Upper Mangatawhiri Dam

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This paper deals primarily with the design and construction of the Upper Mangatawhiri Dam and its place in the planning and development of water supply for metropolitan Auckland. The paper also brings up to date the information on the Auckland water supply system published in a previous paper (1), particularly with respect to basic catchment data and planning.

1. INTRODUCTION

THE commissioning of the Upper Mangatawhiri headworks on 24 March 1965 marked the end of an era. For over 100 years the Auckland City Council had operated to an increasing degree as the bulk water supply authority for the greater part of the metropolitan area. On 1 April 1965 this responsibility was transferred to the recently constituted Auckland Regional Authority.

The Council retains control of its own reticulation, and by arrangement continues to operate bulk supply on behalf of the Authority until April 1967. This delays the disruption of the present Council water supply division and of the close integration of the division with city engineering activities in general, and provides time for the city and the authority to re-organise and organise, respectively, their engineering services to suit the new order.

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TABLE 1

Headworks Data, Programme, and Catchment Potentials

Headworks	Catchment (acre)	Safe mean yield (m.g.d.)	Reservoir storage (10^6gal)	Constructed	Type	Dam data: Crest length (ft)	Max. height (ft)	Volume (yd³)	Lake area (acre)	Lowest intake (el.ft)	Spillway level (el.ft)
Waitakere Reserves											
In service:											
Waitakere	2,040	3.5	407	1907-10: 26-27	Mass concrete	573	83	33,000	69	619	684
Nihotupu auxiliary			69	1921	R.C. slab	275	65	1,585	19	900	930
Upper Nihotupu	2,480	5.0	520	1915-23	Mass concrete	530	165	71,300	36	595	706
Upper Huia	1,940	4.0	536	1926- 9	Mass concrete	544	120	38,000	53	445	540
Lower Nihotupu	3,220	6.0	1,057	1945- 8	Rolled fill	1,250	81	470,000	177	15	66
	9,680	18.5	2,589								
Under development:											
°Lower Huia	3,550	7.5	1,270	1965- 8	Rock fill	1,000	150	800,000		35	124
Total Waitakere	13,230	26.0	3,859								
Hunua Reserves											
In service:											
Cosseys	5,390	10.0	3,180	1951- 5	Rolled fill	550	135	537,900	295	417	516
Wairoa, Stage 1	3,200	2.0	—	1959-60	Intake weir					440	
Upper Mangtawhiri	6,380	14.5	3.640	1961- 5	Rolled fill	1,620	130	1,543,000	337	440	530
	14,970	26.5	6,820								
Future development:											
°Mangatangi 1 and 2	9,580	22.0	6,000	1963-72	Rolled fill	1,300	240	2,800,000		450	640
°Wairoa, Stage 2	—	4.5	2,000	1975- 8	Rolled fill	580	160	350,000		440	570
°Lower Mangatawhiri	5,000	†9.5	3,000	1976- 9	Rolled fill	550	100	500,000		300	370
	14,580	36.0	11,000								
Total Hunua	29,550	62.5	17,820								
Total in service	24,650	45.0	9,409								
Total potential	42,780	88.5	21,679								

° All data for these projects subject to modification.

† Includes 2 m.g.d. first stage: gravity supply, direct into Mangatangi aqueduct.

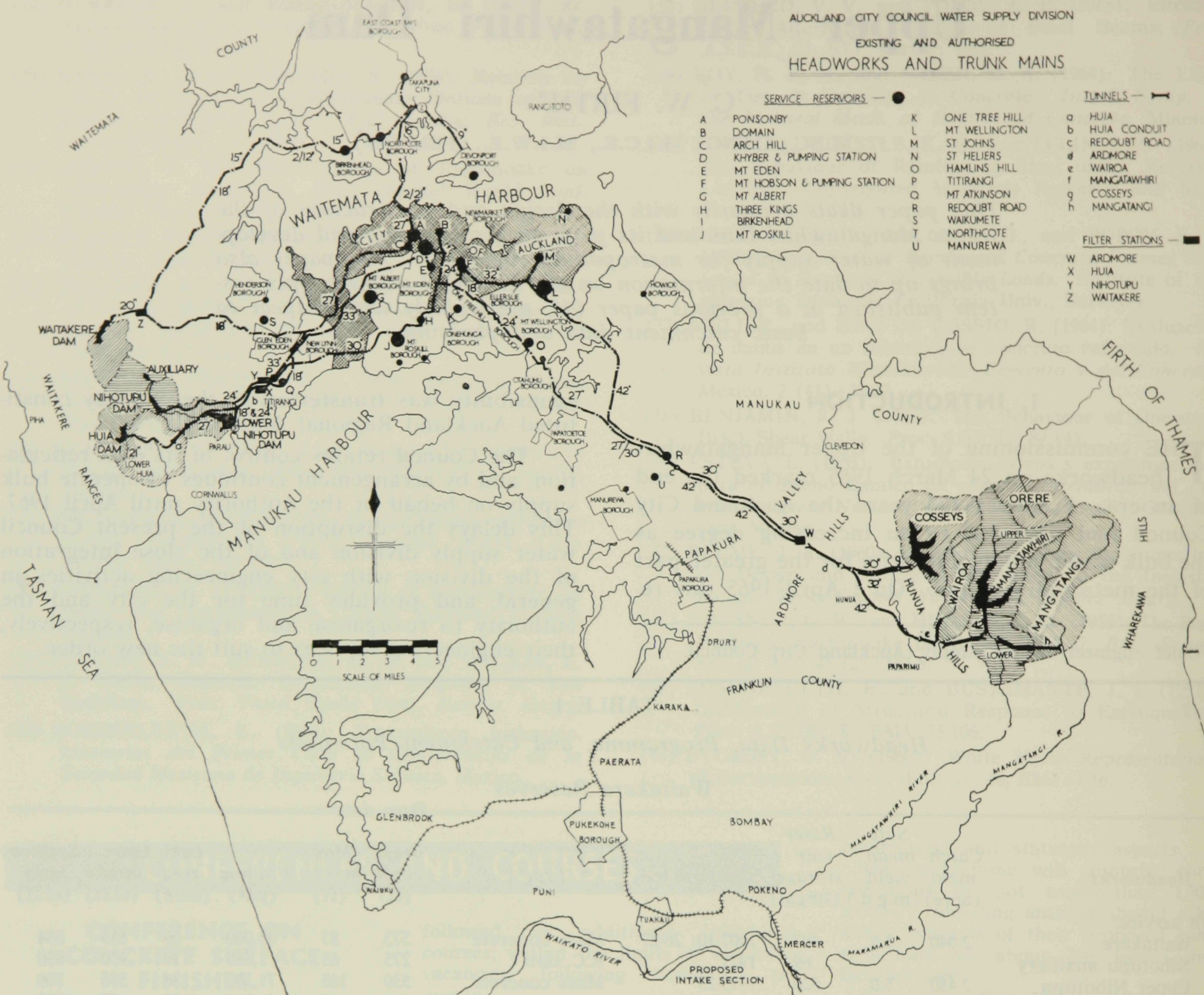

Fig. 1.

1.1. Headworks

Provision of water supply for Auckland, in accordance with policy enunciated in general terms early this century, is based on progressive development of five headworks in the Waitakere upland water supply reserves of 17,000 acres, centred 13 miles west of the city, and five headworks in the 40,000 acres of hill country in the Hunua water supply reserves, centred 26 miles south-east of the city. These upland sources are regarded as sufficient to supply one million people. To cater for further growth it has always been recognised that recourse will have to be made to pumped supply from the Waikato River, probably from the vicinity of Tuakau. A population of one million was originally estimated as being possible about the year 2000. It is now regarded as being probable by 1986.

Headworks have been and are being programmed in economic order, subject to strategic considerations. Thus rapid expansion of demand in South Auckland was a factor in proceeding to Hunua in 1946 instead of following the original policy of fully developing Waitakere resources first, and now, in 1965, the Lower Huia pumping headworks, the final project in the Waitakere reserves, is being given priority over otherwise more advantageous Hunua gravity headworks because of load growth in western suburbs and the need to obtain better balance of supply on either side of the city. Programme review in the light of demand trends and revision of other basic data is undertaken regularly, and at all times takes into consideration supply from the Waikato River. Ideas with regard to the latter are affected by the increasing reliability of power supply on the credit side, and on the debit side by the deterioration in the quality of the river water as the Waikato watershed is developed. Economic advantage still lies with full development of the Hunua reserves before supply is drawn from the river.

An up-to-date tabulation of data on upland catchments is given in Table 1. In this, Orere (Hunua) and four small projects previously listed at Waitakere are now omitted. They would all be expensive, and they have no real value now that average demand is in excess of 35 million gallons per day (m.g.d.) and is increasing at a rate of 2 m.g.d. per year. Figure 1 is a map of the whole supply area, and Fig. 2 shows supply and demand curves, to date and forecast.

To protect the future Waikato River intakes as far as possible, the City Council recently negotiated with the Pollution Advisory Council, under the Water Pollution Regulations 1963 and the Waters Pollution Act 1953, to gain a preliminary class B: "Water Supply" classification for that stretch of Waikato River from Mangatawhiri Stream, Mercer, downstream to Tuakau Bridge.

1.2. Treatment

All Auckland water is treated by alum flocculation and rapid sand filtration, followed by lime dosing to neutral pH and chlorination. For geographic reasons Waitakere waters are treated at three separate filter stations, but all Hunua water is treated at a single station at Ardmore. The Waitakere stations date from 1927 and are typical of that vintage. Ardmore is modern in all respects. It incorporates pre-settlement and automatic control, has already a capacity of 24 m.g.d., and will be continuously extended over the next 15 to 20 years to its ultimate capacity (peak-load draw-off from all Hunua headworks) of about 85 m.g.d. Expenditure on Ardmore since its inception in 1956 now totals £950,000, but this includes several major non-recurring features.

1.3. Transmission

The effective terminal for all transmission mains from filter stations, and the dominant control for distribution is Khyber service reservoir, with top water level at el. 320 ft, at the highest point in the city itself. Water pumped from here to el. 494 ft on Mt Eden supplies the limited higher levels within the isthmus area, and Ponsonby reservoir, at el. 235 ft, controls reticulation within the lower levels of the central city business district.

Transmission from Waitakere headworks is by three trunk mains of combined capacity 24 to 25 m.g.d., ranging from 9 to 15 miles in length, in sizes 18 to 33 in diameter. From Hunua there are at present two mains: No. 1 of 27 and 33 in pipe, 25 miles long, with 7 m.g.d. capacity; and No. 2 of 32 and 42 in pipe, 29 miles long, with 15 m.g.d. capacity. These mains link a series of tunnel aqueducts, totalling 6.2 miles, generally of 6 ft 4 in diameter, which are capable of transmitting the full Hunua yield.

To avoid a proliferation of pipelines from Hunua, advantage is being taken of the increasing reliability of electric power supply to boost-pump transmission mains to double their gravity capacity, and a station to do this is already installed at Ardmore. No. 2 main cost £1,500,000 when completed in 1960 and carries charges of the order of £100,000 per year. Boost-pumping will delay duplication of such a line for about five years and will cost only about £20,000 per year when fully operative.

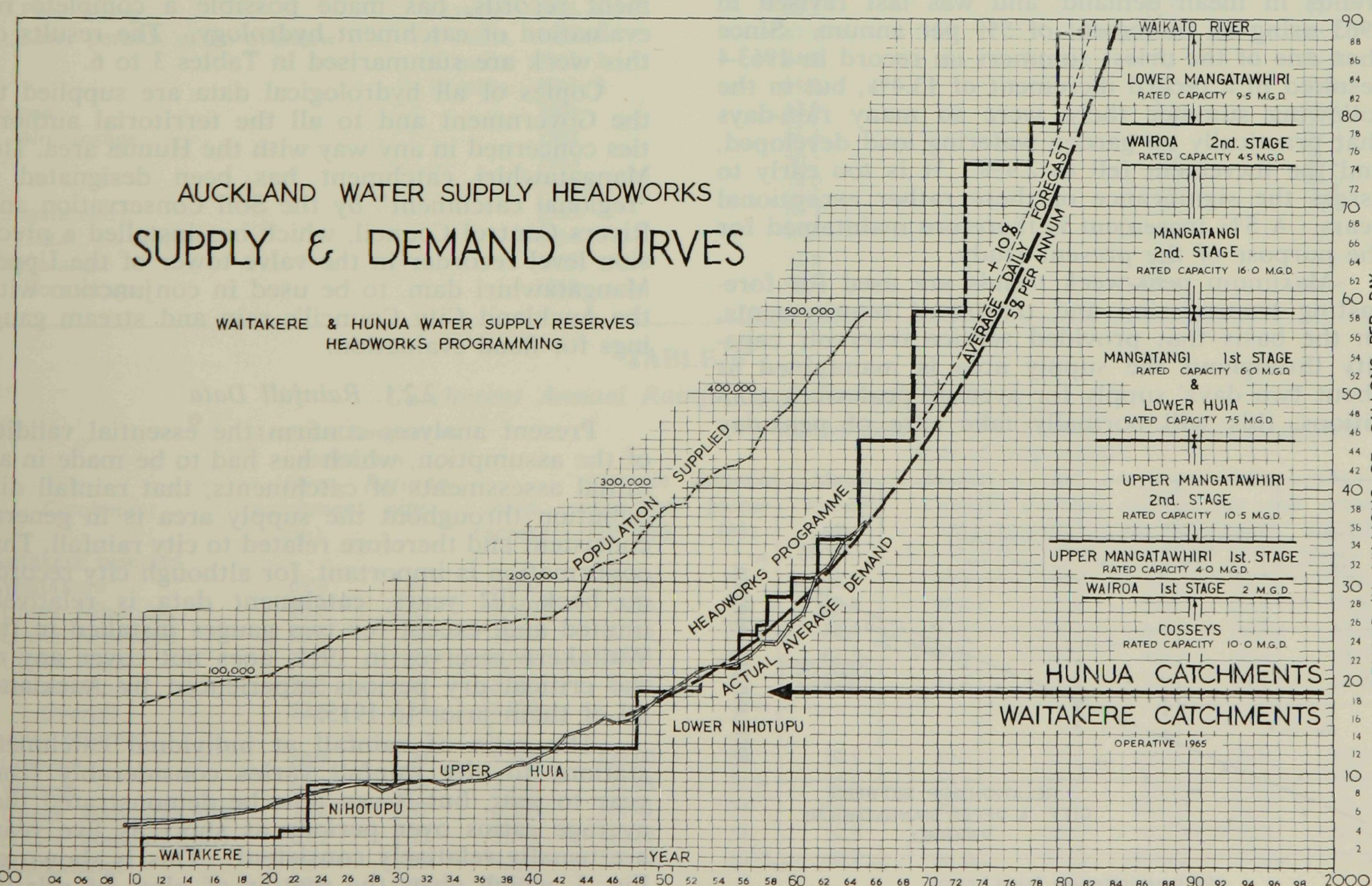

Fig. 2.

TABLE 2

Average Demands Since 1945

Years (ending 31 March)	*Mean demand*		*Annual increments*				*Margin over mean*			
				Peak day		*Peak week*	*Peak day*		*Peak week*	
	Average (%)	*Range* (%)	*Average* (%)	*Range* (%)	*Average* (%)	*Range* (%)	*Average* (%)	*Maximum* (%)	*Average* (%)	*Maximum* (%)
1945-55	3.74	+10.6 —3.4	3.87	+13.0 —4.5	4.22	+14.7 —6.6	32.4	37.5	20.5	24.4
1956-60	3.50	+ 6.5 —0.3	4.96	+10.8 —1.5	4.33	+11.3 —5.6	39.1	42.1	24.5	30.6
1961- 5	6.97	+13.4 +2.9	6.45	+14.7 —0.2	5.34	+16.6 —7.3	37.9	42.8	26.4	29.5

2. DATA FOR PROGRAMMING

2.1. Growth in Demand

The rate of growth of the population supplied has been remarkably steady at 4% per annum over the last 20 years, from 216,000 in 1945 to 476,000 in 1965. It is now 19,000 per year. Growth in demand over the same 20 years has been from an average of 15 to 35 m.g.d., and the rate of this increase has been increasing, for both mean and peak-load conditions. This is in part because of the continuous expansion of secondary industries, dominantly non-seasonal and therefore tending to introduce a steadying factor, but a substantial part of total demand is still of a type affected by weather conditions and therefore, following Auckland's well spread but variable rainfall, increments in demand from year to year vary considerably. The averages for various intervals over the last 20 years shown in Table 2 demonstrate the general tendencies of the demand.

Overall headworks programming is based on trends in mean demand, and was last revised in 1963 using an increment of 5% per annum. Since then one of the driest summers on record in 1963-4 resulted in a record increment of 13.4%, but in the following summer there were so many rain-days that practically no garden watering load developed, and the increment fell to 2.9%. It is too early to assess the significance of these rather exceptional years. A 5% increment is therefore maintained for the purpose of the present study.

Maximum peak-week trends are used for forecasting transmission and treatment requirements, on the basis that provided service reservoir capacity throughout the supply area is maintained at about two days' supply for average peak-week conditions, this will normally take care of peak-day loads in a system such as this with a variety of supply lines. To satisfy this requirement, service reservoir capacity, now 85 million gallons, has to be increased at a present average rate of 2.2 million gallons per year.

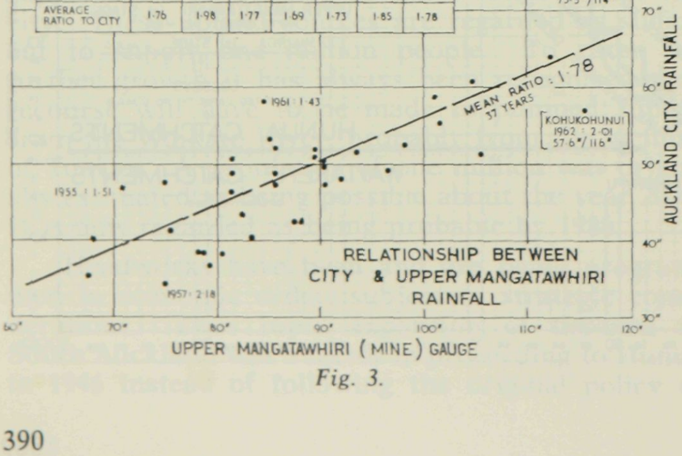

Fig. 3.

In 1963 the criteria used for transmission provision was 5.5% increment on a peak-week figure of demand plus 30%. The latter factor is now increased to 35% to cover more recent trends and to give some coverage for critical periods which may commence with reservoir storage depleted.

In all cases a 10% margin is added to cover year-by-year variations from general trends and provide some latitude for emergencies.

2.2. Hydrology

Since 1957 the establishment of additional rain-gauge stations and of continuously recording stream-gauging weirs on the Huia, Wairoa, Mangatawhiri, and Mangatangi catchments, together with the significant increase in the time covered by catchment records, has made possible a complete re-evaluation of catchment hydrology. The results of this work are summarised in Tables 3 to 6.

Copies of all hydrological data are supplied to the Government and to all the territorial authorities concerned in any way with the Hunua area. The Mangatawhiri catchment has been designated a "regional catchment" by the Soil Conservation and Rivers Control Council, which has installed a precision level recorder in the valve tower of the Upper Mangatawhiri dam, to be used in conjunction with the Auckland City Council's rain and stream gaugings for flood evaluation.

2.2.1. *Rainfall Data*

Present analyses confirm the essential validity of the assumption, which has had to be made in all initial assessments of catchments, that rainfall distribution throughout the supply area is in general consistent and therefore related to city rainfall. This confirmation is important, for although city records go back 102 years, catchment data is relatively limited and, except for two gauges installed in the Waitakere reserves in 1911, does not cover any of the critical dry periods experienced by Auckland, all of them prior to 1915.

The ratio of rainfall at individual catchment stations to city rainfall varies considerably from year to year, but it can now be demonstrated that average ratios over periods as short as five years are usually relatively consistent. This is shown on Fig. 3, which gives the scatter of plots of relative rainfall for the city for a typical gauge, one of the

TABLE 3

Hydrological Data: To 31 December 1964

Rainfall, Auckland city, calendar years

102 year mean since 1863	*Maximum year*	*Minimum year*	*Critical dry periods* Period	*Critical dry periods* Mean	*Critical dry periods* Driest year
	1956	1914	1863- 6 = 4 yrs	40.7 in	1864 = 37.5 in
	75.5 in	28.4 in	1876-82 = 7 yrs	40.9 in	1881 = 34.2 in
47.3 in	mean +60%	mean −40%	1884- 8 = 5 yrs	40.0 in	1885 = 32.5 in
			1911- 4 = 4 yrs	38.7 in	1914 = 28.4 in
Average three driest consecutive years			1912- 4 = 3 yrs	36.6 in	= mean × 0.775 in

TABLE 4

Rainfall: Water Supply Reserves

Reserve	*Recording commenced*	*Gauging stations Present stations* No.	*Average density*	*Instruments Automatic* Month	*Automatic* Day	*Non-automatic* Month	*Non-automatic* Week	*Non-automatic* Day
Waitakere	2 : 1911	15	1 per 1,100 acres	1	1	2	8	4
Hunua	4 : 1928	23	1 per 1,200 acres	2	2	3	12	4

Check non-automatic gauges are installed, also, at each automatic station.

TABLE 5

Run-off Data: Gauged Catchments

Catchment	*Period*		*Annual rain over catchment for period* (in)	*Ratio to mean* (%)	*Run-off factor* (%)
Huia (total): part storage	17 mths.	1962- 3	78.5	+ 4.0	62.5
		1963	64.9	−14.0	55.9
		1964	80.7	+ 6.9	66.2
Cosseys:					
Stream only: no storage	5 yrs.	1947-53	59.5	− 2.3	58
First filling	14 mths.	1955- 6	68.5	+12.5	47
Later periods of no overflow	11 mths.	1959-60	67.5	+10.8	45.2
	12 mths.	1961- 2	64.0	+ 5.1	49.2
	19 mths.	1963- 4	64.4	+ 5.7	45.5
Wairoa:		1961	62.9	− 2.2	58.4
No storage		1962	88.4	+37.5	59.2
		1963	60.5	− 5.9	48.2
		1964	78.0	+21.4	55.9
Mangatawhiri:		1963	63.7	− 9.0	53.3
Total: no storage		1964	83.5	+19.3	58.5
Mangatangi:		1963	71.6	− 5.3	55.0
No storage		1964	92.2	+23.4	61.4

TABLE 6

Catchment Annual Rainfall and Run-off Data

Catchment	*Assessed long-term mean rain* Over catchment (in)	*Ratio to city*	*Area* (acres)	*1 in rain over area* (10^6 gal)	*Run-off data* Adopted factor (%)	*Mean run-off* (in)	*Run-off to city rainfall* (%)	*Storage expressed as rainfall* (in)	*Designed mean yield* (million gallons per day)	(rain per year —in)	(% of mean run-off)
Waitakere reserves											
Waitakere	69.0	1.46	2,040	46.18	55	38.0	80.3	8.8	3.5	27.7	73.0
Upper Nihotupu	78.0	1.65	2,480	56.13	55	42.9	90.6	10.5	5.0	32.4	75.5
Lower Nihotupu	68.5	1.45	3,220	72.88	53	36.4	77.0	14.5	6.0	30.1	82.7
Upper Huia	72.5	1.53	1,940	43.91	58	42.5	90.0	12.2	4.0	33.2	78.0
Lower Huia	76.5	1.62	3,550	80.35	55	42.0	88.8	15.8	7.5	34.1	81.2
Overall	72.6	1.54	13,230								
Hunua reserves											
Cosseys	60.9	1.29	5,390	122.00	50	30.5	64.5	26.1	10.0	29.9	98.0
Wairoa	64.3	1.36	3,200	72.43	51	32.8	69.4	27.6	6.5	32.7	99.0
Upper Mangatawhiri	74.4	1.57	6,380	144.41	54	40.2	85.2	25.2	14.5	36.7	91.3
Lower Mangatawhiri	63.7	1.35	5,000	113.18	51	32.5	68.8	26.5	9.5	30.6	94.0
Mangatangi	75.4	1.59	9,580	216.84	54	40.7	86.2	27.7	22.0	37.0	91.0
Overall	69.6	1.47	29,550								

older ones in the Hunua reserves, and shows that the average ratio is reasonably consistent over the whole range of rainfall intensity. Application of city data to assess "long-term" and "critical-period" catchment rainfall data is therefore now made with reasonable confidence, even for catchment stations established within recent years.

2.2.2. *Run-off*

All stream gauge stations are two-step rectangular weirs, with 4 ft 6 in Lea level recorders and maximum level indicators for floods over 4 ft 6 in head. Data from these stations, summarised in Table 5, cover a limited but useful period including years of high and low rainfall, and are sufficiently significant to indicate that original estimates of catchment yield were too liberal.

Gauged run-off from all four Hunua catchments, without storage, ranges from about 55% in low rainfall years to 60% in substantially above average years, but data from the Cosseys headworks indicate that when storage is operative, these figures will be reduced to a greater extent than was expected. Cosseys Lake is 5.5% of the catchment and therefore involves significant evaporation loss. Further loss is probably associated with groundwater re-charge as lake levels rise. The country rock at Hunua is relatively pervious argillite and greywacke. Considerations of water-table gradients therefore indicate that the re-charge loss as lake levels rise will exceed the yield from groundwater discharge as lake levels fall. Overall run-off from Cosseys for the periods quoted could be higher than the figures for periods of "no overflow" indicate, for run-off in the intervening periods of overflow (spillway discharge is not gauged) will be relatively high. On the other hand, it would be possible for any impounding reservoir designed with a relatively high storage factor, as is Cosseys, not to overflow for several years during a critically dry period, in which case the low run-off factors recorded for Cosseys could be appropriate.

Storage loss factors can be expected to be much less significant in the Waitakere reserves, for here the country rock is massive andesitic grits and conglomerate of low permeability; this also accounts for the high run-off factors recorded at Huia for years of above average rainfall.

2.2.3. *Flood Flows*

Flood data for Auckland catchments are incomplete. Automatic rain-gauging in the city goes back to about 1915, but was not commenced in the catchments until 1949. Non-recording weirs of limited range were installed on Waitakere streams at an early stage, but no full range flow records were available until 1947, when Cosseys (Hunua) recording weir was installed for five years prior to dam construction. Furthermore, although over the last 30 years rainfall has been in general greater than average, the only floods of any significance in the catchments over this period were two major ones in 1937 and 1938, in the Hunua area, for which, unfortunately, no useful data were recorded. For spillway design, therefore, reliance is still placed on analyses made in connection with the design of Lower Nihotupu dam and summarised by A. D. Mead in his paper (2) on this project, based primarily on city intensity data and on a well authenticated maximum flood for the area of 11,300 ft^3/s from 5.2 $mile^2$ (2,175 $ft^3/s.mile^2$) at Huia, on 25 December 1926, during the construction of Huia dam. With due regard for the necessity of providing ample spillway capacity for an earth dam, and with adjustments for catchment shape, topography, rainfall, and ponding characteristics, this gave a limiting flood discharge for Lower Nihotupu of 16,300 ft^3/s from 8.5 $mile^2$, or 1,970 $ft^3/s.mile^2$. On the same bases, Cosseys spillway has been designed to discharge 15,000 ft^3/s from 8.2 $mile^2$ (1,830 $ft^3/s.mile^2$) and Upper Mangatawhiri 24,000 ft^3/s from 9.8 $mile^2$ (2,500 $ft^3/s.mile^2$).

2.2.4. *Safe Mean Yield and Storage*

Catchment yields based on revised assessed run-off factors for critically dry periods are in some cases lower than previously listed, but this has been partially countered in recent headworks by designing impounded storage, wherever economically possible, more on the basis of mean rainfall than on the "three driest consecutive years" basis which has been traditional for water supply (see Tables 1 and 6).

Thus the Waitakere headworks, designed soundly on the "three driest years" in keeping with demands at the time and physical limitations at some sites, have safe mean yields corresponding to from 38 to 43% of mean rainfall on their respective catchments (73 to 83% of long-term mean run-off), whereas at Cosseys, Hunua, a very favourable storage-to-dam cost factor made a safe mean yield of 49% of mean rainfall (98% of long-term run-off) economically obtainable. At Upper Mangatawhiri physical factors at the dam site limited the height of the dam and thereby determined the economic limits of storage, but even so a safe mean yield of 49.5% of assessed mean rainfall (91.3% of long-term run-off) has been obtained.

This departure from the traditional is justified by the present magnitude of the undertaking. At all times, other than in critical years, it permits up to peak-capacity draw-off from the headworks concerned over longer periods than normal, thereby enabling pumped headworks to be used largely as peak-load stations only, with appreciable savings in pumping costs. This has already been demonstrated in the operation of Lower Nihotupu pumped headworks, will be accentuated when Lower Huia pumped headworks, shortly to commence, is in service, and will be of greatest importance when pumping from the Waikato River is a reality.

A mass-curve analysis for Upper Mangatawhiri headworks for the "drought period" 1911-4, simplified by assuming a uniform run-off factor over the whole rainfall range, is shown in Fig. 4. The mean yield "curve" for Cosseys headworks is also shown on this diagram, appropriately adjusted for scale for comparative purposes, to indicate the long cover provided by the storage available there.

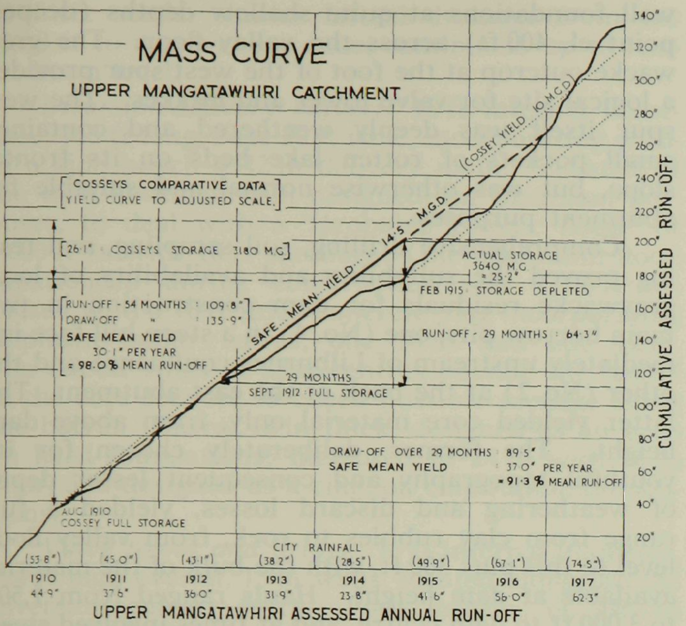

Fig. 4.

3. UPPER MANGATAWHIRI HEADWORKS

3.1. General

This section covers new matter. In 1957 it was stated that site conditions on Mangatawhiri Stream in Moumoukai Valley at an elevation suitable for gravity supply to Khyber Pass reservoir in the city (el. 320 ft) were unfavourable for the construction of an impounding dam, and that therefore development at this level was to be limited to run-of-the-stream supply only. In 1959-60, however, a detailed geological re-survey showed that conditions were in fact satisfactory for rolled filled embankment construction, and further studies demonstrated that sufficient economic storage could be obtained behind such a dam to sustain a safe mean yield of 14.5 m.g.d. instead of a stream-flow yield which could fall to as low as 3 m.g.d. Work on the new proposal was put in hand immediately, the main contract was let in October 1961, and the project was commissioned on 23 March 1965 for a total expenditure, including delivery lines in Moumoukai Valley, of £1,480,000.

Consequentially upon this change from original planning, the whole concept for further development of the Mangatawhiri Catchment has had to be altered, and this in turn has affected transmission and treatment provision for Hunua in general. Originally it had been planned that the main development of Mangatawhiri would be by major impoundment at the outlet of Moumoukai Valley at el. 330 ft, with a separate transmission system to the city at a level 100 ft lower than the present one. The lower Mangatawhiri project will now be so restricted that it will be far more economical to pump its now limited yield into the upper system. The combined effect on transmission of all these changes will be that tunnel aqueducts and the Ardmore filter station, designed and constructed (or planned) to take the peak-yield of 54 m.g.d. from all original high-level headworks, that is those capable of gravity supply to Khyber Pass, will now have to transmit Mangatawhiri low-level yield as well. The new total of up to 85 m.g.d. can be handled by surcharging the tunnels. The Ardmore filters can be readily extended.

3.2. Geological Factors

3.2.1. *Basic Materials*

The general geology of the Hunua reserves and of the individual catchments has already been described (1). Figure 5 shows the basic structure of the area.

Mangatawhiri Stream traverses the greater part of the length of the central part of Hunua Hills block, a 10 mile by 9 mile mass of greywacke basement rock strongly upfaulted along its southern and western boundaries, and flows south, against the general northerly tilt imposed on the block by this faulting, to join the Waikato River near Mercer. For six miles within the hills it is essentially at grade in a remarkable gravel-floored, mature, aggraded valley, Moumoukai Valley, set deep in an otherwise rugged terrain. It leaves the valley at el. 330 ft (lower Mangatawhiri dam site), at the fall-line of a two mile long youthful gorge cut deep across the southern fault line.

Moumoukai Valley in this, its present form, originated in the back-ponding of the stream by fault-tilting of the Hunua block, probably about one million years ago, to form a long, narrow lake at least 100 ft deep, which was in time infilled with gravel and silt, and later drained by overflow-down-cutting of the outlet gorge. Rejuvenation of the stream through the valley by lowering of the outlet has enabled it to down-cut to bedrock throughout the lower half of the valley, and in the new erosion cycle initiated by this lowering the greater part of the old lake-fill has been eroded away. It is still, however, preserved as terrace remnants at intervals throughout the valley. Modern flood-plain silts and gravels related to present stream levels completely dominate the upper half of the valley floor.

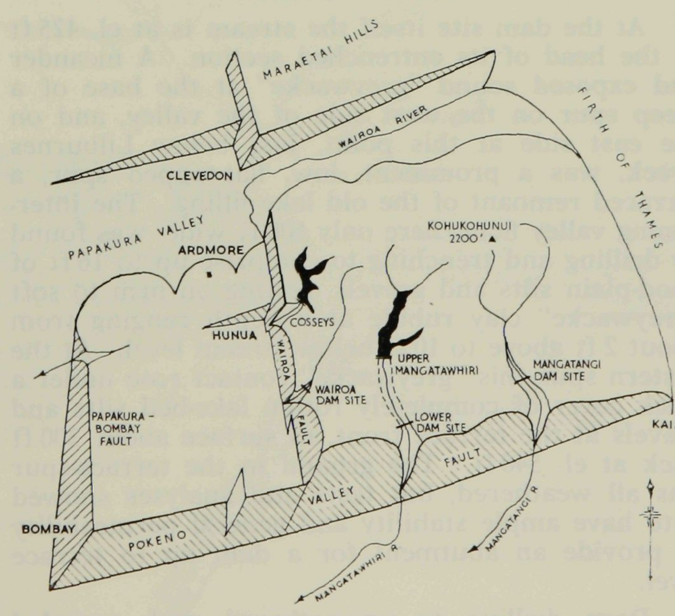

Fig. 5: Block diagram, Hunua Hills area.

Country rock in the Mangatawhiri catchment is typical Trias-Jura "basement greywacke", dominantly argillite with minor true greywacke. Structural trend is N 30° W, more or less paralel to Moumoukai Valley, with dips steep to the west. Manganese oxides are usually present, particularly in the more shattered phases of the argillite, and in two places in the upper watershed are sufficiently concentrated to have been mined in the past. Irregular silicified zones are at times associated with the manganese-bearing formation, and are revealed by resistance to weathering. Otherwise weathering is deep —a feature in keeping with the past history of the area—and owing to steep dip and formational variations, very irregular.

The profile of weathering is typical of "greywacke", except that where the parent rock is manganiferous the upper parts of the profile are deep, leached, reddish-yellow, silty "clays", dominated by gradings in the capillary range and therefore highly water-absorbent, difficult to work, and unsuitable for rolled fill. On the other hand, materials in the profile for more normal "greywacke" are ideal for the purpose and show the usual regular gradation from unweathered "blue" greywacke or argillite, through partly weathered "blue-grey" rock, which quarries out as rubble ideally suitable for shoulder construction, to leached material which similarly produces "brown" and "clay" rubbles ideal for core work, to fine sandy or silty surface clays. The depth of weathering has produced a general surplus of core-type material, and this, plus the manganiferous residual silty clays referred to, expectably results in a high reject and stripping to placed fill ratio.

A close-to-surface occurrence of fine-grained greywacke and shattered, unweathered argillite, massive in bulk owing to partial cementation by silica and other secondary minerals, was located in a handy position in the main borrow pit and provided all facing stone and stone backing for the water face of the dam.

3.2.2. *Site Conditions*

At the dam site itself the stream is at el. 425 ft at the head of its entrenched section. A meander had exposed sound "greywacke" at the base of a steep spur on the west side of the valley, and on the east side at this point, just below Lilburnes Creek, was a prominent, low, flat-topped spur, a terraced remnant of the old lake filling. The intervening valley floor, here only 600 ft wide, was found by drilling and trenching to comprise up to 16 ft of flood-plain silts and gravels, resting on firm to soft "greywacke" clay rubble at a depth ranging from about 2 ft above to 10 ft below stream level. At the eastern spur this "greywacke" contact rose under a thick cover of completely rotten lake-bed silts and gravels at the terrace front, to surface about 300 ft back at el. 540 ft. The ground in the terrace-spur was all weathered, but tests and analyses showed it to have ample stability and general permeability to provide an abutment for a dam up to terrace level.

Deep drilling to unweathered rock revealed sound grey-brown "greywacke" suitable for core-wall foundations at quite shallow depths (deepest point el. 400 ft) across the valley floor. The greywacke outcrop at the foot of the west spur provided a logical site for valve tower and intakes. The west spur itself was deeply weathered and contained small pockets of rotten lake beds on its frontal slope, but was otherwise normal and suitable for abutment purposes.

Comprehensive drilling, soil sampling, and testing proved the suitability and availability of local greywacke residuals for dam construction in two main borrow pits, one (No. 4) in a steep hill face immediately upstream of Lilburnes confluence and the other (No. 2) at the back of the east abutment. The latter yielded core material only, from above dam height. The former, deliberately chosen for its youthful topography and consequent lesser depth of weathering and discard losses, yielded a full range from clay rubbles to rock, from valley floor level to this plus 300 ft, with the bulk of the material available at dam height. Hauls ranged from 1,500 to 3,000 ft to dam centre, and at times involved steep climbs. More conveniently placed material at the western abutment and nearby was unsuitable because of the presence of rotten lake beds or manganiferous silty greywacke residuals.

The open nature of the site gave great flexibility of working. Thus total earthworks of nearly 2,300,000 yd^3 on this job were handled in the same time as that taken for 735,000 yd^3 at Cosseys, where dam construction with similar materials was cramped into a narrow, youthful gorge.

3.3. The Project

3.3.1. *Design Features*

The proving of the site was completed during 1960, and designs were finalised for calling of tenders for dam construction in July 1961. The contract was let to Wilkins and Davies Construction Ltd. in October 1961, for completion by 30 April 1965, with impoundment to commence on 1 December 1964. Both targets were met, thanks in part to better-than-normal weather throughout and to excellent job-relationships between the contractor, the subcontractor for earthworks, Seton Contracting Co. Ltd., and the Auckland City Council. The main features of the work as finalised are illustrated in Figs. 6, 7, and 8. Some details are:

Project rating: Safe mean yield 14.5 m.g.d., from 6,300 acres and 3,640 million gallons storage; peak-load delivery capacity 22 m.g.d. through 6,000 ft of 42 in main to Mangatawhiri tunnel.

Reservoir: 337 acres when full, 1.9 miles long, up to 100 ft deep.

Dam: 1,543,000 yd^3 rolled fill; crest 1,620 ft long at el. 540 ft, 20 ft wide, up to 130 ft above foundations; upstream face 1 in 3, rock-faced; downstream face 1 in 2½, bermed and grassed; maximum width of base 730 ft. Overflow level el. 530 ft, with 3 ft freeboard above highest flood level plus concrete wave-break 3 ft high. Core section, roughly the centre one-third, of "clay" and "brown" greywacke residual rubbles; shoulders of harder "brown" to "grey" rubbles, separated from the core downstream by an 8 ft wide gravel drainage blanket protected by filter zones; similar lateral drainage from the blanket to a gravel toe zone, continuous up to lowest berm level.

Diversion-delivery culvert: 16 ft internal horseshoe, 713 ft long, base 20 ft wide, incorporating within a side bench a 42 in delivery main. Located at the west abutment and designed, for diversion purposes, to deal with a "probable 20-year flood" of 8,000 ft³/s with 10 ft surcharge (the maximum flood experienced during construction was 1,000 ft³/s). Closed with a 10 ft thick concrete plug incorporating duplicate scour valves, one 30 in and one 12 in. Ten cut-off rings at 25 ft centres outside the barrel in the core section, to check possible seepage along the barrel. Barrel 18 in thick, designed as an arch on hard foundations; increased to 24 in with 30 in floor, and designed as a ring, on clay rubble foundations.

Valve tower and bridge: 16 ft internal diameter, 124 ft high overall, 15 in walls, located alongside the diversion culvert entry. Four intakes, el. 440 to 520 ft, to a 42 in standpipe. Valves operated from grid-floored stage platforms. Footbridge to tower, six 40 ft spans, 6 ft wide deck.

Spillway: Located in weathered "greywacke" in the eastern terrace-abutment. A side weir 380 ft long, curved to suit the country, to discharge up to 24,000 ft³/s with 7 ft head into a trapezoidal channel 37 to 52 ft deep below dam crest, 22 to 30 ft wide at the base; 60° side slopes, lined with 7 in concrete, tied back with anchor rods at 10 ft centres and made as a stable structural unit with post-tensioned ribs at 10 ft centres. The design, developed from 1:48 model tests at the School of Engineering, University of Auckland, incorporates a 40 ft long transition section between the weir section and a steep, rectangular discharge channel, 578 ft long, 30 ft wide, walls 8 in thick, top-strutted, 24 to 17 ft high, which ends in a flip-bucket and discharges into an open-cut channel. The transition from trapezoidal to rectangular section was necessary to throttle the discharge to ensure subcritical velocities in the weir section at high discharges.

3.4. Construction

3.4.1. *Pre-contract Work by the Auckland City Council*

This included extensive roadworks, accommodation provision, the installation of an intake weir upstream of the dam site for interim run-of-the-stream supply to the city, the laying of a temporary 33 in main from the weir down to and across the dam site and the laying of a permanent 42 in, 6,000 ft long delivery main to Mangatawhiri tunnel. At the dam site itself, the Council diverted the stream from the site of the diversion culvert by means of an open cut down the centre of the valley, and made a substantial start on centre-line grouting. Completion of grouting was included in the contract. The Council also stockpiled and later supplied to the

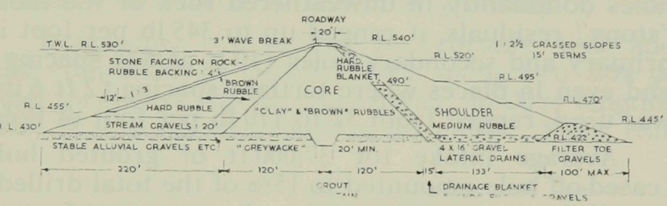

Fig. 6: Upper Mangatawhiri dam, typical cross-section.

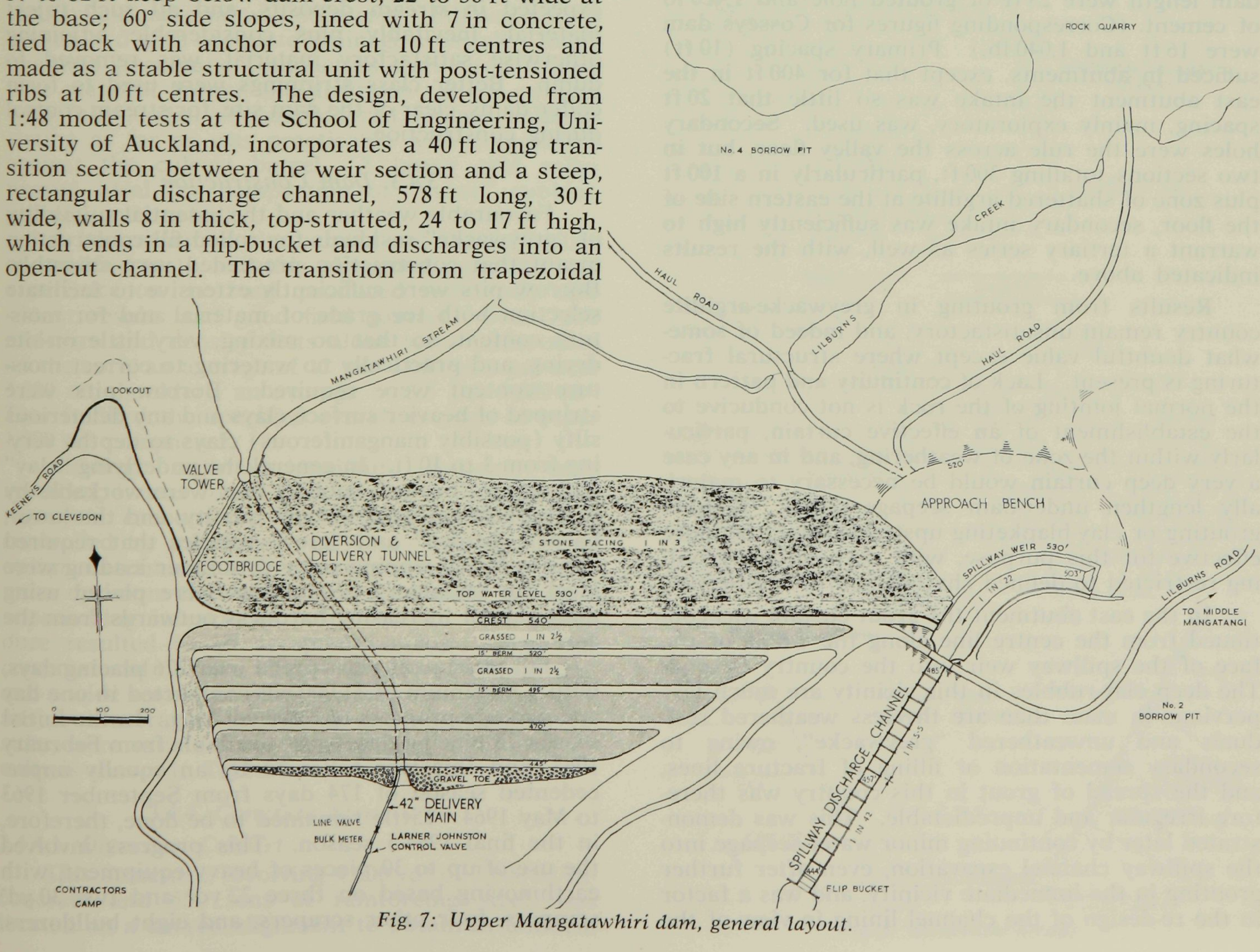

Fig. 7: Upper Mangatawhiri dam, general layout.

contractor all the concrete metal aggregate required, at a nominal figure of £1/yd³ delivered, from crusher plant established originally to process greywacke tunnel spoil for aggregate for tunnel lining. The nearest source of sand for concrete was the Waikato River at Mercer, a haul of about 23 miles.

3.4.2. *Grouting*

Curtain grouting on the dam centre line down at least 30 ft into unweathered "greywacke" was specified, to pressures ranging from 30 lb/in² in top stages to double the ultimate water-head, that is up to 175 lb/in², in bottom stages. The depth of hole below cut-off ranged from between 65 and 105 ft across the valley floor and from 60 to 140 ft in abutments, with grout intake as erratic as is to be expected in "greywacke" country. Thus in the abutments, with holes dominantly in "clay" rubbles, intake varied from 9 to 65 lb per foot of hole over 10 ft stages, whereas across the valley floor, with holes dominantly in unweathered rock or the more "stony" residuals, it ranged up to 345 lb per foot in primary and secondary holes (10 ft at 5 ft spacing), and even in places where tertiary spacing (2 ft 6 in) was used, ranged from 32 to 170 lb per foot of hole.

Average intake for 37,000 ft of grouted hole (cased-off hole amounted to 15% of the total drilled) was 60 lb of cement per foot. Averages per foot of dam length were 23 ft of grouted hole and 1,420 lb of cement. (Corresponding figures for Cosseys dam were 16 ft and 1,040 lb.) Primary spacing (10 ft) sufficed in abutments, except that for 400 ft in the east abutment the intake was so little that 20 ft spacing, mainly exploratory, was used. Secondary holes were the rule across the valley floor, but in two sections totalling 300 ft, particularly in a 100 ft plus zone of shattered argillite at the eastern side of the floor, secondary intake was sufficiently high to warrant a tertiary series as well, with the results indicated above.

Results from grouting in greywacke-argillite country remain unsatisfactory, and indeed of somewhat doubtful value except where structural fracturing is present. Lack of continuity and pattern in the normal jointing of the rock is not conducive to the establishment of an effective curtain, particularly within the zone of weathering, and in any case a very deep curtain would be necessary to materially lengthen under-dam seepage paths. Blanket grouting or clay-blanketing upstream could be more effective for this purpose, with deep-curtain grouting restricted to defined shatter- and/or fault-zones.

In the east abutment the grout curtain was continued from the centre line along the whole of the face of the spillway weir into the country beyond. The deep clay-rubbles in this vicinity are much less pervious in mass than are the less weathered residuals and unweathered "greywacke", owing to secondary cementation or filling of fracture lines, and the spread of grout in this country was therefore irregular and unpredictable. This was demonstrated later by continuing minor water-seepage into the spillway channel excavation, even after further grouting in the immediate vicinity, and was a factor in the re-design of the channel lining in view of the possibility thus revealed of water-pressure developing behind the lining.

3.4.2. *Foundations*

As at Cosseys, reliance for water-seal is upon a central core trench at least 20 ft wide, without a concrete key-wall, carried down into the top of the grout curtain to firm "grey-brown" grouted greywacke, generally defined, across the valley floor, as the limit to which heavy scraper-units, without ripper assistance, could operate. In abutments, owing to lesser head and the relative impermeability in mass of the thick "greywacke" residuals, foundations were taken down to firm, grouted "clay" or "brown" rubbles. In all cases positive integration of fill with in-situ material was demanded.

Trench depths were essentially as expected—somewhat less in general—but local water seepage was experienced in the valley floor section in spite of apparently satisfactory grouting. This seepage was led to two pump-sumps by means of gravel-filled, covered drains, which later, when sufficient weight and height of fill had been placed, were grouted through pipes left in place for this purpose.

The main fill outside the core-trench is founded on in-situ country stripped down a nominal 4 ft on abutments and 2 ft on the flood-plain deposits of the valley floor. The latter were checked for substandard wet-silty or "puggy" pockets such as must be expected in stream deposits, and all such reject material, inevitably plus considerable adjoining otherwise satisfactory material, was removed to dump. Better class strippings were used to form coffer dams, clear of the dam site, for stream control during construction.

3.4.3. *Dam Construction*

Favourable weather and the essential suitability of "greywacke" residuals for rolled fill construction meant that construction proceeded very smoothly. Borrow pits were sufficiently extensive to facilitate selection both for grade of material and for moisture content, so that no mixing, very little on-site drying, and practically no watering to correct moisture content were required. Borrow pits were stripped of heavier surface clays and any deleterious silty (possibly manganiferous) clays to depths varying from 3 to 10 ft. In general the underlying "clay" and softer "brown" rubbles that were workable by scrapers direct were of core quality, and the lower, harder "brown" and "grey" rubbles that required light to heavy ripping prior to scraper loading were ideal for shoulders. Shoulders were placed using materials of increasing hardness outwards from the core.

Fill placement was spread over 276 placing days, with a maximum of 11,000 yd³ compacted in one day and average progress of 6,000 yd³/day. In the initial season 75 placing days were available from February 1963, and this was followed by an equally unprecedented season of 174 days from September 1963 to May 1964. Little remained to be done, therefore, in the final 1964-5 season. This progress involved the use of up to 30 pieces of heavy equipment, with earthmoving based on three 22 yd³ and two 30 yd³ scrapers, four other scrapers, and eight bulldozers.

TABLE 7

Test Results, Dam Material

	Specification		*Core*		*Shoulders*	
	Core	*Shoulders*	*Tests*	*Averages*	*Tests*	*Averages*
Moisture content	28%	Test	4,700	28.4%	56	25.2%
Density:						
Dry weight	95	100	3,800	93 lb/ft^3	3,000	102 lb/ft^3
Wet weight	120	125	3,800	120 lb/ft^3	3,000	127 lb/ft^3
Air voids	—	—	—	3.2%	—	4.3%
Triaxial (total stress)						
Cohesion	2,000	1,000	130	1,495 lb/ft^2	—	1,350 lb/ft^2
Angle of internal friction	10°	17.5°	130	24.5°	—	23.5°
Unconfined shear	—	—	1,500	1,630 lb/ft^2	47	1,910 lb/ft^2
Permeability K (cm/s)	—	—	64	2.9×10^{-6}	16	6.2×10^{-4}

Gradings, permeabilities, and shear values were specified for fill types from the extensive sampling and testing undertaken at the investigational stage, but in practice, control by moisture content and density was all that was really necessary. Gradings in "greywacke" residuals are a function of breakdown effort and are not critically significant in this type of construction. Satisfactory shear values and permeability, provided moisture and density were satisfactory, were readily achieved.

Compaction was normal, averaged six passes of a compactor-type footed roller on each 6 in compacted layer. When work was resumed after the winter close-down, very little treatment of the previously laid material was necessary. Fill was always maintained with adequate drainage crossfall. Hand-operated pneumatic compactors were used to seal core material around the diversion culvert cut-offs, with fully loaded pneumatic-tyred scraper units assisting as far as was practicable with compaction against the culvert barrel. A heavy grid roller proved ideal for compaction of "stony" rubbles, gravels, and rockfill.

A laboratory on site carried out the full range of specified tests, and this plus good materials and experience on the bank produced sufficiently consistent results and good field control. Over 7,500 moisture and density tests alone were undertaken on the core, an average of 12 per 1,000 yd^3, and over 3,000 on shoulders, an average of 3.5 per 1,000 yd^3. This fulfilled the specified overall average of 6 per 1,000 yd^3 compacted.

Although optimum moisture contents were specified, it was found that the wide variations in materials encountered in borrow pits were best controlled by tolerances, +2% to –4%, applied to optima determined at frequent intervals. The maximum optimum that was adopted excluded anything approaching intractable "silty" clays. This procedure resulted in an accepted moisture range for core material of from 18 to 36% (22 to 34% optima) and a wide scatter of plots for moisture against density, but averages, and shear and permeability results, were satisfactory. A summary of job test results is shown in Table 7.

3.4.4. *Concrete Structures*

3.4.4.1. *Diversion Culvert*

This structure, involving 4,600 yd^3 of 3,000 lb/in^2 concrete and 500 tons of reinforcing steel, was located on a curved alignment to obtain as uniform foundations as possible on firm, weathered "greywacke" capable of supporting at least 5 ton/ft^2, but even so a harder bar encountered near the dam centre-line did cause differential settlement and some cracking under full embankment load. These cracks were effectively sealed by grouting.

Concreting commenced in February 1962 and was completed in October 1962, using two sets of steel formwork, inside and outside, in 20 ft lengths, with 7½ in p.v.c. waterbars at all joints. Floor con-

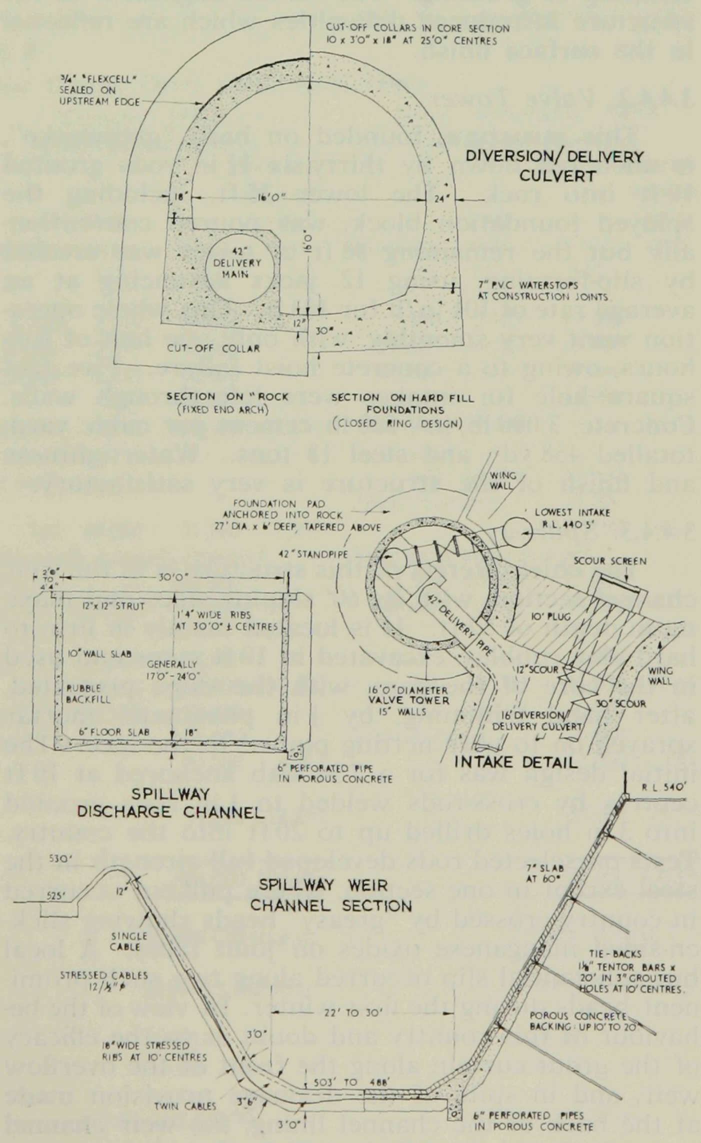

Fig. 8: Structural details.

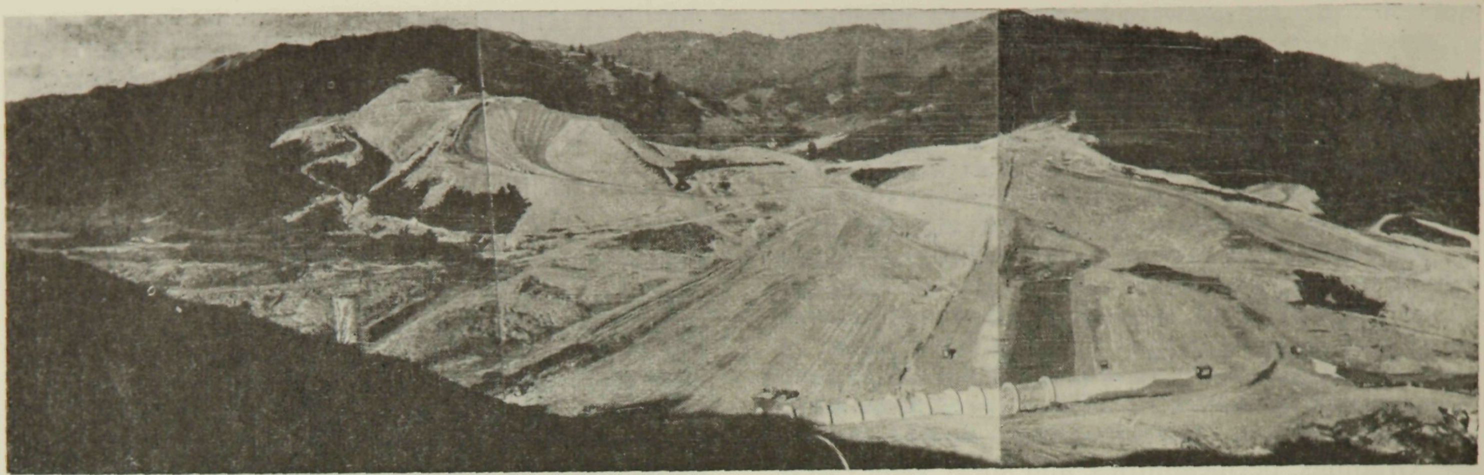

Fig. 9: Panorama of Upper Mangatawhiri dam, from west abutment, March 1963.

creting was carried ahead of arch concreting on a working pad of lean concrete, similarly in 20 ft lengths with waterstops. Closure of the tunnel in November 1964 by a 10 ft plug at the inlet was uneventful, but some seepages did subsequently develop through the tunnel walls, in spite of their 18 to 24 in thickness. These were readily staunched by caulking or grouting. The curved alignment of the structure introduced difficulties which are reflected in the surface finish.

3.4.4.2. *Valve Tower*

This structure, founded on hard "greywacke", is anchored down by thirty-six $1\frac{1}{8}$ in rods grouted 10 ft into rock. The lower 25 ft, including the splayed foundation block, was poured conventionally but the remaining 88 ft of tower was erected by slip-forming, using 12 jacks advancing at an average rate of $10\frac{1}{4}$ in/h for 103 h. This whole operation went very smoothly, with only one halt of two hours, owing to a concrete hoist failure. Five feet square hole for intakes were left through walls. Concrete, 3,000 lb/in^2, 560 lb cement per cubic yard, totalled 458 yd^3, and steel 18 tons. Watertightness and finish of the structure is very satisfactory.

3.4.4.3. *Spillway*

The chief interest of this structure is in the weir channel section, with its 60° sloping sides and maximum depth of 52 ft. It is located mainly in firm to hard clay rubbles, excavated in 10 ft steps and used in the core of the dam, with the slope protected, after hand trimming, by $\frac{1}{2}$ in pneumatic mortar sprayed on to wire netting pinned to the face. The initial design was for a 7 in slab anchored at 10 ft centres by cross-rods welded to 1 in rods grouted into 3 in holes drilled up to 20 ft into the country. Tests on selected rods developed full strength in the steel except in one section, where pull-out occurred in country crossed by "greasy" heads showing slicken-sided manganese oxides on joint faces. A local but substantial slip occurred along two such prominent heads during the first winter. In view of the behaviour of this country and doubt as to the efficacy of the grout curtain along the front of the overflow weir, and in spite of the drainage provision made at the back of the channel lining, the weir channel was redesigned as an independent trough structure, stable against maximum water head outside the lining, using post-stressed ribs at 10 ft centres, continuous down slopes and under the floor. Stressing incorporated twelve $\frac{1}{2}$ in stranded wires in ducts, stressed 130 tons at each end.

Drainage is provided primarily by no-fines concrete 8 in thick behind slope linings for from 10 to 20 ft above floor level and 6 in thick under the floor, and in side drains on each side of the whole length of the spillway channel system incorporating 6 to 12 in perforated concrete pipe. Smaller transverse pipes are located at stressed ribs, and inspection pits are provided throughout.

Shuttering for the sloping weir channel walls was by trussed-steel, plywood-sheathed panels, 20 ft by 6 ft high, carried on rolled steel joist strongbacks 10 ft apart, held top and bottom on the slope and by yokes to the anchor bars grouted into the country. This system was rather difficult to control on the curved faces and involved a moderate amount of joint and face treatment after stripping, but the overall result was good.

The transition section between the trapezoidal weir channel and the rectangular discharge channel involved tricky design and construction, but otherwise construction was essentially conventional.

The cut-off across the end of the discharge channel, at the face of the flip-bucket, was made by driving a line of square-ended 18 in by 6 in reinforced concrete piles through clay rubble to firm "greywacke" at a depth of about 20 ft.

Concrete in the spillway features totalled 3,844 yd^3, with 218 tons of mild steel and 17 tons of stressed steel.

3.4.5. *Ancillary Works*

A 55 ft by 11 ft road-bridge spans the spillway at the head of the discharge channel, based on three prestressed, precast beams.

The 240 ft by 6 ft footbridge to the valve tower was built in place, on trestle piers at 40 ft centres.

Grassing of the downstream face was originally by sodding from prepared nearby grassland, but later this was replaced, to advice given by the Department of Agriculture, and with excellent results, by sowing a special clover mixture directly on consolidated clay and brown rubble on the slopes.

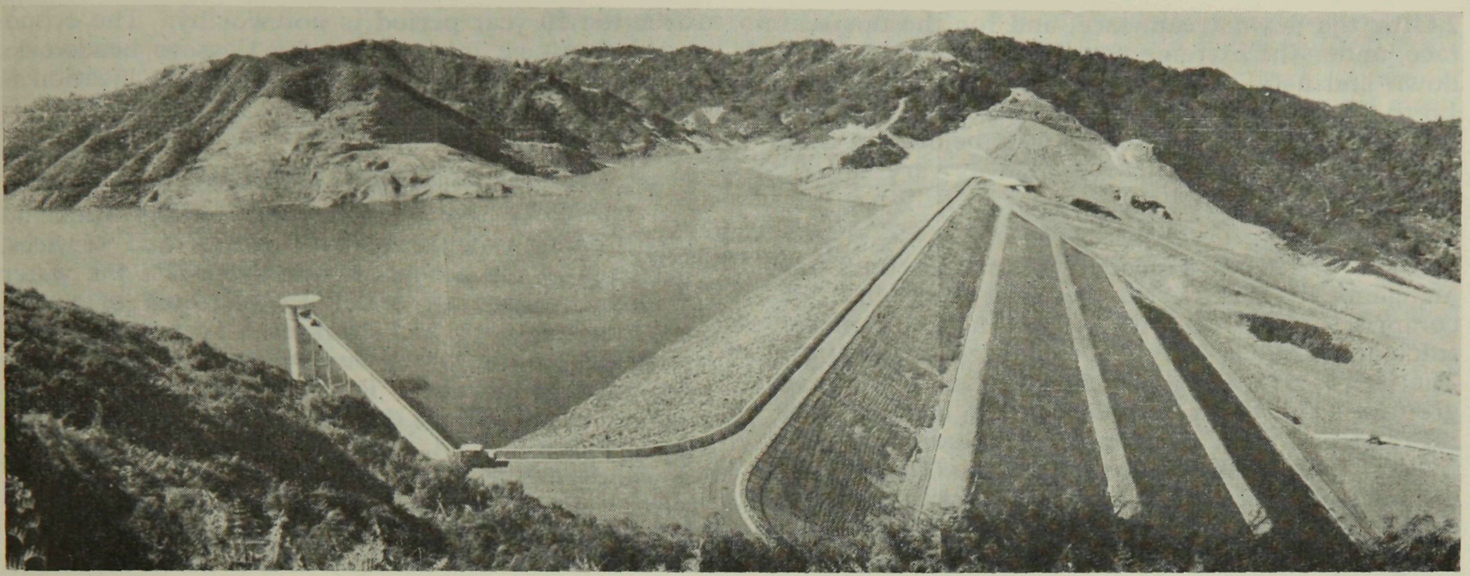

Fig. 10: Panorama of Upper Mangatawhiri dam, from west abutment, May 1965.

3.4.6. *Filling*

Following closure on 1 December 1964 the lake has filled steadily, in spite of fairly heavy draw-off into supply and by July 1965 it was two-thirds full.

3.5. Stability Analysis

Comprehensive analysis of the embankment as constructed, for critical shear circle and critical wedge failure, gave a minimum factor of safety of

TABLE 8

Rolled Fill Headworks: Comparative Costs, Dam Structures Only

	Upper Mangatawhiri: 1961-5 (greywacke country)				*Cosseys: 1951-5 (greywacke country)*				*Lower Nihotupu: 1945-8 (sandstone country)*			
	Quantity	*Unit rate* (£)	*Cost* (£)	*Percentage of total cost*	*Quantity*	*Unit rate* (£)	*Cost* (£)	*Percentage of total cost*	*Quantity*	*Unit rate* (£)	*Cost* (£)	*Percentage of total cost*
Embankment:												
Grouting (ft hole)	43,400	1.590	69,000		8,700	2.775	25,000		7,900	2.405	19,000	
Rolled fill (yd³)	1,505,000	0.374	560,000		510,000	0.255	130,000		470,000	0.250	118,000	
Stripping (yd³)	770,000	0.188	147,000		128,000	0.235	30,000		340,000	0.110	37,000	
Facings (sundries)	—	—	63,000		—	—	28,000		—	—	43,000	
Complete (yd³; gross)	1,543,000	0.543	839,000	59.0%	538,000	0.396	213,000	42.1%	486,000	0.446	217,000	61.1%
Diversion:	16′ culvert				14′ tunnel				11′ tunnel			
Reinforced conc. (yd³)	4,636	31.600	147,000		in rock				440	36.600	16,000	
Excavation (sundries)	—	—	41,000						lined channel		17,000	
Complete (ft)	730	258.000	188,000	13.3%	760	49.000	37,000	7.3%			33,000	9.3%
Spillway:	Side weir and channel				Bellmouth, tunnel, channel				Side weir and channel			
Reinforced conc. (yd³)	3,844	35.600	137,000		20′ x 96′	—	42,000		2,500	10.000	25,000	
Excavation (yd³)	78,000	0.530	42,000		17′9″ x 725	106.000	77,000		40,000	0.375	15,000	
Anchors, drains, etc.	—	—	43,000		675 yd³	26,000	40,000		—	—	3,000	
Complete (ft)	950	234.000	220,000	15.6%	1,040	153.000	159,000	31.4%	860	50.000	43,000	12.1%
Valve Tower: Complete	16′x112′	312.000	25,000		16′x135′	163.000	19,000		16′ x 71′	85.000	6,000	
Bridging and Sundries:	Foot and road		14,000		Foot	—	3,000		Foot & road		6,000	
			39,000	2.8%			22,000	4.4%			12,000	3.4%
Council supervision, clearing, pipework, roads, camp, etc.:	Excluding 42 in delivery line		132,000	9.3%			75,000	14.8%			50,000	14.1%
			1,420,000	100.0%			506,000	100.0%			355,000	100.0%
Cost per million gallons per day yield:			£98,000 (14.5 m.g.d.)				£50,600 (10 m.g.d.)				£60,000 (6 m.g.d.)	
Cost per million gallons per day storage:			£390 (3,640 m.g.)				£159 (3,180 m.g.)				£336 (1,057 m.g.)	
Average basic labour, weekly wage			£14.00				£9.35				£6.25	
Reinforcing steel in place			£120.00				£40.00				£41.00	
Concrete only and reinf. concrete complete			£12.10 and £30.00				£10.00 and £19.70				£6.00 and £10.00	

2.4 for the downstream face, and for the upstream face, under the extreme possibility of sudden draw-down and a fully saturated bank, minima of from 1.6 to 1.8 at water levels ranging between el. 485 and 440 ft, that is, between 45 and 90 ft below top water level. These results are regarded as very satisfactory.

3.6. Costs

The £1,480,000 expended exclusively on Upper Mangatawhiri headworks between 1959 and 1965 comprises £1,279,000 with respect to the dam contract and £201,000 City Council direct expenditure. The latter covers engineering expenses, preliminary grouting (£18,000), reservoir clearing, housing, roading, and first stage development, including the 42 in by 6,000 ft delivery main from the dam (£60,000), but excludes any part of the Mangatawhiri tunnel aqueduct. The latter will serve Mangatangi and Lower Mangatawhiri as well as Upper Mangatawhiri.

For the purpose of general estimating and comparison, Table 8 shows the relative cost of the salient features of the city's three post-war rolled-fill headworks, Lower Nihotupu, Cosseys, and Upper Mangatawhiri, and their respective economic merits as headworks only, exclusive of transmission and treatment costs. The relative steadiness of earthmoving costs compared with increases in other basic commodities of between two and three times over the 20 year period is noteworthy. The economic advantage inherent in the Cosseys headworks because of its very favourable dam site location is also demonstrated.

4. ACKNOWLEDGMENTS

The work was a team effort by the Auckland City Council's department of works and services, particularly the general design office and the water supply division, all under the director of works and city engineer, A. J. Dickson. Technical officers assigned to the water supply division for on-site supervision included: R. W. Sharp, resident engineer; D. A. Wilson, deputy resident engineer; and G. L. Dickson, E. A. Scanlen, and J. N. Clapperton, assistant engineers. S. Cotton, the division's analyst, was in charge of soil testing. Project engineer for the contractors, Wilkins and Davies Construction Co. Ltd., was P. D. Mataga, with J. Seton representing Seton Contracting Co. Ltd., subcontractors for earthworks.

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The Computer as Something to be Reckoned with

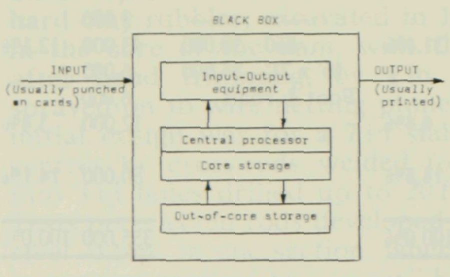
J. H. PERCY*

B.E.(HONS.), B.SC., PH.D.

These notes summarise a talk given to the Auckland Branch of the N.Z. Institution of Engineers. They are intended to explain, from the point of view of the practicing engineer, what a computer is and does, how much it costs, and what services are commercially available to engineers in New Zealand.

1. The Computer

1.1. *The User's Point of View*



1.2. *Input-Output Equipment*

Punched cards
Typewriter
Line printer
Digital plotter
Magnetic tape
Paper tape
Magnetic disc or drum

Also: sense lights, sense switches, digital displays, push buttons, cathode ray tube and light-pen, magnetic card files, document readers.

Off-line: card punch, tape punch, lister.

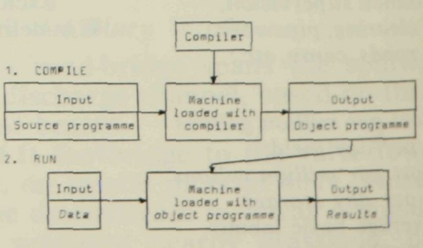
1.3. *Operating Procedure*

Load machine with *programme* to process *input data* and output *results.* Programme is in machine language (unintelligible to engineers).

1.4. *Mnemonic Language Programming Concept*

Write own source programme in mnemonic language (like English, e.g. Fortran, Algol) and translate into own machine-language object programme using machine-language *compiler* programme provided.

Two operations: (1) compile, and (2) run:



These can often be run as one (two-part) operation called *load and go.*

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