CHAPTER 5

HYDRAULICS

1. General Hydraulic Problems

Hydraulics as applied to conservation deals with the measurement and control of run-off from river drainage basins. Measurement has to do with such factors as precipitation - both rain and snow - the topography and vegetative covering of the area and the daily gauging of the flow of the river at selected points. Control deals with the prevention of floods by the use of reservoirs and other structures, and the increase of summer flow.

Floods which are caused by the natural run-off from river basins have occurred from time to time in Southern Ontario ever since records were first kept. Evidence of these can be found in diaries going back well over 150 years and from newspaper records for at least 100 years. Most of this run-off occurs in the spring, with the result that there is too much water in our rivers at the time of the year when it is needed least and very little, if any, during midsummer when it is required most. In addition to the flooding which is caused by spring run-off, occasional floods also occur during the summer on watersheds which have little natural protection. These summer floods do serious damage to crops. Such floods are not confined to a few of our largest rivers, but records show that all rivers of any consequence have from time to time caused serious damage in this way.

When Ontario was mostly covered with forest and the natural reservoirs, such as large swamps, had not been interfered with, severe flooding probably was not as frequent as it is today because these two factors had an ameliorating effect on the flow of water. Land clearing and drainage were necessary to open up the country for agriculture, but in some respects these were carried beyond the point of necessity, thereby aggravating the flood situation. In order now to regain a more or less stable condition of the rivers and streams, certain conservation measures must be carried out, such as the reclaiming of large swamps and water storage areas, the re-forestation of marginal and submarginal land, and also by a program of proper land use as indicated by farm planning, whereby run-off from gently sloping land can be controlled by such methods as contour cultivation and grass land where such is indicated. Such methods aim to control water where it falls on the land. If this could always be done it would be the ideal solution of the flood problem. But to minimize the required flood storage in a large watershed, a program of improved land use would need the co-operation of a great many individual farmers. This would take many years to accomplish. More immediate measures are therefore also necessary, especially where urban centres are frequently flooded.

One of the first problems facing the hydraulic engineer is to estimate or measure the run-off from a drainage basin which causes flooding farther down the valley. This includes a careful examination of rainfall over the years at different times of the year, which in turn presupposes that weather stations have been established in the area. Topography, types of soil, the amount of vegetative covering, particularly tree growth, on the area, and the gradient of the river, which has a bearing on the rapidity with which the water travels to the river's mouth, must all be carefully studied. If no gauging stations have been established then the run-off must be computed by taking the above factors into consideration and an approximate figure of flow is then determined by comparison with a neighbouring drainage basin which has gauge records in order to decide how much protection by the use of reservoirs is required. If, on the other hand, gauges have been established, by which a daily record is kept of the amount of water going down the channel at certain points, then a more accurate determination can be made of how much protection is needed. Fortunately, at London on the North and South Branches of the Thames there are hydrometric records dating from 1915, and at other stations on the watershed for shorter periods, and although the years of records for the latter are short they may be correlated with the long-term records and usually dependable run-off ratios established.

After the amount of run-off has been measured by whichever means are available to the engineer, it will give him a figure of flow which will indicate how much of this water will have to be held back by different methods in order to give the necessary protection where flooding is taking place. This means that a reconnaissance survey of the whole watershed must be made in order that suitable valleys be selected where dams can be built for the storing of the required amount of water. When more than a sufficient number of such reservoir sites have been selected, each must be measured as to its capacity, and the required number chosen to hold back sufficient water to solve the flood problem. In addition, wherever a dam is to be built, some subsurface exploratory work must be done at the site to make certain that the dam will have a proper foundation. Only after this preliminary work has been carried out can the reservoirs be finally chosen, the actual designing of the dam structures undertaken and the work carried through to completion.

While conservation reservoirs are usually built for the purpose of preventing floods, they are needed just as much in Southern Ontario for increasing summer flow. This has become increasingly important in recent years because rivers with extreme low flow and those which dry up entirely are a health menace to the communities through which they pass. Summer flow is necessary for flushing out the channel; to furnish water for industrial plants; for the practice of good agriculture; and is absolutely necessary for dilution where urban municipalities empty the effluent of their sewage disposal plants or raw sewage into the river.

The building of dams for the prevention of flooding and the increasing of summer flow is a comparatively new concept in engineering. It is only since the turn of the century that structures of this kind have been used for this purpose.in North America. The older methods included such projects as straightening and widening the river, narrow bridges and other man-made works which might obstruct the flow or cause ice jams. Also, occasionally, for such work a river was diverted into another watershed, or dikes were built to hold it within its banks. Such practices are aimed at one thing only, namely to get rid of water as quickly as possible. They do not take into consideration the necessity of holding water at the head-waters for deep infiltration or retaining it for summer flow throughout the year. On some rivers in Ontario channel improvements, diversions and even dikes must be carried out and built, especially where dams and reservoirs are not economical and summer flow is not a major problem.

2. The Upper Thames Watershed

(a) General Description

The Upper Thames Watershed comprises the drainage area of the Thames above the confluence of Dingman Creek with the main river ten miles south-west of the city of London. It measures 51 miles in length (north and south), has a maximum width of 37 miles, an average width of 26 miles and an area of 1324.9 sq. miles. The watershed is drained by two main branches and their tributaries, known as the North Branch and the South Branch, their confluence or "the Forks" being near the south-westerly limits of London. The Middle Branch is a major tributary of the South Branch, its confluence being approximately 16 miles above the Forks at London. Major tributaries of the North Branch are the Medway River which joins the North Branch about three miles above the Forks, Trout Creek which joins the North Branch at the town of St. Marys and the Avon River which joins the North Branch about four miles above St. Marys.

The rate of run-off[1] into these rivers and their tributaries is very high; so much so that the banks in places are unable to contain the flood waters and the low areas are flooded. Floods may occur at any time of the year but it is the floods which occur during the spring break-up that are frequent and most severe, and it is these floods which are the concern of this report.

(b) Cause of Floods

The topography, soils and other natural features of the watershed contribute in a great measure to flooding. The impervious clay soils, the high gradient of the river bed, and the steep lateral slopes of the tributaries increase the rate of run-off (Fig. H-1). There are no lakes or swamps which have been left untouched to serve as natural reservoirs; also, a high percentage of the forest has been removed. In addition

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1. Run-off is the amount of water that the drainage area supplies to the open streams and is the excess of precipitation over evaporation, transpiration and deep seepage.

there is a network of municipal drains above London which includes the straightening and widening of smaller creeks. These works undoubtedly increase the rate of run-off and aggravate floods.

The above adverse physical conditions are constant, but the magnitude of spring floods depends largely upon precipitation and temperature and the condition of the ground, that is, whether it is saturated or frozen. The depth and weight of the snow pack and the direction and the velocity of wind are also important factors. The most adverse combinations of factors, however, for a spring flood are frozen or saturated soil covered with a heavy snow blanket, accompanied by heavy rain and unseasonable prolonged high temperatures. Such floods may be further aggravated by ice jams impounding large volumes of water which, when the jams break, surge down and boost the flood peak.

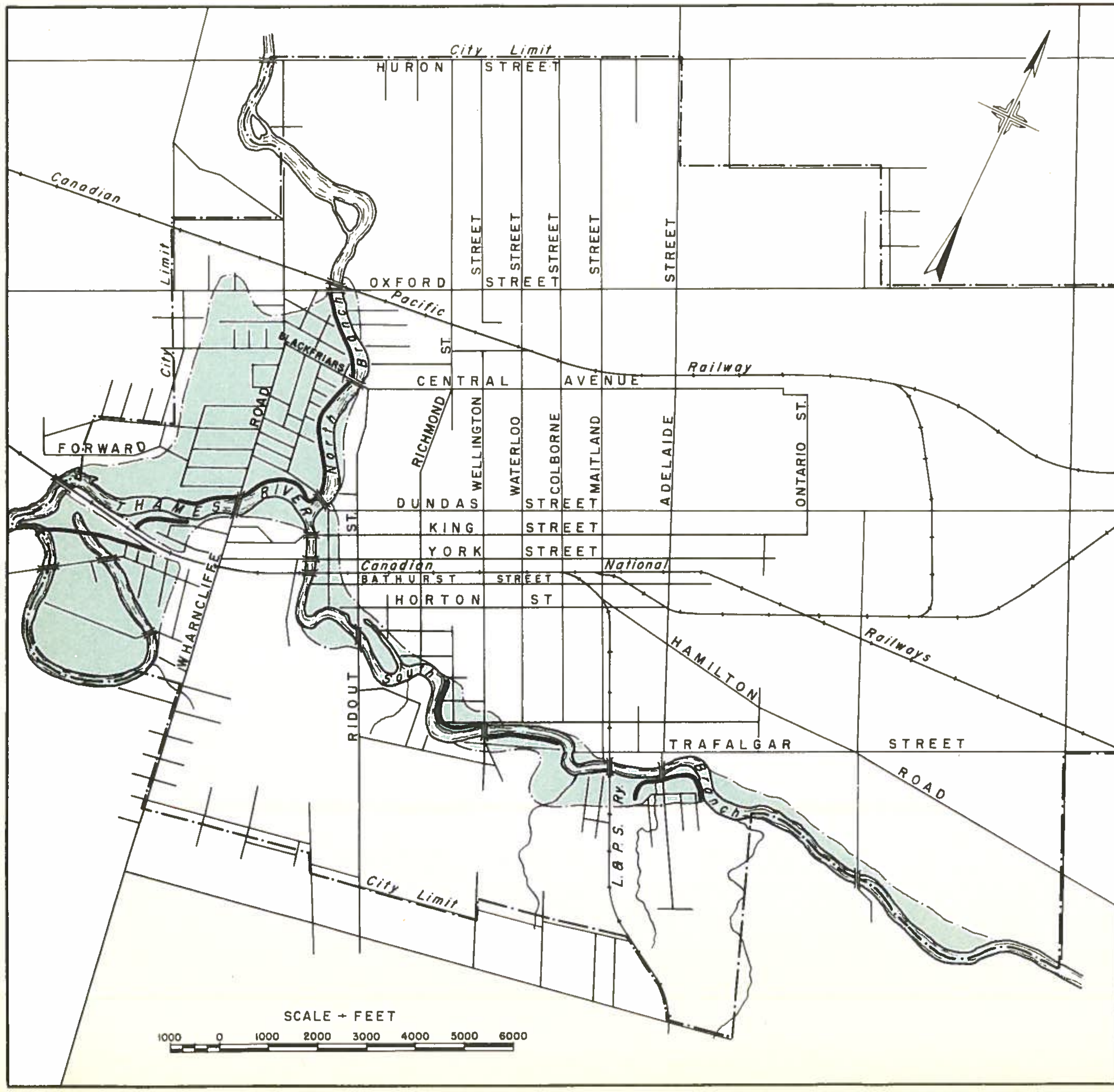
The towns and cities subject to serious floods are London, St. Marys, Mitchell, Woodstock and formerly Ingersoll, now relieved by channel improvement. (Figs H-2, H-3 and H-4 show the flooded area for each of the above places.)

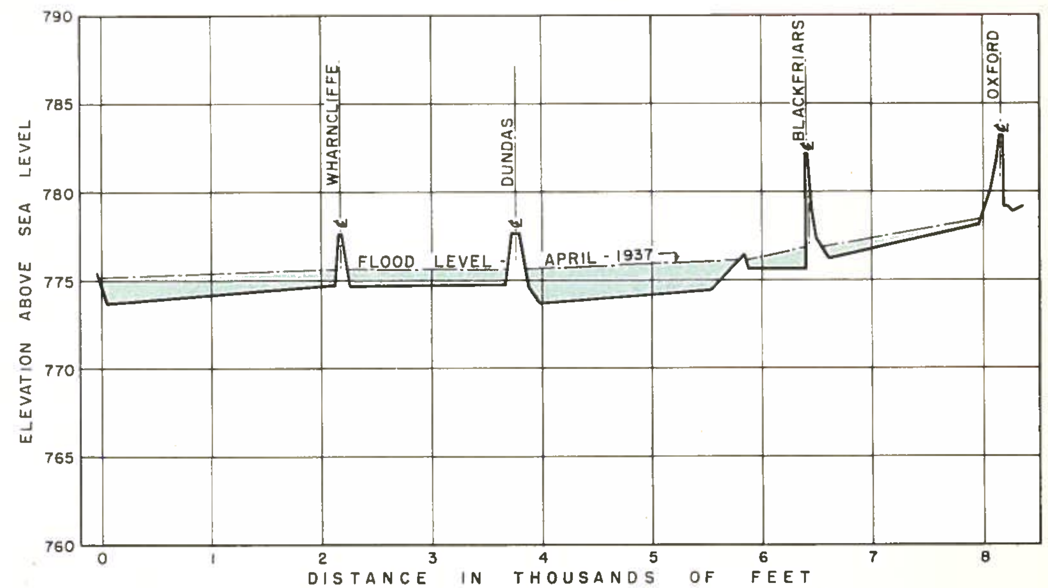
(c) Remedial Measures for Flood Control and Low Summer Flows

Of the above places subject to floods, London has sustained the greatest damage. Owing to its location at the Forks, it is in the most vulnerable position on the river, consequently in planning the location of required storage this fact has been kept in mind; but at the same time requirements for the safety of smaller municipalities up stream have not been overlooked. Low summer flows in both branches of the Thames are also a major problem and measures for both problems are complementary.

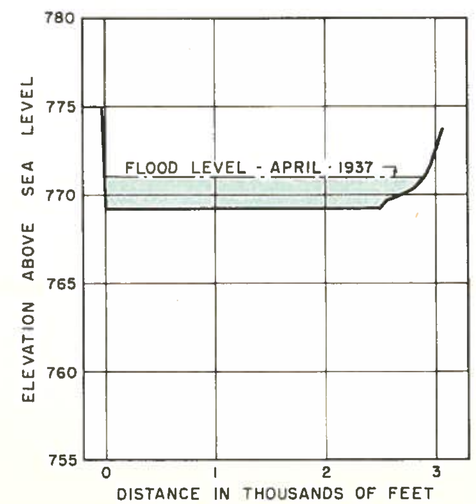
The land use and reforestation conservation measures recommended in this report are an essential part in any plan for flood relief and increasing low summer flows, but of themselves, even though immediately implemented, would not be sufficient to solve either problem and must be supplemented by storage of water in conservation reservoirs up stream which will reduce the flood crests to a safe stage, and later during dry periods may be released to increase the low flows.

Dikes and channels are aids only to a flood problem and should be considered as expedients. The benefit is local and they do not conserve water but have the reverse effect of speeding flood waters past the trouble area and often aggravate flooding down stream. Some diking and channel work, however, is sometimes necessary in conjunction with reservoir control when sufficient storage is not available or is too costly.





PROFILE OF DYKE IN LONDON WEST

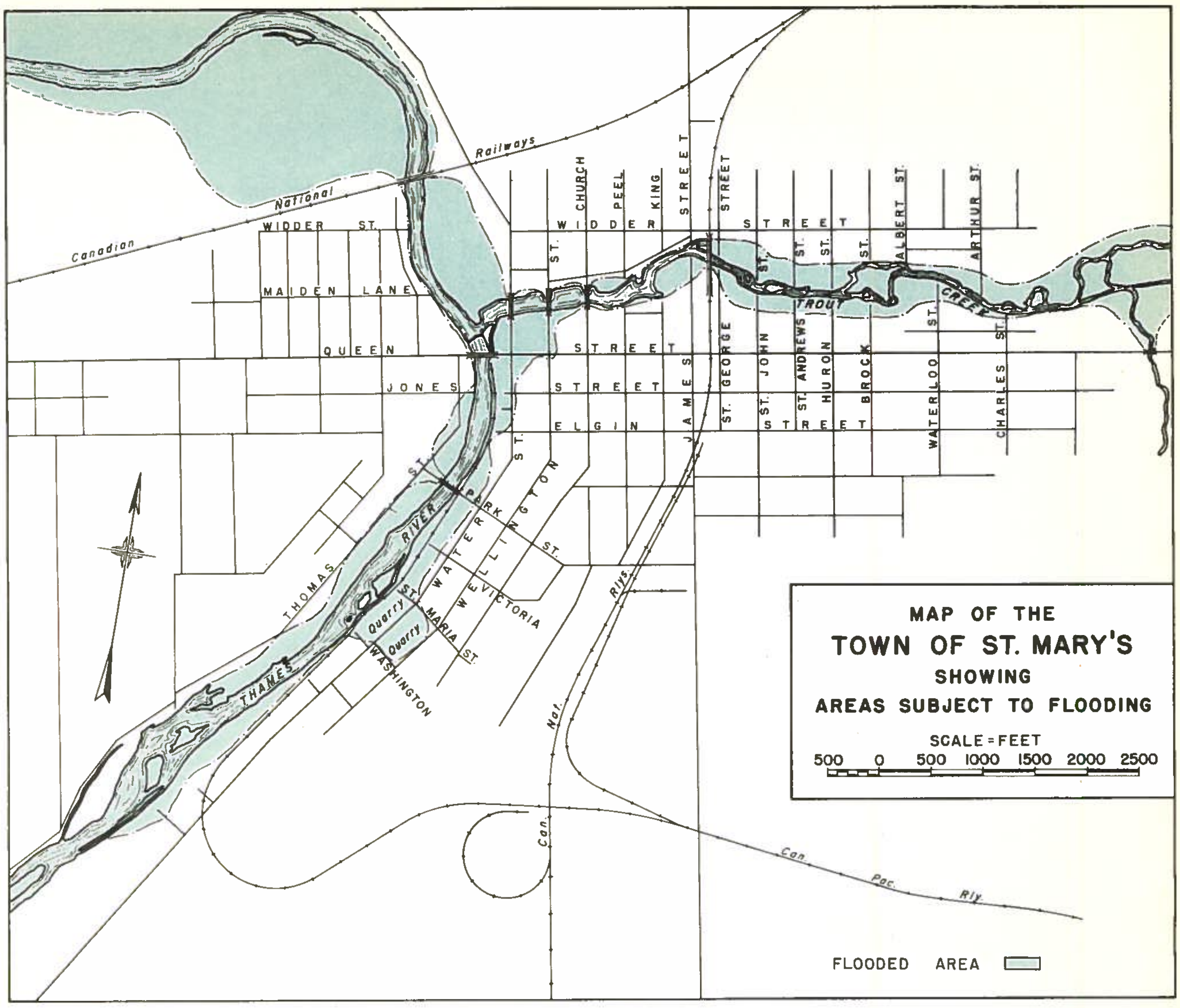


PROFILE OF COVE DYKE

MAP OF THE
CITY OF LONDON
SHOWING
AREAS SUBJECT TO FLOODING AND EXISTING DYKES

FLOODED AREAS
EXISTING DYKES

FIG. H-2


MAP OF THE
TOWN OF ST. MARY'S
SHOWING
AREAS SUBJECT TO FLOODING
SCALE = FEET
500 0 500 1000 1500 2000 2500
FLOODED AREA
Canadian National Railways
Can. Pac. Rly
Can. Nat. Rys.
THAMES RIVER
TROUT CREEK
WIDDER ST.
MAIDEN LANE
QUEEN STREET
JONES STREET
ELGIN STREET
WIDDER STREET
CHURCH ST.
PEEL
KING
WATER ST.
WELLINGTON ST.
PARK ST.
VICTORIA
MARIA ST.
WASHINGTON
THOMAS
Quarry
JAMES STREET
ST. GEORGE ST.
ST. JOHN ST.
ST. ANDREWS ST.
HURON ST.
BROCK ST.
WATERLOO ST.
ALBERT ST.
ARTHUR ST.
CHARLES ST.

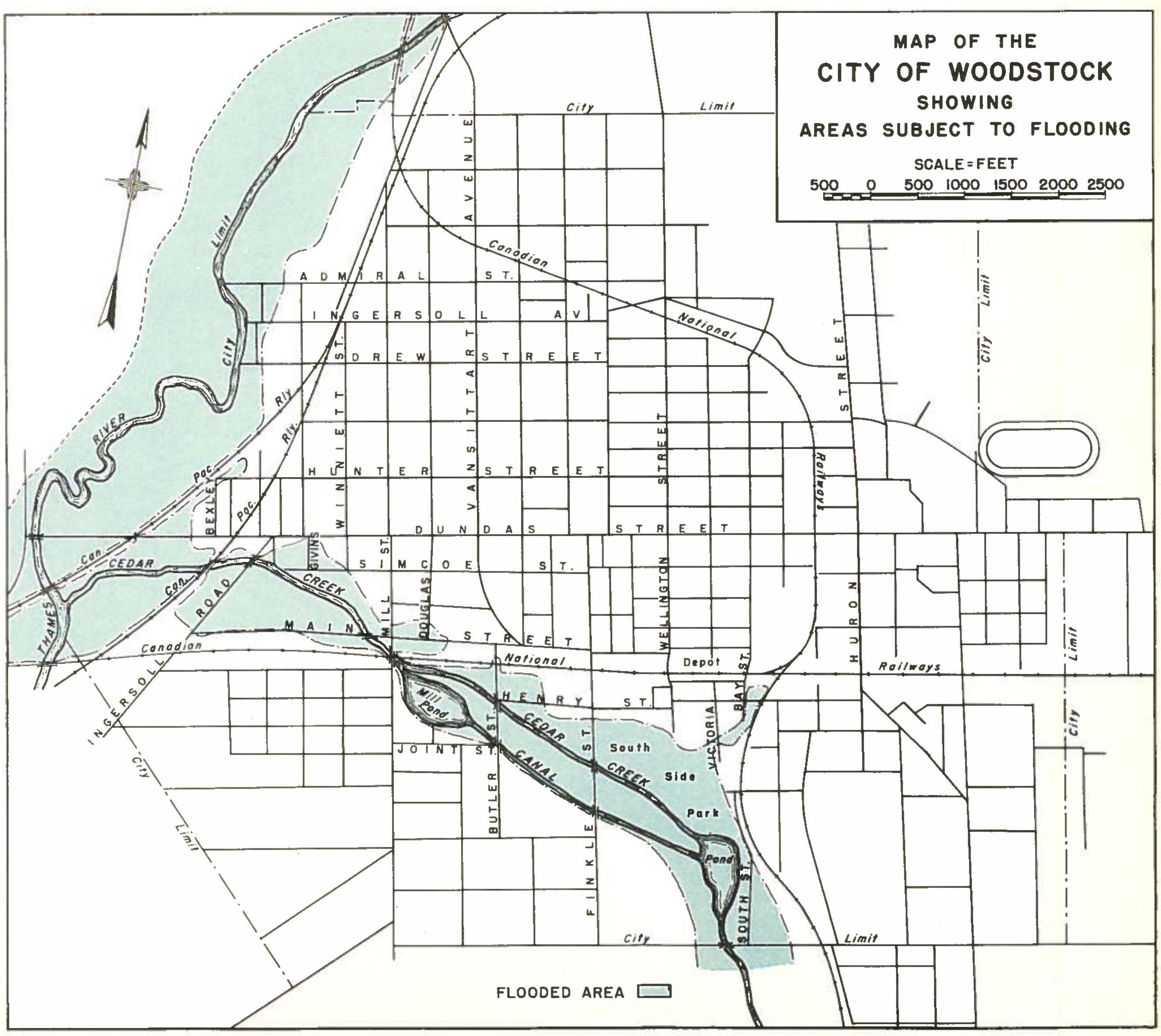
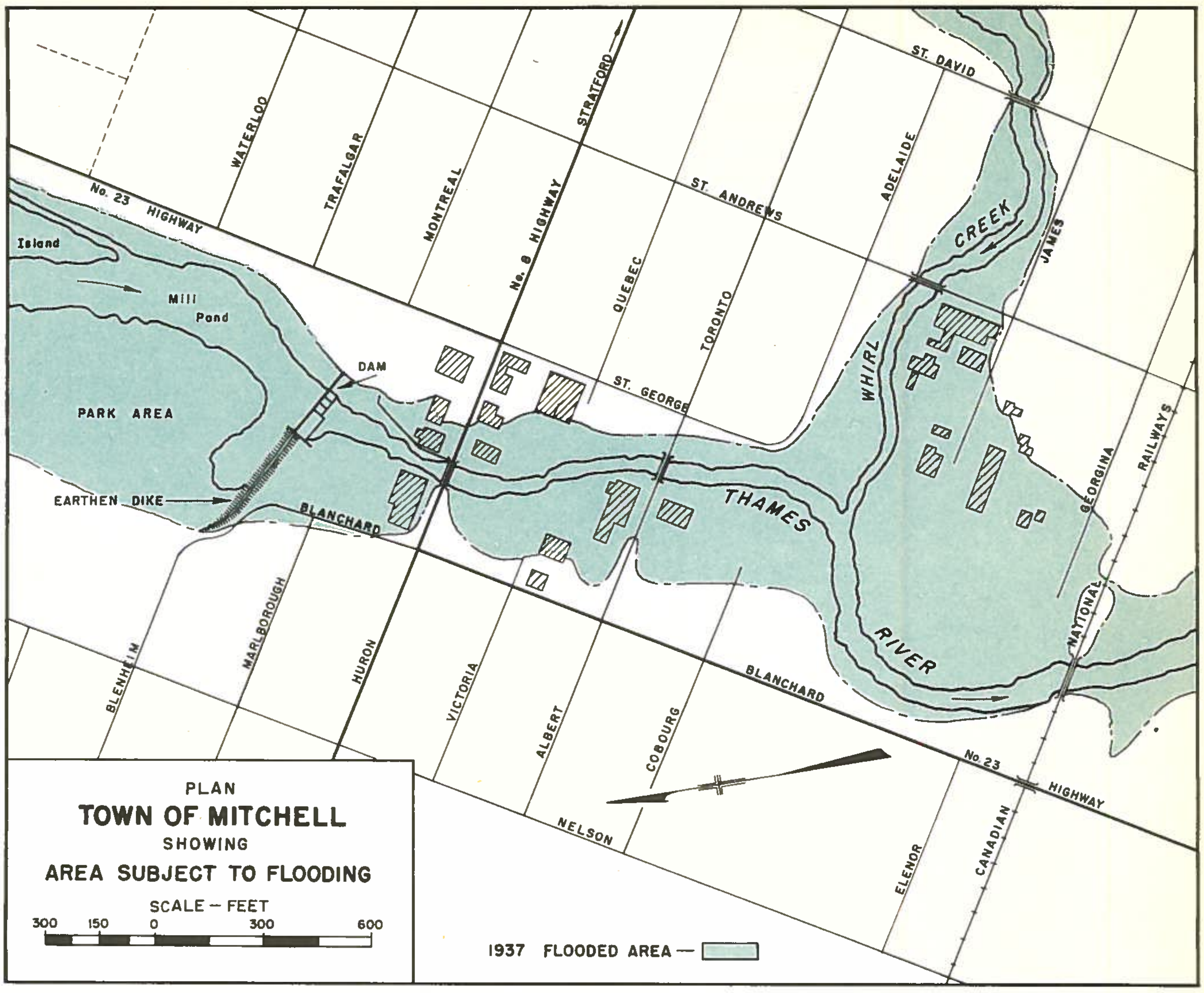

MAP OF THE
CITY OF WOODSTOCK
SHOWING
AREAS SUBJECT TO FLOODING
SCALE = FEET
500 0 500 1000 1500 2000 2500
FLOODED AREA
City Limit
THAMES RIVER
CEDAR CREEK
CEDAR CANAL
Mill Pond
Pond
South Side Park
Canadian National Railways
Can. Pac. Rly
ADMIRAL ST.
INGERSOLL AV
DREW STREET
HUNTER STREET
DUNDAS STREET
SIMCOE ST.
MAIN STREET
HENRY ST.
JOINT ST.
INGERSOLL ROAD
BEXLEY
GIVINS
MILL ST.
DOUGLAS
WINNIETT ST.
VANSITTART AVENUE
WELLINGTON STREET
BUTLER ST.
FINKLE ST.
VICTORIA
BAY ST.
SOUTH ST.
HURON STREET
Depot

FIG. H-3


PLAN
TOWN OF MITCHELL
SHOWING
AREA SUBJECT TO FLOODING
SCALE — FEET
300 150 0 300 600
1937 FLOODED AREA —
WATERLOO
TRAFALGAR
MONTREAL
No. 8 HIGHWAY
STRATFORD
ST. DAVID
ADELAIDE
ST. ANDREWS
No. 23 HIGHWAY
Island
Mill Pond
PARK AREA
DAM
EARTHEN DIKE
QUEBEC
TORONTO
ST. GEORGE
WHIRL CREEK
JAMES
THAMES
RIVER
GEORGINA
RAILWAYS
NATIONAL
CANADIAN
BLANCHARD
BLENHEIM
MARLBOROUGH
HURON
VICTORIA
ALBERT
COBOURG
NELSON
ELENOR
No. 23 HIGHWAY

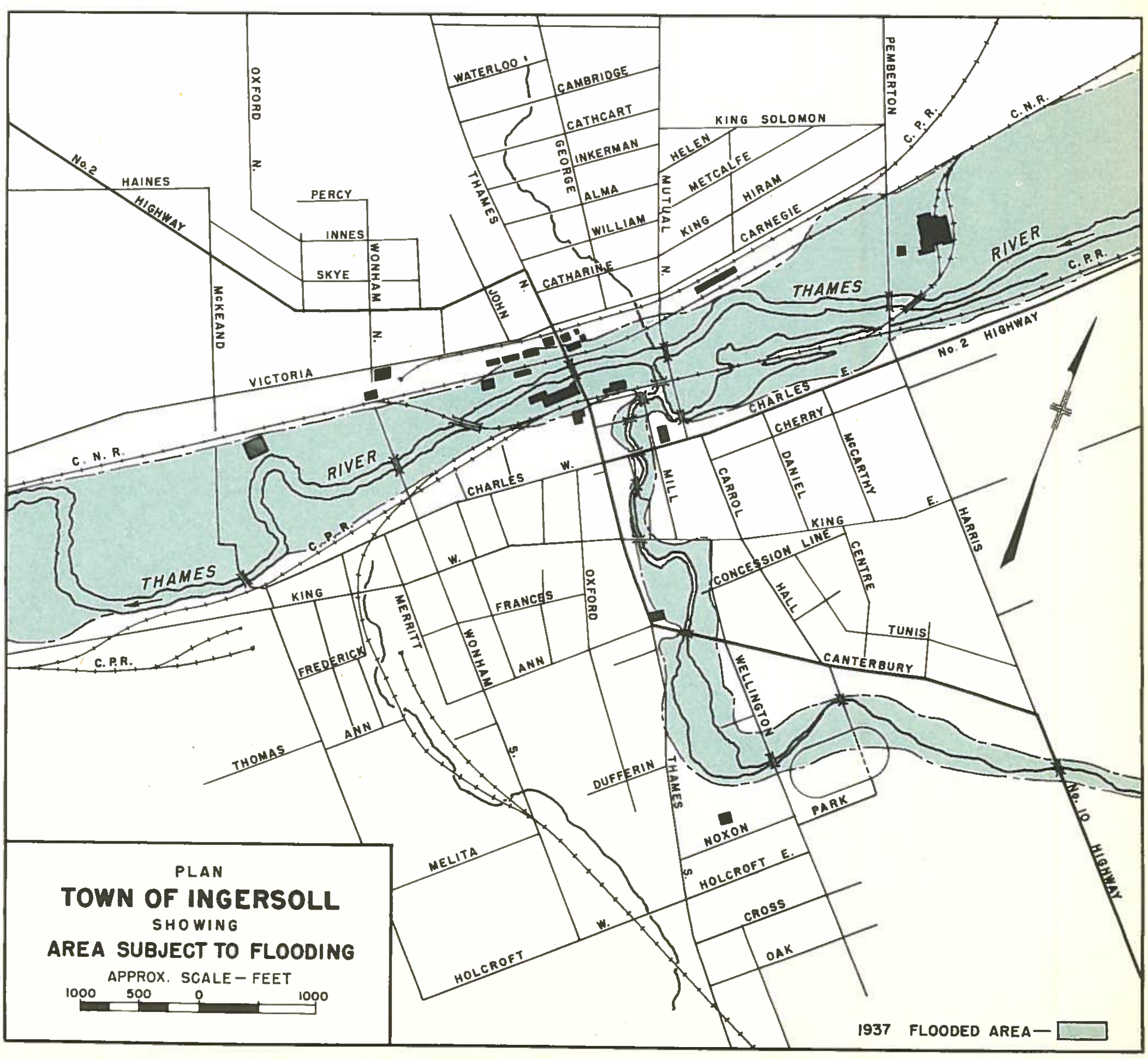

PLAN
TOWN OF INGERSOLL
SHOWING
AREA SUBJECT TO FLOODING
APPROX. SCALE — FEET
1000 500 0 1000
1937 FLOODED AREA —
OXFORD N.
WATERLOO
CAMBRIDGE
CATHCART
GEORGE
INKERMAN
ALMA
WILLIAM
THAMES N.
KING SOLOMON
HELEN
METCALFE
HIRAM
KING
CARNEGIE
MUTUAL N.
PEMBERTON
C.P.R.
C.N.R.
No. 2 HIGHWAY
HAINES
PERCY
INNES
SKYE
WONHAM N.
JOHN
CATHARINE
McKEAND
VICTORIA
THAMES
RIVER
No. 2 HIGHWAY
CHARLES E.
CHERRY
DANIEL
McCARTHY
C. N. R.
RIVER
CHARLES W.
MILL
CARROL
KING E.
HARRIS
CONCESSION LINE
CENTRE
HALL
THAMES
C.P.R.
KING
MERRITT
W.
FRANCES
OXFORD
TUNIS
CANTERBURY
FREDERICK
WONHAM
ANN
WELLINGTON
ANN
THOMAS
S.
DUFFERIN
THAMES S.
NOXON
PARK
No. 10 HIGHWAY
MELITA
HOLCROFT E.
CROSS
OAK
HOLCROFT W.

FIG. H-4

3. The Solution of the Flood Problem

(a) Future Floods

The flood which caused the most damage, at least in recent years, occurred in April 1937. The exceptional flood flows on the South Branch were an all-time high, but on the North Branch the highest flood flows actually occurred in 1947 and would have caused greater damage in London had the South Branch been correspondingly high and had not part of the dike system been raised and extended after the 1937 flood. From these floods it will be seen that, in order to give protection for the future, provision must be made not only for floods of known magnitude but for greater ones which in all likelihood will occur during the years ahead.

It is difficult to state with accuracy what these greater floods might be, but in planning protection for the municipalities of the Upper Thames it has been considered sufficient to provide for protection equal to one-third more than the greatest known flood of the past, namely the flood of the South Branch in 1937 and the flood of the North Branch in 1947 if they should occur together, which they might well do, at some future date. This was the factor of safety provided for by the engineers who prepared the official plan of the Muskingum Conservancy District. Such a probable flood of the future is termed an hypothetical flood and will be referred to as such hereafter in this report.

(b) Storage Required

The flood control storage required for the hypothetical spring flood for the North Branch is 80,734 acre feet[1] and for the South Branch, 30,346 acre feet. The total for the Forks at London is 111,080 acre feet. The plan is designed for such operation that all the reservoirs would be filled to spillway level but not beyond, in the case of a spring run-off

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1. An acre foot is a volume one acre in area by one foot in depth and is equivalent to 43,560 cubic feet.

with a magnitude equal to the hypothetical flood. During such a flood the outflow from the reservoirs, including the run-off from the uncontrolled areas below the reservoirs, would not exceed the safe carrying capacity of the channels below. Should the hypothetical flood be exceeded in magnitude, then the reservoirs would have to discharge the flood waters at a greater rate than planned and there would be some flooding.

(c) Distribution of Storage (Fig. H-5)

The distribution of the 111,080 acre feet of storage should be as near as possible to the ratios of the spring run-off volumes of the tributaries above London. This ratio is 73.77 per cent or 81,944 acre feet for the North Branch and 26.23 per cent or 29,136 acre feet for the South Branch.

The North Branch is well provided with reservoir sites. The chosen sites are well located strategically and their storage capacity is in fair proportion to their respective drainage areas. The South Branch, however, is not so well favoured with reservoir sites. The Thamesford site on the Middle Branch[1] is the only one which has a capacity comparable to those of the North Branch. The other two sites are small and alone are inadequate for the protection of Woodstock. Also over half of the South Branch drainage area is uncontrolled. There is, however, sufficient storage in the South Branch, as the amounts below indicate.

Reservoir storage available in North Branch	80,734 ac. ft.
Reservoir storage available in South Branch	30,346 ac. ft.
Total	111,080 ac. ft.

The uncontrolled area of the North Branch is 97 square miles or 14.76 per cent of the North Branch drainage area. The uncontrolled area of the South Branch is 277.9 square miles or 53.55 per cent of the South Branch drainage area.

Table H-1 shows the storage capacity for each of the reservoirs surveyed and the distribution of the storage relative to London, St. Marys and Woodstock.

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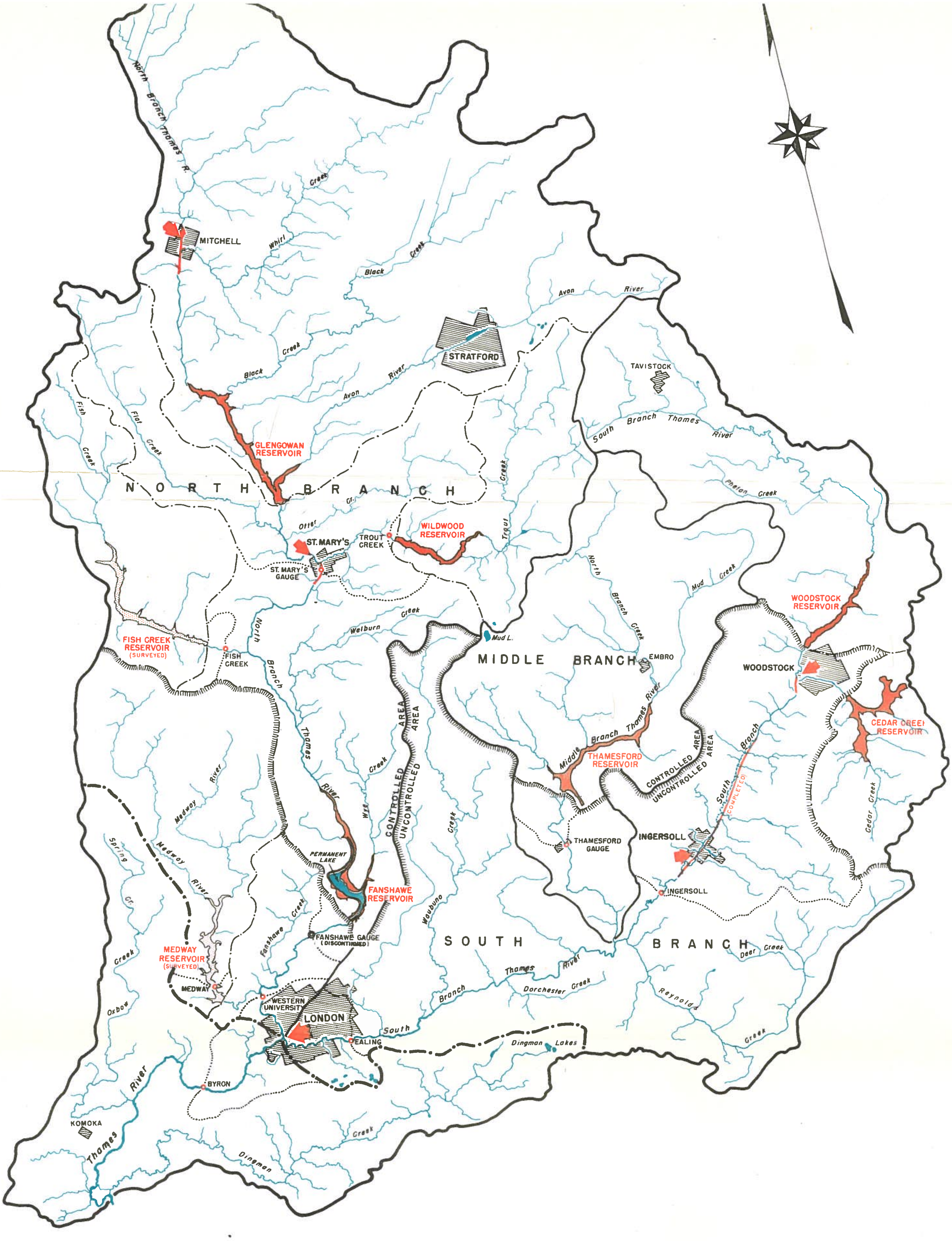
1. A tributary of the South Branch.



UPPER THAMES RIVER WATERSHED

SHOWING

PROPOSED CONSERVATION RESERVOIRS

CHANNEL IMPROVEMENTS

AREAS SUBJECT TO FLOODING

GAUGING STATIONS

AND

DRAINAGE AREAS

SCALE – MILES

2 1 0 2 4 6 8 10

LEGEND

RESERVOIRS
FLOODED AREAS
GAUGING STATION
CHANNEL IMPROVEMENT

DRAINAGE AREAS: ABOVE LONDON; BRANCH; RESERVOIR; ABOVE GAUGE

FIG. H-5

(d) Low and Increased Sustained Flows

Reservoirs which are used for the storage of water for summer flow, as well as for flood control, may be regulated for the summer months only, which would yield more flow during this shorter period; or, more preferably, extended over the balance of the year in order to dilute the effluent from domestic sewage and industrial waste. May is usually a wet month and the reservoirs would be full on June 1, and about that time storage would be used to increase low flows. June 1 to September 20 - 112 days - constitutes a summer period. June 1 to March 1 - 273 days - covers a yearly period, March 1 being about the time of the break-up, when the reservoirs would shortly be filled again.

From Table H-2 it will be seen that the summer of 1939 and the winter of 1940 was the driest yearly period since 1915. The driest summer period was in 1936, the mean monthly flows at London on the North Branch being only 12 c.f.s. in August and on the South Branch being only 22 c.f.s. in July. The mean daily flows were even lower, being 9 c.f.s. for 9 days of the month on the North Branch and 12 c.f.s. for 4 days of the month on the South Branch. Conservation reservoirs will correct this adverse condition, as may be seen in Table H-A below and in Tables H-3 and H-4, which account for the conservation storage. In examining these tables it should be noted that all reservoirs may be used for summer flow except Fanshawe.

It may be pointed out here that the Thames below London will benefit not only by the increased flow but also from the greatly reduced flood crests as well. The proposed reservoir system will control the run-off from 802 square miles or approximately 36% of the entire Thames Watershed area and would be invaluable at such times when the lower Thames River alone was in flood, as in January, 1951. Under these conditions the flow from above London could be cut off completely and held until the flood danger below has passed.

TABLE H-A

SUSTAINED FLOWS AT LONDON BY THE REGULATION OF CONSERVATION RESERVOIRS - (FANSHAWE EXCEPTED)

Branch	Period of Days	Average Daily Discharge from Reservoirs - Conservation Storage Only		Sustained Flows at London	
		Driest Year on Record c.f.s.	Average Year c.f.s.	Driest Year on Record c.f.s	Average Year c.f.s.
North South	112 112	150.2 87.6	152.4 89.1	201.3 161.5	325.5 277.5
North South	273 273	60.0 33.8	61.3 33.9	128.4 104.2	455.6 263.4

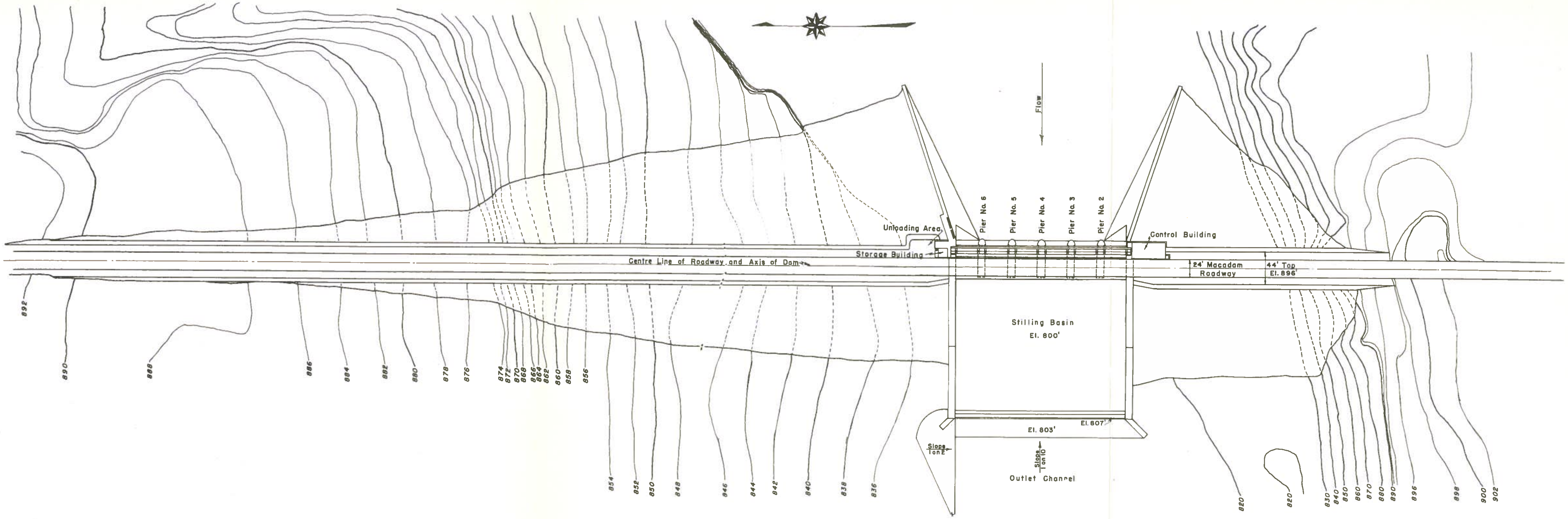
(e) Proposed Dams and Reservoirs

(1) Fanshawe Dam and Reservoir (Fig. H-6)

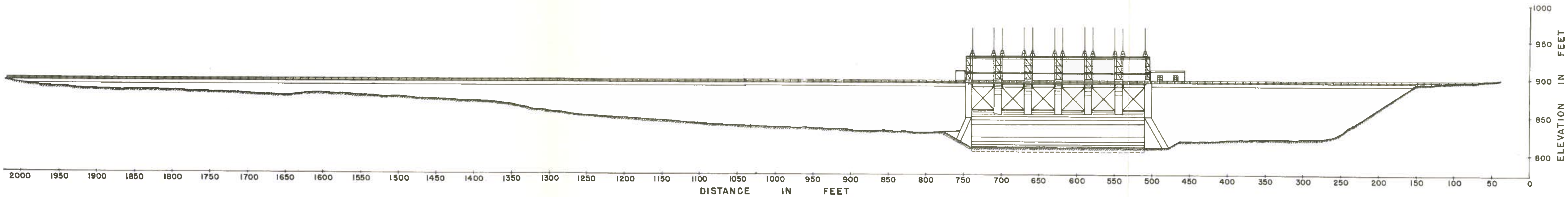
a General Description

The Fanshawe damsite is located on the North Branch of the Thames 5½ miles north of the city of London. The damsite area and reservoir limits were surveyed by the Department in 1945 and the construction of the dam and reservoir was recommended in the preliminary report. Subsequently the firm of H. G. Acres and Company was retained to prepare the plans and the dam is now under construction.

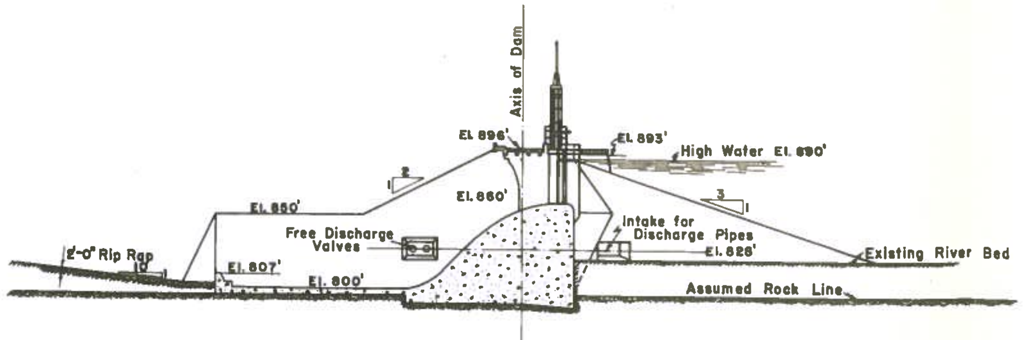
The dam will be of the gravity earth-fill and concrete type. The concrete spillway section will be 250 feet long, 100 feet high (above bedrock) and 115 feet wide at the base. The spillway will be fitted with two 6-foot diameter controlled discharge tubes and six 30 feet by 30 feet spillway gates and have a discharge capacity in excess of 100,000 c.f.s. at maximum water level. In addition the necessary water intake tubes and pumping equipment will be installed in order that the reservoir may be used for domestic water supplies for the city of London.

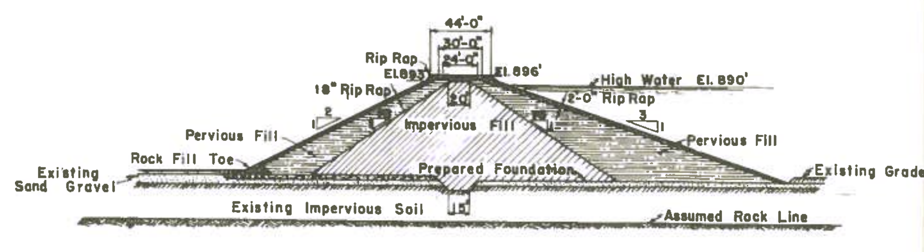


GENERAL PLAN OF EARTH F LL DAM & SPILLWAYS



DOWNSTREAM ELEVATION OF EARTH FILL DAM & SPILLWAYS





FANSHAWE DAM
PLAN, PROFILE AND TYPICAL SECTIONS

SCALE — FEET

50 0 50 100 150

▲ FANSHAWE DAMSITE, *Summer 1947—looking upstream from road between Cons. III and IV London Twp.*

▼ GLENGOWAN DAMSITE—*from bridge on Lot 7 Con. XV Blanshard Twp. Broken white line in this and following photographs indicates (approx.) the crest of the proposed dam.*

The earthen embankments which will join the concrete section to the valley slopes and complete the barrier will have a minimum top width of 44 feet with 3 to 1 and 2 to 1 slopes on the upstream and downstream sides respectively. Total length of the completed structure will be 2,050 feet.

The reservoir when full (elevation 890 feet) will have a water surface area of 1,322 acres, extend northerly for approximately 7 miles, with a maximum depth of 71 feet at the dam and a storage capacity of 38,880 acre feet. At the proposed recreational lake level (elevation 860 feet) the reservoir will extend northerly for a distance of 4.4 miles, with an average width of 1/4 mile and a maximum width of 1/2 mile just north of the dam. The water surface area will be 645 acres and it will have a volume of 10,040 acre feet.

A plan showing the general arrangement of the dam is shown in Figure H-6. The estimated cost of the dam and reservoir including a 24-foot roadway along the top of the dam and the development of the supplementary recreation facilities is $4,711,250.

b The effect of its use as a recreational lake

When a reservoir is used as a recreational lake there is no storage retained for increasing low flows. The water level in the reservoir would be lowered to the fixed lake level as soon as possible after the flood period in order to preserve vegetation, and regulated approximately at that level throughout the following months; otherwise wide fluctuations in water levels would not satisfactorily serve its recreational purpose. The Fanshawe Reservoir therefore will make no contribution in increasing low flows.

(2) Glengowan Dam and Reservoir

The Glengowan site is located on the North Branch three miles north of the town of St. Marys. The survey of the reservoir area and foundation investigations have been carried out and a design of the dam prepared by G. Graham Reid, Consulting Engineer.

The dam will be the same type as the Fanshawe Dam, consisting of a concrete spillway section and earthen embankments. The spillway section will be 280 feet long and will be fitted with five sluiceways and six Taintor type gates each 8 feet high by 40 feet in length. The over-all length of the dam will be 1,200 feet with a maximum height of 64 feet above the bed of the river.

The reservoir, when full, will have a maximum depth at the dam of 59 feet, and the lake would extend northerly for a distance of about 8.7 miles with an average width of about 1/4 mile and a water surface area of 1,195 acres. With a reservoir capacity of nearly 27,000 acre feet it will provide protection to St. Marys for most flood years and would benefit all the municipalities down stream by reducing the high spring flows and providing increased summer flow.

The total estimated cost of this dam and reservoir, including a roadway over the dam, is $2,020,000.

(3) <u>Wildwood Dam and Reservoir</u>

This reservoir is located on Trout Creek near Highway No. 7. The foundation investigations have been completed and a design for the dam prepared by G. Graham Reid, Consulting Engineer. However, since this work was done subsequent calculations and further studies have indicated that a higher dam to provide greater storage is required at this point to give the needed protection for St. Marys and points below.

The proposed dam would be of the same design as that prepared by G. Graham Reid, but the height would be increased from 41.0 feet to a height of 53.0 feet above the bed of the stream. The reservoir for this higher dam would extend south-easterly from the dam for a distance of 6.2 miles with an average width of 1,170 feet and a surface area of 880 acres. The maximum holding capacity of this reservoir would be 14,900 acre feet as compared to 6,400 acre feet for the original dam.

WILDWOOD DAMSITE—*looking upstream from No. 7 Highway 2 miles east of St. Marys.*

THAMESFORD DAMSITE—*panorama of the proposed site 2 miles north of Thamesford Lot 5 Con. XI Nissouri E. Township.*

This dam would also be of the earth-fill and concrete type and would be fitted with five gate-controlled rectangular orifices and 108 feet of free overflow section to provide satisfactory regulation of the discharge from the dam at all times.

The estimated cost for the dam and reservoir is $1,407,000.

(4) Thamesford Dam and Reservoir

The Thamesford Reservoir site is located on the Middle Branch two miles north of the village of Thamesford. The damsite for this reservoir would roughly coincide with the line between Lots 4 and 5, Concession XI in the Township of Nissouri East.

This site is suitable for a dam 49.5 feet in height which would provide 17,500 acre feet of storage. Such a dam would create an artificial lake 6.4 miles long with an average width of 1,550 feet, a surface area of 1,200 acres and a maximum depth of water at the dam of 43.5 feet.

Several bridges and road crossings would be flooded out by this proposed dam, but this site is the only one on the South or Middle Branches which can provide a reservoir of appreciable size and the site should therefore be developed to its maximum for flood control and conservation purposes.

No foundation investigations have been carried out at this site as yet, but the estimated cost of the dam and reservoir based on an assumed depth to rock of 25 feet is $2,440,000.

(5) Woodstock Dam and Reservoir

This dam and reservoir would be located on the South Branch immediately north of the city of Woodstock. The damsite would be located about 1/2 mile east of No. 19 Highway between Lots 20 and 21, Concession XII, Zorra East Township. This would be a low dam 31 feet in height, being limited by the Canadian Pacific Railway which skirts the southerly side of the proposed reservoir area. When full the reservoir would extend north-easterly from the dam for a distance of 4.5 miles, with a width of about 1,050 feet, a surface area of 575 acres and a maximum water depth at the dam of 25 feet.

The capacity of this reservoir would be 5,152 acre feet, and being situated so closely to the trouble area it could be used to full advantage during the critical periods of high flow.

This reservoir is ideally located for a recreational lake and if it should be required for this purpose at some future date the conservation storage so lost could be replaced by constructing one or two small dams farther up stream.

The estimated cost for this dam and reservoir is $760,000.

(6) <u>Cedar Creek Dam and Reservoir</u>

The dam for this reservoir would be located on the Cedar Creek 1 1/2 miles south of the city of Woodstock and would be the smallest dam in the proposed reservoir system. The dam would be of the earth-fill and concrete type 27.5 feet high and 1,100 feet long and would control the run-off from some 31 square miles of drainage area lying south of Woodstock.

The reservoir, at full capacity, would have a water surface area of 1,460 acres and would be the largest of any of the proposed reservoirs in this respect. Actually it would be two separate lakes joined together by narrow channels a short distance above the dam. The southerly lake or arm would extend back from the dam a distance of 3 miles with a water surface area of 600 acres, and the easterly lake or arm would extend eastward from its confluence with the southerly arm a distance of about 2.5 miles, with a water surface area of some 860 acres. A large part of the flooded lands is swamp which has little value except for storing water.

WOODSTOCK DAMSITE—*panorama of the proposed site immediately north of the city, Lot 21 Con. II Blandford Township.*

CEDAR CREEK DAMSITE—*looking downstream from bridge on road between Cons. II and III Oxford E. Township.*

This reservoir would have a storage capacity of 7,728 acre feet with a maximum depth of water at the dam of 21.5 feet. With this amount of storage this project, in conjunction with the Woodstock Reservoir and Channel Improvement, would provide the needed protection for Woodstock and also provide a valuable increase for the much needed summer flows in the South Branch. Further, this is one of the few natural sites available on the South Branch and steps should be taken immediately to obtain these lands before the area becomes too built-up and the land values make the construction of a reservoir in this area too costly.

The estimated cost of this dam and reservoir is $604,000 or approximately $78.00 per acre foot, which is the lowest per acre foot cost of any reservoir on the watershed.

(f) Other Dams and Reservoirs

Eight reservoir areas were surveyed in the watershed, of which six have been selected to make up the reservoir system for flood control and water conservation storage. The two remaining sites, namely Medway and Fish Creek, are good sites and are the only remaining ones where sufficient storage capacity may be had to make the construction of a dam economically feasible. These sites should be held in reserve in the event that some unforeseen conditions make the provision of additional storage in the system advisable or necessary.

(1) Medway Dam and Reservoir

The Medway damsite is located on the Medway River about 3 miles north-west of the city of London. The Medway drains an area of approximately 75 square miles of which about 73 square miles could be controlled by a dam at this site.

A 76-foot dam could be constructed at this point to impound about 21,500 acre feet of water, which would provide useful flood control for the city of London. The reservoir is located too far down the watershed to be used as a dual-purpose reservoir and the cost cannot be justified by flood control alone at the present time.

(2) Fish Creek Dam and Reservoir

The Fish Creek damsite is located on Fish Creek 5½ miles south-west of the town of St. Marys.

This site is suitable for a 41-foot dam which would impound 11,500 acre feet of water and control the run-off from 58 square miles of drainage area. This amount of storage would be most useful should one of the reservoirs above St. Marys be required for a recreational lake or should additional summer flow be required on the North Branch.

Table H-B below gives the data and costs for all the dams and reservoirs included in the proposed scheme, together with the dam and reservoir data for the above two reservoirs which were surveyed but which are not included in the reservoir system. Additional dam and reservoir data is given in Table H-5 at the back of this section of the report.

TABLE H-B

DAM AND RESERVOIR DATA

Name	Dam		Reservoir				Cost
	Length (Ft.)	Height (Ft.)	Length (Miles)	Av. Width (Ft.)	Area (Acres)	Capacity (Ac. Ft.)	$
Glengowan	1,200	64.0	8.7	1,132	1,195	26,954	2,020,000
Wildwood	1,790	50.5	6.2	1,170	880	14,900	1,407,000
Fish Creek x	980	41.0	7.7	1,054	984	11,753	-
Fanshawe	2,050	77.0	7.6	1,469	1,351	38,880	4,711,250
Medway x	950	76.0	5.2	1,560	983	21,507	-
Woodstock	1,440	31.0	4.5	1,052	575	5,152	760,000
Cedar Cr.	1,100	27.5	5.5	2,185	1,459	7,728	604,000
Thamesford	1,200	49.5	6.4	1,548	1,200	17,466	2,440,000
Totals						111,080	11,942,250

x Surveyed but not imcluded in scheme.

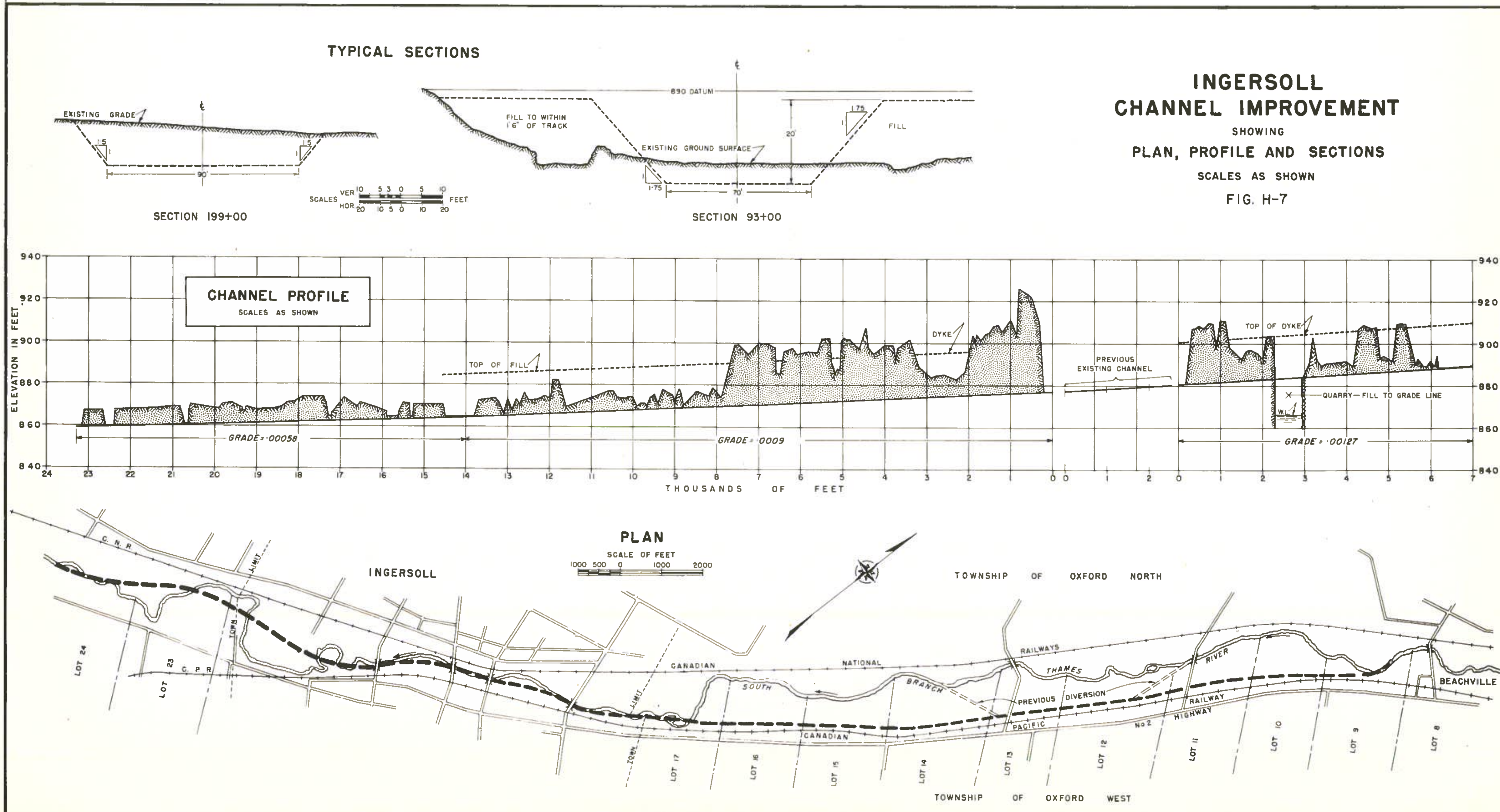
(g) Channel Improvements

(1) Ingersoll Channel Improvement Scheme (Fig. H-7)

In addition to the two conservation reservoirs which are proposed for the South Branch of the Thames River, an extensive channel improvement scheme designed and supervised by the firm of G. Graham Reid, Consulting Engineers, has already been completed. This is located in the Beachville-Ingersoll area and is known as the Ingersoll Channel Improvement Scheme. This work was done to provide immediate flood protection for the town of Ingersoll and the industrial plants and quarries located in the river valley above Ingersoll.

Starting at the village of Beachville the improved channel section parallels the Canadian Pacific Railway line along the southerly side of the valley to the easterly limit of Ingersoll. From this point the new channel follows, in general, the natural river course except for the wide loops below Ingersoll which it cuts through. The length of the new channel is 32,725 feet, as compared to 39,640 feet, the overall distance by the old channel. The new channel has a bottom width of 60 feet for the upper 14,000 feet, then a 70-foot bottom width for 6,500 feet and an 80-foot bottom width for the next 3,300 feet, increasing to a 90-foot bottom width for the remaining 8,925 feet. The side slopes are 1.75 to 1 vertical for the upper and central sections and 1.5 to 1 vertical in the lower part. The grade varies throughout, being 0.127 per cent for the upper part, 0.09 per cent for the central part and 0.058 per cent for the lower part.

Construction of this channel required the excavation of some 1,612,000 cubic yards of earth and about 26,000 cubic yards of rock. Most of the earth was used to raise the height of the banks and the stone for rip-rapping the side slopes. The consolidated earthen dikes built from the waste material have a minimum height of 20 feet above the bed of the channel with a minimum top width of 12 feet.



INGERSOLL CHANNEL IMPROVEMENT

SHOWING

PLAN, PROFILE AND SECTIONS

SCALES AS SHOWN

FIG. H-7

—London Free Press

INGERSOLL CHANNEL IMPROVEMENT

Aerial view of the improved channel through Ingersoll taken from the north-west. Bridge at centre of photograph is the Thames Street Bridge.

The greatest known flow in this area occurred in April 1937. Unfortunately there was no gauge established at Ingersoll to record this discharge, but from the Ealing gauge it has been calculated that the maximum mean daily flow at Ingersoll during this flood was about 7,400 c.f.s. with a probable peak flow of 10,400 c.f.s. A stream gauge was established at Ingersoll in 1938 and flood discharges have been recorded since then. High flows for each year are shown below:

Ingersoll Gauge Readings 1938-50

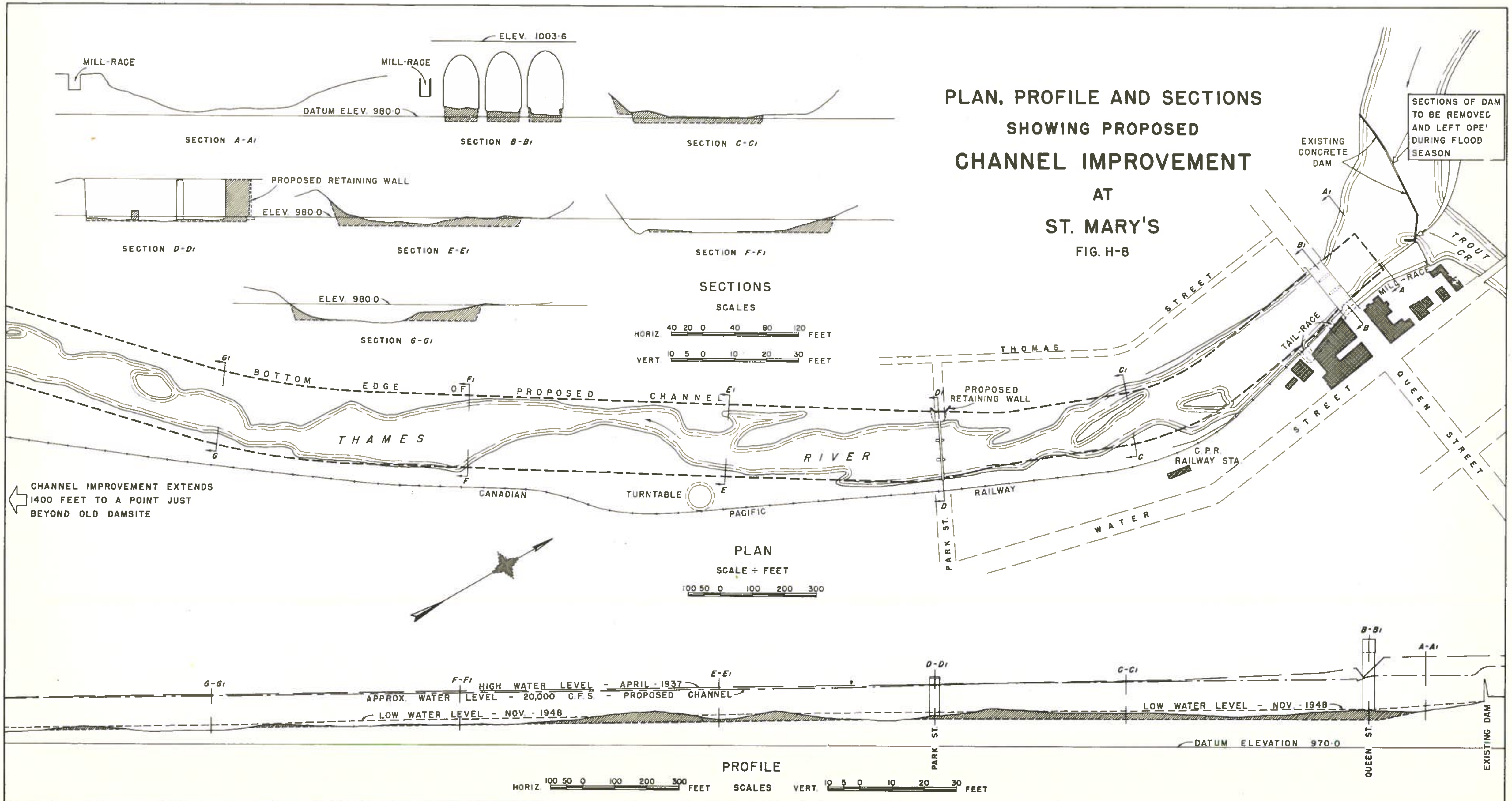
Date	Max. Mean Daily c.f.s.	Probable Peak Flow c.f.s.
1937 April (estimated)	7,400	10,400
1938 February 14	2,230	3,122
1939 April 19	1,810	2,534
1940 April 9	3,030	4,244
1941 April 6	403	564
1942 March 9	2,280	3,192
1943 February 24	2,500	3,500
1944 March 14	1,260	1,764
1945 March 7	1,420	1,988
1946 March 8	2,030	2,842
1947 April 6	3,460	4,844
1948 March 20	2,890	4,046
1949 February 15	2,700	3,780
1950 April 4	2,790	3,906

The new channel was designed to carry a flow of 8,000 c.f.s. and will safely discharge momentary peak flows as high as 11,750 c.f.s., which provides protection against such floods as have occurred in the past. However, the present flow capacity of the Thames Street Bridge in Ingersoll is only 8,650 c.f.s. and in the case of flows such as were experienced in April 1937 the water would be backed up for a short time at the height of the flood flow. However, this channel alone will provide sufficient flood protection for most years and, together with the two proposed conservation reservoirs, will provide for floods of the hypothetical magnitude.

The cost of the work, including a new bridge, was $1,000,000.

(2) St. Marys Channel Improvement Scheme (Fig. H-8)

The town of St. Marys is situated on the North Branch of the Thames at the confluence of Trout Creek 22 miles


PLAN, PROFILE AND SECTIONS
SHOWING PROPOSED
CHANNEL IMPROVEMENT
AT
ST. MARY'S
FIG. H-8
MILL-RACE
ELEV. 1003·6
DATUM ELEV. 980·0
SECTION A-A1
SECTION B-B1
SECTION C-C1
PROPOSED RETAINING WALL
ELEV. 980·0
SECTION D-D1
SECTION E-E1
SECTION F-F1
SECTION G-G1
SECTIONS
SCALES
HORIZ 40 20 0 40 80 120 FEET
VERT 10 5 0 10 20 30 FEET
EXISTING CONCRETE DAM
SECTIONS OF DAM TO BE REMOVED AND LEFT OPE' DURING FLOOD SEASON
TROUT CR
MILL-RACE
TAIL-RACE
QUEEN STREET
STREET
THOMAS
PROPOSED RETAINING WALL
BOTTOM EDGE OF PROPOSED CHANNEL
THAMES
RIVER
C.P.R. RAILWAY STA
WATER STREET
PARK ST.
CANADIAN PACIFIC RAILWAY
TURNTABLE
CHANNEL IMPROVEMENT EXTENDS 1400 FEET TO A POINT JUST BEYOND OLD DAMSITE
PLAN
SCALE ÷ FEET
100 50 0 100 200 300
HIGH WATER LEVEL - APRIL - 1937
APPROX. WATER LEVEL - 20,000 C.F.S. - PROPOSED CHANNEL
LOW WATER LEVEL - NOV. - 1948
DATUM ELEVATION 970·0
EXISTING DAM
QUEEN ST.
PROFILE
HORIZ. 100 50 0 100 200 300 FEET
SCALES
VERT. 10 5 0 10 20 30 FEET

north of the city of London. The combined watersheds of the two streams have a drainage area of 416 square miles at this point, being comprised of 352 square miles for the North Branch and 64 square miles for Trout Creek. The area above St. Marys is flat and intensively drained. There is very little forest cover and these factors together with the steep stream gradients all contribute to a quick run-off from the area and the consequent fluctuations in the flows at St. Marys.

A gauge was established at the Park Street bridge in St. Marys in 1938 and the high spring flows have been recorded for each year since. The highest mean daily flow recorded was 11,800 c.f.s. on March 20, 1948. The peak flow on that day from timed records was 18,668 c.f.s. However, many local residents claim that the 1937 spring flood was greater than that of 1948. This flood occurred before the gauging station was established and the daily flows during the flood period were not recorded. From the Fanshawe gauge records it has been estimated that maximum mean daily flow for the 1937 flood was 15,220 c.f.s. with a corresponding peak flow of 20,240 c.f.s. In the case of a flood of the hypothetical magnitude a maximum daily flow of 23,632 c.f.s. with a corresponding peak flow of 31,500 c.f.s. might be expected.

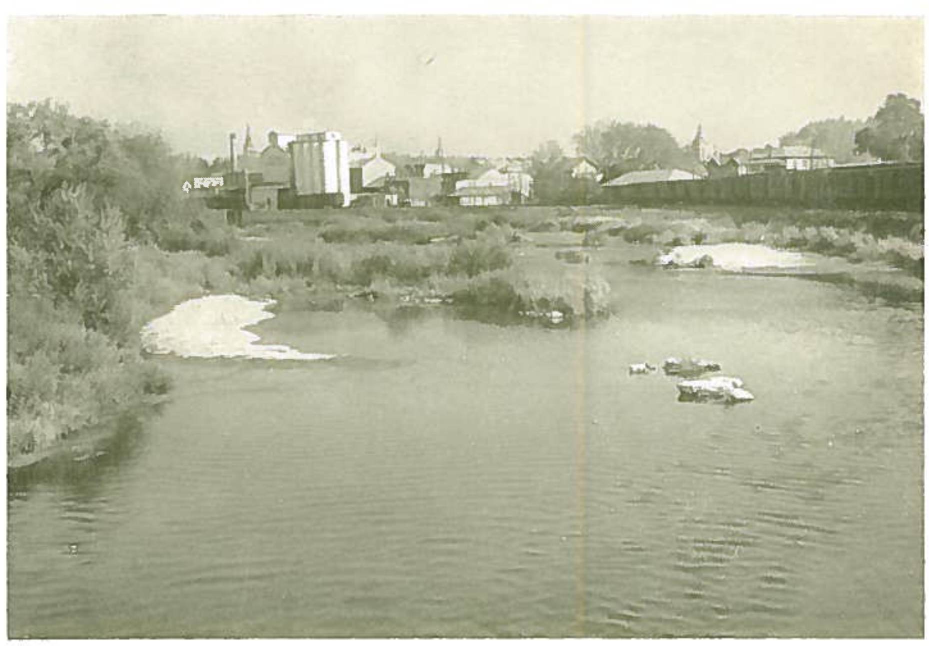
The overall water conservation plan for the Upper Thames Watershed provides for the construction of two storage reservoirs above St. Marys (Glengowan and Wildwood) with a combined storage capacity of 41,854 acre feet of which 38,854 acre feet would be available for flood control purposes. This amount of storage would provide protection from floods such as have occurred in the past but would not be sufficient for the hypothetical flood; consequently some channel improvement would be necessary to supplement the storage and satisfy the flood problem.

It is estimated that the present channel through St. Marys can only accommodate flows up to 10,400 c.f.s. and still provide a margin of safety. To reduce the hypothetical

St. Marys Dam—present dam has not sufficient spillway capacity to discharge the high spring flows safely.

Queen St. Bridge—limited opening at this bridge further aggravates the flood conditions.

River channel north of Park St.—shoals and weedy condition of the stream bed also tend to impede the normal passage of the spring flows.



maximum mean daily flow to this stage would require 57,300 acre feet of storage or 18,400 acre feet more than is available in the proposed Glengowan and Wildwood reservoirs. Thus to give full protection to St. Marys additional storage or an increased channel capacity is necessary. The additional storage could be made available by constructing a third dam above St. Marys, but in view of the fact that the present proposed storage is sufficient for the overall flood and low flow problems it is believed that the channel improvement offers the better solution. Further, in order to make full use of the channel capacity at London and to be able to route the flood waters to the best advantage, St. Marys should be able to discharge its proportion of channel capacity flow or at least 12,100 c.f.s. This flow is 1,700 c.f.s. more than the present channel can safely handle and thus some channel work would be required in any case.

Flood relief for St. Marys is an urgent matter and, as it would probably be some time before the reservoir system would be in operation, it is recommended that the channel improvement work be extended to provide protection against floods up to the magnitude of the 1937 flood or a flow of 20,000 c.f.s. However, in recommending this amount of channel improvement two facts have been kept in mind. Firstly, that this work is only a temporary measure to provide some immediate relief for St. Marys and the amount of work has therefore been kept to a minimum wherever possible in order to avoid unnecessary expenditures. Secondly, that the pond behind the present dam has a certain aesthetic and recreational value and the proposed work has been designed to leave most of the dam intact so that the pond may be restored at a minimum cost.

The estimated cost for this channel improvement is $135,000.

This cost does not provide for restoring the pond above the dam for recreational or power purposes. If the pond is wanted for the summer months then removable timber dams could be constructed across the openings to raise the water to the height of the remaining overflow section and create a pond equal to the present pond. Such a dam would be removed in the fall and then replaced in the spring after the freshet had passed and thus would not be available for power during the winter months.

The estimated cost for the above channel improvement including the timber dams sections is $143,300.

If water power is required throughout the year then it will be necessary to replace the present dam with a whole new structure. The dam would have to be fitted with sufficient gates to provide a discharge capacity of 20,000 c.f.s. without raising the water level above the dam to the flood stage.

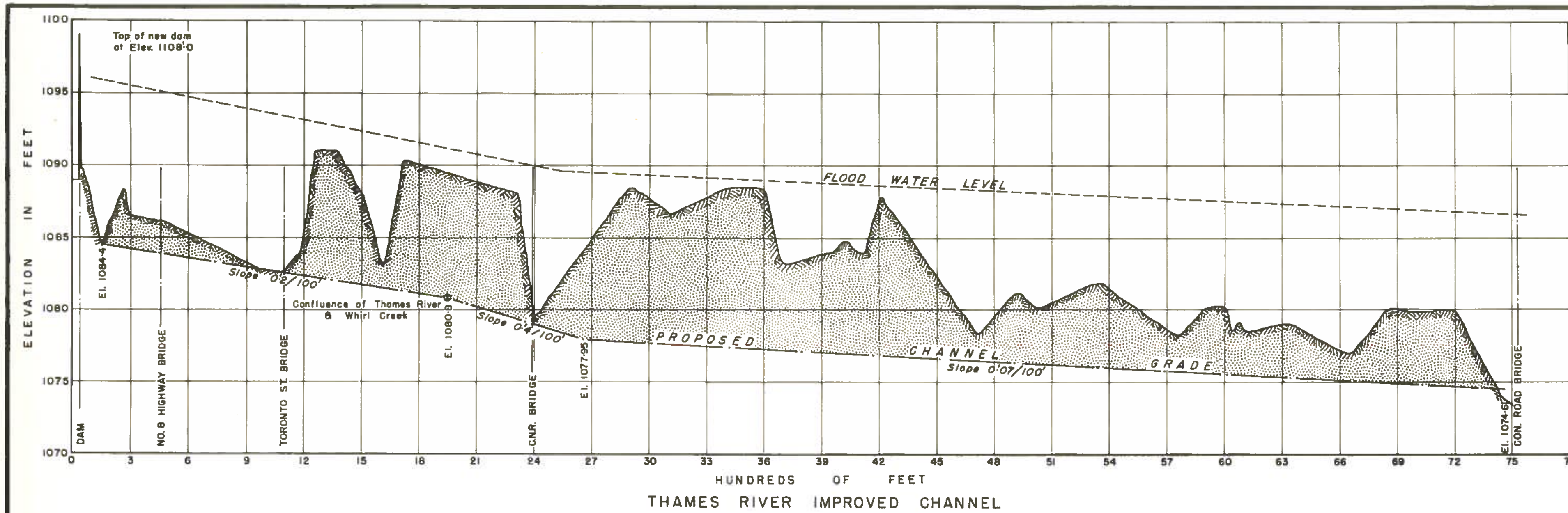
The estimated cost of the channel improvement including a new dam is $280,200.

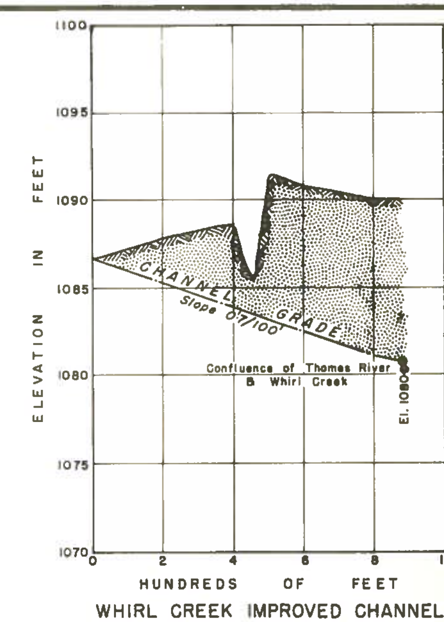
(3) Mitchell Channel Improvement (Including Small Dam) (Fig. H-9)

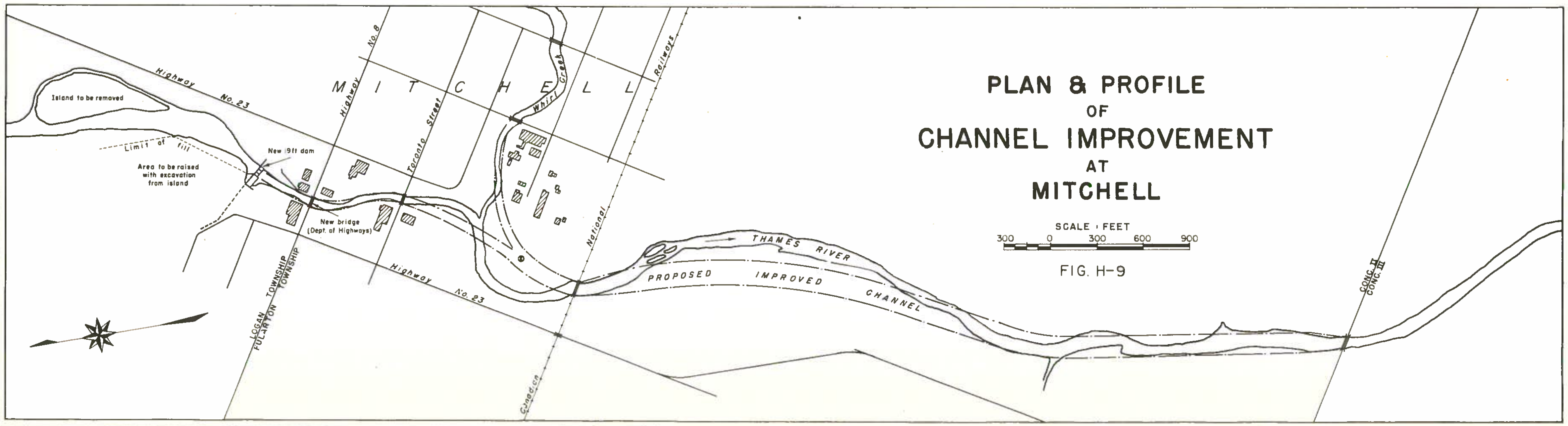
The town of Mitchell is located on the North Branch of the Thames at the confluence of Whirl Creek, 13 miles north-west of the city of Stratford. The total drainage area above the town is 119 square miles, consisting of 64 square miles for the Thames River and 55 square miles for Whirl Creek.

Twenty major floods have been recorded at Mitchell in the past and indications are that these floods have increased in frequency and severity in recent years. The earliest flood on the Upper Thames River in this vicinity of which any record has been found took place in 1858. In the course of the ensuing 93 years, evidence would indicate that clearing land and draining swamps have accelerated the spring run-off, increasing the volume of water in the river at times of high flow and proportionately decreasing the volume of summer flow.

There are no stream gauges located at Mitchell to measure the river flow, but it has been estimated that the flow at the dam has been as high as 6,200 c.f.s. Using this figure the corresponding discharge for Whirl Creek would be 5,300 c.f.s. The unit run-off for flows of this magnitude would be 97 c.f.s.







PLAN & PROFILE OF CHANNEL IMPROVEMENT AT MITCHELL

SCALE : FEET 300 0 300 600 900

FIG. H-9

per square mile. During the summer months the flow is reduced to a mere trickle and in many cases stops altogether. Such extremes in flow have caused much distress and inconvenience to everyone along the river. The floods have caused damage to dams, roads, bridges, buildings and crops and the lack of flow has deprived the town and adjacent countryside of a valuable water supply and of the potential recreational features of the river. Further, such conditions seriously interfere with the disposal of sewage and industrial wastes and present a threat to the health of the community.

While flooding is the major problem at Mitchell and the immediate objective is to control the high spring and flash summer flows, any remedial measures which do not provide for an increased summer flow could not be considered to be complete.

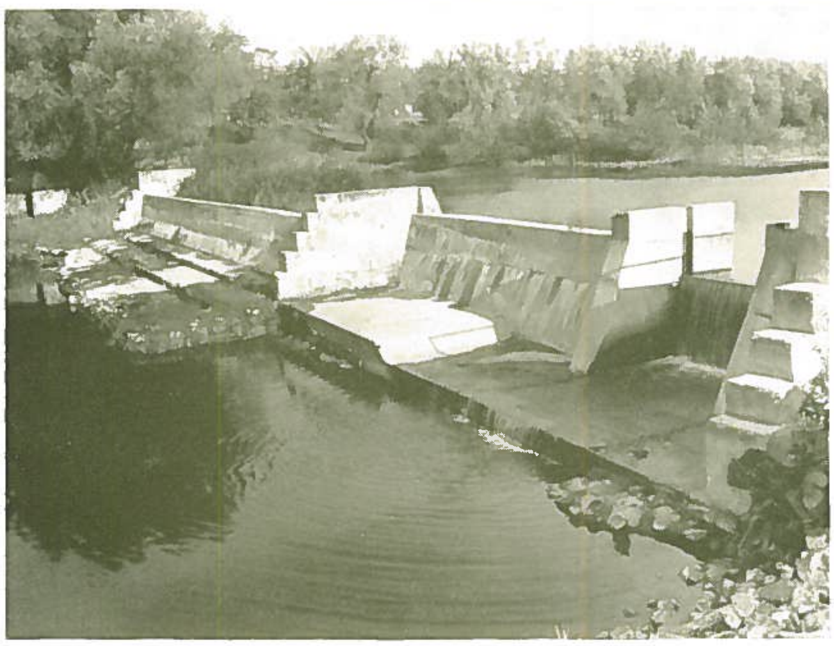
Conservation storage reservoirs would satisfy both phases of this problem. Unfortunately, there are no reservoir sites of economical size available above Mitchell. The present dam at Mitchell could be raised five feet and the pond area dredged to contain some 1,200 acre feet of storage, which would ease the flooding somewhat and provide means for a definite improvement in the summer flow.

This amount of storage situated so close to the trouble area would be invaluable in times of flooding but would not be nearly enough to give complete protection against even such floods as have occurred in the past. Thus other supplementary works are necessary. The required protection could be had by either increasing the channel capacities of the streams through Mitchell or by diverting the Whirl Creek flow around the town. Of these two methods the channel improvement is the better and is the one recommended.

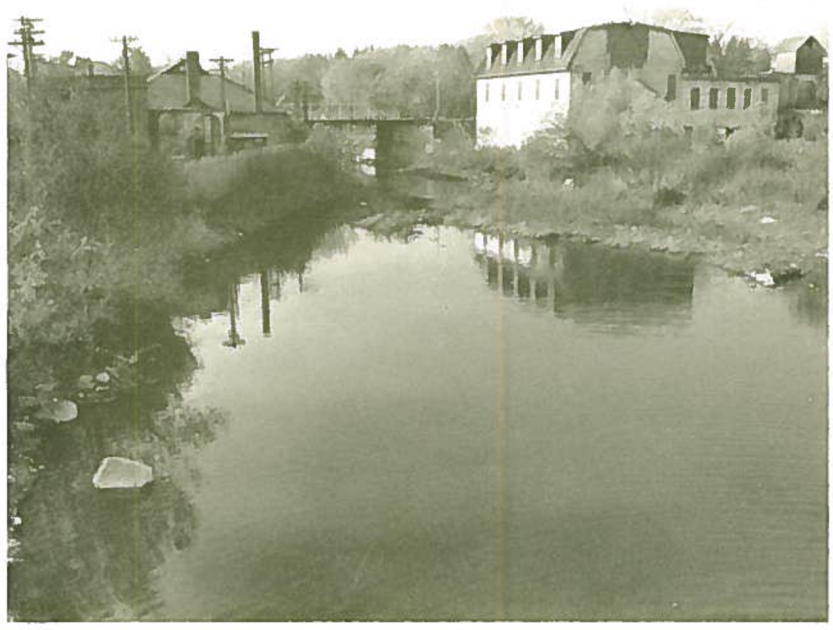
The channel improvement work would consist of removing obstructions such as islands and shoals, deepening, widening and straightening the river channel from the dam to a

Highway Bridge at Mitchell—a new bridge with a 75-foot clear span would be erected here.

Mitchell Dam. This dam would be removed and replaced by a dam 5 feet higher.



Looking downstream from the dam—the river has been confined to its low-water channel by buildings, retaining walls and narrow bridge openings.



point about one mile below the town. The improved channel would vary in bottom width from a minimum of 60 feet above the confluence of Whirl Creek to a maximum of 160 feet below the Canadian National Railway bridge. In addition, a concrete wall would be built at the Flax Mill to protect the mill building and the No. 8 Highway bridge would be replaced by a new bridge with a clear span of 75 feet. The Whirl Creek channel would be graded to a uniform slope and section with a bottom width of 35 feet and side slopes 1½ to 1 from its mouth up stream for a distance of 570 feet. The Whirl Creek channel would also be aligned at the confluence to allow the two flows to come together smoothly, thus permitting a freer passage of the flood waters at this point and minimizing the backwater effect.

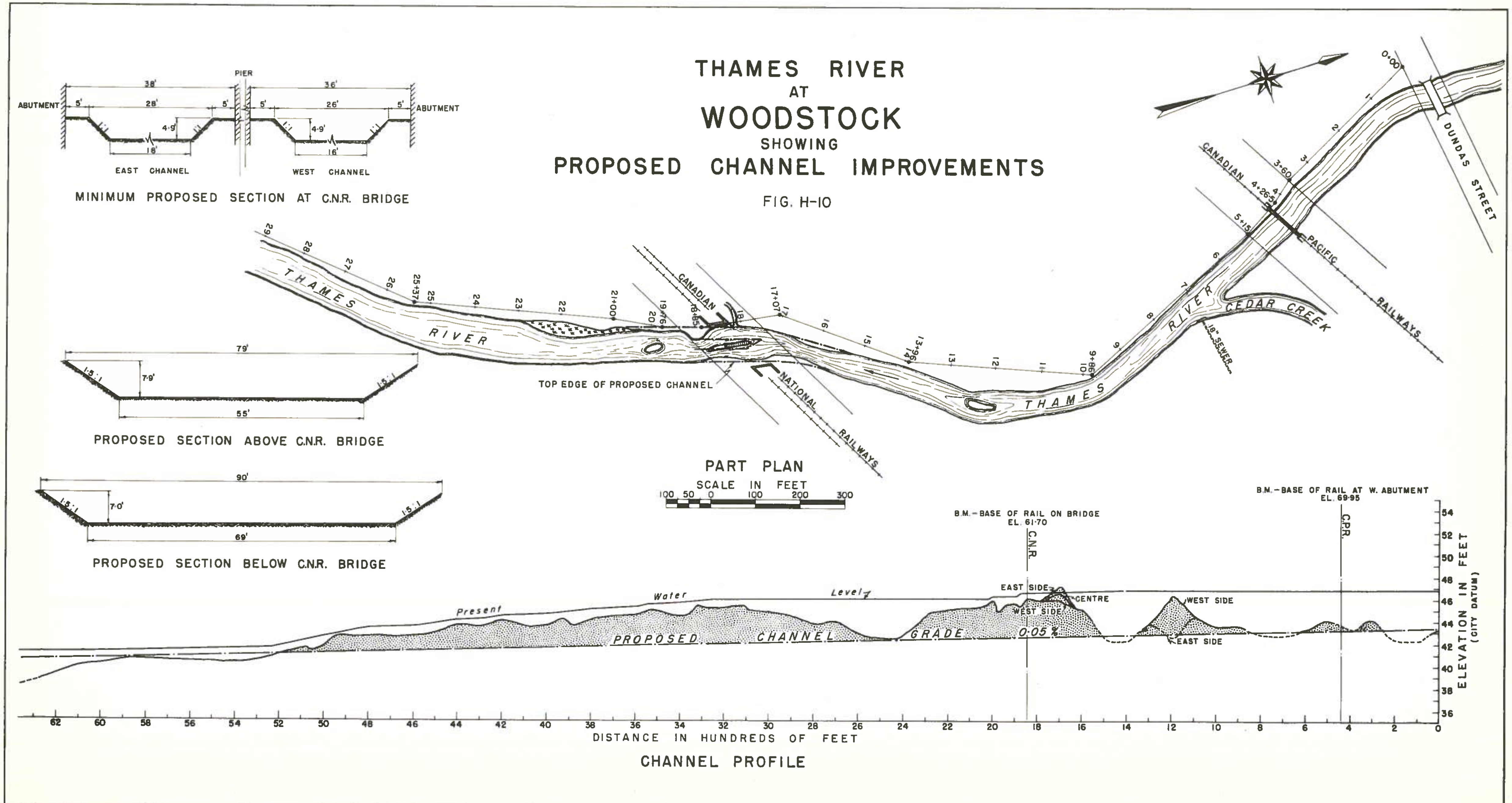
The additional storage being provided at Mitchell will more than compensate for the water which will be hurried through by the improved channel section and should prevent any appreciable increase in the flood flows below the town, and the increased summer flow which would be made available would benefit many along the river from the dam on down.

The plans and profiles for this proposed work are shown in Figure H-9. The ground surveys, flood investigation plans and cost estimates were made by the Kilborn Engineering Company Limited upon instructions from the Upper Thames River Conservation Authority. The above proposal is their recommended method for flood control and improved summer flow and is approved by the Department.

The estimated cost of the dam and channel improvement is $260,000.

(4) Woodstock Channel Improvement (Fig. H-10)

Flooding at Woodstock is caused by solid limestone rock shoals, which in places rise in the river bed, from Dundas Street for nearly a mile down stream. The shoals choke the flood flows and cause a backwater on the Thames River and up Cedar Creek where most of the damage occurs. Even with the


THAMES RIVER
AT
WOODSTOCK
SHOWING
PROPOSED CHANNEL IMPROVEMENTS
FIG. H-10
MINIMUM PROPOSED SECTION AT C.N.R. BRIDGE
EAST CHANNEL
WEST CHANNEL
ABUTMENT
PIER
PROPOSED SECTION ABOVE C.N.R. BRIDGE
PROPOSED SECTION BELOW C.N.R. BRIDGE
DUNDAS STREET
CANADIAN PACIFIC RAILWAYS
CANADIAN NATIONAL RAILWAYS
CEDAR CREEK
18" SEWER
THAMES RIVER
TOP EDGE OF PROPOSED CHANNEL
PART PLAN
SCALE IN FEET
B.M.-BASE OF RAIL AT W. ABUTMENT EL. 69·95
B.M.-BASE OF RAIL ON BRIDGE EL. 61·70
C.P.R.
C.N.R.
Present Water Level
PROPOSED CHANNEL GRADE 0·05%
EAST SIDE
WEST SIDE
CENTRE
ELEVATION IN FEET (CITY DATUM)
DISTANCE IN HUNDREDS OF FEET
CHANNEL PROFILE

shoals removed the channel is not large enough to contain major floods, but their removal, together with the Woodstock and Cedar Creek reservoirs, will provide protection against a flood of the hypothetical magnitude.

A channel capacity of 1,355 c.f.s. requires 18,300 acre feet of storage above Woodstock for the hypothetical flood. The two reservoirs shown below and recommended elsewhere in this report provide the following storage:

Woodstock Reservoir	5,152	acre feet
Cedar Creek Reservoir	7,728	" "
Total storage	12,880	" "
Storage required	18,300	" "
Storage deficiency	5,420	acre feet

With both reservoirs in operation the 5,420-acre feet storage deficiency would require a channel capacity at the confluence of the South Branch and Cedar Creek of 2,000 c.f.s. for the hypothetical flood and it is proposed to excavate and grade the river in order to increase the channel capacity from 1,355 to 2,000 c.f.s. The cost of this work is estimated at $75,000 if completed during a period of low water.

TABLE H-C

CHANNEL IMPROVEMENT DATA

Name	Length Feet	Bottom Width Feet	Cost $
Ingersoll	32,725	60 to 90	1,000,000
St. Marys	5,960	150 to 225	135,900
Mitchell	7,500	60 to 160	260,000[1]
Woodstock	5,900	55 to 69	75,000
Total			1,470,900

1. Including cost of small dam.

4. Hydrology of the Upper Thames River

(a) Hydrometric Gauges and Records

The storage required for the hypothetical spring flood has been determined from the records of the hydrometric gauges which in past years have been installed in the river and which will be hereafter referred to as "gauges".

Gauges are rods graduated in feet, tenths and hundredths of a foot which are installed at a section of a river, whereby, with a rating curve prepared from flows determined for various stages of the water level of the river, flows or discharges of the river at the gauge are known approximately for any reading on the gauge. The gauges in general are of two types: automatic, with which time and gauge heights are graphically recorded on a chart, and staff gauges which are read by observers, usually twice daily and oftener during flood periods.

This service is administered and the records published by the Water Resources Division, Ottawa (formerly called the Water and Power Bureau[1]). Table H-6 gives the names, drainage areas and period of records for all the gauges on the Upper Thames River and also shows the drainage areas for the main tributaries, places and damsites.

The published records do not show the time of recorded flows but show an average flow for each day which is called "mean daily" flow. It may be pointed out that although the records show that the maximum mean dailies for the North and South Branches coincided on many days, their peaks however may have been several hours apart. There may have been occasions when the peaks of spring freshets coincided at the Forks, but if so there is no mention of it in the published records.

The Hydro-Electric Power Commission began installing gauges in the rivers of Southern Ontario in 1910 and continued setting them up and operating them until 1919 when this service was taken over by the Bureau. Both the Hydro and the Bureau were interested only in those rivers which had power possibilities, and for this reason many of the gauges which had been set up were discontinued after a few years of

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1. For brevity hereafter called the "Bureau".

operation. The flashy rivers which have a high rate of run-off during the spring freshet, causing floods followed by extremely low flows during the summer, have little power potential, and for this reason gauges were not installed in most of these rivers until 1945 when they were set up by this Department at its inception. Gauges were installed however at Ealing on the South Branch and at Fanshawe (transferred to Western University in 1944) on the North Branch in 1915 and fortunately have continuous records to date. Also a gauge was set up in 1914 on the main stream below London at Kilworth bridge, which was transferred to Byron in 1922. Byron gauge was discontinued in 1932 and resumed in 1938 for the spring months only.

There are now nine gauges in operation on the Upper Thames Watershed. The locationsof the gauges are shown on Fig. H-5 and their rating curves in Figs. H-11 and H-12.

(b) Run-Off and Run-Off Ratios

Run-off is the amount of water that the drainage area supplies to the open stream and is the excess of precipitation over evaporation, transpiration and deep seepage.

The amount of run-off varies greatly with the classification and condition of the soil and is less than 10 per cent of the precipitation on dry sandy soils, to 90 per cent or more in the spring with frozen clay, ice or rock. The run-off of a drainage area above a gauge is expressed as a rate in c.f.s. per square mile and may be determined by the equation

$$\text{c.f.s. / square mile} = \frac{\text{c.f.s.}}{\text{Drainage area in square miles}}$$

With flood and low flow problems it is the extreme conditions which may occur that have to be considered, and for reliable values, hydrometric records of 20 years or more are necessary. The Western and Ealing gauges have long-term records and provide the data for the London flood problem. However, the rate of run-off varies over the watershed and the London gauges alone could not be used by the direct proportion of drainage areas to determine flows at St. Marys, Mitchell and Woodstock unless reliable adjustments were made for differences in geophysical and climatic conditions, a method which is sometimes necessary but gives results which are only approximate.

The short-term gauge records (six years) at or near the trouble areas above London can be used and a satisfactory rate of flow determined for those places by establishing run-off ratios between the years of the short-term and the corresponding years of the long-term records.

Table H-D shows the run-off ratios of the short-term gauges relative to the Western University and Ealing gauges. The ratios were determined by comparing the volume of flow for an eleven-day period for each year of the short-term records with the volume of the corresponding flood period of the long-term records at London, the run-off ratios being:

$$\frac{\text{The volume of 11 days spring flood flow at the short-term gauge}}{\text{The volume of 11 days spring flood flow at the long-term gauge}}$$

The weighted average of the six ratios for each of the short-term gauges gives a ratio which may be applied to any rate at Western University and Ealing gauges at London.

TABLE H-D

FLOW RATIOS RELATIVE TO WESTERN UNIVERSITY AND EALING GAUGES

Ratios determined from the Max. mean daily and the flow volume over the flood periods for the available years of record

Ratios Relative to Western University Gauge Drainage Area - 657.2 sq. miles			
	Ratio		
Gauge	Drainage Areas	Volume for Flood Period	Max. Mean Daily for Period
St. Marys	0.63320	0.66308	0.60785
Fish Creek	0.08780	0.06845	0.05700
Trout Creek	0.08308	0.06619	0.09165
Medway	0.10620	0.11712	0.10800
Ratios Relative to Ealing Gauge Drainage Area - 519.1 square miles			
Ingersoll	0.41290	0.39247	0.33520
Thamesford	0.22650	0.31588	0.33975

(c) The Hydrograph

> "[1]The Hydrograph is a correct expression of the detailed run-off of a stream, resulting from all the varying physical conditions which have occurred on the drainage area above the gauging station previous to the time which it represents."

It is by means of the hydrograph that the volume of storage herein is determined. Figs. H-13 and H-14 show continuous hydrographs for all the years of records for the Western University and Ealing gauges.

Fig. H-15 shows on a larger scale hydrographs for the major spring floods of record. The vertical measurements represent c.f.s., and the horizontal measurements time. The area of the hydrograph for any period of time represents the volume of water which has passed the gauge. In order to avoid the use of astronomical numbers, volume is expressed in acre feet[2] instead of cubic feet.

Flood hydrographs are roughly triangles or a series of triangles rising from and receding to the normal or base flow. The left limb (the rising or accession limb) rises from the base flow to the maximum mean daily or the apex of the triangle. Actually, for timed flows the apex is not a point but is slightly blunted or rounded. When at the apex, all of the surface run-off has reached the river and the right limb (the falling or recession limb) represents the lowering of the "valley storage" or the falling stages of the water level of the river until it is down to base flow. The rising and falling limbs are slightly concave but may be assumed as straight lines and the apex assumed as a point in calculations without serious error. The interval of time between base flows is the "flood period".

(d) The Hydrograph for a Hypothetical Spring Flood

When hydrometric records are available the amount of storage that would be required to control any particular

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1. Definition given in "Hydrology" by Professor D. W. Mead.
2. One c.f.s. flow for one day = 1.98347 acre feet or approximately 2 acre feet per day.

flood in the past is not a difficult problem. For a future or "hypothetical flood", however, having a greater magnitude than any known flood, the problem becomes extremely complex and particularly so with ice and snow conditions at the time of the spring break-up. Under these circumstances there is no means whereby storage may be determined with mathematical certainty. There are various approaches to the problem but all are approximations. Methods used are:

(i) The unit hydrograph method, which is a correlation of hydrometric and meteorological records. At the present time this method is considered by American hydrologists to be the best approach. Their flood problems however are caused chiefly by rain, and this method is not so well adapted to our widely variable conditions of the ground, temperature, snow, ice, and rain.

(ii) The frequency curve whereby, with 20 or more years of hydrometric records, from a curve developed by the theory of probability, extrapolated values for a maximum run-off for a period of once in 50, 100 and up to 2,000 or more years may be determined. As a matter of interest frequency curves were prepared for the North and South Branches at London (Fig. H-16) and they show that the hypothetical spring flood flows would occur but once in about 450 years on the North Branch and about once in 200 years on the South Branch.

These curves, however, are fallacious and the best that may be said for them is that they are a hoped-for indication. They are based on the "law of averages", but floods do not follow that law, as is evidenced by the frequency and increased magnitude of floods in the last decade, due possibly to a change in climatic conditions. Isolated freak floods are also a possibility which upsets the theory. According to the frequency curve the South Branch, as for a maximum mean daily, had a flood in the spring of 1937 which should not occur again for 350 years.

(iii) With 20 years or more of records, a run-off graph for spring freshets showing maximum mean daily flows against total run-off between base flows may be developed. By increasing the maximum run-off by one-third or more the corresponding mean daily flow and total run-off for a possible future flood are determined.

When hydrometric records are available over a period of 20 years or more, many hydraulic engineers consider from 1-1/3 to 1-1/2 times the greatest rate of run-off for that period is safe to use as a maximum. With 36 years of hydrometric records, many of which record major floods, it is believed that the construction of a hydrograph at the gauges for the hypothetical spring flood based on the run-off graph using a factor of 1-1/3 will give the best answer to the storage problem. The construction of these hydrographs is based upon three concepts which will be compared with flood hydrographs of record and shown to be logical:- These concepts are:

(1) That the hypothetical hydrograph is approximately triangular in shape.
(2) That the flood period or the base of the triangle may be determined approximately.
(3) That the maximum mean daily or the apex of the triangle may be determined approximately.

(1) <u>Hypothetical hydrograph is approximately triangular in shape</u>

An examination of the hydrographs in Fig. H-13 and Fig. H-14 for the summer storms of June 1922, August 1926, July 1927, and July 1945 for the Fanshawe and Ealing gauges shows that they are definitely triangular in shape. With summer storms, where the hydrographs have been broken and have two or more apexes, the cause may be traced to a break in the storm or to an unequal distribution of rainfall.

During the spring run-off period it is ice jams, intervals of freezing temperatures, or rain which cause breaks in the hydrograph. If there were no ice jams or heavy rains and the temperatures remained above freezing there would be no halt in run-off and the hydrograph would be continuous, having only one apex, excepting the case of major tributaries which may break up before or after the main stream.

Prolonged freezing intervals during the spring break-up lower the stage or water level in the river providing "channel storage" capacity, extend the time of the run-off period allowing the snow and ice to dissipate slowly and reduce the flood crests to lower levels or stages than would be the case with no freezing intervals and only one flood "run" or one apex. Fig. H-17, a hydrograph of the 1945 spring run-off, shows the influence of precipitation and temperatures on flood flows.

Since a definite volume of flow has been provided for the hypothetical flood period and one apex gives the highest rate of flow, therefore in theory an unbroken hydrograph or triangle would result in the highest peak for the given volume of flow.

Also if there should be a lag in the break-up of any of the major tributaries of the Thames such as the North, South or Middle Branches of the Thames, there would be a break in the hydrograph at the Forks. If the peaks, however, coincide at London there would be but one apex and the most adverse condition would be satisfied.

A flood period, however, of less magnitude than the hypothetical spring flood may have two peaks and cause a flood if the second peak or run has not been anticipated and space provided in the reservoirs for the second run. The soundness of the single triangular hydrograph is therefore contingent upon the reliability of meteorological forecasts and snow surveys prior to and during the spring flood period and all measures necessary for the efficient management and control of the dams.

Large ice jams, if they formed above a trouble area, could cause a break in the hydrograph and boost the flood peaks when the jam broke. The hydrograph for the hypothetical spring flood makes no provision for ice jams.

(2) Determination of the Base

An examination of the hydrographs of the flood periods at the Western University and Ealing gauges showed that the duration period did not vary greatly for summer storms, it being from 5 to 7 days only. With the spring floods, however, there is a wide spread from 7 days to as much as 24 days, but, as was pointed out in the previous sub-section, the long periods have been dissipated and have several minor peak flows. The major floods vary by only a few days; the pattern being that as the peak increases the duration of the flood period decreases, or in general the greater the flood magnitude the shorter the duration period, depending upon the breaks in the hydrographs. Eleven days is about the average duration for the major floods at these gauges. This is the duration period which is used in the next sub-section in preparing the run-off graph.

(3) The Run-Off Graph and Determination of the Apex or Maximum Mean Daily

The apex of the hypothetical hydrograph has been determined by means of a "run-off graph" (Figs. H-18 and H-19) which is a graphical correlation of the two variables, namely the volume of flow in acre feet and the maximum mean daily in c.f.s. for all of the spring flood periods of record. From these graphs the maximum mean daily or apex may be determined by extrapolation for the volume of flow which has been provided for the hypothetical spring flood period.

A study of the major flood periods showed that the duration at the London gauges varied between 10 and 12 days for the North Branch and from nine to eleven days for the South Branch. For the purpose of comparison three run-off graphs were prepared for the North Branch and three for the South Branch - 10, 11, and 12 days duration for the former and 9, 10 and 11 days for the latter. There was less than two per cent difference in the storage determined from each of the run-off graphs, a flood period of 11 days giving the greatest storage and the best result for both branches.

The run-off graphs were prepared as follows:-

(i) The area or volume of flow for the period was found by summing the flow records for each day and converting the result into acre feet. The volume for the flood period was plotted on cross-section paper relative to the maximum mean daily flow for the period, the volumes being to a horizontal scale and the maximum mean dailies to the vertical. All of the spring flood periods from 1916 to 1950 inclusive were plotted.

(ii) The arithmetic mean was determined for the volumes of all the periods and also for the maximum mean dailies, by their summation and dividing the results repectively by the number of periods. The arithmetic mean was plotted and a horizontal and vertical axis drawn through the point.

(iii) Moments were taken on both sides about the horizontal axis, the moment $M = YQ$, Y being the vertical measurement of the points to the horizontal axis and Q the volume for the period. The sum of the moments on each side of the horizontal axis were divided by the number of the periods on each side respectively. These two points A and B when plotted and joined by a straight line, the line passes through the arithmetical mean. Moments were taken in a similar manner about the vertical axis and the corresponding points C and D plotted. The straight line joining these points also passed through the arithmetical mean. These lines do not coincide but form an angle[1] at the arithmetical mean, due to the variable agencies which influence spring run-off.

(iv) The lines AC and DB were bisected at E and F respectively, which gives an average for the co-ordinates, and points on the line EF and EF produced are co-ordinates of a correlation between the maximum mean daily relative to any volume of flow for an eleven-day period.

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1. This angle is much smaller for summer storm periods than for those for spring run-off.

(e) <u>The Maximum Mean Daily for the Hypothetical Spring Flood at the London Forks</u>

(1) <u>North Branch</u>

The greatest flood on the North Branch occurred in April 1947.

The flow at London for 11 days was 206,562 acre feet and for the hypothetical flood amounts to 206,562 x 1-1/3 = 275,416 acre feet.

From Fig. H-18, the corresponding maximum mean daily flow for the hypothetical flood is 38,878 c.f.s. Figure H-20 shows the developed hydrograph for the hypothetical flood in the North Branch superimposed on the 1937 and 1947 flood hydrographs.

(2) <u>South Branch</u>

The greatest flood occurred on the South Branch in April 1937.

The flow at London for 11 days was 127,289 acre feet and for the hypothetical flood amounts to 127,289 x 1-1/3 = 169,719 acre feet.

From Fig. H-19, the corresponding maximum mean daily flow for the hypothetical flood is 20,700 c.f.s. Figure H-21 shows the developed hypothetical flood hydrograph superimposed on the 1937 and 1947 flood hydrographs.

From Figs H-19 and H-21, it will be noted that the maximum mean daily for the South Branch 1937 flood is slightly greater than that of the hypothetical flood. This great flood was exceptional for the South Branch drainage area, having been caused by heavy rain with a storm centre not far from Woodstock. 6.97 inches of rain was recorded at Woodstock between April 21 and 28, 3.37 inches of which fell in one day. Such a freak storm might occur again; if so, the storage provided by the increase in volume for the hypothetical flood would reduce peak flows to a safe stage at London. Figure H-22 shows the developed hydrographs for the 1937, 1947 and hypothetical spring floods at the Forks.

(3) Adjustment of the Base or Flood Duration

Using the determined area and altitude functions in the triangular hypothetical hydrograph and solving for the base gives a duration of:-

$$\text{base} = \frac{2 \text{ area}}{\text{height} \times 1.98347} \qquad \frac{2 \times 450560}{60302 \times 1.98347} = 7.534 \text{ days}$$

The triangle (Fig H-22) has been adjusted to conform to the 11-day period by extending the average base flow at the up and down limbs to 11 days and reducing the area of the triangle by an equal amount.

The similarity of the hydrographs in Figs H-20 and H-21 is noteworthy. They indicate that the triangular concept for flood periods is fundamentally sound. The storage which will be determined from the hypothetical hydrograph may not be absolute but it is believed to be a close approximation.

(f) Channel Capacity at London

The "channel capacity" for a place which is subject to flooding is the maximum flow in cubic feet per second which can be contained within the river channel without flooding. Expressed in another way, it is the highest stage or water level that the river can reach without overflowing its banks. Where dikes are proposed or already in place it may be related to some stage on the dikes.

A reliable "channel capacity" for each place flooded is essential for the solution of the flood problem at that place. If there is a rated gauge at the flooded area in question, the channel capacity flow can be determined at any time by relating the elevation of the overflow point with the same elevation of the gauge and the rate of flow for channel capacity determined from the gauge's rating curve. If the gauge is not too far away it may be obtained in the same way if the slope of the river or the difference in elevation is known between the gauge and the point of overflow. If the gauge is some distance away and the average rate of flow between the points is known, it may be determined at flood time by having an observer at the flooded area and another at the gauge, each noting the time that the river reaches the "channel capacity" flow.

The Forks (Fig. H-5) is the critical point of overflow and tests were made for the channel capacity for London at that place during the spring flood periods of 1948 and 1950. Timed flows were observed at the Western University, Ealing and Byron gauges and were related to timed stages at the City of London's gauge at their Douglas Avenue Pumping Station which is located on the main stream about half a mile below the Forks. The gauge at Douglas Avenue is not rated and in order to determine a direct and absolute value for channel capacity and water level slopes between the Forks and the Douglas Avenue gauge, steps have been taken to have the river rated at the gauge and another gauge installed at the Forks. In the meantime the 1948 and 1950 tests have been used to determine the storage above London and, although tentative, it is believed that later tests will not show any substantial difference in the storage.

The right[1] bank of the North Branch from Oxford Street to the Forks and on the main stream to Douglas Avenue is protected by a continuous earth embankment, dike or breakwater with a concrete facing. The elevation of the top of the dike at the Forks is 774.0 feet[2].

The critical point of overflow is on the opposite side of the river at the junction of Dundas Avenue and James Street, the ground surface elevation being 770.0. From observations in 1948 by the City Engineer's Department, the Douglas Avenue gauge read 768.5 at the time when the river rose to the point of flooding at the Forks, with a water level slope at that time of 1.5 feet between the Forks and the gauge. This slope was used in both the 1948 and 1950 tests, allowance having been

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1. Facing down stream.
2. Geodetic Survey of Canada datum.

made for work done by the City in 1949 to ease this flow at the Forks; accordingly the channel capacity at the Forks is 35,670 c.f.s.

(g) Storage Required Above London

Space is provided in reservoirs for four independent purposes and the amount for each determined, viz:

(1) Channel capacity storage

(2) Dead storage

(3) Operational storage

(4) Boost discharge storage

(1) Channel Capacity Storage

The channel capacity storage above London is the equivalent of the volume of water which overflows at London. It is represented on the hypothetical hydrograph (Fig. H-23) by the area above the line 35,670 c.f.s., the channel capacity rate of flow. The triangular area above this line is

$$\frac{1.5385}{2} \text{ in. x } 4.926 \text{ in.} = 3.7893 \text{ sq. in.}$$

1 square inch = 19834.7 acre feet

Therefore the channel capacity storage =

3.7893 sq. in. x 19834.7 = 75,160 acre feet

(2) Dead Storage

A reservoir is never drained bone dry. A certain amount of water is retained to protect the discharge tubes at the foot of the dam and to facilitate the silting of the reservoir bottom in the immediate vicinity of the dam in order to protect natural and artificial seals against damage. Dead storage space therefore is not used for flood control nor is the water available to supplement low flows at the end of a dry period.

The amount of dead storage depends upon the gradient of the bed of the reservoir and its width in the vicinity of the dam, and will vary for each reservoir.

The total amount of dead storage required is estimated (Table H-7) at 2,100 acre feet.

(3) <u>Operational Storage</u>

Operational storage is a cushion which will enable the control of the flows through London to approach but not exceed the channel capacity.

The following section on Water Conservation shows that from 48,000 to 50,000 acre feet of water should be stored in the reservoirs during the spring run-off period in order to increase and sustain the low flows which follow. If cheap storage were available this "conservation storage" for increasing low flows, reservoir space would be provided over and above that required for flood control and this part of the storage problem would be simplified. The cost of storage, however, on the Upper Thames, as with most watersheds in Southern Ontario, is very high and it is necessary to use the space in the reservoirs called "dual-purpose reservoirs" for both purposes, which means that during the spring flood period the controller of the dam system must operate them in a manner which will prevent a flood and at the same time fill the reservoirs to the conservation storage level, a very uncertain and exacting task. He has much worry at this time. He has to regulate the discharges of many dams, most of which are some distance upstream, time and route their discharges so that the flows through London (explained in the next section) will conform approximately to the hypothetical hydrograph and not exceed the channel capacity. He has to make provision for ice jams and regulate the dams in such a manner as to prevent flooding at the other trouble areas above London. His main concern, however, is the amount of snow cover, the condition of the soil and in particular what the weather ahead will be during and immediately following the spring run-off. It is assumed that, prior to the spring break-up, the condition of the soil will be known and that the average weight of snow pack for the various areas of the watershed will have been determined by field observers, from which the controller will know approximately the potential volume of run-off. What he does not know is the behaviour of the snow melt, or if he will also have to contend with heavy rainfall. Meteorological forecasts are indispensable and future scientific research and more stations will enhance their dependability, but at the present time "No method of accurately forecasting the weather for more than 48 hours in advance has yet been devised that meets with the approval of the larger part of the meteorological profession"[1] whereas at least two weeks is required.

From the foregoing remarks it is obvious that during the spring run-off period the operator cannot control the flows at London, holding them at the channel capacity stage, but must have a lower stage to provide a margin of freeboard or a cushion below channel capacity, for an objective.

With the complexities of spring run-off and the above unpredictable agencies, it is not possible to determine the depth of freeboard by any analysis or mathematical means, and for the present, only an estimate based on considered judgment can be made. It is estimated that 9 inches or 0.75 feet of channel capacity freeboard is necessary. The flow at elevation 769.25 at the Forks is 31,725 c.f.s. and the triangular area above this line (Fig. H-23) is $\frac{1.785}{2}$ in. x 2.8577 in. = 5.101 sq. in. 5.101 x 19834.7 = 101,177 acre feet. The operational storage = (101,177 - 75,160) = 26,017 acre feet. From Fig. H-23 it will be seen that an inch of freeboard below channel capacity averages $\frac{130,775 - 75,163}{18}$ = 3,090 acre feet, or approximately

$309,000, a considerable amount for an inch of error in the estimate. The plan, however, will be a long-term program, and later experience in operation may show whether too much or too little channel capacity freeboard has been provided herein for operational storage. If it is not enough or is too much there will be time to adjust the plan accordingly.

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1. Authority: Charles D. Hopkins, Jr. Ass. M. A.S.C.E., U. S. Weather Bureau, River Forecast Center.

The operational storage estimate is contingent upon the following premises:

(i) That the control of the dams is under efficient management with adequate personnel and equipment.

(ii) That surveys are made to observe the condition of the soil, the depth to the ground water table, to record the accumulation of snow during the winter, and particularly the condition of the soil and the depth and the weight of the snow pack at the spring break-up.

(iii) The rapid assembly at the control centre of rainfall and run-off information, stream gauge records at the areas subject to flooding, at the dams and any other strategic places.

(iv) A sound and rapid system of communication together with an emergency operating plan in the event of failure of the communication system. The system should be such that information from the operation personnel at gauges, dams, meteorological stations and any other places that would influence the control of the dams could be transmitted to the control centre, and that instructions issued by the control centre would be transmitted quickly and with the assurance that the information is reliable and the instructions sent out understood and carried out.

(v) That a basic plan is installed at each reservoir.

(vi) That weather forecasts are made by the best means available.

(vii) That discharge curves are prepared for the discharge tubes control, valves, gates and spillways together with curves for storage capacities and time of wave travel.

(viii) Any other measures employed that would increase the efficiency of the control centre.

(4) <u>Boost Discharge Storage</u>

Boost storage is a necessary increase in the head of water at the dam in order that the dam, during the first part of the spring run-off period, may be able to discharge a volume of water equal to the inflow into the reservoir. In order that the function of the boost storage may be understood, some particulars concerning dam regulation which conform to the hypothetical hydrograph should be explained.

For the hypothetical spring flood period, and for floods approaching this magnitude, the regulation of the dams should be such that the flows through London should conform approximately to the hypothetical hydrograph; which means that the flow should follow the rising limb, until the flow reaches the stage between the operational and channel capacity rates (30,410 c.f.s. and 35,670 c.f.s. respectively); and if, during this interval of the rising limb, the discharge from the dams is equal to the inflow into the reservoirs, the flows through London will conform to the hydrograph. After the flows have reached that stage, the inflow into the reservoirs would be greater than the discharge from the dams, and the water level in the reservoirs would rise and the reservoirs would be full at the time the falling limb reached the channel capacity rate of flow. At this time the peak of the flood has passed, the surface run-off is complete except for the contribution of ground water flow, the water is all in the river and its tributaries; and, provided there is no further precipitation or added discharge from the reservoir, the stages at London would conform to the falling limb and recede to base flow.

If, during the rising interval, the discharge from the dams should be less than the inflow into the reservoirs, the reservoirs would fill prematurely, and if the run-off were of the hypothetical magnitude would result in a flood. Consequently, in order that the dams may be able to discharge the inflow, provision must be made in both the design of the discharge tubes and the head of water over them.

At the beginning of the spring break-up, any conservation storage left in the reservoirs would be dumped to the dead storage level to provide space for the approaching flood.

would this cause ice jamming below NW FPT.

At the dead storage level it would not be practicable to provide discharge tubes of sufficient size and number to discharge the inflow at such a low head, hence the head must be increased or "boost discharge storage" provided to the extent that tubes may be designed economically to discharge the inflow.

The volume of boost storage required depends upon the drainage area of the reservoir, the amount of the run-off and the design of the dam, and will vary for each reservoir. The design of the dams is beyond the scope of this report, but an estimate of the boost flow storage is necessary at this time, and is 7,800 acre feet.

Table H-7 shows the reservoir distribution of this storage and it may be noted that there has been, comparatively, a greater amount allocated to Fanshawe than to the other reservoirs. This was done partly to enable the Fanshawe tubes to discharge an amount approaching channel capacity and also because of the possibility that the additional storage might be needed as a domestic supply for London, in which case it would be considered as dead storage. Actually the storage available in the six reservoirs was 1,070 acre feet greater than the determined storage, and this excess was put into the boost storage at Fanshawe, which explains why the determined storage happens to be exactly the same as the capacity of the reservoirs.

(5) <u>Total Storage Required Above London</u>

A summation of the foregoing storage classifications is tabulated below:

TABLE H-E

Storage	Acre Feet	Percentage of Total Storage
(1) Channel capacity storage	75,160	67.67
(2) Dead storage	2,100	1.89
(3) Operational storage	26,017	23.42
(4) Boost discharge storage	7,803	7.02
Total storage required above London	111,080	100.00

From the above, the efficiency in the operation of dual-purpose reservoirs is $\frac{75,160}{75,160 + 26,017}$ x 100 = 74.28 per cent.

5. Water Conservation Storage and Increased Flows

Excluding Fanshawe, the other five reservoirs when full have a total capacity of 64,500 acre feet. It is assumed that the reservoirs would be full at the end of the spring flood period and unless the month of May were dry they would remain full or nearly so until June 1. This is about the time when the flows in the rivers need to be increased and the impounded water in the reservoirs may then be released to sustain the flow.

All of the above potential storage is not available as conservation storage and deductions must be made for certain losses.

(a) Space for a hypothetical rainstorm.

(b) Loss by water surface evaporation and ice formation

(c) Dead storage

The above storage losses are shown in Tables H-3 and H-4 and an explanation of each follows:

(a) Space for a Hypothetical Rainfall

The hypothetical rainfall is a planned-for rain that might occur at any time after the spring freshet and for which space must be made available in the reservoirs as soon as possible after the spring run-off in the case that they have been filled to capacity or above the limiting stage. For such a rainfall the same pattern has been followed as for the spring floods, viz: a run-off volume 1-1/3 times the greatest run-off on record by rainfall only; the preparation of run-off graphs of the greatest rainfall run-off of record (for both branches at London); the construction of the hydrograph at the Forks; and from it the determination of the storage space to be made available for the hypothetical rainfall.