PHYSICAL HYDRAULIC MODEL STUDY OF SEWAGE OVERFLOW MITIGATION, AUCKLAND, NEW ZEALAND

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Abstract

The Newmarket Gully (Auckland, New Zealand) waste water outlet, designed in 1983, is prone to flooding. On average, the frequency of flooding into Newmarket stream is 102 spills per annum, with a total annual spill volume of 227,193m^3 (Watercare Services Ltd. 2007). In order to reduce this flooding and to provide additional storage capacity, Sinclair Knight Merz (SKM) proposed the addition of a storage tank on top of the existing energy dissipater and the widening of the channel downstream. The effects of those changes were tested on a scale model (1:22) in The University of Auckland's Fluid Mechanics laboratory.

The construction and testing consisted of three phases, as the design was iteratively updated based on the results of the previous phase. The initial design (Design Scenario 1) posed problems with a jet forming from the 1200∅ pipe. The precast design (Design Scenario 2) featured a modified 1200∅ inlet so that the flow was streamlined along the system and the velocity reduced, however issues arose further downstream as the system pressurised. The final design (Design Scenario 3) increased the volume of the system so that for the maximum tested flow rate (41m^3/s) pressurisation did not occur.

Key Words: Stormwater Outfall, Hydraulic Model, Overflow Mitigation, Newmarket Gully

1 INTRODUCTION

The aim of this project is to investigate the proposed changes to the Newmarket stormwater outfall system (Newmarket Gully) originally designed in October 1983 (Melville 1983). The existing stormwater system has a frequency of overflowing into Newmarket stream on average 102 spills per annum with a total annual spill volume of 227,193m^3 (Watercare Services Ltd. 2007). This is expected to reduce to approximately two spills per annum with the installation of the proposed storage tank and the completion of an infiltration and inflow reduction programme in the catchment by Metrowater. The proposed modifications to the existing stormwater structure are required to be built over the existing outfall for installation of a new storage tank.

The Metrowater Integrated Catchment Study (ICS) model of the Newmarket Drainage Management Area (DMA) predicts one in 100 year flows of 27.4m^3/s, comprising of flows of 15m^3/s (from the original single stormwater outfall pipe) and 8.4m^3/s from an open channel that will be carried in a new 1200∅ pipe (AWT 2005). The wastewater tank will be situated above the energy dissipater, enclosing the overall system. Stormwater will discharge via two outfall pipes located in the open chute into Newmarket Stream.

The existing stormwater structure is to be enlarged to accommodate the tank and it will also include two outfall pipes currently not present. The wastewater tank requires support; piles are proposed for this purpose, affecting the continuous flow at their positions within the stormwater channel. As a result of the confinement of the main stormwater system, caused by accommodating the tank, it will no longer feature open channel flow throughout.

2 HYDRAULIC MODEL

2.1 Scale Ratios

The Froude Law is appropriate for this type of hydraulic model, leading to the following scale ratios (all expressed as model size to prototype size):

Geometric scale	1:22
Velocity scale	1:4.69
Discharge scale	1:2270
Time scale	1:4.69

Froude law modeling is appropriate for flow systems that operate with a free water surface. This is the case for our model except for the centre section at very high flow rates. However, it is assumed that the viscous forces are relatively minor in this section, because the water pressures are only slightly in excess of hydrostatic pressure.

2.2 Model Construction

The model was constructed in three phases. Initially a Perspex and timber model was built of the existing energy dissipater and downstream channel. The 1200∅ and 1800∅ pipes were constructed from PVC piping. This model was then modified by changing the 1200∅ inlet pipe in an attempt to streamline the flow. At this stage, the additional tank overflow (1200∅) connection and the open channel were added to the model. The tank overflow connection was modeled with PVC piping and the Middleton Road open channel was modeled with a timber open channel connected to a 950∅ PVC pipe. The third construction phase (Figure 1: Hydraulic Model) was to increase the volume of the system through the centre section. Figure 1, Figure 2 and Figure 3 show the final model, a technical drawing of the channel plan and provide a schematic overview, respectively.

2.3 Experimental Apparatus and Testing Procedure

The model was tested in three stages following each construction phase. Each testing stage involved operating the model at different flow rates up to its maximum capacity and measurements of flow rate and water levels were made. The flow rates were measured by timing the collection of 200 – 400 litres of water in a weigh-tank.

Figure 1: Hydraulic Model

Figure 2: Technical drawing of the channel plan (additional storage tank in red, existing energy dissipater in light grey and new proposed structure in dark grey)

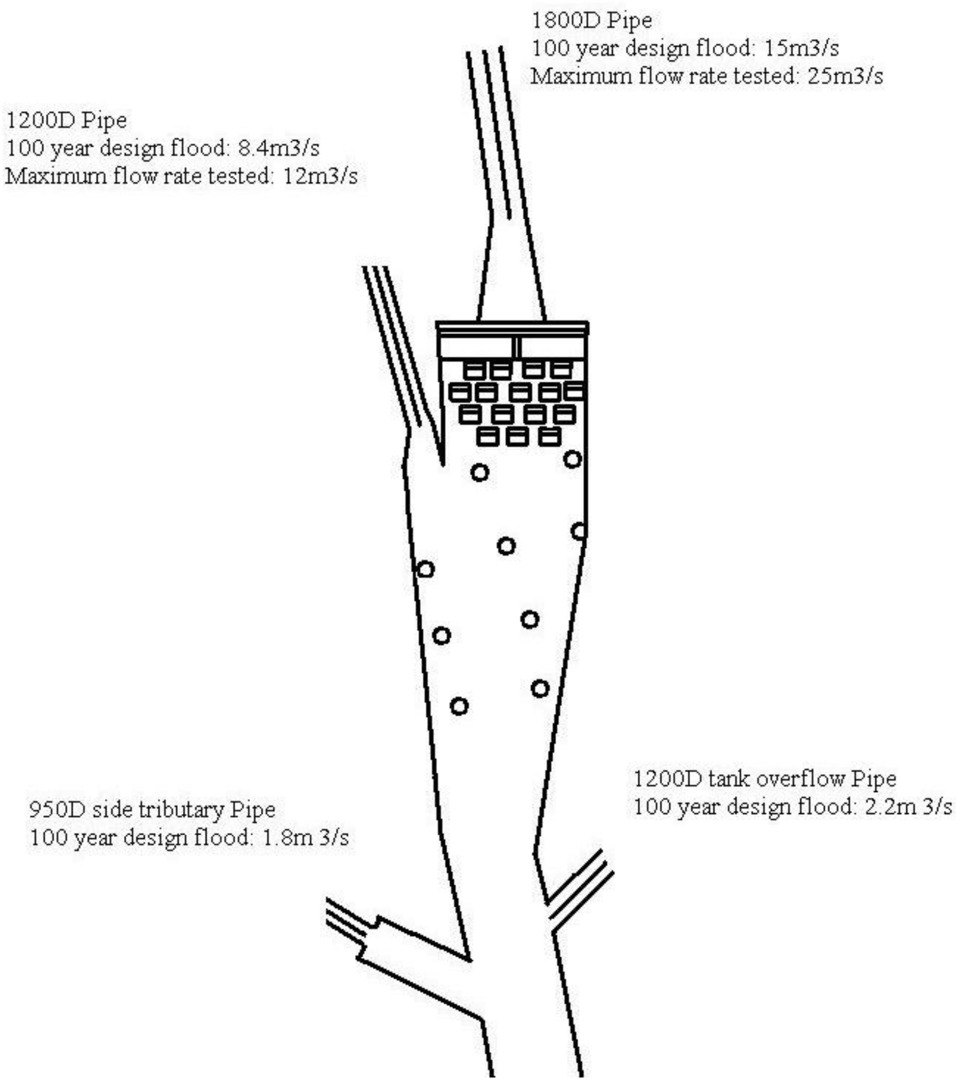

Figure 3: Schematic overview of outlets and flow rates

3 RESULTS AND DISCUSSION

3.1 Design Scenario 1

The 1800∅ inlet and energy dissipater appeared very effective when operated at the 100 year storm design flood ($15m^3/s$) and up to the maximum flow tested of $25m^3/s$. The 1200∅ inlet, however, presented a problem. When operated at the 100 year design flood ($8.4m^3/s$) and up to the maximum flow tested of $12m^3/s$ the pipe acted like a jet with the water hitting the opposite wall of the chute at very high velocities. This effect was more severe when the flow rate in the 1800∅ pipe was below $20m^3/s$. Figure 4 shows the jet formed as a result of a zero flow rate in the 1800∅ pipe and a flow rate of $8.4m^3/s$ in the 1200∅ pipe.

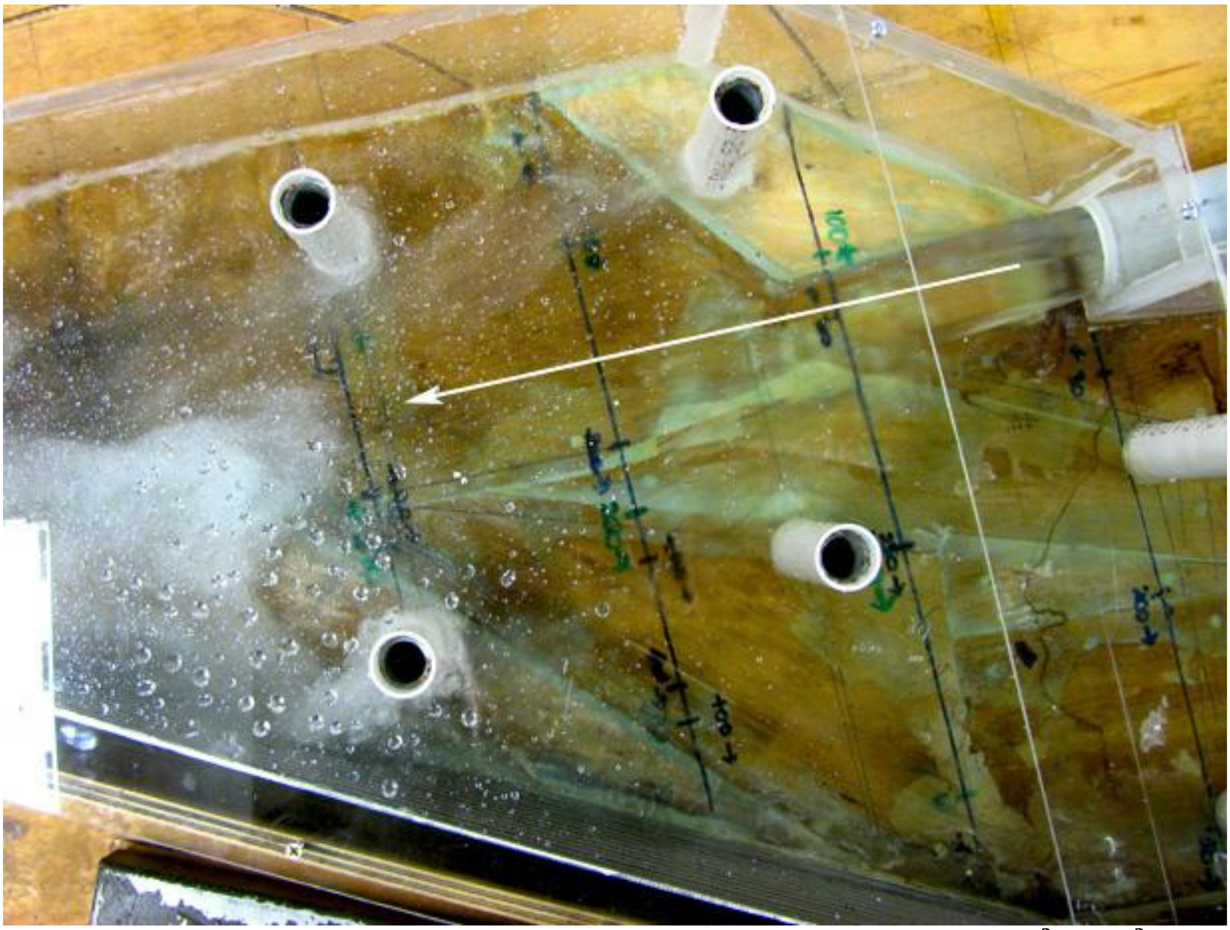

Figure 4: Jet of water forms in the 1200∅pipe. Best seen at flow rate of 8.4m^3/s (0m^3/s in the 1800∅pipe 8.4 m^3/s in the 1200∅ pipe)

3.2 Design Scenario 2

For the purpose of reducing the jet effect present in Design Scenario 1, modifications were made to the 1200∅ outlet. The outlet was expanded to a rectangular cross section with a flared end as well as being moved back to be in line with the piles. The wall originating at the 1200∅ outlet towards the box culvert was also straightened to help streamline the flow.

These changes significantly improved the flow from the 1200∅ outlet. This jet was no longer formed and high flow rates could now discharge from the 1200∅ outlet without causing problems. The body of the system was deemed ineffective due to it pressurising at a low flow rate of 34.96m^3/s. Although slightly above design flow rate of 27.4m^3/s, it was still below the maximum tested flow rate of 41m^3/s. Pressurisation points were also calculated with just the 1800∅ and the 1200∅ outlet pipes running (see Table A).

Table A: Pressurisation and depressurisation flow rates for Design Scenario 2

	Full system:1800∅ 1200∅ 1200∅ tank over flow and 950∅ side tributary all running	1800∅ and 1200∅ pipes running only
Design flow rate (m^3/s)	27.4	23.4
Maximum flow (m^3/s)	41	37
Flow at pressurisation point (m^3/s)	30.4	35
Flow at depressurisation point (m^3/s)	20.3	21.7

It is interesting to note that depressurisation did not occur at the same flow rate as pressurisation. Instead, depressurisation occurred after the flow rate had decreased to $20.3m^3/s$ which is significantly lower than the pressurisation flow rate of $30.4m^3/s$ and the design flow rate ($27.4m^3/s$) for full system testing.

It important to note that air pressure modeled below a 1:1 scale is very inaccurate. It is possible that this pressurisation will pose serious problems on the tank and tank overflow, and the pressurisation flow rates may in fact be lower than those determined from the scale model. Therefore, ventilation should be considered in the vicinity where the energy dissipaters are located (Figure 3), the exact position depending on structural requirements of the superstructure.

3.3 Design Scenario 3

To eliminate the pressurisation present in the model, further modifications were made to increase the stormwater system capacity. The left hand side wall was extended in order to widen the channel and the drop in roof height was also removed. This meant that a standard precast box culvert would no longer be suitable in construction, which may increase the overall expense and construction time.

When tested at design flow ($27.4m^3/s$) all sections of the new model performed satisfactorily. No pressurisation of any section occurred, the 1200∅ pipe remained effective and no jet formed. At the maximum tested flow rate of 41m3/s the system was still running efficiently and unpressurised. The drawback of the design was that the system was over flowing at the end (Figure 5). This problem could be addressed by increasing the height of the channel section from this point.

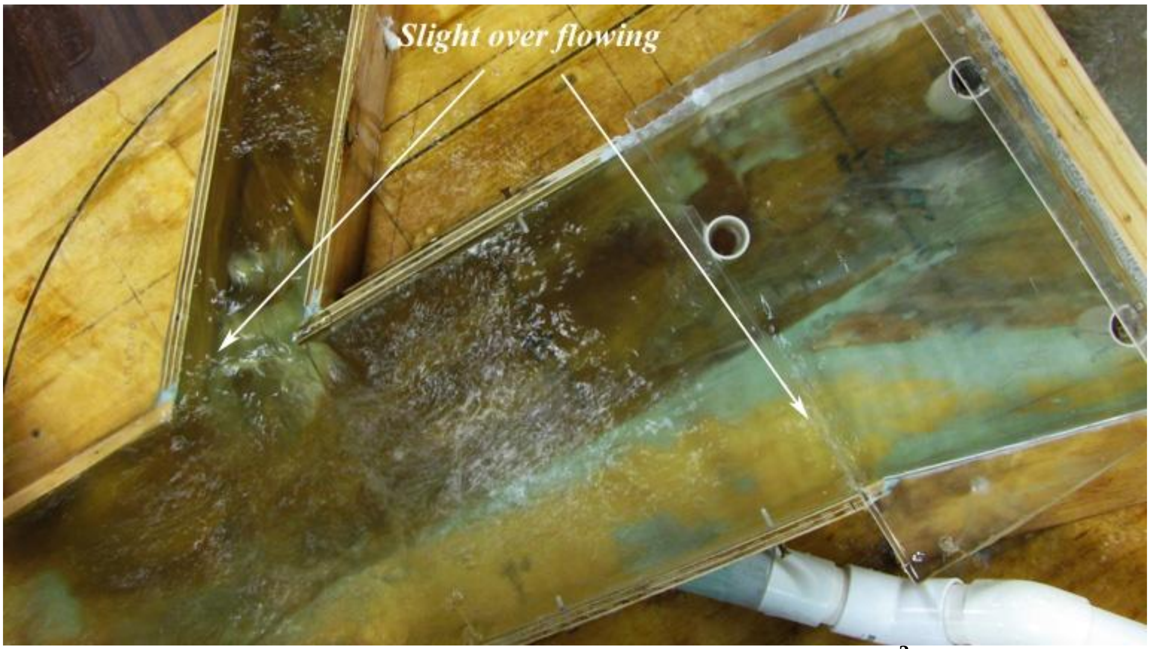

Figure 5: Overflowing at maximum flow rate (41m^3/s)

3.4 Design Scenario 3 Including Effects of Weir

The next stage of testing involved the use of a weir to simulate the back water effect. Using Manning's equation the design water level was calculated for the outlet of the scale model and a weir was used to raise the outlet to this level. The main concern was that the back water effect may cause problems further upstream in the model as seen in Design Scenario 2 with pressurisation occurring. However, this was not the case. While the weir did cause the water level to rise throughout the system it was neither significant enough to result in system pressurisation nor to hinder the efficiency of the baffle blocks and the existing energy dissipater. The only cause for concern (Figure) was the rising water level in the uncovered section of the model. With the addition of the weir the model now overflowed at design flows and more intensely at maximum tested flow rate (41m^3/s).

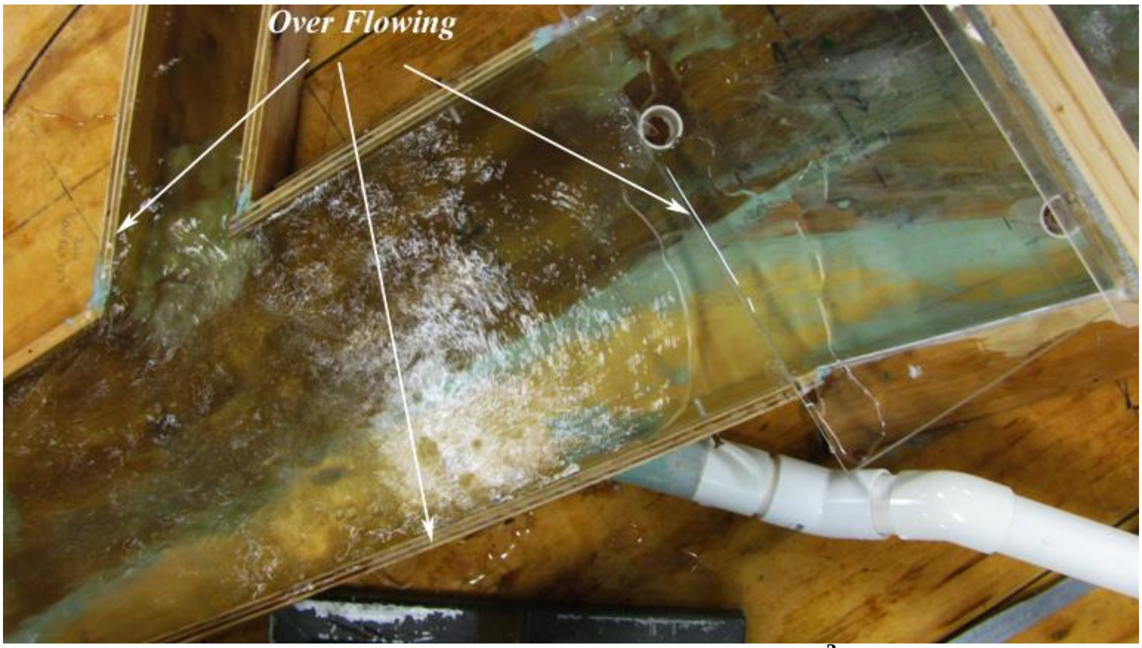

Figure 6: Overflowing at maximum flow rate (41m^3/s) with weir

4 CONCLUSIONS

The final design (Design Scenario 3) has adequate capacity to cope with the 100 year design flood of 27.4m^3/s and was recommended for implementation. From an economic point of view, it is not the most cost-effective solution, however, given the problems encountered with possible structural damage during operation due to high jet forces (Design Scenario 1) and severe pressurization (Design Scenario 2), it is the most viable option overall.

With the modifications made to the 1200∅ outlet, no extra erosion protection will be required for the piles. The outlet was expanded to a rectangular cross section with a flared end and no longer forms a jet which hits the piles. During initial operation of the new project, forces acting on the piles should be observed and if necessary, minor damage should be repaired as part of a maintenance program. Furthermore, ventilation should be considered in the vicinity of where the energy dissipater is located.

To prevent flooding during severe storms the height of the end section of the system could be increased slightly (to approximately 2.7m). Implementing the channel geometry of Design Scenario 3 also minimises head losses.

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