Institutionen för vattenbyggnad
Chalmers Tekniska Högskola

Department of Hydraulics
Chalmers University of Technology

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SEDIMENT IN SEWERS
Research Leaves in England

PART ONE: WALLINGFORD, October-November 1989

PART TWO: NEWCASTLE, March-April 1990

Gustavo Perrusquia

Report Series B:52

Göteborg 1990

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Chalmers tekniska högskola
412 96 GÖTEBORG

Telefon: 031–72 10 00

ABSTRACT

Experiments were made in part-full pipes with a deposited sediment bed both at Hydraulics Research, Wallingford and the University of Newcastle upon Tyne in the United Kingdom. Tests were carried out using different pipe diameters, sand sizes and sediment thicknesses.

Discharge rates, flow depths, sediment transport rates and bedform dimensions were measured. The aim of these studies was to gather information for an oncomming report on the effects that the relative flow depth, the relative sediment depth, the relative grain size, the pipe diameter (in other words: shape factors) and the bedform dimensions produce on the sediment transport in part-full pipes.

The preliminary results showed that a functional relationship of the type $\Theta_b = f(\Phi_b)$ can be used to describe the transport capacity of such pipes.

PREFACE

During the fall of 1987 contacts were established between the Department of Hydraulics at Chalmers University of Technology and both Hydraulics Research Ltd at Wallingford and the University of Newcastle upon Tyne in the United Kingdom. The aim was to start technical co-operation in the field of Sediment in Sewers.

After performing an extensive series of experiments at the Department of Hydraulics, I was able to spend six weeks at Hydraulics Research in the fall of 1989 and five weeks at the University of Newcastle in the spring of 1990. The purpose of these visits was to do some experimental work to investigate among other things whether scale effects have any influence in the rate of sediment transport in pipe channels with a deposited bed. Different pipe diameters, pipe materials, sand sizes and sediment bed thicknesses were used. Only bed load transport and part-full flow were studied. This report contains exclusively the work carried out during both visits. The analysis of results from these experiments will be done and published in my Doctor's thesis.

These trips were possible thanks to funds from the Swedish Council for Building Research (BFR) through the Urban Geohydrology Research Group. Also Åke and Greta Lisshed Foundation and Chalmers Research Fund provided financial support which is gratefully acknowledged.

I wish to thank my hosts Mr. Richard May at Hydraulics Research and Dr. Chandra Nalluri at the University of Newcastle for all the facilities they gave me to conduct my experiments.

Göteborg, September 1990

Gustavo Perrusquia

LIST OF CONTENTS

		Page
ABSTRACT		i
PREFACE		ii
LIST OF CONTENTS		iii
PART ONE: WALLINGFORD		1
1.	EXPERIMENTAL PROCEDURE	2
1.1	Calibration of sediment supply	2
2.	EXPERIMENTAL RESULTS	7
PART TWO: NEWCASTLE		12
3.	EXPERIMENTAL PROCEDURE	13
4.	EXPERIMENTAL RESULTS	14
5.	SUMMARY AND CONCLUSION	18
APPENDIX I:		
	Longitudinal bed profiles, Wallingford	20
APPENDIX II:		
	Longitudinal bed profiles, Newcastle	30

PART ONE: WALLINGFORD, October-November 1989

1. EXPERIMENTAL PROCEDURE

The experiments were carried out in a 21 m long, 450 mm diameter concrete pipe placed on a tilting flume. The pipe was constructed of 2.52 m long sections of spun concrete. Each pipe length had two 900 x 90 mm slots cut along the pipe's crown. Water was recirculated between two tanks, one at each end of the pipe. The water discharge rates were measured using a 1.234 m wide rectangular thin plate weir. Sediment was also recirculated by a slurry pump and the velocity measured using an electromagnetic current meter. The sediment concentration was measured using an infrared sensor. A layout of the test rig is shown in Figure 1. Flow depths were measured using five digital depth gauges along the pipe length. Uniform flow conditions were gradually established with the help of an adjustable sluice gate at the downstream end. Sediment bed thickness and bedform dimensions were measured using a digital point gauge while sediment bed width using a ruler. The sediment used was sand with D_{50} = 0.72 mm. A more detailed description of the test arrangement can be seen in a report by Hare (1988) from which Figure 1 has been taken.

The water pump was turned on first and the slurry pump was not turned on until the water had reached the downstream end. In this way a constant head over the slurry pump was kept. Each test was allowed to run for several hours so that bedforms occurred along the whole pipe length. No suspended load was detected. Both pumps were turned off while the sluice gate was shut off so that the bedforms were undisturbed. The pipe was then slowly emptied of all water and the bed profile measured the following day.

1.1 Calibration of sediment supply

Before the experimental series was started, it was necessary to calibrate the infrared sensor which measures sediment concentration. This was done for a range of velocities in the pipe that transported the mixture sand-water. The infrared light source was installed on the outside of a 1 m long transparent PVC section and the sensor mounted opposite the source, see Figure 2. The signal detected by the sensor and modified by the amount of sand passing along the pipe was fed to an amplifier unit and converted to a voltage. The signal could be monitored by both a chart recorder and a counter. Figure 3 shows a schematic layout of sensor equipment. The calibrations were done according to the following steps:

1) A pre-weighed amount of sand was supplied to the slurry pump over a given period of time. The rate of supply was converted to kilograms per second.

2) Knowing the rate of supply and the discharge in the sediment pipe (using the electromagnetic current meter) the concentration in the sediment pipe could be computed as

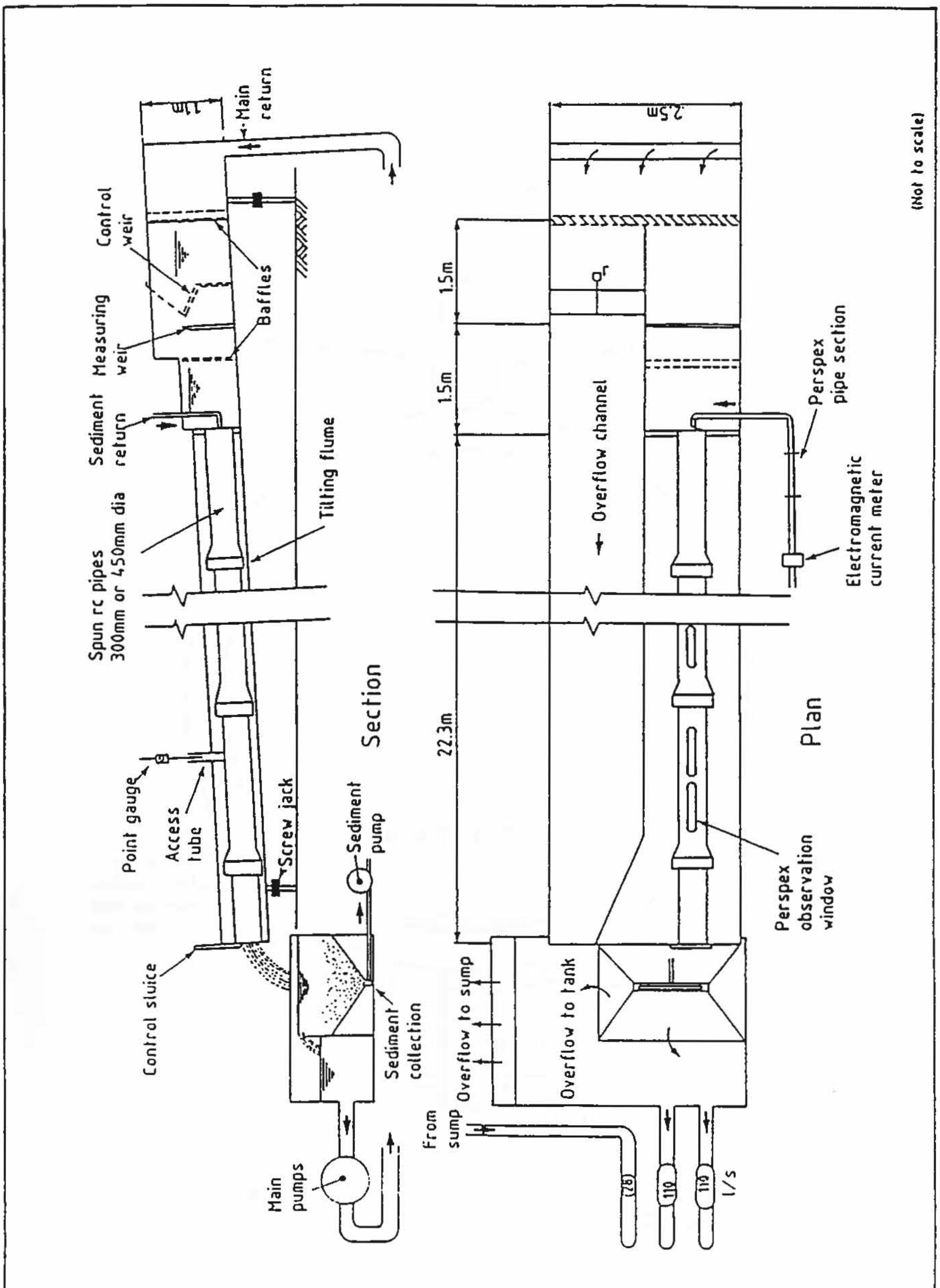

Figure 1. Layout of the test rig for the 450 mm pipe. (Published with permission, HR Wallingford)

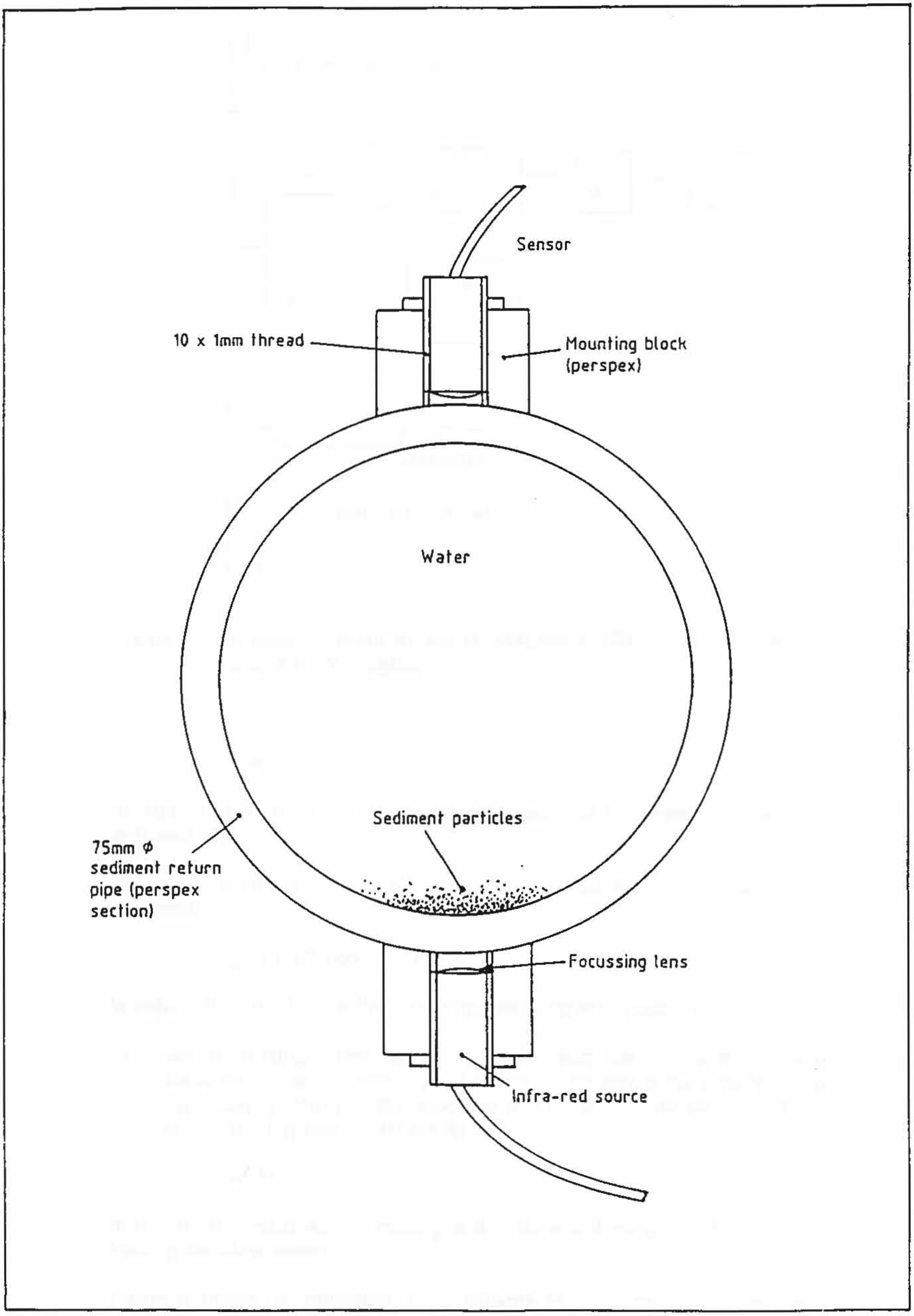

Figure 2. Cross section through sediment sensor. (Published with permission, HR Wallingford)

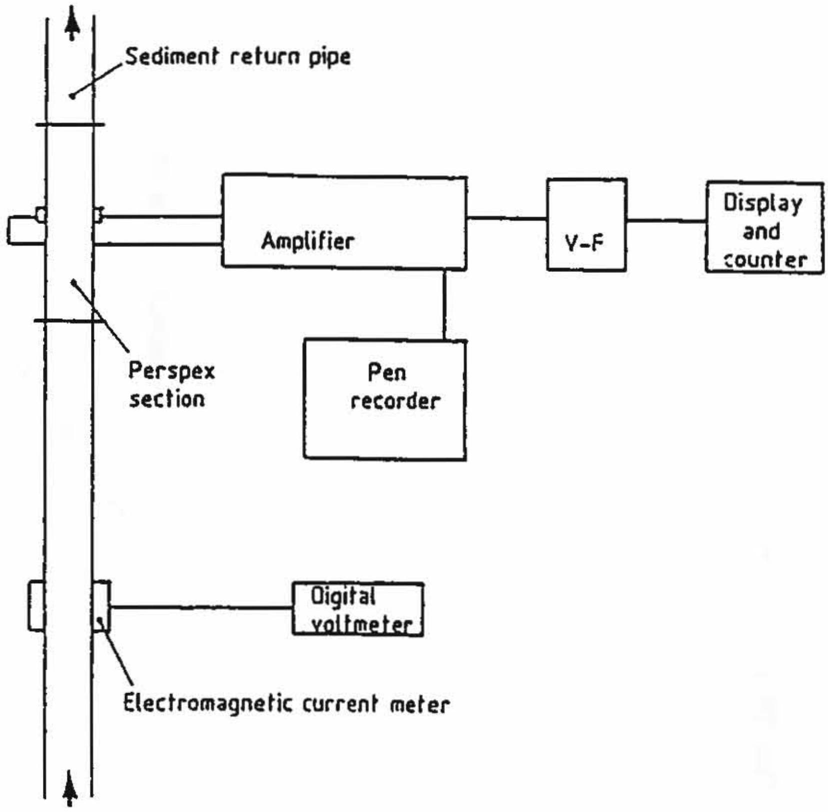

Figure 3. Schematic layout of sensor equipment. (Published with permission, HR Wallingford)

$$C = T_b / Q_p \qquad (1)$$

in which C = concentration, T_b = transport rate and Q_p = water discharge in sediment pipe.

The water discharge in the sediment pipe Q_p, could be computed using the relationship

$$Q_p = 0.003065\ (V-2)\ m^3/s \qquad (2)$$

in which V = reading on the electromagnetic current meter in volts.

3) Sensor readings were made for each particular concentration and discharge in the sediment pipe. This was done during the time the sand was being supplied to the pipe and at 100-second intervals. Then the sensor reading was expressed as

$$(M-M_c)/1000 \qquad (3)$$

in which M = mean sensor reading with sediment flowing and M_c = sensor reading for clear water.

Figure 4 shows the calibration for a velocity of 2.775 m/s in the sediment pipe.

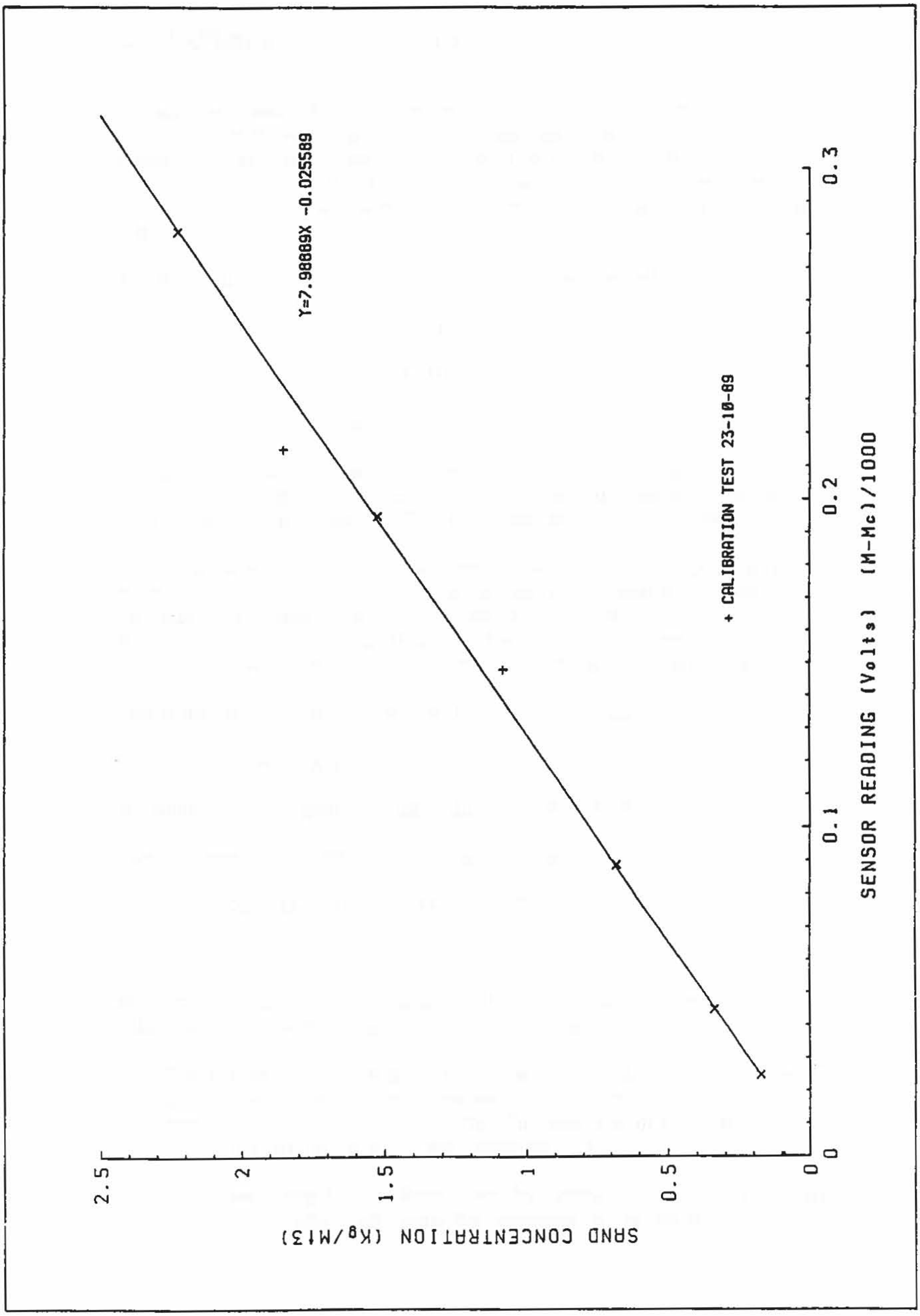

Figure 4. Sensor calibration at 2.775 m/s, Q = 12.26 l/s, ECM = 6 V. (Published with permission, HR Wallingford)

2. EXPERIMENTAL RESULTS

It took two weeks to complete the calibrations. On the third week two preliminary runs were made just to test the condition when a deposited bed builds along the pipe invert. It was found that the best calibration curve was the one shown in Figure 4 because the range of concentrations measured were within the region where the sensor gives an approximately linear response.

The water discharge over the weir was found using the following relationship

$$Q_w = 2/3\ (2g)^{1/2}\ C_0\ B\ H_e^{3/2} \qquad (4)$$

$$C_0 = 0.602 + 0.083\ (H/P) \qquad (5)$$

$$H_e = H + 0.0012 \qquad (6)$$

in which g is the acceleration of gravity, C_0 is the discharge coefficient, B is the width of the weir = 1.234 m, H_e is the energy head over the weir, H is the head over the weir and P is the height of the weir = 0.807 m.

The mean sensor reading with sediment flowing, M, was found taking the mean value of ten consecutive readings, each representing the mean concentration for a 1000 s period. The sensor reading for clear water M_c was fairly constant and equal to 0.22 V. The calibration curve could then be used to find the value of the concentration in the sediment pipe C in kg/m^3.

The solid discharge was computed using the expression

$$Q_s = C\ Q_p\ /\rho_s \qquad (7)$$

in which ρ_s is the density of the sediment = 2650 kg/m^3.

The volumetric concentration was finally computed as

$$C_v = Q_s \cdot 10^6\ /\ \underbrace{(Q_p+Q_w)}_{Q} \qquad (8)$$

Bedform dimensions (height and length) were also measured. The mean values were computed using the following criteria:

1) The height of a particular bedform was taken as the difference in elevation between the bedform crest and the mean of the elevations of the troughs at the ends of the bedform. The mean height is the mean of all individual heights taken on the centerline of the sediment bed.

2) The mean length was determined by averaging the length of all individual lengths taken along the centerline of the sediment bed.

Table 1 summarizes the results. All tests were carried out with the pipe flowing below half-full. The sediment thickness was kept reasonably constant and consequently the sediment width did not present large variations. The column denominated "TOTAL DEPTH" is the sum of the sediment thickness and the flow depth. The water temperature was kept relatively constant. The most significant longitudinal bed profiles are shown in Appendix I.

No further analysis of the results is done in this report because the limited amount of data is unsufficient to study scale effects. This will be done in conjunction with the main series of experiments conducted by the author (CTH runs). However, the relationship between the mobility number Θ_b and the transport parameter Φ_b is shown in Figure 5. The mobility number is defined as

$$\Theta_b = R_b \, S \, / \, (s-1) \, D_{50} \tag{9}$$

in which R_b = hydraulic radius of the bed; S = hydraulic gradient; s = specific density; and D_{50} = particle diameter of bed material 50% being finer. The hydraulic radius of the bed was determined using the side wall elimination procedure of Einstein (1942).

The transport parameter is defined as

$$\Phi_b = q_b \, / \, [(s-1)g \, D_{50}^3]^{1/2}$$

or (10)

$$\Phi_b = C_v Q \, / \, P_b \, [(s-1)g \, D_{50}^3]^{1/2}$$

in which q_b = unit solid discharge; g = acceleration of gravity; and P_b = wetted perimeter of the sediment bed. C_v is the volumetric sediment concentration as defined in Equation (8) and Q is the total flow discharge. The so called CTH runs were performed using different methodologies from those of the Wallingford runs. Also, pipe and sediment dimensions were different. The pipe diameter in the Wallingford runs was double the size of the pipe used in the CTH runs. Even though no shape factors have been included for plotting the points shown in Figure 5, a preliminary curve can be drawn to fit the data.

A complementary set of results is summarized in Table 2.

TABLE 1. EXPERIMENTAL DATA, WALLINGFORD RUNS, OCT–NOV 1989

D = 450 mm ; D_{50} = 0.72 mm

RUN	SLOPE	SEDIM THICK	SEDIM WIDTH	DISCHARGE	FLOW DEPTH
	$^o/oo$	mm	mm	l/s	mm
1	1	73	332	16.5	87
2	1	87	355	26.4	123
3	1	75	335	39.1	204
4	2	80	344	24.5	121
5	2	80	344	31.4	145
6	2	87.5	356	48.6	214
7	3	78	340	20.7	134
8	3	86	354	36.3	168
9	4	83	349	26.2	115

RUN	TOTAL DEPTH	WATER TEMP	TRANSP RATE	VOL SED CONC	SOLID DISCH
	mm	^oC	kg/s	ppm	kg/m^3
1	160	15	0.0120	276	0.9794
2	210	14	0.0093	133	0.7557
3	279	15	0.0039	38	0.3187
4	201	13	0.0141	217	1.1448
5	225	13	0.0143	171	1.1600
6	301.5	13	0.0097	75	0.7900
7	212	13	0.0154	281	1.2526
8	254	13	0.0100	104	0.8132
9	198	13	0.0210	303	1.7100

RUN	BEDFORM DIMENSIONS HEIGHT	LENGTH
	mm	mm
1	7.4	347
2	12.6	236
3	–	–
4	13.1	396
5	19.7	386
6	28.6	463
7	33.1	567
8	34.1	718
9	37.7	673

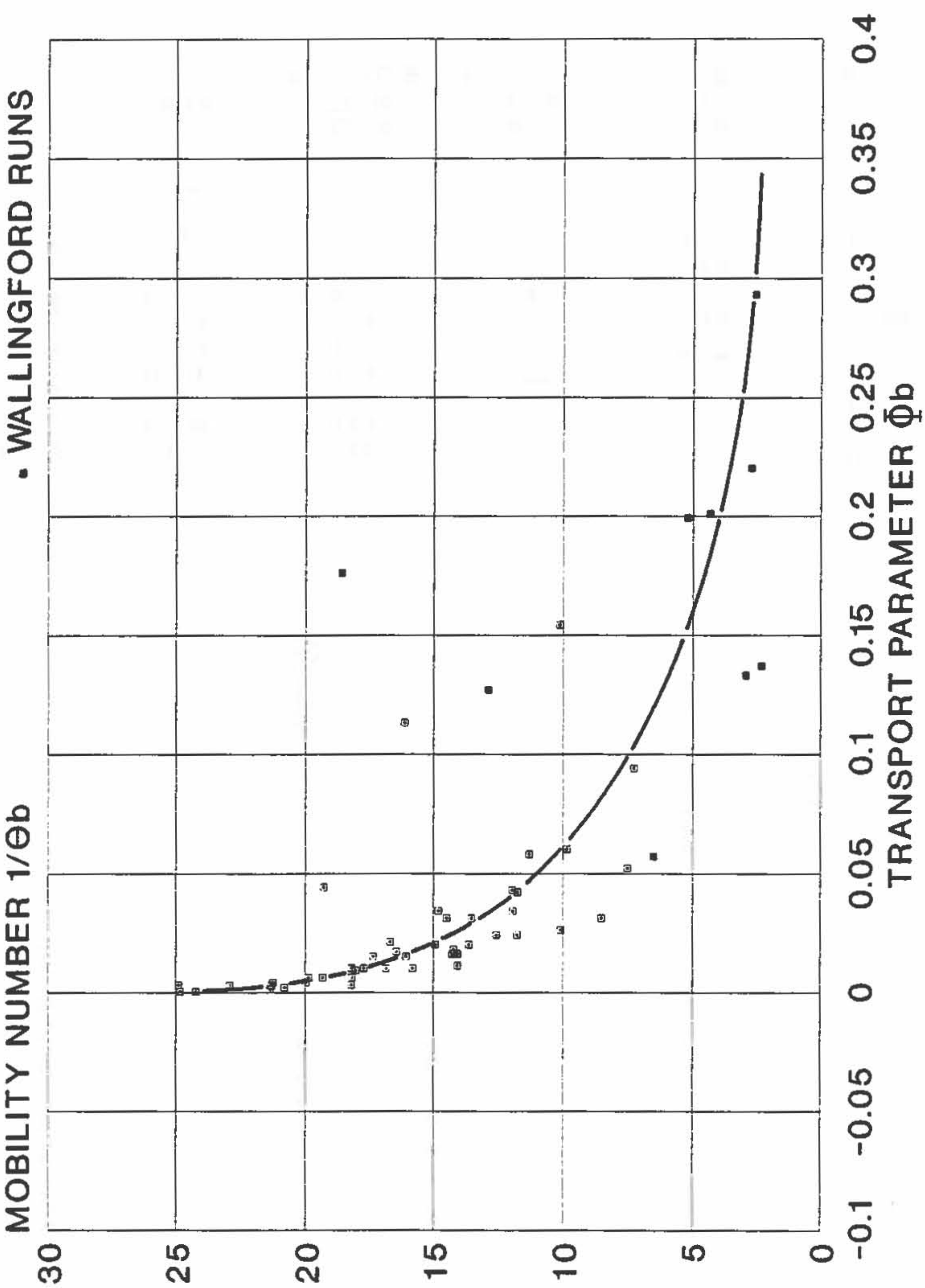

Figure 5. Transport parameter versus mobility number for all CTH and Wallingford runs.

TABLE 2. COMPLEMENTARY EXPERIMENTAL DATA, WALLINGFORD

RUN	FLOW VELOC	RELATIVE SEDIM DEPTH	RELATIVE FLOW DEPTH	INVERSE MOBILITY NUMBER	TRANSP PARAM
	m/s				
1	0.486	0.16	0.19	18.6	0.176
2	0.516	0.19	0.27	12.9	0.127
3	0.454	0.17	0.45	6.50	0.057
4	0.494	0.18	0.27	5.19	0.199
5	0.520	0.18	0.32	4.32	0.201
6	0.531	0.19	0.48	2.93	0.133
7	0.375	0.17	0.30	2.71	0.220
8	0.509	0.19	0.37	2.31	0.137
9	0.555	0.18	0.26	2.57	0.293

PART TWO: NEWCASTLE, March-April 1990

3. EXPERIMENTAL PROCEDURE

The experiments were carried out in a 20 m long, 154 mm diameter PVC pipe placed on a tilting frame. Water was recirculated between two tanks, one at each end of the pipe.

Water discharge rates were measured using a V-notch according to the following relationship

$$Q = 1.365\ (H + 0.002)^{2.5} \qquad (11)$$

in which H is the head over the V-notch. Sediment rates were measured by a sediment trap located just downstream of the sediment test section. A layout of the test rig is shown in Figure 6. Flow depths were measured using nine point gauges along the pipe length. Uniform flow conditions were gradually established with the help of a tail gate at the downstream end; in the cases this was not possible the slopes were corrected by using non-uniform flow equation. Sediment was supplied by a feeder just upstream of the sediment test section. Bedform dimensions were measured using a specially designed ruler. The sediment used was sand with D_{50} = 1.0 mm and a density ρ_s = 2593 kg/m^3.

After establishing uniform flow each test was allowed to run for some time until bedforms occurred along the sediment test section. At first, sediment was supplied by simply checking that at the beginning of the test section neither erosion nor accumulation of sediment occurred. Later, once samples had been taken and weighed while running each test, the feeder was adjusted to supply the same rate of sediment that was falling into the sediment trap. This process took several hours.

The pump was turned off while the tail gate was shut off so that the bedforms were undisturbed. The pipe was slowly drained and the bed profile measured later.

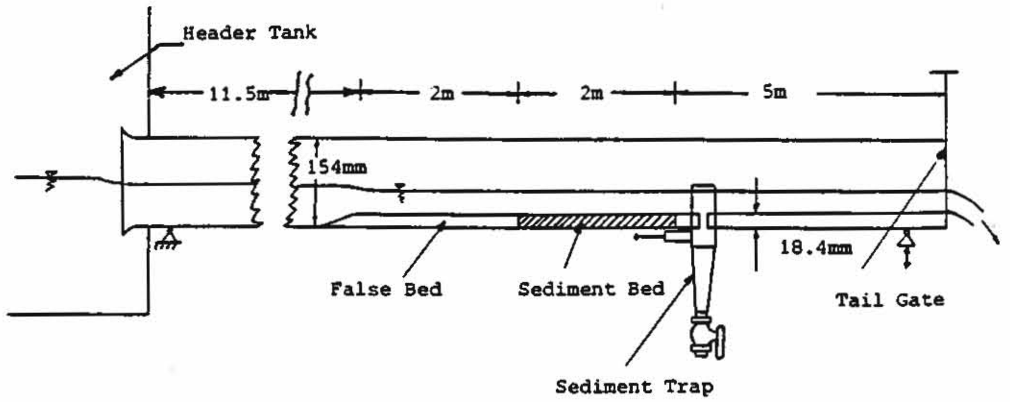

Figure 6. Experimental layout for the 154 mm diameter pipe. (Published with permission, University of Newcastle)

4. EXPERIMENTAL RESULTS

The parameters presented in Table 3 are similar to those shown in Table 1. Both sediment thickness and width are considered constant because the false bed had fixed dimensions and the average values for the loose sediment bed were considered to be equal to those of the false bed. This was decided after the longitudinal bed profiles had been analyzed, see Appendix II for the most significant bed profiles.

Figure 7 shows the plot of the mobility number against the transport parameter as defined by Equations (9) and (10) along with the preliminary curve drawn in Figure 5. Table 4 contains a complementary set of results.

The scatter in Figure 7 is considerably less than in Figure 5. It should be mentioned that the pipe diameter in the Newcastle runs is almost two thirds the size of the pipe used in the CTH runs. However, no consideration has been taken with regard to any scale or shape factor in plotting Figure 7.

TABLE 3. EXPERIMENTAL DATA, NEWCASTLE RUNS, MAR–APR 1990

$D = 154$ mm ; $D_{50} = 1.00$ mm

RUN	SLOPE	SEDIM THICK	SEDIM WIDTH	DISCHARGE	FLOW DEPTH
	$^o/oo$	mm	mm	l/s	mm
1	2.4	41.5	136.7	1.64	33.2
2	2.4	41.5	136.7	3.29	55.0
3	2.3	41.5	136.7	4.93	79.4
4	3.2	41.5	136.7	2.10	35.8
5	2.6	41.5	136.7	4.92	65.3
6	3.0	41.5	136.7	5.04	66.9
7	3.8	41.5	136.7	2.64	40.4
8	3.8	41.5	136.7	2.32	36.4
9	1.9	41.5	136.7	1.68	35.4
10	2.0	41.5	136.7	4.43	70.8
11	2.1	41.5	136.7	2.75	47.8

RUN	TOTAL DEPTH	WATER TEMP	TRANSP RATE	VOL SED CONC	SOLID DISCH
	mm	^oC	kg/s	ppm	kg/m^3
1	74.70	18	0.0003	78	0.1989
2	96.50	18	0.0011	133	0.3371
3	120.9	18	0.0014	111	0.2812
4	77.30	18	0.0009	156	0.3961
5	106.8	18	0.0023	179	0.4541
6	108.4	18	0.0024	186	0.4724
7	81.90	20	0.0015	217	0.5498
8	77.90	20	0.0017	280	0.7100
9	76.90	20	0.0002	34	0.0874
10	112.3	20	0.0013	113	0.2871
11	89.30	18	0.0006	84	0.2135

RUN	BEDFORM HEIGHT	DIMENSIONS LENGTH
	mm	mm
1	–	–
2	–	–
3	4.3	543
4	–	–
5	4.6	768
6	4.9	256
7	–	–
8	–	–
9	–	–
10	4.1	359
11	–	–

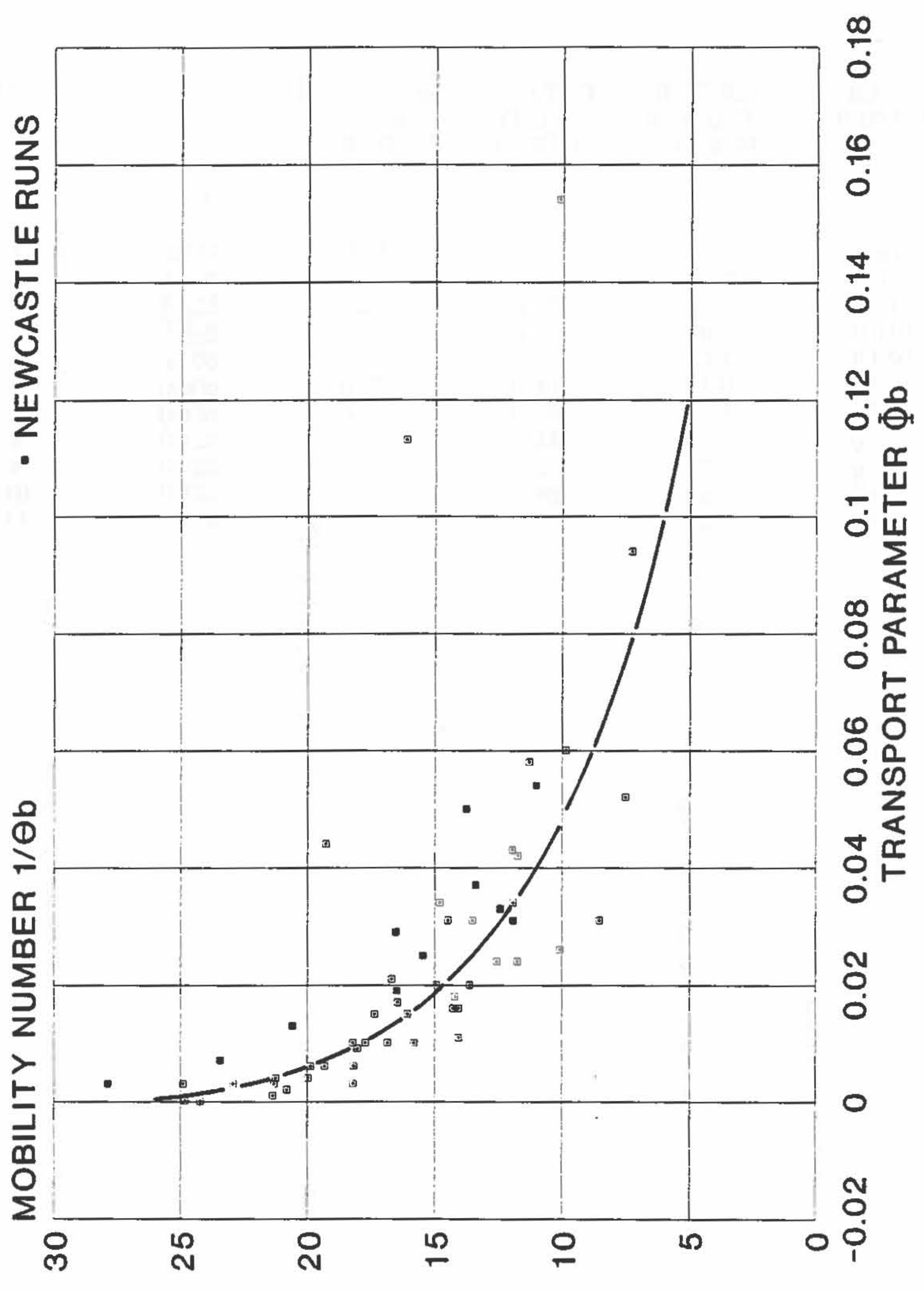

Figure 7. Transport parameter versus mobility number for all CTH and Newcastle runs.

TABLE 4. COMPLEMENTARY EXPERIMENTAL DATA, NEWCASTLE

RUN	FLOW VELOC m/s	RELATIVE SEDIM DEPTH	RELATIVE FLOW DEPTH	INVERSE MOBILITY NUMBER	TRANSP PARAM
1	0.334	0.27	0.22	23.4	0.007
2	0.399	0.27	0.36	15.5	0.025
3	0.423	0.27	0.52	11.9	0.031
4	0.395	0.27	0.23	16.5	0.019
5	0.505	0.27	0.42	13.8	0.050
6	0.506	0.27	0.43	11.0	0.054
7	0.439	0.27	0.26	12.4	0.033
8	0.429	0.27	0.24	13.4	0.037
9	0.320	0.27	0.23	27.9	0.003
10	0.422	0.27	0.46	16.5	0.029
11	0.384	0.27	0.31	20.6	0.013

5. SUMMARY AND CONCLUSIONS

Transport capacity and bedform dimensions in part-full pipes with a deposited bed have been measured. Experiments were performed both at Hydraulics Research, Wallingford and the University of Newcastle upon Tyne using the facilities listed below.

Series	Pipe diameter mm	Sand size mm	Number of runs	Slope range °/₀₀	Relative sediment depth
WALLINGFORD	450	0.72	9	1-4	0.18
NEWCASTLE	154	1.00	11	2-4	0.27

The preliminary results show that a functional relationship of the type $\Theta_b = f(\Phi_b)$ can be used to describe the transport capacity in such pipes as shown in Figure 8. Even before any shape factors are introduced it can be seen that the preliminary results fit fairly well to those obtained by the author in an extensive series of experiments (CTH runs).

The effects (if any) that the relative flow depth, the relative sediment depth, the relative grain size, the pipe diameter and the bedform dimensions produce on the sediment transport rate will be studied in an oncoming report. To that purpose, the results published herein will be extremely helpful.

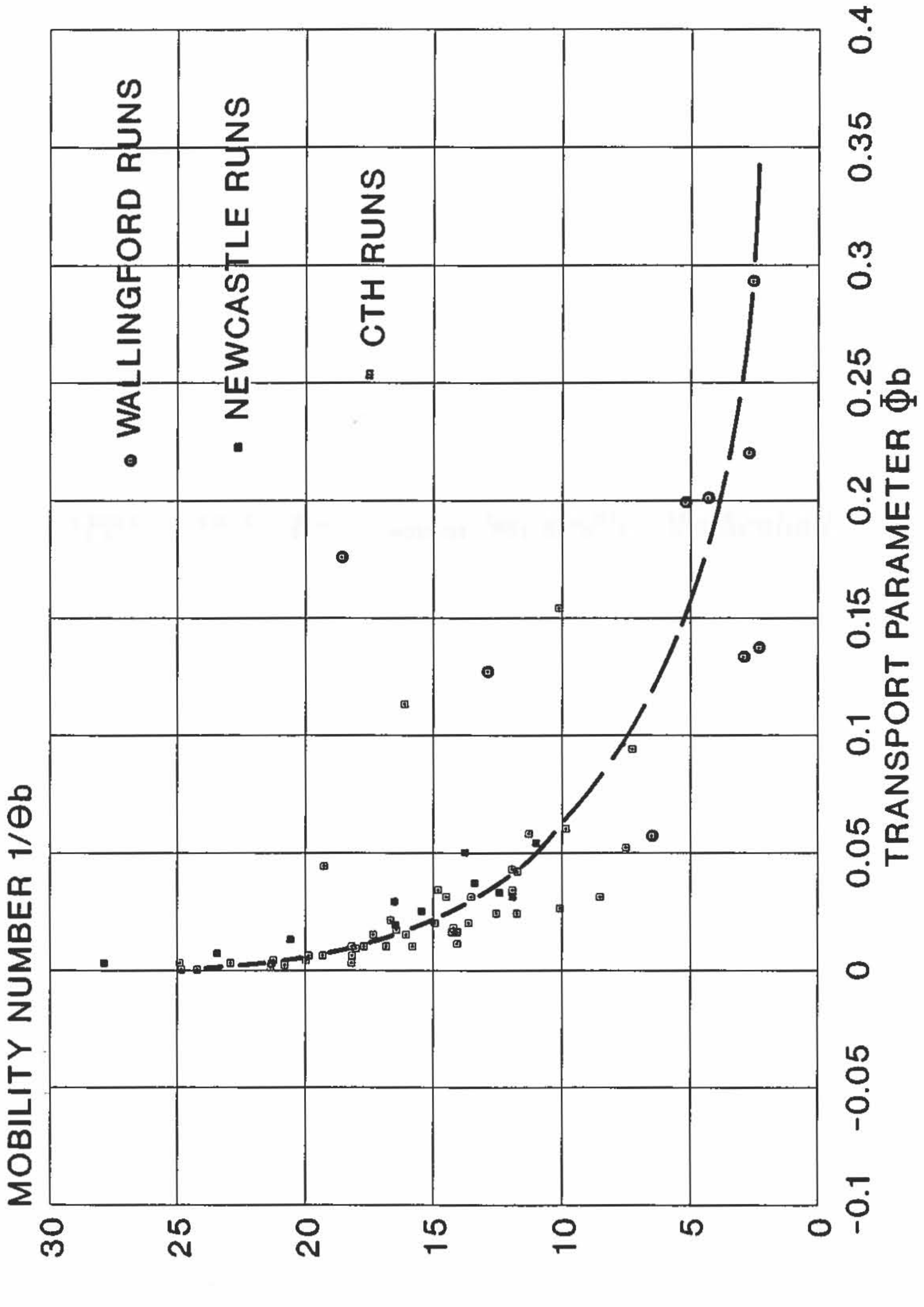

Figure 8. Transport parameter versus mobility number for all CTH, Wallingford and Newcastle runs.

APPENDIX I: Longitudinal bed profiles, Wallingford

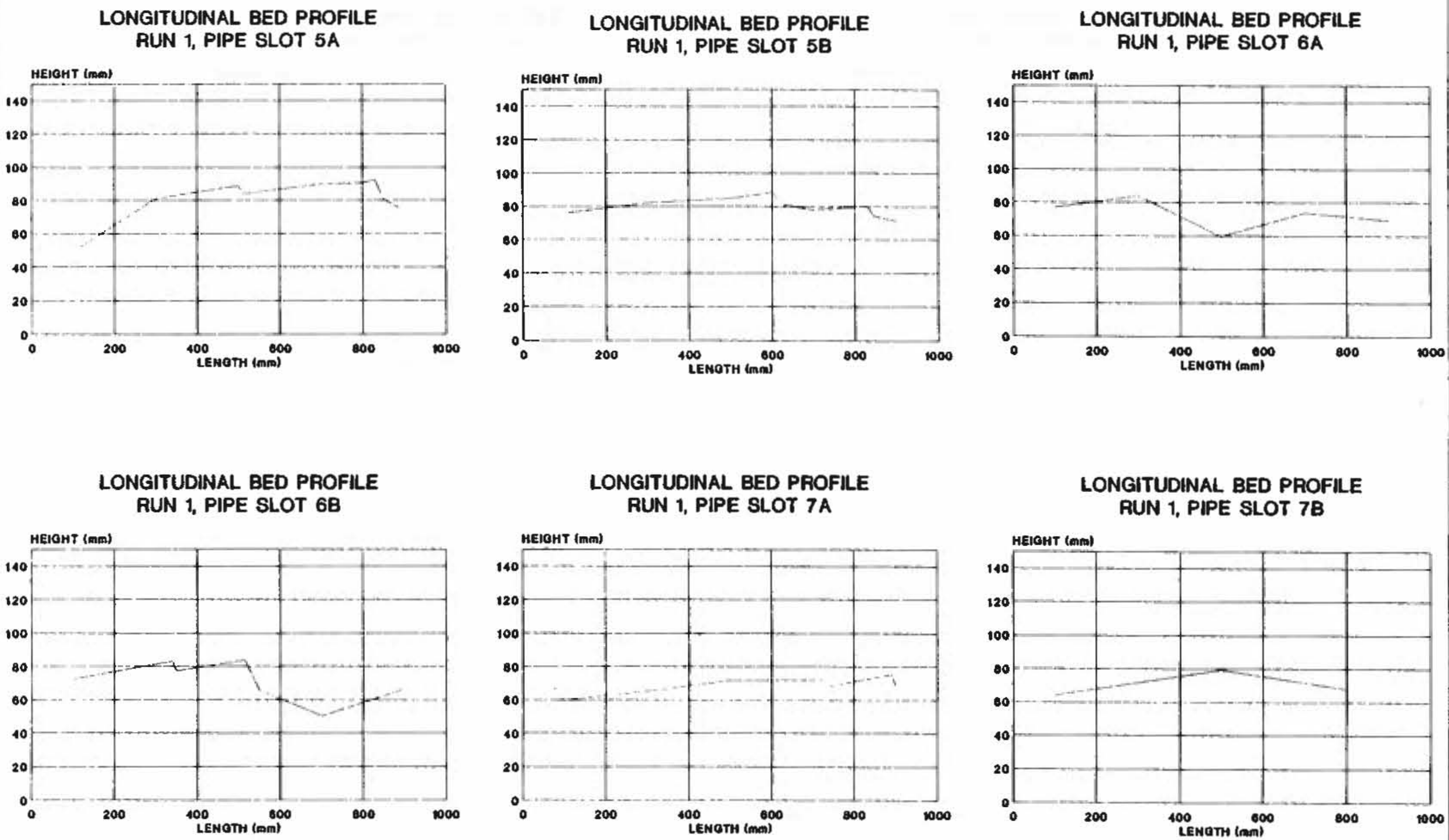

Figure 9. Longitudinal Bed Profiles for Wallingford Run 1

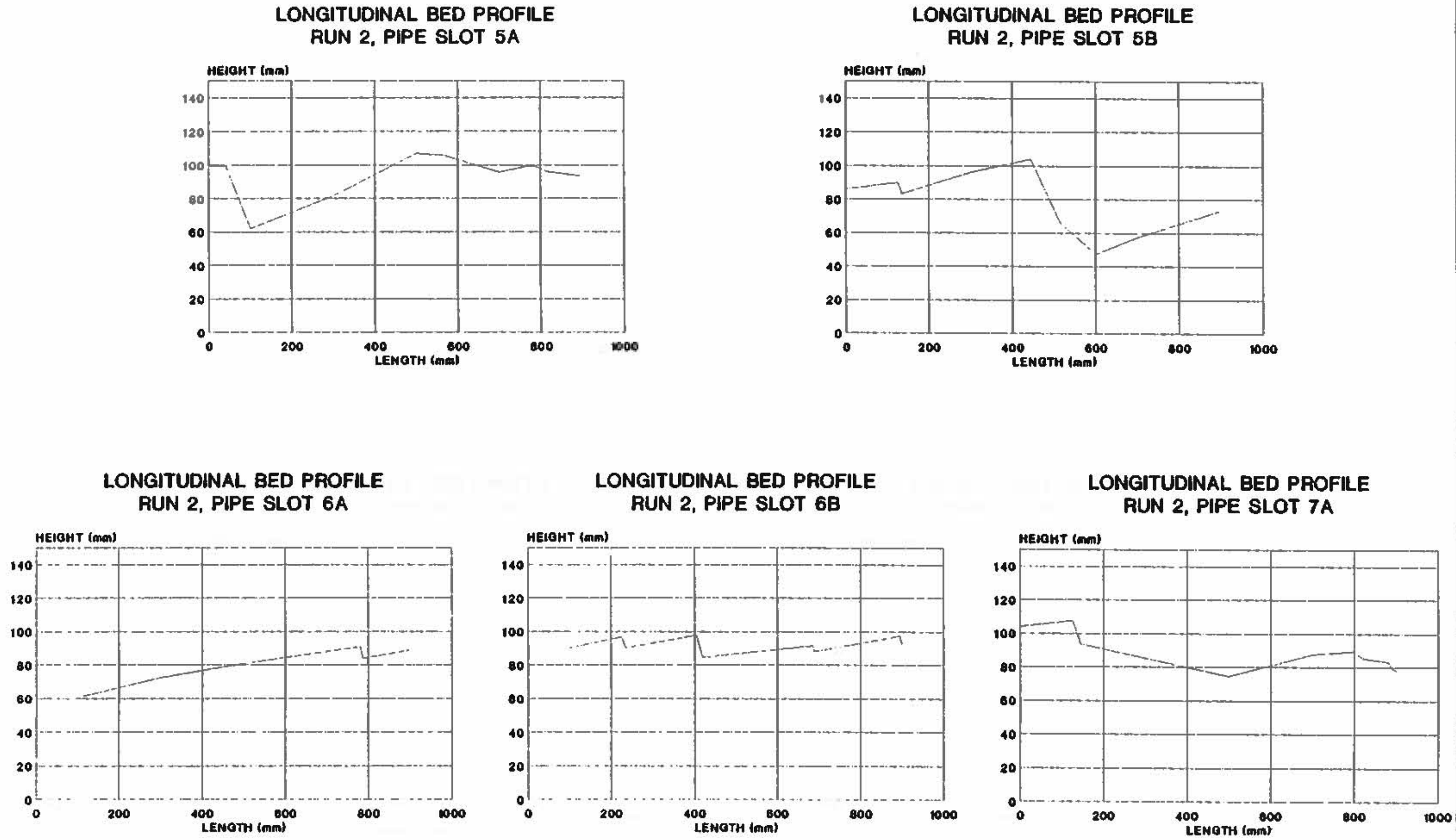

Figure 10. Longitudinal Bed Profiles for Wallingford Run 2

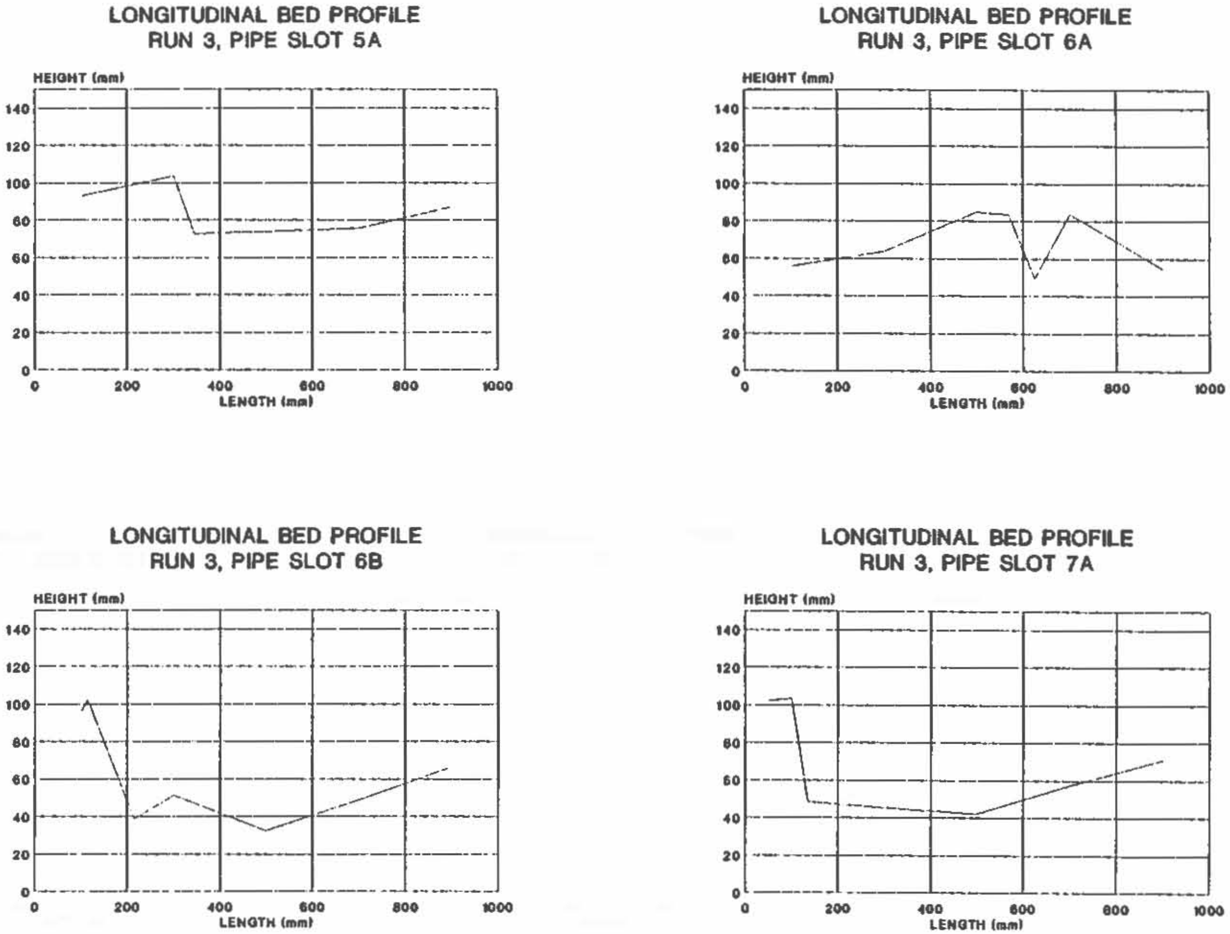

Figure 11. Longitudinal Bed Profiles for Wallingford Run 3

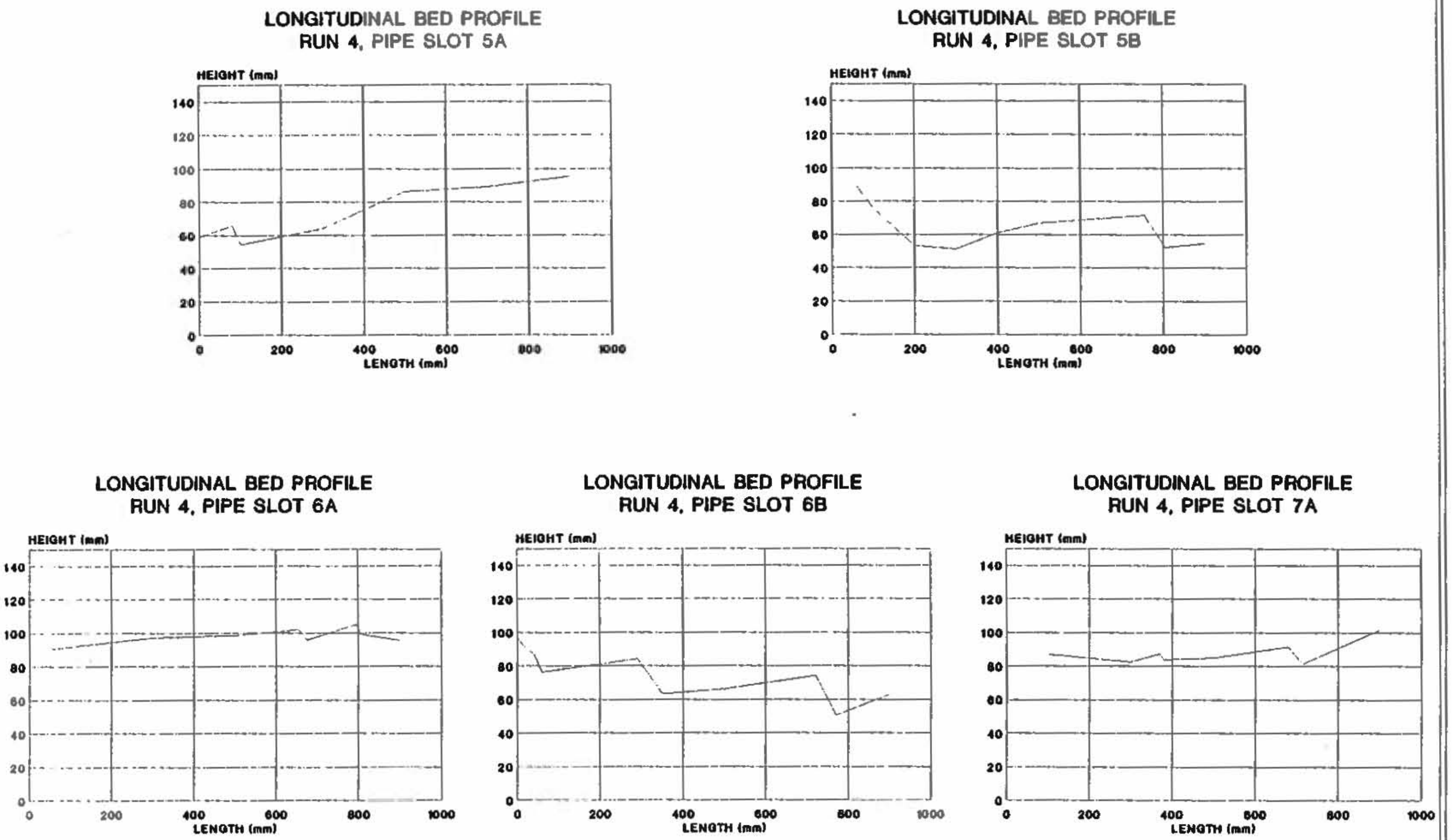

Figure 12. Longitudinal Bed Profiles for Wallingford Run 4

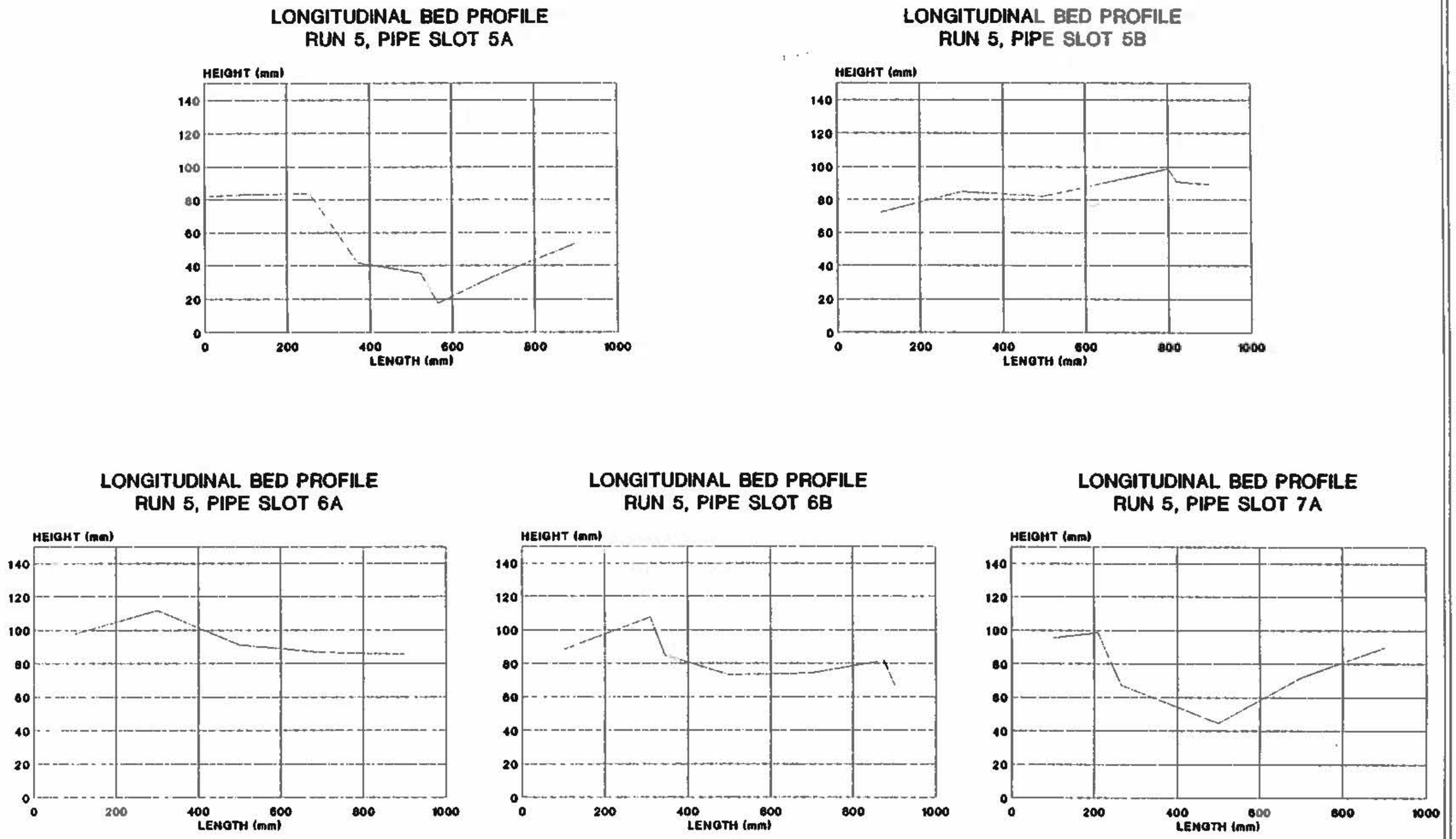

Figure 13. Longitudinal Bed Profiles for Wallingford Run 5

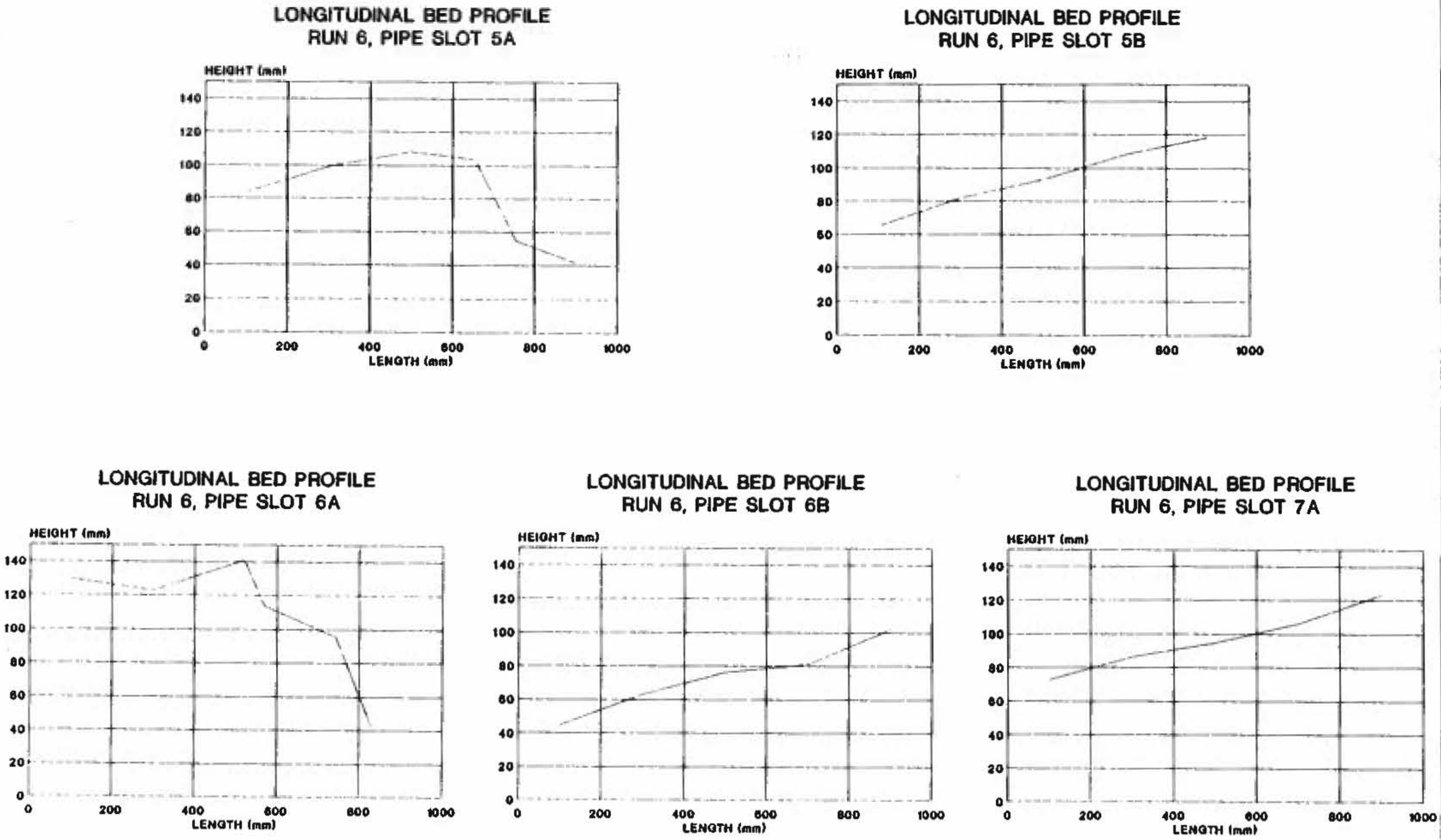

Figure 14. Longitudinal Bed Profiles for Wallingford Run 6

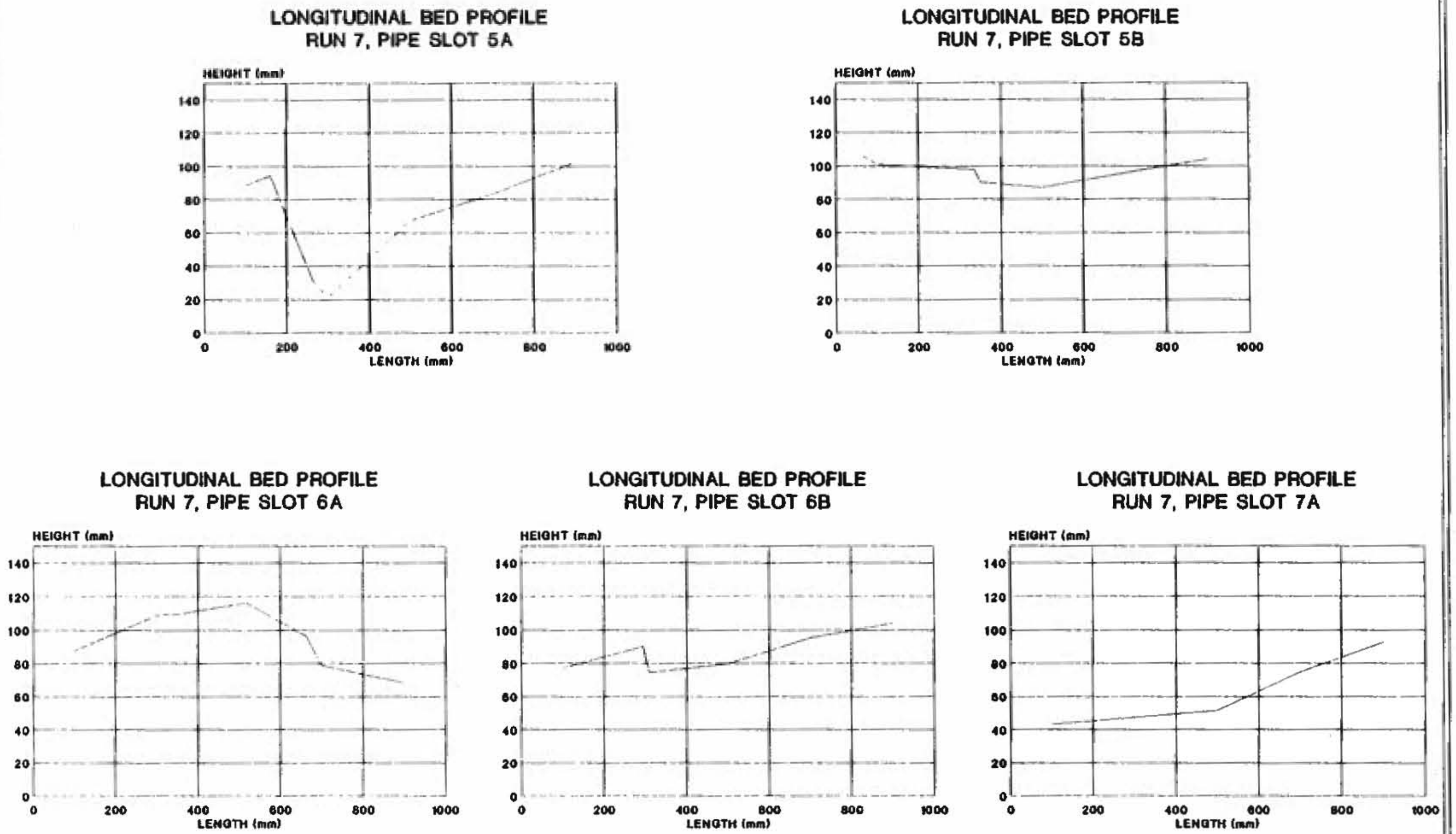

Figure 15. Longitudinal Bed Profiles for Wallingford Run 7

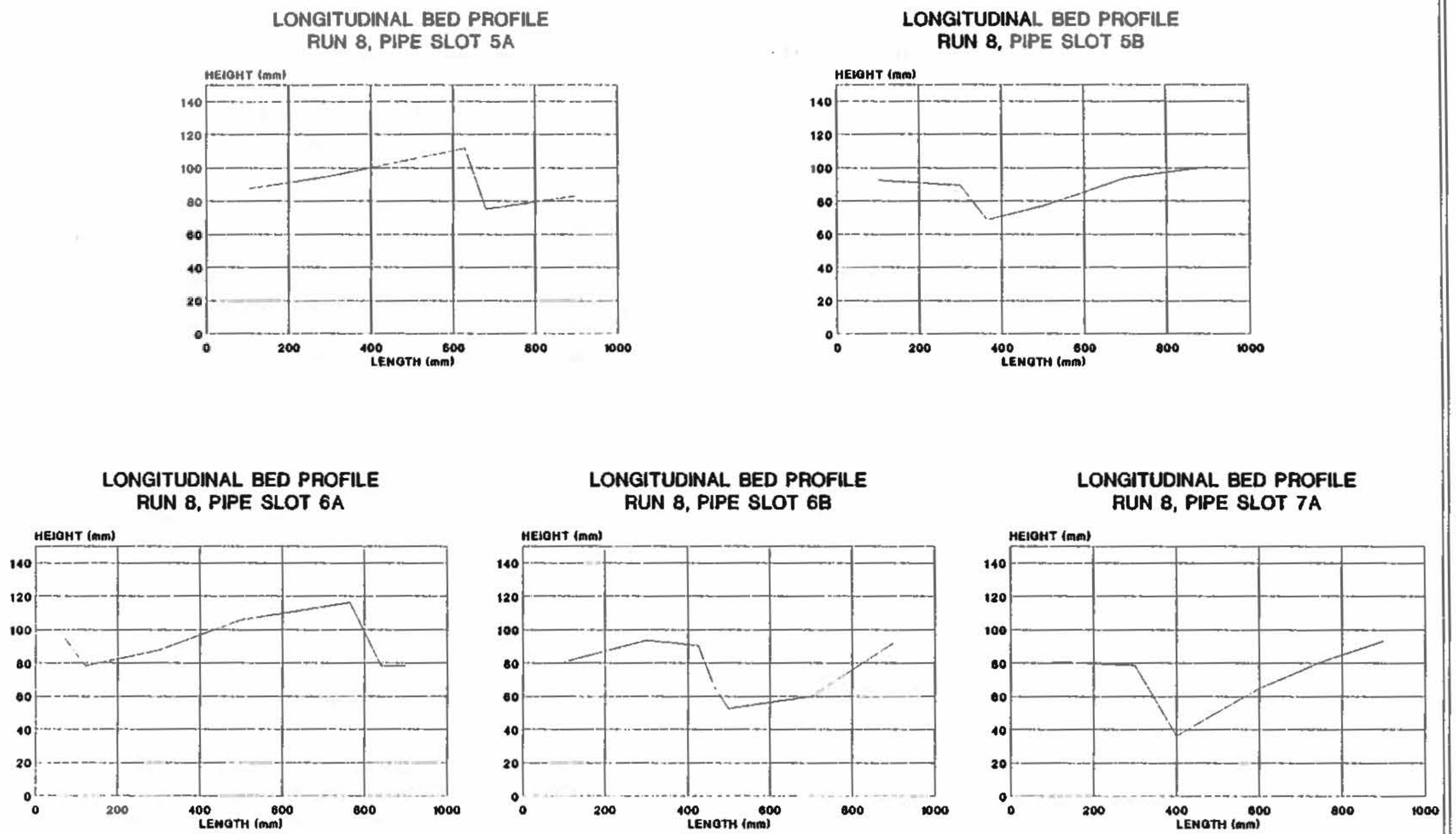

Figure 16. Longitudinal Bed Profiles for Wallingford Run 8

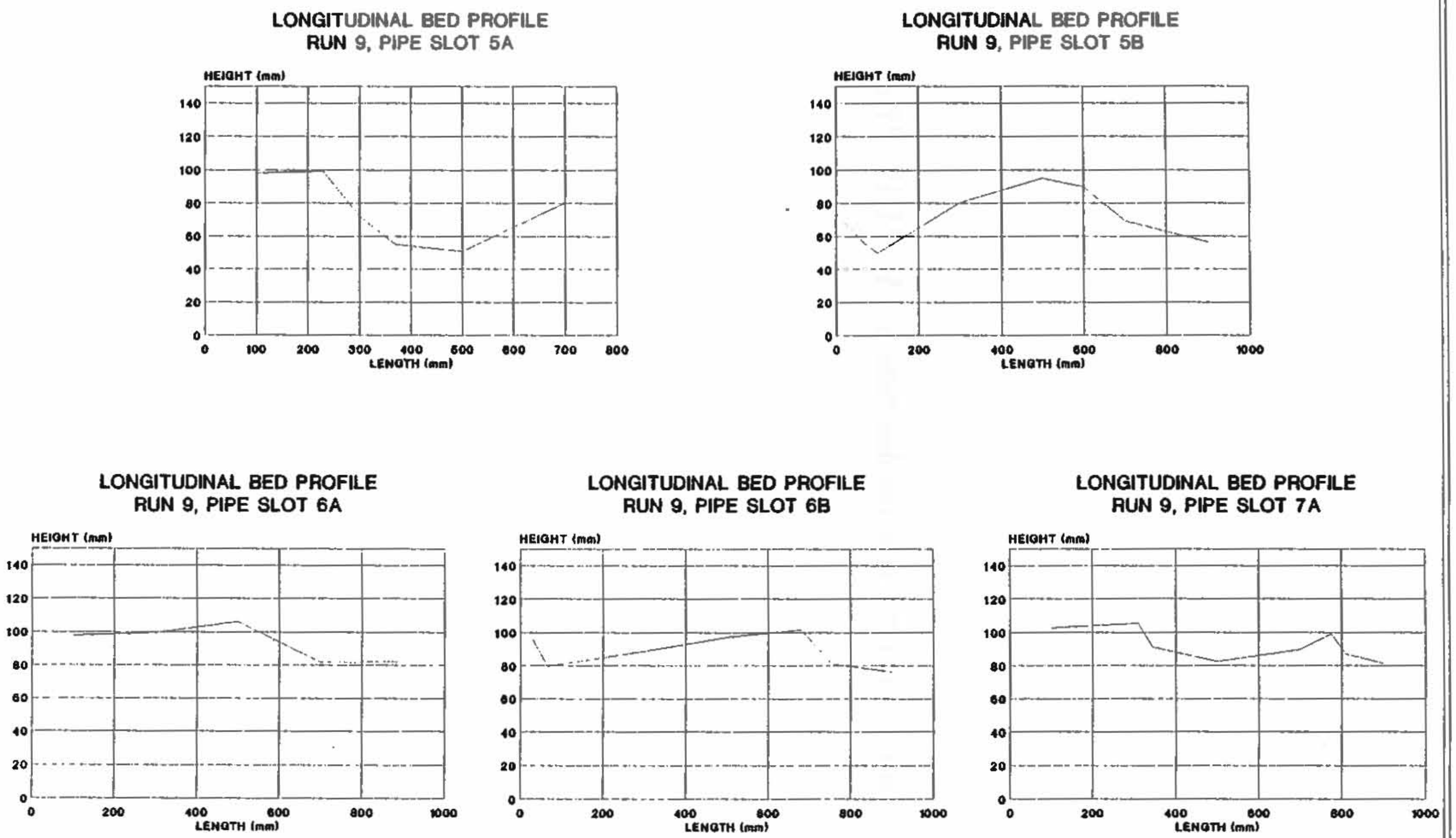

Figure 17. Longitudinal Bed Profiles for Wallingford Run 9

APPENDIX II: Longitudinal bed profiles, Newcastle

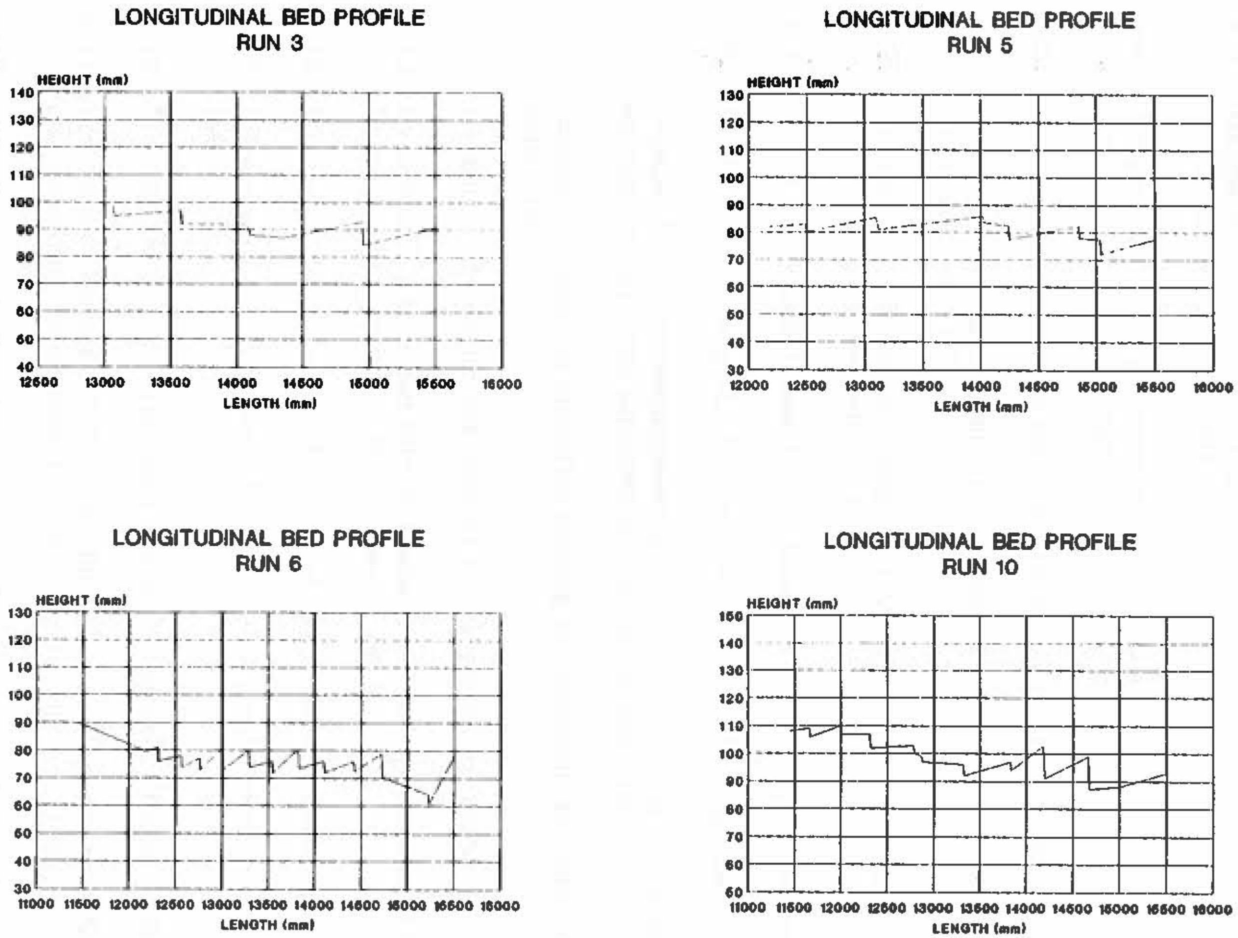

Figure 18. Longitudinal Bed Profiles for Newcastle Runs

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