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PUBLIC WORKS DEPARTMENT, N.S.W.

LOWER WYONG RIVER MODEL INVESTIGATION

Report No. 437

October 1985

Manly Hydraulics Laboratory

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SUMMARY

The Lower Wyong River is a flood prone river emptying into Tuggerah Lake about 70 km north of Sydney. Increased pressure for further development within the flood plain has necessitated an investigation of the flow conditions by a physical model. Model tests were made with the present level of development included in the model. Measurements of the velocity field were made with the steady state flows corresponding to the peak discharges for a 50 year flood and a 100 year flood. Flow distributions for the in-bank and overbank areas were also measured for the 100 year flood.

The water levels were measured at ten points throughout the model. Of these, two points corresponded to the known down-stream water levels which were obtained for model operation from a previous numerical model study.

A comparison between the water levels in the physical model and a 1D numerical model of a previous investigation showed that they were in general agreement but maximum discrepancies of 16cm for the 50 year flood and 10cm for the 100 year flood were noted.

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1. INTRODUCTION

The Manly Hydraulics Laboratory was commissioned by the Rivers and Ports Branch of the Public Works Department to carry out a physical model investigation of the flooding by the Lower Wyong River.

The general area prone to flooding extends from the Pacific Highway in the east to Tuggerah Lake in the west with a north-south width of about 3 km (see Fig.1). The model was used to determine the velocity field and flow paths in the flood plain and along the river. The present level of development was to be included in the model.

2. SITE DESCRIPTION

The study area proper was contained within a loop of the Lower Wyong River and bounded to the north by McDonagh Road. The river was generally bounded by trees and undergrowth along the river banks with some trees that had fallen into the river. Figure 2 shows the residential band of housing present along the northern river bank. It was estimated from a site visit that about 10% of this development was elevated above the ground level on stilts or supported by a central concrete core. The character of the other overbank areas in the main study area and in other regions of the model ranged from roads and grasslands to bushlands with dense undergrowth.

3. PROJECT DATA

3.1 Survey Data

The river was surveyed in 1981 at five cross-sections. This provided the basis for modelling 2.5 km of the Lower Wyong River. The primary source of survey data for the overbank area was a 1:4000 plan of the Lower Wyong River flood plain mapping. This was part of the Tuggerah Power Station Mapping and dated April 1984.

This information was supplemented with two series orthophotomaps (which were constructed from aerial photographs) from the Department of Lands. The Wyong 1:4000 series had a contour interval of 2m and was dated February 6, 1973. The Wyong 1:2000 series covered a more limited area. It had a contour interval of 1m and was dated March 27, 1977. The regions covered by the two series had some overlap and where there was some conflict, the information from the 1:2000 series was given precedence because of its greater accuracy (estimated at ± half a contour interval).

3.2 Flow Data

Steady state model tests were conducted for two flows which corresponded to a 50 year and 100 year flood. The input discharges used were the peak flows of the flood hydrographs and were 1150 m^3/s for the 50 year flood and 1550 m^3/s for the 100 year flood. Due to the paucity of field data these flows had been determined from a unit hydrograph analysis (Reference 1).

4. THE MODEL

4.1 Extent of the Model

The model boundaries were selected on the basis of providing a sufficiently large model to provide a reasonable representation of the hydraulic processes while limiting the extent of the model in order to reduce its construction costs. Reference 1 was used as a guide to the main flow paths and hence to determine the model boundaries.

The extent of the model is indicated in Figure 6. There is one inflow boundary and two outflow boundaries. The discharge through both outflow boundaries ends up in Tuggerah Lake. The other sides of the model are bounded by high land or (in the case of the southern boundary) regions where the water is relatively slow flowing and remote from the main study area.

4.2 Model Scales

4.2.1 General Scales

Being a free surface flow model,the Froude criterion was used as a basis for determining the relation between the vertical length scale, the horizontal length scale and the time scale.

The vertical scale was chosen as 60 in order to promote rough turbulent flow throughout the model. Due to the slower velocities and shallower depths on the flood plain than in the river channel, it was more difficult to ensure fully rough turbulent flow in the overbank regions of the model. It was the conditions of flow over the flood plain therefore which were the dominating factor in determining the vertical length scale of $Y_r = 60$. The subscript 'r' is used to denote the ratio of prototype to model value. There were regions of the model removed from the main study area in which the flow was expected to be in the transition zone of turbulent flow. The likelihood of obtaining fully turbulent flow was greater for the more critical 100 year flood than the 50 year flood.

The sinuosity of the river and the presence of the three-dimensional structure of the flow as water spilled over from the river channel onto the flood plain, required that the model distortion to be kept to a minimum. However, a horizontal scale of $X_r = 60$ (with $Y_r = 60$) would have resulted in an expensive model with an area of 1000 m^2. By distorting the length scales, to $X_r/Y_r = 4.5$, the model area was reduced by a factor of 20 ($\simeq 4.5^2$). The subsequent reduction in model costs resulted in a feasible physical model. The horizontal length scale was $X_r = 4.5 \times 60 = 270$. A consequence of distorting the length scales is to render the modelling of those areas of the model with significant vertical accelerations (and hence also velocities) less reliable. Such areas were present along the banks of the river where there were some levees. When these levees were clearly identifiable, they were modelled undistorted, i.e. with $X_r = Y_r = 60$.

The velocity and time scale are then defined by the Froude criterion. The scales are now -

Froude No. scale	:	$F_r = 1$
horizontal length scale	:	$X_r = 270$
vertical length scale	:	$Y_r = 60$
velocity scale	:	$V_r = Y_r^{1/2} = 7.75$
time scale	:	$t_r = X_r Y_r^{-1/2} = 34.9$
discharge scale	:	$Q_r = X_r Y_r^{3/2}$ $= 1.255 \times 10^5$

4.2.2 Roughness Scale

The roughness scaling of the model was carried out separately for the in-bank flows and the overbank flows.

In-Bank Roughness

Based on a site inspection and Reference 2, the Mannings 'n' for the in-bank prototype flow was estimated to be .035 for the whole of the river modelled.

In-bank roughness was achieved using small permeable discs (4 cm in diameter, 2 cm thick) made up of a synthetic, fibrous matted material. These discs were glued to the sides and bed of the river channel in the model. The number of discs in the channel was varied until the measured backwater curve corresponded to a calculated backwater curve for n_p = .035. The subscript 'p' is used to denote a prototype quantity. For the in-bank flow, the roughness scale is given by -

$$(C_h)_r = \sqrt{(X/R_h)_r} \qquad (1)$$

where R_h = hydraulic radius
C_h = Chezy coefficient

The measured and calculated backwater curves are plotted up in Figure 3.

Overbank Roughness

The roughness for the overbank flow was achieved using cylindrical pegs (see Figure 4). The effects of the friction forces on the water in the prototype were simulated in the model by the drag forces on the pegs and the shears developed on the concrete model base. An unusual aspect of the model overbank flow was that some regions of the model were well within the transition region of the Moody diagram. This implied that the shears due to the viscosity of water may not be negligible compared to the turbulent shear forces. The consequences of modelling in the transition region are examined in the Appendix. Here it is argued that due to the model distortion of 4.5 (= X_r/Y_r), the frictional forces and hence the roughnesses required in the model flood plain are much greater than the existing combined 'roughness' due to both the concrete model surface and the viscous drag of the water flowing at Reynolds numbers as low as about 5,000 (where, as in the Moody diagram for pipes, the Reynolds No. is defined as $R_m = 4V_m h_m/\nu$). The roughness scale for the overbank flow is given by -

$$(C_h)_r = (X/Y)_r^{1/2} \qquad (2)$$

With a knowledge of the Chezy coefficients in the prototype, equation 2 was used to calculate the required Chezy coefficients throughout the model. The spatial distribution of Chezy coefficients in the prototype was determined by first dividing up the area modelled into twelve regions. Each region was then characterised by a Mannings 'n' value {or equivalently a Chezy coefficient $\{(C_h)_p = (h_p^{1/6})/n_p\}$. These values were based upon an underbrush survey, aerial photography and values recommended in Reference 2. Figure 5 shows the various regions and the Mannings 'n' values adopted for the model study.

Pegs of 0.95 cm diameter dowel were glued onto the hydraulic model to achieve the required model roughness as determined by Equation 2. The density of peg placement (number of pegs per square metre) was found from -

$$N = \frac{2g}{C_D D h_m} \left[\frac{1}{(C_h)_m^2} - \frac{1}{(C_h)_{con}^2} \right]$$

where

N = number of pegs/m^2

C_D = drag coefficient for a cylinder

D = peg diameter = 0.95 cm

h_m = depth of flow in model (m)

$(C_h)_m$ = target Chezy coefficient for the model from equation (2)

$(C_h)_{con}$ = Chezy coefficient for the concrete capped model without the added roughness elements.

For the Reynolds numbers expected in the model, a drag coefficient of C_D=1 was sufficiently accurate. This is in spite of the fact that some regions of the model were located in the turbulent transition region of the Moody diagram. However, it has been shown in the Appendix that if the roughness criterion adopted assumes fully turbulent flow, then the errors are insignificant. When these errors are translated into a placement density for the roughness elements, the error in the number of pegs per square metre will not amount to more than about 3.5% for a prototype Manning's 'n' value of 0.04. For larger Manning's 'n' values, the error would be even less. In any case, these considerations only apply to the smaller 50 year flood which, when compared to the 100 year flood, are associated with the smaller Reynolds numbers.

On the flood plain the pegs were arranged in a pattern which was as isotropic as possible from different flow directions. No placement pattern of the pegs was completely satisfactory from this point of view, but a triangular arrangement was preferable to a rectangular one. In addition, it was necessary to arrange the placement pattern with an orientation to suit the expected flow directions in each of the regions of the model. With this care taken, it was possible to avoid the situation whereby the rows of pegs were aligned with the velocity direction. In such a situation, the drag force exerted by the pegs on the water would not be mobilised to the full extent.

4.3 Model Construction

The model surface levels were set using vertical masonite templates in both the river channel and the flood plains. Between the templates, a concrete capping was applied to a sand base.

The model roughness was achieved using 4 cm diameter discs, 2 cm thick of a synthetic matted material in the river channel and .95cm diameter dowel pegs in the overbank area.

Houses and other developments were modelled by foam blocks. The sizes and distribution of these buildings were measured from small format aerial photographs.

4.4 Model Operation And Boundary Conditions

The model had one inflow and two outflows. At the inflow end the required discharge passed from a V-notch weir into a sump with baffles. The water then flowed over into the model and finally passed out into one of two sumps at the downstream ends and subsequently over a tailgate. Water passing through the modelled area was destined for "Tuggerah Lake" irrespective of which tailgate the water passed over. The downstream water levels were fixed by adjusting the two tailgate settings, such that the water levels in the sumps matched the stage contours from Reference 1. Since the two flow paths to Tuggerah Lake were of different length, with different discharges and were characterised by different bed roughnesses, the two tailgate settings were different. In addition, the tailgate settings for the 50 year flood and the 100 year floods were different from each other. This reflected not only the different flows but also the different Tuggerah Lake levels.

4.5 Model Tests And Results

Tests were conducted for both the 50 year flood and the 100 year flood. The quantities measured were the velocities and depths for both floods and the flow distributions for the 100 year flood.

i) The velocity field was measured throughout the model on a grid with a mesh spacing corresponding to 125m in the prototype. Due to the presence of arrays of roughness elements, velocity measurements were made by timing the movement of dye patches over a distance corresponding to 54 m. Hence the velocities reported in Figures 6 and 7 are spatially averaged <u>surface</u> velocities. The directions of the velocity vectors were measured using a compass with an estimated accuracy of ± 15°. Development in the flood plain has been included in these figures because some of the results were markedly affected by it.

ii) The water levels in the model were measured at ten fixed points throughout the model using electronic water level followers. The results for the two floods are contained in Figures 8 and 9.

iii) The flow distributions were determined for the in-bank flow and the left and right overbank areas through the three sections indicated in Figure 10. The flow through these sections (shown as dashed lines) were calculated by dividing them up into a number of straight line segments. The normal flow through each segment was found by measuring the depth and resolved normal velocity. Summing the normal flows through the segments then yielded the overbank flows.

The river flows at sections 1 to 3 in Figure 10 were measured over each cross-section using a mini-current meter mounted on a small traversing rig. Between 30 and 40 velocity measurements were made at various depths across each section.

4.6 Discussion Of Model Results

The velocities in Figures 6 and 7 are surface velocities. The ratios of V/V_s (where V = depth averaged velocity and V_s = surface velocity) were determined separately for the river flow and the over-bank flow for the 100 year flood. For the in-bank flow the velocity measurements across each of the three sections were considered sufficiently detailed to permit a reasonably accurate calculation of the channel flow. The average velocity was then determined and compared to the surface velocity measurement.

For the over-bank flow, the ratio of V/V_s was found using the measurements taken for the flow distributions in Figure 10. Firstly, the overbank discharge was obtained as the difference between the total inflow and the river inflow. From the overbank depths, overbank flow and the measured surface velocities, the values of the ratio V/V_s for each of the three sections was calculated for the overbank area. The results below were the average of these sections and are:

i) for the in-bank flow, $V/V_s = .81$
ii) for the overbank flow, $V/V_s = .85$

It is recommended that these same ratios be applied to the velocities for the 50 year flood (i.e. Figure 6).

The water levels are contained in Figures 8 and 9. For the 50 year flood, the downstream water levels near the two outflows were set to the reference levels 2.2 m. and 2.69 m (A.H.D.) which were taken from the results of the numerical model (Reference 1). A comparison of the upstream water levels predicted by the physical and numerical models for the <u>50 year flood</u> shows that the water levels predicted by the physical model were generally higher. The overall hydraulic gradients in the physical model are greater by 5% to 20%.

For the <u>100 year flood</u>, the downstream water levels near the two outflows were set to the reference levels 2.57 m. and 2.85 m.(A.H.D.). A comparison between the water levels from the physical model and the numerical model shows that the overall hydraulic gradients in the physical model were less by about 5% to 10%.

The numerical model HEC-II, which was used in the first study documented in Reference 1, is a strictly 1D model which did not have the facility for the representation of levees as flow controls. The physical model, however, did not have these limitations, although it did have some scale effects. These were caused by the overbank flow not being fully turbulent for the smaller 50 year flood. These scale effects were not generally present throughout the model and only occurred in patches. Moreover, it has been demonstrated in the Appendix that these effects were not significant.

Different roughness distributions were used in the numerical and physical models. The numerical model roughnesses were calibrated to the 1964 flood of 960 m^3/s. In doing this, it needs to be remarked that the flow paths over the flood plain were necessarily pre-defined. Hence the schematisation of the flows needed for the 1D numerical model would be inextricably coupled to the roughnesses obtained from the model calibration. The extent of the various overbank regions which were assigned particular Manning's 'n' values is not clear from the report (Reference 1). It needs to be stated, however, that the delimiting boundaries that were provided between the various zones of constant roughness in the numerical model did not correspond to those in the prototype - eg: regions which were modelled with constant roughness in the numerical model did not reflect the variations in roughness that were clearly evident in the prototype. It is considered that the roughness distribution adopted in the physical model was a more accurate reflection of the physical reality. A somewhat crude comparison between the (prototype) Manning's 'n' values used in the two models is tabulated below where all the values are scaled to the prototype values.

The unbracketed numbers refer to the 'n' values upon which the physical model was based (see Figure 5). The bracketed numbers refer to the 'n' values adopted in Reference 1 for the numerical model.

	IN-BANK ROUGHNESS	LEFT OVERBANK ROUGHNESS	RIGHT OVERBANK ROUGHNESS
Upstream Reach	.035 (.035)	.04 to 0.08 (0.08)	.035 (.08)
Middle Reach	.035 (.025)	.04 to 0.1 (.08)	.035 to .08 (.06)
Downstream Reach	.035 (.025)	.06 to 0.08 (.08)	.035 to 0.1 (.08)

The physical model was generally "rougher" than the numerical model in the in-bank regions, but not in the overbank regions.

5. CONCLUSIONS

Tests with the physical model were run for a 50 year flood and a 100 year flood to determine the velocity and depth fields. Due to the greater possibilities of more accurate representation of the physical processes, it is considered that the results of the physical model are more reliable than those of the numerical model reported in Reference 1.

The placement and attachment of the roughness elements to the model were not insignificant components of the model study cost. It was therefore decided to have the same spacing for both the 50 year and the 100 year floods. As a result, the roughness adopted in the model was underestimated for the 100 year flood and overestimated for the 50 year flood. This infers that for the 100 year flood, the depths are underestimated and the velocities overestimated and vice versa for the 50 year flood. It is considered that the errors were within the accuracy requirements for the model. Moreover, if the criterion upon which flood plain policy for development is to be formulated is one based on the product of depth and velocity, there is a tendency for the errors to cancel.

The velocity fields for the 50 year and 100 year floods in Figures 6 and 7 are <u>surface</u> velocities which were spatially averaged over (prototype) distances of 54 metres. In order to obtain depth averaged velocities, it is recommended that the surface velocities be scaled down to 81% for river velocities and 85% for overbank velocities.

6. REFERENCES

1. Lower Wyong River Flood Study, Public Works Department, N.S.W., January 1984, Report No. PWD 83020

2. Open-Channel Hydraulics, V.T. Chow, 1959, McGraw-Hill

SUMMARY

The Lower Wyong River is a flood prone river emptying into Tuggerah Lake about 70 km north of Sydney. Increased pressure for further development within the flood plain has necessitated an investigation of the flow conditions by a physical model. Model tests were made with the present level of development included in the model. Measurements of the velocity field were made with the steady state flows corresponding to the peak discharges for a 50 year flood and a 100 year flood. Flow distributions for the in-bank and overbank areas were also measured for the 100 year flood.

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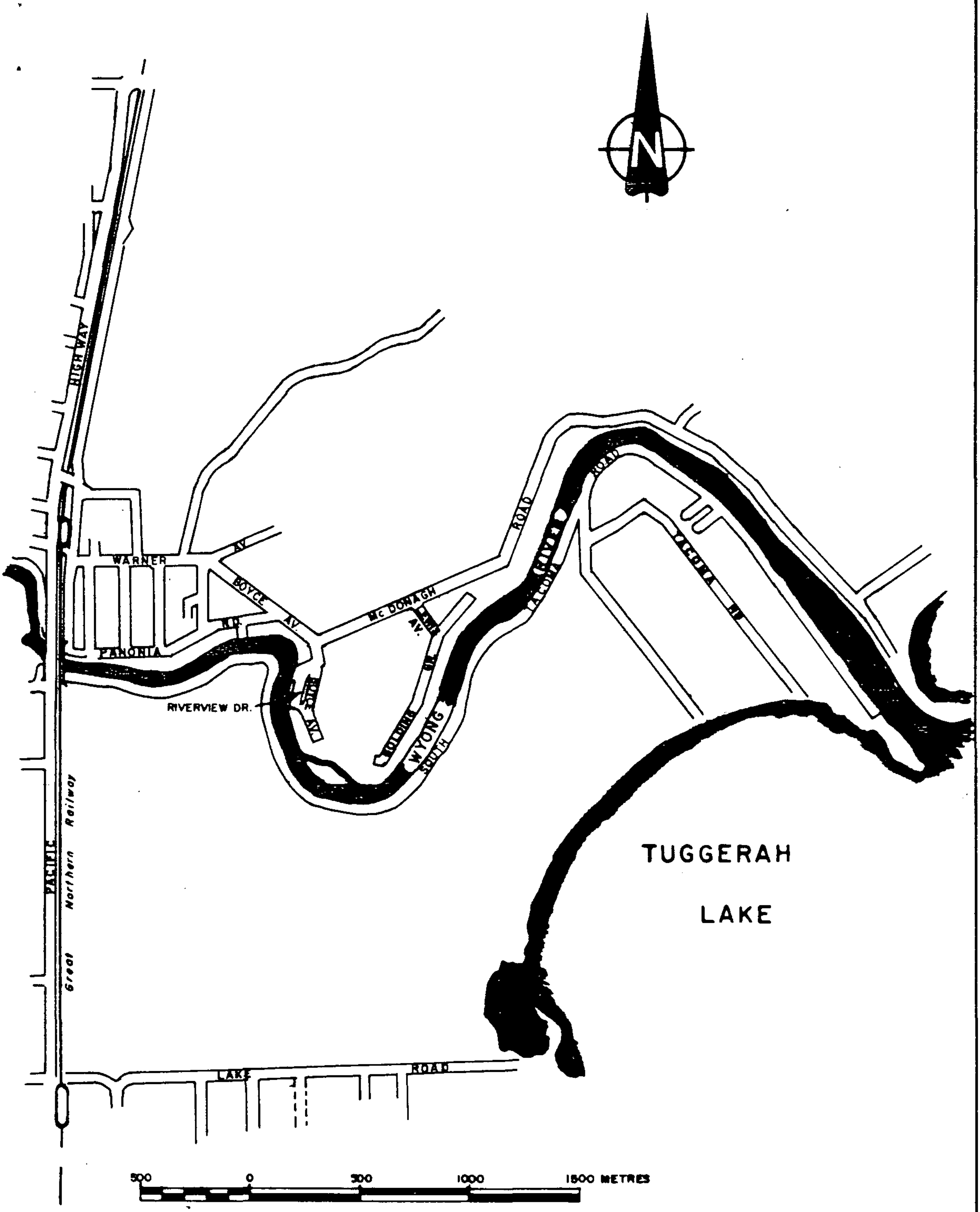
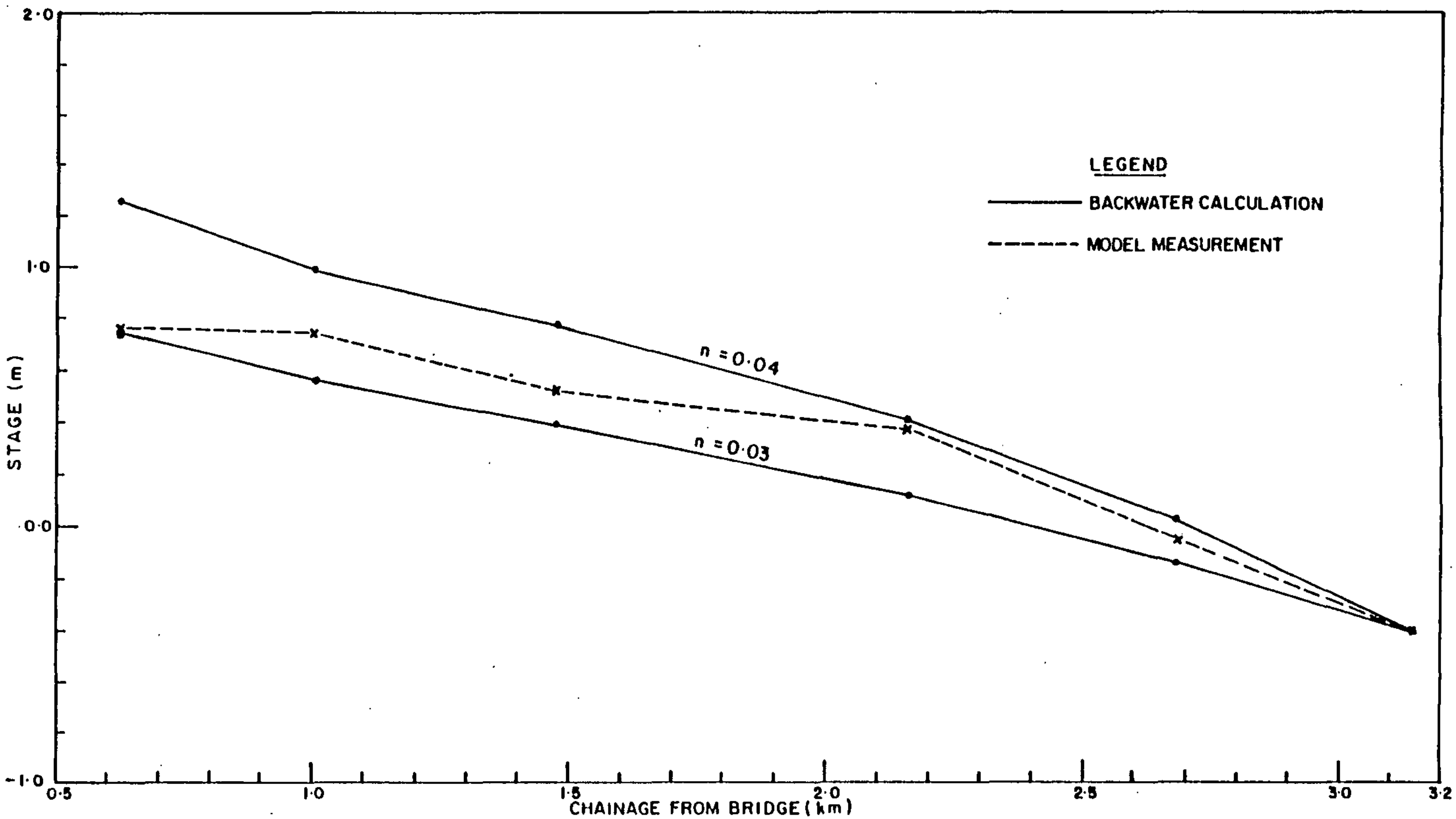
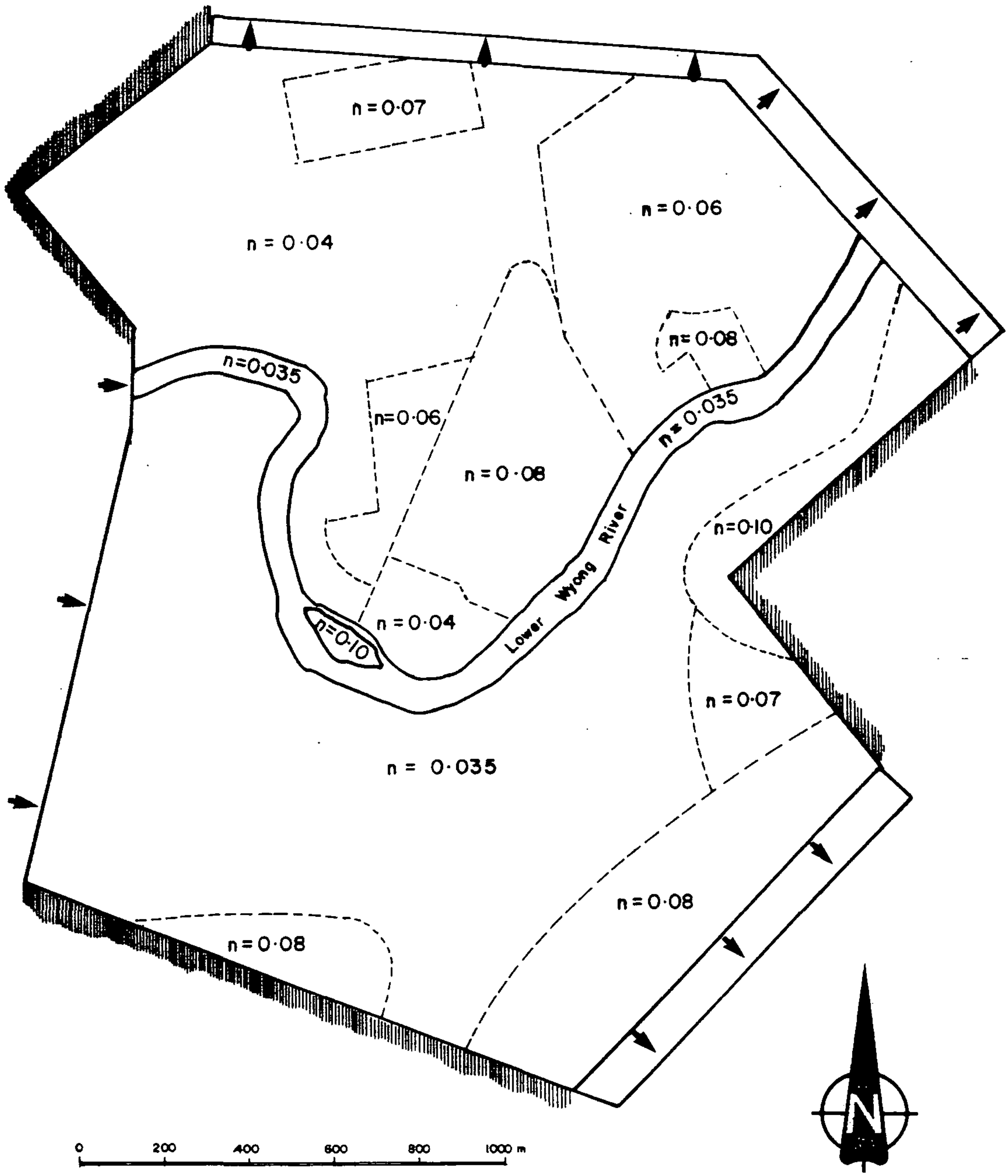


LOCALITY SKETCH

Figure 1

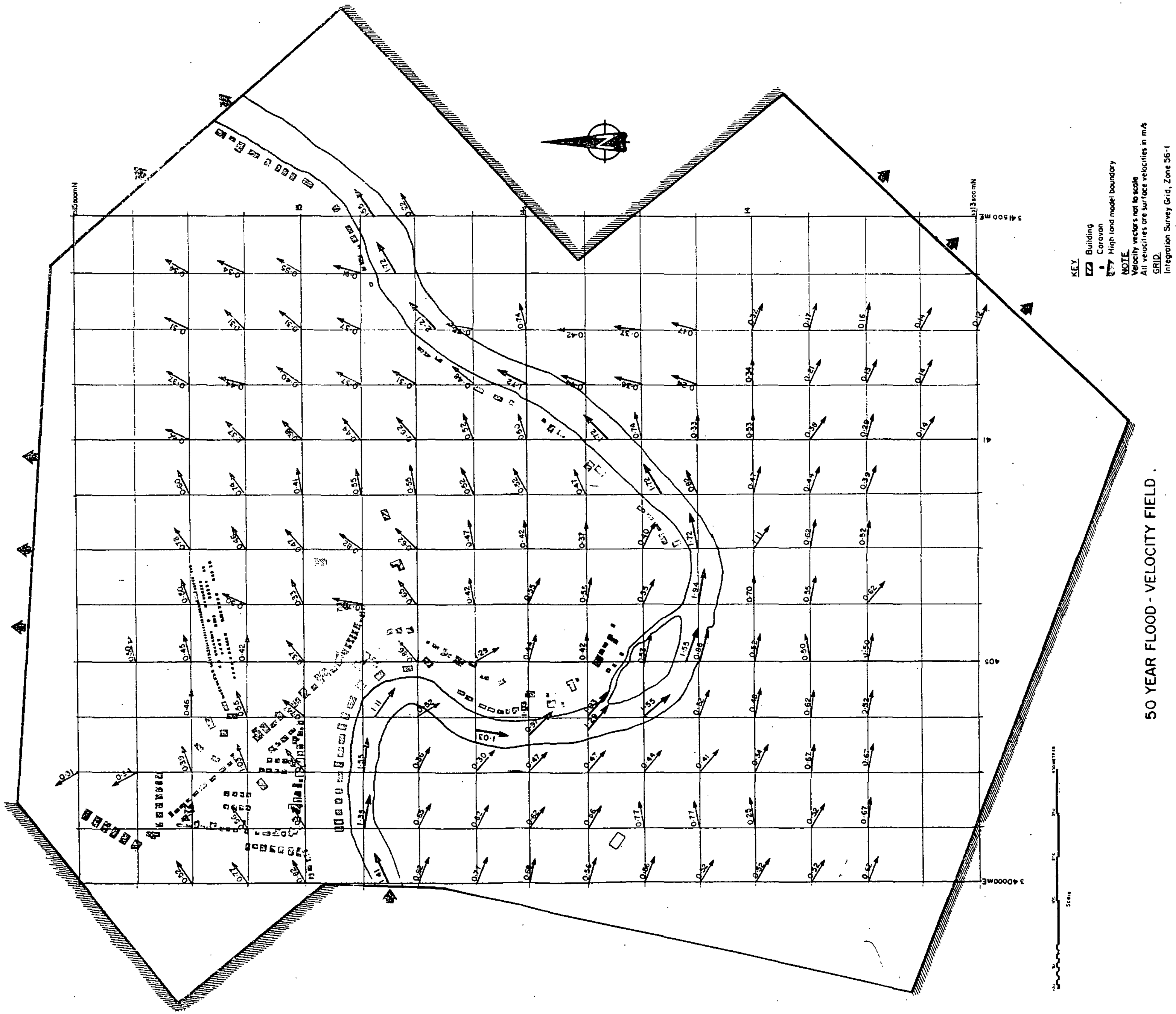


BACKWATER CURVES FOR IN-BANK ROUGHNESS : DISCHARGE = 402 m³/s
(Australian Height Datum)

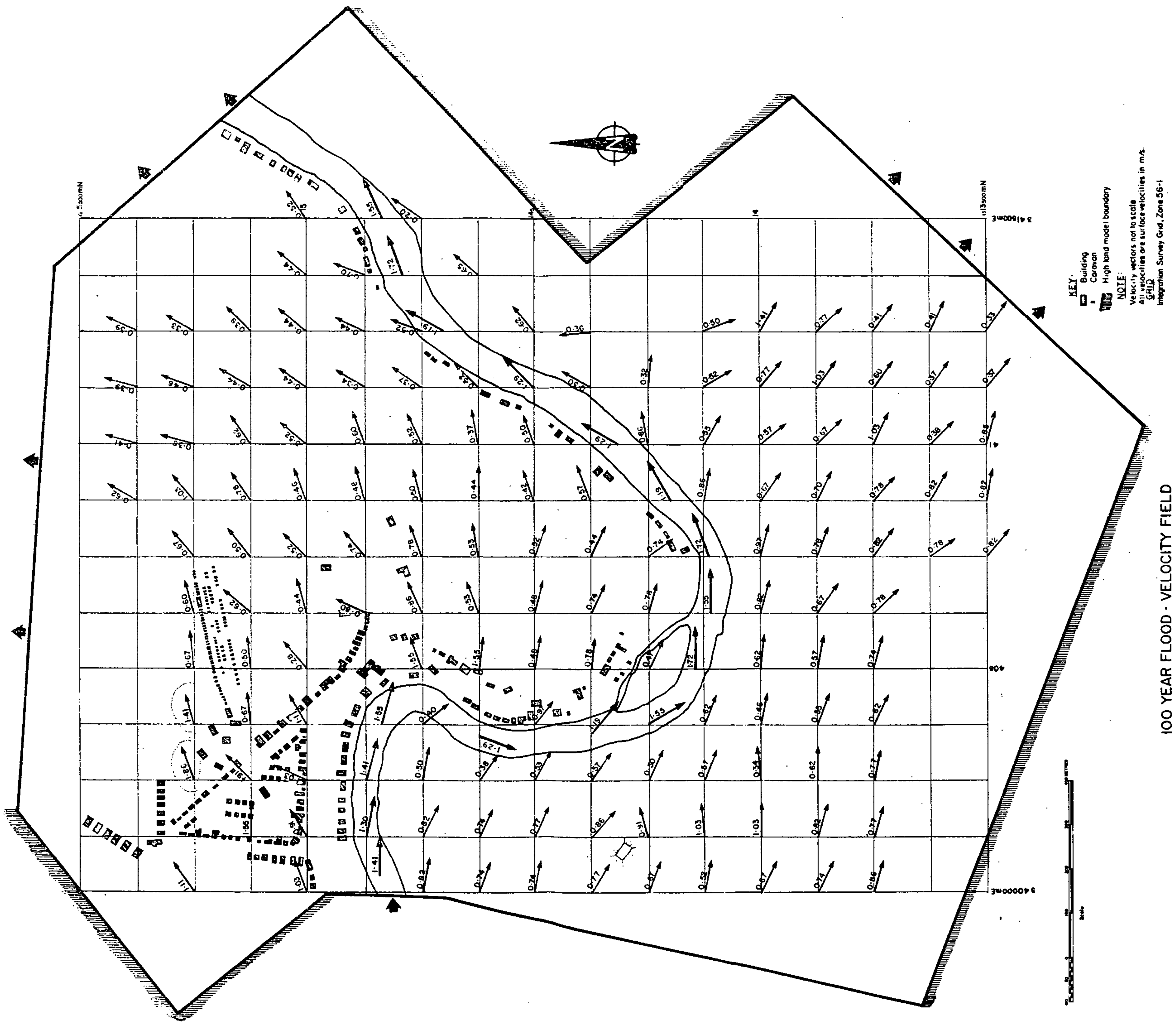


MANNING'S 'n' (PROTOTYPE) VALUES ADOPTED FOR MODEL STUDY

Figure 5

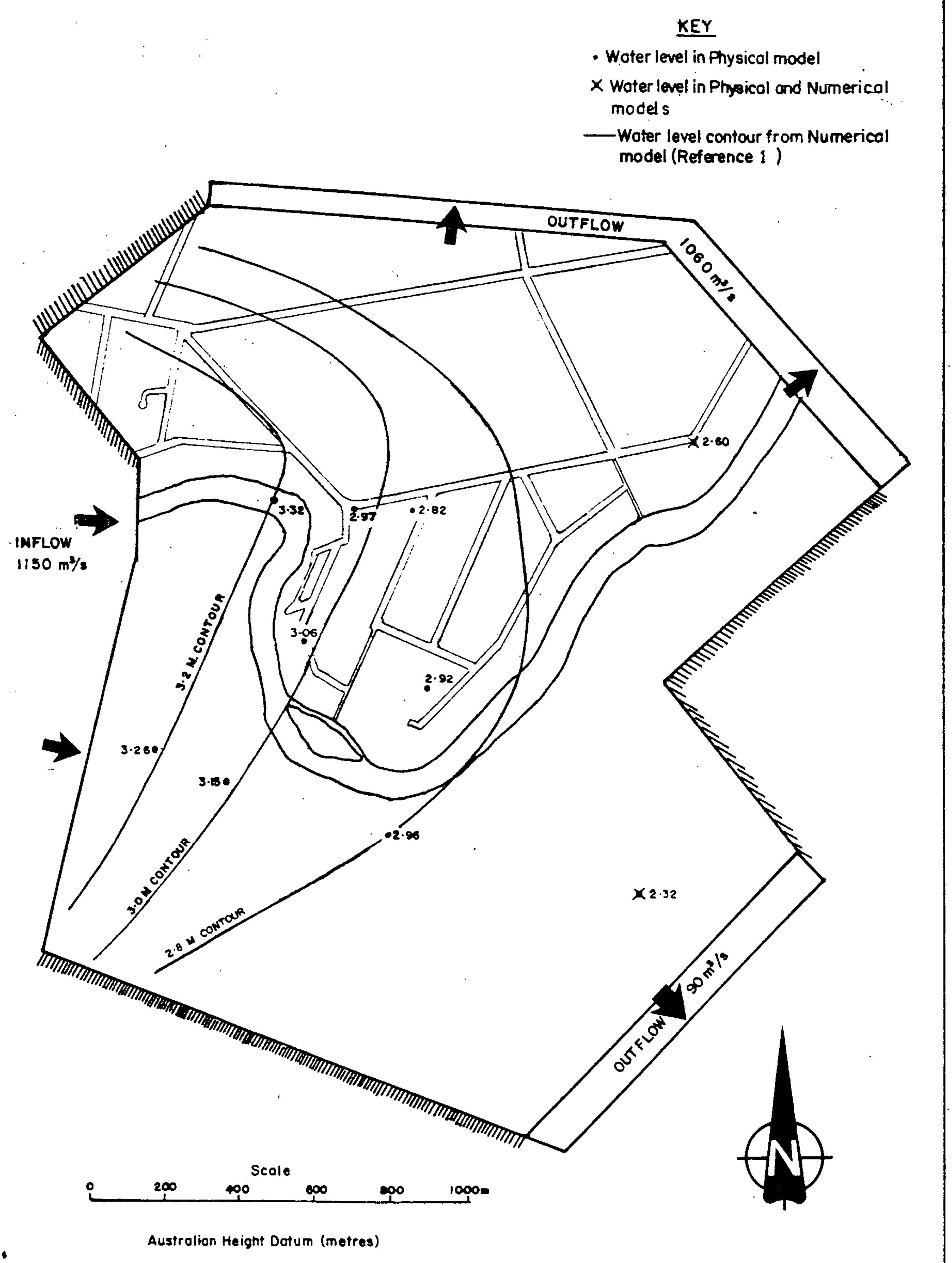


50 YEAR FLOOD - VELOCITY FIELD

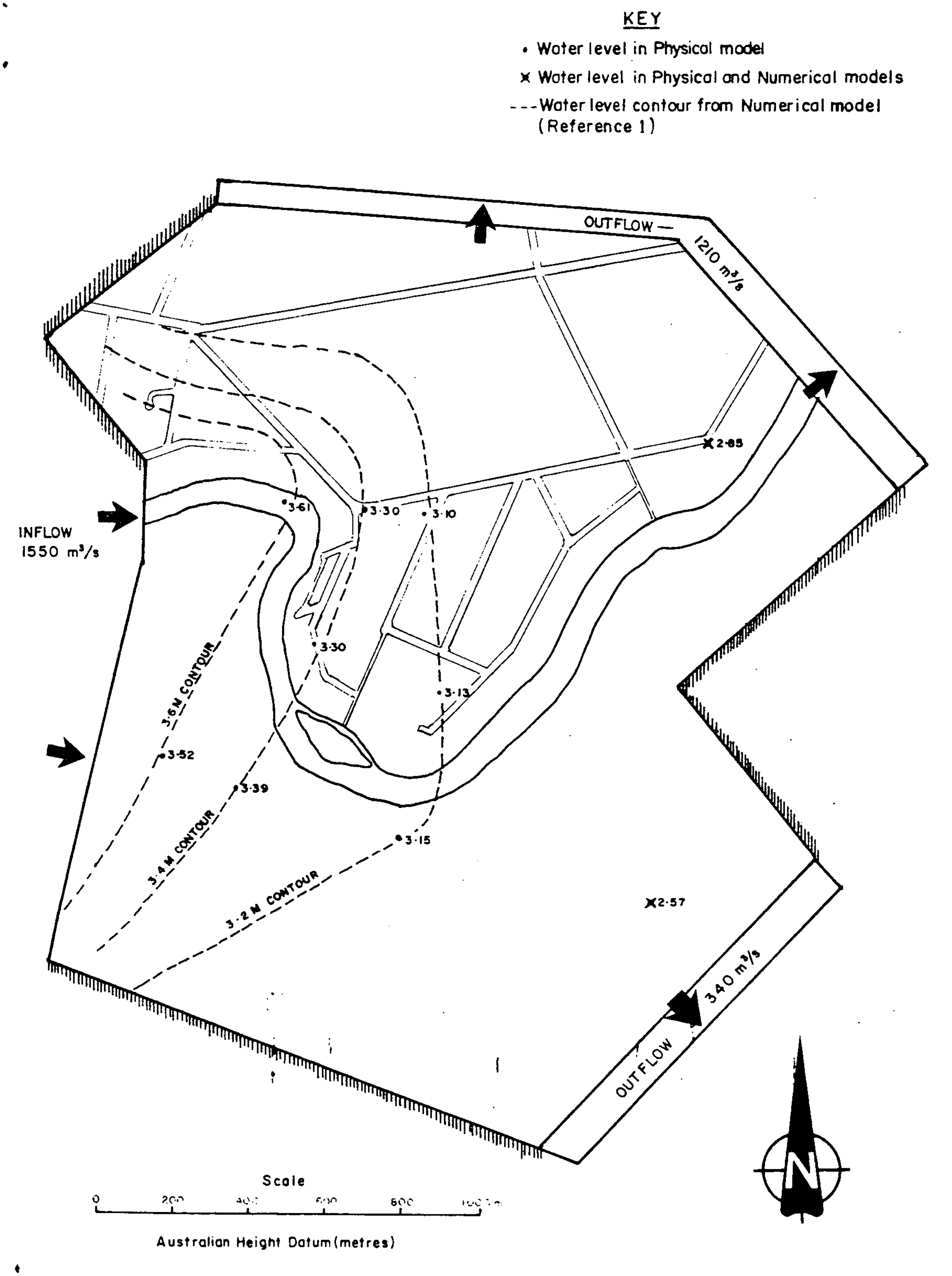


100 YEAR FLOOD - VELOCITY FIELD

Figure 7



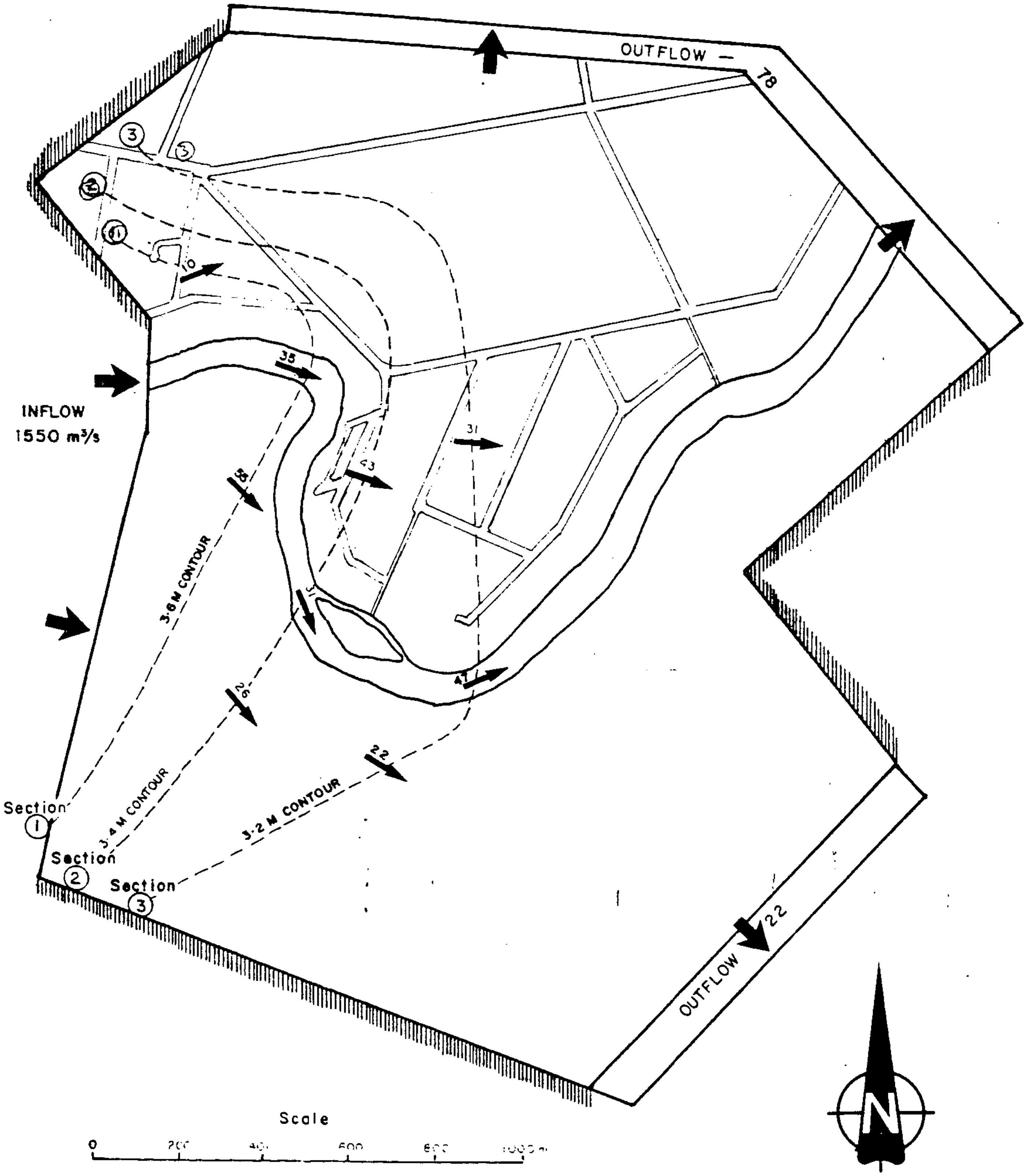
50 YEAR FLOOD LEVELS



100 YEAR FLOOD LEVELS

Figure 9

Note. Numbers refer to the percentages of left and right overbank flow and river flow



FLOW DISTRIBUTIONS FOR THE 100 YEAR FLOOD

Figure 10