run-off from the Canning catchment have only been kept a comparatively short time. Should an abnormal flood above the capacity of the bywash occur, a downstream slope of 7 in 10 will provide sufficient stability in the wall section to permit an overflow of 12 in. over the dam. It was, therefore, decided to adopt a downstream slope of 7 in 10.

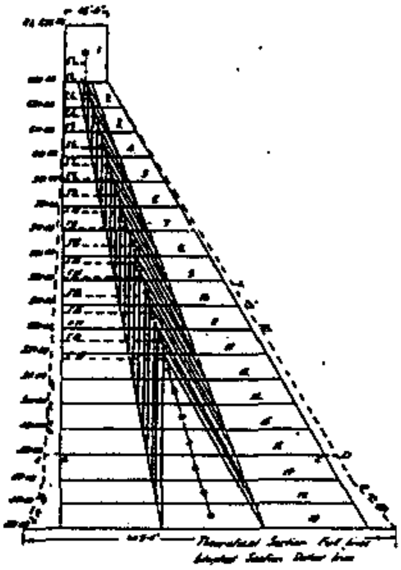


Fig. 5.—Section showing Lines of Pressure.

The upstream face slope has been kept vertical from the top of the dam to R.L.611.00 at which level the resultant of vertical forces, reservoir empty, coincided with upstream extremity of middle third of base. From R.L. 611.00 downwards the upstream face batter has been fixed at 0.6 in 10. This corresponds with Maroondah (Victoria) and Elephant Butte (United States of America) dams. It is a very little flatter than most other Australian dams, and about the average of the large American dams.

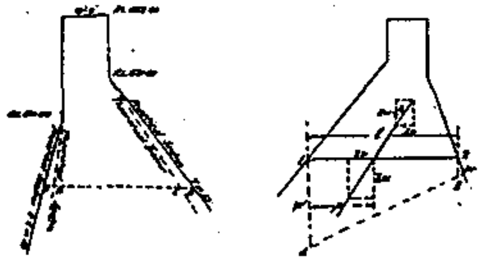


Fig. 6.

These slopes are maintained until the lower levels where limitation of toe pressures compels a flatter slope to be adopted.

Practical Section.

Table II gives the widths at the various levels of the theoretical minimum section compared with those of the practical section as adopted. As will be seen later the limiting inclined toe pressure of 14 tons per square foot will further increase the width of the adopted practical section at the lower levels :—

TABLE II.

R.L.	*AB* ft.	*BC.* ft.	*CD.* ft.	*AD.* ft.
653		15.0		15.0
631		15.0		15.0
621		19.54	2.46	22.0
611		25.41	3.59	29.0
601	0.30	32.90	3.80	36.6
591	0.20	40.20	3.80	44.2
581	0.38	47.34	4.08	51.8
571	0.68	54.60	4.12	59.4
561	1.04	61.83	4.13	67.0
551	1.45	69.03	4.12	74.6
541	1.88	76.26	4.06	82.2
531	2.43	83.22	4.15	89.8
521	2.96	90.27	4.17	97.4
511	3.49	97.32	4.19	105.0
501	4.05	104.34	4.21	112.6
491	4.62	111.33	4.25	120.2
481	5.19	118.32	4.29	127.8
471	5.77	125.31	4.32	135.4
461	6.34	132.30	4.36	143.0
451	6.92	139.29	4.39	150.6

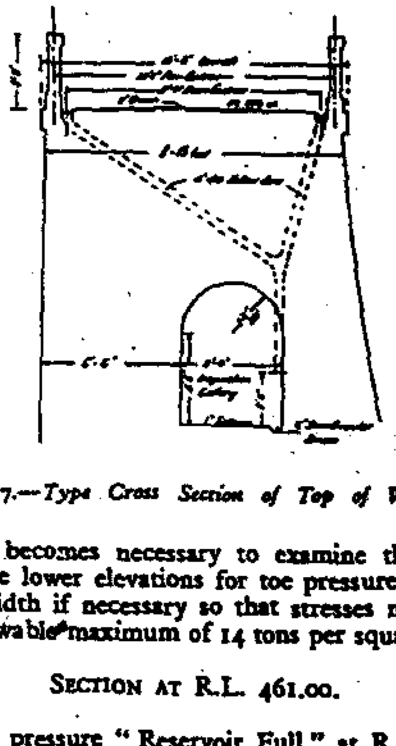


Fig. 7.—Type Cross Section of Top of Wall.

It now becomes necessary to examine the practical section at the lower elevations for toe pressures to provide additional width if necessary so that stresses may not exceed the allowable maximum of 14 tons per square foot.

Section at R.L. 461.00.

The toe pressure "Reservoir Full" at R.L. 481.00 = 12.63 tons per sq. ft. The toe pressure "Reservoir Full" at R.L. 471.00 = 13.4 tons per sq. ft.

It may, therefore, be assumed that for a similar section the toe pressure at R.L.461.00 would be approximately 14.2 tons per sq. ft.

It is seen from the formula, Toe pressure $= \frac{2\,\Sigma W}{l}\left(2 - \frac{3\mu}{l}\right)\sec^2\theta$, that toe pressures vary almost inversely as *l*.

It is, therefore, necessary to increase *l* by $\left(\frac{0.3}{14}\right)$ at R.L. 451, above the regular increase adopted previously, in order to obtain less than 14 tons per sq. ft. toe pressure. This amounts to approximately 3 ft.

To avoid danger of cracking at downstream toe and also to *conform* with Enger's theory, it is not *desired* to flatten the slope from 7 in 10 more than necessary. For trial this slope was increased to 8 in 10.

The calculations then reveal that the following conditions obtain :

Reservoir Full. Sec. θ = 1.178.

Distance from upstream heel to point of intersection of resultant with base = 90.4 ft.

Pressure on downstream toe on inclined plane normal to resultant = 13.62 tons per sq. ft.

Reservoir Empty. Distance from upstream heel to point of intersection of resultant with base = 52.3 ft.

Pressure at upstream heel normal to base = 11.2 tons per sq. ft.

The resultants fall within the middle third and toe pressures do not exceed 14 tons per sq. ft.

Contraction Joints.

In order to obviate vertical cracking in a long masonry dam, modern design now requires that the dam be built in monoliths or sections with a distinct division or lack of bonding between each monolith. Any expansion or contraction is taken up at these joints and so vertical cracking is obviated.

Temperature variations in concrete dams are due to three causes :—

(1) chemical action ;
(2) daily atmospheric variation ; and
(3) seasonal atmospheric variation.

Investigations tend to confirm that the initial high increase in temperature due to *chemical action* will, in the large Canning dam, be dissipated as construction proceeds. There will then be the gradual radiation of the last few degrees of setting heat spread over a number of years.

Numerous observations show that expansion or contraction in a mass gravity dam, due to daily variations in temperature, is negligible.

The Boonton dam observations indicated that the seasonal range in temperature of the concrete varied with the seasonal range in the temperature of the air, and with the distance from the dam face in accordance with Merriman's formula

$$R = \frac{T}{3\sqrt[3]{D}}$$

where T = extreme seasonal range of temperature of air, in degrees Fahrenheit.
R = extreme seasonal range of temperature of concrete, in degrees Fahrenheit.
D = distance from nearest face, in feet.

On the Boonton dam, $T = 135°$ F.

In Perth, Western Australia, highest mean sun temperature (January 1880) = 156° F.
Lowest mean sun temperature = 47.7° F.
Maximum seasonal variation = 108.3° F.

It is reasonable to allow for the Canning dam site where winter temperatures are lower than in Perth, a maximum *seasonal* variation of 110° F.

Design of Contraction Joints for Canning Reservoir.—The length of the wall and other features lend themselves to *monoliths* approximately 90 ft. 0 in. in length, which in accordance with overall lengths of monoliths in gene-

D	T	R	R
5'	118°	21°	12°C
10'	"	17	9
20'	"	13	"
30'	"	12	6.7

Upper Gill 14°C seasonal
Lower Gill 5°C

practice. This length has been tentatively adopted and will be subdivided as required for contraction in the upper parts of the section.

From Merriman's formula, previously quoted,

D = distance from the nearest face in feet.

Adopting $T = 110°$ F.

The thickness of the upper portion of the wall is 15 ft. 0 in., so that D at this section = 7.5 feet.

Adopting coefficient of expansion for concrete = 0.000006

$$R = 18.7° \text{ F.}$$

For a 90 ft. 0 in. length monolith, the contraction equals ⅛ inch.

It is, therefore, necessary at this thickness of wall to reduce the length between contraction joints to 45 ft.

At 80 ft. 0 in. below the top of the wall, i.e. at R.L. 573.00, the thickness of the wall = 57.8 feet.

$$\therefore D = 28.9 \text{ feet}$$

$$\therefore R = 12° \text{ F.}$$

$$\therefore \text{Contraction} = 0.078 \text{ in.}$$

i.e., approximately 1/12 in.

Contraction joints have therefore, been spaced as below: 90 ft. 0 in. apart on radial lines running from the top of the dam to the foundations, with intermediate joints from the top to 80 ft. 0 in. down.

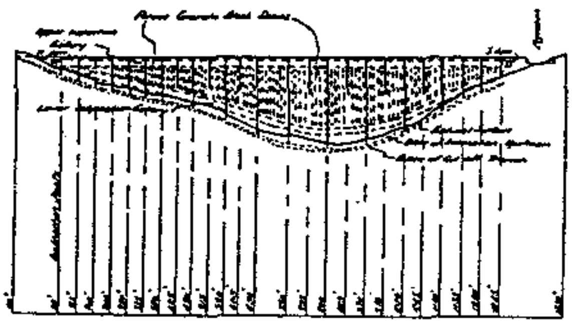


Fig. 8.—Key Elevation of Dam, showing Contraction Joint, Drainage Inspection Galleries, Adits, etc.

Control Design.

The maximum head, rate of discharge of water, type of dam, foundation material, and any special variations in draw off must be considered in the design of any outlets. Messrs. Gaylord and Savage deal fully with this question in "High Pressure Reservoir Outlets," a Bureau of Reclamation (United States of America) publication.

The Bureau of Reclamation reservoir projects are without exception for irrigation purposes and the control of necessity must be such as to release a large volume of water immediately below the dam either into the stream bed, or through power generating sets, and thence into the stream. Under these conditions very high velocities occur in the tunnel or outlet through the wall and in the earlier installations considerable damage was done to the controlling gates or valves, and to the tunnel walls. The controls necessary for a city water supply through a main are free from many of these problems in that the available head and length of supply line preclude the velocities from mounting to a dangerous value. It is necessary in the design of the Canning dam to provide a scour outlet at a lower level than the service pipe line, through which to pass the water during construction, and also to prevent accumulation of silt, etc., against the upstream face, and to provide for emptying the dam should the necessity arise. The problems connected with the control of this outlet will be exactly similar to those dealt with in Messrs. Gaylord and Savage's treatise.

Modern practice in masonry dam design is to provide for control of all outlets through the dam so that repairs may be made under all conditions and that draw-off may be made under all heads and in any quantities up to the capacity of the pipe line.

The following methods of control have been adopted in various masonry dams.

Cataract Dam, New South Wales, 1902/8.—Upstream control—screens and gates. Downstream control—needle valves.

Avon Dam, New South Wales, 1921/6.—On the outlets: Upstream control—removable screens and stop logs, emergency roller gate, and penstock gates at head of pipes. Downstream control—needle valve.

Cordeaux Dam, New South Wales, 1918/26.—On the outlets: Upstream control—removable screens and stop logs, with emergency roller gate and sleeve valve at head of pipe. Downstream control—gate valve and needle valve.

Hume Reservoir, 1919, *incomplete.*—Upstream control—non-removable trash racks set in reinforced concrete frame, with Stoney sluice gates. Downstream control—needle valve.

Chichester Reservoir, New South Wales, 1917/26.*—Upstream control—screens and inlet gate valves at various levels. Downstream control—gate valve and needle valve.

Burrinjuck Dam, New South Wales.†—Upstream control—screens only. Downstream control—gate valves with by-passes on mains to power house. Stoney sluices control passages through the dam for release of irrigation water.

Maroondah Dam, Victoria, 1920/8.—Outlet tower is entirely separated from main dam. Provision is made for take off at several levels. Emergency sleeve valves protect the intake ends of the pipes at each level outside the tower and inside the tower hydraulically-operated gate valves regulate the draw-off through each pipe.

Silvan Dam (Earthen) Victoria, 1926‡.—Two outlet towers provide for take off at 6 levels, which is controlled at each level by a sleeve emergency valve outside the tower, and a 24 in. x 30 in. hydraulically operated valve on the inside. From each tower a 54 in. diameter pipe passes under the bank in a reinforced concrete dry tunnel and discharges into a stilling pool. The outlet end of the pipe of 54 in. is controlled by a 32 in. gate valve and offtake is regulated by a special regulator valve designed by Mr. J. M. Sutherland, Melbourne Metropolitan Board of Works.

Owyhee Dam, United States of America, 1929.—Upstream control—reinforced concrete frame with wrought iron or mild steel bars for trash racks; also emergency gate hydraulically operated. Downstream control—needle valve.

McKay Dam, United States of America.—Control similar to Owyhee Dam.

Gibson Dam, United States of America, 1926.—Upstream control—trash rack consisting of bars set in reinforced concrete frame. Downstream control—emergency hydraulically operated gate immediately upstream of needle valve.

Stony Gorge Dam, United States of America, 1928.—Control similar to Owyhee Dam.

O'Shaughnessy Dam, United States of America, 1922/5.—Upstream control—coarse bar screens and stop planks. Sluice gates at each entrance level to wet well, and sluice gates at each offtake from wet well but no control downstream.

Croton Falls Dam, United States of America.—Upstream control—wooden gratings and sluice gates at each entrance to wet well. Downstream control—gate valve.

Cross River Dam, United States of America.—Control similar to Croton Falls dam.

Wachusett Dam, United States of America.—Upstream control—stop logs and screens with gate valves at entrances to wet well. Sleeve valve at pipe offtake at foot of wet well. Downstream control—standard gate valve.

In the earlier irrigation projects built by the Bureau of Reclamation, control of the outlets was placed at the upstream end. Under the very high velocities involved the regulating devices and conduits were seriously damaged, most of the damage being due to the effects of vacuum in

*See The Journal, Vol. 4, No. 3, March, 1932, p. 107.
†See *The Transactions,* I.E.Aust., Vol. IX., 1928.
‡See The Journal, Vol. 2, No. 12, December, 1930, p. 455.

the conduits immediately below the regulating devices. The most generally accepted theory, according to Gaylord and Savage, is that nascent oxygen is released from the air in solution as it enters the area of low pressure. This powerful oxidising agent acts on the material present producing a relatively soft oxide which is carried away by the stream.

The vacuum also produces surging in the water in the conduit, tearing up concrete linings, metal castings, etc. This is partly due to the high velocities and partly to the violent surges resulting from breaking and re-forming the vacuum, which occur when the outlet is running nearly full.

Quoting in full from Messrs. Gaylord and Savage :—

Although it is difficult to explain the results of the action of the water it is easy to determine the source of the destructive energy. Assume for example that the area of the tunnel is twice the area of the gate opening, and that the velocity of discharge through the gate into a perfect vacuum is 100 ft. per second, the gradient of the tunnel being level.

Let the tunnel be arranged so that it will run full, let no air vent be provided and let the gate opening be 4 ft. square. The velocity head will be 156 feet at the gate and 39 feet at the end of the tunnel, the drop being 117 feet. The increase in pressure head is from zero to 1 atmosphere, or 34 ft. which leaves a net drop of 83 feet. The energy corresponding to this loss of head must be converted into heat within the tunnel. The conversion must be made by operating the water and since the volume subject to agitation is small, the disturbances must be violent. The disturbances will be distributed over the length of tunnel necessary to secure an approximately uniform distribution of velocities. The remedy to be applied is the introduction of air immediately below the regulating devices to relieve the vacuum. In the design of outlets for high heads any conditions tending to produce vacuum should be avoided as far as practicable, and if this is impossible an ample air supply should be provided. *If the water can be safely brought through the dam under pressure and the regulating valve, usually a needle valve, be allowed to discharge freely into the air all danger of vacuum is eliminated.* This form of outlet is proving very satisfactory for high heads . . . .

After discussing various types of control as embodied in various dams, Messrs. Gaylord and Savage state *that the most approved type for high heads is that in which trash rack, and guard or emergency gates are provided near the upstream end of the conduit, the flow being taken through a metal lined conduit and discharged through a needle valve into free*

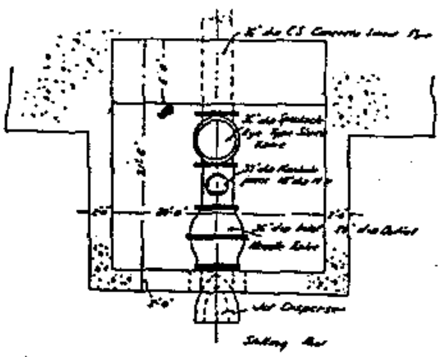


Fig. 9.—Arrangement of Valves, Downstream End, Scour Pipe Line.

ir. With a stilling pool as provided for in the Canning am, and solid granite floors, the energy is dissipated without amage. Omission of the metal lining from the conduit rough the dam is considered by Gaylord and Savage to e dangerous for high head outlets.

This type of control has been adopted for the scour stem at Canning dam, with the additional safeguard that a special type of gate valve called a " spectacle eye " valve has been placed immediately above the needle valve to ensure easy and complete shutting off of water for repairs to the needle valve at any time.

The spectacle eye valve which is always operated full open or full shut is provided with a ring working in a slot way. The ring is lifted when the gate is raised and fills the grooved seat of the gate, thus providing stream line flow through the valve.

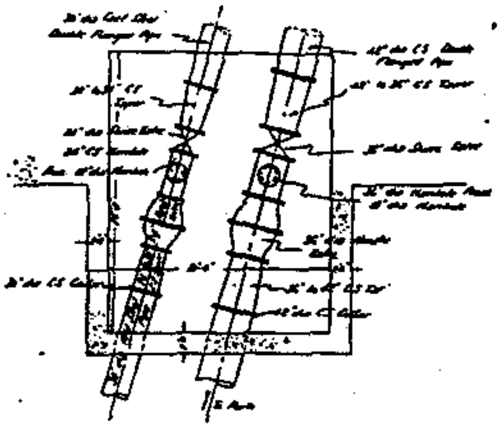


Fig. 10.—Arrangement of Valves, Downstream End, Service Pipe Lines.

A similar system has been adopted for the service control, regulation of supply from the reservoir through the pipe lines being effected below the dam by needle valves, while sluice valves are placed immediately above them.

Service Mains Through Dam.

Reduced Level of Invert of Pipes.—A pipe-head dam has been constructed above the dam site and a 30 in. pipe laid from this to the city. The R.L. of invert of this pipe line passing across the dam site = 475.00. The quantity of water stored in the basin below this level is only 8,000,000 gallons, a negligible quantity and the bed of the stream is 12 ft 0 in. below the invert. Owing to natural features this level is also the most economical for the laying of the pipe line below the dam. R.L. 475.00 has, therefore, been adopted for the invert of the service mains.

Dimensions of Pipes.

The average annual rainfall of Perth, totalling 34.32 inches, which very closely approximates in its monthly distribution the rainfall on the reservoir catchment, is as follows :—

January	0.34	inches	1.00%
February	0.45	„	1.31
March	0.79	„	2.30
April	1.65	„	4.81
May	4.97	„	14.48
June	6.92	„	20.16
July	6.57	„	19.14
August	5.63	„	16.40
September	3.41	„	9.94
October	2.21	„	6.44
November	0.81	„	2.36
December	0.57	„	1.66
	34.32	inches	100.00%

The percentage distribution of the total annual consumption of the Metropolitan Water Supply Department in each month over the last three years, was :—

January	14.40%
February	12.85%
March	12.50%
April	8.49%
May	5.31%
June	4.42%
July	4.27%
August	4.12%
September	4.73%
October	6.50%
November	9.33%
December	13.08%
	100.00%

The daily gaugings of the Canning River since 1915 indicate that approximately 7,000,000,000 gallons are available per annum, providing a sufficient storage is provided to carry over dry seasons. This figure does not provide any allowance for losses due to evaporation, etc.

The very low water consumption during June, July, August, and September, can readily be provided for by pipeheads, bores and other storages. Basing the draw-off from the Canning reservoir for the remaining months approximately on the above rates, it is possible to supply 45,000,000 gallons daily during December, January and February. The capacity of the pipe lines for peak-draw-offs and emergencies was, therefore, fixed at 50,000,000 gallons per day.

The main city service reservoirs are located at Mount Eliza at R.L. 236.00.

A 36 in. diameter main is at present laid to Kelmscott, a distance of 17 miles from Perth. At Kelmscott branch mains run to Wongong River pipe head, Canning River pipe head and Churchman Brook reservoir.

The capacity of the present 30 in. diameter main from Canning dam site to Kelmscott, plus the 36 in. diameter main Kelmscott to Perth, = 17,520,000 gallons per 24 hours with water level in the dam at R.L. 580.00.

If a 42 in. diameter pipe were laid from Perth to the dam site the discharge at water level in the dam of R.L. 580.00 would be 32,530,000 gallons per 24 hours. That is, the discharge through the two pipes would be 50,050,000 gallons daily. When the city demand became sufficient, the discharge capacity at lower levels than R.L. 580.00 could be increased by inserting a breeches pipe in either main below the dam and laying a third main.

The dimensions of the pipes through the dam were fixed at 30 in. and 42 in. diameters, to give 50,000,000 gallons per day capacity.

The 42 in. diameter main to the city would not be laid until the city's requirements demanded it, and this pipe through the dam in conjunction with the scour pipe will be available to control flood waters during construction operations.

Breeches pipe connections at a later date will increase the capacity of draw-off to any required figure.

Scour Pipe Through Dam.

Size of Pipe.—The scour pipe has 3 services to fulfil :

(*a*) During construction with the assistance of the service mains to pass the water flowing in the stream through the wall on which construction is proceeding;

(*b*) To empty the reservoir if required; and

(*c*) To keep the intake level of the service mains free from silt by occasional flushing.

The invert of the 42 in. diameter service main, which is available as a scour, is at R.L. 475.00.

The invert of the scour pipe at intake is set at R.L. 467.50.

The length of the scour pipe is approximately 132 feet from intake to discharge end, and that of the 42 in. diameter main approximately 142 ft. 0 in.

The highest gauged flows in the river for any period were in September, 1923, when over a span of 11 days an average of 1,024 cusecs was recorded.

A maximum velocity through the pipes of approximately 50 feet per second has been allowed for, if required specially during construction operations. This velocity is frequently exceeded in American practice, but is above general English practice limits. Under normal construction conditions, however, 40 ft. per sec. flow through the pipes is set as the maximum and all surplus water above their capacity is to be passed through a gap left in the wall as provided for in the specification. The 30 in. diameter pipe line, in which all scour cocks would be open, will also assist materially in discharging the flow. The balancing effect of the basin upstream as the wall rises will also aid in the control of floods.

The discharges of various diameter scour pipes were investigated. It was found as shown below that a 36 in. diameter pipe would satisfactorily deal with the average river flow in conjunction with the 42 in. diameter main, and also of a size suitable for requirements (*b*) or (*c*).

As will be discussed later, a needle valve will, after the dam has reached a certain height, be attached to the discharge end of the scour pipe to control the discharge under high heads. With the maximum velocity of 50 ft. per second the discharge from the 42 in. and 36 in. pipes would be 834 cusecs.

To determine the head at which this velocity of 50 ft. per sec. is attained, Tutton's formula was used

Velocity head in bell mouth.—

$V^2 = 2gh$; 38.8 feet head.

Head lost in 36 in. diameter pipe conduit—132 ft. 0 in. long from

$$h = \frac{flv^n}{d^x}$$

where h = head lost in feet.
l = length in feet.
v = velocity.
d = diameter of pipe in feet.
n = constant.
x = constant.
f = coefficient of friction.

Values in Tutton's formula as used by Blakeborough & Sons Ltd., dealing with water under high velocities are as under :

$n = 1.96$
$x = 1.29$
$f = 0.00068$

$\therefore h = 53.2$ ft. head lost in 36 in. pipe; *i.e.*, velocity of 50 ft. per sec. in 36 in. scour pipe is attained when W.L. $= 467.50 + 38.8 + 53.2 = 559.50$.

Head lost in 42 in. diameter pipe conduit = 41.04 feet.

That is to say, the velocity of 50 ft. per second in 42 in. diameter main is attained when W.L. = 554.84.

Therefore, when the wall height approaches 90 ft. 0 in., the maximum allowable discharge through the pipes occurs and with the large storage capacity of the basin upstream and the assistance of the 30 in. diameter service main only portions of floods will top the wall under construction.

Control. Service Mains Upstream.

It was decided to have 3 levels at which draw off may be made, namely, R.L.'s 475, 530, and 585.

The method of draw off to be adopted is by a vertical wet well shaft which is to be built out from the upstream face. The walls of the shaft are to be of concrete reinforced where necessary to sustain full hydrostatic pressure due to the water in the reservoir, assuming the wet well to be empty. This condition may occur during repair work to pipe intakes, etc.

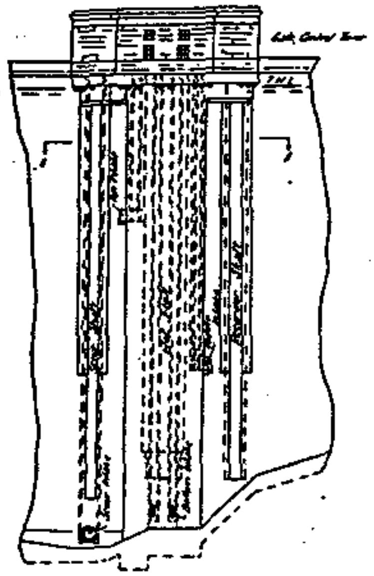


Fig. 11.—*Elevation showing Wet Well, Scour and Recorder Shafts.*

The basis of the design of all fittings connected with the dam has been that wherever possible the fittings shall be removable; and where not removable they shall be concrete lined or else made of some non-corroding metal.

In accordance with this principle, removable rectangular gates with phosphor bronze frames set in the concrete are to be used for the control of the inlets to the wet well. The type of gate adopted is made of cast iron fitted with a pair of roller trains. This gate can be operated by a winch set on

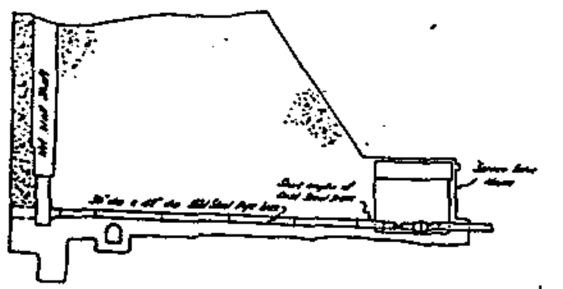


Fig. 12.—*Section on Service Pipe Lines.*

op of the dam and grooves are provided in the walls of the vet well for the lowering of the gate into position, and raising he gate when required to the surface. Should any emergency ccur in which the downstream valves could not function, hese gates are made of sufficient weight to be dropped into osition and to close against full head pressure.

The tractive effort to move this type of gate has been ound to be only 30 lb. per ton of pressure load with the ollers in clean condition. The weight of the gate must, herefore, be ample to overcome this frictional resistance hich at the lowest intake level is approximately 1,230 lb. for an 8 sq. ft. area gate. The gate would, therefore, be approximately 1 ton in weight.

The dimensions of the two services mains have been fixed at 42 in. and 30 in. diameters. These two pipes have a sectional area of 14.5 sq. ft. It is, therefore, necessary to provide inlets at each offtake level of a size permitting gates and racks easily to be handled and of a combined sectional area approximating 14.5 sq. ft. Two openings, each 4 ft. 0 in. x 2 ft. 0 in., have been adopted, providing 16 sq. ft. of area for entrance at each of the three draw off levels.

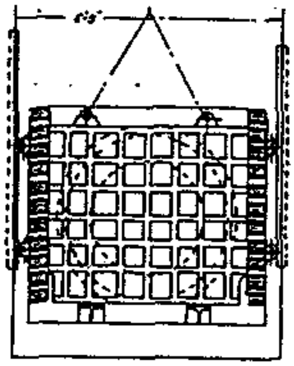


Fig. 13.—*Emergency Roller Gate.*

Gate designs providing this area of opening have been submitted by makers of the type of roller gate required, and slots are provided in the walls of the wet well of dimensions to suit the gates. A gate tower is to be built above the wet well carrying a hand-operated travelling winch. The gates are raised or lowered by means of a wire rope connected to the winch and special safety clips are to be provided above each groove so that the rope may be disconnected from the winch and yet prevented from falling in the slotway.

Trash Racks.—The flow from the Canning reservoir catchment at the dam site is never turbid, the water, even in

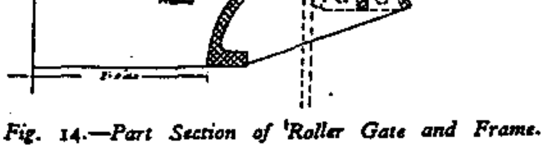


Fig. 14.—*Part Section of Roller Gate and Frame.*

flood conditions being clear. It has also been found in the adjoining Churchman Brook reservoir that, at depths exceeding 30 to 40 feet from the surface, the fine screens catch practically nothing, small fish, tadpoles, etc., only occurring when the water level falls to within the above distance above the screens. As draw-off at the Canning reservoir can always be regulated to be at depths exceeding 30 ft. 0 in. below the surface of the water, it has not been considered necessary to provide for fine screening. Movable trash racks operated in a similar manner to the emergency roller gates are provided to work in a groove just upstream of the roller gate groove. These trash racks are constructed of mild steel flat bars at 2 in. centres in a channel iron frame, and provided with 4 rollers. In case of a wire operating rope breaking, an emergency fishing tackle has been devised enabling connection to be made with the rack for lifting out. If from future experience fine screens are found to be necessary, the design of the trash rack is such that the fine screen may be incorporated with it.

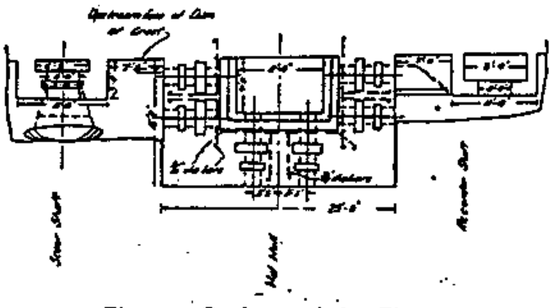


Fig. 15.—Section at A.A.—Fig. 11.

Service Mains—Downstream Control.

To avoid the drawing and erosion which occur in sluice valves when used to regulate discharge in pipe lines from high-pressure outlets, needle valves have been inserted. Immediately upstream of each needle valve is placed a standard double flanged sluice valve. This valve will permit removal or repair of the needle valve at any time. The sluice valve will never be other than fully open or completely closed and, therefore, will suffer a minimum of wear. As the loss of head is negligible and the reduction in cost is considerable, 24 in. diameter valves have been placed in the 30 in. diameter service main, and 36 in. diameter valves in the 42 in. diameter service main. Cast steel taper specials enable this to be done. The 24 in. and 36 in. diameter sluice valves are hand-operated through suitable gearing, and are provided with 4 in. diameter and 6 in. diameter by-passes respectively. The needle valves are hydraulically and manually operated.

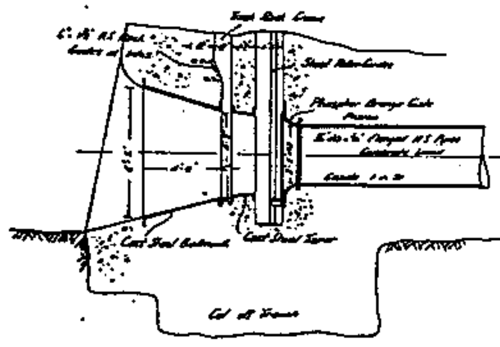


Fig. 16.—Detail of Inlet End of Scour Pipe Line.

Scour System—Upstream Control.

The control of the upstream end of the scour system is by means of trash rack and roller emergency gate functioning in a similar manner to the various trash racks and emergency roller gates at the various service main intakes. A bell mouth entrance is provided and stream lining is effected as shown in Fig. 16. The gate frame of phosphor bronze is provided with a flange at the back to which connection with the 36 in. diameter pipe is made.

Scour System—Downstream Control.

In view of the high velocities involved and the great amount of energy released at the discharge outlet, considerable research was made into this phase of the design.

Messrs. Gaylord and Savage's "High Pressure Reservoir Outlets" was studied carefully, also a paper read by Prof. Sutton before the South African Institution of Engineers, 8/11/28. The design was also discussed by correspondence with Messrs. Blakeborough & Sons Ltd., and Messrs. Glenfield and Kennedy of England, the recognized British authorities on such problems. Copies were also received, and studied, of correspondence between Messrs. Glenfield and Kennedy and the Bureau of Reclamation United States of America on high-pressure outlet design. Numerous articles in technical publications were also consulted.

The conduit velocities in various Bureau of Reclamation undertakings* do not exceed 68 ft. per sec. at normal designed capacity in the case of the dams studied. They are, on the average, about 30 ft. per sec. The maximum figures recorded are 70 ft. per sec. in the cases studied.

The total available head at the intake of the scour main of the Canning dam is 185.50 feet.

In order to ensure a maximum velocity not exceeding approximately 50 feet per second in the main, Blakeborough and Sons Ltd., recommended that a needle valve 36 in. diameter at the intake end, and 28 in. diameter at the discharge end, be used.

As shown by the following calculations, which were arrived at by trial and error, a velocity of 52 feet per second is attained.

Velocity head in bellmouth = 42 feet head lost.

Using the method adopted and the coefficients described under the design of the scour pipes, it is found that the head lost in pipe conduit, 132 ft. 0 in. long = 57.5 feet.

Velocity head lost in needle valve.—

Spouting velocity at needle valve outlet = 86 ft. per sec.

The head lost = 73 feet.

Friction head lost in needle valve (from Blakeborough's formula).—

$$H = \frac{V_o^{1.83}}{214 d^{0.33}};$$

where H = head lost, ft.
V_o = spouting velocity, ft. per sec.
d = diameter in feet.
= 11.5 feet head lost.

Total head lost.—

Intake screens, etc.	...	1.00 ft.
Velocity head in bellmouth	...	42.00 „
Friction loss in pipe	...	57.50 „
Velocity head needle valve	...	73.00 „
Friction loss in needle valve	...	11.50 „
Total	...	= 185.00 ft.

Total available head = 653.00 − 467.50 = 185.50 feet.

Energy in jet.—

Energy lost in friction.		
Intake loss, etc.	...	1.00 ft.
Friction in pipe	...	57.50 „
Friction in needle valve	...	11.50 „
		70.00 ft.

∴ Net energy head = 185.50 − 70.00 = 115.50 ft.
∴ Horse power of jet = 4,825 h.p.

There are three methods of absorbing this very large amount of energy.

*Vide Messrs. Gaylord and Savage.

1. Allow the jet to fall into a pool of such depth and size that the whole of the shock is taken up in the stationary mass of water;

2. By means of a cast iron or cast steel spoon-shaped disperser to throw the jet up into the air thus tending to break it up and destroy a considerable amount of its energy by friction with the air. A stilling pool is also required in this case but not of such depth as where no disperser is provided. This type of disperser is entirely separated from the needle valve.

3. By means of the "Glenfield" patent jet disperser to disintegrate the whole mass of water by shattering it into drops in the air a few feet beyond the escape orifice. It is claimed by the makers that the entire energy is cushioned against the atmosphere and that the water merely falls as heavy rain on the rock bed or on the surface of the collecting pond beneath. The Glenfield disperser, when used, is incorporated with the Glenfield needle valve.

The American practice is to follow method No. 1. In the case of the Arrowrock dam, erosion of the rock immediately below the dam has reached a depth of approximately 23 ft. 0 in. It would be considered extremely dangerous with most foundations to permit such an excavation in the vicinity of the toe of the dam.

C. M. Day, Chief Mechanical Engineer, Bureau of Reclamation, United States of America, in a communication to the South African Institution of Engineers, 18/6/29, writes,

> The balanced type of valve, which is operated by the proper control of the hydraulic forces within the valve, has proven so successful, and also so much cheaper than mechanically operated valves of equal capacity that the engineers of the Bureau of Reclamation are convinced they are the most reliable means available for securing close regulation of the discharge from reservoirs under heads in excess of 100 ft. and experience has shown that the needle type of valve properly protected by a gate, discharging below a dam where conditions are favourable for absorbing the destructive forces in the jet, is a most economical and satisfactory type of outlet. For reasonably short pipes leading to these needle valves where the length may not exceed 350 feet, the diameter of the pipe is proportioned to the nozzle diameter of the valve in the ratio of 6 to 5. With this ratio pipe line velocities at the maximum discharge seldom exceed 50 feet per second, and, to date, conduits connecting the reservoirs with the needle valves, whether of steel or concrete have shown practically no signs of deterioration. The jet disperser developed for attachment to, or inclusion in, discharge valves, which changes the solid jet of water from a practically solid stream to a broken stream which is spread over a considerable area, thus greatly reducing the destructive forces in the discharged water, appears to have merit, and is specially adaptable to installations where good rock conditions or space for an adequate water cushion do not obtain below a dam.

With reference to the damage immediately below the outlets, Mr. Day continues :—

> At the Arrowrock dam soundings in the pool below the dam show practically no change in bottom contour, therefore apparently the water cushion is affording ample protection. At Minatare dam the reinforced steel in the concrete jet deflectors is now exposed in places. However, as no repairs have been required after 14 years' service, and the cost of repairing these jet reflectors will be small, we consider they have proven successful under the conditions existing at that dam. At Pathfinder dam of the two 58 in. balanced needle valves in the North Tunnel outlets the outer valve discharges into the pool below the dam. The inner valve impinges on the sloping rock wall surface which is coated with a green slime that may act as a lubricant. In any event no damage has occurred below the dam from the discharge from these two valves, or from the discharge of water from the tunnel below the 6 valves in the south tunnel where the water, at times amounting to 5,000 second feet, drops over 60 feet into the pool.

> The experience at Pathfinder and Arrowrock dams indicates that with reasonably sound rock below the dam a water cushion will successfully absorb the destructive forces in the jets from the needle type of valve.

The design adopted for Canning reservoir outlet comprised :—

(*a*) 36 in. to 28 in. hydraulically and mechanically-operated needle valve;

(*b*) Separate spoon shaped cast iron disperser; and

(*c*) Stilling pool.

The design was based on the following reasoning :—

1. The special type of dispersing needle valve is more costly than the ordinary stream flow needle valve. In addition the wear and therefore maintenance must be greater in the case of the special type of needle valve;

2. The needle valve will only be operated for short intervals each year after construction is completed;

3. The rock below the dam is solid unweathered granite which is equal to any rock in resisting erosion;

4. The spoon-shaped disperser is a very cheap casting bolted to a concrete foundation very easily replaced when worn out and quite separate from the needle valve.

5. The construction of a small wall across the stream bed below the dam provides a sufficient stilling pool.

Design of Bywash.

The principal consideration in the design of a bywash will be the probable run-off, which depends on the type and area of the catchment, and the likely intensity of rainfall.

The required discharge will depend on the height of the dam, the capacity of the basin at top levels, and the configuration of the site.

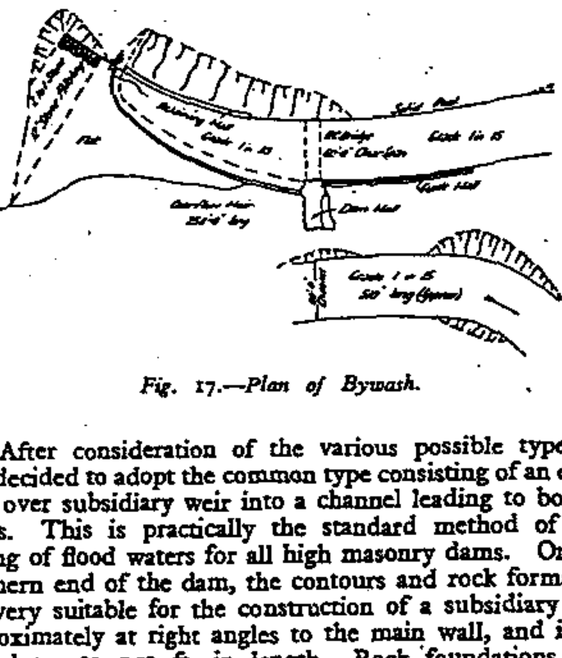


Fig. 17.—Plan of Bywash.

After consideration of the various possible types, it was decided to adopt the common type consisting of an overflow over subsidiary weir into a channel leading to bottom levels. This is practically the standard method of disposing of flood waters for all high masonry dams. On the southern end of the dam, the contours and rock formation are very suitable for the construction of a subsidiary wall approximately at right angles to the main wall, and if required up to 250 ft. in length. Rock foundations also exist for the channel to convey the flood water down to river bed well below the dam.

The area of the catchment is 282 square miles. The catchment is heavily timbered with jarrah, red gum and blackbutt forest, with a considerable amount of undergrowth. With the exception of the portion in the vicinity of the dam site, very little exposed rock surface is included, and approximately 50% of the catchment area consists of comparatively flat ironstone gravel country. The basin is fed by three streams, Canning and Canning South rivers and Death Adder Creek. No defined stream exists through the ironstone gravel area, but a number of small creeks bring the water down in flood periods. It will thus be seen that the run-off factor from the catchment will not be a high one. It is regrettable that while two gauging weirs have been erected on the river, neither has been of sufficient capacity to register the maximum winter floods. Information supplied by the ranger as to the maximum flood level reached at one point, together with the cross section of the river at this point, and an assumed velocity of 5 feet per second gave a discharge of 1,814 cusecs.

Another flood mark at a point down the river by similar calculations gave 2,000 cusecs.

These figures gave an average maximum discharge of catchment of approximately 7 cusecs per sq. mile.

The Mundaring reservoir in the same Darling Ranges and approximately 20 miles north of Canning reservoir has a very similar catchment area of 569 square miles. The maximum recorded overflow to date after 25 years records = 5,700 cusecs or approximately 10 cusecs per square mile.

The Harvey reservoir of catchment area 141 square miles, during June, 1930, had a maximum discharge lasting only a few hours of 3,214 cusecs, equivalent to 22.8 cusecs per square mile. The rainfall was almost a record for the month and the catchment had been thoroughly saturated by early rains. The Harvey catchment which is in the Darling Ranges, is more rugged and rocky with a greater proportion of cleared land than in the case of the Canning or Mundaring catchments, and, therefore, should give a greater percentage run-off.

Churchman Brook catchment of approximately 6 square miles area, in 1926, when approximately 90 inches (including 21 inches in July) of rain fell, did not at any time record a run-off exceeding 30 cusecs per square mile. This catchment is situated approximately 3 miles south of Canning dam site. 1926 was the wettest year for 50 years.

The heaviest rainfalls, for a 24-hour period, recorded in the Darling Ranges since records were kept, are given below.

Wongong Valley	... ...	390 points.
Victoria reservoir	... ...	350 „
Canning Valley	... ...	530 „

Various formulae exist for calculating the maximum off-flow from an area. Each area has its own characteristics. As stated by Turneaure and Russell, "Each stream is an individual problem which must be studied in the light of all available and pertinent information."

1. Fanning's formula, applicable to New England, United States of America, streams

$$Q = \frac{200}{M^{\frac{1}{6}}}$$

in which Q = discharge in cubic feet per second per square mile; and M = drainage area in square miles.

In the case of Canning, adopting the above formula
Q = 77.82 cusecs per square mile.

2. Murphy's formula (Turneaure & Russell, p. 71)

$$Q = \frac{46790}{M \times 320} + .15$$

derived from data of streams in the north-eastern United States of America, applied to Canning, gives a discharge of 93 cusecs per square mile.

3. The discharge of the Burrinjuck dam in May, 1925, was possibly the greatest flood since white settlement. The catchment area is 5,000 square miles, over which an average of 6½ inches of rain fell during 4 days. The average rate of inflow over 9 hours exceeded 340,000 cusecs, or 68 cusecs per square mile. The catchment area comprises principally granite and compact slates and shales, with scattered deposits of limestone. The area embraces undulating table land with an elevation of 1,000 to 2,000 ft. and also rugged mountains rising 4,000 to 5,000 ft. above sea level. (H. H. Dare, The Transactions, Vol. IX., 1928, The Institution of Engineers, Australia.)

The Burrinjuck catchment, from Mr. Dare's description and from actual comparison by the author, although larger than Canning catchment, will have a much greater run-off per square mile.

Discharge over Weir.—The formula generally adopted is the Francis. It applies to sharp crested weirs, and must be modified for broad-crested sections.

For sharp crested weirs it is
$Q = 3.33\ L.H.^{3/2}$ where—

Q = discharge cusecs;
L = length of crest in feet;
H = depth of water over crest.

If a sharp crested weir were assumed, the maximum capacity of the 250 ft. long bywash without overflowing the dam = 12,250 cusecs, equivalent to 43.59 cusecs run off per square mile, i.e., approximately 6 times the highest recorded flood discharge. As an extra precaution, emergency provision has been made for the dam to overflow 12 in. depth over the top of the wall when the bywash capacity increases to 15,435 cusecs, to which must be added approximately 3,000 cusecs flowing over the dam.

The storage in the basin at this level per 12 in. rise is approximately 300,000,000 gallons, which is equivalent to 555 cusecs for 24 hours, and this will assist the bywash by flattening the peak of the discharge until the water level reaches the maximum height, i.e., 7 feet over the bywash.

Design of Overflow Weir.—Actually the weir would not be sharp crested and considerable research was made into the type of overflow weir to be used to obtain the greatest efficiency; coefficients given by Messrs. Hazen and Williams have been adopted. Owing to the length of the weir, in all calculations of discharge any effect due to end contractions was neglected.

The coefficients for the type of weir shown in Fig. 18, which was finally adopted, are given in Table III.

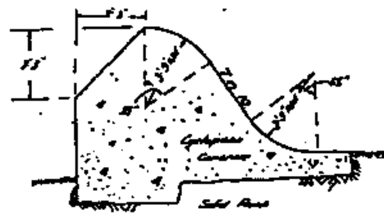


Fig. 18.—Typical Section of Bywash Weir—Type K.

TABLE III.

Head in feet over crest of weir.	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Coefficient	0.971	1.040	1.092	1.126	1.146	1.163	1.177	1.190

It will be seen that if this type, which was known as type K, is adopted the discharge over the weir will be approximately 19% greater than given by the formula.

The capacity of the bywash, including 12 in. over the wall becomes (15,435 x 1.19) + 3,000 = 21,367 cusecs. equivalent to 75.7 cusecs per square mile.

The next stage is to design the bywash channel to convey the waters below the dam. The whole hillside, below the dam, consists of solid granite so that excessive velocities may be permitted during peak overflows.

The Burrinjuck dam bywash channels, in similar country, have gradients of 1 in 14 to 1 in 5.

An invert gradient of 1 in 15 has, therefore, been adopted for Canning dam, as this will enable the water to be discharged about 500 ft. below the dam over a ledge of rock in a position which prevents any water working its way back to the dam.

Adopting Hazen and Williams formula for open channels

$$V = c\, r^{0.63}\, s^{0.54}\, 0.001^{-0.04}$$

where $c = 70$

A 60 ft. 0 in. channel flowing 4 ft. 0 in. deep will discharge 11,280 cusecs with an average velocity of 47.07 ft. In actual practice, as shown by photographs of Burrinjuck discharges, the disturbance and turbulence are so great that no formula can be accurate. Provision has been made in the channel for a flow up to 7 ft. 0 in. in depth, which should amply cover all contingencies.

The outlet end of the bywash channel will be fanned out during construction to spread the overflow as much as possible. The extent of this fanning out will be determined from the fact that it may be possible economically to open up a secondary spalls quarry in this location.

APPENDIX I.

Some details of the reasons for the adoption of certain values for the variables assumed in the specification for the design of the cross section of the wall are given below.

Uplift.—Uplift is provided for in the design of all modern masonry dams. Owing to the variation in the character of foundations, it is necessary to examine each particular site in order to decide what allowance for uplift is necessary.

The appended table refers to some recently constructed high dams in the design of which—with the exception of Elephant Butte—extreme values of the percentage of the base assumed to be subjected to uplift pressure have been adopted.—

Wachusett ...	... 66%
Cross River	... 66%
Elephant Butte	... 33%
Olive Bridge	... 66%
Kensico ...	... 66%
Lock Raven	... 66%

Foundations for the Canning dam are in hard sound granite rock; a deep cut-off trench is to be provided and behind this cut-off trench a drain to relieve uplift.

After reference to the latest authorities, an allowance for uplift of 50% full hydrostatic pressure at the upstream heel, reducing by straight line variation to zero at downstream toe exerted over 50% of the area, has been adopted for Canning dam design.

With a view to making the design of similar structures capable of greater accuracy as regards uplift, provision will be made for the measurement of uplift pressure at various points along the base of the dam with various heads of water in the reservoir.

Drainage and Internal Inspection.—Every modern dam is provided with an internal drainage system combined with inspection tunnels and drainage outlet tunnels for discharge of the collected water. The advantages to be gained by such a system are:—

(i) Prevention of seepage on downstream face;

(ii) Elimination of uplift on any horizontal joint;

(iii) By using a more impermeable concrete on the upstream section of the wall, a less impervious, and therefore cheaper, concrete can be used downstream of the drainage system.

(iv) Inspection tunnels enable leakage to be localized and watched; and

(v) Leakage through contraction joints can be collected and led to drainage tunnel without appearing on downstream face.

Experiments indicate rate of flow increases as the thickness of concrete decreases, but in a much larger inverse ratio.

To cut-off theoretically all internal drainage in the wall it would be necessary to have a continuous porous vertical sheet throughout the dam. This is not practicable. In practice the seepage will be along horizontal construction joints, vertical cracks or contraction joints. Seepage along the latter will be dealt with later. In a large dam of this description defective work allowing seepage along the *horizontal* joint which could find access to the downstream face would extend at least 6 feet near the upstream face. Therefore any system of tubes, spaced 6 ft. apart between inner edges, would collect the water and would effectively eliminate any uplift on any horizontal joint.

The type of drainage blocks, distance from upstream face, and distance apart can only be decided from what has proved *effective* in other dams. Table A.

TABLE A.

Dam.	Details of tubes.	Distance apart centre to centre.	Distance of centre of drainage tube from upstream face at upper inspection gallery.	Distance from upstream face at lower inspection gallery.	Approx. batter.
Kensico	Porous blocks 36 in x 36 in.	15 ft.	10 ft.	30 ft.	1 in 14
Arrow Rock	Core 16 in. diar. 8 in. diar. tubes	10 ft.	12 ft.	25 ft.	Vertical and 1 in 13
Elephant Butte	12 in. dia. tubes	4 ft. staggered.	5 ft.	18 ft.	Vertical
Cataract	6 in. x 6 in. tubes.	—	6 ft.	6 ft.	
Olive Bridge or Ashokan	Porous blocks 36 in. x 36 in. Core 12 in. diar.	12 ft.	11 ft.	26 ft.	1 in 15
Cordeaux	18 in. x 18 in. porous drains	—	10 ft.	12 ft.	Vertical
Avon	18 in. x 18 in. porous drains	15 ft.	8 ft.	17 ft.	Vertical

Following average practice in other dams, the cut-off trench for Canning dam was fixed with its upstream face 3 ft. 6 in. downstream of the upstream heel of the dam and this distance is maintained throughout the dam.

In order to seat the dam on solid rock free from weathering, surface jointing, etc., and also to provide a *thrust* face against the downstream toe as a safety factor against sliding, the foundations over the whole area of the base are carried a minimum of 5 ft. below the natural surface of the rock.

The passage of water under pressure, other things being equal, will vary inversely with the distance to be travelled (i.e., in the case of a cut-off *trench* with twice the depth plus the width or $2d + w$.), and directly with the hydrostatic pressure.

To prevent water finding its way along a fissure below the cut-off trench, the trench has been specified to go at least 1 ft. 6 in. below the fissures, the existence of which was indicated by the borings. A 24 in. width has also been adopted as a minimum economic width for the trench at the higher levels.

At the lowest point on the section, R.L. 463, after allowing for the foundations being 5 ft. below natural surface, it is found necessary to design the cut-off trench to go down an additional 12 ft. in order to be 1 ft. 6 in. below the fissure level as estimated from the bore information.

In agreement with the widths generally adopted in other dams, the width of the cut-off trench at this point, i.e., where R.L. of natural surface of rock = 463.00, has been fixed at 10 ft.

At R.L. 653.00 the top of the dam, the depth of cut-off trench = nil.

For economic working 24 in. has been adopted as the minimum width of the cut-off trench at the higher levels.

The provision of the cut-off trench and other precautions specified to ensure the bonding of the concrete to the rock foundation during construction, have the effect of making the entry of water under pressure into the foundations as difficult as possible. There will, however, be a certain proportion which will make its way along the junction of the concrete and rock, or through small porous layers in the cut-off trench. This water will, unless removed, create an uplift force which may, however, be dealt with in several ways.—

(1) Horizontal holes may be drilled at intervals across the site in a row just below the cut-off, and provided with a free connection to tail water;

(2) Internal drainage tubes of dam may be continued through base of the dam into the foundations; and

(3) Construct an open pipe and stone drain longitudinally the full length of the dam, and just downstream of the cut-off trench, the drain to have connection through the base of the dam by a series of tubes with the lower inspection gallery. This system has the advantage that the actual pressure underneath the dam may be measured at a series of points.

System No. 3 was adopted as being more effective than Nos. 1 and 2 in a non-stratified rock, such as granite. No. 3 system deals with the full length of the base, systems Nos. 1 and 2 only tap at isolated points.

With the non-stratified granite foundations of Canning dam, the main object has been to make seepage as difficult as possible, and then to remove at once such water as may get through. The sub-foundation drain has, therefore, been placed just downstream of the cut-off trench and connection with the lower inspection gallery will be made by means of 4 in. diameter galvanized wrought iron pipes. The drain will have very little water to carry and has been designed 2 ft. 0 in. deep by 2 ft. 0 in. wide throughout, with 2 ft. 0 in. lengths of 6 in. diameter concrete pipes in the centre. The pipe joints are to be 1 in. apart, and the pipes to be surrounded by hand-packed granite spalls, 6 in. maximum and 2 in. minimum sizes.

In order to measure the sub-foundation uplift pressure on any section of the foundation the longitudinal sub-drain has been divided into 50 ft. lengths, each being shut off from the adjoining section by a 12 in. thick concrete wall across the drain. A 4 in. diameter drain pipe leading to the lower inspection gallery taps each 50 ft. 0 in. section at the lowest point in that section. At the discharge end of this 4 in. pipe, a plug with connections for a small pressure gauge can be screwed in whenever desired and the actual pressure along the particular 50 ft. 0 in. section thus measured. By this means, should considerable seepage occur, the area of seepage can be localized, and steps can then be taken to deal with the trouble.

Weight per Cubic Foot of Material on which Design is to be Based.—Seven granite samples from the quarry site at Canning reservoir were tested by the University of Western Australia in 1928, giving the following specific gravities:—2.59; 2.60; 2.61; 2.56; 2.54; 2.58; 2.59. Average = 2.58. Weight = 161.25 lb. per c. ft.

The weight of concrete is very little influenced by changing the proportions of the ingredients.

A block of concrete, approximately 1 c. ft. in volume, composed of Canning reservoir quarry granite 2½ in. aggregate, proportions 1 : 3 : 6, weighed after 2 months 145 lb. per c. ft. after very careful measurements to ascertain the true volume.

Assume 15% plums throughout the structure, the weight of the cyclopean concrete would be as follows:—

145 lb. x 0.85	=	123.25 lb.
161.25 lb. x 0.15	=	24.19 lb.
		147.44 lb.

The design was, therefore, based on a unit weight of 145 lb. per c. ft. for the masonry throughout the dam. Any additional weight due to the inclusion of plums is to be regarded as an extra factor of safety for the stability of the structure. The limiting toe pressures adopted are amply safe for any increase that may be caused by any excess in the actual weight of the concrete over the assumed weight.

For a dam such as Canning, founded on solid granite where toe pressures become determining factors only in the extreme bed of the gorge and for a length of approximately 100 feet along the section, knowing that either theory is conservative in such foundations the formula $P_i = P_v \sec^2\theta$ is adopted, as recommended by Creager, and in accordance with Bouvier's theory; θ being the angle made by the resultant of forces with the vertical.

Tests have been made of the compressive strengths of various rock types both in Australia and elsewhere and it is found that, for granite, the tests indicate a range of strength from 432—1,497 tons. In the case of the Canning reservoir site two 6 inch cube test blocks were compressed to the limit of the University of Western Australia's testing machine, viz. 200 tons, without breaking. This is equivalent to a compressive stress of 800 tons per square foot without crushing.

Limiting toe pressures have been adopted in various existing dams, which vary from 8 to 30 tons on the horizontal plane.

For the Canning dam design a maximum inclined compressive stress of 14 tons per sq. ft. was adopted, the pressures to be calculated on an average unit weight per cubic foot of concrete = 145 lb. The percentage of plums (approximately 15%) will increase this weight, but the high compressive strength of the granite is easily sufficient to allow for the slight increase in stress caused by the plums.

Resistance to Sliding.—In order to calculate the stability of a dam against sliding, it is necessary to ascertain a maximum allowable coefficient of friction.

Creager, p. 51, states "values of f for masonry on masonry and masonry on good rock foundations have been assumed variously between 0.6 and 0.75. In general, and for careful work, a value 0.75 is not excessive. Turneaure and Russell, p. 351, gives the following table:—

TABLE B.

Material.	Coeff. f.
Granite roughly worked on gravel and sand (wet) ...	0.41
Granite on sand (dry)	0.65
Granite on sand (wet)	0.47
Point dressed granite (medium) on like granite	0.70
„ „ „ „ „ common brickwork...	0.63
„ „ „ „ „ smooth concrete ...	0.62
Fine cut granite on like granite	0.58
Concrete blocks pressed on like concrete blocks ...	0.66
Common bricks on common bricks	0.64

Wegmann, p. 35, M. Krantz, and other authorities place the limiting value of f at 0.75.

On good construction work, with non-stratified foundations, where considerable care is taken to roughen and step the rock foundation surface and to leave keys and ties at the end of each day's concreting, authorities agree that the coefficient of friction will be at least twice that found for evenly surfaced specimens of the same materials. If, therefore, 0.7 be adopted as the limiting coefficient of friction, a factor of safety of at least 2 against sliding is provided.

The tangent of the angle of inclination with the vertical of the resultant of all the forces including uplift acting on the dam above any horizontal plane must be less than the coefficient of friction at that plane, i.e., $\tan\theta$ must always be less than 0.7.

APPENDIX II.

The top width of the wall had been assumed at 15 feet and, allowing for uplift and assuming that the resultant intersects the base at the downstream extremity of the middle third, it can be shown that the level at which the downstream face should begin to batter will be 21.6 ft. As the top of the wall is at R.L. 653.00, the wall can be battered from R.L. 631.00.

In the calculations which follow, which are sample calculations to indicate the procedure adopted, the following symbols have been used in the computations:—

ΣW_E = sum of vertical forces acting on a joint—reservoir empty.
ΣW_F = sum of vertical forces acting on a joint—reservoir full.
ΣW_{XE} = sum of moments of vertical forces about origin—reservoir empty.
ΣW_{XF} = sum of moments of vertical forces about origin—reservoir full.
ΣP_F = sum of horizontal forces acting on a joint—reservoir full.
ΣP_{XF} = sum of moments of horizontal forces acting on a joint—reservoir full.

Origin adopted was located 10 ft. upstream from upstream edge of roadway.

θ = Angle made by resultant force with the vertical.

T = distance from upstream heel to intersection of resultant with base—reservoir empty.

Minimum widths necessary for stability at planes at 10 feet intervals downwards, from R.L. 621 to R.L. 451, were calculated, neglecting for the toe pressures in the first trial.

At R.L. 621 the condition is as follows:—

TABLE C.

Level R.L. 621.00.

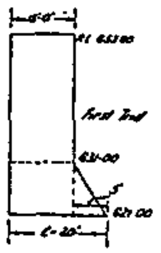
1st trial—assume l = 20 ft.

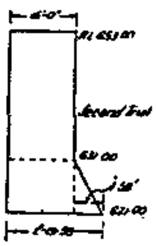
			Symbol.	Force.	Lever.	Symbol.	Moment.
Vertical forces	Masonry above R.L. 621.00	$32 \times 15 \times 145$		69,600	7.5′		522,000
		$\frac{5 \times 10}{2} \times 145$		3,625	16.7′		60,537
			ΣW_E	73,225		ΣW_{XE}	582,537
	Uplift pressure	$\frac{32 \times 20 \times 62.5}{8}$		5,000	6.66′		33,333
			ΣW_F	68,225		ΣW_{XF}	549,204
Horizontal forces	Water pressure	$\frac{32 \times 32 \times 62.5}{2}$	ΣP_F	32,000	10.66′	ΣP_{XF}	341,332

$\Sigma W_{XF} + \Sigma P_{XF}$ = 890,536

For reservoir full : $\frac{2l}{3} = \frac{890536}{68225} \therefore l = 13.053 \times \frac{3}{2} =$ 19.579 ft.

For reservoir empty : $T = \frac{582537}{73225} = 7.95.$





2nd trial—assume l = 19.58 ft.

Vertical forces	Masonry above R.L. 621.00	$32 \times 15 \times 145$		69,600	7.5		522,000
		$\frac{4.58 \times 10}{2} \times 145$		3,320	16.53		54,879
			ΣW_E	72,920		ΣW_{XE}	576,879
	Uplift pressure	$\frac{32\ 19.58 \times 62.5}{8}$		4,895	6.53		31,964
			ΣW_F	68,025		ΣW_{XF}	544,915
Horizontal forces	Water pressure	$\frac{32 \times 32 \times 62.5}{2}$	ΣP_F	32,000	10.66	ΣP_{XF}	341,332

$\Sigma W_{XF} + \Sigma P_{XF}$ = 886,247

For reservoir full : $\frac{2l}{3} = \frac{886,247}{68,025} \therefore l = 19.54$ ft.

For reservoir empty : $T = \frac{576,879}{72,920} = 7.91$ ft.

$\tan \theta = \frac{32,000}{68,025} = 0.47$ which is well within limit of 0.7 allowed.

Note.—All moments from R.L. 601.00 downwards are taken about a vertical line 10 ft. upstream of the upstream edge of the roadway. This method is used because, as will be seen, the upstream face at or about this level begins to batter in an upstream direction.

TABLE D.

		Factor	Symbol	Force	Lever	Symbol	Moment.
Vertical forces	Rectangle 653-621	$32 \times 15 \times 145$		69,600	17.5		1,218,000
" "	" 621-611	$10 \times 19.5 \times 145$		28,275	19.77		558,997
" "	" 611-601	$10 \times 25.4 \times 145$		36,850	22.7		836,495
" "	Triangle 631-621	$\frac{10 \times 4.5}{2} \times 145$		3,262	26.51		86,476
" "	" 621-611	$\frac{10 \times 5.9}{2} \times 145$		4,278	31.49		134,714
" "	" 611-601	$\frac{10 \times 6.8}{2} \times 145$		4,930	37.7		185,861
			ΣW_E	147,195		ΣW_{XE}	3,020,543
" "	Uplift	$\frac{32.2 \times 52\ 62.5}{8}$		13,081	20.73		271,169
			ΣW_F	134,114		ΣW_{XF}	2,749,374
Horizontal forces	Water pressure	$\frac{52 \times 52 \times 62.5}{2}$	ΣP_F	84,500	17.33	ΣP_{XF}	1,464,385

$\Sigma W_{XF} + \Sigma P_{XF}$ = 4,213,759

Reservoir full. $10 + \frac{2l}{3} = \frac{4,213,759}{134,114} = 31.42 \therefore l = 32.13.$

Reservoir empty. $10 + T = \frac{3,020,543}{147,195} = 20.52 \therefore T = 10.52.$

In this case it is seen that with the reservoir empty the resultant falls outside the upstream edge of the middle third. It is obvious therefore that the upstream face of the dam should begin to batter at R.L. 611.00 and the amount of offset from the vertical at R.L. 601 for the 2nd trial should be 0.2 ft.

By similar calculations the widths at elevations down to R.L. 451.00 have been obtained, such that the resultant pressures with reservoir full or empty fall just within the middle third. Table E.

It must be remembered that in these calculations there has been no limitation as to toe pressures. These may now be examined at elevation 451.00. Table F.

TABLE E.

Elevation.	y	l	Tan θ
653.00	10.0	15.0′	—
631.00	10.0	15.0′	—
621.00	10.0	19.54′	0.47
611.00	10.0	25.41′	0.568
601.00	9.7	32.50′	0.627
591.00	8.98	40.20′	0.656
581.00	8.58	47.34′	0.67
571.00	8.28	54.60′	0.65
561.00	8.04	61.83	0.686
551.00	7.85	69.03	0.688
541.00	7.68	76.26	0.693
531.00	7.63	83.22	0.693
521.00	7.56	90.27	0.695
511.00	7.49	97.32	0.696
501.00	7.45	104.34	0.696
491.00	7.42	111.33	0.697
481.00	7.39	118.32	0.697
471.00	7.37	125.31	0.696
461.00	7.34	132.30	0.697
451.00	7.32	139.29	0.697

TABLE F.
Level 451.00.

Item.	Factor.	Symbol.	Force.	Lever.	Symbol.	Moment.
Masonry above 461			1,831,620			94,221,898
Rectangle 461-451	132.33 x 10 x 145		191,878	73.49		14,101,114
Upstream triangle	$\frac{0.02 \times 10}{2} \times 145$		15	7.32		110
Downstream triangle	$\frac{7 \times 10}{2} \times 145$		5,075	142.0		720,650
		ΣW_E	2,028,588		ΣW_{XE}	109,043,772
Uplift	$\frac{202 \times 139.35 \times 62.5}{8}$		—219,912	53.76		—11,822,469
			1,808,676			97,221,303
Vertical component	2.69 x 122 x 62.5		20,511	8.36		171,472
		ΣW_F	1,829,187		ΣW_{XF}	97,392,775
Horizontal water pressure	$\frac{202 \times 202}{2} \times 62.5$	ΣP_F	1,275,125	67.33	ΣP_{XF}	85,854,166

$$\Sigma P_{XF} + \Sigma W_{XF} = 183,246,941$$

Reservoir full: $y + \frac{2l}{3} = \frac{183,246,941}{1,829,187} = 100.18$

Reservoir empty. $y + \frac{l}{3} = \frac{109,043,772}{2,028,588} = 53.75$

$$\therefore \frac{l}{3} = 46.43 : l = 139.29$$

$$y = 7.32$$

$\left\{\begin{array}{l}\tan\theta \\ \text{reservoir full}\end{array}\right. = \frac{1,275,125}{1,829,187} = 0.697. \quad \theta = 34°53.'$

Toe pressure $= \frac{2\Sigma w}{l} \cdot \sec^2\theta = \frac{2\Sigma W_F}{l} \times (1.219)^2.$

Reservoir full : Toe pressure $= 2 \times \frac{1,829,187}{139.29} \times 1.486$

$= 17.41$ tons sq. ft.

Reservoir empty : Toe pressure $= 2 \times \frac{2,028,588}{139.29}$

$= 13.00$ tons sq. ft.

To obtain vertical toe pressures (1) reservoir full and (2) reservoir empty, use the formula

$$Pr' = 2\,\frac{\Sigma W}{l}\left(2 - \frac{3\mu}{l}\right).$$

$$Pr'' = 2\,\frac{\Sigma W}{l}\left(\frac{3\mu}{l} - 1\right)$$

To obtain downstream toe pressures on inclined plane normal to resultant, multiply by $\sec^2\theta$ for reservoir full.

Discussions & Communications.

A GRAPHICAL SOLUTION OF STATICALLY INDETERMINATE FRAMES.

By R. C. Robin.*

Mr. C. N. Ross (Member, Brisbane Division).—A comparison of the author's method of dealing with statically indeterminate frames with that of distributing the fixed-end moments† was interesting. Two of the author's numerical examples had been selected for this purpose.

The first example was that shown in Fig. 21 of the paper. This frame is shown in Fig. A.

The ratio $\frac{I}{l}$ to $\Sigma\,\frac{I}{l}$ for all members meeting at any one joint is shown at each end of each member, also the ratio $\frac{I}{l^2}$ to $\Sigma\,\frac{I}{l^2}$ for all the columns is shown below each column.

The writer stated that, briefly, the Cross method was as follows:

(1) Assuming that each end of each member in the frame is clamped, write down the fixed end moments due to the given load system, taking clockwise moments as positive and counter-clockwise moments as negative. These are the first figures shown at each end of each member. (The calculated moments are 1,000 times those shown on the figure.)

(2) Unclamp each joint in turn. The resultant moment at any joint is resisted by all the members meeting at that joint in proportion to the relative stiffness $\left(\frac{I}{l} \div \Sigma\,\frac{I}{l}\right)$ of each member, so that the moment on that joint is zero. This is called distributing the moments.

(3) Each distributed moment induces a moment at the opposite end of the member which for the time being is considered clamped. These are called the carry over moments and, for members of uniform section, are equal to one half of the moment causing them.

Therefore, place a moment equal to one half of each distributed moment at the opposite end of the same member.

Now show the column moments from the first three operations at the first set of brackets shown on the figure. The shear in any column equals the sum of the moments at its ends, divided by its length.

Therefore shear in $AB = \frac{-123.0}{16} = -7.69$

" " $FC = \frac{11.34}{20} = +0.567$

" " $GD = \frac{24.96}{25} = +1.00$

" " $LE = 0$

therefore the unbalanced shear $= -7.69 + 1.567 = -6.123.$

Each column resists this shear in proportion to its $\frac{I}{l^2} \div \Sigma\,\frac{I}{l^2}$ for all the columns.

Therefore AB takes a shear of $+ 0.507 \times 6.123 = 3.105$
" FC " " " " $+ 0.216 \times 6.123 = 1.322$
" GD " " " " $+ 0.1385 \times 6.123 = 0.848$
" LE " " " " $+ 0.1385 \times 6.123 = 0.848$

These shears induce moments at each end of each column equal to shear $\times$ half height of column (since bases of columns are fixed).

Therefore moments for $AB = 3.105 \times 8 = 24.84$
" " " $FC = 1.322 \times 10 = 13.22$
" " " $GD = 0.848 \times 12.5 = 10.60$
" " " $LE = 0.848 \times 12.5 = 10.60$

Place these moments on the columns as shown in the figure.

This completes the first cycle of operations. Each succeeding cycle is similar, that is, distribute moments, then carry over, then balance shears in columns.

*For text of paper see THE JOURNAL, Vol. 5, No. 5, May, 1933, p. 145.
†"Analysis of Continuous Frames by Distributing the Fixed-End Moments" by Hardy Cross, *Proceedings of the American Society of Civil Engineers*, May, 1930.

TRANSACTIONS OF THE INSTITUTION

The Construction of the Canning Dam, Western Australia.

BY RUSSELL JOHN DUMAS, M.E., *Member.** AND VICTOR CRANSTON MUNT, B.A., B.E. *Associate Member.**

Summary.—The construction of the Canning Dam, now being built in Western Australia to supply water for the City of Perth and Metropolitan Area, has various novel features. This paper deals with the plant employed, the method of construction, and the costs of the more important items, particular reference being made to the use of bulk cement and the very substantial savings effected by this method of construction.

INTRODUCTION.

The construction of the Canning Dam, now being built in Western Australia to supply water for the City of Perth and metropolitan area, has various features which are either novel to Australian contruction methods or are extended developments of previous practice.

For the first time in Australia, so far as the authors are aware, bulk cement is being used. The method of placing concrete by means of chuting and distributing through a compound truss of such large radius is also an advance on previous Australian practice.

The work is being carried out as part of the Western Australian Government's Employment Scheme, approximately 90 per cent. of the employees being engaged on a part time basis.

A paper covering the details of the design of the Canning Dam was published in THE JOURNAL, Vol. 5, No. 9, September, 1933. This design provided for a wall 175 ft. high. It was decided later to raise the wall, and the dam now under construction will, when completed, stand 218 ft. above the river bed.

The main features, including allowances for maximum toe pressures, uplift, weight of concrete, etc., are the same as in the original design. The major differences in the new design are :—

(*a*) A length of 360 ft. of the southern wing of the dam will as an overflow section, discharging into a channel, blasted from the rock, which will convey the floodwaters down the granite hillside well below the dam.

(*b*) The pre-cast porous drainage blocks have been superseded by a system of practically vertical 8 in. diameter ducts in the concrete wall set at 5 ft. intervals along its full length.

This paper will deal with the plant employed, the method of construction and the costs of the more important items.

INITIAL STAGES.

The report recommending the raising of the height of the dam was submitted to Cabinet on 6th Sept., 1933. This report was adopted and Cabinet instructions to proceed with the work were given on 18th Sept., 1933. At that date no drawings existed for the dam of increased height and no consideration had been given to the layout or design of plant. As one object of carrying out the work was to provide useful employment for workers on sustenance, it was imperative that the employment stage be reached as rapidly as possible. A limited number of men was employed immediately on preliminary work, such as preparation of camp sites, access roads, erection of cubicles, water supply, etc. Portable air compressors were hired and the blasting of rock from the by-wash channel and in the bed of the stream for the foundations was the first work undertaken.

In the meantime the amended design for the dam was expedited to the utmost. Draughtsmen and designing staff worked overtime on designs and layout for the various items of plant, every portion of which, excepting machinery such as crushers, concrete mixers, etc., was designed by officers of the Public Works Department. The greatest concentration in the drawing office was on the design of the tower and chuting plant. Tenders were called for the construction and erection of this unit. A programme was drawn up by which all other units of plant would be purchased and erected by the time the chuting plant was ready for operation. In addition, sufficient of the foundation was to be excavated to permit concreting to commence. It being summer time, the river was diverted through a series of 30-in. dia. pipes and the loose overlaying rock blasted out of the river bed. It had been known from diamond drill bores that a diorite dyke, approximately 20 ft. wide, existed along the bed of the river. At the junctions of the diorite with the granite, on each side, approximately 4 in. of semi-decomposed rock occurred. A shaft 6 ft. by 3 ft. was sunk across each junction for a depth of 57 ft. below the original bed of the stream. At this depth the junction material was much more compact. Grout holes were drilled across the joints, but it was impossible to force in any grout. The shafts were filled with 1 : 2 : 4 concrete with a very low slump and this was handpacked into position to fill completely the shafts and act as a seal across the joints. Great care was exercised in bonding the shaft concrete to the concrete in the body of the dam.

This work was completed and a sufficient area of cut-off trench and foundations excavated to permit concreting to start immediately the plant was ready. On 20th April, 1934, concreting commenced and has been continued steadily since that date.

At the commencement of construction an estimated progress graph was plotted, and as shown, it has been found possible to conform extraordinarily closely to this chart. See Fig. 1.

COSTING.

The cost of carrying out a work is the only true criterion —providing that quality is maintained—as to the efficiency of the plant and method of construction employed. For

*This paper, No. 548, which originated in the Perth Division of The Institution, will be presented before the Engineering Conference to be held in Perth in April, 1936.

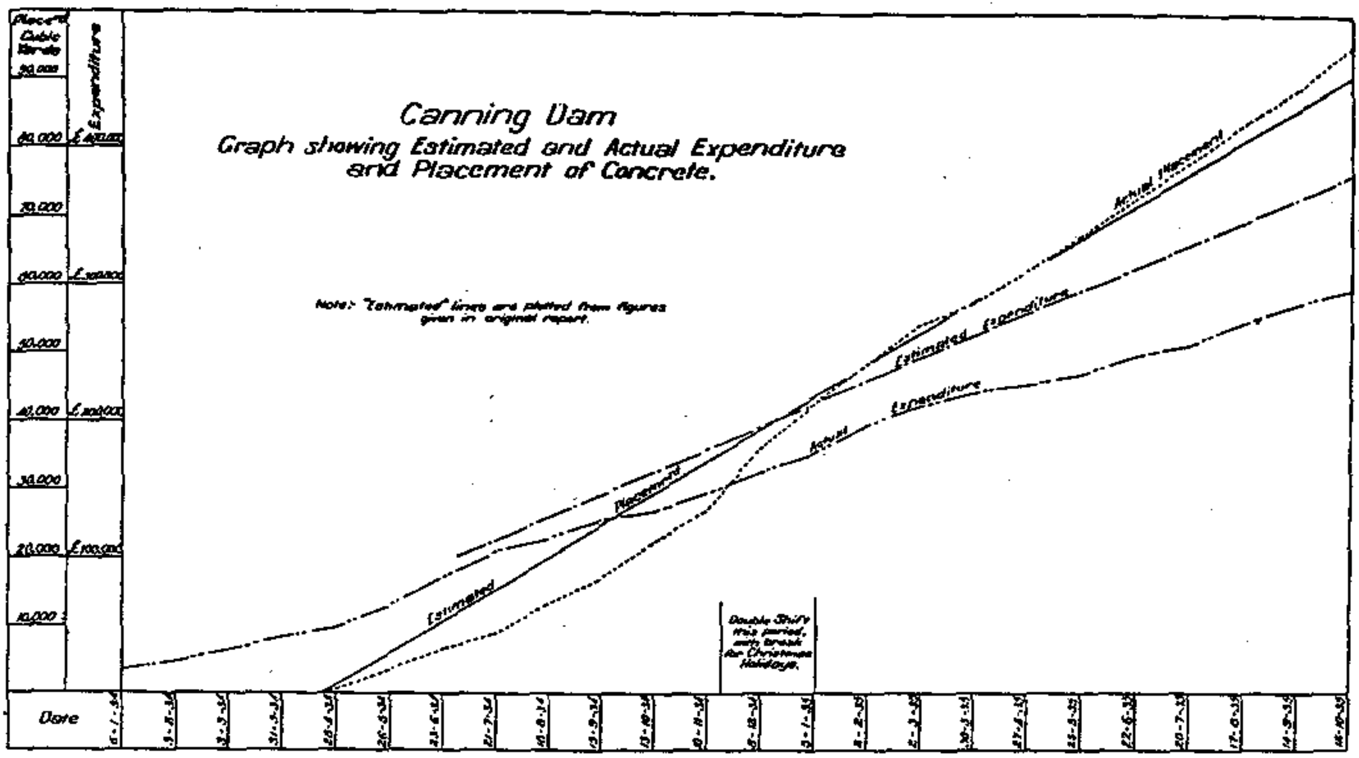


Fig. 1.—Progress Chart.

purposes of comparison it is essential that costs for various jobs should be on a standard or uniform basis. Many costs are not "honest" and it is considered that The Institution might well draft some standard costing system to be adopted on major construction works.

The costing system adopted for the Canning Dam is a conservative one. Purchases of plant and erection of same, including temporary construction buildings, have been debited to a Plant Suspense account. For each cu. yd. of concrete placed this account is credited with 2s. 6d., which is charged against the concrete item. In addition, all credits received for returned empty cement bags, etc., are credited to a "dismantling" account.

By this means the "plant suspense account" will have been almost completely written off at the end of the work, and the "dismantling account" will provide the fund for dismantling plant, etc. The individual item costs will, therefore, not be disturbed at the end of the work.

One of the authors, who was responsible for the introduction, to Western Australia, of the chuting system with the compound truss, has had intimate constructional experience with the 10-ton Lidgerwood type cableway. On account of its flexibility, capacity, low capital cost, and other features, the chuting system is proving more economical for construction of concrete dams than the 10-ton cableway. The cableway will develop, as in the United States of America, to the 20-ton or 25-ton capacity and then, the only way in which the Engineer can determine which system is preferable, must be by comparison of costs.

Details of Dam.

Before proceeding with a description of the construction of the dam, some details of the work will first be given.

The basin comprised approximately 1,200 acres of heavily-timbered jarrah, black-butt and red-gum country. This was cleared and burnt with sustenance labour.

The main dimensions of the dam are:—

Height above stream bed	218 feet.
Width at crest	20 "
Length of crest	1,600 "
Width at stream bed level	170 "
Estimated quantity of concrete	350,000 cu. yd.
Original estimated cost	£1,257,300
Revised estimated cost	£1,050,000
Expenditure to 12/10/1935	£298,213
Concrete placed to 12/10/1935	95,585 cu. yd.

Location.

The site of the dam is situated 10 miles distant from the railhead at Armadale and approximately 27 miles from Perth.

The main Perth-Albany Road, which has a bitumen surface, passes within 3½ miles of the site. A gravel road has been constructed from this point to the dam, and all supplies, including cement, sand, etc., are carried by motor transport over this road. The gravel road is graded approximately once a week with a Wehr grader, beyond which it requires very little maintenance.

Sand.

At the foothills of the Darling Ranges are extensive deposits of typical "Perth" yellow sand.

After a wide survey an excellent deposit was located 11 miles from the site of the dam, and within 100 yd. of the main road. The depth of sand, shown by borings, varies from 20 to 30 ft. with an average overburden of approximately 2 ft. of sandy loam.

Tests made by The University of Western Australia from samples of this sand taken from the borings gave the average results shown in Table I.

The quantity of clay and silt in the sand (Decantation test) averaged 7.2 per cent. The test for organic impurities gave a satisfactory result.

TABLE I.

Breaking stress in lb. per sq. inch.	In tension.		In compression.
	3 days.	7 days.	28 days.
3 : 1 mortar—Canning Dam sand	456	526	5,648
3 : 1 mortar—Australian Standard sand	358	445	4,140

The clearing and stripping are done by the Department. The top three or four feet of the sand contain roots of vegetation and this is screened; otherwise no screening is necessary. Tenders are called once every 12 months for the screening, loading, carting, and delivery of sand from this pit into the bins above the concrete mixers at the dam. This system has proved most satisfactory. In cases of breakdown on the works, or a short day's concreting, the contractor keeps his trucks running and dumps the sand on prepared beds at the dam. He re-loads and delivers from these dumps at his own cost on days of heavy draw.

The quantity of sand used per week averages 650 cu. yd. and varies from 80 to 160 cu. yd. per day. With the assistance of dumping and the use of one emergency truck the system is most elastic. Payment is on a cu. yd. basis, measured in the truck at the point of delivery at the dam. The rate for the existing contract is 2s. 11d. per cu. yd.

BROKEN STONE.

The stone is obtained from a quarry about one-quarter mile downstream from the southern wing of the dam.

A granite outcrop, approximately 150 ft. wide and bounded on either side by diorite dykes, has been opened up, and is now developed to a face depth of approximately 60 ft. The width permits of seven 24 in. gauge tracks into the face.

The quarry is operated by a contractor who delivers balls from steel tip trucks into bins above the primary crushers. The Department strips the overburden from the quarry and pays the contractor for each cu. yd. measured in the solid removed from the quarry.

After passing through the crushing plant, the "crusher run" stone, screened to 2½ in. size, is delivered by a series of conveyor belts into the stone bins above the mixers. Samples of the granite have been tested to a compressive strength exceeding 800 tons per square foot. Approximately 6 per cent. of the quarried stone has been rejected.

CEMENT.

The cement is purchased from the Swan Portland Cement Co. Ltd., Rivervale. The cement works are situated on the eastern bank of the River Swan. The principal materials are immediately adjacent to the works. Deposits of shell, of considerable depth, exist along the bed of the River Swan. These shells are loaded by grab dredges on to barges which are towed to the works. A clay deposit has also been opened up within a short distance of the works. The coal is brought by rail from the Collie mines, a distance of 125 miles, and is delivered at the company's siding.

All cement before consignment to the dam must pass the British Standard three-day and seven-day tests, a supply of Leyton Buzzard sand having been obtained for this purpose. The University of Western Australia carries out all tests on behalf of the Government.

BULK CEMENT.

The large savings to be made from the use of bulk cement had appealed to the authors during the initial stages of the work, but it was not possible to give detailed attention to this phase until plant designs had been completed, and the actual construction of the dam was proceeding smoothly.

Tests showed that the average amount of cement left in each jute bag, after emptying into a hopper by hand and shaking, was approximately 1¼ lb. Over the whole job this would have represented 1,000 tons. In addition, the weight of the bag is also paid for as cement. Inquiries from the manufacturers indicated that the cost of the bags, including filling, sewing, tagging, etc., represented approximately 9s. per ton. As the result of conferences with the Railway Department's experts, the system now in force was evolved, resulting in a reduction in railway freight to railhead of 2s. 6d. per ton, and a reduction in road cartage from railhead to dam of 11d. per ton.

The bag system entails the employment of a large number of men stacking the cement in the sheds, more men wheeling to the mixers, more men opening the bags and dumping into the mixer hopper and more men collecting, tying in bundles and loading the empty bags on to motor trucks for return to the cement works.

With the bulk cement and automatic batching, as adopted, the contractor conveys to the dam six tons of cement in bulk, and dumps it, without any assistance, into the 60 ton capacity steel tank.

BULK CEMENT COSTS.

The total cost of the bulk cement plant was £1,875, exclusive of cost of containers and railway trucks which were made by the Railway Department and not charged to the works. This gives a unit cost of 1.32d. per cu. yd. of concrete, or 5d. per ton of cement.

The following figures show the savings effected by using bulk instead of bagged cement :—

	Bulk per Ton.				Bag per Ton.		
	£	s.	d.		£	s.	d.
Capital cost ...			5	... ...	Neglect		
F.O.R. cement works	3	16	1½	... ...	4	4	10
Freight		6	2	... ...		8	8
Cartage		3	4¼	... ...		4	3
Testing fees		1	3	... ...		1	3
Repairs to containers			1	Sheeting			1
Labour of handling to mixer and repairs ...			3½	Labour of handling into shed and to mixer		2	4
Loss of cement ¼% max.			3	Loss of cement in bags 1%, say, ...		1	0
Total	£4	7	11¼	... ...	£5	2	5

Payment for the weight of the bag at cement rates, plus collecting in bundles, freight, etc., equalises any credit which may accrue from return of empty bags.

The net saving is, therefore, 14s. 5¾d. per ton or, say, 3s. 7½d. per cu. yd. of concrete, or a total of approximately £63,000.

HANDLING OF BULK CEMENT.

The first step necessary was for the cement company to erect four concrete silos, each of approximately 120 tons capacity, to allow time for the seven-day tests to be completed, and yet to keep the dam going. The capacity of the silos, however, does not quite enable this to be done, but whenever the silos cannot supply tested cement, the company fills the steel containers, which will be described later in the paper, by emptying in cement from bags, a stock of which is always held in reserve at the cement works.

Steel containers have been constructed in accordance with the dimensions shown in Fig. 2.

These containers are designed so that two can be placed, end to end, on a standard 3ft. 6in. gauge flat top railway waggon and be readily transferred from it to the chassis of a motor truck. The rectangular design was considered preferable to the cylindrical shape as it enabled the centre of gravity to be kept down while being transported on the motor chassis, and also because it can be fitted more readily to a motor chassis and emptied by raising one end.

The container empty weighs approximately 26 cwt., and up to 6 tons of cement are carried in each container; the gross weight when filled, therefore, being approximately $7\frac{1}{4}$ tons.

Two Leyland (Badger *TA* 6 model, rated capacity 5 tons) trucks have been employed and these have for 10 months conveyed an average of approximately 300 tons per week from railhead to dam site, a distance of 11 miles, with practically no breakdowns.

A centre opening 18 in. in diameter is provided in the cover plate of the container and the cement is filled in through this opening. One end of the container forms a hinged door faced around the edges with a strip of rubber insertion to make a perfect closure when clamped tight.

The cement is conveyed from the foot of the silos by means of a screw conveyor, raised to the necessary height by a bucket elevator and delivered through a canvas spout into the containers, which are on railway trucks. Experiments made before the containers were designed indicated that cement filled in this way did not weigh more than 80 lb. per cu. foot. Sufficient volume in the container was, therefore, provided on this basis. Tests were also made on the angle of repose of cement discharged from a spout, and these indicated that one opening in the top plate was sufficient for filling the containers.

The container is transferred, at railhead at Armadale, from the railway waggon to the motor chassis by means of an electric gantry which straddles the track. The gantry itself traverses in a direction parallel to the railway line, and is equipped with twin electric hoists which move in a transverse direction. In this way considerable mobility is obtained, and the transfer is effected in two or three minutes. Each hoist is fitted with a steel bridle and hook. which grapple lugs set at the corners of the containers The gantry is operated by the drivers of the motor trucks. On the underside of the container, at the discharge end, are two lugs through which two pins from the motor chassis pass. The other end is clamped to the chassis.

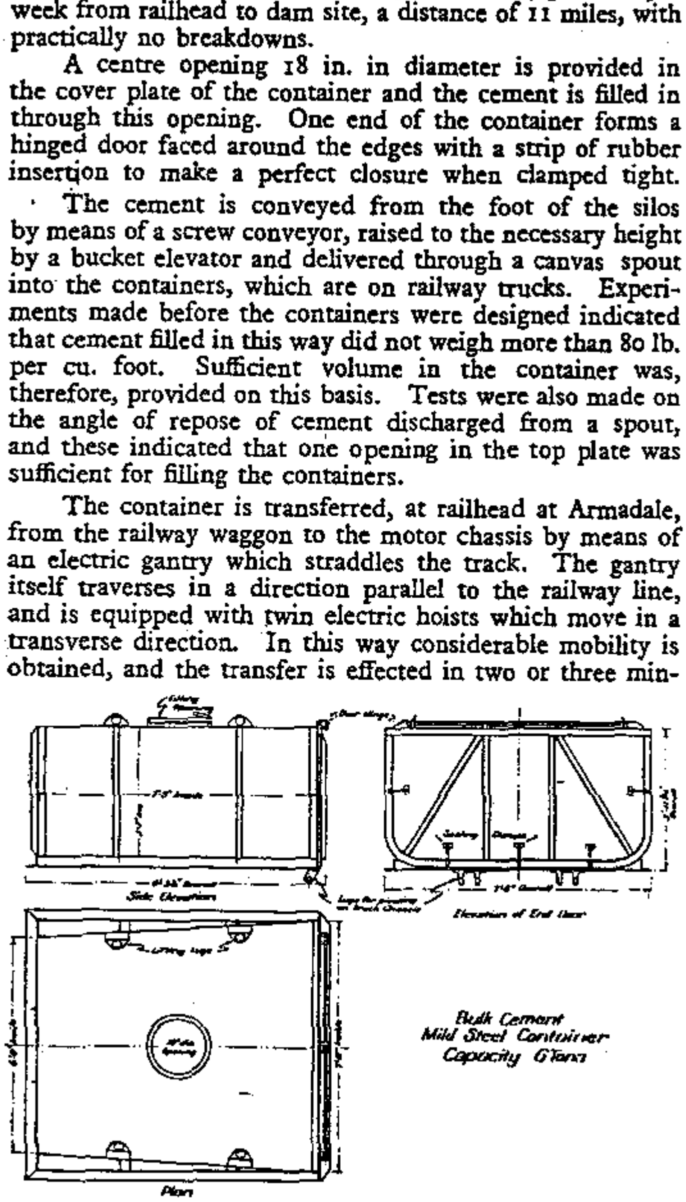


Fig. 2.

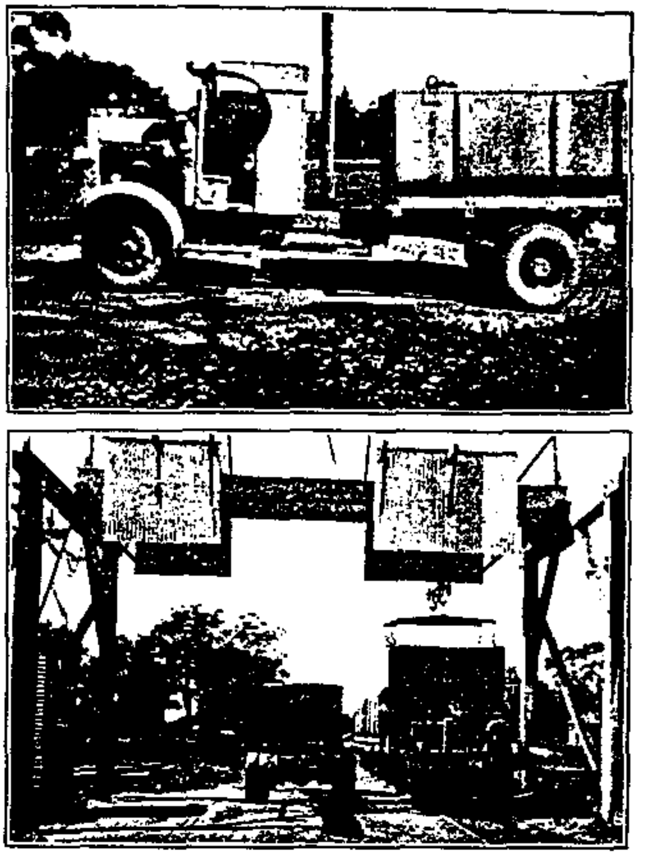


Fig. 3.—*Bulk Cement Container on Motor Truck.*
(*Below*)—*Transferring Containers from Railway Trucks.*

The main cement storage bin at the dam is located at the end of a wide timber ramp, with a gantry erected over the bin for the purpose of emptying the containers. The bin, which has a capacity of 60 tons, is 10 ft. square in section for the top 8 ft. and tapers down on three sides over the bottom 14 ft. to an opening 24 in. by 18 in. The bin and gantry are totally enclosed with corrugated iron to ensure weather tightness, a low entrance shed and doorway giving access to the trucks which are entirely sheltered while tipping is in progress. The motor truck is backed along the trestle into position at the front of the bin, and the front of the container is elevated by a friction winch and tackle suspended from the gantry until the container assumes an angle of about 55° with the horizontal. The rear end of the container pivots on two cast iron saddle lugs about a shaft attached to the rear end of the chassis. The clamps on the door at the back are previously unfastened, and as the front of the container is raised the door swings open and the cement is delivered into the bin.

Operation Problems.

In the original design of the plant, the cement was then fed through a door in the bottom of the bin into the boot of a vertical bucket elevator which delivered it through a 12 in. dia. pipe, on an incline of 52°, to a secondary bin of 2.5 tons capacity located above the mixer. The rate at which the elevator was to be fed was to be regulated by the adjustment of the door, which moved horizontally between guides and was fitted with a hand wheel and rack and pinion gearing to facilitate operations. It was here that the lack of actual experience in handling the material led to the first set of difficulties. In the first place, the cement was found to be so fluid in its action that it discharged through the smallest of pin holes in the welding at the corners of the bin and squeezed out between the door and the door guides. The welding was retouched thoroughly with a caulking tool and a baffle welded to the door to prevent the escape of cement in this way. The next difficulty was in the operation of the door itself. It was found that a door opening of an inch or two gave the required rate of feed for a time. but the cement then formed an arch above the door and this arch had to be broken before feeding of the elevator could continue. Breaking of the arch by increasing the door opening invariably resulted in the flooding of the boot to such an extent as to prevent the operation of the elevator. Each flooding meant a considerable waste of time and labour, as portion of the casing had to be removed and the cement scraped out and bagged. This operation took nearly three-quarters of an hour.

The problem was to devise some means of preventing the formation of the arch. The first step taken was the installation of a chain agitator, which consisted of an endless $\frac{3}{8}$ in. chain suspended in the cement from the top of the bin to within about 6 in. of the door, and driven through gearing by a 7 h.p. motor. It was found, however, that the chain could not be made to re-enter the surface of the cement and merely worked upwards leaving the arch unbroken.

It was then decided to fit compressed air jets to the sides of the bin above the door, the idea being to prevent arching by keeping the cement well agitated. In the first test, six $\frac{3}{8}$ in. dia. jets were placed on two sides of the bin about 12 in. above the door and connected to the air line. This showed promise for a time, but the arch continued to form, and seven more jets were placed in the sloping back to a height of 8 ft. above the door. The consequent aeration of the cement in the vicinity of the jets meant that the door opening had now to be reduced to about $\frac{1}{8}$ in. to prevent flooding. This opening gave the required rate of feeding for about one-half hour, when the opening became blocked by the accumulation of cement on its edges. The door had then to be opened wider and greater flooding occurred. It was then found that, although the cement in the immediate vicinity of the jets was more fluid than before, the cement above the jets was unaffected and the arch formed higher up, even though the air would bubble the cement to the top of the bin. A $\frac{1}{2}$ in. air pipe was, therefore, led into the cement from the surface to within a few feet of the door, but the arch remained unbroken.

After the shape of the door had been altered by the insertion of check plates in the sides and front of the bin, without success, it was thought that the formation of the arch might be prevented by continued hammering of the exterior of the bin. An air gun, of the type used in scabbling, fitted with a flattened tool, was mounted in such a position as to operate against the bin at the spot where the arch formed. This method met with more success than those already mentioned, but the effect on the bin was far from beneficial, and there was no certainty about its action. Flooding still continued on the breaking of the arch, and the method was abandoned after a time.

It was then decided to modify the design of the plant in an endeavour to rectify the trouble. The direct method of feeding under gravity from the main bin to the elevator boot was abandoned and the shape of the bin at the bottom was altered to allow of the installation of a short screw conveyor, which forced cement from the bin horizontally to the boot. It has now been in operation for some months and has proved quite satisfactory. The door is left open to its full extent, thus reducing the chance of an arch forming, and the conveyor feeds at a constant rate, irrespective of any flooding from the bin.

From the elevator boot the cement is hoisted vertically in buckets attached at 18 in. centres to a 10 in. wide canvas belt carried on head and tail pulleys and driven by a 7 h.p. motor. The elevator is housed in a mild steel casing to protect the cement from weather. The buckets discharge at the head of the elevator into a 12 in. dia. pipe, through which it gravitates to the small secondary bin. From this bin it is drawn off through a second door to the batching hopper, where the required batch is weighed and passed to the mixer. Should a blockage occur in the 12 in. dia. pipe, backflooding of the elevator is prevented by means of an overflow pipe which takes off well below the elevator head and by-passes the cement back to the main storage bin.

The door connecting the secondary bin and the batching hopper, when first installed, also gave considerable trouble. It was a flat plate door moving horizontally between guides, and designed to be operated automatically by compressed air. Considerable leakage occurred, however, between the door and the opening, and although efforts were made to prevent this by means of rubber insertion and other means, little success was achieved. The door now in use consists of a horizontal cylinder in which a piston moves back and forth to open or close a circular shaft passing through the vertical axis of the cylinder. A second piston, joined to the first by a distance piece moves forward with it, and when the door is open the two pistons, one on each side of the opening, block the escape of cement through the cylinder. Cement lodging in the clearance between the pistons and the cylinder forms a gasket and prevents further leakages.

Plant General.

The outstanding features of the plant employed are the tower and chuting system for the placing of the concrete, and the system of conveyors for transporting the broken stone from the crushers to the mixing plant.

The power is electrical, high tension lines at a potential of 20,000 volts being transformed at the works to 440 volts, 40 cycles. A unit rate of $1\frac{1}{8}$d. per kW., measured on the high tension side, is charged by the Electricity Supply Department.

Crushing Plant.

The quarry, as previously described, is approximately one-quarter mile downstream from the southern wing of the dam. The crushing plant is in duplicate, so that in case of repairs or overhaul the output may be maintained. The primary unit in each set is a Jacques No. 5 gyratory crusher, driven by a 48 h.p. motor, and of capacity approximately 300 tons per eight-hour shift. The broken metal discharges on to a 24 in. troughed conveyor belt of 70 ft. centres which elevates it and discharges it into a revolving cylindrical screen, 4 ft. dia. and 9 ft. in length, the holes

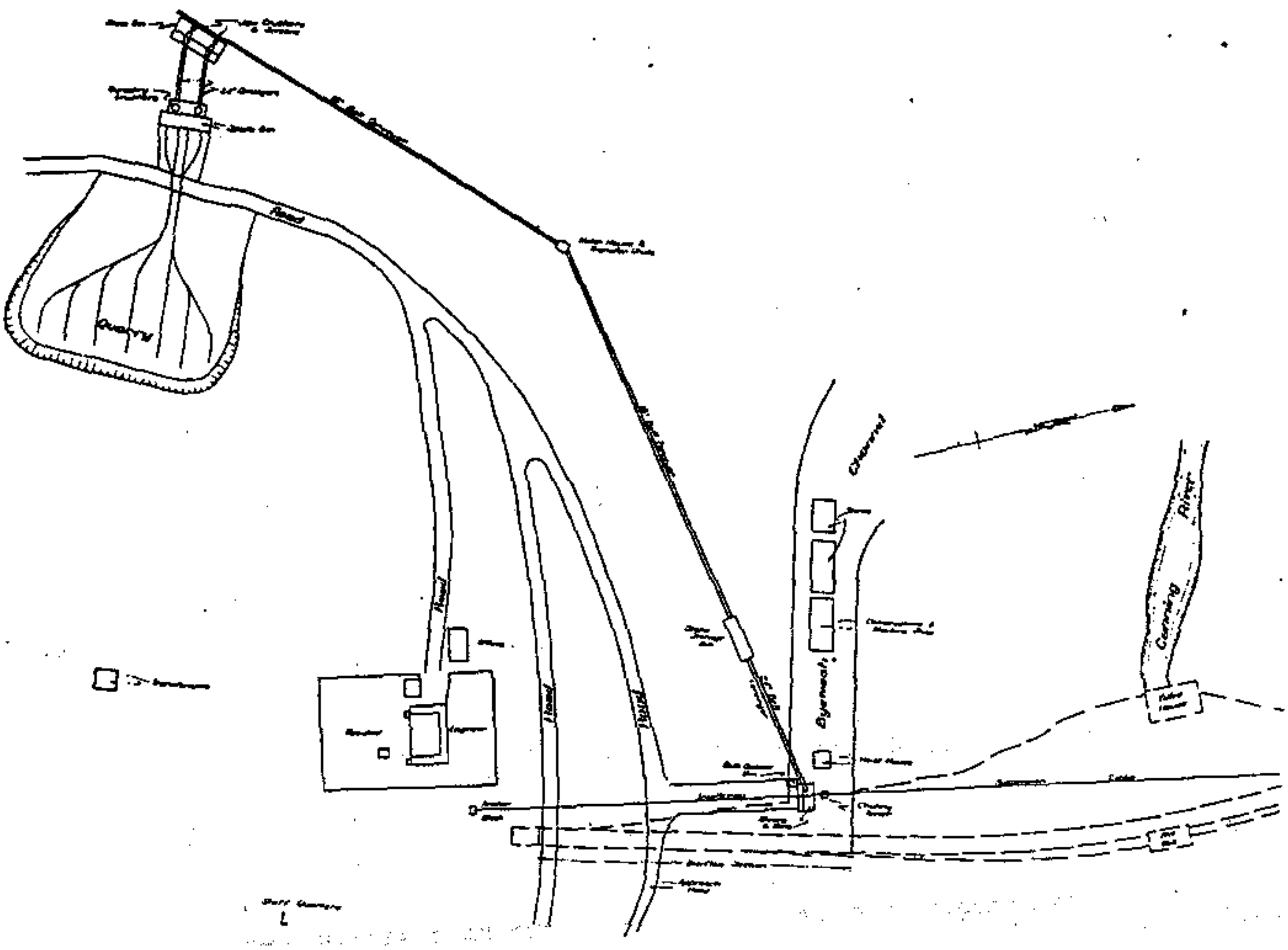


Fig. 4.—Plan of Canning Dam, Showing Layout of Plant.

in which are $2\frac{1}{2}$ in. diameter. The stone passing through the screen falls into the bins below. The oversize stone is fed to a 24 in. by 12 in. jaw crusher, and broken to smaller dimensions; this stone is not further screened but falls below to the bin with the screened stone. The cost of the crushing plant complete was £8,727, or 6.16d. per cu. yd.

The crushed stone is fed from the bins on to an 18 in. five-ply, flat belt conveyor of 525 ft. centres. This belt again feeds on to another similar belt of 410 ft. centres, set at an angle of 35° from the line of No. 1 belt, and by this means the stone is conveyed around the side of a hill and discharged into a rectangular timber bin of 300 cu. yd. capacity, which is regarded as the "service reservoir" of the system. The stone is conveyed as required from this bin to the stone bins above the mixer by a 24 in. trough belt conveyor, 275 ft. long, set at an angle of 12° with the horizontal.

The cost of the conveyor system from the crushed stone bins to the mixer bin, and including the intermediate storage bin, was £6,230, or 4.3d. per cu. yd. of concrete. One man is employed as greaser and door attendant and, including his wages plus the very little repair work necessary, the delivery of the crushed stone to the mixer bin costs only 5d. per cu. yd. The belts have already done one-third of their work and should be in good condition at the conclusion of the job.

The total cost of the coarse aggregate averages 5s. 4d. per cu. yd. of concrete, made up principally of contractor's price of 6s. 3d. per solid cu. yd. of rock excavated, stripping of overburden and also crushing and maintenance costs. To the above 5s. 4d. may be added the plant overhead of 10.46d., assuming the plant has no value at the conclusion of the job.

Concrete Chuting Plant.

The concrete chuting plant consists essentially of a $\frac{3}{4}$ yd. hoisting bucket in a steel tower 7 ft. square and 255 ft. high, and a receiving hopper which discharges the concrete into the chute line.

The tower legs are 8 in. by 8 in. by $\frac{3}{4}$ in. mild steel angles, divided into panels of 7 ft. 6 in., and the tower is guyed with $\frac{7}{8}$ in. dia. steel wire ropes at every sixth panel point.

The bucket is hinged at the bottom to a steel bridle equipped with rollers, which roll between two 4 in. by 4 in. timber guides attached to the sides of the tower. Two rollers fitted on each side of the tipping lip engage in a trip gear at the receiving hopper, and the concrete is dumped by continuing the hoisting. If the bridle is hoisted too far, the bucket will either jam or turn upside down.

The bucket is carried automatically under the mixer chute at the rear of the tower by bottom throw-out guides, which engage the bottom rollers of the bridle.

The hoisting winch is driven by a 150 h.p. motor, giving a rope speed of 800 ft. per min., and a bucket speed of 400 ft. per minute.

The sliding frame, carrying the receiving hopper, slides between 3 in. by 3 in. angles cleated to the outside of the tower legs, and can be bolted at any desired panel. It is raised and lowered with a crab winch.

There are two suspension cables of $2\frac{1}{4}$ in. dia., 280 ton breaking strength, locked coil rope, one carrying the continuous chute line, and the other the counterweight chute trusses. These cables pass over two specially shaped wooden cradles at the top of the tower, and are anchored to deadmen on either side of the valley. These deadmen are 30 in. dia. steel pipes filled with concrete, reinforced with old rails and buried 13 ft. deep.

The continuous chute line is hung from the cable by $\frac{5}{16}$ in. dia. wire rope tackles, the lower block engaging in a sister hook, which in turn grips the two hanger bands at the end of each chute section; the upper block hangs from a single wheel trolley riding on the cable, and each trolley is kept apart the required distance by a spacing line of 1 in. dia. rope.

The chutes are each 18 ft. long, nominally 16 in. dia., and are approximately of parabolic section. Angles, $\frac{3}{4}$ in. by $1\frac{3}{4}$ in. by $\frac{3}{16}$ in. are used at the sides, with a 16 B.G. bowl and a 12 B.G. renewable liner. The chutes are slip jointed in the continuous line, and the sister hooks are designed to give flexibility and yet ensure that the weight of each chute is equally transmitted to the pulley blocks. The chutes are raised and lowered by specially designed 15 cwt. crab winches, anchored on the downstream side. Two men can raise comfortably a chute during concreting.

Distribution of the concrete is effected by the use of three counterweight balanced trusses of 45 ft., 25 ft., and 18 ft. radius, respectively, giving an effective radius of nearly 90 ft. Each truss is suspended from the main pulley block, or the nose of the preceding truss, as the case may be, by a single suspension rod passing through the truss hopper, and these rods are supported by crossheads and ball bearings to give the needed flexibility. The trusses are suspended by an eight-part line, the upper block hanging from a two-wheeled trolley on the main cable.

A two ton air hoist stationed at the foot of the tower raises and lowers the trusses, and a similar hoist stationed well to the rear of the tower at the main deadman traverses the trusses along the cable.

The swivel plate chute connecting the chute line to the main truss is so arranged as to traverse with the trusses.

Fig. 5.—View from North, Showing Progress to October, 1935.
Chuting Tower, Mixing Plant and Stone Conveyors in Background.

Fig. 6.—View from South, Showing Progress to October, 1935.

The total cost of the chuting plant, including electric hoist, wire ropes, winches, etc., was £7,488, of which purchase of major items cost £5,731, and erection, including minor materials, £1,757. This expenditure represents a capital cost of 5.3d. per cu. yd. of concrete.

Method of Operation and Behaviour of Chuting.

As will be mentioned elsewhere in the paper, the daily blocks to be poured are approximately 45 ft. by 30 ft. by 6 ft. high. The design provides for vertical contraction joints at 90 ft. intervals along the wall, and it is the practice to pour concrete within one 90 ft. monolith for three weeks or more before moving the truss above the adjacent monolith.

It has been found that the workable grades of the chute line lie between $1\frac{1}{2}$ to 1 and $2\frac{1}{4}$ to 1. At steeper grades the stone will race ahead of the mortar, with consequent separation and danger to the men working below, while at flatter grades the concrete is so sluggish that the slightest check to the flow will cause clogging of the chute line.

Where possible, the grade is kept at about 2 to 1, but concrete is poured over $2\frac{1}{4}$ to 1 with a long chute line and for long periods with very little trouble. Steeper grades than 2 to 1 are rarely used and then only on short leads close to the tower.

With such a wide choice of grade it follows that the receiving hopper on the tower can be fixed in one position for pouring in the one monolith for three or more weeks. During this time it is necessary to raise or lower the trusses six feet or so or to traverse them 18 or 36 ft. on an average of every second day. The former operation takes six men about 20 minutes while the latter takes the same number of men about 30 minutes. These times include lining up the chute line.

When raising or lowering only, the trusses are not disconnected from the chute line, but when traversing, the swivel plate chute joining the trusses to the line is unbuttoned from the chute line and the trusses are hauled the required distance, the continuous line being then broken and the swivel plate chute again buttoned up. The chutes disconnected from the line are left hanging approximately in their previous position.

Moving from one monolith to another is a longer process, owing to the necessity for moving the sliding frame and the handling of a longer chute line. To move the trusses 200 ft. and the sliding frame eight sections took, in two instances, 16 men $2\frac{1}{4}$ hours and $1\frac{3}{4}$ hours, respectively.

The difference in time is due to the longer chute line lowered in the first instance.

It will be mentioned later that the slumps of the No. 1 and No. 3 concrete average 2 in. and 3 in., respectively. These mixtures give very little trouble on long leads of 450 ft. and grades of $2\frac{1}{2}$ to 1. Concrete of 1 in. slump has been poured, over 350 ft. and on grades of 2.2 to 1, for days at a time, but extra care is needed at the mixer. Should a wetter mixture be tipped into the chute line, then the higher velocity of the wetter concrete will cause building up in the chute with consequent overflows and, eventually, blockages. In fact, wet concrete causes more trouble in the chute than dry because, due to the lack of cohesion, the mortar will settle to the bottom and any irregularity in the sides of the chute will check the stone and cause a blockage. Also the concrete is more sensitive to any slight change of grade in the chute line, causing the stone to race ahead and then to build up on the flatter sections.

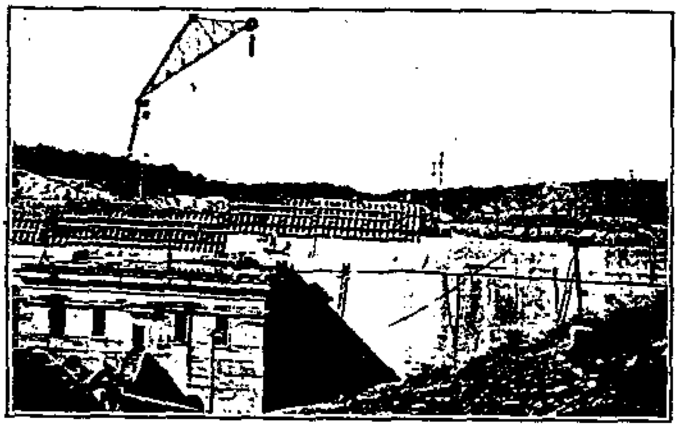


Fig. 7.—View Looking Upstream, October, 1935, Showing Valve House.

It might also be mentioned that the chuting plant is almost equivalent to another mixer. The concrete is well mixed when entering the tower skip from the mixer hopper and it is completely turned over when emptied into the receiving hopper on the tower face. The chute line gives very little mixing effect but the three vertical falls and changes in direction at the trusses must do. This extra mixing probably explains why such workable concrete is obtained with low water-cement ratios.

At the present time the output of the concrete mixer is limited by the bucket hoisting winch. At the lower levels of the tower 40 cu. yd. per hour can be maintained comfortably, while at the top of the tower 32 cu. yd. per hour is the limit. The highest yardage for one week's concreting was 1,802, and for one month, 6,312.

The cost of handling and of maintenance of the chuting plant up to September, 1935, when 90,000 cu. yd. of concrete had been placed, was as follows :—

Labour cost per cu. yd.	4d.
Materials	3d.
Total	7d.

Add to the above the capital cost of 5.3d., and the total cost of handling the concrete by means of the chuting plant is 1s. per cu. yd. This figure does not include the cost of winch driver, tower man, etc.—while concreting is in progress—these are a charge on mixing and placing of concrete.

The 12 B.G. renewable steel liners last from 6,000 to 8,000 cu. yd. of concrete. The cost of renewing these liners is 1d. per cu. yd. and is included in the above 7d.

Mixing and Batching.

In order to understand the operations connected with the proportioning of the various materials which are passed into the mixer when a " batch " of concrete is made, it is necessary briefly to describe the layout of the bins, batching plant, and mixers. The trestleway previously referred to, is a timber runway, 130 ft. long and 34 ft. wide, constructed at the level of the approach road and extending north as far as the hoisting tower. At the tower end the sand and stone bins are located, side by side and extending the width of the trestle. The primary 60 ton bulk cement bin is situated in front of these to the western side, the small secondary $2\frac{1}{2}$ ton container being housed in the sand bin. The mixers, of which there are two, are at ground level some 20 ft. below the bottom of the bins. Midway between is the batching floor where the apparatus controlling the doors and the weighing gear is housed. The bag cement storage shed is on the same level as the batching floor, so that in the event of a breakdown occurring in the bulk cement plant, bags may be wheeled to the mixer hopper on hand trollies. The chief mixing unit is of $1\frac{1}{2}$ cu. yd. capacity, the second mixer of $\frac{3}{4}$ cu. yd. capacity only being used in the event of a breakdown. This small mixer is not connected with the bulk cement plant, and has to be fed with the bagged product, of which one day's supply is always kept in stock.

The batching for the large concrete mixer is done by separate weighing of the stone, sand, and cement. In the floor of the stone bin is an opening tapering from 18 in. square to 15 in. square. This opening is controlled by a steel plate door carried on small flanged wheels running on light rails. The plate door is connected to the piston rod of a cylinder of 10 in. internal dia. with a piston stroke of 18 inches. This arrangement is shown in Fig. 8.

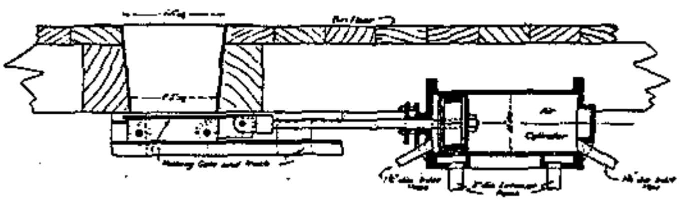


Fig. 8.—Detail of Stone Bin Gate and Cylinder.

The exhaust and inlet ports of this cylinder are connected by $1\frac{1}{2}$ in. and 2 in. galvanised tubing to a piston master valve in the operator's cabin. This master valve, which is moved in one direction by hand and reversed by the action of a solenoid, is so designed that compressed air can be admitted to either end of the cylinder and the door under the stone bin thus be opened or shut. The sand bin door is similarly controlled. The cement bin door is cylindrical and is operated by hand.

Steel batching boxes are provided, one each for stone, sand, and cement. Each box is suspended from its own system of levers, the weight being transferred to its recording beam which can be adjusted to lift when any desired weight is in the batch box. This beam is wired so that immediately it lifts a contact is made, closing the solenoid circuit which reverses immediately the relative master valve and the door under the bin closes.

The stone, sand, and cement batch boxes are suspended above a large hopper which discharges directly into the mixer. Thus, after the weights required have been measured into the batch boxes, the batcher man and his mate stand by for the signal that the mixer has been emptied when they immediately trip the doors, discharging the contents into the mixer. The outlet doors of the batch boxes which are counterweighted are then closed by hand, the master valves are reversed, thus opening the doors under the bins, and batching proceeds again. The complete process can be done easily within two minutes and the mixer never gets ahead of the batching. It will be realised that there is a "lag" factor in the closing of the doors. This has been found much smaller than might be anticipated, and extraordinarily consistent. This factor is allowed for in the setting of the beam weight. Constant and irregular checking of the batching against standard cast iron weights is made, in addition to which a representative of the Toledo Scale Co., which manufactured the weighing mechanism locally, visits the works once a month to check up the accuracy of its operation.

The Concrete.

The design provided that concrete between the upstream face and the 8 in. dia. drainage ducts was to be No. 1 concrete of a nominal mix of 1 : 2 : 4, and the balance to the downstream face was to be No. 3 concrete of a nominal mix of 1 : 3 : 6. The aim was to maintain a concrete of sufficient strength, viz., 2,000 lb. per sq. in. at 28 days, with due regard for workability, i.e., 2 in. slump for No. 1 concrete and 3 in. for No. 3.*

As previously explained, crusher run is used. During the early stages of the job the proportions were based on the Void Theory but this gave a concrete with an excess of mortar, so proportions based on the Fineness Modulus Theory were adopted.

The average fineness modulus for both crusher run and sand was found by taking a considerable number of samples. The calculation, using the Fineness Modulus Theory, gave a proportion of 1 : 2.06 : 6.73. Pending the results of compression tests taken under working conditions it was decided to test out concrete with proportions of 1 : 1.76 : 5.74. These proportions gave a concrete which was much too harsh so the quantity of sand was increased and the stone decreased giving 1 : 2 : 5.5. The proportions for No. 1 concrete were altered to 1 : 1.57 : 4.32.

As the results of the compression tests became known and showed that the aim of 2,000 lb. was being comfortably maintained, so the quantity of cement in the No. 3 concrete has been reduced, each time by 10 lb. per batch. At the present time the proportions for the No. 3 mixture have been reduced to 1 : 2.09 : 5.75.

It should be understood that crusher run is variable as regards voids and fineness modulus, due to conditions in the quarry, the state of the crushing plant, etc. Fineness modulus tests are taken each day and as long as the average is maintained no alterations are made in the proportions of sand and stone. The original average of 7.75 has remained constant.

The aggregates are all batched by weight and this necessarily gives variations in the volume of stone; each day isolated volumes of stone as batched are taken and recorded, and, periodically, consecutive volumes for from one hour up to eight hours. It is always found that the average does not vary one per cent., although rare single batches may vary as much as seven per cent. above or below the average.

An indirect check is also made in the following manner:

The number of batches is counted automatically and the concrete poured each day is measured up by cross-sectioning at 10 ft. intervals. Daily records are tabulated and, although there may be variations of three per cent. in any day's comparison, there is rarely a variation of more than one per cent. at the end of a week.

In regard to the water content, it has not been found practicable to fix the quantity of water per batch, due, very largely, to the above variations. A minimum quantity of water is set by means of a gauge connected electrically to a lamp and when necessary the mixer man adds a little more. The water-cement ratio used is tested periodically by averaging, over four hours, the extra time that the valve is opened and then, later, measuring the total water. The water-cement ratio lies between the ranges of 0.88 and 0.94.

Slump tests of the concrete in the box are taken frequently—often six times a day. Unfortunately individual slump tests are very unreliable, as the standard slump cones are too small for the size of the aggregate used. Quite often the same sample of concrete will give slumps of 1 in., 2 in., and 3 in. The tests, however, enable a standard to be maintained, and correct any tendency to become gradually wetter or dryer.

Two 8 in. dia. by 12 in. high test cylinders are made each day for testing at the University at seven and 28 days. The sample is taken from the chute and not from the mass in the box, and very little selection is made. This method eliminates any tendency to select the better concrete from the mass, and ensures that the average test block is a measure of the concrete in the wall.

Slump Tests.

Table II gives the result of daily slump tests for the past 10 months.

TABLE II.

Over 4in.	4in. to 3½in.	3½in. to 3in.	3in. to 2½in.	2½in. to 2in.	2in. to 1½in.	Under 1½in.
11	24	41	44	34	37	22

Concrete Compression Tests.

Tables III and IV give the result of compression tests since commencement of work.

TABLE III.

No. 1 Concrete.

Seven-day test—lb. per sq. in.			
Over 3,000	3,000 to 2,000	2,000 to 1,600	Under 1,600
6	62	16	1 (1,550)
Twenty-eight-day Test—lb. per sq. in.			
Over 4,000	4,000 to 3,000	3,000 to 2,000	Under 2,000
2	52	38	1 (1,865)

*A small quantity of No. 2 concrete, of nominal mix 1 : 2½ : 5, was used in a few positions. No. 4 concrete, of nominal mix 1 : 2 : 4, aggregate ⅜in. screenings, is specified for certain reinforced work.

TABLE IV.

No. 3 CONCRETE.

Seven-day Test—lb. per sq. in.			
Over 2,200	2,200 to 1,800	1,800 to 1,400	Under 1,400
10	42	57	4 (2 — 1,390) (1 — 1,350) (1 — 1,340)

Twenty-eight-day Test—lb. per sq. in.			
Over 3,000	3,000 to 2,000	2,000 to 1,900	Under 1,900
16	82	5	2 (1 — 1,840) (1 — 1,645)

FOUNDATIONS.

The wall is founded throughout on granite. The design provided for a minimum of 5 ft. of rock to be removed so that the concrete wall should be well keyed into the granite. Everywhere the foundations have been taken to a solid unfractured floor of granite; in one case up to 16 ft. of rock have been removed. The granite floor provides a comparatively smooth surface on which the resistance to sliding would be low. To roughen this, surface holes are drilled 6 in. deep at 6 in. intervals in lines parallel to the face and 24 in. apart. Each hole is loaded with one-quarter plug of ⅞ in. gelignite and fired. This has given a well-corrugated surface without fracturing the rock face. Excavation for foundation is carried out as follows.—

Jackhammers drill a series of holes to a depth of approximately 5 ft. and the rock is blasted out to this depth. Light shooting then proceeds until approaching the solid floor, the approximate level of which is known from the adjoining completed section of foundation. The whole area is then carefully retouched with hammer and gad and every drummy or loose piece of rock is knocked off. The cut-off trench is excavated similarly with light shooting followed up by the hammer and gad men. Careful inspection is made of every section of the foundation before it is approved for the placement of concrete.

After excavation is completed, holes are consistently drilled approximately 14 ft. below the foundation, and an attempt is made to force in grout under 80 lb. per sq. in. pressure. In no case has this been possible in the smallest degree. Approved rock from the foundations is broken down into "one man" stone, and placed, by hand, as spalls in the concrete.

GENERAL CONCRETING PRACTICE.

Before placing new concrete on old, the surface is scabbled, with pneumatic tools, in runs about 3 in. apart and from ½ to ¾ in. deep, thereby removing the old surface. There is practically no laitance present, as the concrete, in the first instance, is sufficiently dry to prevent its formation to any appreciable extent, and because the bags placed on the concrete absorb what little there is.

The scabblings and finer rubbish are removed with a compressed air and water jet, held about 9 in. from the surface, and any loose concrete that may happen to remain is then picked out.

The sides and ends of the boxes are not scabbled Standard corrugated iron forming, vertical at the ends and horizontal at the sides, is used. The corrugated concrete is blown with the jet and any loose fragments are picked off. The cleaning of the concrete is always done by a night shift to ensure a thoroughly wet and fresh surface for the new concrete.

One half-inch layer of 1 : 2 cement-mortar is poured on the floor and swept in with stiff wire brooms immediately prior to pouring the concrete.

The hourly rate of pouring averages 35 cu. yd. per hour. Five men equipped with shovels or spades have proved sufficient for the working of the No. 3 mixture and six men for the stiffer and richer concrete at the upstream face.

Spading of the concrete along the edges of the forms is not done at the expense of the inner concrete, yet the number of honeycombed patches exposed is exceedingly small.

The concrete is placed by the chute in 9 in. layers and the chute is not allowed to deposit the concrete nearer than two feet from any angle of forming. This prevents occasional larger falls of concrete from falling against the formwork and then being insufficiently worked.

Immediately concreting is finished, cement bags are placed on the surface and are then lightly watered. These bags are left on until the surface is to be prepared for new concrete. During the warmer months all concrete up to two weeks old is kept moist and also for one week prior to concreting. In spite of these precautions vertical hair cracks do develop, in the richer concrete upstream, about halfway along each block and also occasionally in the No. 3 concrete. Where there is any possibility of sealing these cracks, grout holes are drilled across them. Doubtless these cracks are caused by the initial increase in temperature, due to chemical action while setting.

Granite spalls taken from the foundation excavation are stacked along the edges of the box the day previous to pouring. Only selected spalls are used as they must be solid, free of quartz or decomposed material. They are washed where necessary.

After each layer of concrete has been packed, the spalls are spread on the concrete and two or three men trample and press them in, turning any concave surfaces uppermost. The next layer of concrete is then placed on top.

Twelve inches of concrete must be poured before spalls are placed; they must be 6 in. from any face and must be 3 in. apart. Up to 11 per cent. spalls, solid measurement, are placed.

DRAINAGE SYSTEM.

The design provides for a drainage grid of 8 in. dia. vertical ducts, spaced 5 ft. apart horizontally, connecting the lower and upper inspection galleries. These ducts were, at first, formed by setting up pre-cast porous, concrete pipes and concreting around these. This was found to be very costly, and a collapsible steel tube was evolved. The tube is constructed in three parts with vertical joints. The centre section has a central shaft connected by hinged ribs to the tube segments. The diameter at the foot of the composite tube, which is 8 ft. long, is slightly less than at the head, so that the tube can always be stood up in a duct. The tubes are set up and tied to the formwork to prevent movement and the concrete, to a depth of 6 ft., is poured round them. After the concrete has set, the centre shaft of a tube is raised by hand, the hinged ribs causing the attached segments to close in, similar to a closing umbrella. This section is withdrawn and the two side sections are then lifted easily from the duct. The tubes are constructed strongly, all joints being welded.

Contraction Joints.

The design provides that to a depth of 80 ft. from the top the wall shall be provided with vertical contraction joints, spaced 45 ft. apart, and below that level the joints are to be spaced 90 ft. apart.

higher in level and upstream and downstream sides overlap 3 ft. deep for a distance of from 5 to 12 feet.

The daily blocks are, therefore, thoroughly keyed into one another and any water, which may tend to follow the vertical or horizontal joints, would be compelled to change direction through 90 degrees a large number of times before appearing on the downstream face. The three-feet overlapping tongue limits the forming required along up and downstream sides to 3 ft. high. Keyways are provided in sides and top of each block.

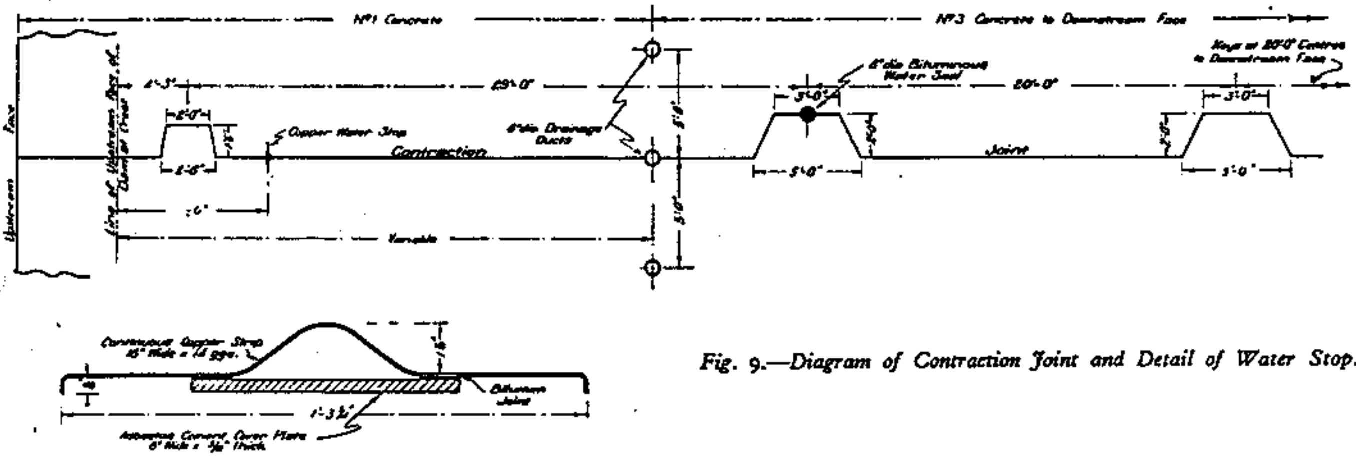


Fig. 9.—Diagram of Contraction Joint and Detail of Water Stop.

A typical contraction joint showing cut-offs, drainage and keying system is shown in Fig. 9, also the method of setting-up the copper cut-off strip to ensure its effective operation. This method was employed at Wyangala Dam, in New South Wales, and has proved most satisfactory and simple.

Before concrete is placed in an adjoining monolith, the junction face of the set concrete, at the contraction joint, is trowelled, where necessary, with cement-mortar to give a smooth surface. After this has set, and immediately before the concrete is poured, the face is painted with oil. The object is to prevent the concrete of one monolith adhering to the concrete of the adjoining monolith; this would interfere with the effective operation of the contraction joint. The bitumen stop is filled with Trinidad asphalt (40-50) soon after the concrete has set, and care is taken to prevent any foreign matter getting down the duct before the bitumen is poured. The same care is exercised to keep open the space between the copper strip and the cement asbestos sheet, to which it is attached by means of bitumen.

Block System.

A block system has been evolved which ensures that no horizontal or vertical joint between each pouring will be continuous through the dam. Each block contains an average day's pour, varying from 240 to 330 cu. yd., depending mainly upon the distance from the tower, the quantity of spalls available, and the class of concrete, and must be completed with continuous pouring.

Two blocks are formed between each contraction joint and are each of irregular, rectangular outline in plan. The centre joint is staggered, and the leg so formed is always alternated in consecutive lifts and also in adjacent blocks. See Fig. 10.

Each block is 6 ft. high and the two between contraction joints are on the same level. Adjoining blocks are 3 ft.

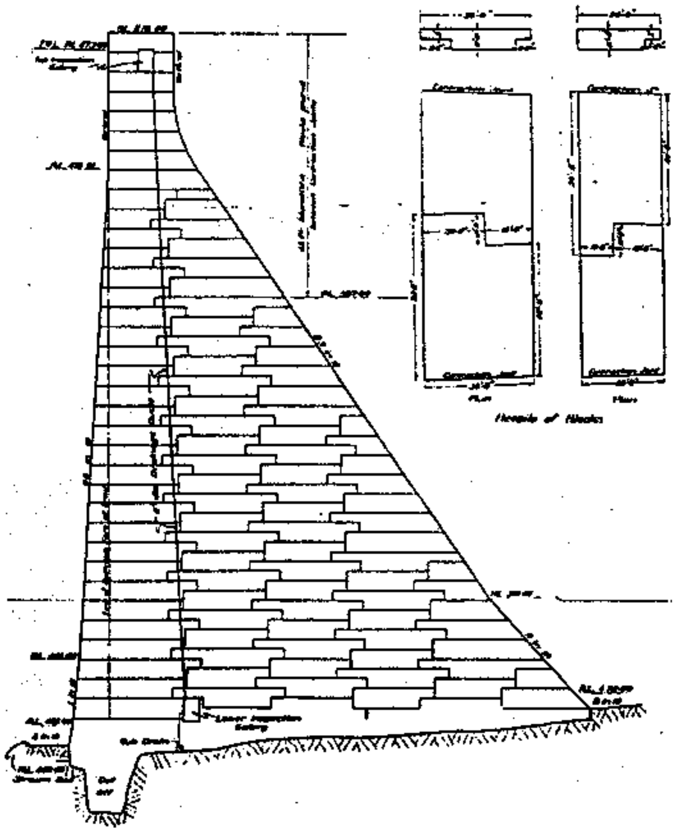


Fig. 10.—Profile of Dam, Showing Blocking System.

During a short period when concrete was poured by double shift, the blocks were enlarged for two-shift capacity. Boxes were enlarged in plan and depth increased to 9 feet.

Forming.

Standardised panel forming is used almost entirely. On the up and downstream face each panel is a complete unit 10 ft. wide and is lifted by light winches and frames. The vertical studs are 7 in. by 2½ in. Karri, at 1 ft. 9 in. centres with 6 in. by 1 in. t. & g. Jarrah sheeting, covered with 20 B.G. galvanised iron, to protect the sheeting from the concrete. Top and bottom walings only are used. These are 5 in. by 3 in. steel angles, back to back, spaced 1 in. apart and curved to the radius of the dam. The bottom waling is held by four ⅞ in. bolts set into the previous lift of concrete and these bolts are later screwed out and the hole plugged with cement mortar.

The top waling is held by four ½ in. dia. rods at 45°, set into the previous lift of concrete. One ⅞ in. rod tapped to take the ½ in. rod passes through the steel waling and connects the form to the tie rod. This ⅞ in. rod serves a dual purpose: it saves about 2 ft. of ½ in. rod and also, when the concrete is poured higher than the point where the tie rod passes through the sheeting, the ⅞ in. rod is screwed out and the hole plugged with mortar.

In order to eliminate toms or spreaders, and to facilitate lining up, the vertical studs are carried about 2 ft. 6 in. below the bottom waling to a second row of bolts, thus giving a cantilever effect.

When used against concrete, jarrah or karri sheeting will twist and shrink to such an extent that it can be used only two or three times on exposed faces. It is believed that, with the galvanised iron protection, the forms will complete the job. Besides the saving in timber a very much neater appearance is achieved.

Each lift of the forming is carefully lined up by surveyors to within ⅛ in.

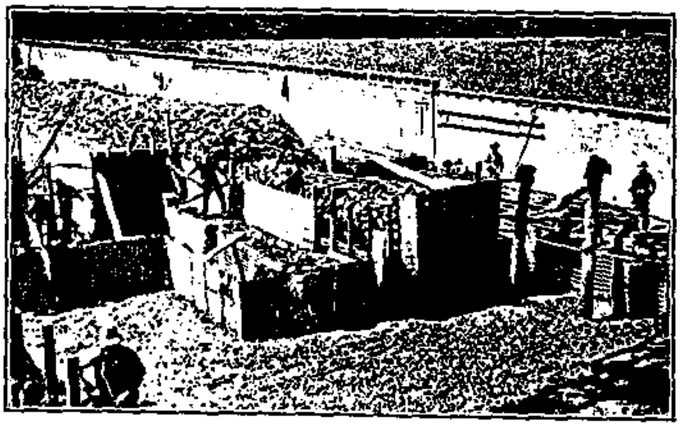


Fig. 11.—Box being Poured. Showing also Corrugated Iron Forms and Collapsible Steel Tubes for Drainage Ducts.

The block system, as previously described, needs only 3 ft. and 3 ft. 6 in. high forming on the east and west faces of the inside blocks. Until recently, jarrah formwork was used. The studs were braced-knee frames out of 4 in. by 2 in. at 2 ft. centres, clamped near the knee to the old lift of concrete by means of a 6 in. by 4 in. waling and a ⅞ in. bolt set previously in the concrete. The sheeting was made up in 10 ft. panels, 18 in. wide, out of 6 in. by 1 in. faced with 22 B.G. galvanised iron. The ends of the box were built generally with loose 6 in. by 2 in. studs with the above panel sheeting and held at the top by walings and 8 S.W.G. tie wire.

The use of corrugated iron formwork completely altered the system. Steel knee frames at 5 ft. centres instead of 2 ft. held down as above are now used. Steel channels 4 in. by 3 in. at top and bottom act as walings, and light 3 in. by 2 in. vertical studs of jarrah are clamped loosely to the walings. The waling fits into a lug welded to the knee frames. Standard corrugated iron (16 B.G.) is then nailed lightly to the studs.

The interior ends from 6 ft. to 7 ft. high are constructed in an exactly similar manner except that, as the corrugations are now vertical, the 3 in. by 2 in. jarrah studs are unnecessary and also that three walings are used instead of two.

Plain sheeting is necessary along the expansion joints. The forming does not lend itself to standardisation, so loose studs with 10 ft. jarrah panels are used.

Costs.

The costs of the various items are kept in considerable detail, and a high degree of accuracy is insisted upon.

The major item, mass concrete, includes all concrete except such special work as valve house and wet well.

The cost to date of mass concrete is 38s. 3d. per cu. yd., whereas the period costs for the past six months average 34s. 10¾d. per cu. yd., the difference being accounted for, mainly, by the reduced price of cement since May last and also by a gradual fall in the remaining items.

The only expenditures not charged against this item which would, by any stretch of imagination, be considered as a charge, are:—

1. *Roads and Maintenance*—£3,400. The greater part of this expenditure was in the construction of approach roads which will later be in the nature of a national concern.

2. *Water Supply* for works and men—£1,200.

3. *Accommodation and sanitation*—£11,900. This item includes erection of weatherboard quarters for men and families, office buildings, provision of recreation hall, school, etc. Rent is charged but this money is not credited to the job authority.

The major item is divided into 11 sub-items as follows. All costs are per cu. yd. of concrete.

1. *Forming.*—Average cost over past six months, 2s. 2d., labour cost 1s. 6d. per cu. yard. This includes cost of surveys, timber and all materials.

2. *Quarry and Crushing.*—Average cost over past six months, 5s. 3d., labour, including contractor's payments, costs 4s. 3.7d. A detailed dissection of this item has already been given.

3. *Mixing and Placing.*—Average cost, 1s. 7d., labour cost 1s. 1d. This includes mixer crew, winch drivers, tower men, and those working in the box. It also includes costs of mixer overhauls, proportion of power and compressed air costs.

4. *Chuting.*—Average cost, 7d., Labour cost, 4.2d. Includes all maintenance to tower and chuting plant and costs of handling chuting generally. Detailed costs are already given.

5. *Sand.*—Costs per cu. yd. of sand are already given. The cost per cu. yd. of concrete runs out at 1s.

6. *Cement.*—Average cost, 19s. 5d. Labour cost negligible. This item varies from period to period due largely to the varying proportions of No. 1 to No. 3 concrete. Any maintenance to bulk cement plant is included.

7. *Spalls.*—Average cost, 6d. Labour cost, 6d. Includes handling of spalls from excavation into box.

8 & 9. *Water Stops and Curing.*—Two minor items costing 0.75d. per cu. yard.

10. *Cleaning up.*—Average cost, 1s. 10d. Labour cost, 1s. 6d. Includes cost of scabbling surfaces of concrete and preparation of box ready for concrete and cleaning up generally. Carries its proportion of air and power costs.

11. *Erection of Plant.*—A Plant Erection Suspense Account was opened and 2s. 6d. per cu. yd. is credited to this item and debited against mass concrete each monthly period. The total in the account, including erection of the second tower, will be £46,000, and £41,000 will be credited by the conclusion of the works. This leaves £5,000 to be covered in the value of the plant.

A Plant Dismantling Account is also carried in Suspense and such credits as the return of empty cement bags are credited to this account.

The mass concrete item is also subdivided into No. 1 and No. 3 concrete and costs to date are 41s. 1d. and 35s. 10d., respectively.

Staff.

The principal officers connected with the construction of Canning Dam are Messrs. E. Tindale, B.C.E., M.I.E.Aust., Director of Public Works; R. J. Dumas, M.E., M.I.E.Aust., Engineer, Metropolitan Water Supply, Sewerage and Drainage Department, directing design and construction work; R. J. Cavanagh, A.M.I.E.Aust., Inspecting Engineer; and V. C. Munt, B.A., B.E., A.M.I.E.Aust., Resident Engineer.

APPENDIX I.

Plant and Erection.

Item.	Purchase of major plant.		Erection.			Grand Total. £	Remarks.
	Item. £	Total. £	Wages. £	Material £	Total. £		
Approach Trestle.	—	—	875	482	1,357	1,357	Cost per cu. yd. concrete, 0.96d.
Bulk Cement Bin.							
Bulk bin.	380						
Elevator.	350						
Secondary bin.	49						
Cement batchers.	305						Cost, cu. yd. concrete, 1.32d.
Worm gear screw.	154						,, ,, Cement, 5.03d.
Motors, 7 h.p., second-hand.	14	1,252	336	287	623	1,875	
Compressors and Shed.							
Compressor.	360						Second-hand. Includes 60 h.p. motor.
,,	258						
,,	20						Overhaul charges only.
Receiver	18						
,,	50						
Motor, 65 h.p.	136						
,, 65/70.	123	965	112	213	325	1,290	Cost, cu. yd. concrete, 0.91d.
Conveyors.							
1,860 ft. Belting, 18 in.	1,102						This item includes second-hand materia ex-Wellington Dam.
548 ft. ,, 24 in.	330						
Idlers, 18 in. and pulleys.	1,042						
,, 24 in. ,, ,,	259						
Motor, 20 h.p. second-hand.	20						
,, 28 h.p.	143						
,, 28 h.p.	143	3,039	1,483	1,147	2,630	5,669	Cost, cu. yd. concrete, 4d.
Crusher Plant.							
3—Jacques gyratory crushers.	1,834						
—Screens.	212						
2—Jaw crushers, 20 by 12, say.	400						On hire.
Belt conveyor, 24 in.	125						
Idlers, 24 in.	261						*Includes approx. £600 steel plating.
Motors, (2), 48 h.p.	234						
,, (2), 35 h.p.	200						
,, 35 h.p.	104	3,370	3,332	2,025*	5,357	8,727	Cost, cu. yd. concrete, 6.16d.
Intermediate Bin.			251	310	561	561	Cost, cu. yd. concrete, 0.4d.
Mixer and Bins.							
Mixer, 1½ cu. yd.	650						Mixers second-hand.
,, ¾ ,,	354						
Motor, 48 h.p.	196						
,, 37½ h.p.	150						
Stone meas.: batcher.	29						
,, weigh.: ,,	187						
Sand ,, ,,	187						
,, meas.: ,,	29	1,782	872	1,005	1,877	3,659	Cost, cu. yd. concrete, 2.5d.
Tower.							
Tower complete.	2,193						
Rope.	1,438						
Chuting.	1,174						
Steam winches, second-hand.	132						
Electric hoist, including 150 h.p. motor.	794	5,731	936	821	1,757	7,488	Cost, cu. yd. concrete, 5.3d.

APPENDIX II.

Mass Concrete.

Summary of Cost of Work to 12th October, 1935 *(including all material, freight, transport and power costs).*

Four Weeks Period 15/9/1935—12/10/1935.

Unit of Measurement—cubic yard.

Quantity for period: No. 1 concrete—2,203 cu. yd. @ 34s. 6d.
No. 3 „ —4,109 cu. yd. @ 30s. 9½d.

6,312 cu. yd.*

Description of Work.	Rate per cu. yd. s.	d.	Expenditure for period. £	s.	d.
1. Forming	1	7.60	515	9	8
2. Quarrying & crushing ...	4	10.74	1,544	18	11
3. Mixing & placing ...	1	5.00	447	3	10
4. Chuting		3.88	101	19	9
5. Sand		11.04	290	5	2
6. Cement	18	2.84	5,755	10	0
7. Spalls		7.42	195	5	1
8. Water stops		1.14	30	0	0
9. Curing		0.96	25	2	7
10. Clean up	1	4.48	433	7	3
11. Erection of plant ...	2	6.00	789	0	0
	32	1.10	10,128	2	3

*Record yardage for single shift; usual maintenance work.

Totals to Date.

Quantity to date:

No. 1 concrete placed to date 36,751 cu. yd. @ 41s. 4½d.
No. 3 „ „ „ „ 57,265 „ „ @ 35s. 10½d.
Other „ „ „ „ 1,569 „ „ @ 40s. 4d.

95,585 „ „

	s.	d.	£	s.	d.
1. Forming	2	6.55	12,156	11	7
2. Quarrying & crushing	5	4.29	25,604	8	9
3. Mixing & placing ...	1	9.83	8,692	0	8
4. Chuting		6.79	2,704	0	3
5. Sand	1	6.39	7,325	18	4
6. Cement	21	5.90	102,727	3	10
7. Spalls		7.07	2,814	19	9
8. Stops		1.63	650	5	8
9. Curing		0.43	171	1	9
10. Clean up	1	9.50	8,560	2	8
11. Erection of plant ...	2	2.07	10,384	3	0
	38	0.45	181,790	16	3

APPENDIX III.

Formula for Determining Costs of No. 1 and No. 3 Concrete.

Let X = Net quantity of No. 3 concrete } allowing for 10 per cent.
Y = Net quantity of No. 1 concrete } spalls.

Based on 658 lb. and 864 lb. per batch with allowance of 3 per cent. for bulking No. 1 Concrete.

Then $\dfrac{11X}{11X + 14Y}$ = proportion of total cement cost to be charged to No. 3.

$\dfrac{14Y}{11X + 14Y}$ = proportion of total cement cost to be charged to No. 1.

Cost of item for "Spalls" is divided proportionately between No. 3 and No. 1 with spalls.

Cost of remaining items is assumed the same per cu. yd. for each separate class of concrete.

Discussions & Communications.

THE EFFECT OF WELDING VARIABLES ON THE PHYSICAL PROPERTIES OF ELECTRIC ARC WELD METALS.

By R. R. Blackwood.*

Mr. D. V. Isaacs (Associate Member, Melbourne Division).—The importance of the author's work lay in his having demonstrated that the physical properties of weld metal were practically unaffected by welding variables, and that, therefore, the best weld was the one which could be executed most economically to give the best stress distribution.

Economy of execution of welds was usually achieved by using heavy runs and heavy currents, and the writer's own research work, which was as yet incomplete, indicated an improved stress distribution under these same conditions. On this basis the designer of welded work was thus free to choose his runs to secure what appeared to him the most desirable characteristics, with the confidence that the welds, if reasonably well done, would consist of a given quality of metal. Whether this statement applied however, to electrodes of makes other than those tested by the author was questionable.

There was also the aspect of parent metal properties, and this the author had not touched. He could hardly be expected to touch it however, considering the great extent of his work as described in the paper. The zone of parent metal affected by the heat of welding must be subject to quite marked variation in extent, with variation of welding variables. With weld metal which had high elastic limits, the parent metal properties near the weld were of importance, and for example, the fatigue resistance of a weld might therefore be affected by these parent metal properties, and their mode of variation near the weld. The writer had noted quite definite variations in parent metal yield points and ductility.

In regard to the Izod tests which the author described as being usually better in weld metal containing gas holes, the writer was inclined to accept Mr. W. D. Chapman's explanation as covering most of the phenomenon. Mr. Chapman considered the gas holes to imply metal in contact with reducing gases, and therefore of good quality. There was also the possibility however, that the gas holes had a geometrical effect, altering the distribution of stresses. Since the energy absorbed per cubic inch of metal near the fracture in an Izod specimen was not constant from point to point, gas holes might alter this energy per cubic inch and increase the test result. The gas holes might also tend, by their geometrical distribution, to change a bending fracture to a shear fracture.

Mr. W. M. Telford (Associate Member, Brisbane Division.)—In the tests to determine the natural variations set out in Tables IV and V, it must be agreed that all the variables under the control of the welding operator were kept as uniform as practicable during the series of tests, and that the finished test pieces were free from obvious defects; nevertheless there were considerable variations between the maximum and minimum values of ductility and Izod tests, while the tests further showed that tensile tests taken by themselves were an unreliable guide to the quality of weld metal. Commenting on the general results, the author stated "these variations are definitely attributable to some variable condition which is at present unknown." The need to determine this unknown variable was evident.

During the course of investigations that the writer carried out some years ago, he found a number of variable factors, other than those mentioned by the author, had a considerable influence on the quality of the weld metal. Such factors were the arc voltage, the rate at which the point of the electrode was moved over the work, and magnetic deflection of the arc, to mention the more important, while the time taken to complete the finished weld, after commencement of the work, had an important bearing, particularly on heavy sections. In the tests referred to, he butt-welded standard flex covered electrodes by an automatic welding machine, in which the arc voltage and travel along the seam were under accurate control, and he suggested that this method be used in making future tests.

In view of the permissible use of higher welding current than had generally been used in the past, it appeared desirable that the type of joint be modified considerably, with a view to reducing contraction stresses, and cheapening the cost of welding.

*For text of paper, see The Journal, Vol. 6, No. 11, November, 1934, p. 421, and Vol. 7, No. 2, February, 1935, p. 61, for discussion.

The dam's designer

SIR RUSSELL DUMAS, K.B.E., C.M.G., M.E., Hon. F.I.E. (Aust.) 1887-1975

Sir Russell was born in South Australia and educated at Prince Alfred College, Adelaide. He was awarded a Bachelor of Engineering from the University of Adelaide in 1919, and in 1931 was awarded the degree of Master of Engineering for a thesis submitted on the design of high masonry dams.

After experience in the construction of the Murray River Locks in South Australia, Sir Russell joined the Metropolitan Water Supply, Sewerage and Drainage Department, Perth, in 1925.

In 1928 he was employed by the Public Works Department. He was placed in charge of the construction of Canning Dam in 1933.

In 1934 he was appointed Chief Engineer, Metropolitan Water Supply and in 1941 was appointed Director of Works and Buildings and Chief Hydraulic Engineer for Western Australia.

In 1946, while retaining his appointment as Director of Works, Sir Russell was appointed the first Chairman of the State Electricity Commission.

Sir Russell was also responsible for the Goldfields and Agricultural Areas Comprehensive Water Scheme, constructed between 1946 and 1962.

He was Knighted in 1959.

"...a monument to enterprise..."

The Canning Dam was officially declared open by the Premier, the Hon. J. C. Wilcock, on September 6, 1940. An article in "The West Australian" newspaper at that time declared that it was *"the most ambitious water storage scheme in the history of the State . . . a monument to West Australian enterprise."*

It was almost exactly seven years since the same newspaper had reported a statement by the Director of Works (Edward Tindale) that: "preliminary work on Canning Dam was commenced on September 8, 1933, when the first party of eight men were sent to the site. Engineers and Surveyors also took up residence at the same time."

The start of construction was the culmination of planning, extending back to 1904, for a hills water scheme, which would give Perth its first fully reliable supply of good quality water.

The first dam to serve Perth was the Victoria Reservoir — a concrete dam constructed in 1891 and still in use.

The second hills dam was constructed on Bickley Brook in 1921. This had only a short career and was retired in 1936. It now provides a lake used for recreational purposes.

In 1924 a dam was commenced on Churchman Brook. At this time, pipehead dams were also built on the Canning River

The then Minister for Public Works, the Hon Alex McCallum, unveils a plaque to commemorate the commencement of construction on Canning Dam in September 1933. The plaque is now placed on a pillar at the south end of the dam.

Photograph courtesy of "The West Australian"

Some of the staff pictured at the dam in 1935. Left to right -

Photograph courtesy of Mr H W Lance

Top row: Hugh Lance, timekeeper; Eric Stoddart, engineer; Don Munro, engineer; Frank Bottrell, engineer; Cliff Sadlier, cler
Middle row; John Cohen, surveyor; Merv Whitfield, engineer; Victor Munt, resident engineer; "Mac" McAdam, storekeeper.
Bottom Row: Fred Pratt, clerical assistant; Bill Fawcett, surveyor; Bob Collins, foreman-in-charge, workshop;
Bill Slingsby, general foreman; Harold Riva, clerk-in-charge; Bill Davidson clerk-in-charge, costing.

and Wungong Brook to divert stream flow during the winter months directly into the trunk mains supplying the metropolitan area.

The pipehead dam on the Canning River was to be replaced by the major dam within a few years. However, the structure on Wungong Brook remained in service until the late 1970's when it was replaced by the Wungong Dam.

The Canning Dam was designed by Russell Dumas who had been brought from South Australia in 1924 to supervise the construction of Churchman Brook Dam. In 1934 Dumas was appointed Chief Engineer of the Metropolitan Water Supply Sewerage & Drainage Department.

The Resident Engineer in charge of construction of Canning Dam, under the direction of Dumas, was Victor Munt. His Principal Assistant Engineer was Don Munro.

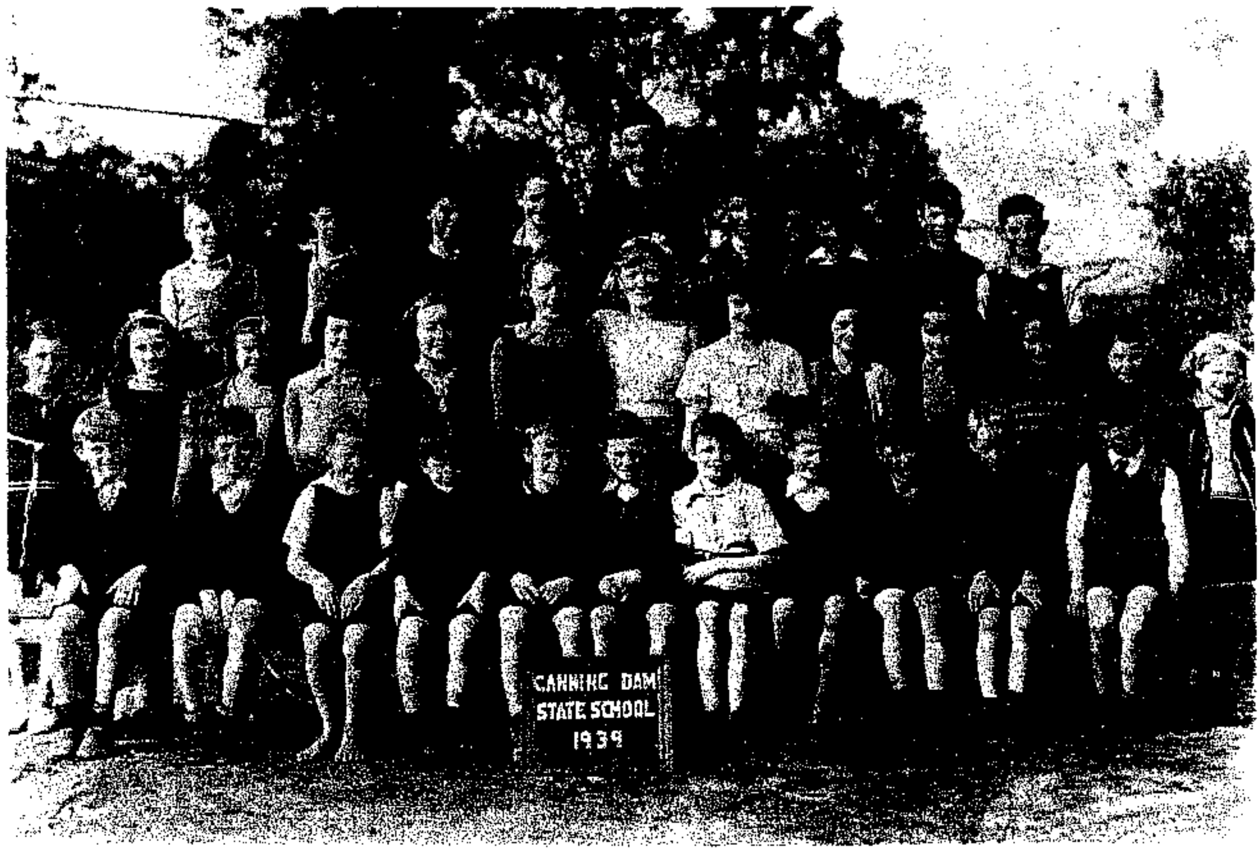
Some of the married quarters.
Photograph courtesy of Mrs A Milligan.

The construction of Canning Dam was commenced at the peak of the economic depression. Although the need for the major storage reservoir had been recognised for many years, the Government's decision to undertake such a vast project, estimated to cost in excess of 1.1 million pounds, and continuously employ almost 500 men as well as thousands more indirectly, was at that time as popular as it was reassuring.

The bleakest period occurred during the 1936-37 financial year, when work was suspended for six months because of a shortage of funds.

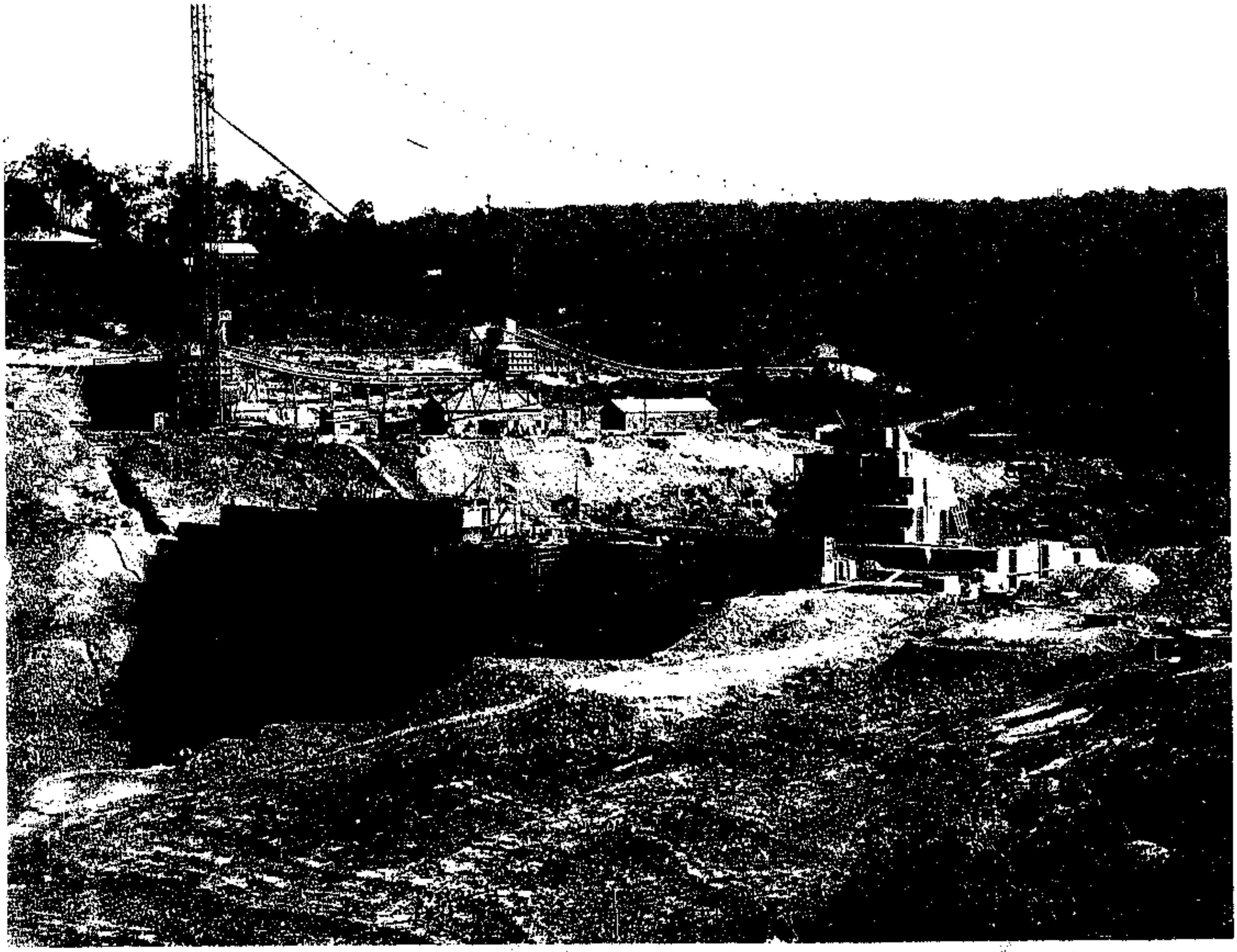
By the end of 1933 the preliminary site clearance, and foundation excavation work were well in hand. The once quiet valley in the Darling Ranges, 32 kilometres from Perth, became a scene of intense activity.

The pay queue.



The senior class of '39.

Photograph courtesy of Mrs P Rowe.



An early view of the dam construction, with the construction tower, concrete mixing plant, stone bin and conveyors in the background.

As part of the celebrations to mark the 50th anniversary of the start of construction of Canning Dam, the Metropolitan Water Authority has planned three interesting historic walks around the dam.

On the next two pages you will find a map of the dam and its precincts. The three walk trails are clearly shown. Each trail is well signposted and information boards, as you go around, describe the more significant features.

Sites and plans used in the dam's construction are illustrated also in this commemorative booklet.

You are invited to use the trails, and the barbecue facilities provided.

The Water Authority hopes that you will enjoy Canning Dam - and the story of its construction.

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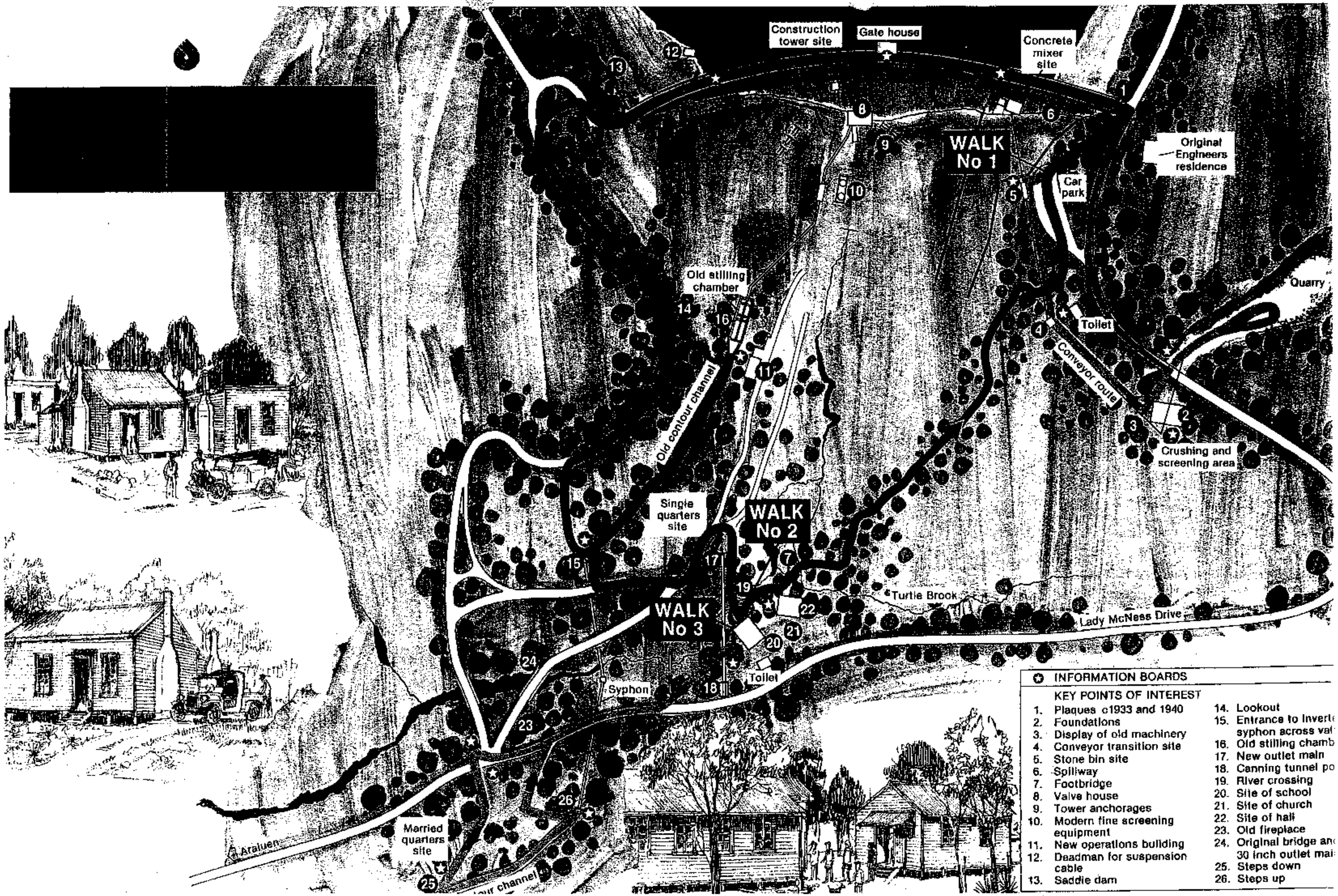
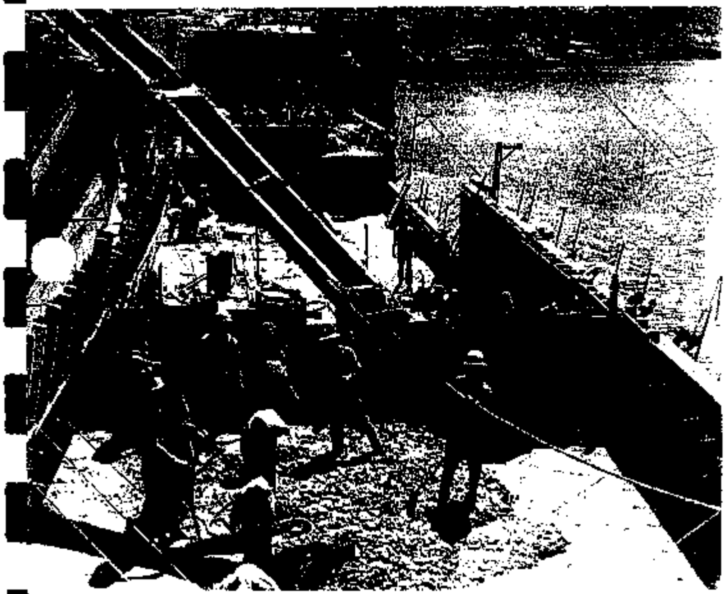
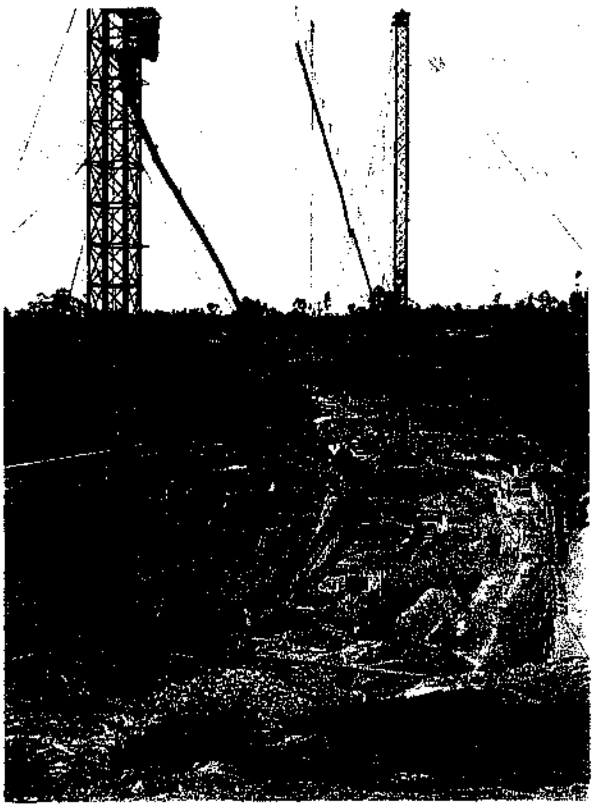

Construction tower site
Gate house
Concrete mixer site
Original Engineers residence
Car park
WALK No 1
Quarry
Toilet
Conveyor route
Crushing and screening area
Old stilling chamber
Old contour channel
Single quarters site
WALK No 2
WALK No 3
Turtle Brook
Lady McNess Drive
Toilet
Syphon
Married quarters site
Araluen
INFORMATION BOARDS
KEY POINTS OF INTEREST
1. Plaques c1933 and 1940
2. Foundations
3. Display of old machinery
4. Conveyor transition site
5. Stone bin site
6. Spillway
7. Footbridge
8. Valve house
9. Tower anchorages
10. Modern fine screening equipment
11. New operations building
12. Deadman for suspension cable
13. Saddle dam
14. Lookout
16. Old stilling chamb
17. New outlet main
18. Canning tunnel po
19. River crossing
20. Site of school
21. Site of church
22. Site of hall
23. Old fireplace
25. Steps down
26. Steps up



Concrete placing



A view from the Southern abutment.
Photographs courtesy of "The West Australian"

Concrete placing commenced in April 1934, and by the winter of 1934 a small township had taken shape at the western end of the valley. One of the principal objectives in undertaking the construction of Canning Dam was to provide useful work at a time when unemployment was running at record levels. Eighty per cent of the workmen on the job were unskilled. They were engaged under "sustenance conditions" whereby a person's period of employment was fixed according to a person's number of dependents.

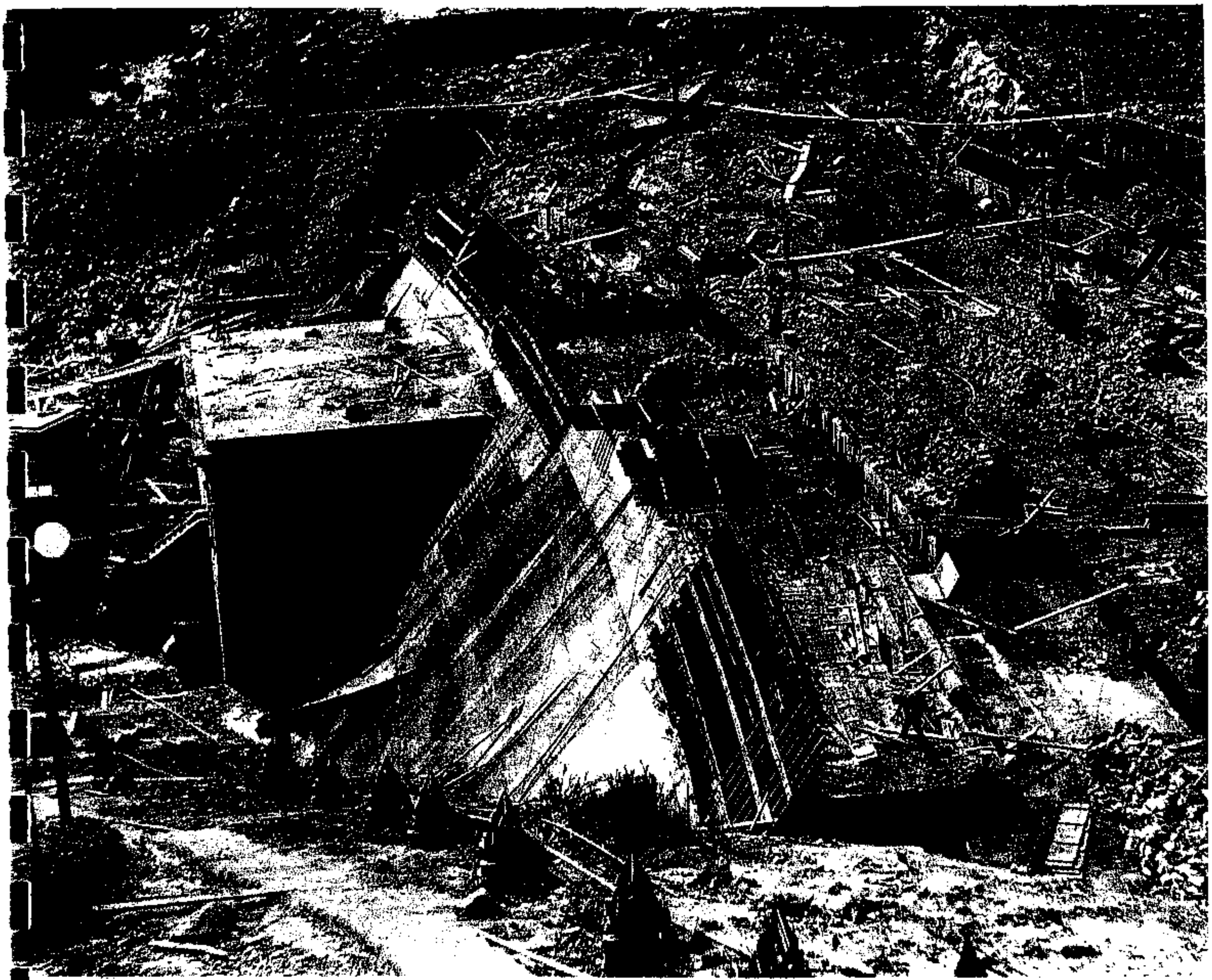
The work was very hard and at times quite dangerous. Blockages, which caused concrete to spill from the overhead chuting system, were a regular problem. Never the less considering the size and duration of the job there were surprisingly few serious accidents.

Many men came in daily from Armadale and Kelmscott but the majority lived on site. The initial accommodation was in tents, but gradually wooden huts were built. A boarding house and mess quarters were built, and soon a general store and butcher's shop were operating. By September 1934 sixty-seven married quarters (three or four room wooden cottages) had also been erected.

The growth of community spirit at the township was fostered by the construction of a recreation hall in September 1934. It was used for dances, card evenings, school concerts, and also as a picture theatre. There was a hall committee, which employed a caretaker to keep the hall in order and organize many of the activities.

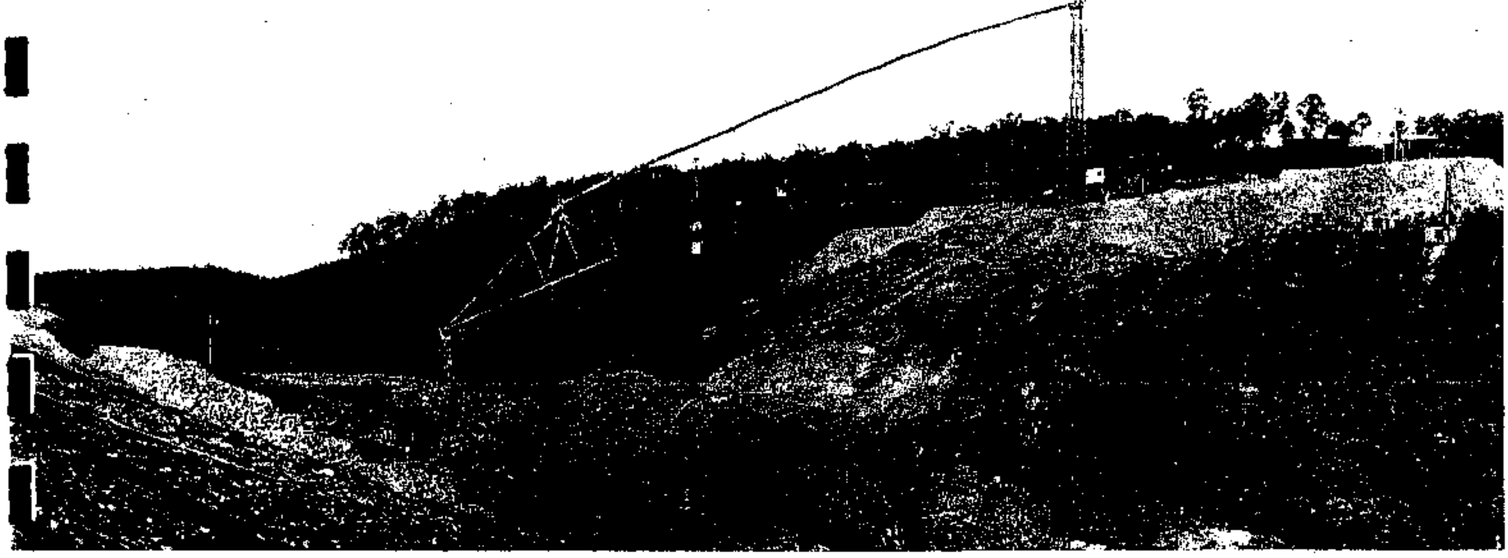
A school was built in March 1934 to serve the 29 children, aged between 6 and 14 years, who then lived on site. By 1935 the little school was overflowing and the hall was used as an annexe. The school reached its maximum enrolment of 107 pupils in 1938. The classes in the hall operated under difficulties, for the hall was unlined and cold in winter.

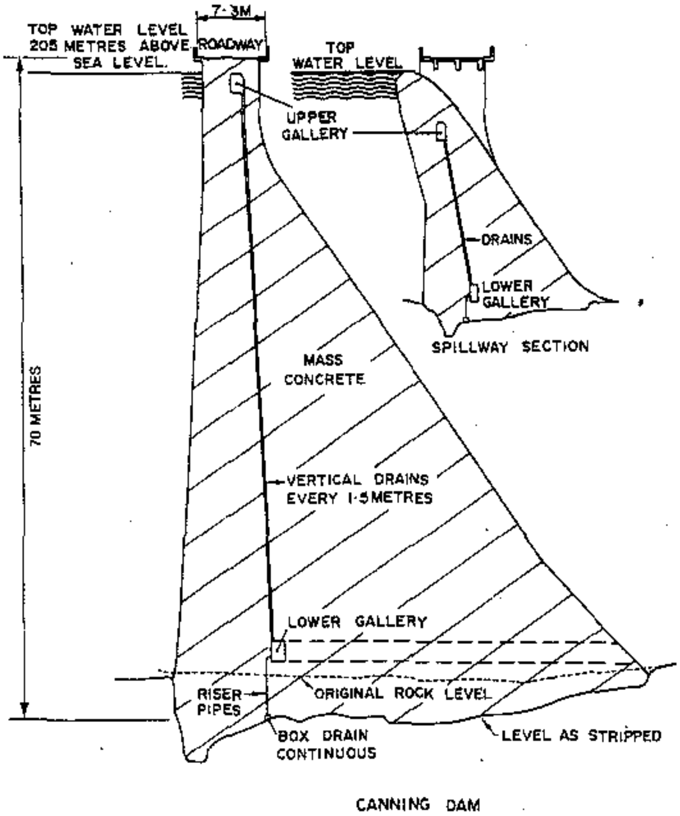
In 1938 a small Anglican church was built by volunteer labour. It was dedicated as the Church of St. Michael and All Angels.



Photograph courtesy of "The West Australian"

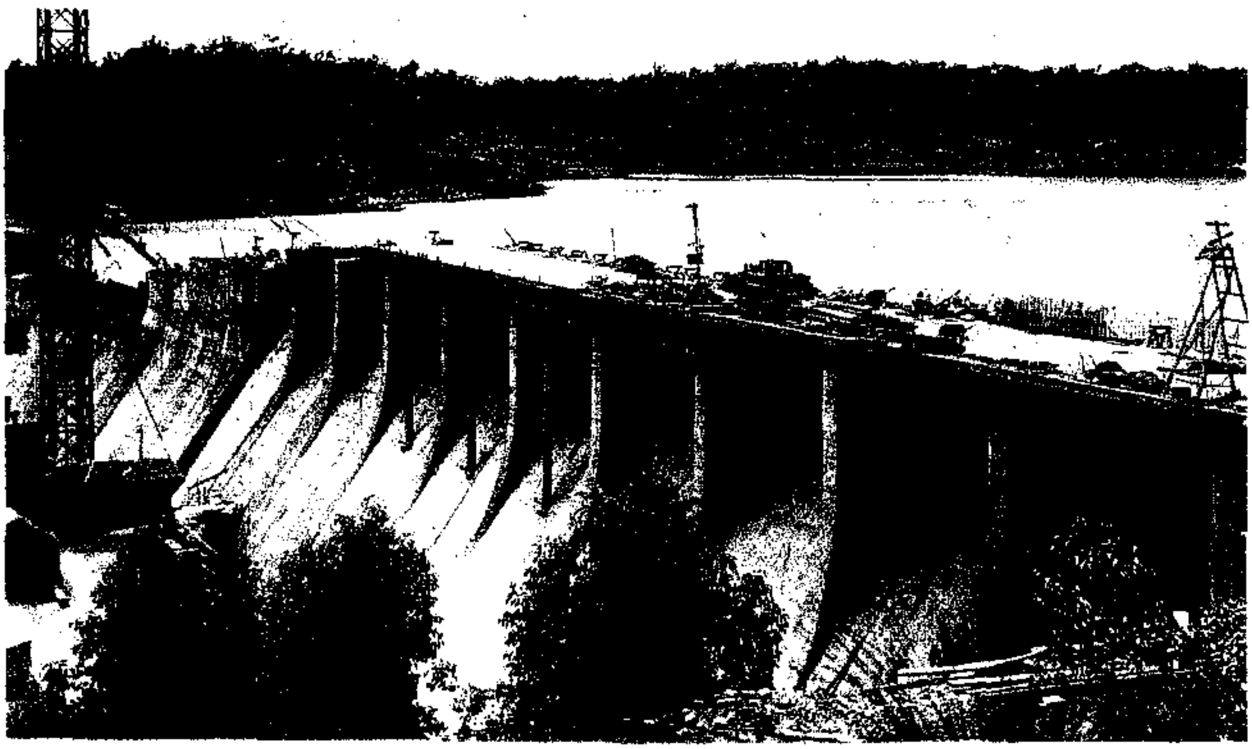
Early stages of construction.





A sketch showing a cross-section of the dam. The photographs show the early stages of the dam's construction.

The spillway nears the completion.



Photograph courtesy of "The West Australian"

The Canning Dam is a concrete wall with a total length of 466 metres and a maximum height of 70 metres. The dam stores 91 million cubic metres of water. At the time of its construction, Canning Dam was one of the largest in the world.

However, it was not only its size that caused it to be recognised as such a major engineering achievement.

Construction techniques were adopted which were quite new to Australia. These included the use of bulk cement; weight batching of all aggregate and cement; the use of internal vibration to compact the concrete; and improved methods for the treatment of construction joints.

The dam is located on an ideal site with sound massive granite rock outcropping on both sides of the valley.

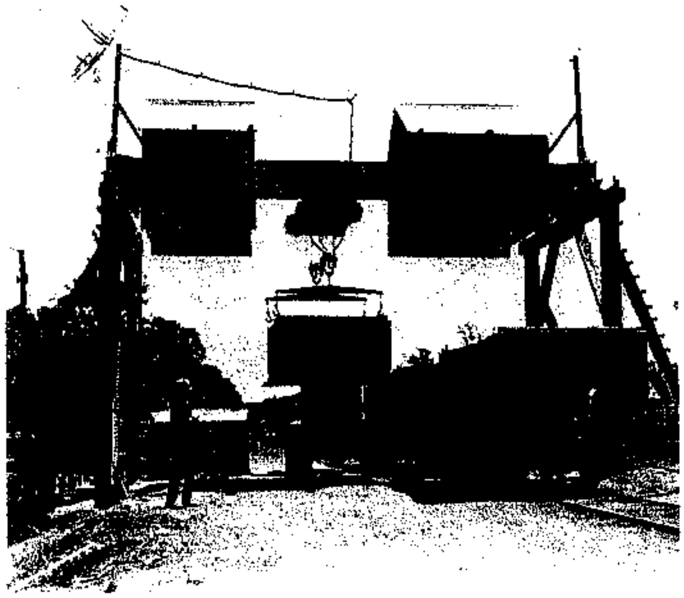
The dam is made up of a series of large vertical concrete monoliths forming an arch across the valley. Each monolith is tightly keyed into the next, while strips of copper were used to seal the vertical joints between the monoliths, and ensure that the dam remains watertight.

Two galleries traverse the full length of the wall close to the upstream face — one near the top of the wall, and the other just above the foundation. These galleries collect seepage water and provide access for inspection purposes.

Rock used for the concrete aggregate was obtained from a quarry about 400 metres downstream from the southern abutment of the dam. The quarry face was approximately 20 metres high.

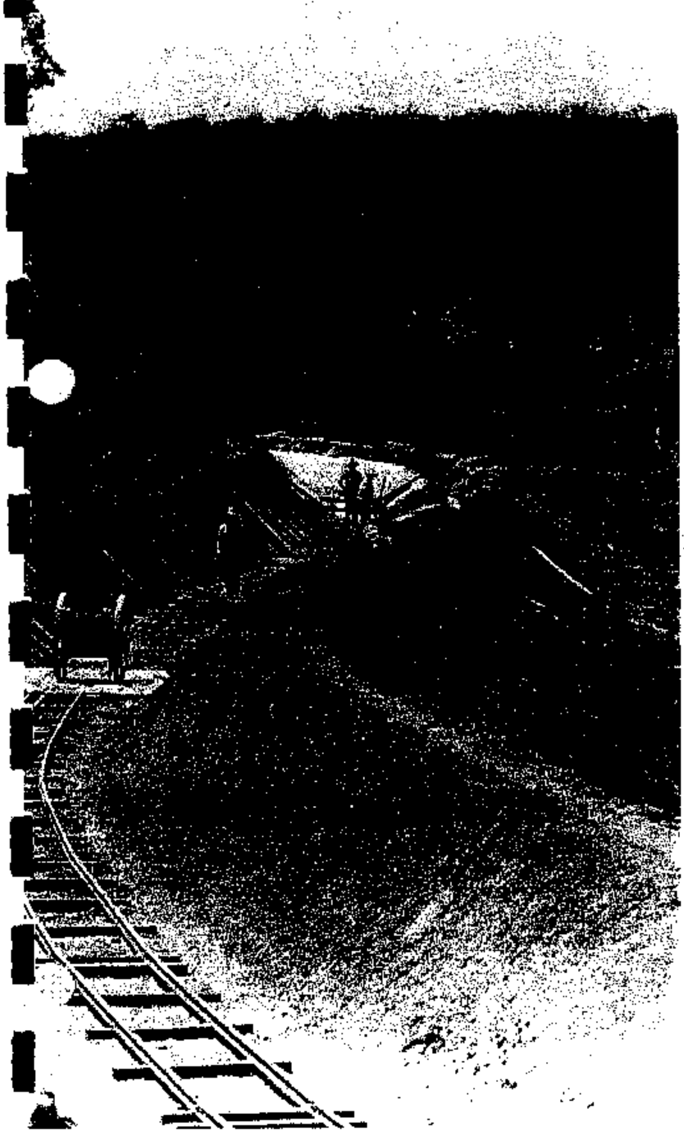
The rock spalls from the quarry were crushed and screened to a maximum 75mm size, before being delivered by conveyor belts to a concrete mixing plant which was located close to the southern abutment of the dam.

Cement for the concrete was obtained from the Rivervale works of the Swan Portland Cement Company. It was delivered in steel tanks each of 6 tons capacity to the railhead at Armadale and then by road truck to the dam site.



Transport of bulk cement.
Photograph courtesy of "The West Australian"

The quarry.
Photograph courtesy of "The West Australian"



Construction of the contour channel.

The sand was obtained from a pit located on the coastal plain between Armadale and Kelmscott.

The most outstanding feature of the plant employed on the site, was the two massive steel towers, each 76 metres high, to which were fixed the cables that supported the long chuting system.

Concrete from the mixing plant was hoisted in skips to the required level on the tower and tipped into the chutes that distributed it into the required position on the wall below.

To convey water from the dam a concrete lined channel was constructed during 1935-37 down the Canning Valley to a point in the foothills above Gosnells — a total distance of approximately 16 kilometres. From there it joined the trunk main system through which the water passed to the service reservoir at Mount Eliza.

Since the completion of the work at Canning Dam in 1942 major improvements have taken place. In 1951 the inflow to Canning Dam was increased by the construction of a concrete lined channel to divert stream flow from the adjoining Kangaroo Gully Catchment.

In 1975, the supply channel to Gosnells was retired. It was replaced by a 2.4 metre diameter tunnel, 5.6 kilometres long between the dam and Roleystone, from where a piped connection was made to the metropolitan trunk mains.

Canning Dam overflowing in 1974.