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NATIONAL WATER RESEARCH INSTITUTE
INSTITUT NATIONAL DE RECHERCHE SUR LES EAUX

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PREAMBLE

The City of Toronto commissioned the National Water Research Institute (NWRI) to evaluate the hydraulic performance of a sewer junction at the Bathurst and Fairlawn intersection. The knowledge of the operation of this junction is important for finding the causes of infrequent surcharging of this junction and basement flooding in this area, and developing corrective measures.

Head losses at the Bathurst/Fairlawn sewer junction of two opposed laterals were studied in a 1:4 scale model installed in the hydraulics laboratory. The observed data indicate that the junction produces relatively small losses ($\leq$ 0.36 m), which are smaller than the actual design allowance for head losses (at this particular junction) of 0.47-0.55 m, for laterals L1 and L2, respectively. The observed head losses could be probably somewhat reduced by changing the junction geometry, but with limited practical benefits. The worst possible scenario of junction choking flow would result from formation of a vortex at the junction. Vortex flow patterns were not observed at the Fairlawn junction for the range of experimental parameters studied, nor could they be induced by manipulating flows in both laterals.

It would appear that sewer surcharge and spillage into basements in this area is caused by hydraulic problems further downstream of the junction (and downstream of the large outlet pipe serving to provide storage), and if indeed that is the case, junction improvements would not mitigate the problems encountered in the sewer system.

PRÉAMBULE

La ville de Toronto a chargé l'Institut national de recherche sur les eaux (INRE) d'évaluer le rendement hydraulique d'un raccordement d'égout à l'intersection des rues Bathurst et Fairlawn. Il est important de connaître le fonctionnement de ce raccordement si on veut découvrir la cause de sa surcharge occasionnelle et des inondations de sous-sols qu'elle cause afin d'élaborer des mesures correctives.

On a étudié les pertes de charge qui se produisent au raccordement de deux conduites latérales opposées à l'intersection Bathurst/Fairlawn à l'aide d'un modèle à l'échelle 1:4 installé dans le laboratoire d'hydraulique. D'après les données observées, le raccordement ne produit que des pertes de charge relativement faibles (≤0,36 m), inférieures aux tolérances prévues de 0,47 m et de 0,55 m pour les conduites latérales L1 et L2 à ce raccordement particulier. On pourrait probablement réduire légèrement les pertes de charge observées en modifiant la géométrie du raccordement, mais cela n'aurait guère d'avantages pratiques. Le pire scénario d'engorgement possible serait dû à la formation d'un tourbillon à ce raccordement. Or, on n'a constaté aucune indication d'un écoulement tourbillonnaire au raccordement Fairlawn dans tous les intervalles de valeurs des paramètres expérimentaux étudiés, pas plus qu'on n'a pu induire ce genre d'écoulement en manipulant les débits dans les conduites latérales.

Il semblerait que les surcharges et les déversements dans des sous-sols qui ont été constatés dans cette zone soient causés par des problèmes hydrauliques en aval de ce raccordement (et en aval du grand tuyau de refoulement servant à emmagasiner l'eau). Dans ce cas, les améliorations éventuelles apportées au raccordement ne permettraient pas d'atténuer les problèmes constatés dans le système d'égout.

ABSTRACT

Head losses at the Fairlawn sewer junction of two opposed laterals were studied in a 1:4 scale model installed in the hydraulics laboratory. The observed data indicate that the junction produces relatively small losses (≤ 0.36 m), which are smaller than the actual design allowance for head losses (at this particular junction) of 0.47-0.55 m, for laterals L1 and L2, respectively. The observed head losses could be probably somewhat reduced by changing the junction geometry, but with limited practical benefits. The worst possible scenario of junction choking flow would result from formation of a vortex at the junction. Vortex flow patterns were not observed at the Fairlawn junction for the range of experimental parameters studied, nor could they be induced by manipulating flows in both laterals. It would appear that sewer surcharge and sewage spillage into basements in this area is caused by hydraulic problems further downstream of the junction (and downstream of the large outlet pipe serving to provide storage), and if indeed that is the case, junction improvements would not mitigate the problems encountered in the sewer system.

Key Words

wastewater, basement flooding, head loss, sewer junctions, laboratory testing

DISCLAIMER

INTRODUCTION

The Water and Wastewater Services Division, City of Toronto identified a specific case of sewage back-up into basements in North Toronto and engaged the National Water Research Institute in hydraulic testing of a sewer junction at the Bathhurst Street/Fairlawn Avenue intersection, known as Fairlawn Avenue Sanitary Manhole #1 (further called the Fairlawn junction) and thought to be contributing to or causing this problem.

In recent years, two major rainfall events caused basement flooding in several homes immediately upstream of this junction. Such a condition can occur when the sewer system capacity is exceeded, sewers become surcharged and local head losses may bring the hydraulic gradeline above the elevation of basement floors in the serviced area. In order to determine if, or under what conditions, this sewer junction could become surcharged, a hydraulic scale model was constructed and tested in the Hydraulics Laboratory of the National Water Research Institute in Burlington, Ontario.

The main purpose of the study was to assess the potential of the sewer junction studied to cause sewer pipe surcharging at this location, particularly by acting as a vortex valve (brake) restricting outflow from the junction and thereby causing surcharging upstream of the junction. Past experience with sewer junction design indicated that a junction of two opposite laterals could cause relatively high energy losses, estimated as two to three outlet velocity heads (Marsalek, 1985). However, such losses would not be sufficient to cause the reported surcharging of the Fairlawn junction. If the manhole would function as a hydraulic bottleneck, recommendations to redesign the junction geometry would be made to improve its hydraulic effectiveness and improve the operation of the sewer system.

A hydraulic model of the sewer junction was constructed and tested. This work was conducted by NWRI staff, in space and facilities provided at the Hydraulics Laboratory of the National Water Research Institute in Burlington, Ontario.

Experimental Set-up

The experimental apparatus used is shown in Figs. 1-3. The apparatus consisted of a 1:4 scale laboratory model built according to the drawings and field measurement data provided by the City of Toronto, head tanks supplying the lateral inflows into the model, and a tail box fitted with a calibrated V-notch weir for outflow measurement (Q_{out}). Lateral pipe L_1 was fitted with a magnetic flow meter measuring pipe discharge denoted Q_1, and discharge through lateral pipe L_2 was determined as $Q_2 = Q_{out} - Q_1$. Piezometer ports were fitted in both lateral pipes as well as the outflow pipe to measure pressure head, P [m], in these pipes, and, the energy head, E[m], was determined as the pressure head plus the velocity head, $E = P + v^2/2g$, where v is the mean velocity in the corresponding pipe and g is the acceleration due to gravity (9.81 m/s^2).

The model scale used (4:1) was found adequate for this study and produced quantified data that could be scaled up to the prototype. The model was designed and operated according to the Froude similarity, which is applicable to flows governed by forces of gravity. In this case, all model dimensions are geometrically similar to the prototype, but scaled down 1:4, and the scale for discharge is 1:32. Consequently, both laterals were built as plastic pipe with a diameter of 0.0875 m (0.35 m in prototype), and the outlet pipe diameter was 0.225 m (0.90 m in prototype). The angle formed by both laterals is 175°; the change of flow direction for lateral L1 can be expressed as 71°, for lateral L2 as 114°.

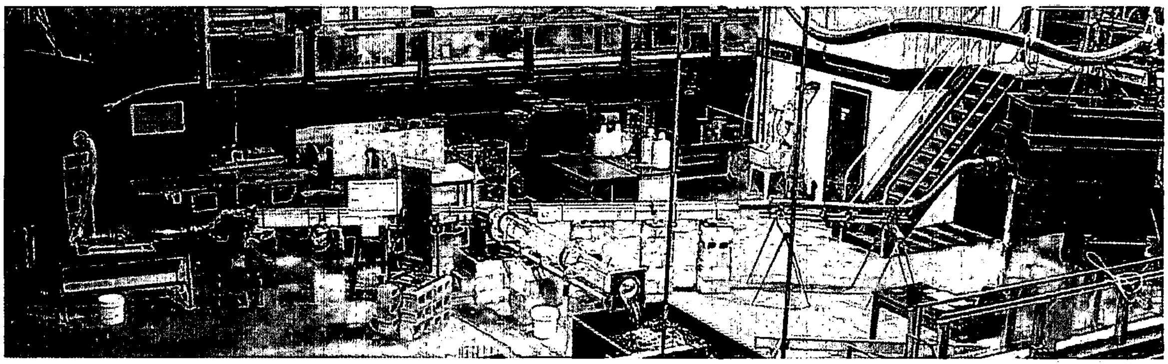

Figure 1: Laboratory Model - 1:4 Scale

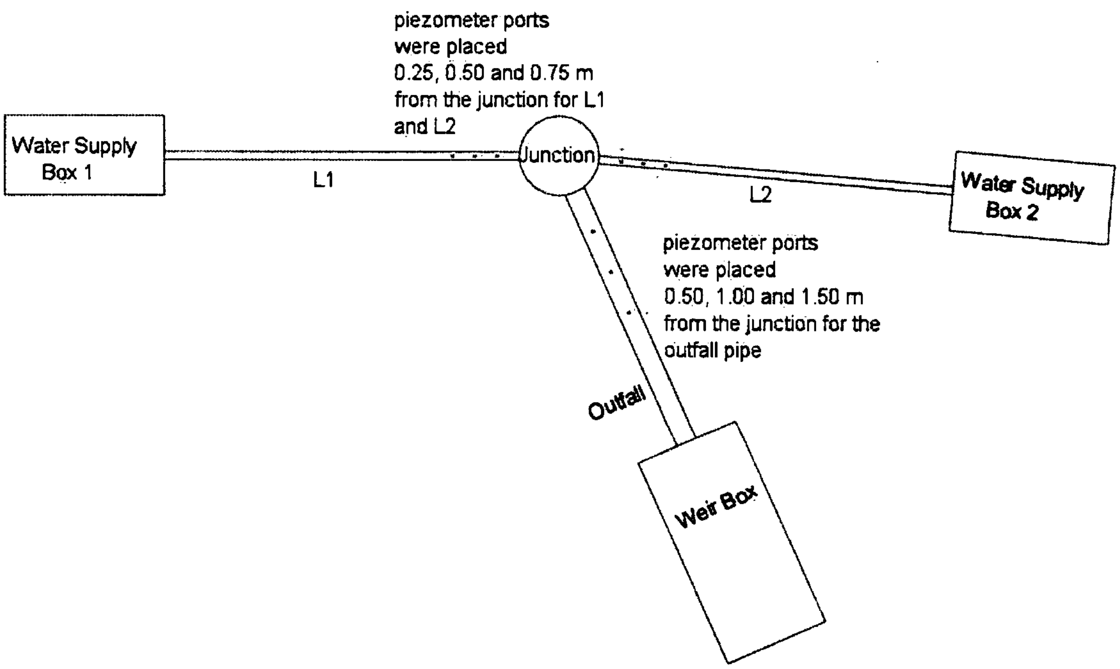

Figure 2: Experimental Layout

Figure 3: Junction of Two Opposed Laterals

Experimental Program

The model sewer junction was examined in two series of experimental runs designed to explore head losses caused by the junction for a wide range of experimental conditions. The main purpose of these experiments was to check whether a vortex forms at the junction. If it were the case, then the junction would function as a vortex valve and outflow from the junction would be relatively constant, regardless of the pressure head upstream of the junction. In fact, such a head would increase the intensity of the vortex motion and lead to a great dissipation of energy, with limited flow conveyance. This principle of using vortex to restrict outflow from storage is used as a combined sewer control under various commercial names (Brombach, 1990).

In the first series of runs, head losses for both laterals (L1 and L2) were determined for a broad range of junction surcharge depths (0.36 m – 0.935 m in model), flow rates ranging from 0.006 to 0.0131 m^3/s, and ratios of discharges in laterals Q_1/Q_2 ranging from 0.6 to 1.17. The main purpose of those runs was to assess the magnitude of junction head losses for a wide range of conditions. Junction surcharge was varied by increasing the downstream pressure head by throttling a gate valve at the downstream end of the outlet pipe. Changes in lateral discharges were achieved by opening valves controlling inflows into lateral pipes.

Piezometric pressure head data were processed in two ways: (a) taking readings from piezometers located in both laterals and the outflow pipe, adding velocity heads ($v^2/2g$) to these pressure heads to obtain energy heads, fitting least square lines approximating the energy grade lines for individual pipes, and extrapolating these lines to the junction centre. The headloss for each lateral was then defined as the extrapolated lateral energy head minus the outlet energy head; (b) in the second approach, energy heads in individual pipe lines were averaged, and the energy grade lines were approximated by horizontal lines over a short distance (0.5 m) to the junction centre and the rest of the procedure was the same as in the first case. Both procedures produced comparable data; the averaging of hydraulic heads (rather than fitting sloping grade lines) was deemed less susceptible to errors inherent to fitting least square lines through relatively few points and was adopted in the final data analysis. Examples of both procedures are shown in the Appendix.

The second set of tests focused on the risk of vortex formation at the junction for various ratios of flows in both laterals and different degrees of surcharge. Thus, in each experimental series, a ratio of Q_1/Q_2 was selected and kept constant, and head losses were measured for five levels of surcharges. Thus, this series comprised 25 runs altogether.

Table 1. Second experimental series

Run No.	Q1(fraction of maximum)	Q2(fraction of maximum)
1 A-E	0	1
2 A-E	0.27	0.73
3 A-E	0.48	0.52
4 A-E	0.68	0.32
5 A-E	0.75	0.25

Results and Discussion

Altogether, almost 40 experimental runs were made in the sewer junction model. The results from the first series of 12 runs are presented in Table 2, which displays basic experimental data in both model and prototype dimensions.

Table 2. Head Losses at the Fairlawn sewer junction for various degrees of surcharge and minor variation of Q_1/Q_2

Run	Junction surcharge [m]		Q_1/Q_2	Q outflow [m^3/s]		Head loss ΔE_1 [m]		Head loss ΔE_2 [m]	
	model	prototype		model	prototype	model	prototype	model	prototype
1	0.359	1.44	0.92	0.0094	0.301	0.031	0.12	0.040	0.16
3	0.533	2.13	0.77	0.0123	0.394	0.053	0.21	0.067	0.27
10	0.546	2.18	0.98	0.0099	0.317	0.036	0.14	0.042	0.17
7	0.605	2.42	0.78	0.0105	0.336	0.038	0.15	0.045	0.18
6	0.641	2.56	0.86	0.0108	0.346	0.032	0.13	0.038	0.15
2	0.675	2.70	0.80	0.0081	0.259	0.019	0.08	0.026	0.10
11	0.696	2.78	1.17	0.0100	0.320	0.055	0.22	0.037	0.15
5	0.700	2.80	0.65	0.0117	0.374	0.052	0.21	0.073	0.29
4	0.740	2.96	0.59	0.0118	0.378	0.058	0.23	0.084	0.34
12	0.803	3.21	0.62	0.0131	0.419	0.054	0.22	0.089	0.36
9	0.829	3.32	1.06	0.0060	0.192	0.031	0.12	0.018	0.07
8	0.935	3.74	1.00	0.0060	0.192	0.028	0.11	0.018	0.07

Mean	0.041	0.16	0.048	0.19

Data in Table 2 indicate broad variation of observed head losses for both laterals. For lateral L_1, the average head loss was 0.16 m, with minimum and maximum values of 0.08 and 0.23 m, respectively. Head losses in lateral L_2 were somewhat larger, which follows from the junction geometry characterized by a greater change in direction (180° - 66° = 114°) of lateral L_2 than that of lateral L_1 (180° - 109° = 71°). The average head loss for L_2 was 0.19 m, with minimum and maximum values of 0.07 and 0.36 m, respectively. The fact that lateral L2 head losses ΔE_2's are greater than ΔE_1's can be also seen in Fig. 4, which indicates that ΔE_2's are about 25% larger than ΔE_1's (at $r^2 = 0.65$).

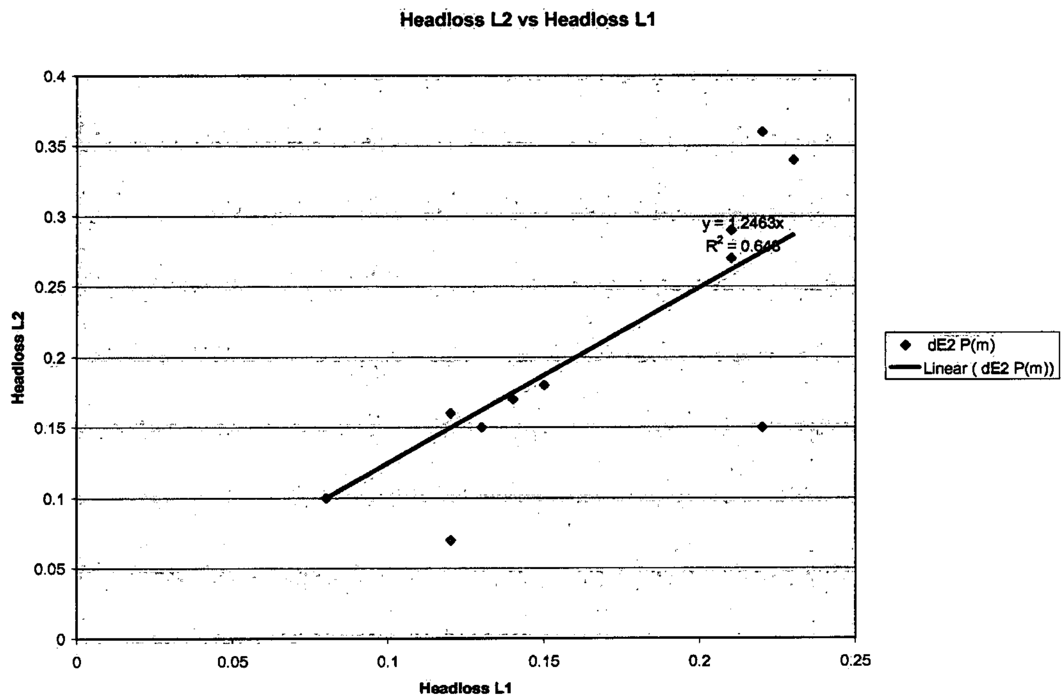

Fig. 4. Head loss in lateral L_2 vs. head loss in lateral L_1

The earlier research on junctions of two laterals (Marsalek, 1985) indicated that head losses at such junctions could be expressed as:

$$K = f\,(Q_1/Q_2,\ D_1/D_{out},\ D_2/D_{out},\ \text{junction geometry}) \qquad (1)$$

where K is the head loss coefficient = ΔE_1 / ($v_{out}^2/2g$), v_{out} is the mean velocity in the outlet, g is the acceleration due to gravity (9.81 m/s^2), D is the pipe diameter, and subscripts 1, 2, and 'out' refer to lateral L1, lateral L2, and outlet, respectively.

Recognizing that in runs listed in Table 1 the only variable among those included in eq. (1) was Q_1/Q_2, head losses in laterals 1 and 2, ΔE_1's and ΔE_2's, were plotted vs. the relative discharge Q_1/Q_2 and regression trend lines (2nd degree polynomial) were fitted to the experimental data, with $r^2 = 0.69$ and 0.81, for laterals 1 and 2, respectively (Fig. 5 and 6). Thus, head losses in both laterals indeed vary with the relative discharge.

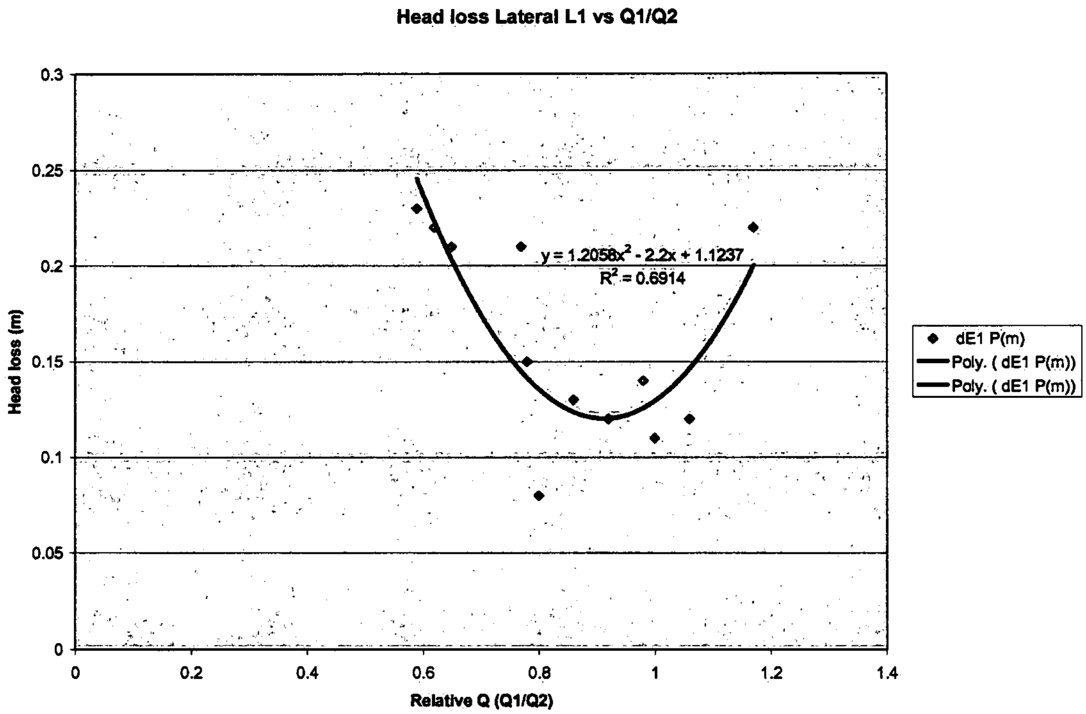

Fig.5. Lateral L_1 head loss vs. relative discharge Q_1/Q_2

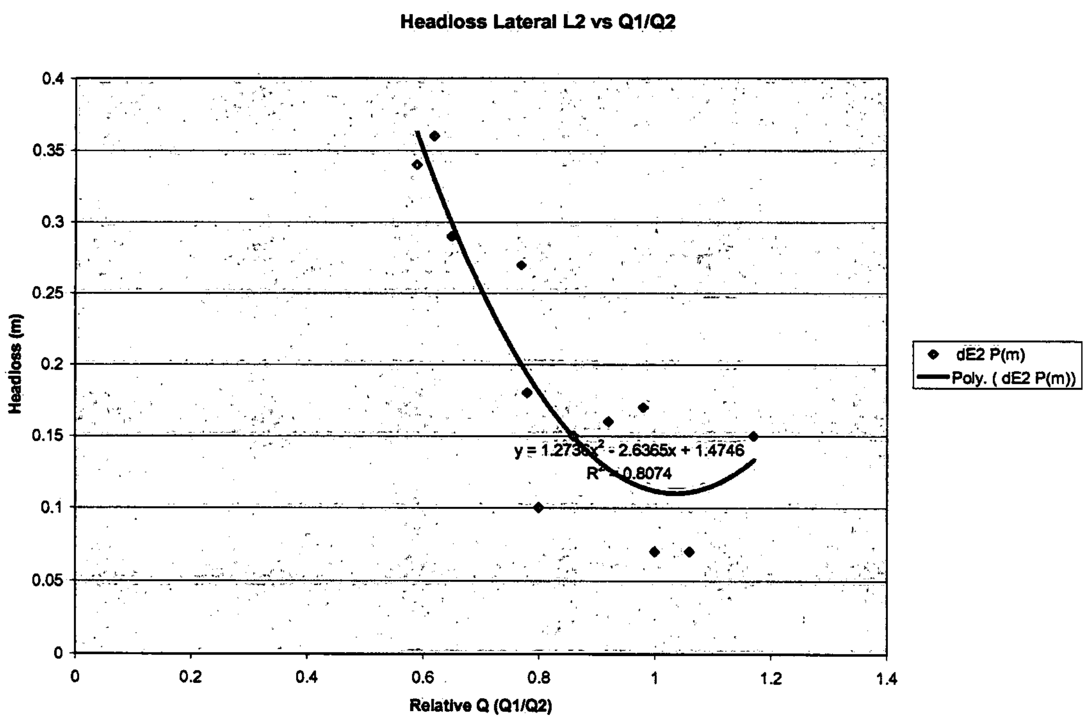

Fig.6. Lateral L_2 head loss vs. relative discharge Q_1/Q_2

In the second series of experimental runs, the relative discharge Q_1/Q_2 was varied in five steps, and for each step, five runs with different surcharge heads were conducted. It was noted that the surcharge head did not affect the losses and consequently, the data for various surcharge heads were aggregated into single average values. The final results determined for both the scale model and the prototype are shown in Table 3.

Table 3. **Head Losses at the Fairlawn sewer junction for varying relative discharge Q_1/Q_2**

Run	**Q_1/Q_2**	**Q outflow [m^3/s]**		**Headloss ΔE_1 [m]**		**Headloss ΔE_2 [m]**	
		Model	prototype	model	prototype	model	Prototype
A	0	0.0055	0.176	0.0	0.00	0.039	0.16
B	0.37	0.0051	0.163	0.010	0.04	0.017	0.07
C	0.92	0.0054	0.173	0.011	0.04	0.010	0.04
D	2.13	0.0053	0.170	0.024	0.10	0.016	0.06
E	3.00	0.0061	0.195	0.034	0.14	0.021	0.08
Mean				0.016	0.06	0.021	0.08

The second series results indicate that head losses do not depend on the surcharge head at the junction, and that even for high variation of the relative discharge (Q_1/Q_2 as high as 3), head losses did not attain some unexpectedly high values. Neither experimental series produced hydraulic conditions in the Fairlawn junction indicating the presence of a vortex brake (valve).

Finally, it is a good practice in sewer design to compensate for junction head loss by dropping sewer pipe invert by a head equivalent to the junction head loss. In the Fairlawn junction, the invert drop for lateral 1 is 0.47 m, and for lateral 2, about 0.55 m. Such drops appear to be adequate for the maximum head losses determined in this experimental study as 0.23 and 0.36 m, for laterals 1 and 2.

Conclusions

A scale model study of the Fairlawn sewer junction of two opposed laterals indicates that for a wide range of experimental conditions studied, the junction produces relatively small losses ($\leq$ 0.36 m), which do not exceed the actual design allowance for such losses of 0.47-0.55 m. These losses could be probably somewhat reduced by changing the junction geometry, but with limited practical benefits. The worst possible scenario of junction choking flow would result from formation of a vortex at the junction; under such circumstances, outflow from the junction would hardly increase with increasing head upstream of the junction. Vortex flow patterns were not observed at the Fairlawn junction for the experimental conditions studied, nor could they be induced by manipulating flows in both laterals. It would appear that sewer surcharge and sewage spillage into basements in this area is caused by hydraulic problems further downstream of the junction (and downstream of the large outlet pipe serving to provide storage), and if indeed that is the case, junction improvements would not mitigate the problems encountered in the sewer system.

Recommendation

The data indicate that the capacity of the junction manhole is sufficient to convey the maximum specified flows under all tested conditions. Before considering any changes of the junction, detailed studies of flow conveyance downstream of the junction outlet pipe should be conducted.

Acknowledgement

The excellent laboratory work of Mr. Bill Warrender is acknowledged. He was responsible for construction of the benching in the manhole, setting up the model components, piping for flows, and assisting with flow measurements.

References

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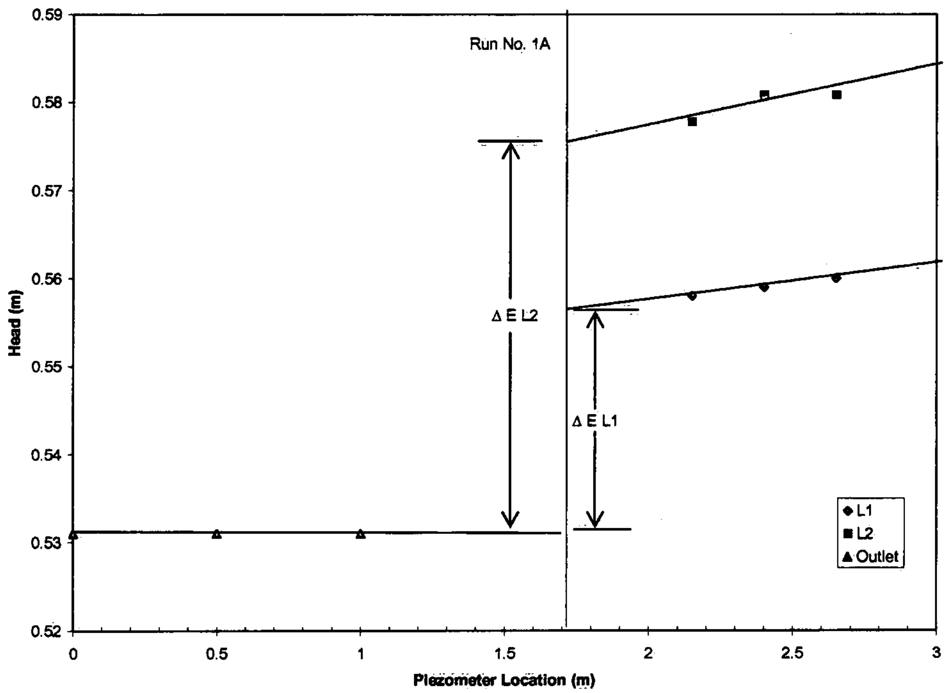
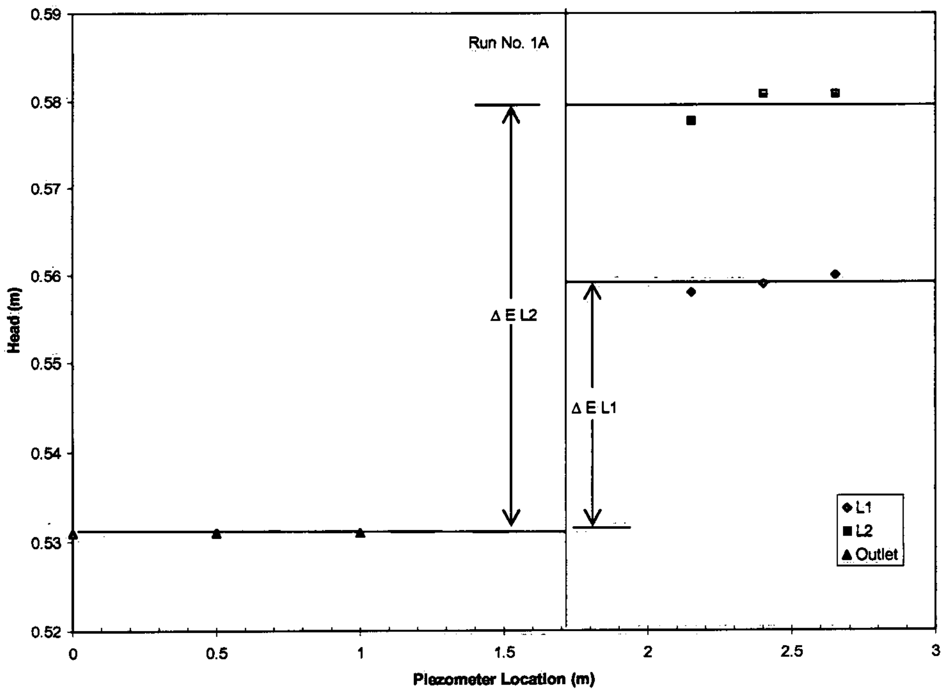
Appendix - Examples of fitting and extrapolating energy grade lines to experimental data.

Case 1 – fitting the best-fit grade lines



Case 2 – extrapolating average energy heads by horizontal lines to the junction.



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