A Scale Model Study of Flow Behavior of Storm Diversion Structure

A Thesis

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CERTIFICATE OF RESEARCH

This is to certify that this thesis work has been done by me and neither this thesis nor any part of thereof has been submitted elsewhere for the award of any degree or diploma.

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ABSTRACT

The low-lying area behind the Sonargaon Hotel and the Hatirjheel lowlands, extending from the east of Tejgaon diversion road up to the Pragati Shwarani at Rampura, receive significant discharges through a number of major storm sewer outfalls. Illegal connections of both domestic and industrial wastewaters to the storm sewer network are usual case. As a result, during dry season, the storm sewers mainly carry significant flows of domestic sewage as well as industrial wastewater. The untreated domestic sewage and industrial wastewater drains through this low land via the Begunbari khal-Norai khal into the Balu river; the Balu river eventually discharges into the Sitalakhya river. During monsoon, the pollution level drops to some extent due to dilution of domestic sewage and industrial effluent by rainwater/ stormwater. Over the years, the lowlands behind Sonargaon Hotel and Hatirjheel have virtually been turned into wastelands. In order to manage this combined water flow, storm water diversion system can be introduced.

Storm Diversion structure (SDS) is a structure which is used to separate dry weather flow from storm water flow. Working principle of SDS is that storm water combined with dry weather sewerage flow is used to divert excess flows received during storm events into nearby receiving water body (lake), thus relieving other hydraulic structures within the area and reducing the risk of flooding in urban areas.

In this study, attempt has been made to study the hydraulic behavior of a Storm Diversion Structure (SDS) in laboratory model under various flow conditions. In addition the overflow gate operation at various flow conditions have also been studied. The flow condition of the receiving watercourse and within the structure chamber has been observed for various gate operations. For a typical scale model study outfall Q4 of hatirjel- Begunbari area located near Tejgaon diversion road has been selected. Physical modeling facility (46m x 11 m) of Department of Water Resources Engineering (DWRE), BUET has been used for this purpose. Froudian law has been applied to design the laboratory scale model. Based on the availability of the space and the discharge capacity in modeling facilities, an undistorted model of scale 1:4 has been selected. A total number of 13 test runs were performed. Total of eight different discharges and seven different overflow gate openings have been considered in the present study. The bypass conduit opening was fixed at 76.2 mm as that of the prototype condition.

It reveals from model study that the gate heights should be maintained in such as way that the water flow through combined system and dry weather flow through bypass pipe can be maintained. To do so, the overflow gate height is to be decreased to increase the total flow of combined sewer and vice versa (that has to be increased for decrease of total flow of combined sewer). The Froude number at various components have been calculated and found that the flow is always subcritical in storm diversion structure and found supercritical flow at bypass pipe for the flow rate greater than the dry weather flow. A hydraulic jump was observed in the under flow bypass pipe. To check the prototype design, normal depth of the flow through bypass pipe has been calculated from the Froude number and depth of water at storm diversion structure. All the model results and observations have been transformed in to prototype designed condition and found satisfactory agreement.

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LIST OF NOTATIONS AND SYMBOLS

Notation	Meaning	Unit
F_r	= Froude number	(-)
g	= acceleration due to gravity	(m/s^2)
h	= average water depth	(m)
h_x	= distance downstream submerged vanes	(m)
Q	= discharge	(m^3/s)
R_e	= Reynolds Number	(-)
R_b	= radius of bypass pipe	(m)
S_r	= lateral water surface slope	(-)
t	= time	(s)
Δt	= change in time	(s)
U	= depth averaged stream wise velocity	(m/s)
Ua	=Cross-sectional average velocity	(m/s)
u	= depth averaged flow velocity	(m/s)
$\bar{u}$	= depth averaged value of local velocity in x-direction	(m/s)
$\bar{u}_v$	= depth averaged approach flow velocity for vane	(m/s)
v	= perpendicular flow velocity due to horizontal vortex	(m/s)
V	= lateral near bed velocity	(m/s)
v_2	= perpendicular flow velocity due to vertical vortex	(m/s)
v_{max}	= maximum perpendicular flow velocity due to horizontal vortex	(m/s)
v_{max2}	= maximum perpendicular flow velocity due to vertical vortex	(m/s)
$\bar{v}$	= depth averaged mean velocity of flow	(m/s)
X,Y	= cartesian coordinate system	(m)
Z	= vertical co-ordinate	(m)
v	= Kinematic viscosity	(m^2/s)
•	= density	(kg/m^3)

CHAPTER 1

INTRODUCTION

1.1 Background

The low-lying area behind the Sonargaon Hotel and the Hatirjheel lowlands, extending from the east of Tejgaon diversion road up to the Pragati Shwarani at Rampura, receive significant discharges through a number of major storm sewer outfalls. Illegal connections of both domestic and industrial wastewaters to the storm sewer network are usual case. As a result, during dry season, the storm sewers mainly carry significant flows of domestic sewage as well as industrial wastewater. The untreated domestic sewage and industrial wastewater drains through this low land via the Begunbari khal-Norai khal into the Balu river; the Balu river eventually discharges into the Sitalakhya river. During monsoon, the pollution level drops to some extent due to dilution of domestic sewage and industrial effluent by rainwater/ stormwater. Over the years, the lowlands behind Sonargaon Hotel and Hatirjheel have virtually been turned into wastelands. (Rajuk, 2005)

The Government of Bangladesh is now being carried integrated development of Hatirjeel-Bagunbari khal system . One of the development component of Hatirjeel area is to separate the dry weather flow from the storm water, so that the lake water can be at free of pollution Thus it is necessary to design a flow separator so called named as Storm Diversion Structure (SDS) in the outflows of the existing dry weather flow system. This system has to be adopted with the existing system. As the SDS has to bear a resemblance to the existing outflows, the present study aims to develop a scale model to carry out study the flow behavior of the structure under various discharge and outflow gate operation conditions. From this study, hydraulic behavior of storm diversion structure will be tested in a scale model so that flow behavior of storm diversion through the overflow and under flow pipe system can be understood before the construction.

1.2 Objectives with specific aims

The specific objectives of this study are as follows:

1) To design and develop a scale model in laboratory to conduct the study for hydraulic behavior of Storm Diversion Structure (SDS).
2) To study of overflow gate operation at various flow condition.

1.3 Organization of project thesis

The first chapter of this project report deals with the general background, scope and objectives of the study. In second chapter, review of literature has been overviewed and in chapter three design of storm diversion structure with specific reference of Hatirjhil Begunbari integrated project has been detailed. Experimental facility, experimental setup and test run have been discussed in Chapter four. Physical model results and discussions have been given in Chapter five. Finally in Chapter six conclusions and future recommendations are reported.

CHAPTER 2
REVIEW OF LITERATURE

2.1 Various types of system for Treatment of pollutants

Discharges from catchment contain both foul sewage and storm water and therefore contain large amounts of pollutants, including gross solids and finely suspended solids in solution. These pollutants can have a significant aesthetic, oxygen demand or toxic impact on the quality of the receiving water. In general, either or combination of the following four types of system can be used to treat the pollutants

- A combined sewer overflow structure (CSO)
- An emergency overflow at a pumping station and / or detention tanks
- An overflow from storm tanks at a sewage / wastewater treatment works, and
- An overflow from an emergency spill weir at a sewage / wastewater treatment works.

In order to manage combine water flow, storm water diversion system can be introduced. This combined flow regulatory is based on the conveyance of domestic and industrial effluents and the surface runoff from catchments surfaces in underground conduits or open drains. The storm water combined with dry weather sewerage flow is used to divert excess flows received during storm events into nearby receiving water body (lake), thus relieving other hydraulic structures within the area and reducing the risk of flooding in urban areas.

Diversion system needs to be inspected internally and each chamber and screen are to be subjected to a detailed mechanical, structural and operational inspection. During operation, the condition of the receiving watercourse has to be assessed wherever possible. (Burrian, 1999)

2.2 Combined Sewer Overflow structure (CSOs)

A combined sewer is a sewer that is designed to carry both sanitary sewage and storm water runoff in a single pipe system. Discharge from a combined sewer system occurs in response to rainfall because the carrying capacity of the sewer system is exceeded. These discharges do not receive all treatment that is available and utilized under ordinary dry weather conditions (normally during dry weather conditions the wastewater is transported to a wastewater treatment facility where it receives appropriate treatment prior to discharge). This excess discharge which have to disposed to cannel or river is referred as Combined Sewer Overflow (CSO) and the structure that used to diverted this overflow to the cannel is referred as Combined Sewer Overflow structure (CSOs), specially here termed as Storm Diversion Structure (SDS).

From the late 1800s through the 1940s, engineers designed combined sewers (sewers that carry sewage and stormwater runoff in a single pipe) to convey sewage, horse manure, street and rooftop runoff, and garbage from city streets to the nearest receiving body of water. Combined sewers can cause serious water pollution problems due to combined sewer overflows, which are caused by large variations in flow between dry and wet weather. This type of sewer design is no longer used in building new communities. (Crow, 2008)

A Combined Sewer Overflow, or CSO, is the discharge of wastewater and stormwater from a combined sewer system directly into a river, stream, lake or ocean. Overflow frequency and duration varies both from system to system, and from outfall to outfall, within a single combined sewer system. Some CSOs outfalls discharge infrequently, while others activate every time it rains. During heavy rainfall when the stormwater exceeds the sanitary flow, the CSO is diluted.

The storm water component contributes a significant amount of pollutants to CSO. Each storm is different in the quantity and type of pollutants it contributes. For example, storms that occur in late summer, when it has not rained for a while, have the most pollutants. Pollutants like oil, grease, fecal coliform from pet and wildlife waste, and pesticides get flushed into the sewer system. In cold weather areas, pollutants from cars, people and animals also accumulate on hard surfaces and grass during the winter and then are flushed into the sewer systems during heavy spring rains.

Combined sewer systems are sewers that are designed to collect rainwater runoff, domestic sewage, and industrial wastewater in the same pipe. Most of the time, combined sewer systems transport all of their wastewater to a sewage treatment plant, where it is treated and then discharged to a water body. During periods of heavy rainfall, however, the wastewater volume in a combined sewer system can exceed the capacity of the sewer system or treatment plant. For this reason, combined sewer systems are designed to overflow occasionally and discharge excess wastewater directly to nearby streams, rivers, or other water bodies.

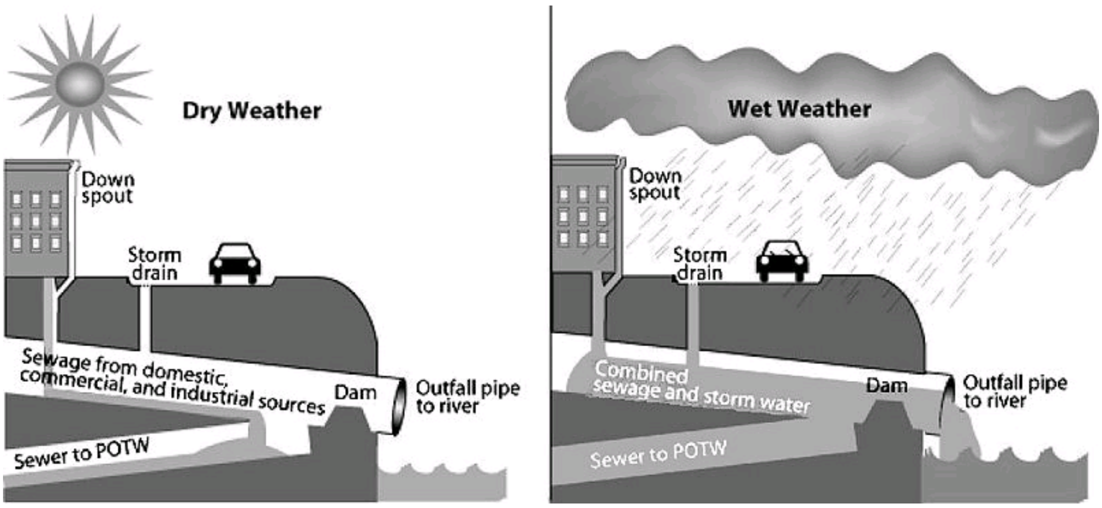


Fig- 2.1: Definition sketch Combined Sewer System.

In order to manage combine water flow, storm water diversion system can be introduced. This combined flow regulatory has designed based on the conveyance of domestic and industrial effluents and the surface runoff from catchments surfaces in underground conduits or open drains. The storm water combined with dry weather sewerage flow is used to divert excess flows received during storm events into nearby receiving water body (lake), thus relieving other hydraulic structures within the area and reducing the risk of flooding in urban areas.(Wikipedia)

But now a day, most sewer systems were built as separated systems (sewage in one pipe; stormwater in another pipe). In the late 1950s, treating wastewater became the standard. Interceptor pipes were built to transport all wastewater (from either combined or separated systems) to treatment plants.

In Dhaka city illegal connections of both domestic and industrial wastewaters to the storm sewer network are usual case. As a result, during dry season, the storm sewers mainly carry significant flows of domestic sewage as well as industrial wastewater.

Diversion system needs to be inspected internally and each chamber and screen are subjected to a detailed mechanical, structural and operational inspection. During operation, the condition of the receiving watercourse has to be assessed wherever possible.

2.2.1 Principal of CSOs/SDS

In this simplified illustration, the combined sewer line is blocked by a low weir, or dam, before it reaches the stream. The weir diverts the flow into the interceptor sewer, which takes it to a sewage treatment plant. In dry weather, all of the flow is sanitary sewage, and the interceptor line can handle it. In wet weather, stormwater mixes with the sanitary sewage, increasing the flow. If the flow is large enough, part of the water may flow over the weir and through the CSO into the stream. Most overflows are equipped with valves that allow overflow water to enter the stream, but prevent stream water from entering the sewers. The simplest is the flap valve, shown here. Some valves may be inside the sewer lines, where they cannot be seen from the creek.

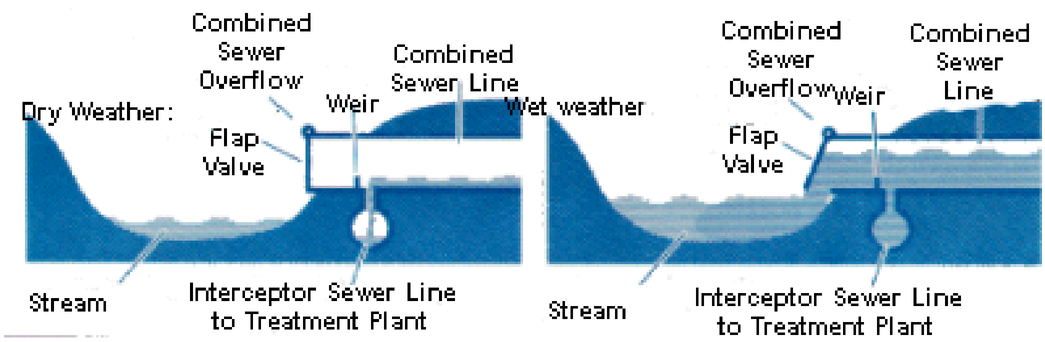


Figure 2.2: A simple CSOs, (Flap valve)

2.3 Sluice gate in CSO Chamber

A sluice gate is traditionally a wooden or metal plate which slides in grooves in the sides of the channel. Sluice gates are commonly used to control water levels and flow rates in rivers and canals. They are also used in wastewater treatment plants and to recover minerals in mining operations, and in watermills.

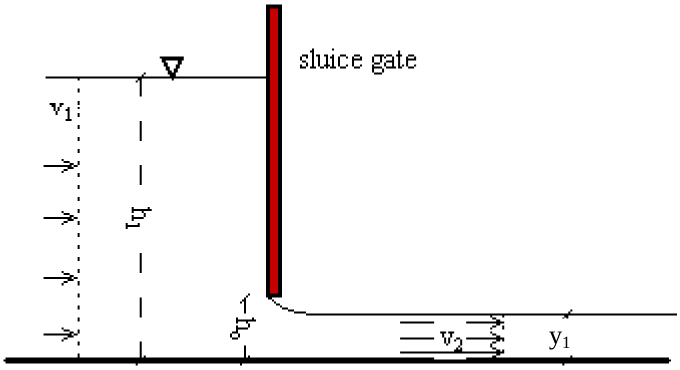


Figure 2.3: Definition sketch of sluice gate.

The sluice gate flow rate measurement is based on the Bernoulli Equation principles and can be expressed as:

$\frac{1}{2}\ v_1^2 + g\ h_1 = \frac{1}{2}\ v_2^2 + g\ h_2$ (2.1)

Where, h is elevation height and v is flow velocity

The pressure components are in general irrelevant since the pressures upstream and downstream are the same ($p_1 - p_2 = 0$).

Assuming that the velocity profiles are uniform in the upstream and downstream section the continuity equation gives:

$q = v_1\ A_1 \ = v_2\ A_2$ (2.2)

Where, q is flow rate, A is flow area

Equation (2.2) can be written as

$q = v_1\ h_1\ b \ = v_2\ h_2\ b$ (2.3)

Where, b is width of the sluice, h_1 is upstream height, h_2 is downstream height

Combining Equation (2.1) and (2.3), Equation (2.2) can be written as:

$q = h_2\ b\ [\ 2\ g\ (h_1 - h_2) / (1 - (h_2 / h_1))\ ]^{1/2}$ (2.4)

Assuming $h_1 >> h_2$ (2.4) can be modified to:

$q = h_2\ b\ [\ 2\ g\ h_1\ ]^{1/2}$ (2.5)

This is approximately true when the depth ratio h_1 / h_2 is large, the kinetic energy upstream is negligible (v_1 is small) and the fluid velocity after it has fallen the distance $(h_2 - h_1) \bullet h_1$ - is:

$v_2 = [\ 2\ g\ h_1\ ]^{1/2}$ (2.6)

The equation (2.5) can be modified with a discharge or contraction coefficient:

$q = c_d\ h_2\ b\ [\ 2\ g\ h_1\ ]^{1/2}$ (2.7)

Where, c_d is discharge or contraction coefficient

The discharge coefficient c_d is a function of the opening height and the height of vena contracta. Again vena contracta is a function of slope of bypass pipe and bypass pipe sluice gate opening.

Henderson proposed an equation for the contraction coefficient δ for radial (Tainter) gate which depends on inclination angle θ

$$\delta = 1 - 0.75\left(\frac{\theta}{90^o}\right) + 0.36\left(\frac{\theta}{90^o}\right)^2 \qquad (2.8)$$

The expected error is less than 5% provided that $\theta < 90^o$. Thus the discharge coefficient is given by $C_d = \dfrac{\delta}{\left(1 + \dfrac{\delta w}{y_1}\right)^{0.5}}$ (2.9)

The discharge coefficient ranges between 0.607 to 0.596.

2.4 Hydraulic jump in CSO

Hydraulic jump is a classical hydrodynamic phenomenon. The figure below depicts a cross section of water flowing through bypass pipe with a fixed gate opening of 76.2mm. Given certain conditions of flow rate, height of water, and channel widths a hydraulic jump will occur where the water level rises downstream. The area between the two water levels is the transition or turbulent region. Calculating the overall energy before and after the jump there is an energy loss. Most references attribute this loss to turbulence or otherwise stating that energy is dissipated when there is turbulence.

The studies provide formulas to tell us when to expect a hydraulic jump to occur. The energy loss in a hydraulic jump is attributed to losses due to turbulence in the transition region.

Energy loss in a hydraulic jump (• E)

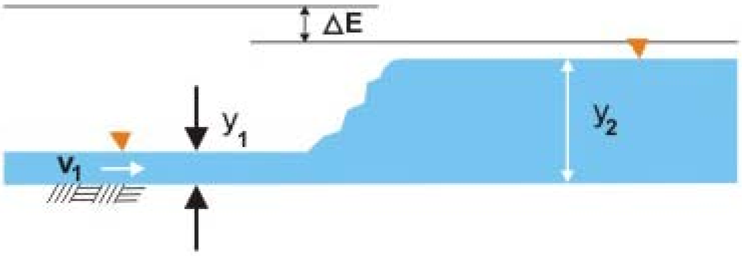


Fig: 2.4 Hydraulic jump in bypass pipe

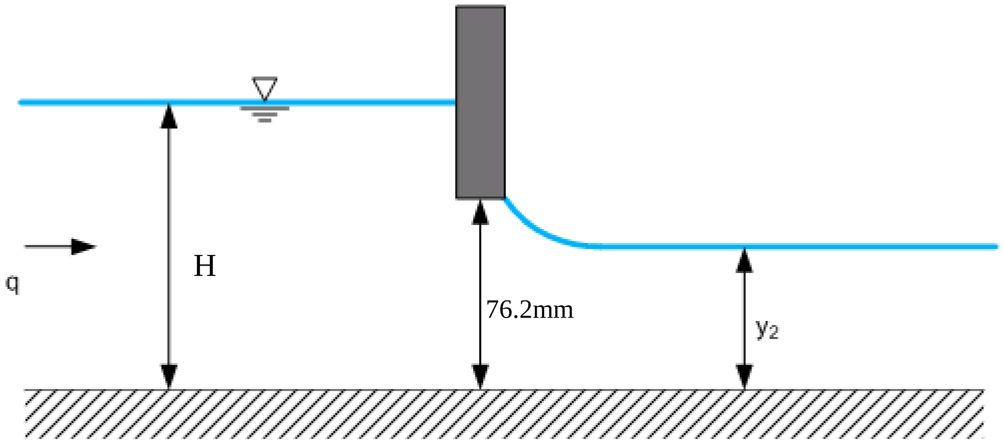


Fig: 2.5 Flow through sluice gate of bypass pipe.

A hydraulic jump is a natural phenomenon that occurs when a higher velocity, v_1, supercritical flow upstream is met by a subcritical downstream flow with a decreased velocity, v_2, . The depth of supercritical flow, y_1, 'jumps' up to its subcritical conjugate depth, y_2. The result of this abrupt change in flow conditions is considerable turbulence and Energy Loss, E_L. Figure 2.6 shows a schematic of typical jump characteristics where E_1 is the energy of the upstream flow, E_2 is the energy of the downstream flow and L_j is the length of the hydraulic jump. A series of small surface rollers are formed in a standing wave like the one shown in Figure 2.6.

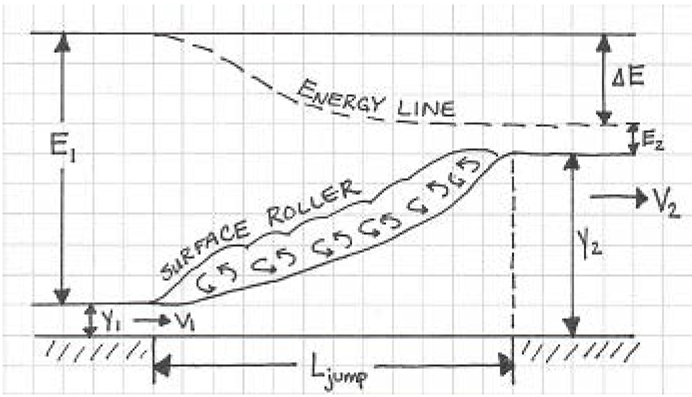


Figure 2.6: Hydraulic Jump Overall Schematic

The height of the hydraulic jump, similar to length, is useful to know when designing the bypass pipe. The height of the hydraulic jump is simply the difference in flow depths prior to and after the hydraulic jump. The height can be determined using the Froude number and upstream energy

Height of jump, $h_j = y_2 - y_1$ (2.10)

$$y_2 = \frac{y_1}{2}\left(\sqrt{1+8F_r^{\,2}} - 1\right) \quad (2.11)$$

The types of jump in the bypass pipe can be determined based on Froude number in bypass pipe. Table 2.2 can be used to identify the type of the jump.

Table 2.1: Types of Hydraulic Jumps

Name	Froude’s Number	Energy dissipation	Characteristics
Undular Jump	1.0-1.7	<5%	Standing waves
Weak Jump	1.7-2.5	5-15%	Smooth rise
Oscillating Jump	2.5-4.5	15-45%	Unstable; avoid
Steady Jump	4.5-9.0	45-70%	Best design range
Strong Jump	>9.0	70-85%	Choppy, intermittent

CHAPTER 3

DRAINAGE OUTFALLS OF HATIRJHIL BEGUNBARI LAKE

3.1 GENERAL

As mentioned in chapter 2 (article 2.2) the Government of Bangladesh is now being carried integrated development of Hatirjeel-Bagunbari khal system. One of the development component of Hatirjeel area is to separate the dry weather flow from the storm water, so that the lake water can be free of pollution Thus it is necessary to design a flow separator so called named as Storm Diversion Structure (SDS) at the outflows of the existing dry weather flow system. This system has to be adopted with the existing system. As the SDS has to bear a resemblance to the existing outflows, the present study aims to develop a scale model to carry out study the flow behavior of the structure under various discharge and outflow gate operation conditions.

The works related to the construction of main diversion sewer system include the following works which has been done by Government of Bangladesh:

(i) Identification of major sewer outfalls discharging into the project lowlands;

(ii) Measurement of invert levels and the dry weather flow through these major outfalls;

(iii) Setting alignment of the main diversion sewer along the southern and northern boundaries of the project site;

(iv) Design of the main diversion sewer, including hydraulic, structural and foundation design;

(v) Design of Combined Sewer Overflow structure (CSOs) for all major storm sewer outfalls in the project area, along with necessary treatment facilities; and

(vi) Design of manholes at appropriate locations along the main diversion sewer lines. The works completed so far are summarized below.

3.2 Hatirjhil Begunbari integrated project

The low-lying area behind the Sonargaon Hotel and the Hatirjheel lowlands, extending from the east of Tongi diversion road up to the Pragati Shwarani at Rampura, receive significant discharges through a number of major storm sewer outfalls. However, illegal connections of both domestic and industrial wastewaters to the storm sewer network in rampant. As a result, during dry season, the storm sewers mainly carry significant flows of domestic sewage as well as industrial wastewater. The untreated domestic sewage and industrial wastewater drains through this low land via the Begunbari khal-Norai khal into the Balu river; the Balu river eventually discharges into the Sitalakhya river. During monsoon, the pollution level drops to some extent due to dilution of domestic sewage and industrial effluent by rainwater/ stormwater. The domestic sewage and industrial wastewater flowing through the lowlands have already degraded the environment of the area very severely. Over the years, the lowlands behind Sonargaon Hotel and Hatirjheel have virtually been turned into wastelands.

The Integrated development of Hatirjeel Begunbari projects are aimed at improving the appallingly degraded environment of the area and protecting the wetlands, which will also perform the very important function of detention of the storm water during monsoon. Alleviation of traffic congestion through connecting the east-west missing link from Tongi diversion road to Pragati Shwarani is another major objective of the project. The integrated development project is being implemented jointly by RAJUK of the Ministry of Housing and Works, and DWASA and LGED of the Ministry of Local Government, Rural Development and Cooperatives. The proposed Hatirjheel development project plans to develop the Hatirjheel area as a retention pond/lake, which would be used as a recreational facility by the city dwellers. However, this requires that the water quality of the low lying areas behind the Sonargaon Hotel and the Hatirjheel areas are maintained at an acceptable level. Considering the present situation with regard to uncontrolled, untreated domestic and storm sewage disposal within and around the areas, restoration of the water quality to the desired level would certainly be a major challenge.

In order to prevent pollution, remove water logging, and to improve water quality condition of the entire wetlands, DWASA has planned to take up a project that would divert all the domestic and industrial wastewaters currently being discharged into the Begunbari-Hatirjheel area. The diversion of the wastewaters would be implemented through construction of large diameter diversion sewers running along the southern and northern peripheries of the project site. A major challenge of this diversion process is the separation of wastewater from rainwater/stormwater during the wet season. For taking care of this, it has been proposed that combined sewer overflow structure (CSOs) will be constructed at the outfalls of all major storm sewers currently discharging into the Hatirjheel. These overflow structures would divert the dry season flow coming through the storm sewers, consisting of mostly domestic wastewater, through the main diversion sewers; while during the wet season, the heavier portion of the combined flow coming through the sewers would be diverted through this sewer, while the lighter fraction, consisting mostly of storm runoff would be discharged into the proposed Begunbari-Hatirjheel lowlands. As additional safety measures, some treatment of the “overflow water” has also been proposed.

In addition to the main diversion sewers, the proposed project also plans to design and construct local diversion sewers for diversion of domestic sewage of the un-sewered areas located adjacent to the project site, that are now discharged into the project lowlands. In addition, the project also plans to install three deep tubewells (DTWs) and construct DTW pump stations, in anticipation of increased water demand resulting from increased residential and commercial development of the area after completion of the project. The project also plans to lay water supply lines (mains) along the peripheral roads to be constructed as a part of this project.

All the above activities have been considered in the DWASA component of the “Integrated Development of Hatirjheel Area including Begunbari Khal” project. The DPP of the project was approved by the ECNEC of Bangladesh Government on 08 October 2007. Subsequently an agreement was signed between DWASA and BRTC, BUET on 07 January 2008 for related studies and design of DWASA activities for the Integrated

Development of Hatirjheel Area including part of Begunbari Khal. (Matin and Asifur,2009)

3.3 Major Drainage Outfalls

The first and very important task in connection with the design of the main diversion sewers was the identification of major storm sewer outfalls currently being discharged into the lowlands behind Sonargaon Hotel and Hatirjheel. For this purpose, available maps on storm sewer network and drainage were collected from DWASA. The major storm sewers and drainage lines currently discharging into the project area were identified from these maps. Relevant DWASA officials assisted the BUET team in identification of these storm sewer outfalls.

The next step was identification of these storm sewer outfalls and other major discharge points, if any, in the field. For this purpose, a detailed field reconnaissance survey was carried out during January 2008 by a team consisting of BUET consultants, DWASA officials and officials of the 16 ECB, SWO, Bangladesh Army. During this survey, all major storm/domestic/industrial wastewater outlets currently discharging into the project lowland were identified and their GPS coordinates were recorded.

Figure 3.1 shows the locations of 9 major outfalls along the periphery of the project area; the outfalls are identified as Q1 through Q9. In addition to these 9 major outfalls, a number of smaller discharges have also been identified [notably in Moghbazar, Ulan, Tejgaon, and Gulshan (close to Aarong and the National Shooting Complex) areas]. These 9 major outfalls discharging into the project lowlands are as follows:

(1) The major storm sewer (6.2mx6.0m and 2.6mx3.9m twin box-culvert) outfall behind Soargaon Hotel (Q1, Photograph 3.1), which discharges stormwater/wastewater coming from through Panthapath area in the west through a box-culvert and those coming through a brick sewer (formerly Paribagh khal) from the south-west.

(2) The 96-inch diameter storm sewer outfall (Q2) coming from the south and discharging into the Hatirjheel immediately to the east of Tongi diversion road (Photograph 3.2).

(3) The storm sewer outfall coming from the Karwan Bazar area and discharging into the project lowlands at a location in front of the entrance of the BGMEA building (Q3).

(4) The 48 inch diameter storm sewer outfall (Q4) coming from the north and discharging into the Hatirjheel immediately to the east of Tongi diversion road. This sewer carries a part of industrial discharges from the Tejgaon industrial area (Photograph 3.3).

(5) The storm water drainage khal that discharges into the Hatirjheel at Madhubagh, Nayatola (at Gudaraghat). The Tejgaon Sewage Lifting Station of DWASA is located just on the other side of Hatirjheel at this location. A 48-inch diameter main domestic sewer of DWASA (often referred to as “mother sewer”) crosses the Hatirjheel at this location (see Figure. 3.1).

(6) The major storm sewer outfall at Niketon (5.5mx3.8m box-culvert). The discharge through this outfall [Q6(s)] mixes with that of Banani Lake [Q6(L)] (see Photograph 3.5), crosses the Tejgaon-Gulshan Link road underneath the bridge located close to Gulshan Aarong store and the combined flow (Q6) then discharges into the Hatirjheel.

(7) The 66-inch diameter storm sewer (Q7) coming from Mouchak-Rampura area and discharging into the Hatirjheel immediately to the west of the Progati Shwarani (close to Rampura Bridge) (see Photograph 3.6).

(8) The storm sewer [Q8(s)] coming from Badda area, mixing with the discharge of Gulshan Lake [Q8(L)]; the combined flow (Q8) then discharges into the Hatirjheel (close to and behind the National Shooting Complex).

(9) The 48-inch diameter storm sewer outfall (Q9) that comin from the Badda area and discharging into the Hatirjheel immediately to the west of the Progati Shwarani (close to Rampura Bridge) (see Photograph 3.7). This sewer appears to carry industrial discharges as is evidenced from the colored wastewater coming through this outfall (Photograph 6).

Photograph 3.1: Storm sewer outfall behind Sonargaon Hotel (Q1)

Photograph 3.2: 96-inch diameter storm sewer outfall located immediately to the east of Tongi Diversion Road (Q2)

Photograph 3.3: Out fall located at the North-west Tajgaon diversion road (Q4).

Photograph 3.4: Illegal Cacha Toilet Polluting Hatirjeel-Bagunbari khal.

Photograph 3.5: A view of the box-culvert storm sewer at Niketon; flow from the Banani Lake is coming from the right and mixing with the storm sewer discharge (Q6)

Photograph 3.6: Storm sewer coming from Mouchak-Rampura area and discharging into Hatirjheel near Rampura bridge (Q7)

Photograph 3.7: Greenish colored industrial wastewater being discharged near into the Hatirjheel (near Rampura bridge) through the storm sewer (coming from Badda area, Q9)

3.4 Measurement of Invert Levels of Major Outfalls

The proposed main diversion sewer for diverting the dry weather flow currently being discharged into the project lowlands would be built as an "intercepting sewer" that would receive flows coming only through the major sewer outfalls. No lateral or branch sewers would be connected to this main diversion sewer. For setting alignment and profile of such an intercepting sewer, a very important piece of information is the invert levels of the main sewers (in this case major sewer outfalls) that would discharge into this main diversion sewer. It was therefore necessary to measure the invert elevations of major outfalls (listed in Table: 3.1) with very good accuracy.

In order to accurately measure the invert elevations, it was necessary to establish a temporary bench mark at the project site. The Survey of Bangladesh (SOB) was therefore requested to establish a benchmark at the project site. In response to the request, the SOB set a bench mark at the top of the DWASA concrete manhole located immediately to the east of Tongi diversion road. The RL at the top of this manhole, as determined by the SOB, is 7.3266 m (SOB). Using this bench mark, invert elevations of all 9 outfalls (listed above) were determined by the survey team of BRTC, BUET. The final RL of all points were expressed as "m (PWD)", which is equal to the "RL in m (SOB) + 0.4599 m". The invert elevations of the major outfalls are reported in Figure 3.1. It should be noted that the invert elevation of Q5 at Nayotala was measured at the end point of box the box-culvert, which then becomes a small khal and discharges into the Hatirjheel. The invert elevations of the major outfalls are also listed in Table 3.1.

Table 3.1: Invert elevations of all major outfalls discharging into the Begunbari khal-Hatirjheel lowlands (Matin and Asif,2009)

Outfall Identification	Invert Elevation (m PWD)	Remark
Q1 (behind Sonargaon Hotel)	1.76	Invert elevation 6.2x6.0 m box culvert
Q2 (east of Tongi diversion Raod)	2.28	Measured at the point of discharge
Q3 (storm sewer from Karwan Bazar)	2.96	Measured at the manhole located at the entrance of the BGMEA building
Q4 (east of Tongi diversion Raod)	2.81	Measured at the point of discharge
Q5 (Modhubagh, Nayatola)	3.53	Measured at the end of the box culvert, upstream of the khal
Q6 (S) (Niketon outfall)	1.35	Invert level of box culvert
Q7 (outfall on the southern side of Rampura bridge)	2.29	Measured at the point of discharge
Q8 (S) (storm sewer from Badda)	1.87	
Q8(L)	1.29	
Q9 (outfall on the northern side of Rampura bridge	2.77	Measured at the point of discharge
Top of Kazi Nazrul Islam Avenue	8.04	
Top of Tongi Diversion Road	7.59	

Figure 3.1 shows that among the outfalls located along the southern periphery of the project site, the lowest invert level is 1.7555 m (PWD) for outfall Q1 (outfall behind Sonargaon Hotel). For the outfalls along northern periphery, the lowest invert level is 1.3530 m (PWD) for the outfall Q6(S).

3.5 Measurement of Dry Weather Flow

Estimation of dry weather flow through the major outfalls listed above was a very important task, based on which the hydraulic capacity of the main diversion sewers will have to be determined. Estimation of dry weather flow through all 9 major outfalls was carried out in January 2008. In addition to the flow measurement at the main outfall locations, flow measurements were also carried out at different sections of the main khal (channel) flowing through the lowlands behind Sonargaon Hotel and Hatirjheel. The additional flow measurements (along the khal) were carried out for cross-checking the flow measurement values at the outfall locations and also for estimating (through back calculation) the relatively small discharges that flow into the project lowlands through numerous point and non-point sources and eventually are carried along the main khal, during the dry season.

A number of techniques were used for discharge measurements. Majority of discharge measurements were accomplished through measurement of flow velocity through a current meter and determination of cross-sectional area of flow. Flow velocities were also estimated by measuring the time required by a naturally floating substance in water to traverse a fixed distance. In case of channel/khal, the cross-sectional area of flow was estimated by measuring depth of flow at different sections along the width of the channel. In case of box-culvert [e.g., Q6(S)], the depth of flow and width of box-culvert provided estimates of x-sectional area of flow; while in case of circular pipe (e.g., Q7, Q9 shown in Photograph 3.6 and Photograph 3.7), the maximum depth of flow (i.e. at the mid-section of the pipe) and the diameter of the pipe provided estimates of x-sectional area of flow. Besides current meter, some discharge measurements were also carried out with Acoustic Doppler Equipment, fitted with an online data logger. Results obtained through these two methods were in good agreement.

Because of difficulties in setting the current meter, discharge through the twin box-culvert behind Sonargaon Hotel (Q1) was measured at three different sections along the khal/channel behind Sonargaon Hotel. The first section was located about 500 ft downstream of the discharge point the second section was located behind the BIAM building, and the third section was located close to the BGMEA building.

Because the discharge through the outfall Q4 was relatively low and the shallow depth of flow did not allow setting the flow meter, discharge at this point was estimated by measuring the time required for the flow to fill bucket of know volume (60 L). The combined flow from the first 4 major outfalls (i.e., Q1, Q2, Q3, Q4 and any additional flows) was measured at the khal/channel section about 500 ft downstream from the Tongi diversion road; this has been referred to as Q_c in Table 3.2. Because of difficulties in setting flow meter at Q3 location, the discharge at this point was estimated by deducting the measured discharges Q1, Q2 and Q4 from the combined discharge measured at the khal/channel. Discharge at Modhubagh, Nayatola outfall was measured at the drainage khal (downstream of the box-culvert) that discharges into the Hatirjheel. At Q6 location, discharge was measured separately at the storm sewer outfall [Q6(S)] and at Banani Lake. In addition, measurement of the combined flow [Q6(S) and Q6(L)] plus smaller additional flows (e.g. industrial discharge through a pipe beneath the bridge on Tejgaon-Gulshan Link Raod) were measured at a location about 800 ft downstream of the of the bridge; this flow is reported as Q6 in Table 3.2. Similarly, for Q8 locations, discharge was measured separately for the sewer [Q8(S)] and lake [Q8(L)].

Table 3.2: Discharge through major outfalls discharging into the lowland behind Sonargaon Hotel and Hatirjheel (Matin and Asifur,2009)

Discharge	Date of Measurement	Time of Measurement	Discharge (cfs)	Remark
Q1	07-01-08	10:00-12:00	61	Average of measurements at 3 different sections
Q2	13-01-08	11:00-11:10	10	
Q3	13-01-08	12:00-12:10	7	(=Qc-Q1-Q2-Q4)
Q4	**13-01-08**	**10:30-10:40**	**2**	
Qc	13-01-08	11:45-12:30	80	Combined flow (Q1+Q2+Q3+Q4+others)
Q5	13-01-08	13:45-14:00	4	
Q6(S)	21-01-08	11:00-11:30	22	
Q6(L)	21-01-08	11:30-12:00	14	
Q6	21-01-08	12:15-13:15	38	Combined flow [Q6(S)+Q6(L)+Others]
Q7	21-01-08	14:15-14:30	11	
Q8(S)	11-02-08	10:50-11:00	3	
Q8(L)	11-02-08	10:35-10:45	14	
Q9	21-01-08	13:45-14:00	4	

Table 2 shows the measured discharge at the major outfalls. Table shows that among the 9 outfalls, the outfall behind Sonargaon Hotel (Q1) carries the highest discharge (61 cfs). It is also interesting to note that there are significant discharges from Banani Lake (14 cfs) and Gulshan Lake (14 cfs) to Hatirjheel Lake during the dry season. In the absence of any rainfall, these discharges possibly represent the domestic wastewater that flows into these lakes from the surrounding localities.

Photograph 3.8: A pipe discharging industrial wastewater underneath the bridge on Gulshan-Tejgaon Link Road

3.6.1 Storm Runoff Estimation

For the hydraulic design of such diversion the system requires the overflow weir at lake side, a high side weir, stilling pond, and hydrodynamic separator. Following calculations has been done for determination of Capacity of Overflow weir under various overflow head conditions Size and Crest level of the overflow weirs.

As a typical test case, the SDS out fall located at the North-west Tajgaon diversion road (Q4) has been selected. After preliminary assessment of the existing conditions at the various outfalls, runoff was estimated on extreme storm events using the rational formula. Prior to this action inlet of delineated drainage area has been collected for Dhaka WASA and rainfall intensity of the area has been used from available data (FAP 8-A). Measured dry weather flow data has been collected from available report. Based on existing condition of the outfall and calculated runoff values, preliminary design (Dimensioning) of the SDS has made.

3.6.2 Estimation of Domestic Wastewater Flow

Estimation of domestic wastewater flow was made from estimates of population of the Eskaton area and assuming that wastewater from half of this population would be carried through the proposed local diversion sewer. These estimates and subsequent design calculations revealed that the estimated wastewater flow is not very significant and that sewers of less than 300 mm dia would be enough to carry this flow. However, it is anticipated that after completion of this project, significant additional commercial and residential developments will take place in these areas, resulting in significant increase of wastewater flow. The sewer sizes for the local diversion were finally decided considering these issues and taking into account the DWASA practice of not using sewers below 450 mm dia (considering maintenance issue).

3.6.3 Dry Weather Discharge data

Measured dry weather data of various outlets were measured are given in Table -3.3.

Table 3.3: Existing size and flow conditions of major outfalls (Matin and Asifur,2009)

Outfall Identification	Existing size (m)	Dry weather flow Qd (m^3/s)
Q1	6.2x6.0	1.726
Q2	2.439	0.283
Q3	1.372	0.198
Q4	**1.220**	**0.566**
Q5	1.524	0.1132
Q6	5.5x3.8	0.622
Q7	1.524	0.311
Q8	0.762	0.084
Q9	1.220	0.113

3.6.4 Hydraulic Design

For the hydraulic design of such diversion the system requires the overflow weir at lake side, a high side weir, stilling pond, and hydrodynamic separator Thus, following calculations have been done:

i) Determination of Capacity of Overflow weir under various overflow head conditions

ii) Size and Crest level of the overflow weirs.

3.6.5 Water Level

Maximum allowable level at the Hatirjeel begunbari lake has been considered as +5.5 m PWD and minimum level is 3.00 m PWD. The Riverside Pond level out side the Rampura Bridge was considered as +2.67m PWD. During high seasonal storm flow, pumps at Rampura outfall should be operated to maintain water level at the lake maximum at +5.5 m PWD. (BRTC, 2007).

Table 3.4: Capacity of over flow structures at overflow depth at H=1.75 m, Critical depth yc=1.28 m (Matin and Asifur,2009)

Outfall Identification	Existing size (m)	Maximum overflow capacity m^3/s
Q1	6.2x6.0	28.36
Q2	2.439	11.16
Q3	1.372	6.27
Q4	**1.220**	**5.58**
Q5	1.524	6.67
Q6	5.5x3.8	25.16
Q7	1.524	6.97
Q8	0.762	3.48
Q9	1.220	5.58

CHAPTER 4

EXPERIMENTAL SETUP AND TEST RUN

4.1 GENERAL

In order to assess the hydraulic behavior under extreme flow conditions, attempt has been made to study the diversion structure in a scale model. Physical modeling facility of Department of Water Resources Engineering (DWRE), BUET has used for this purpose. Froudian law has been applied (Chow, 1959, Matin, 1995) to design the scale model. Depending on the availability of the space and the discharge capacity of the physical model facility of DWRE, an undistorted model of scale 1:4 has been selected. Based on this scale, plan and elevation of the designed SDS model are shown in Figure 4.1, Figure 4.2 and Figure 4.3. Two 3D view of SDS has been shown in figure 4.4 and figure 4.5 (one is shows bypass pipe and another is shows bypass opening and overflow gate)

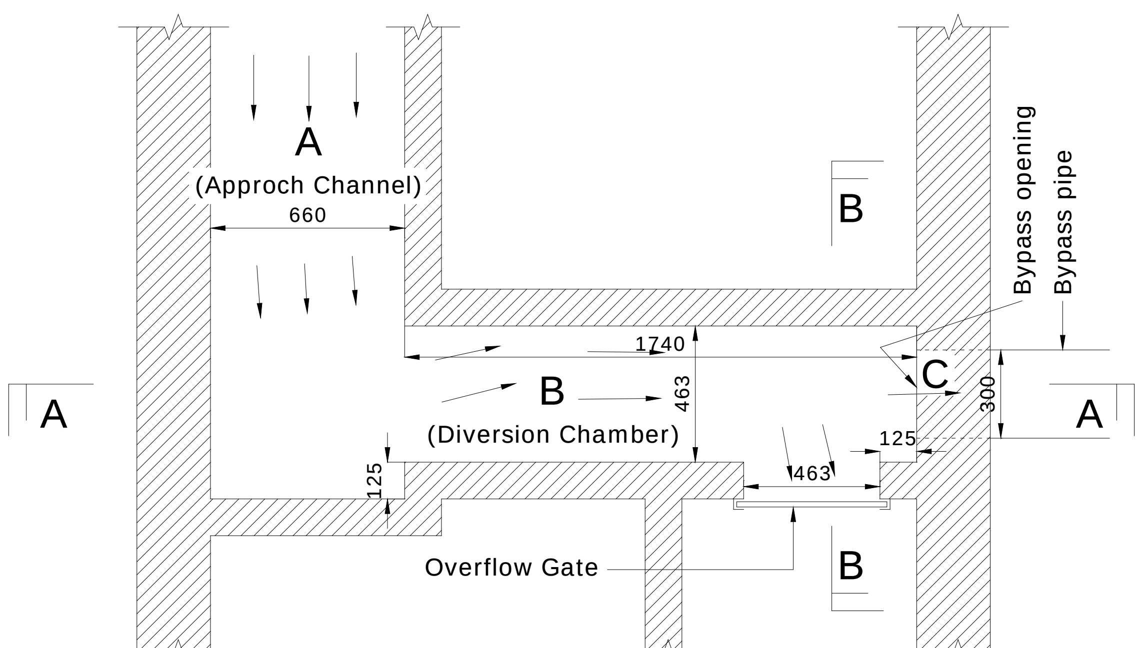


Figure: 4.1 Plan of Model of Flow Diversion Structure

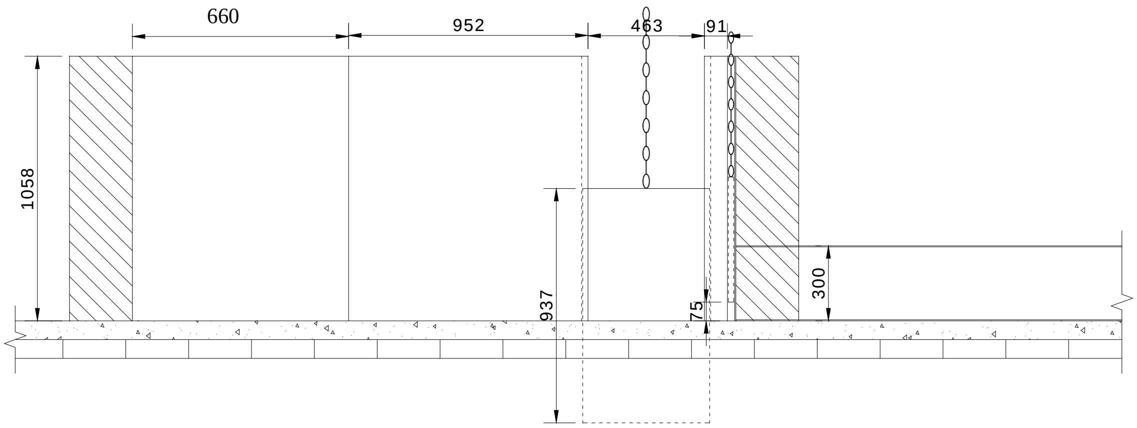


Figure: 4.2 Section A-A (Section along Underflow Pipe of Flow Diversion Structure)

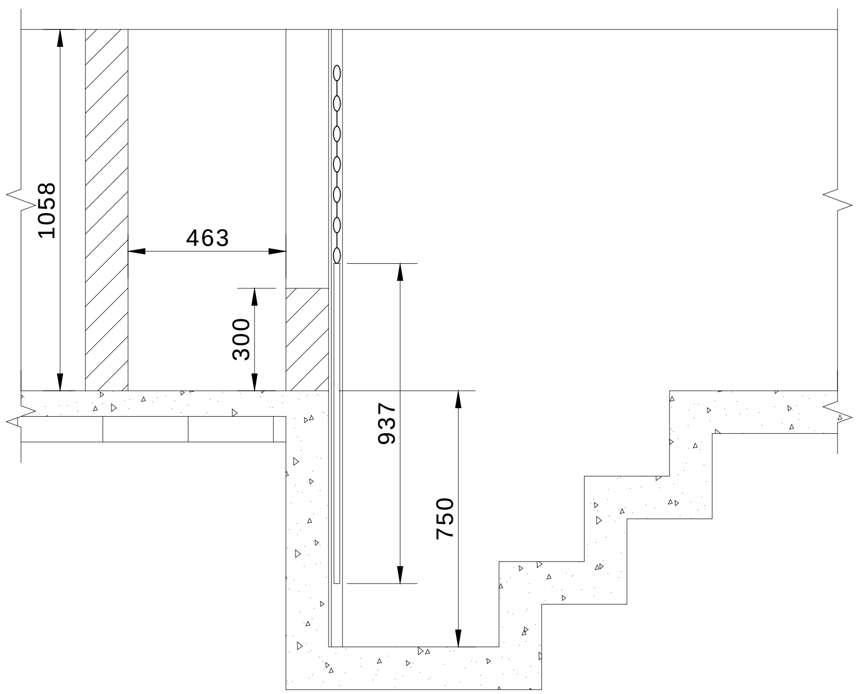


Figure:4.3: Section B-B (Section along Overflow Gate of Flow Diversion Structure)

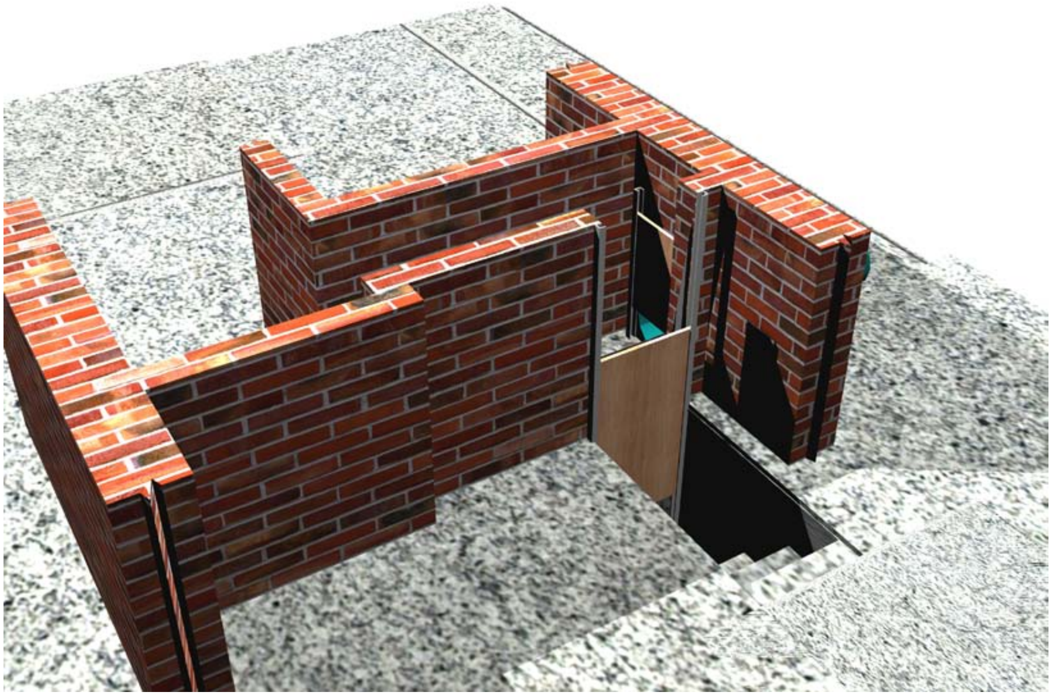


Figure 4.4 : 3D View of proposed model with over flow gate and bypass opening

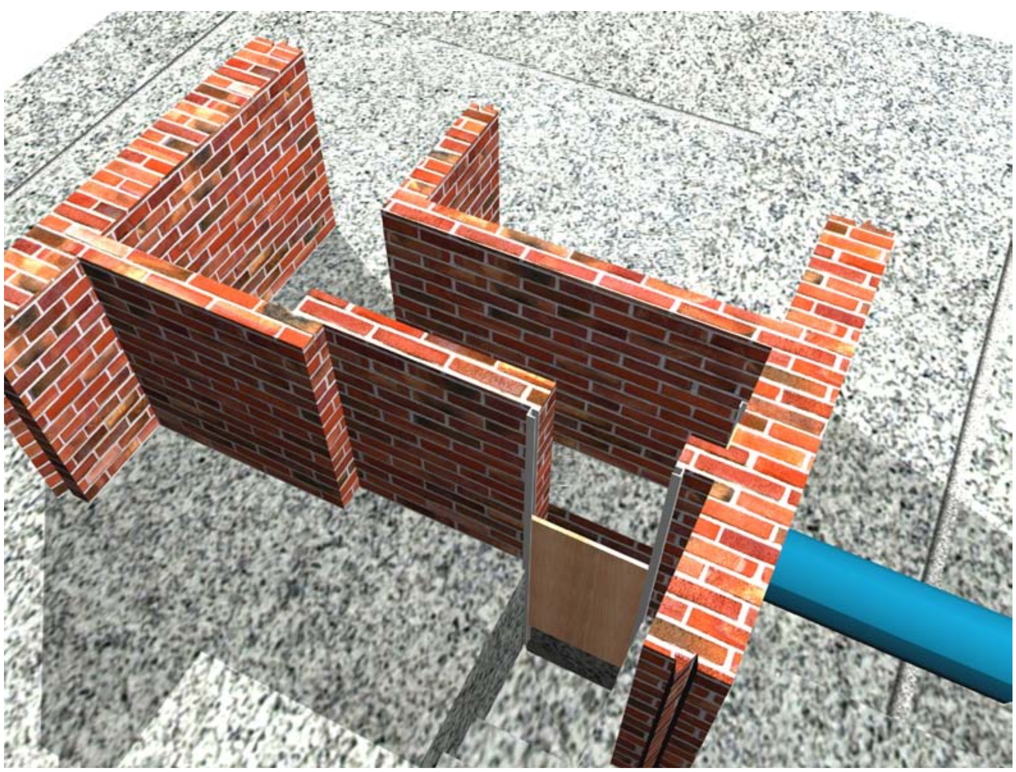


Figure 4.5 : 3D View of proposed model with bypass pipe

4.2 General Layout of Model Facility

The physical model facility is built in an open air space of about 75.5 m long and 47.25 m wide behind the Institute building of BUET, Bangladesh. Figure 4.6 shows the physical model.

Photograph 4**.1: Overview physical model facility**

The physical model facility comprises of a straight flume, storage pools for water supply, a re-circulating canal, measuring device i.e. Rehbock weirs, a measuring bridge, point gauges, tail gates, etc. The dimensions of the different components of the physical model facility are summarized in table 4.1. Moreover the components will be described in section 4.3 in brief. A drawing of the facility, including photographs, is displayed on Figure 4.6.

Table 4.1: Dimensions of the various components of Physical Model Facilities

(Zahidul,-2009)

Component	Length [m]	Width [m]	Depth [m]	Capacity [m3]
Storage pool	10.67	6.09	3.20	210.00
Upstream reservoir	10.67	3.05	1.37	44.61
Straight flume	45.00	2.45	0.46	-
Sediment trap	10.67	3.66	1.40	55.00
Downstream reservoir	10.67	1.52	0.61	9.75
Re-circulating canal	52.44	0.76	1.28	-

4.3 Components of Physical Model Facilities

4.3.1 Straight flume

The straight flume (Figure 4.6) is bounded by about 1.1 m high and 0.28 m wide smoothened concrete sidewalls. The size of the bed is 45 m long and 2.45 m wide. The elevation of the sidewalls is a little higher than the adjacent ground. So the flume remains dry in all seasons. The northern wall (flume entrance) is provided with a 1.5 m wide Rehbock weir to make it possible to determine the quantity of water that enters the model. On the southern wall there are 5 tailgates installed for controlling the water level in the model.

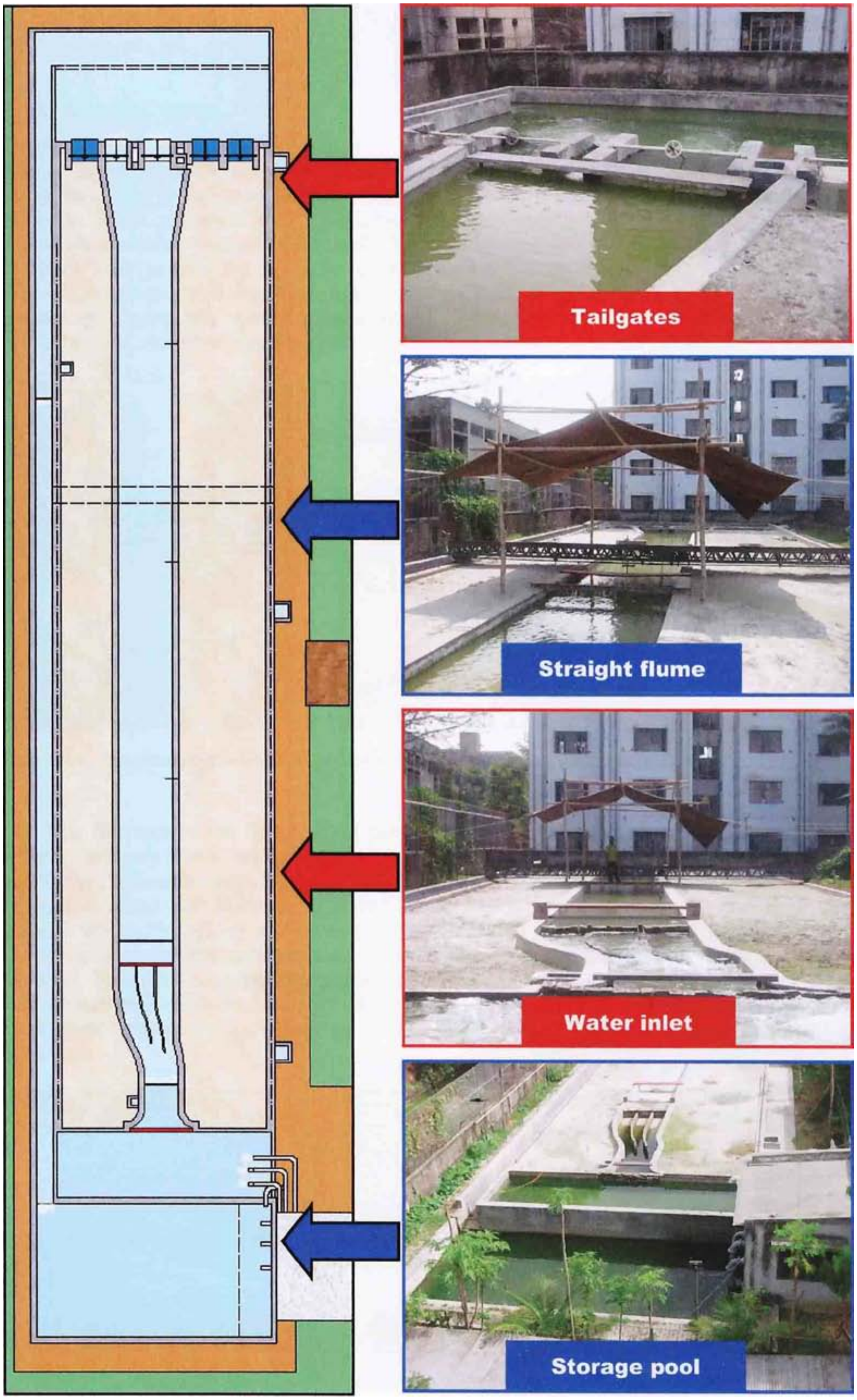


Figure 4.6: Components of physical model facility.

Photograph 4**.2: The straight flume with fixed bed in downstream (1) and upstream (r) direction**

The flow that enters the flume is led past baffle walls, guide vanes and flow dividers in order to achieve uniformity in the approach flow. The water inlet at the upstream end of the flume is provided with a wall of PVC pipes. A flow divider is installed downstream of the guide vanes and consists of a perforated wooden beam, resting on the sidewalls, through which PVC pipes are lowered at an interval of about 5 cm. Upstream of the flow divider and at the downstream end of the guide vanes, a baffle wall of PVC pipes is situated. This wall counters the turbulence in the flow past the submerged weir

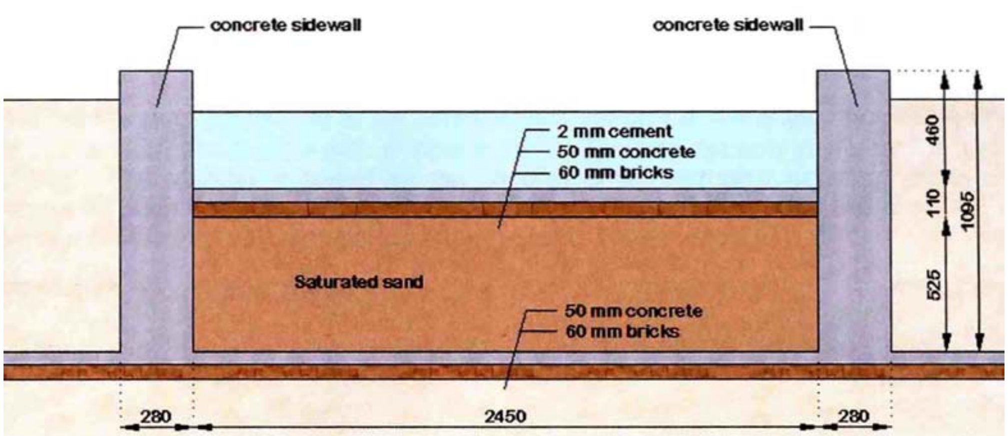


Figure 4.7: Cross-section of the flume with a fixed-bed

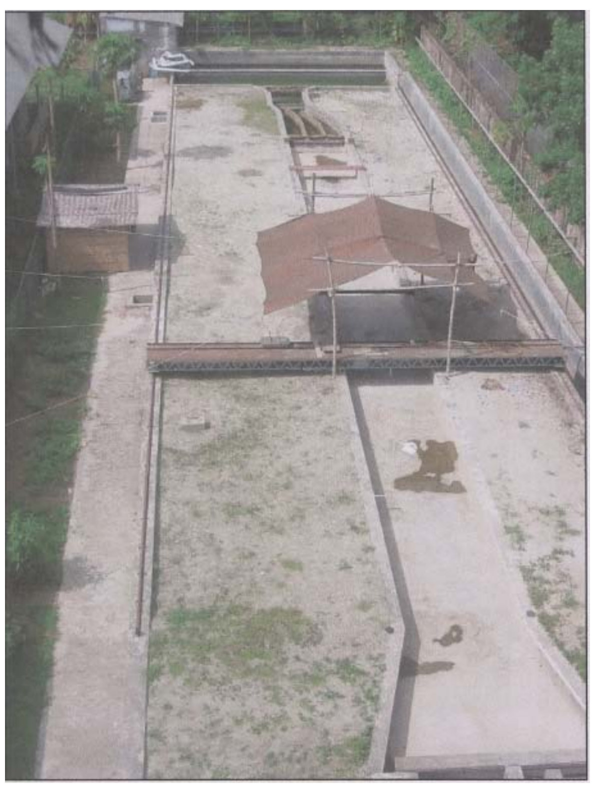
The fixed-bed is supposed to be completely horizontal, but shows some elevation irregularities on the surface. Also the bed roughness is not equal at all places. Photograph-4.3 gives an overview of the flume facility with a fixed-bed for both tested bed configuration with and without scour holes.

4.3.2 Water supply system

The water supply is of course essential for the physical model facility. The water supply in the present case is ensured by the components described in the next sections.

Storage pool

The storage pool (Photograph-4.4) is the northernmost element of the physical model facility. Its size is 10.67 m x 6.09 m with a depth is 3.2 m. The total capacity of the pool is about 210 m^3. This capacity is based on the concept of uninterrupted water supply to the system and the filling up of the other reservoirs. To the western side, an extra 1.2 m depth is provided to accommodate intake pipes of three pumps.



Photograph 4**.3: Overview flume facility with fixed-bed,**

Photograph **4.4: The storage pool with the pumps in the back**

Centrifugal pumps

The pump house (Photograph-4.5) is located on the western side of the storage pool. There are three centrifugal electric water pumps (Photograph-4.6) installed in this house, which are capable of running continuously to ensure water supply to the flume. The total capacity of the pumps is 220 I/s. The capacity of the northern and middle pump is 80 I/s. The southern pump's capacity is 60 I/s. One larger pump has a loss line to control supply of water to the upstream reservoir and returning excess water to the storage pool.

Photograph 4**.5: Pumps discharging water into the upstream reservoir, with the pump house on the left photograph**

Photograph 4.6**: Three centrifugal pumps seen from inside the pump house (I) and the pump outlets (r)**

Upstream reservoir

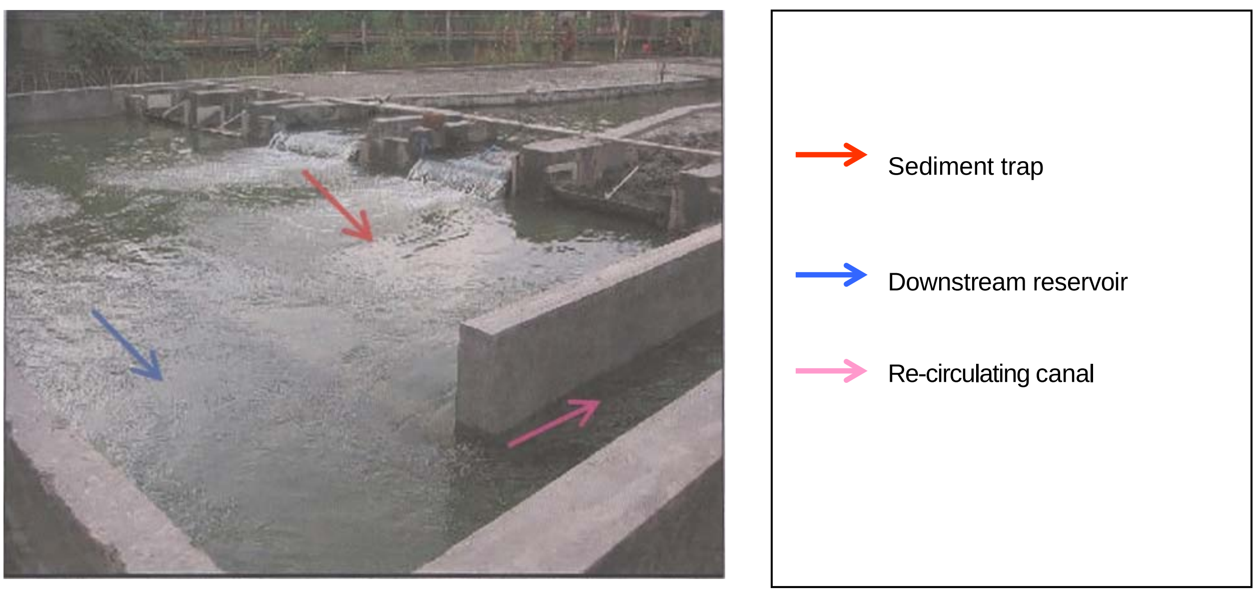
The upstream reservoir (Photograph-4.7) is located between the storage pool and the flume. The purpose of this reservoir is to store water, bring it to a steady state before delivery to the flume. The size of this reservoir is 10.67m x 3.05 m and the depth equals 1.37 m. The capacity is 44.61 m^3. On the southern wall there is a Rehbock weir installed which can be used for measuring the quantity of water that is entering the flume.

Photograph 4.7**: Upstream reservoir adjacent to the storage pool (left picture in front) and a close-up on the reservoir (r)**

Sediment trap and downstream reservoir

The sediment trap tank (Photograph-4.8) is located between the flume bed and the downstream reservoir and was used for experimentation in a mobile bed. The sediment trap tank is designed to trap sediments before reaching the downstream reservoir (Photograph 4.8) and is based on factors i.e. sediment size, weight, shape, velocity and water depth. Sediment will fall to the base due to gravity. Its size is 10.67 m x 3.66 m.

The downstream reservoir is located between the sediment trap on the north and the re-circulating canal on the east. It is the southernmost element of the physical model facility. Its size is 10.67 mxl.52 m and the depth is 0.61 m. The capacity of this reservoir is 9.75 m^3/s. The purpose of this reservoir is to store water before delivery to the re-circulating canal.



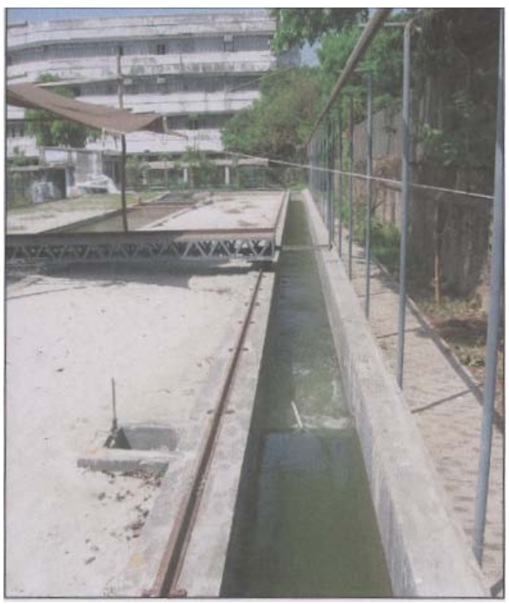
Photograph 4.8**: Sediment trap, downstream reservoir and re-circulating canal**

Re-circulating canal

The re-circulating canal (Photograph-4.9) is located on the eastern side of facility. It connects the downstream reservoir and the storage pool and allows the water to fall back for re-supply. It is 52.44 m long, 0.76 m wide and 1.28 m deep. It is capable of draining all the water supplied to the system. There are three drainage plugs connecting the sand-bed and the re-circulation canal. There is also a 0.754 m long sharp-crested Rehbock weir installed across the re-circulation canal for measuring the quantity of water flowing through the flume.

4.4 Measuring devices

The measuring devices are also an essential part of the physical model facility. Through these devices the operation of the facility is controlled. The different measuring devices will be treated in this section.



Photograph 4.9**: The re-circulating canal (with weir) seen in downstream (I) and upstream (r) direction**

4.4. Rehbock weirs

A weir with a sharp upstream corner or edge such that the water springs clear of the crest is a sharp-crested weir.The rectangular weir is the most commonly used thin plate weir. Weirs are typically installed in open channels such as streams to determine discharge (flow rate). The basic principle is that discharge is directly related to the water head (h)

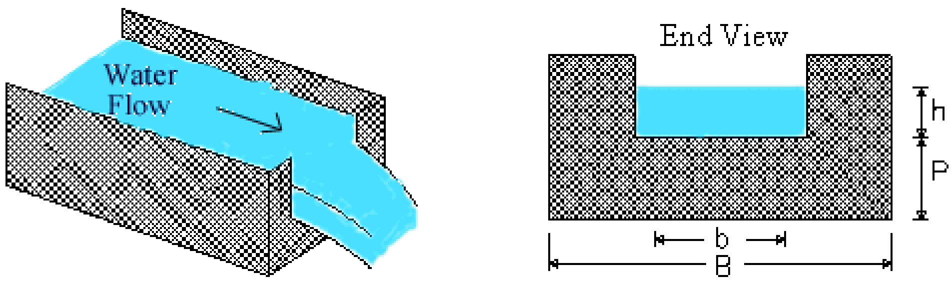


Figure- 4.8: Definition sketch of weir.

in the Figure above; h is known as the "head." Rectangular weirs can be "suppressed," "partially contracted," or "fully contracted." Suppressed means there are no contractions. A suppressed weir's notch width (b) is equal to the channel width (B); thus, there really isn't a notch - the weir is flat all the way along the top. For a weir to be fully contracted, (B-b) must be greater than $4h_{max}$, where h_{max} is the maximum expected head on the weir (USBR, 1997). A partially contracted weir has B-b between 0 and $4h_{max}$. Weir contractions cause the water flow lines to converge through the notch.

USBR (1997) provides equations for a "standard" fully contracted rectangular weir and a "standard" suppressed weir. The U.S. Bureau of Reclamation has conducted many weir tests over several decades using weirs with particular dimensions - usually b's in 1 ft. increments up to about 10 ft. Therefore, any weir outside their tested dimensions is non-standard, and their equations should not be used. To provide a single reliable, accurate method to model all rectangular weirs (suppressed, partially contracted, and fully contracted), the Kindsvater-Carter equation (Kindsvater and Carter, 1959) was developed. It is considerably more complex than the USBR standard weir equations. However, USBR (1997) states that the Kindsvater-Carter method is at least as accurate, if not more, than the standard weir equations for suppressed and fully contracted weirs. And further, the Kindsvater-Carter equation reliably models partially contracted weirs. ISO (1980), ASTM (1993), and USBR (1993) all recommend using the Kindsvater-Carter method for all rectangular weirs.

All other weirs are classed as weirs not sharp crested. Sharp-crested weirs are classified according to the shape of the weir opening, such as rectangular weirs, triangular or V-notch weirs, trapezoidal weirs, and parabolic weirs. Weirs not sharp crested are classified according to the shape of their cross section, such as broad-crested weirs, triangular weirs, and trapezoidal weirs.

The channel leading up to a weir is the channel of approach. The mean velocity in this channel is the velocity of approach. The depth of water producing the discharge is the head.

Sharp-crested weirs are useful only as a means of measuring flowing water. In contrast, weirs not sharp crested are commonly incorporated into hydraulic structures as control or regulation devices, with measurement of flow as their secondary function.

The Kindsvater-Carter rectangular weir equation (ISO, 1980):

$$Q = C_e \frac{2}{3}\sqrt{2g}\left(b + k_b\right)\left(h + k_h\right)^{\frac{3}{2}} \tag{4.1}$$

Where,

Q= Discharge [L^3/T], C_e= Discharge coefficient

g = Acceleration of Gravity [L/T^2], b= Notch Width [L]

h=Head [L]

K_b and K_h account for effects of viscosity and surface tension [L]

The sum b+K_b is called "effective width" and the sum h+K_h is called "effective head." The value for g is 9.8066 m/s^2 and K_h=0.001 m. C_e is a function of b/B and h/P, and K_b is a function of b/B. Our "Solve for Flowrate" calculation is analytic, but our "Solve for Head" and "Solve for Notch Width" calculations require numerical solutions since C_e and K_b cannot be computed directly, as they are functions of h and/or b.

ISO (1980) provides a graph and equations for C_e vs. h/P for b/B=0, 0.2, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, and 1.0. To account for other b/B values, LMNO Engineering developed an equation for all b/B values. The LMNO Engineering curves are plotted below with the ISO curves, so you can see our goodness of fit. Likewise, ISO (1980) provides values of K_b for b/B=0, 0.2, 0.4, 0.6, 0.8, and 1.0, and LMNO Engineering fit an equation through the data to facilitate solving numerically for b. The LMNO Engineering fit is shown below with the ISO data. Our numerical solutions utilize a cubic solver routine. The computations are performed in double precision (the calculations on all of our web pages use double precision). (www.usbr.gov)

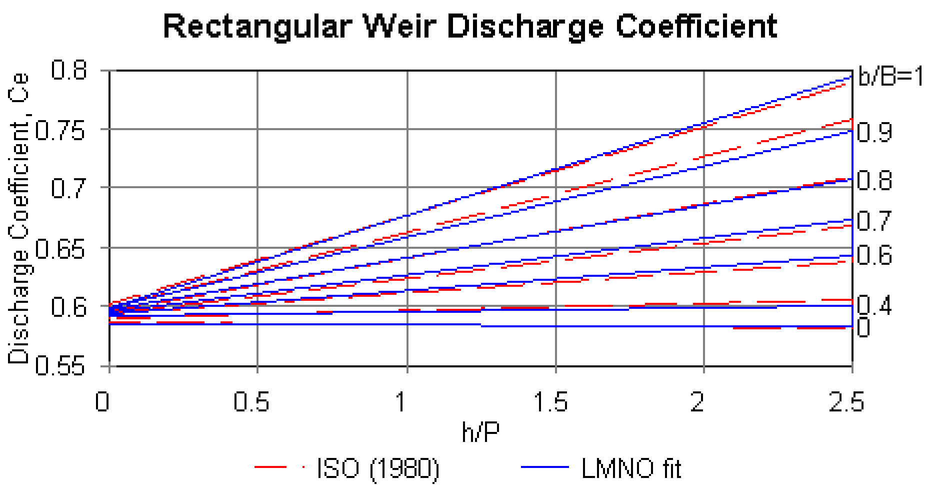


Figure- 4.9: Rectangular weir discharge coefficient.

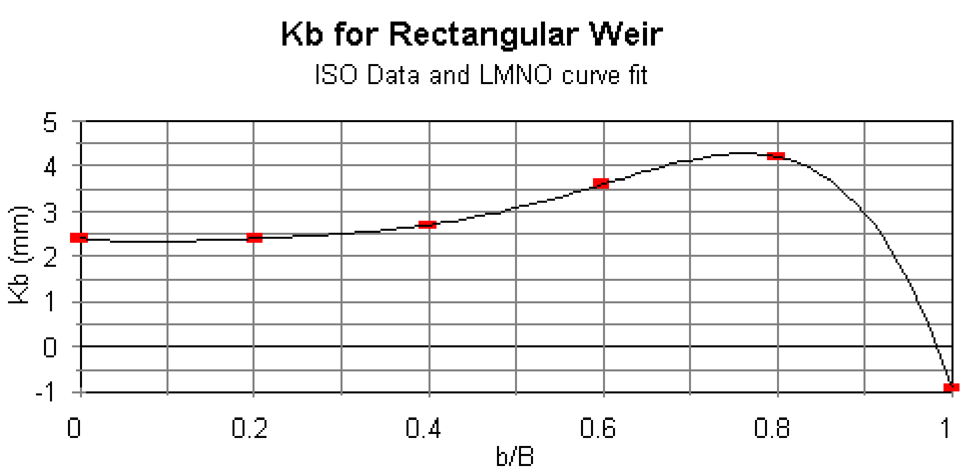
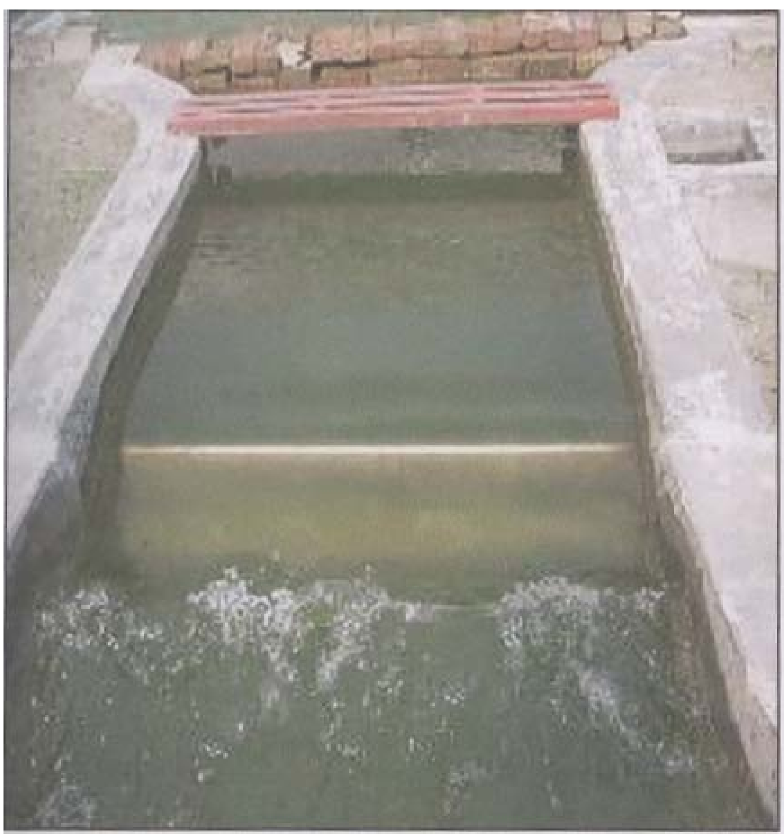


Figure- 4.10: K_b for rectangular weir.

There are two sharp-crested Rehbock weirs installed in this facility. The first one is located between the upstream reservoir and the flume. It helps to measure the quantity of water flowing to the model, by calculating the head from the nearby observation well. The other weir is installed in the re-circulating canal. This one also helps to measure the quantity of water that is flowing trough the flume. It can be calculated from reading the point gauge in the adjacent stilling basin. The difference between the two weirs gives the water loss in the system. During experimentation in 2011, only use has been made of the weir in the start point of the flume.



Photograph 4.10**: Rehbock weirs at the start point of the flume**

4.5 Experimental setup

The experiments has been carried out in a laboratory flume having a 40.4 m overall length, 1m depth and 3m inside width. The flume has an adjustable tail water gate located at downstream. Scale model of proposed SDS has been constructed as designed at the middle reach of the flume. The diversion chamber has constructed using the brick masonry with cement plastering. Over flow and under flow gate has been fabricated with wood and attached with the diversion outlets with steel angle.

4.5.1 Model Preparations

Due to space availability in the physical model facility model scale ratio have been selected 1:4 ($L_r = 4$). For this scale ratio, $V_r = \sqrt{L_r}$; i.e. $\frac{V_p}{V_m} = 2$ and $Q_r = L_r^{5/2}$; i.e.

$$\frac{Q_p}{Q_m} = 4^{5/2} = 32$$

Before the start of an experiment, the model bed was superficially cleaned from any dirt. The flume could be checked for damages also, which had to be repaired. Also other components of the flume were frequently cleaned, like the dirt separator at the end of the re-circulating canal.

4.5.2 Adjusting the Boundary Conditions

It was important to keep the boundary conditions as constant as possible to execute the experiments properly. Unfortunately this was not always feasible. The power supply was sometimes erratic and variable. Fluctuations in electric potential influenced the pumps and hereby the discharge many times. The fact that the boundary conditions were difficult to keep stable. It was important to measure and control it often.

In the model there were two main boundary conditions to control; water level and overflow gate height . These two conditions are interdependent. Because water levels in the flume depend on overflow gate height. Deviations in overflow gate height simultaneously cause deviations in water level. Hence it was important to first set the correct discharge before adjusting the water level. The flow chart of Figure 4.11 was used during experimentation.

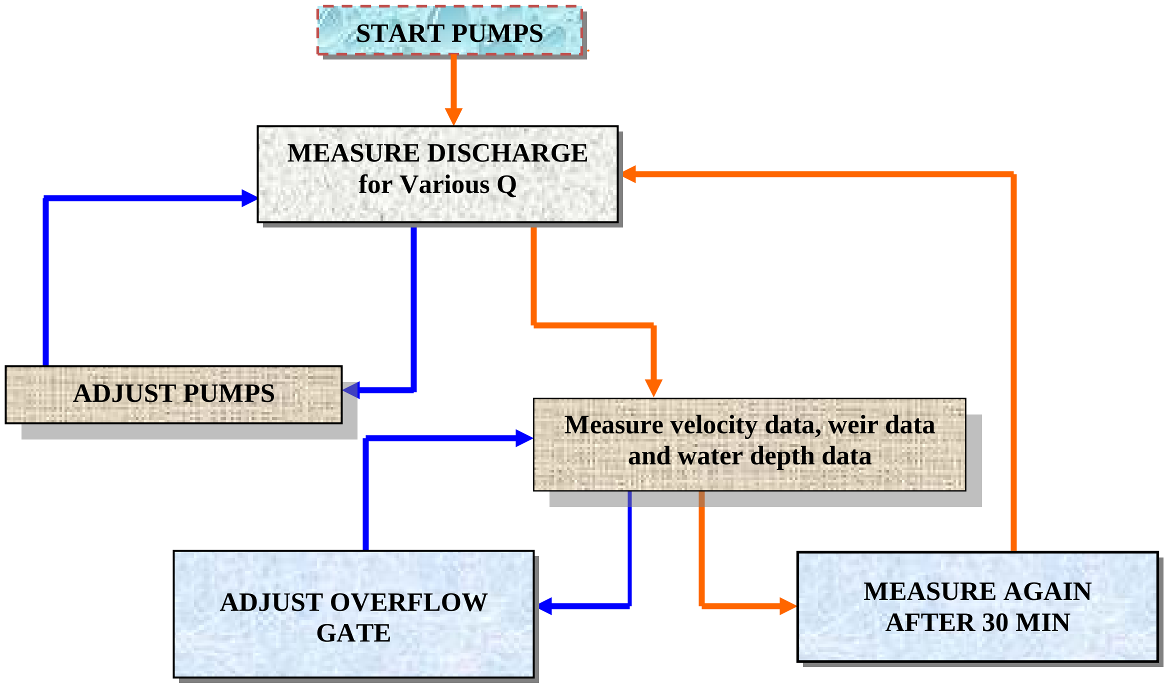
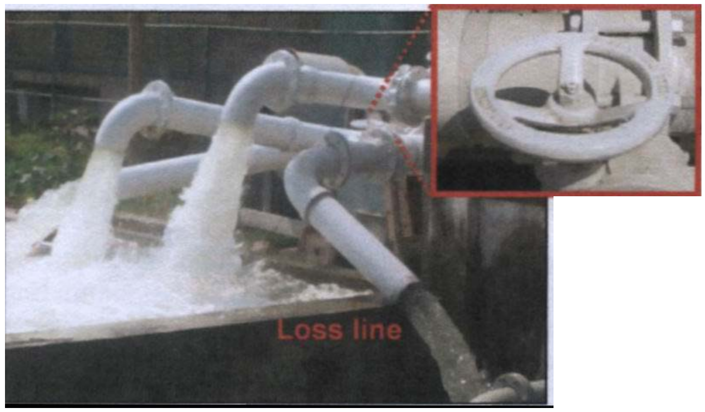


Figure 4.11 : Flow chart of reaching boundary conditions

The discharge could be measured by use of the Rehbock weir in the re-circulating canal and by reading the point gauge in the stilling basin. If necessary the discharge could be regulated by means of valves at the pump-pipe outlets. The second pump had a loss line also, consisting of a pipeline that directly retuned excess water back into the storage pool. The readings from the point gauge were taken every 30 minutes to check the correctness of the magnitude of the discharge.



Photograph 4.11 : The loss line (l) with valve (in frame) and flow depth control by means of the tail gates(r)

4.5.3 Water Level Measurements

Water level was measured by a point gage along the center line of the diversion structure at different point (Shown in Photograph 4.25 to Photograph 4.28). Water level was also measured at total outlet channel and overflow weir channel for discharge calculation. The difference in water level readings provided information about the slope of the water surface. The different water level measurements were taken every different arrangement of setup, at the same time of a discharge reading.

4.5.4 Velocity Measurements

The flow velocity measurements were carried out with current meter (Shown in Photograph 4.22, Model no-Z-215 Brand-HEEL, Made in West Germany), consisting of propeller unit and a meter unit. The required time for every 30 rotation of propeller was taken. Then rotation per second (n value) was evaluated. By this n value we measured velocity from pre-calibrated equation.

The sequence of model setup and test run has shown in photograph 4.12 to 4.29.

Photograph 4.12 : Layout of experimental setup

Photograph 4.13: Construction of diversion chamber

Photograph 4.14: 12”dia diversion pipe

Photograph 4.15: Sliding channel preparation

Photograph 4.16: Removing bed sediment from channel

Photograph 4.17: Measurement of the model dimension.

Photograph 4.18: Measurement of the overflow gate height with a level pipe.

Photograph 4.19: Water recycling pump.

Photograph 4.20: First run of the model

Photograph 4.21: Measurement of u/s weir discharge.

Photograph 4.22: Measurement of velocity at diversion structure with current meter.

Photograph 4.23: Model is running low gate height and high discharge. (discharge= 0.0928 m^3/s and gate height=201mm)

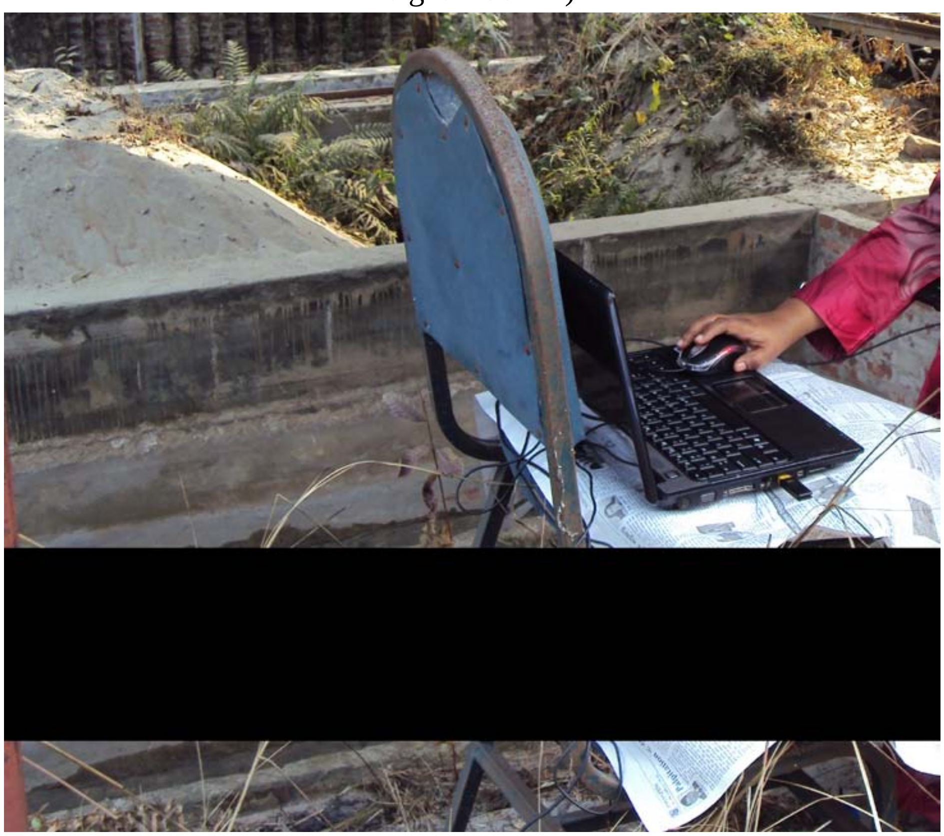
Photograph 4.24: Model is running average gate height and low discharge (discharge= 0.0443 m^3/s and gate height=484mm)

Photograph 4.25: Model is running highest gate height and high discharge. (discharge= 0.109 m^3/s and gate height=692mm)

Photograph 4.26: Model is running on it’s average capacity (discharge= 0.0901 m^3/s and gate height=412mm)

Photograph 4.27: Model is runs highest gate height and high discharge. (discharge= 0.0927 m^3/s and gate height=882mm)

Photograph 4.28: Eight run of the model (discharge= 0.0574 m^3/s and gate height=736mm)



Photograph 4.29: Model data collection with a laptop computer.

CHAPTER 5

MODEL RESULTS AND DISCUSSIONS

5.1 Model Test scenarios

Initially several trial runs had been conducted for various discharges (low, medium and high flow cases) for a certain gate openings. Also for constant discharge, flow conditions were observed for various gate openings. It was observed that the model flow conditions and outlets flow are sensitive to all the options, particularly for higher flow and raised gate at overflow weir.

A total of 13 test runs have been conducted to observe the flow behavior of the model. These are shown in Table 5.1

Table 5.1 : Model Test scenarios

Run No.	**Overflow gate height**	**Upstream weir head**	**Upstream weir discharge**	**Overflow weir head**	**Overflow weir discharge**
	(mm)	(mm)	m^3/s	(mm)	m^3/s
1	201	63.4	**0.043977**	97	**0.028574**
2	201	104.4	**0.092775**	160	**0.060865**
3	201	80.4	**0.062689**	122	**0.040363**
4	412	102.4	**0.090114**	145	**0.052418**
5	412	72.4	**0.053599**	103	**0.031273**
6	412	84.4	**0.067416**	121	**0.039865**
7	612	81	**0.063390**	105	**0.032191**
8	736.8	75.8	**0.057402**	90.2	**0.025621**
9	882	104.4	**0.092775**	118	**0.038382**
10	692	116.4	**0.109301**	146	**0.052967**
11	484	104.4	**0.092775**	143	**0.051325**
12	484	91.4	**0.075970**	125	**0.041872**
13	484	63.7	**0.044287**	88	**0.024689**

5.2 Model Observations

For all the test runs following observations have been made:

i) Flow velocity and water level at the Storm Diversion Structure (SDS), diversion pipe and overflow outlet

ii) Water surface condition within the diversion chamber.

iii) Flow behavior under various overflow gate operation and for different discharge condition.

All the test runs measurements are given in Table 5.2. The results are also presented in graphical forms

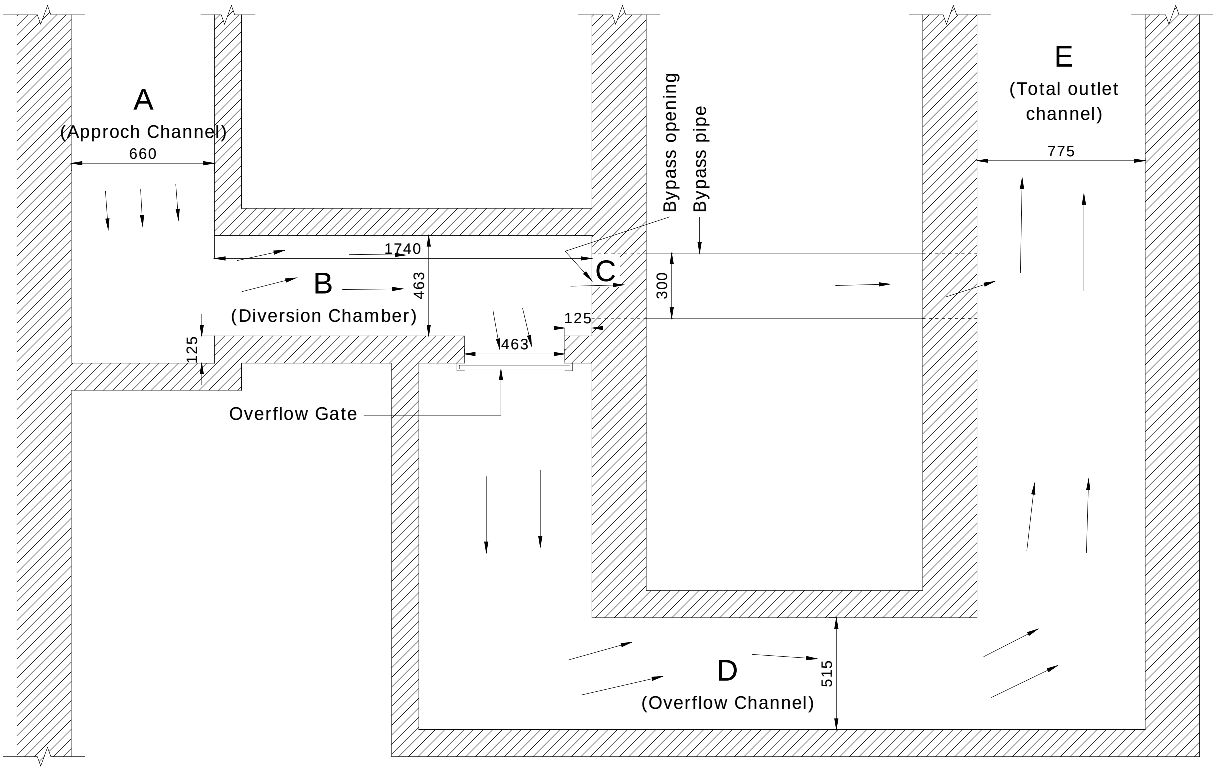


Fig 5.1: Locations of measurement points.

5.3 SDS Model Results and Discussions

Design of the scale model has given in the chapter 4. Flow velocities have been measured in longitudinal direction for some arrangement of gate height and discharge mentioned in chapter 4 (Experimental Setup and Test Runs). These relationships of discharge and velocities with overflow gate height have been plotted. For this experiment 13 test runs were made. To determine the discharge at every arrangement water level at SDS approach channel, Diversion Structure, overflow channel, Total outlet channel and bypass pipe was measured.

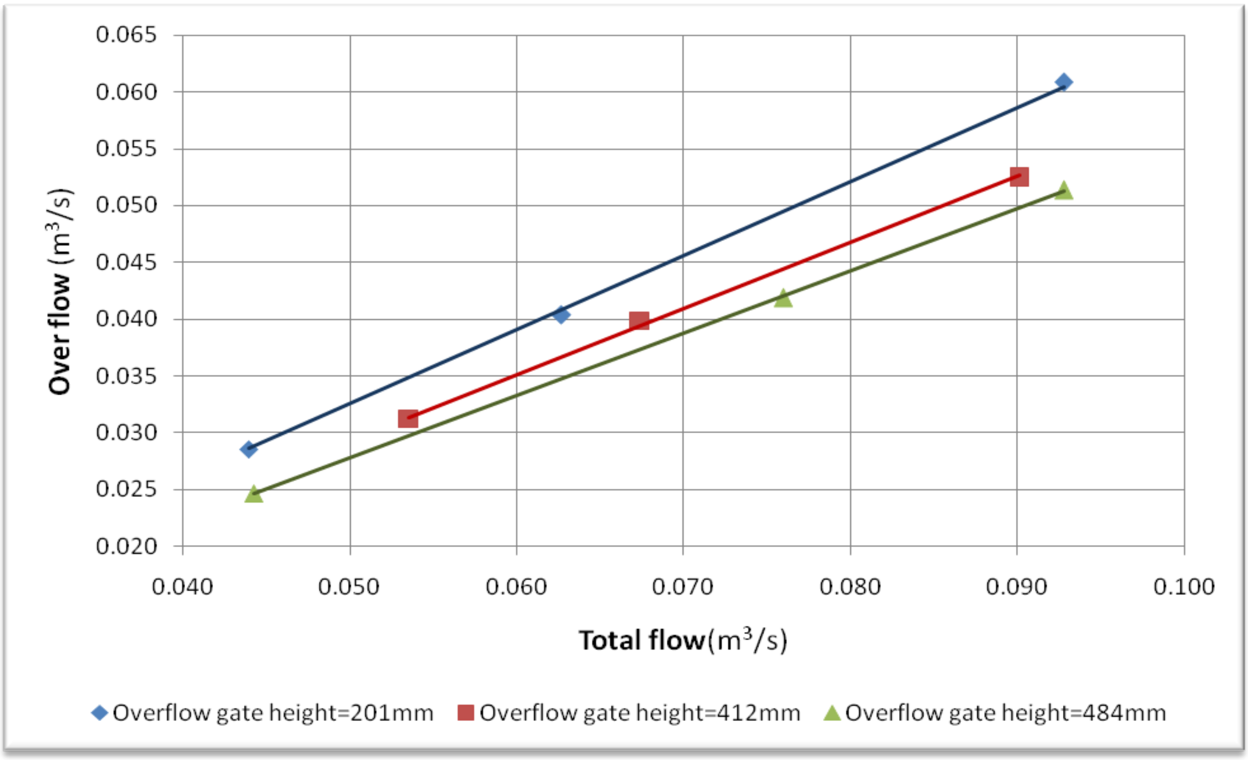


Fig 5.2: Relation between total flow versus overflow (overflow gate height=201mm, 412mm and 484 mm)

5.3.1 Relation between overflow and total inflow

For overflow gate height fixed at 201 mm, 412 mm and 484 mm (Figure-5.2) it was found that overflow rate is proportional to total inflow of Diversion structure. The proportionality is decreasing with increase of gate height, i.e. gate height is inversely proportional to ratio of overflow and total inflow of Diversion structure.

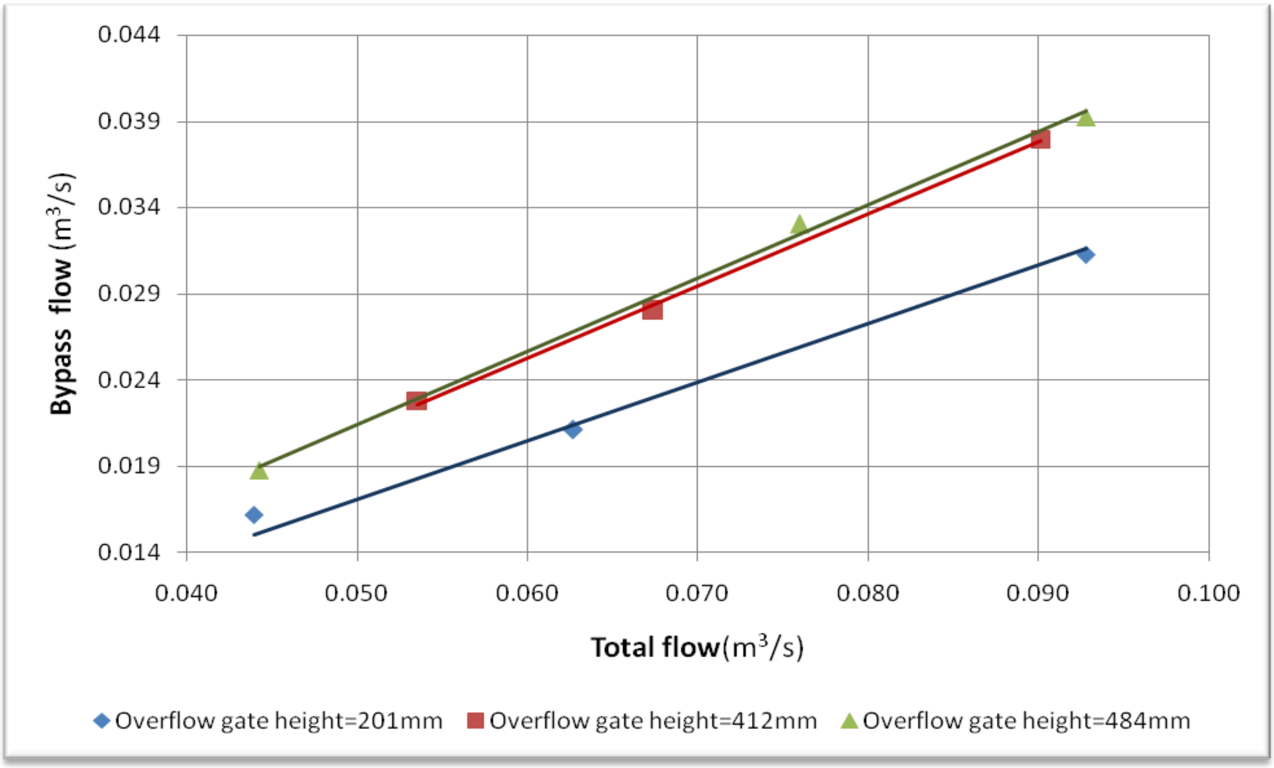


Fig 5.3: Relation between total flow versus bypass flow (overflow gate height=201mm, 412mm and 484 mm)

5.3.2 Relation between bypass flow and total inflow

An observation has also made on bypass flow and total inflow in the SDS. For overflow gate height fixed at 201 mm, 412 mm and 484 mm (Figure-5.3) it was observed that bypass flow rate is proportional to total inflow of Diversion structure.

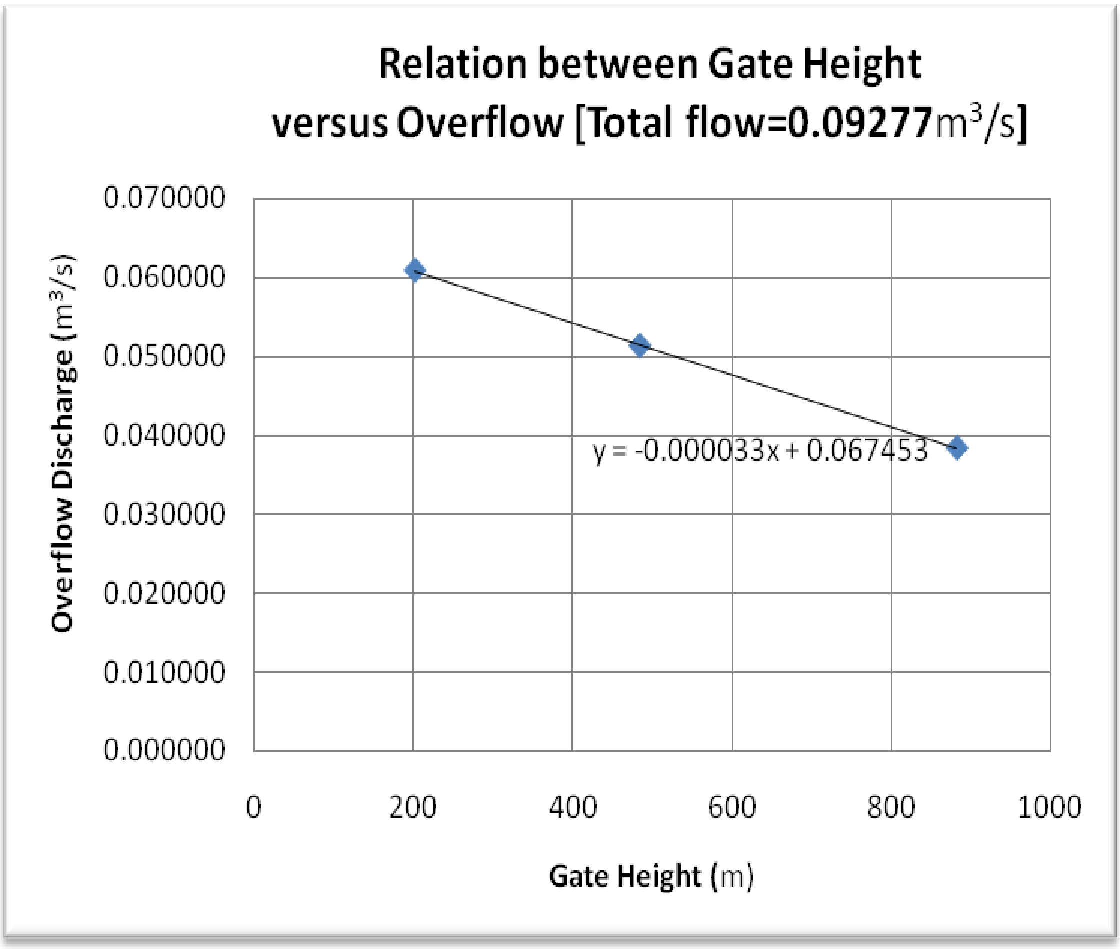


Fig 5.4: Relation between overflow gate heights versus overflow (Total flow=0.09277 m^3/s)

5.3.3 Relation between overflow and gate height

For a constant total inflow of Diversion structure (Fig 5.4 for constant Q=0.09277m^3/s, See run no. 2, run no. 9 and run no. 11), It was found that overflow rate is inversely proportional to gate height and correlation coefficient is 0.999. This correlation is very good correlation.

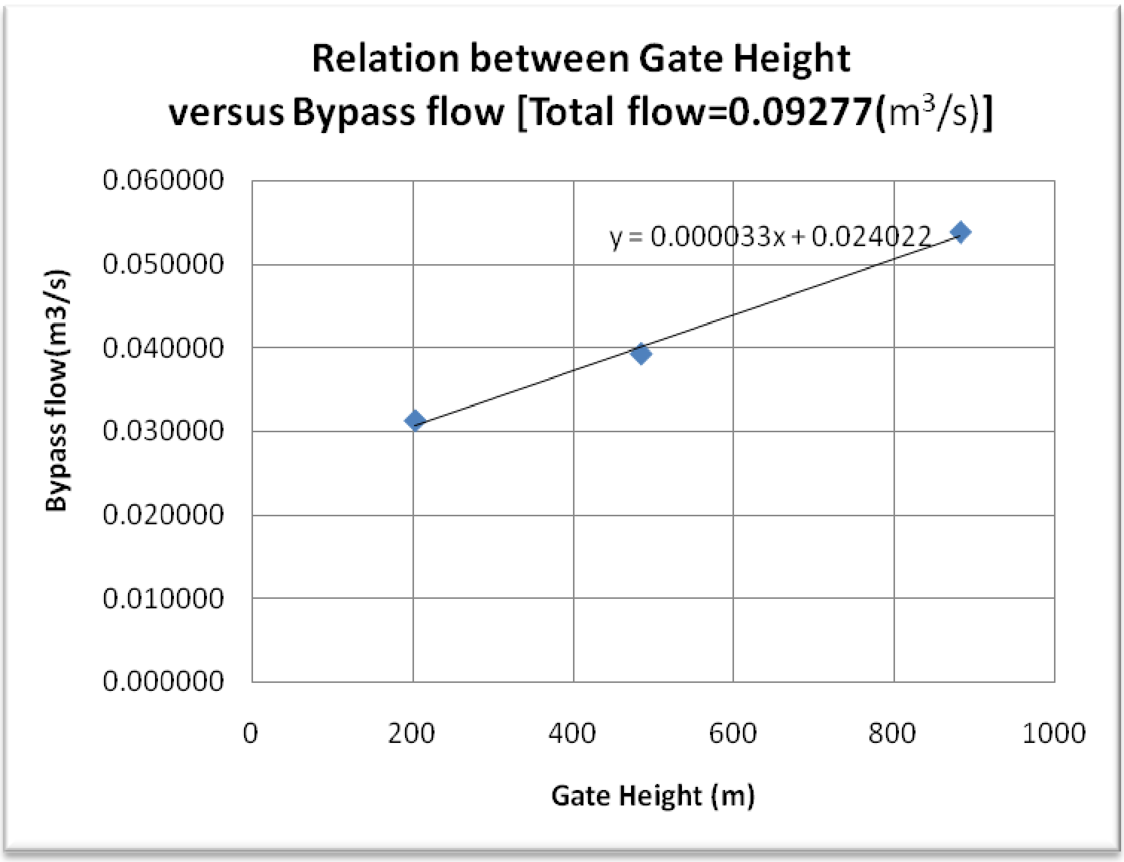


Fig 5.5: Relation between overflow gate heights versus bypass flow (Total flow=0.09277 m^3/s)

5.3.4 Relation between bypass flow and gate height

An observation has also made on bypass flow and gate height. For a constant total inflow of Diversion structure (Fig 5.5 for constant Q=0.09277m^3/s, See run no. 2, run no. 9 and run no. 11), It was observed that bypass flow rate is proportional to gate height. Correlation among them has evaluated and found correlation coefficient is 0.995. This is also good correlation.

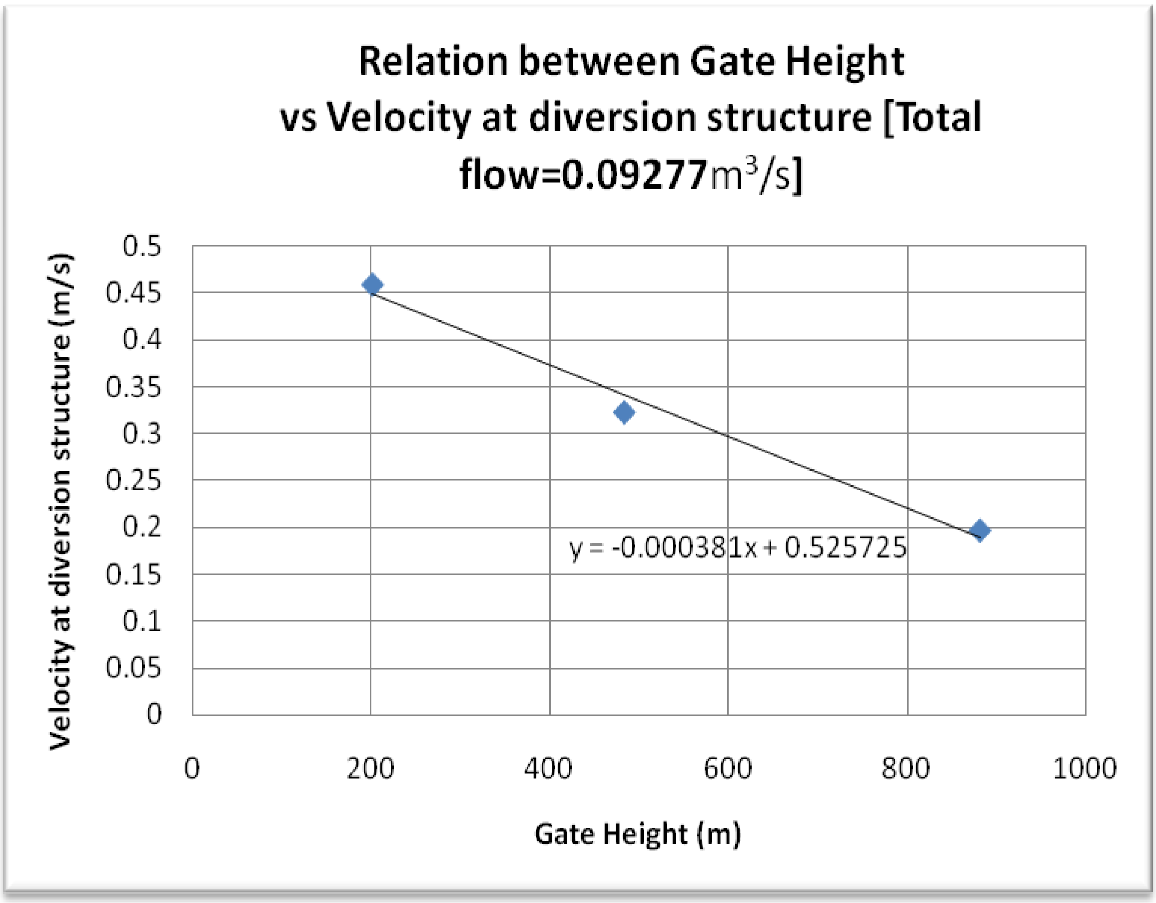


Fig 5.6: Relation between overflow gate height versus velocity at diversion structure (Total flow=0.09277 m^3/s)

5.3.5 Relation between velocity in diversion structure and gate height

The velocity in the SDS has also measured and found for a constant total inflow of Diversion structure (Fig 5.6 for constant Q=0.09277m^3/s, See run no. 2, run no. 9 and run no. 11) velocity in diversion structure is inversely proportional to gate height and the correlation coefficient is 0.985 which is also good correlation.

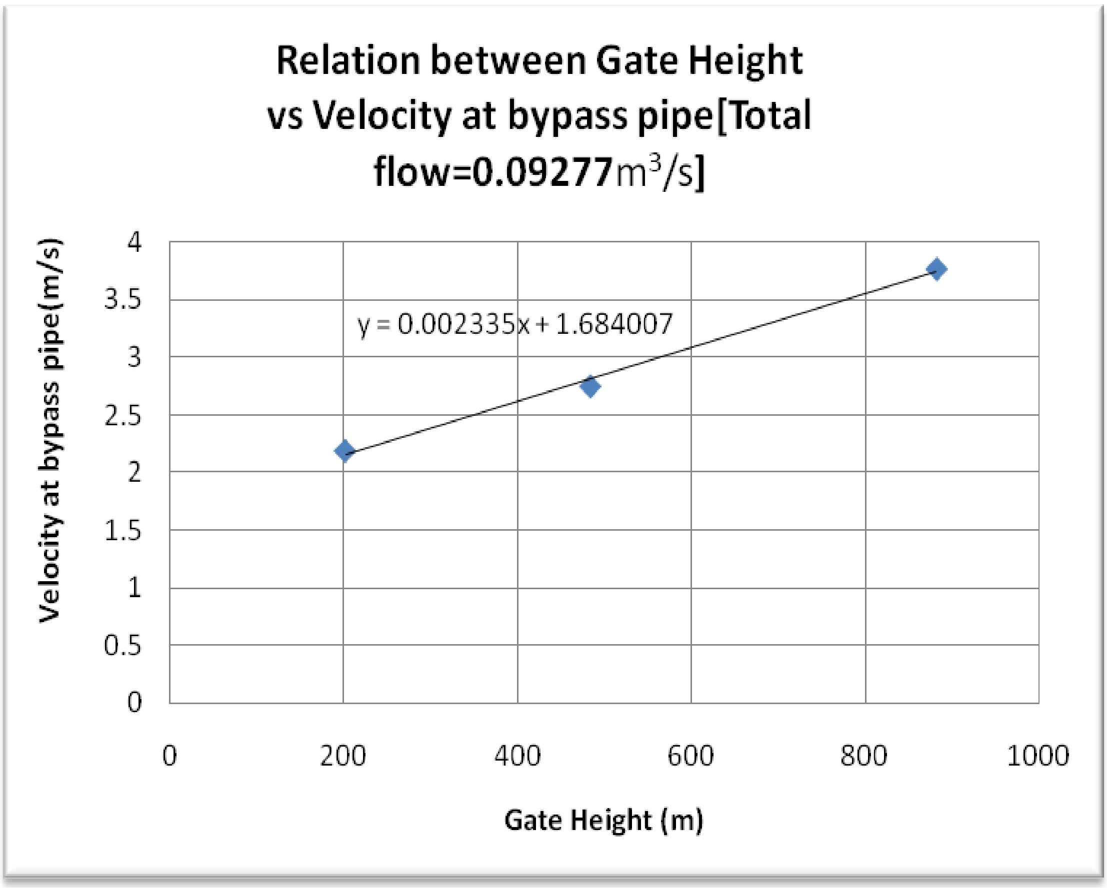


Fig 5.7: Relation between overflow gate height versus velocity at bypass pipe (Total flow=0.09277 m^3/s)

5.3.6 Relation between velocity in bypass pipe and gate height

When the inflow of SDS is constant (Fig 5.7 for constant Q=0.09277m^3/s, See run no. 2, run no. 9 and run no. 11), It was seen that velocity in bypass pipe is proportional to gate height and the correlation coefficient is 0.995 which is also good correlation.

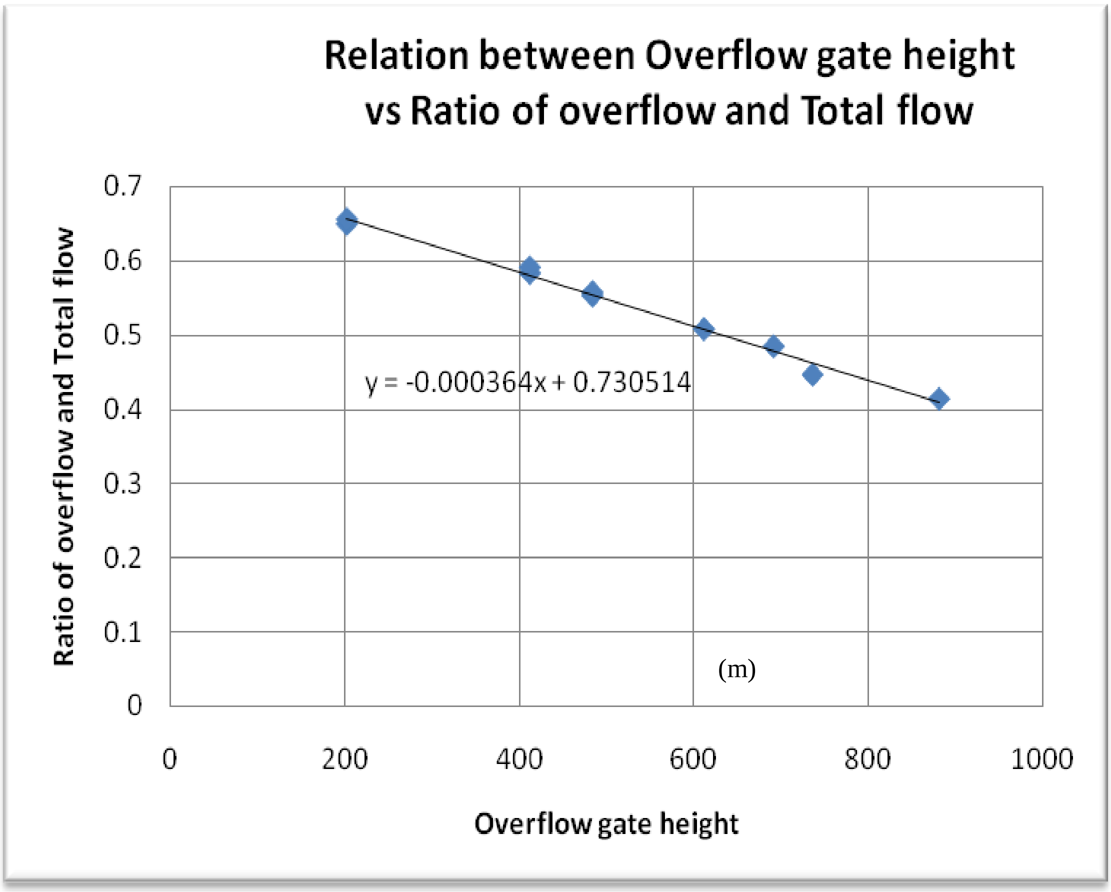


Fig 5.8: Relation between overflow gate height versus ratio of overflow and total flow

5.3.7 Ratio of bypassflow and total inflow and gate height

As discussed in article 5.3.1 and Fig. 5.8 it was observed that gate height is inversely proportional to ratio of overflow and total inflow of Diversion structure and the correlation coefficient is 0.991 which is also a good correlation. From this correlation we can evaluate required gate height for any certain overflow (water to be disposed in cannel) and total inflow of Diversion structure (combined sewer flow).

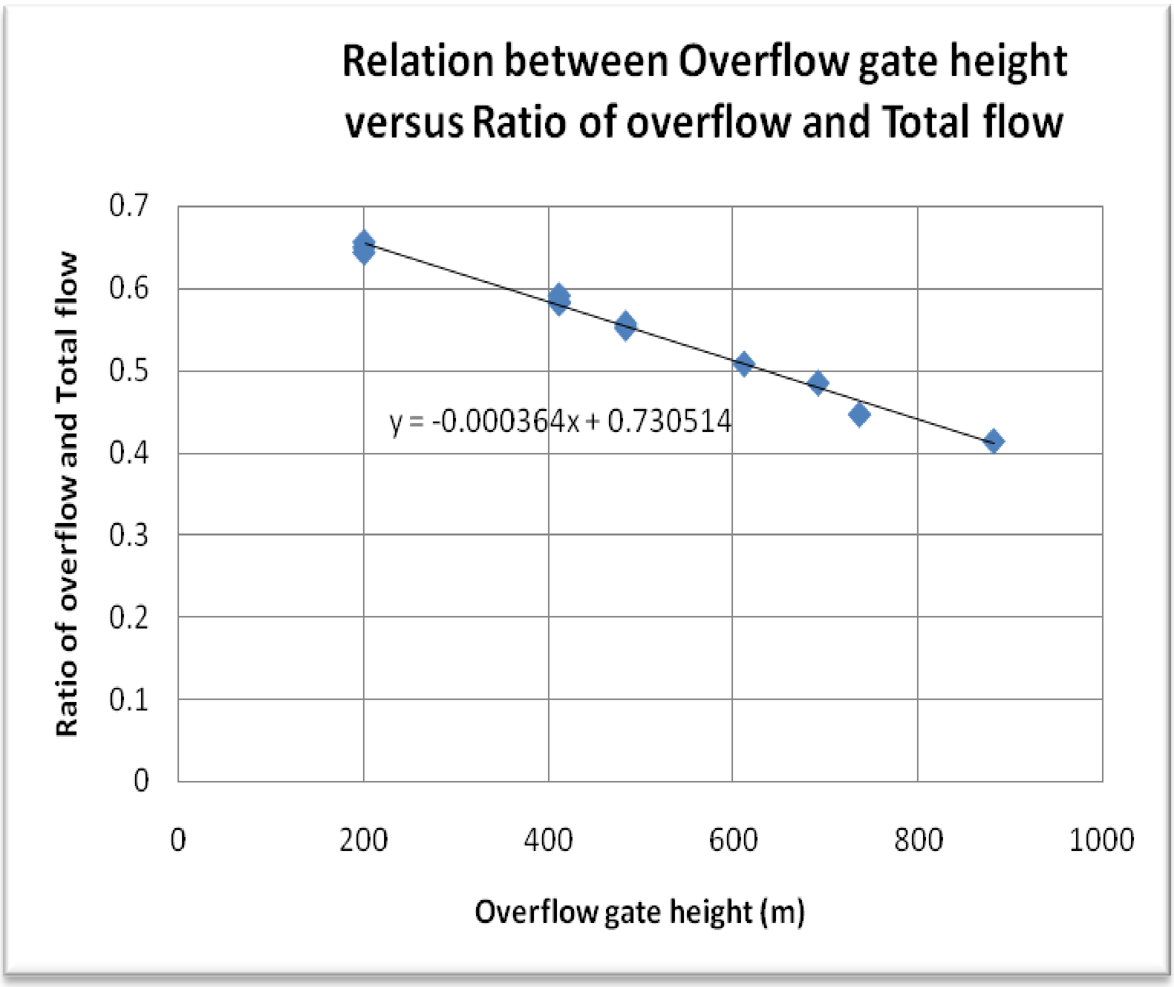


Fig 5.9: Relation between overflow gate height versus ratio of bypass pipe discharge

5.3.8 Ratio of overflow and total inflow and gate height

It is observed from article 5.3.2 and Fig. 5.9, the overflow gate height is directly proportional to ratio of bypass flow and total inflow of Storm Diversion Structure and the correlation coefficient is 0.948 which is also a good correlation. From this correlation we can evaluate required gate height for any certain bypass flow (dry weather flow) and total inflow of Diversion structure (combined sewer flow).

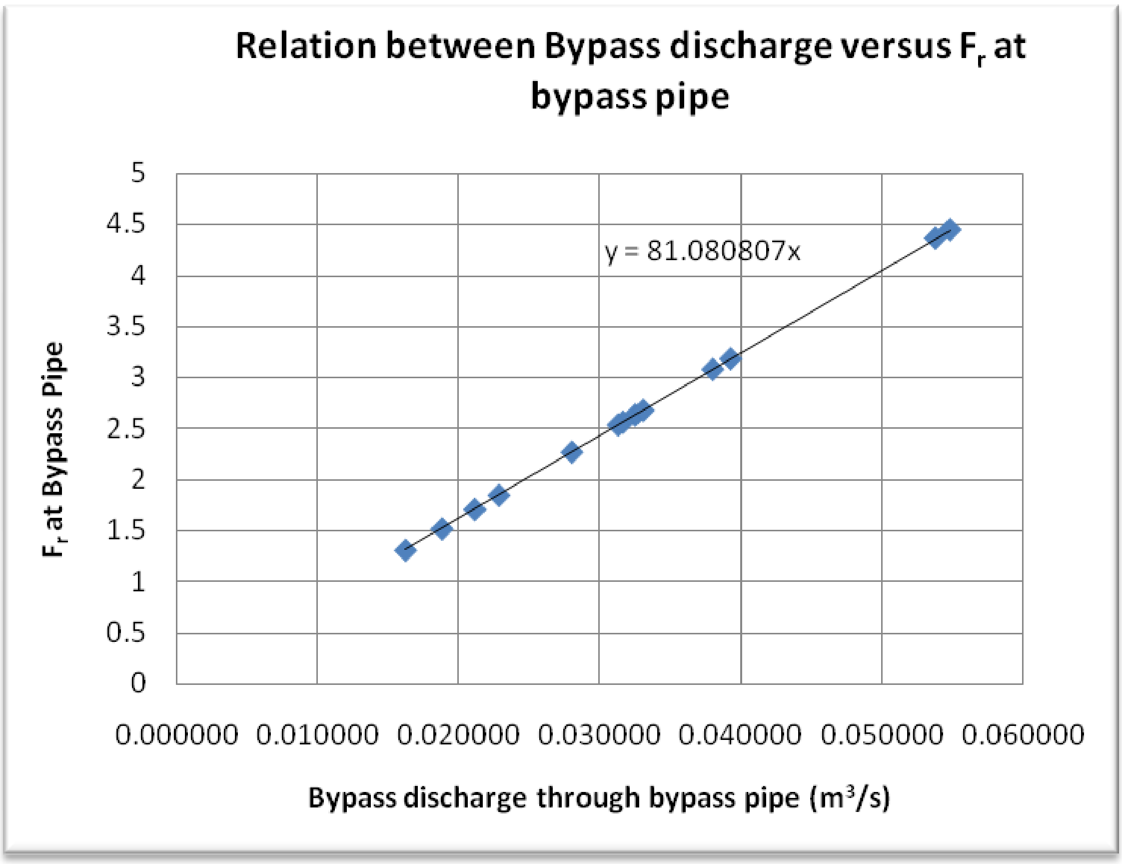


Fig 5.10: Relation between Froude Number and bypass discharge

5.4 Relation between Froude Numbers in bypass discharge through bypass pipe

From Fig. 5.10 It was found that bypass discharge is directly proportional to Froude Number in bypass pipe and the correlation coefficient is 0.991 which is very good correlation. From this correlation we can evaluate Froude number for any certain bypass flow (dry weather flow). Type of hydraulic jump and height of jump in the bypass pipe can be determined from this figure and article 2.4 of chapter two (Review of Literature).

5.5 SDS Model Results in prototype situation

Flow velocities of model have been measured in longitudinal direction for some arrangement of gate height and discharge mentioned in article 5.3. These relationships of discharge and velocities with overflow gate height have been transformed in prototype velocity and discharge and plotted. To determine the discharge in every arrangement water level at SDS approach channel, Diversion Structure, overflow channel. Total outlet channel and bypass pipe was measured in model.

5.6 SDS prototype Results and discussions

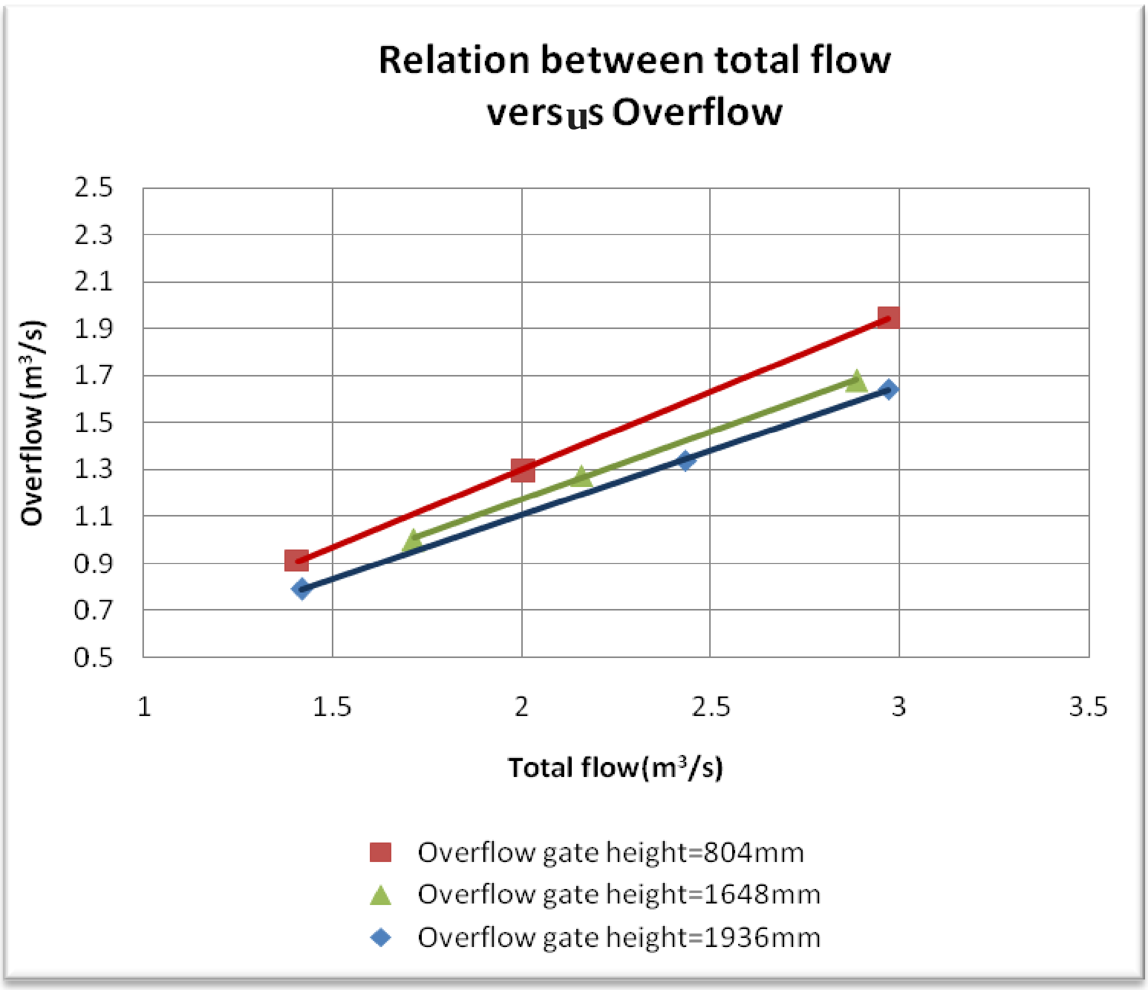


Fig: 5.11 Relation between total flow versus overflow of prototype (overflow gate height=804mm, 1648mm and 1936mm)

5.6.1 Relation between overflow and total inflow

For overflow gate height fixed at 804 mm, 1648 mm and 1936 mm (Figure-5.11) it was found that overflow rate is proportional to total inflow of Diversion structure. The proportionality is declining with increase of gate height, i.e. gate height is inversely proportional to ratio of overflow and total inflow of Diversion structure.

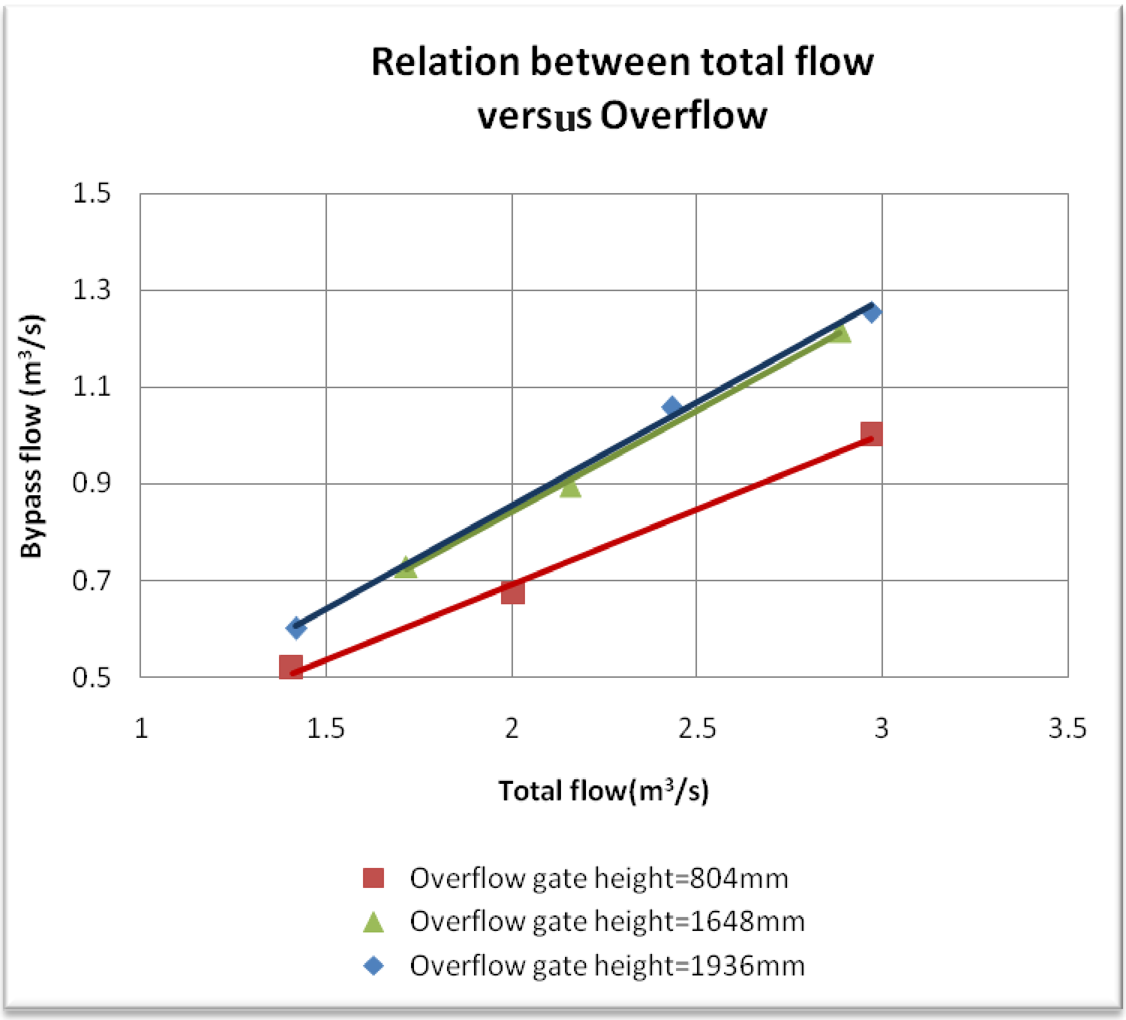


Fig: 5.12 Relation between total flow versus bypass flow of prototype (overflow gate height=804mm, 1648mm and 1936mm)

5.6.2 Relation between bypass flow and total inflow

An observation has also made on bypass flow and total inflow in the SDS Prototype data. For overflow gate height fixed at 804 mm, 1648 mm and 1936 mm (Figure-5.12) it was found that bypass flow rate is proportional to total inflow of Diversion structure.

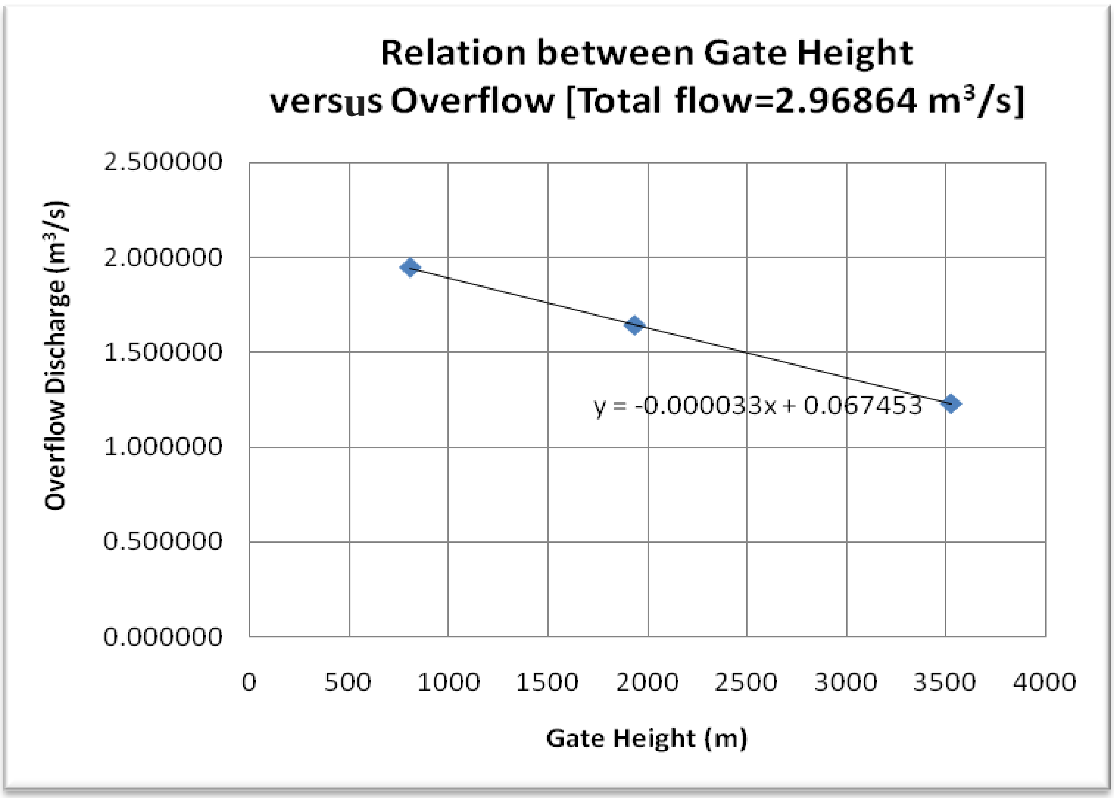


Fig: 5.13 Relation between overflow gate height versus overflow of prototype (Total flow=2.96864 m^3/s)

5.6.3 Relation between overflow and gate height

For a constant total inflow of Diversion structure (Fig 5.13 for constant Q=2.96864 m^3/s), It was found that overflow rate is inversely proportional to gate height and correlation coefficient is 0.999 which is very good correlation.

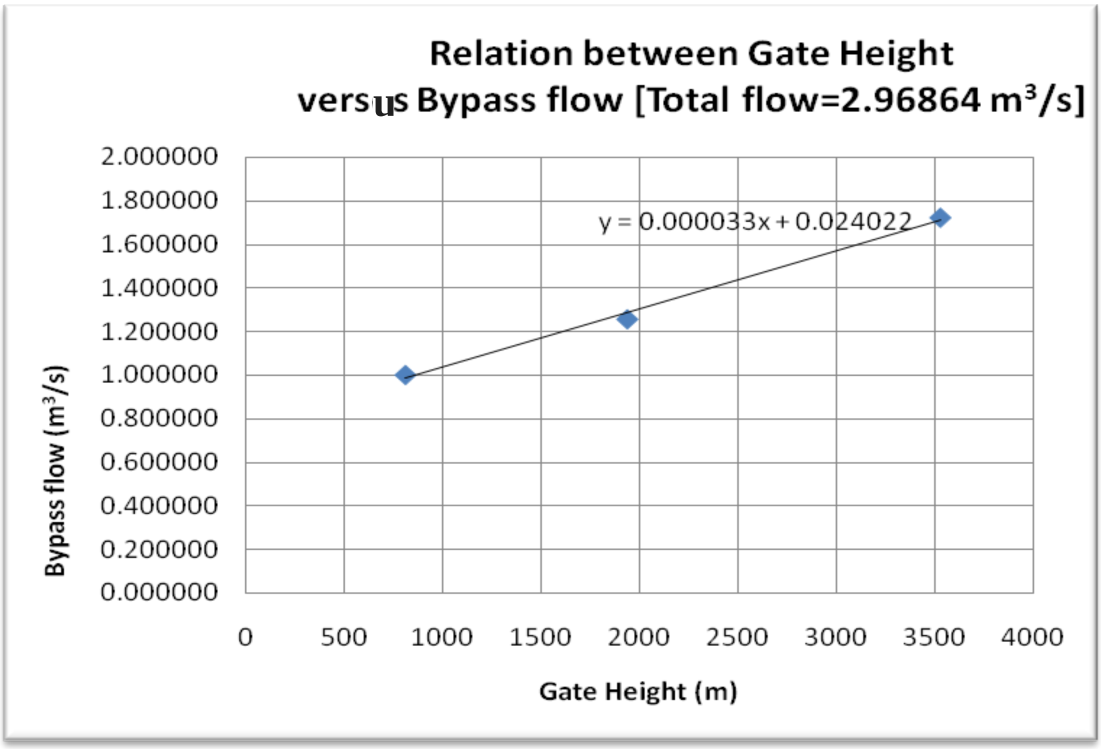


Fig: 5.14 Relation between overflow gate height versus bypass flow of prototype (Total flow=2.96864 m^3/s)

5.6.4 Relation between overflow and gate height

An observation has also made on bypass flow and gate height of prototype. For a constant total inflow of Diversion structure (Fig 5.14 for constant Q=2.96864 m^3/s), It was found that bypass flow rate is proportional to gate height and the correlation coefficient is 0.995 which is also good correlation.

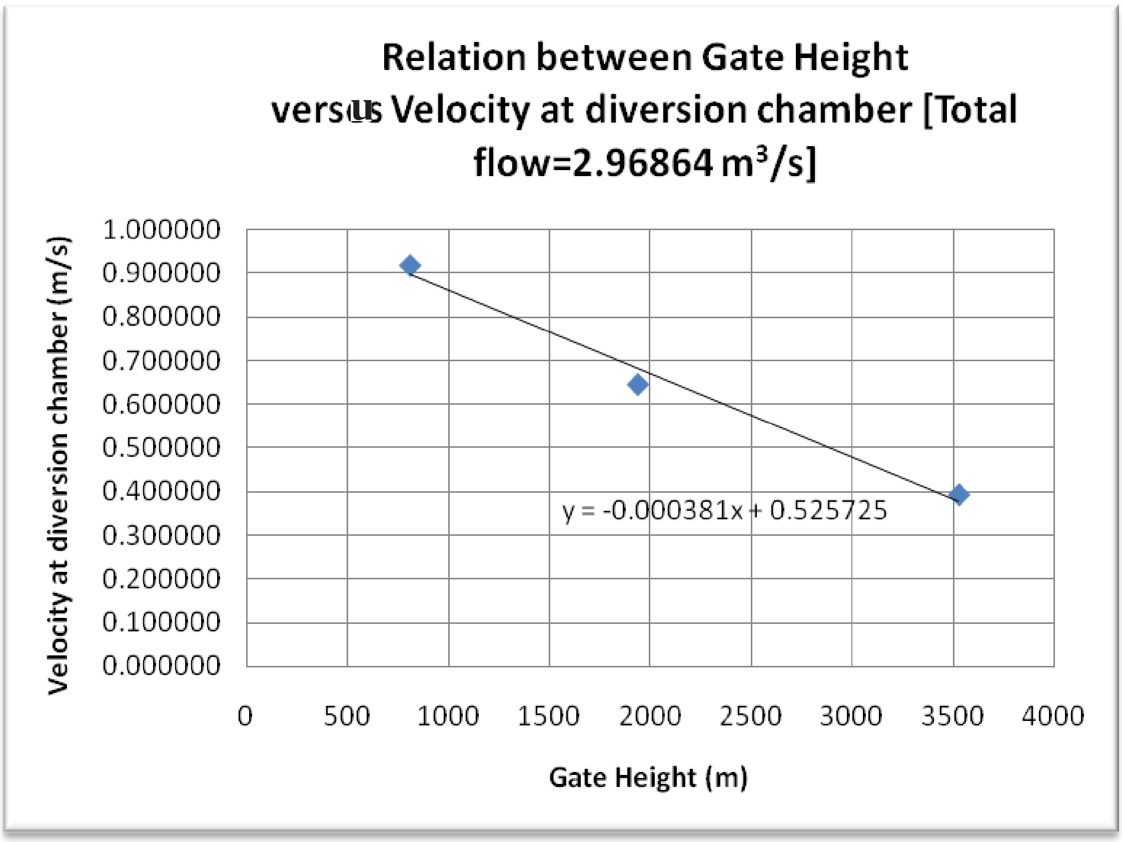


Fig: 5.15 Relation between overflow gate height versus velocity at diversion structure of prototype (Total flow=2.96864 m^3/s)

5.6.5 Relation between velocity in diversion structure and gate height

For a constant total inflow of Diversion structure (Fig 5.15 for constant Q=2.96864 m^3/s), It was found that velocity in diversion structure is inversely proportional to gate height and the correlation coefficient is 0.985 which is also good correlation.

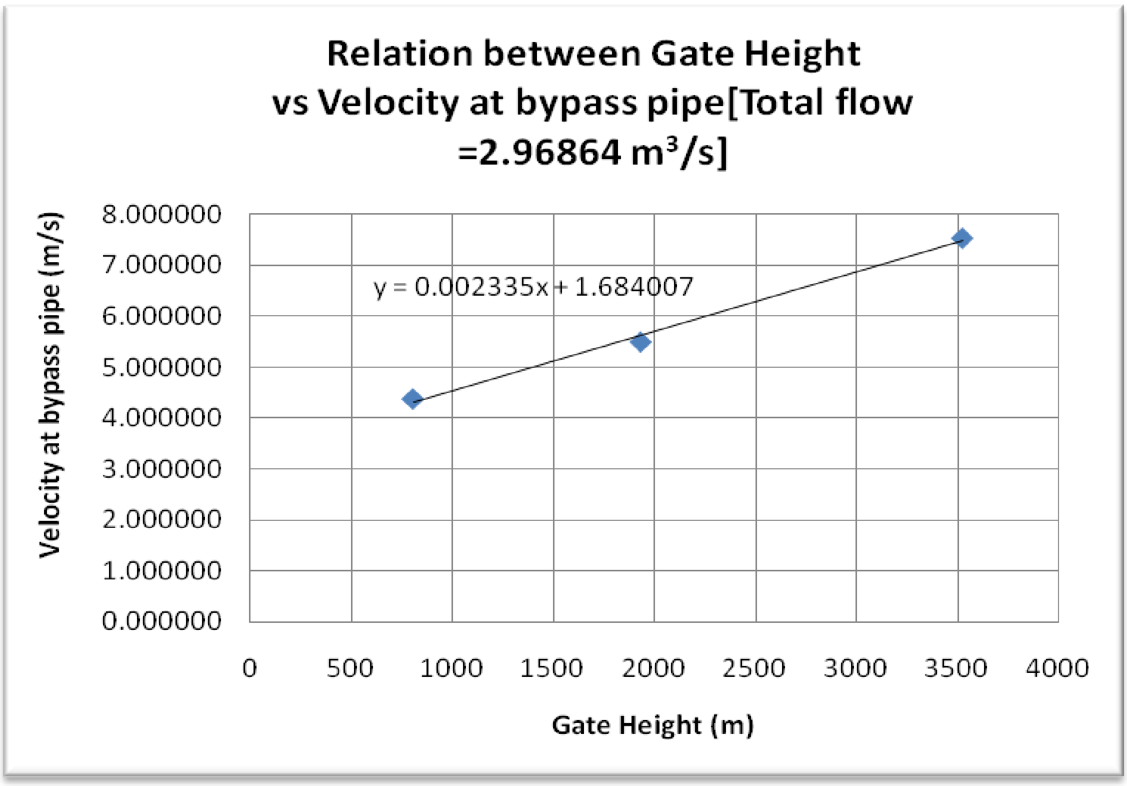


Fig: 5.16 Relation between overflow gate height versus velocity at bypass pipe of prototype (Total flow=2.96864 m^3)

5.6.6 Relation between velocity in bypass pipe and gate height

The velocity in the SDS has also measured and found for a constant total inflow of Diversion structure. For a constant total inflow of Diversion structure (Fig 5.16 for constant Q=2.96864m^3/s), It was found that velocity in bypass pipe is proportional to gate height and the correlation coefficient is 0.995 which is also good correlation.

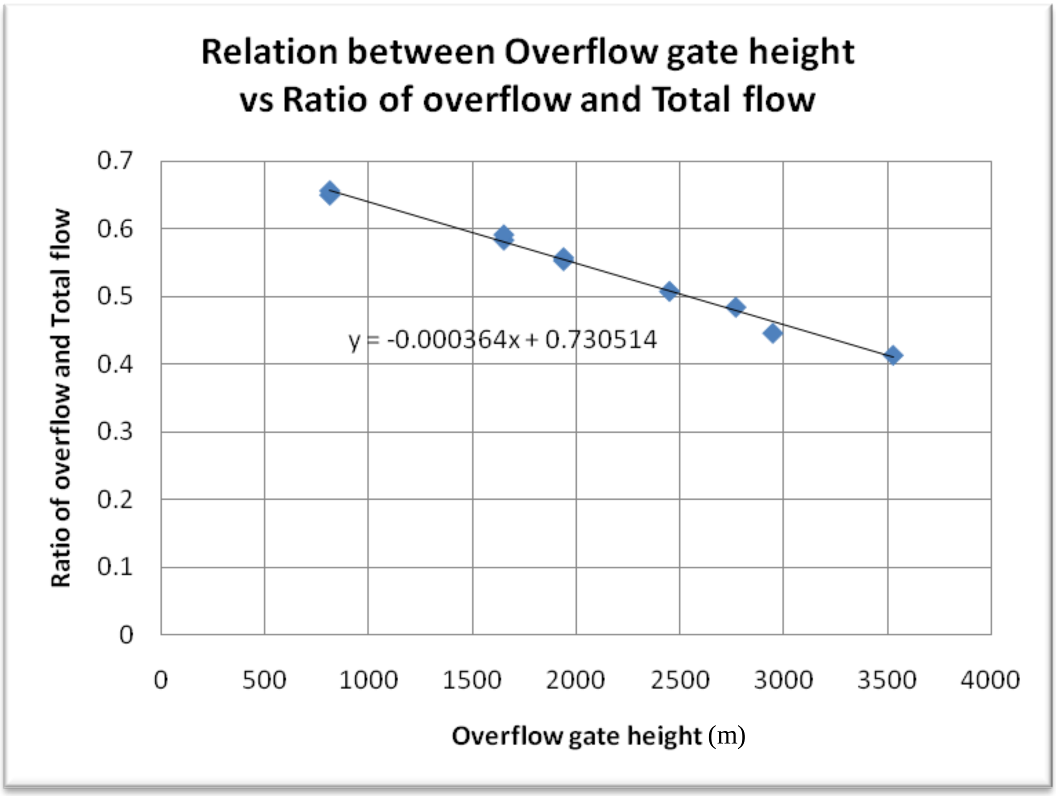


Fig: 5.17 Relation between overflow gate height versus ratio of overflow and total flow of prototype

5.6.7 Relation between ratio of overflow and total inflow and gate height

As discussed in article 5.6.1 and shown in Fig. 5.17, it was observed that gate height is inversely proportional to ratio of overflow and total inflow of Diversion structure and the correlation coefficient is 0.991 which is also a good correlation. From this correlation we can evaluate required gate height for any certain overflow (water to be disposed in cannel) and total inflow of Diversion structure (combined sewer flow).

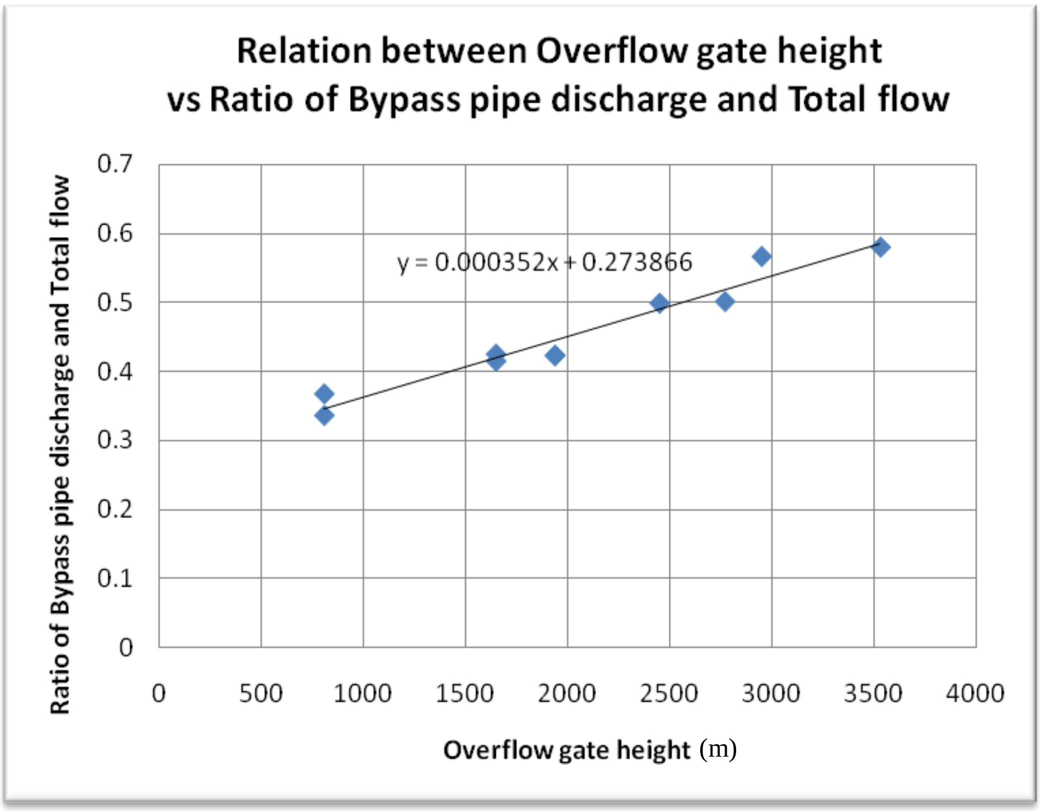


Fig: 5.18 Relation between overflow gate height versus ratio of bypass pipe discharge of prototype

5.6.8 Relation between ratio of bypass flow and total inflow and gate height

From the discussion in article 5.6.2 and shown in Fig. 5.18, it was observed that gate height is directly proportional to ratio of bypass flow and total inflow of Diversion structure and the correlation coefficient is 0.948. This is also a good correlation. From this correlation we can evaluate required gate height for any certain bypass flow (dry weather flow) and total inflow of Diversion structure (combined sewer flow).

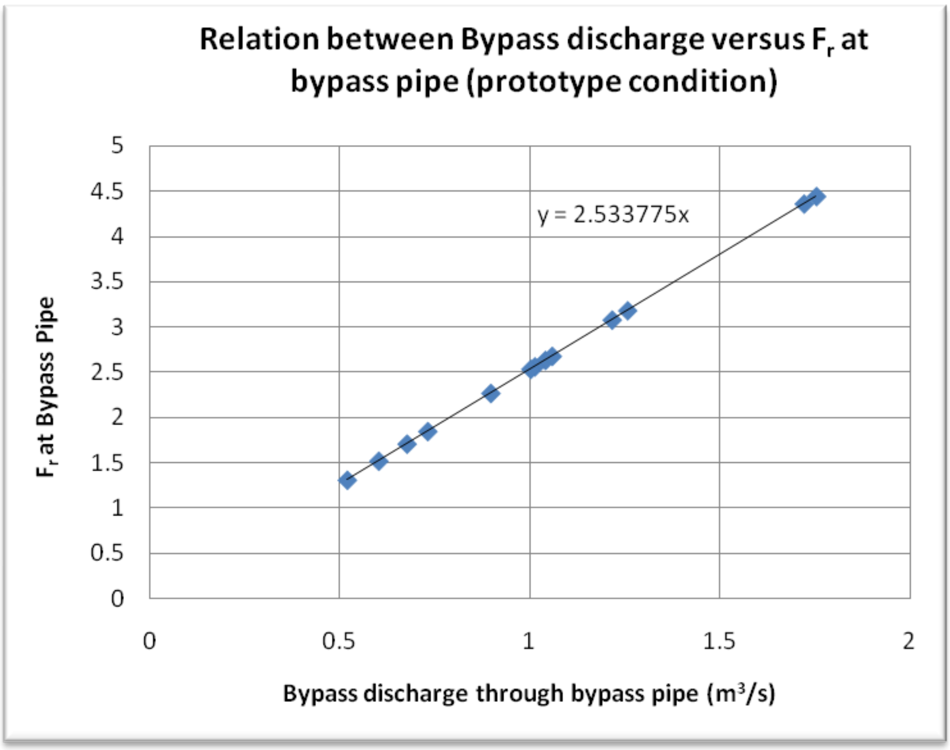


Figure: 5.19 Relation between flow at bypass pipe versus F_r at bypass pipe.

5.4 Relation between Froude Number bypass discharge through bypass pipe

From Fig. 5.19 It was found that bypass discharge is directly proportional to Froude Number in bypass pipe and the correlation coefficient is 0.991 which is very good correlation. From this correlation we can evaluate Froude number for any certain bypass flow (dry weather flow). Type of hydraulic jump and height of jump in the bypass pipe can be determined from this figure and article 2.4 of chapter two (Review of Literature).

5.7 Some empirical relationship

From the graphical relationship of both model and prototype as discussed in the previous article, the following empirical relationship has been developed. This relationship can be used to predict overflow rate for given overflow gate height and dry weather flow is known. Froude number at different part of the Storm Diversion Structure (SDS) has also calculated and drawn relationship with discharge at that section.

$$\frac{Q_O}{Q_T} = -0.000364 \times H_O + 0.73051 \tag{5.1}$$

$$and \ \frac{Q_B}{Q_T} = 0.000352 \times H_O + 0.273866 \tag{5.2}$$

Where,

Q_O = Discharge at Overflow Gate (m^3)

Q_B = Discharge at Bypass Pipe (m^3)

Q_T = Total flow at Combined Sewer Pipe (m^3)

H_O = Overflow Gate Height (m)

Again, It is observed that (from Figure 5.19)

$$F_r = 2.533775 Q_b \tag{5.3}$$

From Equation 2.11,

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + 20.27 Q_b{}^2} - 1\right) \tag{5.4}$$

Again $y_1 = C_d \times h_o$ (5.5)

Here, y_1=initial depth, y_2 = sequent depth, Q_b=bypass flow, h_o=bypass gate opening and C_d = discharge coefficient.

The continuity of inflow and outflow discharge rate has been assessed for the model run. It is found that, these two discharges are very close. The error is estimated and found only about 2% due to little leakage and instrumental measurement. Velocities within the diversion chamber are measured and found to be ranges between 0.1433 m/s and 0.4593 m/s as shown in Table 5.2(c). At the vicinity of the gates the velocities are also measured as shown in Table 5.2(c). At underflow exit (dry weather flow pipe) the velocity is found to be extremely high (3.84 m/s), indicating highly supercritical flow with increased Froude number (Fr =4.4419).

TABLE 5.2a: EXPERIMENT DATA TABLE.

Run No.	Overflow gate height	Bypass gate opening	Upstream weir reading	Upstream weir head	Overflow weir reading	Overflow weir initial gaze reading	Overflow weir head	Current meter reading at point “A”			Current meter reading at point “B”			Current meter reading at point “C”	Current meter reading at point “D”			Current meter reading at point “E”		
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(n)			(n)			(n)	(n)			(n)		
								0.6d	0.2d	0.8d	0.6d	0.2d	0.8d	Center	0.6d	0.2d	0.8d	0.6d	0.2d	0.8d
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	201	76.2	155	63.4	803	900	97	20.1	22	19.2	13.7	14.6	13.2	3.62	14.8	16.2	14.5	11	11.9	10.6
2	201	76.2	196	104.4	740	900	160	10.8	14.5	13.1	9.3	11.2	7.6	1.85	7.7	8.5	7.2	5.46	5.8	5.4
3	201	76.2	172	80.4	778	900	122	14.6	13.8	15.1	10.2	9.8	10.8	2.76	10.2	9.8	10.9	7.6	7	8.1
4	412	76.2	194	102.4	545	690	145	20.4	19.8	21.5	12.5	12.1	12.8	1.52	5.3	5	5.5	2.7	2.5	3
5	412	76.2	164	72.4	587	690	103	26.8	28.4	25.5	20.5	20.5	19.5	2.55	7.4	8.1	7	9.5	10.2	9
6	412	76.2	176	84.4	569	690	121	23.3	24.7	22	16.5	17	16	2.07	6.4	6.8	6.3	6	6.4	6
7	612	76.2	172.6	81	385	490	105	36.2	38	34.5	24.5	26	23.5	1.83	9	9.5	8.5	6.8	7.2	6.5
8	736.8	76.2	167.4	75.8	242	151.8	90.2	52.5	55.9	50	35	37	33.5	1.78	15	16	14.2	10.2	11	9.7
9	882	76.2	196	104.4	102	220	118	35	37	33.5	23.8	25.5	22.5	1.07	13.5	14.5	12.9	8.2	9.1	7.8
10	692	76.2	208	116.4	26	172	146	30.5	32	29	21	22.5	20	1.05	10.4	11	10	6.1	6.5	5.8
11	484	76.2	196	104.4	475	618	143	19.5	20.7	19	13.6	14.5	13	1.47	10.6	11.5	10	7.6	8	7.3
12	484	76.2	183	91.4	493	618	125	25.3	25	25.8	17.6	16.8	18.1	1.75	5.2	4.9	5.5	6.2	6	6.5
13	484	76.2	155.3	63.7	530	618	88	52.4	55.5	50	34.5	37	33	3.11	9	9.5	8.5	11	12	10.5

TABLE 5.2 b: EXPERIMENT DATA TABLE.

Run No.	Depth of water at point "A" (mm)	Depth of water at point "B" (mm)	Area of bypass pipe at point "C" (mm^2)	Depth of water at point "D" (mm)	Depth of water at point "E" (mm)	Upstream weir discharge m^3/s	Overflow weir discharge m^3/s	Discharge through point "A" m^3/s at 0.6d	Discharge through point "A" 0.2d & 0.8 d	Discharge through point "B" m^3/s at 0.6d	Discharge through point "B" 0.2d & 0.8 d	Discharge through point "C" m^3/s	Discharge through point "D" m^3/s at 0.6d	Discharge through point "D" 0.2d & 0.8 d	Discharge through point "E" m^3/s at 0.6d	Discharge through point "E" 0.2d & 0.8 d
	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
1	286	299	14264	182	148	**0.043977**	**0.028574**	0.043187	0.042273	0.044358	0.043778	0.016184	0.028012	0.027155	0.045056	0.044129
2	348	362	14264	213	133	**0.092775**	**0.060865**	0.092048	0.073490	0.076819	0.076053	0.031272	0.060194	0.059105	0.07854	0.076651
3	309	322	14264	193	142	**0.062689**	**0.040363**	0.062003	0.062586	0.062682	0.062115	0.021098	0.04188	0.041315	0.061141	0.061525
4	610	562	14264	130	77	**0.090114**	**0.052418**	0.090931	0.089972	0.090657	0.090991	0.037972	0.052495	0.052977	0.090182	0.088574
5	450	515	14264	107	154	**0.053599**	**0.031273**	0.053124	0.052876	0.053349	0.054510	0.022801	0.031399	0.030807	0.053739	0.053215
6	511	533	14264	116	125	**0.067416**	**0.039865**	0.067913	0.067789	0.066867	0.066867	0.027993	0.039089	0.038233	0.067425	0.065341
7	689	717	14264	134	132	**0.063390**	**0.032191**	0.063652	0.063583	0.063716	0.063169	0.031609	0.032688	0.032688	0.063173	0.062734
8	827	860	14264	165	175	**0.057402**	**0.025621**	0.057610	0.057255	0.056953	0.056631	0.032486	0.025136	0.024985	0.057146	0.056375
9	973	1012	14264	232	232	**0.092775**	**0.038382**	0.092331	0.091809	0.092177	0.091522	0.053767	0.038884	0.038367	0.092965	0.090369
10	1030	1075	14264	245	205	**0.109301**	**0.052967**	0.109243	0.109243	0.109050	0.107937	0.054783	0.052212	0.05175	0.10884	0.107992
11	595	620	14264	248	215	**0.092775**	**0.051325**	0.092261	0.090836	0.092596	0.091676	0.039249	0.051924	0.051251	0.092573	0.092
12	611	636	14264	102	145	**0.075970**	**0.041872**	0.075712	0.075460	0.075335	0.075909	0.033036	0.041951	0.041951	0.075795	0.075215
13	644	670	14264	102	145	**0.044287**	**0.024689**	0.044924	0.044708	0.044883	0.044370	0.018770	0.024882	0.024882	0.044143	0.043234

TABLE 5.2 c: EXPERIMENT DATA TABLE.

Run No.	Ce for Upstream Weir	Ce for Upstream Weir	Upstream weir discharge	Overflow weir discharge	Bypass pipe discharge	Velocity at point "A"		Velocity at point "B"		Velocity at point "C"	Velocity at point "D"		Velocity at point "E"	
			L/s	L/s	L/s	(m/s)		(m/s)		(m/s)	(m/s)		(m/s)	
						at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d	center	at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d
	37	38	39	40	41	42	43	44	45	46	47	48	49	50
1	0.6076	0.61137	**43.98**	**28.57**	**16.18**	0.2281	0.2233	0.3211	0.3169	1.1345	0.2989	0.2897	0.3928	0.3847
2	0.6122	0.61844	**92.77**	**60.87**	**31.27**	0.3996	0.319	0.4593	0.4547	2.1922	0.5487	0.5388	0.762	0.7436
3	0.60951	0.61418	**62.69**	**40.36**	**21.1**	0.3031	0.306	0.4214	0.4175	1.479	0.4214	0.4157	0.5556	0.5591
4	0.61198	0.61676	**90.11**	**52.42**	**37.97**	0.2252	0.2228	0.3492	0.3504	2.6619	0.7841	0.7913	1.5112	1.4843
5	0.60861	0.61204	**53.6**	**31.27**	**22.8**	0.1783	0.1775	0.2242	0.2291	1.5984	0.5698	0.5591	0.4503	0.4459
6	0.60996	0.61406	**67.42**	**39.86**	**27.99**	0.2008	0.2004	0.2715	0.2715	1.9623	0.6543	0.64	0.696	0.6745
7	0.60958	0.61227	**63.39**	**32.19**	**31.61**	0.1396	0.1394	0.1923	0.1907	2.2159	0.4737	0.4737	0.6175	0.6132
8	0.60899	0.61061	**57.4**	**25.62**	**32.49**	0.1052	0.1046	0.1433	0.1425	2.2773	0.2958	0.294	0.4214	0.4157
9	0.6122	0.61373	**92.77**	**38.38**	**53.77**	0.1433	0.1425	0.1972	0.1958	3.7692	0.3254	0.3211	0.517	0.5026
10	0.61355	0.61687	**109.3**	**52.97**	**54.78**	0.1602	0.1602	0.2196	0.2173	3.8404	0.4138	0.4101	0.6851	0.6797
11	0.6122	0.61653	**92.77**	**51.33**	**39.25**	0.2342	0.2306	0.3233	0.3201	2.7514	0.4065	0.4013	0.5556	0.5521
12	0.61074	0.61451	**75.97**	**41.87**	**33.04**	0.1872	0.1866	0.2564	0.2583	2.3159	0.7986	0.7986	0.6745	0.6693
13	0.60764	0.61036	**44.29**	**24.69**	**18.77**	0.1054	0.1049	0.145	0.1433	1.3158	0.4737	0.4737	0.3928	0.3847

TABLE 5.3: RELATIONSHIP OF EXPERIMENTAL DATA

Run No.	Overflow gate height	Ratio of Velocity & Discharge at point "A"		Ratio of Velocity & Discharge at point "B"		Ratio of Velocity & Discharge at point "C"	Ratio of Velocity & Discharge at point "D"		Ratio of Velocity & Discharge at point "E"		Ratio of overflow and Total flow	Ratio of Bypass pipe discharge and Total flow
		at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d	center	at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d		
1	201	5.281727	5.281727	7.239138	7.239138	70.10194	10.66894	10.66894	8.718396	8.718396	0.65	0.368
2	201	4.34073	4.34073	5.979288	5.979288	70.10194	9.116186	9.116186	9.701674	9.701674	0.656	0.337
3	201	4.888589	4.888589	6.722056	6.722056	70.10194	10.06087	10.06087	9.086779	9.086779	0.644	0.337
4	412	2.476351	2.476351	3.851427	3.851427	70.10194	14.93652	14.93652	16.75744	16.75744	0.582	0.421
5	412	3.356831	3.356831	4.202917	4.202917	70.10194	18.14717	18.14717	8.378718	8.378718	0.583	0.425
6	412	2.956114	2.956114	4.06098	4.06098	70.10194	16.7392	16.7392	10.32258	10.32258	0.591	0.415
7	612	2.192415	2.192415	3.018831	3.018831	70.10194	14.49065	14.49065	9.775171	9.775171	0.508	0.499
8	736.8	1.826571	1.826571	2.516863	2.516863	70.10194	11.76817	11.76817	7.373272	7.373272	0.446	0.566
9	882	1.552491	1.552491	2.138836	2.138836	70.10194	8.369602	8.369602	5.561735	5.561735	0.414	0.58
10	692	1.466577	1.466577	2.01349	2.01349	70.10194	7.9255	7.9255	6.294256	6.294256	0.485	0.501
11	484	2.53878	2.53878	3.491133	3.491133	70.10194	7.829627	7.829627	6.0015	6.0015	0.553	0.423
12	484	2.472298	2.472298	3.403305	3.403305	70.10194	19.03674	19.03674	8.898776	8.898776	0.551	0.435
13	484	2.345612	2.345612	3.2306	3.2306	70.10194	19.03674	19.03674	8.898776	8.898776	0.557	0.424

TABLE 5.4a: PROTOTYPE DATA TABLE.

Run No.	Overflow gate height	Upstream weir discharge	Overflow weir discharge	Discharge through point "A"		Discharge through point "B"		Discharge through point "C"	Discharge through point "D"		Discharge through point "E"	
	(mm)	m^3/s	m^3/s	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d	m^3/s	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d	m^3/s at 0.6d	0.2d & 0.8 d
1	804	1.407252	0.914381	1.381999	1.35272	1.41947	1.40089	0.517886	0.896384	0.868964	1.4418	1.412125
2	804	2.968792	1.947688	2.945536	2.351678	2.458206	2.433706	1.000711	1.92621	1.891349	2.513272	2.452832
3	804	2.006046	1.291617	1.984112	2.002737	2.005829	1.987695	0.675131	1.340172	1.322086	1.956527	1.968808
4	1648	2.883636	1.67736	2.909784	2.879094	2.901034	2.911717	1.215097	1.679844	1.695251	2.88583	2.834367
5	1648	1.715174	1.000728	1.699969	1.692045	1.707154	1.744312	0.72964	1.004782	0.985835	1.719645	1.702886
6	1648	2.157301	1.275664	2.173228	2.169247	2.139743	2.139743	0.895762	1.250836	1.223461	2.1576	2.0909
7	2448	2.028491	1.030111	2.036877	2.034651	2.038902	2.021412	1.011503	1.046008	1.046008	2.021544	2.007481
8	2947	1.836869	0.819856	1.843517	1.832167	1.822495	1.812185	1.039544	0.804339	0.799535	1.828672	1.803993
9	3528	2.968792	1.228228	2.954588	2.937872	2.94966	2.928696	1.720551	1.244291	1.227745	2.974892	2.891814
10	2768	3.49764	1.694931	3.495773	3.495773	3.489605	3.453973	1.753071	1.67079	1.655993	3.482873	3.455756
11	1936	2.968792	1.642415	2.952357	2.906745	2.96307	2.933646	1.255976	1.661575	1.640044	2.962347	2.943996
12	1936	2.431033	1.339892	2.422774	2.414713	2.410705	2.429083	1.057138	1.342441	1.342441	2.425444	2.406875
13	1936	1.417175	0.790051	1.437565	1.430652	1.436266	1.419851	0.600641	0.796215	0.796215	1.412574	1.383501

TABLE 5.4 b: PROTOTYPE DATA TABLE.

Run No.	Overflow gate height	Upstream weir discharge	Overflow weir discharge	Velocity at point "A"		Velocity at point "B"		Velocity at point "C"	Velocity at point "D"		Velocity at point "E"	
	(mm)	m^3/s	m^3/s	(m/s)	(m/s)	(m/s)	(m/s)	(m/s)	(m/s)	(m/s)	(m/s)	(m/s)
				at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d	center	at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d
1	804	1.407252	0.914381	0.45621	0.44654	0.6422	0.6338	2.269	0.5977	0.5794	0.7856	0.7695
2	804	2.968792	1.947688	0.79911	0.638	0.9186	0.9095	4.3845	1.0975	1.0776	1.5239	1.4873
3	804	2.006046	1.291617	0.60622	0.61191	0.8427	0.8351	2.958	0.8427	0.8313	1.1112	1.1181
4	1648	2.883636	1.67736	0.45035	0.4456	0.6983	0.7009	5.3238	1.5682	1.5826	3.0224	2.9685
5	1648	1.715174	1.000728	0.35666	0.35499	0.4484	0.4582	3.1968	1.1396	1.1181	0.9005	0.8918
6	1648	2.157301	1.275664	0.40152	0.40078	0.5431	0.5431	3.9247	1.3086	1.28	1.392	1.349
7	2448	2.028491	1.030111	0.2791	0.2788	0.3847	0.3814	4.4318	0.9473	0.9473	1.2351	1.2265
8	2947	1.836869	0.819856	0.21046	0.20916	0.2867	0.2851	4.5546	0.5916	0.5881	0.8427	0.8313
9	3528	2.968792	1.228228	0.28669	0.28506	0.3943	0.3915	7.5384	0.6509	0.6422	1.0341	1.0052
10	2768	3.49764	1.694931	0.32043	0.32043	0.4391	0.4347	7.6809	0.8276	0.8203	1.3701	1.3595
11	1936	2.968792	1.642415	0.46846	0.46122	0.6465	0.6401	5.5029	0.8131	0.8026	1.1112	1.1043
12	1936	2.431033	1.339892	0.37436	0.37312	0.5128	0.5167	4.6317	1.5972	1.5972	1.349	1.3386
13	1936	1.417175	0.790051	0.21075	0.20973	0.29	0.2867	2.6316	0.9473	0.9473	0.7856	0.7695

TABLE 5.5: FROUDE NUMBER AT DIFFERENT COMPONENTS OF SDS.

Upstream weir discharge	Overflow gate height	Froude number at point "A"		Froude number at point "B"		Froude number at point "C"
		At 0.6d	avg. of .2d&.8d	At 0.6d	avg. of .2d&.8d	At center
0.043977	201	0.3676	0.3598	0.5175	0.5107	1.3122
0.044287	484	0.1698	0.169	0.2337	0.231	1.5219
0.053599	412	0.2874	0.286	0.3613	0.3692	1.8487
0.057402	736.8	0.1696	0.1685	0.231	0.2297	2.634
0.062689	201	0.4885	0.4931	0.679	0.6729	1.7106
0.063390	612	0.2249	0.2247	0.31	0.3073	2.5629
0.067416	412	0.3235	0.3229	0.4376	0.4376	2.2697
0.075970	484	0.3017	0.3007	0.4132	0.4163	2.6786
0.090114	412	0.3629	0.3591	0.5627	0.5648	3.0788
0.092775	201	0.6439	0.5141	0.7402	0.7329	2.5356
0.092775	882	0.231	0.2297	0.3177	0.3155	4.3595
0.092775	484	0.3775	0.3716	0.521	0.5158	3.1824
0.109301	692	0.2582	0.2582	0.3539	0.3502	**4.4419**

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

6.1 Introduction

In this present study, flow behaviors of the SDS for different overflow gate conditions have been observed. Froude number at different part of the Storm Diversion Structure (SDS) has also been calculated to check the type of flow. Based on the model results, some empirical relationships have been developed for the use in prototype conditions.

6.2 Conclusions

Based on the detail experimental investigation, analysis and discussion presented in forgoing chapters, following conclusions may be drawn

1. The ratio of overflow discharge and total inflow of SDS is inversely proportional to overflow gate height.

2. The ratio of bypass flow discharge and total inflow flow of SDS is directly proportional to overflow gate height.

3. The overflow gate needs to be controlled depending on the storm water to be disposed to the lake. Equation 5.1 can be used to estimate the overflow discharge. The amount of flow to be disposed at Hatirjhil can be calculated by deducting the known dry weather flow from the incoming flow of SDS.

4. Equation 5.2 can be used to calculate bypass flow rate for respective overflow gate height.

5. The Froude number at different locations of SDS was evaluated. It is found that the flow is always subcritical in storm diversion chamber and that is supercritical at bypass pipe for flow rate grater then dry weather flow.

6. A hydraulic jump was observed in bypass pipe. The sequent depth of the flow through bypass pipe can be calculated from the Froude number and depth of water at storm diversion structure (Using Equation 5.3, 5.4 and 5.5). Froude number in the bypass pipe depends on combined sewer flow rate. This Froude number can be calculated at bypass pipe of SDS by using the equation 5.3

6.3 Future recommendation

It is to be noted that the proposed SDS does not include the pollutant treatment facilities, thus only hydraulic condition of the diversion flow chamber and outlets are studied in the model. The maximum discharge of this study was 0.092775 m^3/s which is commensurate with outfall for Tejgaon (Q4). Similar model study is necessary for other outfalls to investigate the flow behavior , especially culvert outfall (Q1) at panthapath.

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Appendix-A

Model Data collection

Table : Field data table.

Run No.	Overflow gate height	Bypass gate opening	Upstream weir reading	Upstream weir head	Overflow weir reading	Overflow weir initial gaze reading	Overflow weir head	Current meter reading at point “A”			Current meter reading at point “B”			Current meter reading at point “C”	Current meter reading at point “D”			Current meter reading at point “E”			Depth of water at point “A”	Depth of water at point “B”	Area of bypass pipe at point “C”	Depth of water at point “D”	Depth of water at point “E”
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(n)			(n)			(n)	(n)			(n)			(mm)	(mm)	(mm^2)	(mm)	(mm)
								0.6d	0.2d	0.8d	0.6d	0.2d	0.8d	Center	0.6d	0.2d	0.8d	0.6d	0.2d	0.8d					
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
1																									
2																									
3																									
4																									
5																									
6																									
7																									
8																									
9																									
10																									

a. Upstream weir initial gaze reading= mm   b. Upstream weir width, L=1500 mm   c. Overflow weir width= 512 mm

* Point “A”- inlet of CSO Structure, width=662mm   * Point “B”- inside of CSO Structure, width=463mm   * Point “C”- inlet of bypass pipe, A= mm^2

* Point “D”- outlet of bypass pipe, A= mm^2 * Point “E”- inside of overflow channel, width=515mm * Point “F”- Total outlet channel, width=775mm
* Current meter reading at 0.5 d throughout the channel and center of pipe for 30 rev.

Run No.	**Upstream weir discharge**	**Overflow weir discharge**	**Discharge through point "A"**		**Discharge through point "B"**		**Discharge through point "C"**	**Discharge through point "D"**		**Discharge through point "E"**	
	m^3/s	m^3/s	m^3/s		m^3/s		m^3/s	m^3/s		m^3/s	
			at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d		at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d
1	26	27	28	29	30	31	32	33	34	35	36
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											

Run No.	Ce for Upstream Weir	Ce for Upstream Weir	Upstream weir discharge	Overflow weir discharge	Bypass pipe discharge	Velocity at point “A”		Velocity at point “B”		Velocity at point “C”	Velocity at point “D”		Velocity at point “E”		Conservation check (Percent of error between mesured upstream weir discharge and sum of overflow and bypass discharge)	Remarks
			L/s	L/s	L/s	(m/s)		(m/s)		(m/s)	(m/s)		(m/s)		%	
						at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d	center	at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d		
1	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52
1																
2																
3																
4																
5																
6																
7																
8																
9																
10																
11																
12																
13																

FILL UP PROCEDURE OF DATA TABLE:

Collumn-1:- Overflow gate height from u/s bed

Collumn-2:- Bypass gate opening at center of pipe

Collumn-3:- Gaze reading at upstream weir in mm

Collumn-4:- (Collumn-3) - (Note-a)

Collumn-5:- Gaze reading at upstream of overflow weir in mm

Collumn-6:- Overflow weir initial gaze reading in mm

Collumn-7:- (Collumn-5) - (Collumn-6)

Collumn-8:- Current meter reading, n at 0.5 d at point “A” for 30 rev

Collumn-9:- Current meter reading, n at 0.5 d at point “B” for 30 rev

Collumn-10:- Current meter reading, n at center of pipe at point “C” for 30 rev

Collumn-11:- Current meter reading, n at center of pipe at point “D” for 30 rev

Collumn-12:- Current meter reading, n at 0.5 d at point “E” for 30 rev

Collumn-13:- Current meter reading, n at 0.5 d at point “F” for 30 rev

Collumn-14:- Depth of water at Point “A”- inlet of CSO Structure from direct measurement

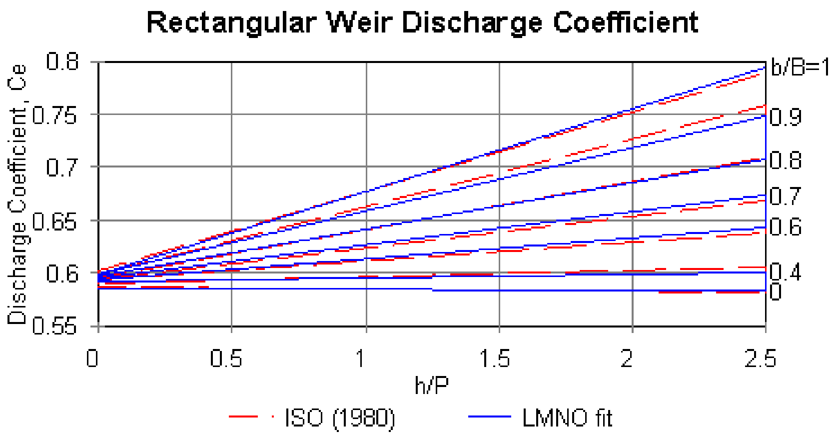
Collumn-15:- Depth of water at Point “B”- inside of CSO Structure from direct measurement

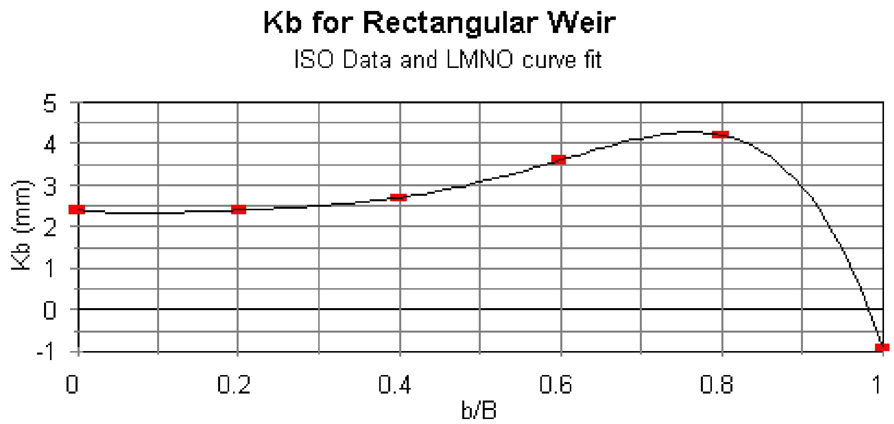
Collumn-16:- Area of bypass pipe at point “C”=14264 mm^2 for 76.2 mm opening

Collumn-17:- Area of bypass pipe at point “D”=14264 mm^2 for 76.2 mm opening

Collumn-18:- Depth of water at Point “E”- inside of overflow channel from direct measurement

Collumn-19:- Depth of water at Point “F”- Total outlet channel from direct measurement





Collumn-20:- Upstream weir discharge $Q = C_e \frac{2}{3}\sqrt{2g}\left(b+k_b\right)\left(h+k_h\right)^{\frac{3}{2}}$; $Q = C_e \times \frac{2}{3}\sqrt{2\times 9.8066}\times\left(1.5+0\right)\left(h+0.001\right)^{\frac{3}{2}}$ <h=col4>

H	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
C_e	0.602298684	0.6039737	0.604960055	0.6059474	0.606942	0.6089078	0.6059474	0.610881575	0.6118684	0.6128552	0.613842125	0.614828975	0.615815825	0.6168026	0.6187763	0.619763115	0.62075	0.62173685	0.62173685

Collumn-21:- Upstream weir discharge $Q = C_e \frac{2}{3}\sqrt{2g}\left(b+k_b\right)\left(h+k_h\right)^{\frac{3}{2}}$; $Q = C_e \times \frac{2}{3}\sqrt{2\times 9.8066}\times 0.512\left(h+0.001\right)^{\frac{3}{2}}$ <h=col7>

Collumn-22:- Discharge through point “A”

$$Q = AV = 0.662\times h\times V = 0.662\times(Col-14)\times 0.1334\left(\left(Col-8\right)+0.29\right);$$

$$V = 0.1334(n+0.29);\ n = (col-8)$$

Collumn-23:- Discharge through point "B"

$$Q = AV = 0.463 \times h \times V = 0.463 \times (Col - 15) \times 0.1334((Col - 9) + 0.29);$$

$$V = 0.1334(n + 0.29);\ n = (col - 9)$$

Collumn-24:- Discharge through point "C"

$$Q = AV = 0.01426494 \times V = 0.01426494 \times 0.1334((Col - 10) + 0.29);$$

$$V = 0.1334(n + 0.29);\ n = (col - 10)$$

Collumn-25:- Discharge through point "D"

$$Q = AV = 0.01426494 \times V = 0.01426494 \times 0.1334((Col - 11) + 0.29);$$

$$V = 0.1334(n + 0.29);\ n = (col - 11)$$

Collumn-26:- Discharge through point "E"

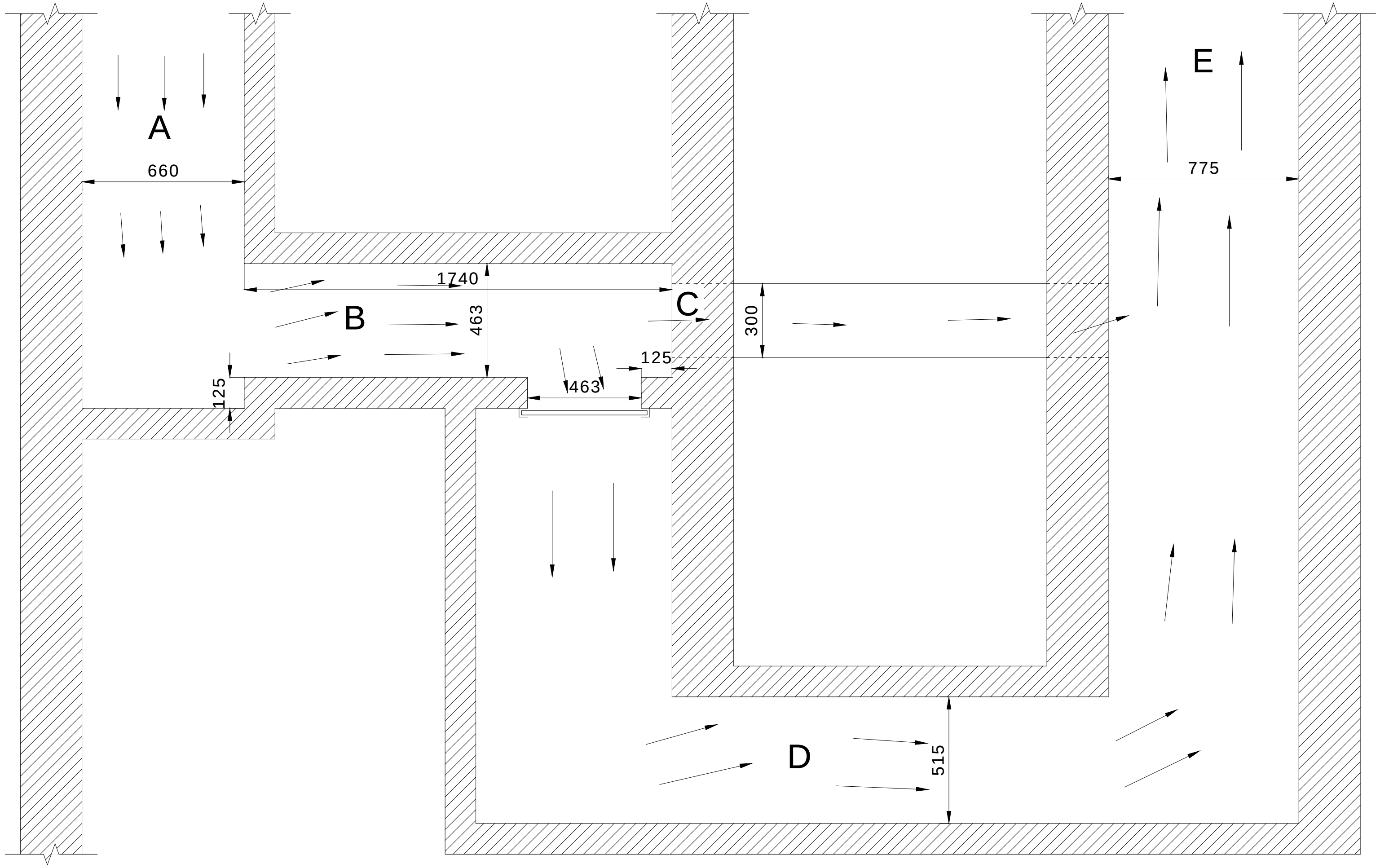
$$Q = AV = 0.515 \times h \times V = 0.515 \times (Col - 18) \times 0.1334((Col - 12) + 0.29);$$

$$V = 0.1334(n + 0.29);\ n = (col - 12)$$

Collumn-27:- Discharge through point "F"

$$Q = AV = 0.775 \times h \times V = 0.775 \times (Col - 19) \times 0.1334((Col - 13) + 0.29);$$

$$V = 0.1334(n + 0.29);\ n = (col - 13)$$



A
660
E
775
1740
B
463
C
300
125
463
125
D
515

TABLE : EXPERIMENT DATA TABLE

a. Upstream weir initial gaze reading = 91.6 mm

Run No.	Overflow gate height	Bypass gate opening	Upstream weir reading	Upstream weir head	Overflow weir reading	Overflow weir initial gaze reading	Overflow weir head	Current meter reading at point "A"			Current meter reading at point "B"			Current meter reading at point "C"	Current meter reading at point "D"			Current meter reading at point "E"			Depth of water at point "A"	Depth of water at point "B"	Area of bypass pipe at point "C"	Depth of water at point "D"	Depth of water at point "E"
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(n)			(n)			(n)	(n)			(n)			(mm)	(mm)	(mm^2)	(mm)	(mm)
								0.6d	0.2d	0.8d	0.6d	0.2d	0.8d	Center	0.6d	0.2d	0.8d	0.6d	0.2d	0.8d					
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
1	201	76.2	155	63.4	803	900	97	20.1	22	19.2	13.7	14.6	13.2	3.62	14.8	16.2	14.5	11	11.9	10.6	286	299	14264	182	148
13	484	76.2	155.3	63.7	530	618	88	52.4	55.5	50	34.5	37	33	3.11	9	9.5	8.5	11	12	10.5	644	670	14264	102	145
5	412	76.2	164	72.4	587	690	103	26.8	28.4	25.5	20.5	20.5	19.5	2.55	7.4	8.1	7	9.5	10.2	9	450	515	14264	107	154
8	736.8	76.2	167.4	75.8	242	151.8	90.2	52.5	55.9	50	35	37	33.5	1.78	15	16	14.2	10.2	11	9.7	827	860	14264	165	175
3	201	76.2	172	80.4	778	900	122	14.6	13.8	15.1	10.2	9.8	10.8	2.76	10.2	9.8	10.9	7.6	7	8.1	309	322	14264	193	142
7	612	76.2	172.6	81	385	490	105	36.2	38	34.5	24.5	26	23.5	1.83	9	9.5	8.5	6.8	7.2	6.5	689	717	14264	134	132
6	412	76.2	176	84.4	569	690	121	23.3	24.7	22	16.5	17	16	2.07	6.4	6.8	6.3	6	6.4	6	511	533	14264	116	125
12	484	76.2	183	91.4	493	618	125	25.3	25	25.8	17.6	16.8	18.1	1.75	5.2	4.9	5.5	6.2	6	6.5	611	636	14264	102	145
4	412	76.2	194	102.4	545	690	145	20.4	19.8	21.5	12.5	12.1	12.8	1.52	5.3	5	5.5	2.7	2.5	3	610	562	14264	130	77
2	201	76.2	196	104.4	740	900	160	10.8	14.5	13.1	9.3	11.2	7.6	1.85	7.7	8.5	7.2	5.46	5.8	5.4	348	362	14264	213	133
9	882	76.2	196	104.4	102	220	118	35	37	33.5	23.8	25.5	22.5	1.07	13.5	14.5	12.9	8.2	9.1	7.8	973	1012	14264	232	232

11	484	76.2	196	104.4	475	618	143	19.5	20.7	19	13.6	14.5	13	1.47	10.6	11.5	10	7.6	8	7.3	595	620	14264	248	215
10	692	76.2	208	116.4	26	172	146	30.5	32	29	21	22.5	20	1.05	10.4	11	10	6.1	6.5	5.8	1030	1075	14264	245	205

Run No.	Upstream weir discharge	Overflow weir discharge	Discharge through point “A”		Discharge through point “B”		Discharge through point “C”	Discharge through point “D”		Discharge through point “E”		Ce for Upstream Weir	Ce for Upstream Weir	Upstream weir discharge	Overflow weir discharge
	m^3/s	m^3/s	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d	m^3/s	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d	m^3/s at 0.6d	m^3/s 0.2d & 0.8 d			L/s	L/s
	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	**0.043977**	**0.028574**	0.043187	0.042273	0.044358	0.043778	0.016184	0.028012	0.027155	0.045056	0.044129	0.6076	0.61137	**43.976615**	**28.574**
13	**0.044287**	**0.024689**	0.044924	0.044708	0.044883	0.044370	0.018770	0.024882	0.024882	0.044143	0.043234	0.60764	0.61036	**44.286716**	**24.689**
5	**0.053599**	**0.031273**	0.053124	0.052876	0.053349	0.054510	0.022801	0.031399	0.030807	0.053739	0.053215	0.60861	0.61204	**53.599199**	**31.273**
8	**0.057402**	**0.025621**	0.057610	0.057255	0.056953	0.056631	0.032486	0.025136	0.024985	0.057146	0.056375	0.60899	0.61061	**57.402154**	**25.621**
3	**0.062689**	**0.040363**	0.062003	0.062586	0.062682	0.062115	0.021098	0.04188	0.041315	0.061141	0.061525	0.60951	0.61418	**62.688936**	**40.363**
7	**0.063390**	**0.032191**	0.063652	0.063583	0.063716	0.063169	0.031609	0.032688	0.032688	0.063173	0.062734	0.60958	0.61227	**63.390334**	**32.191**
6	**0.067416**	**0.039865**	0.067913	0.067789	0.066867	0.066867	0.027993	0.039089	0.038233	0.067425	0.065341	0.60996	0.61406	**67.415659**	**39.865**
12	**0.075970**	**0.041872**	0.075712	0.075460	0.075335	0.075909	0.033036	0.041951	0.041951	0.075795	0.075215	0.61074	0.61451	**75.969772**	**41.872**
4	**0.090114**	**0.052418**	0.090931	0.089972	0.090657	0.090991	0.037972	0.052495	0.052977	0.090182	0.088574	0.61198	0.61676	**90.113628**	**52.418**
2	**0.092775**	**0.060865**	0.092048	0.073490	0.076819	0.076053	0.031272	0.060194	0.059105	0.07854	0.076651	0.6122	0.61844	**92.774754**	**60.865**
9	**0.092775**	**0.038382**	0.092331	0.091809	0.092177	0.091522	0.053767	0.038884	0.038367	0.092965	0.090369	0.6122	0.61373	**92.774754**	**38.382**

11	**0.092775**	**0.051325**	0.092261	0.090836	0.092596	0.091676	0.039249	0.051924	0.051251	0.092573	0.092	0.6122	0.61653	**92.774754**	**51.325**
10	**0.109301**	**0.052967**	0.109243	0.109243	0.109050	0.107937	0.054783	0.052212	0.05175	0.10884	0.107992	0.61355	0.61687	**109.30126**	**52.967**

Run No.	Bypass pipe discharge	Velocity at point "A"		Velocity at point "B"		Velocity at point "C"	Velocity at point "D"		Velocity at point "E"		Conservation check (Percent of error between mesured upstream weir discharge and sum of overflow and bypass discharge)	Remarks
	L/s	(m/s)		(m/s)		(m/s)	(m/s)		(m/s)		%	
		at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d	center	at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d		
	41	42	43	44	45	46	47	48	49	50	51	52
1	**16.184**	0.2281	0.2233	0.3211	0.3169	1.1345	0.2989	0.2897	0.3928	0.3847	**1.78%**	**OK**
13	**18.77**	0.1054	0.1049	0.145	0.1433	1.3158	0.4737	0.4737	0.3928	0.3847	**-1.87%**	**OK**
5	**22.801**	0.1783	0.1775	0.2242	0.2291	1.5984	0.5698	0.5591	0.4503	0.4459	**0.89%**	**OK**
8	**32.486**	0.1052	0.1046	0.1433	0.1425	2.2773	0.2958	0.294	0.4214	0.4157	**1.23%**	**OK**
3	**21.098**	0.3031	0.306	0.4214	0.4175	1.479	0.4214	0.4157	0.5556	0.5591	**-1.96%**	**OK**
7	**31.609**	0.1396	0.1394	0.1923	0.1907	2.2159	0.4737	0.4737	0.6175	0.6132	**0.65%**	**OK**
6	**27.993**	0.2008	0.2004	0.2715	0.2715	1.9623	0.6543	0.64	0.696	0.6745	**0.65%**	**OK**
12	**33.036**	0.1872	0.1866	0.2564	0.2583	2.3159	0.7986	0.7986	0.6745	0.6693	**-1.40%**	**OK**
4	**37.972**	0.2252	0.2228	0.3492	0.3504	2.6619	0.7841	0.7913	1.5112	1.4843	**0.31%**	**OK**
2	**31.272**	0.3996	0.319	0.4593	0.4547	2.1922	0.5487	0.5388	0.762	0.7436	**-0.69%**	**OK**
9	**53.767**	0.1433	0.1425	0.1972	0.1958	3.7692	0.3254	0.3211	0.517	0.5026	**-0.67%**	**OK**

11	**39.249**	0.2342	0.2306	0.3233	0.3201	2.7514	0.4065	0.4013	0.5556	0.5521	**-2.37%**	**OK**
10	**54.783**	0.1602	0.1602	0.2196	0.2173	3.8404	0.4138	0.4101	0.6851	0.6797	**-1.42%**	**OK**

A SCALE MODEL STUDY OF FLOW BEHAVIOR OF STORM DIVERSION STRUCTURE

Md. Asifur Rahman
Student No: 040816017(P)

In partial fulfillment of the requirement for the degree of

Master of Engineering (Water Resources)

Department of Water Resources Engineering

Bangladesh University of Engineering and Technology, Dhaka.

12th March, 2011

Organization of presentation

1. Introduction
2. Background of the study
3. Experimental Setup and Data Collection
4. Results and Discussions
5. Conclusions and Recommendations

Organization of thesis chapters

Chapters of the thesis has been organized as per literature and working sequence. The first chapter, Introduction deals with the general overview of the thesis. The second chapter, Literature has overviewed about thesis related theory. Chapter three has discussed design of storm diversion structure with specific reference of hatirjhil begunbari intigrated project.

Organization of thesis chapters (Contd.)

Experimental facility, experimental setup and test run has discussed in the chapter four. Physical model results and discussions have given in chapter five. Finally chapter six has discussed conclusion and future recommendation.

Introduction

The low-lying area behind the Sonargaon Hotel and the Hatirjheel lowlands, extending from the east of Tejgaon diversion road up to the Pragati Shwarani at Rampura, receive significant discharges through a number of major storm sewer outfalls. Illegal connections of both domestic and industrial wastewaters to the storm sewer network are usual case. As a result, during dry season, the storm sewers mainly carry significant flows of domestic sewage as well as industrial wastewater. The untreated domestic

Introduction (Contd.)

sewage and industrial wastewater drains through this low land via the Begunbari khal-Norai khal into the Balu river; the Balu river eventually discharges into the Sitalakhya river. During monsoon, the pollution level drops to some extent due to dilution of domestic sewage and industrial effluent by stormwater. Over the years, the lowlands behind Sonargaon Hotel and Hatirjheel have virtually been turned into wastelands. In order to manage this combine water flow, storm water diversion system can be introduced.

Introduction (Contd.)

Storm Diversion structure (SDS) is a structure which is used to separate dry weather flow from storm water flow. Working principle of SDS is that storm water combined with dry weather sewerage flow is used to divert excess flows received during storm events into nearby receiving water body (lake), thus relieving other hydraulic structures within the area and reducing the risk of flooding in urban areas.

Introduction (Contd.)

In this study, attempt has been made to study the hydraulic behavior of a Storm Diversion Structure (SDS) in laboratory model under various flow conditions. In addition the overflow gate operation at various flow conditions have also been studied. The flow condition of the receiving watercourse and within the structure chamber have been observed for various gate operations.

Introduction (Contd.)

For a typical scale model study outfall Q4 of hatirjel- Begunbari area located near Tejgaon diversion road has been selected. Physical modeling facility of Department of Water Resources Engineering, BUET has been used for this purpose. Froudian law has been applied to design the laboratory scale model. Based on the availability of the space and the discharge capacity in modeling facilities, an undistorted model of scale 1:4 has been selected.

OBJECTIVES WITH SPECIFIC AIMS

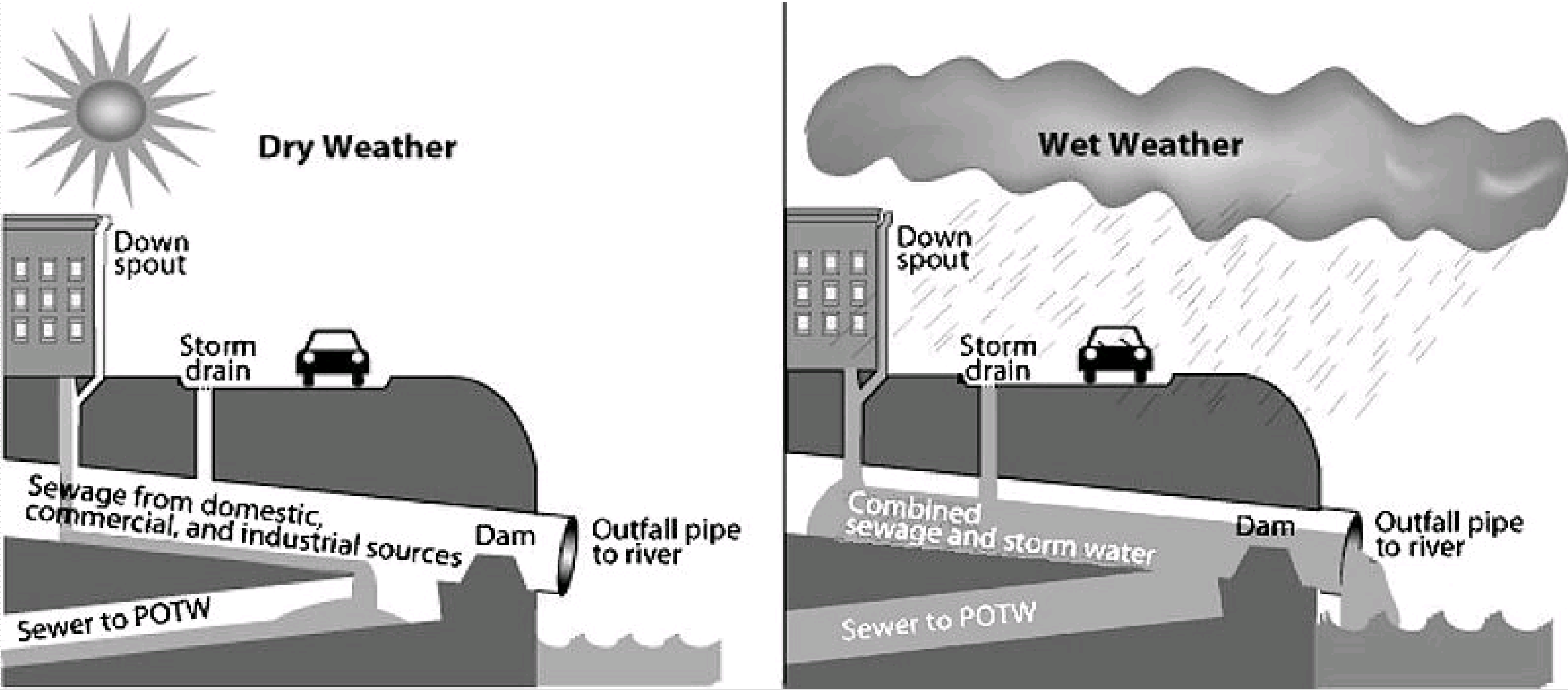
The specific objectives of this study are as follows:

- To design and develop a scale model in laboratory to conduct the study for hydraulic behavior of Storm diversion Structure (SDS) system.
- To study of overflow gate operation at various flow condition.

Definition of CSO

A combined sewer is a sewer that is designed to carry both sanitary sewage and storm water runoff in a single pipe system. Discharge from a combined sewer system occurs in response to rainfall and/or snowmelt because the carrying capacity of the sewer system is exceeded. These discharges do not receive all treatment that is available and utilized. under ordinary dry weather conditions (normally during dry weather conditions the wastewater is transported to a wastewater treatment facility where it receives appropriate treatment prior to discharge). Both the combined sewer overflow structure and the discharge from the structure are referred to as Combined Sewer Overflow structure "CSOs"

DEFINITION OF CSO



Definition sketch of Combined Sewer System.

Definition of CSO

Most of the time, combined sewer systems transport all of their wastewater to a sewage treatment plant, where it is treated and then discharged to a water body. During periods of heavy rainfall or snowmelt, however, the wastewater volume in a combined sewer system can exceed the capacity of the sewer system or treatment plant. For this reason, combined sewer systems are designed to overflow occasionally and discharge excess wastewater directly to nearby streams, rivers, or other water bodies.

Definition of CSO

In order to manage combine water flow, storm water diversion system can be introduced. This combined flow regulatory based on the conveyance of domestic and industrial effluents and the surface runoff from catchments surfaces in underground conduits or open drains. In Dhaka city illegal connections of both domestic and industrial wastewaters to the storm sewer network are usual case. As a result, during dry season, the storm sewers mainly carry significant flows of domestic sewage as well as industrial wastewater.

Various types of system can be used treat the pollutants

In general, either or combination of the following four types of system can be used treat the pollutants

- A combined sewer overflow structure (CSO)
- An emergency overflow at a pumping station and / or detention tanks
- An overflow from storm tanks at a sewage / wastewater treatment works, and
- An overflow from an emergency spill weir at a sewage / wastewater treatment works.

Identification of Major Outfalls

The first and very important task in connection with the design of the main diversion sewers was the identification of major storm sewer outfalls currently being discharged into the lowlands behind Sonargaon Hotel and Hatirjheel. Nine major outfalls were found along the periphery of the project are. the outfalls are identified as Q1 through Q9. a number of smaller discharges have also been identified. The 48 inch diameter storm sewer outfall (Q4) coming from the north and discharging into the Hatirjheel immediately to the east of Tongi diversion road.

Identification of Major Outfalls

Out fall located at the North-west Tajgaon diversion road (Q4)

Drainage outfalls of hatirjhil begunbari intigrated project

Dry Weather Flow

Estimation of dry weather flow through the major outfalls was a very important task, based on which the hydraulic capacity of the main diversion sewers will have to be determined. Estimation of dry weather flow through all 9 major outfalls was carried out in January 2008. A number of techniques were used for discharge measurements, majority of them were accomplished through measurement of flow velocity through a flow meter and determination of cross-sectional area of flow.

Dry Weather Flow

Discharge through major outfalls discharging into the lowland behind Sonargaon Hotel and Hatirjheel

Discharge	Date of Measurement	Time of Measurement	Discharge (cfs)
Q1	07-01-08	10:00-12:00	61
Q2	13-01-08	11:00-11:10	10
Q3	13-01-08	11:00-11:10	7
Q4	13-01-08	10:30-10:40	2
Qc	13-01-08	11:45-12:30	80
Q5	13-01-08	13:45-14:00	4
Q6(S)	21-01-08	11:00-11:30	22
Q6(L)	21-01-08	11:30-12:00	14
Q6	21-01-08	12:15-13:15	38
Q7	21-01-08	14:15-14:30	11
Q8(S)	11-02-08	10:50-11:00	3
Q8(L)	11-02-08	10:35-10:45	14
Q9	21-01-08	13:45-14:00	4

Drainage outfalls of hatirjhil begunbari intigrated project

Flow calculation

- Storm Runoff estimation
- Estimation of Domestic Wastewater Flow .

Experimental Setup and Test Run

In order to assess the hydraulic behavior under extreme flow conditions, attempt has been made to study the diversion structure in a scale model. Physical modeling facility of Department of Water Resources Engineering (DWRE), BUET has used for this purpose. Depending on the availability of the space and the discharge capacity of the physical model facility of DWRE, an undistorted model of scale 1:4 has been selected. Based on this scale, plan and elevation of the designed SDS model are shown in Figureure 5(a) ,(b) and (c).

Experimental Setup and Test Run

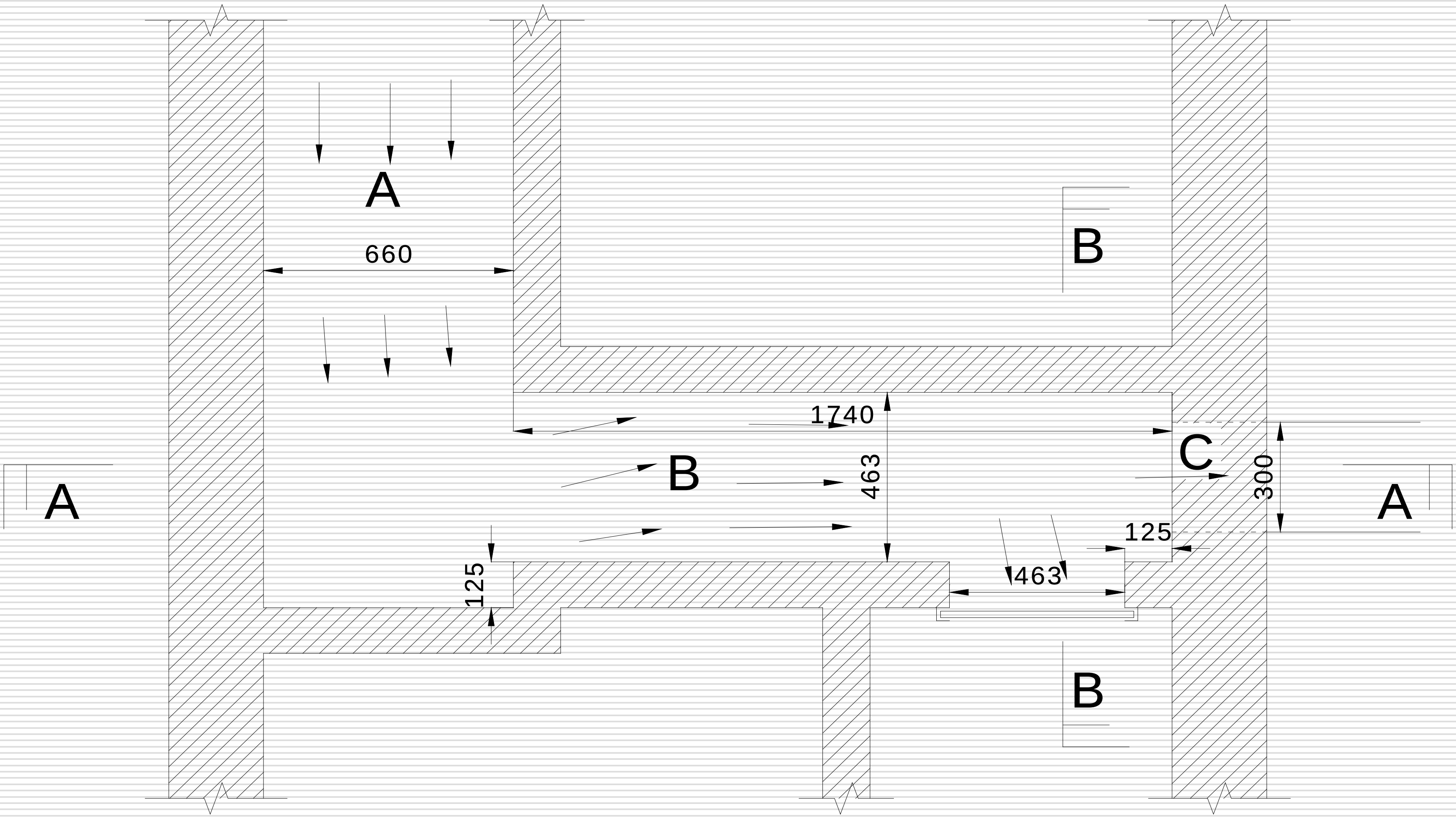


Figure: Plan of Model of Flow Diversion Structure

Experimental Setup and Test Run

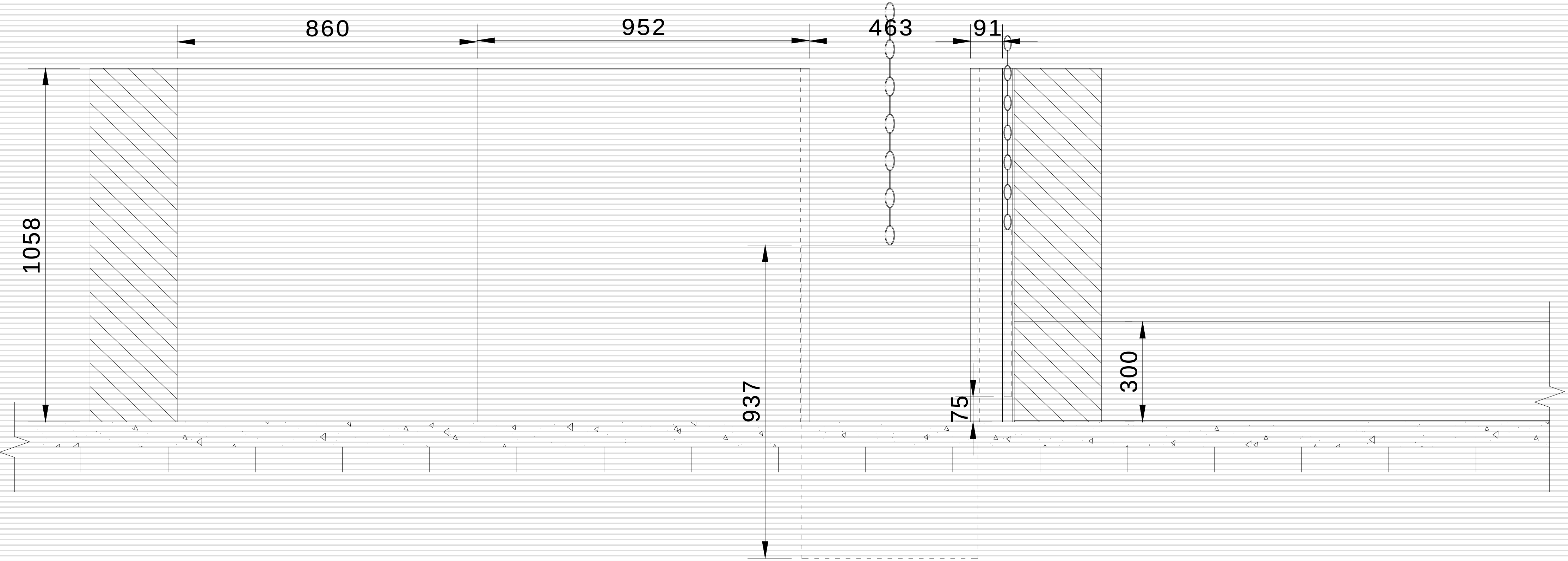


Figure: Section A-A (Section along Underflow Pipe of Flow Diversion Structure)

Experimental Setup and Test Run

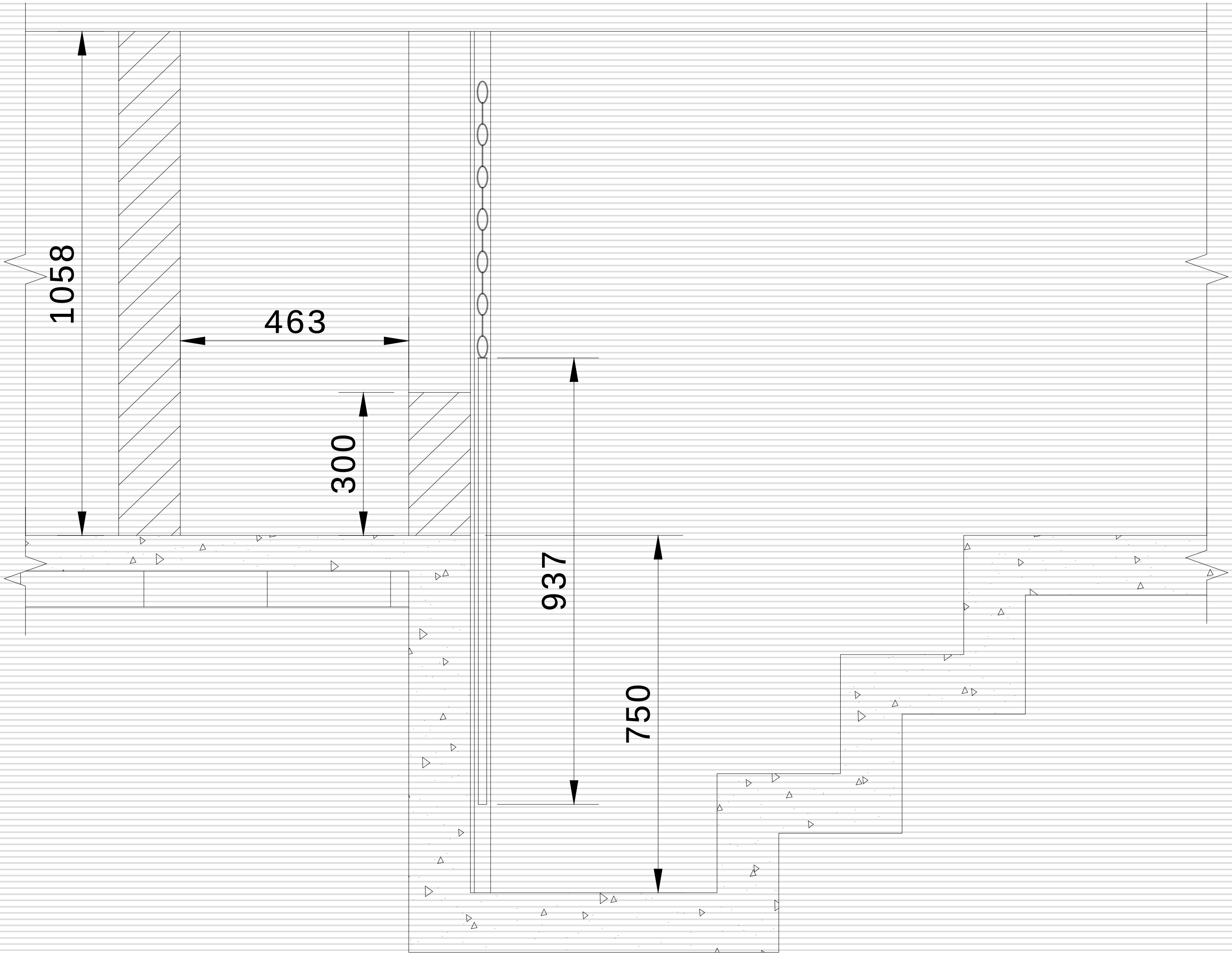


Figure: Section B-B (Section along Overflow Gate of Flow Diversion Structure)

Experimental Setup and Test Run

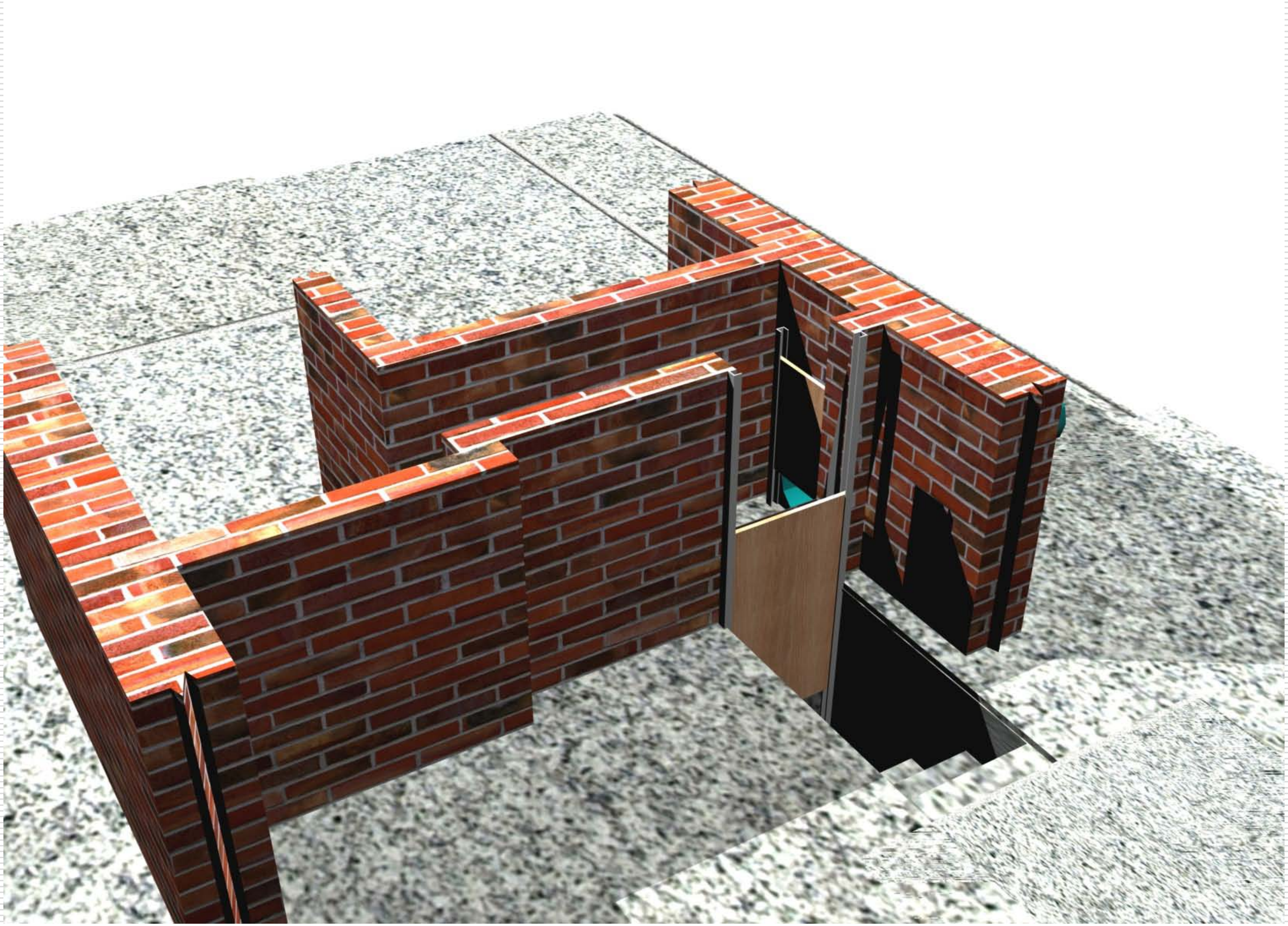


Figure:3D View of proposed model with over flow gate

Experimental Setup and Test Run

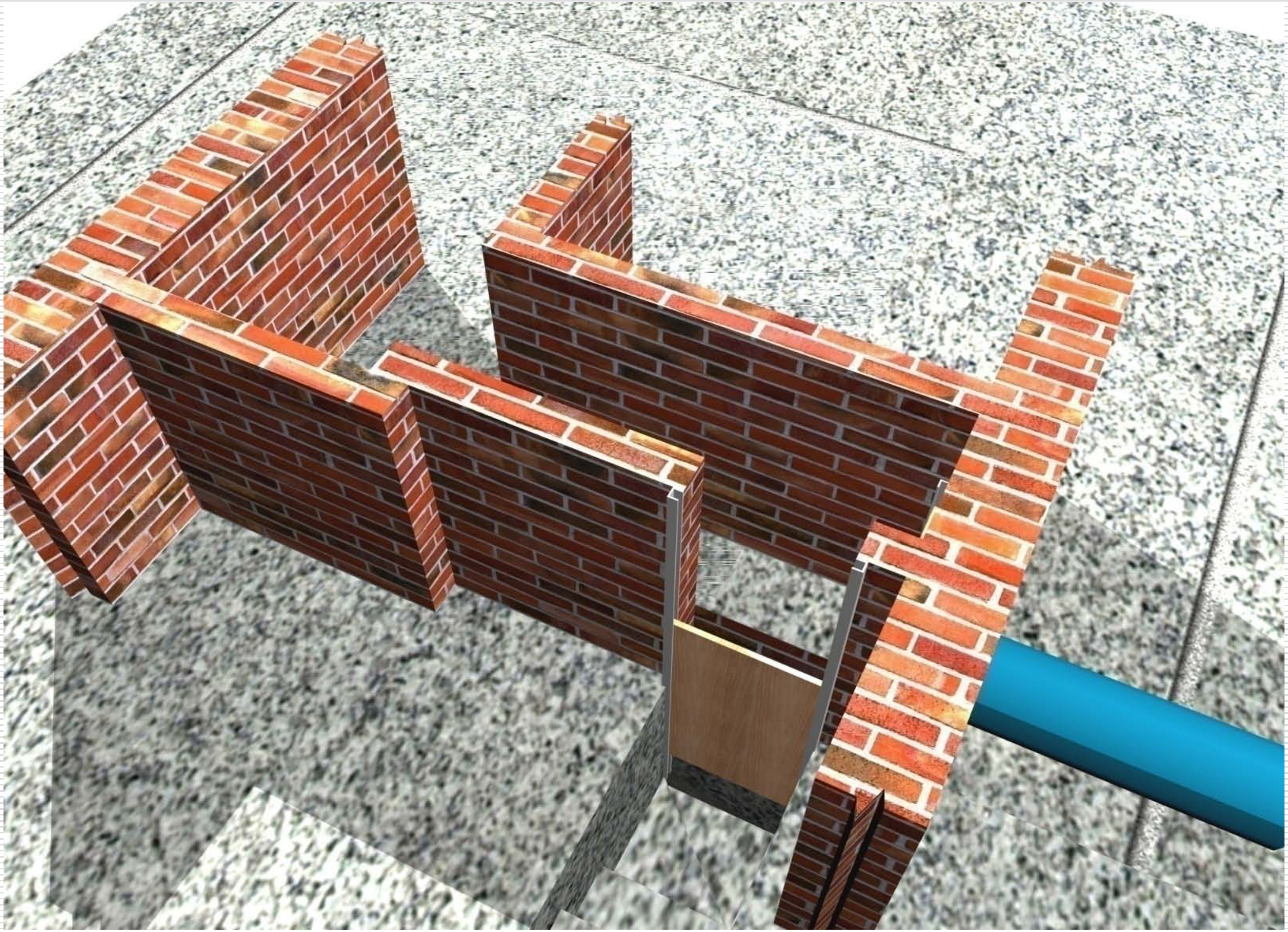


Figure: 3D View of proposed model with diversion pipe

Experimental Setup and Test Run

Figure: **Overview physical model facility**

Experimental Setup and Test Run

Table : Dimensions of the various components of Physical Model Facilities

Component of model facility	Length [m]	Width [m]	Depth [m]	Capacity [m3]
Storage pool	10.67	6.09	3.20	210.00
Upstream reservoir	10.67	3.05	1.37	44.61
Straight flume	45.00	2.45	0.46	-
Sediment trap	10.67	3.66	1.40	55.00
Downstream reservoir	10.67	1.52	0.61	9.75
Re-circulating canal	52.44	0.76	1.28	-

Experimental Setup and Test Run

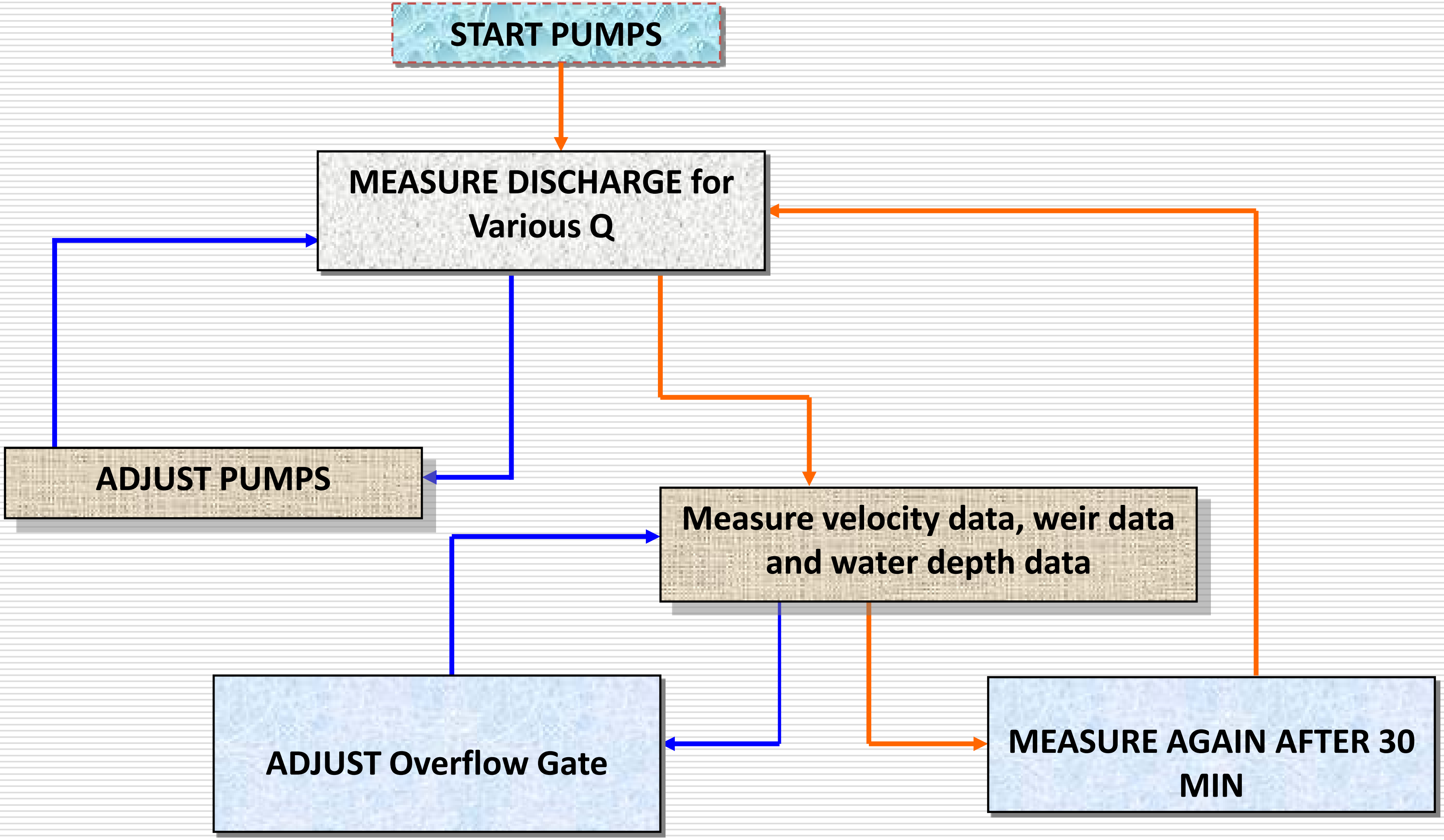


Figure : Flow chart of reaching boundary conditions

Experimental Setup and Test Run

Figure : Layout of experimental setup

Experimental Setup and Test Run

Figure : Construction of diversion chamber

Experimental Setup and Test Run

Figure : 12”dia diversion pipe

Experimental Setup and Test Run

Figure : Measurement of the overflow gate height with a level pipe

Experimental Setup and Test Run

Figure : Water recycling pump.

Experimental Setup and Test Run

Figure : First run of the model

Experimental Setup and Test Run

Figure : Measurement of u/s weir discharge.

Experimental Setup and Test Run

Figure : Measurement of velocity at diversion structure with current meter.

Experimental Setup and Test Run

Figure : Model is running low gate height and high discharge.
(discharge= 0.0928 m^3/s and gate height=201mm)

Experimental Setup and Test Run

Figure : Model is running average gate height and low discharge (discharge= 0.0443 m^3/s and gate height=484mm)

Experimental Setup and Test Run

Figure : Model is running highest gate height and high discharge.
(discharge= 0.109 m^3/s and gate height=692mm)

Experimental Setup and Test Run

Figure : Model is running on it's average capacity (discharge= 0.0901 m^3/s and gate height=412mm)

Experimental Setup and Test Run

Figure : Model is runs highest gate height and high discharge.
(discharge= 0.0927 m^3/s and gate height=882mm)

Experimental Setup and Test Run

Figure : Eight run of the model (discharge= 0.0574 m^3/s and gate height=736mm)

Experimental Setup and Test Run

Figure : Model data collection with a laptop computer

Model Results and Discussions

Model Test scenarios

Initially several trial runs had been conducted for various discharges (low, medium and high flow cases) for a certain gate openings. Also for constant discharge, flow conditions were observed for various gate openings. It was observed that the model flow conditions and outlets flow are sensitive to all the options, particularly for higher flow and raised gate at overflow weir.

A total of 13 test runs have been conducted to observe the flow behavior of the model. These are shown in Table

Model Results and Discussions

Model Test scenarios

Run No.	Overflow gate height	Upstream weir head	Upstream weir discharge	Overflow weir head	Overflow weir discharge
	(mm)	(mm)	m^3/s	(mm)	m^3/s
1	201	63.4	**0.043977**	97	**0.028574**
2	201	104.4	**0.092775**	160	**0.060865**
3	201	80.4	**0.062689**	122	**0.040363**
4	412	102.4	**0.090114**	145	**0.052418**
5	412	72.4	**0.053599**	103	**0.031273**
6	412	84.4	**0.067416**	121	**0.039865**
7	612	81	**0.063390**	105	**0.032191**
8	736.8	75.8	**0.057402**	90.2	**0.025621**
9	882	104.4	**0.092775**	118	**0.038382**
10	692	116.4	**0.109301**	146	**0.052967**
11	484	104.4	**0.092775**	143	**0.051325**
12	484	91.4	**0.075970**	125	**0.041872**
13	484	63.7	**0.044287**	88	**0.024689**

Model Results and Discussions

Model Observations

For all the test runs following observations have been made:

- Flow velocity and water level at the Storm Diversion Structure (SDS), diversion pipe and overflow outlet
- Water surface condition within the diversion chamber.
- Flow behavior under various overflow gate operation and for different discharge condition.

All the test runs measurements are given in Table 5.2. The results are also presented in graphical forms

Model Results and Discussions

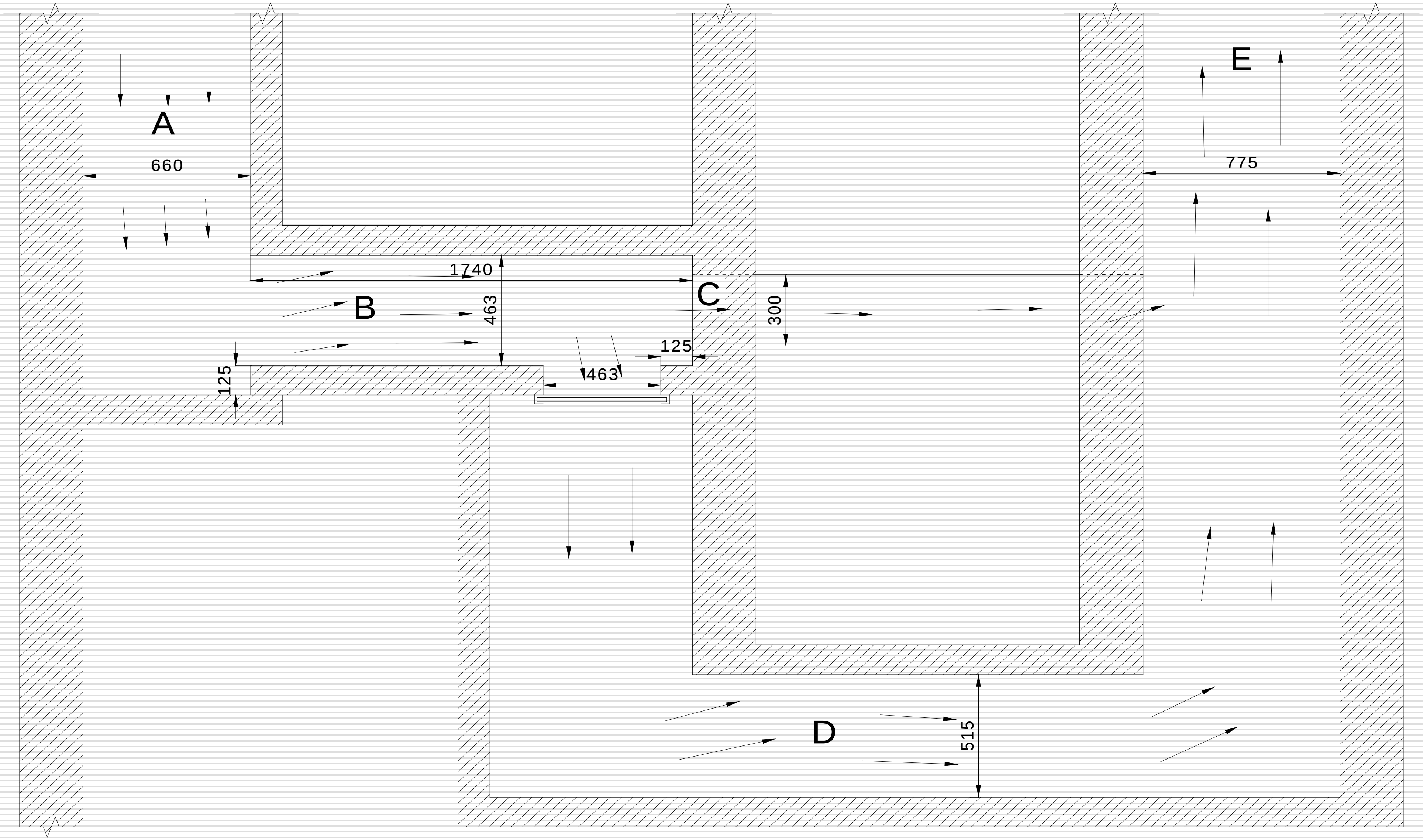


Figure : Different location of the model at which velocity was measured.

Model Results and Discussions

Model Observations

Run No.	Overflow gate height	Bypass gate opening	Upstream weir reading	Upstream weir head	Overflow weir reading	Overflow weir initial gaze reading	Overflow weir head
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
	1	2	3	4	5	6	7
1	201	76.2	155	63.4	803	900	97
2	201	76.2	196	104.4	740	900	160
3	201	76.2	172	80.4	778	900	122
4	412	76.2	194	102.4	545	690	145
5	412	76.2	164	72.4	587	690	103
6	412	76.2	176	84.4	569	690	121
7	612	76.2	172.6	81	385	490	105
8	736.8	76.2	167.4	75.8	242	151.8	90.2
9	882	76.2	196	104.4	102	220	118
10	692	76.2	208	116.4	26	172	146
11	484	76.2	196	104.4	475	618	143
12	484	76.2	183	91.4	493	618	125
13	484	76.2	155.3	63.7	530	618	88

Model Results and Discussions

Model Observations

Run No.	Current meter reading at point “A”			Current meter reading at point “B”			Current meter reading at “C”	Current meter reading at point “D”			Current meter reading at point “E”		
	(n)			(n)			(n)	(n)			(n)		
	0.6d	0.2d	0.8d	0.6d	0.2d	0.8d	Center	0.6d	0.2d	0.8d	0.6d	0.2d	0.8d
	8	9	10	11	12	13	14	15	16	17	18	19	20
1	20.1	22	19.2	13.7	14.6	13.2	3.62	14.8	16.2	14.5	11	11.9	10.6
2	10.8	14.5	13.1	9.3	11.2	7.6	1.85	7.7	8.5	7.2	5.46	5.8	5.4
3	14.6	13.8	15.1	10.2	9.8	10.8	2.76	10.2	9.8	10.9	7.6	7	8.1
4	20.4	19.8	21.5	12.5	12.1	12.8	1.52	5.3	5	5.5	2.7	2.5	3
5	26.8	28.4	25.5	20.5	20.5	19.5	2.55	7.4	8.1	7	9.5	10.2	9
6	23.3	24.7	22	16.5	17	16	2.07	6.4	6.8	6.3	6	6.4	6
7	36.2	38	34.5	24.5	26	23.5	1.83	9	9.5	8.5	6.8	7.2	6.5
8	52.5	55.9	50	35	37	33.5	1.78	15	16	14.2	10.2	11	9.7
9	35	37	33.5	23.8	25.5	22.5	1.07	13.5	14.5	12.9	8.2	9.1	7.8
10	30.5	32	29	21	22.5	20	1.05	10.4	11	10	6.1	6.5	5.8
11	19.5	20.7	19	13.6	14.5	13	1.47	10.6	11.5	10	7.6	8	7.3
12	25.3	25	25.8	17.6	16.8	18.1	1.75	5.2	4.9	5.5	6.2	6	6.5
13	52.4	55.5	50	34.5	37	33	3.11	9	9.5	8.5	11	12	10.5

Model Results and Discussions

Model Observations

Run No.	Depth of water at point "A"	Depth of water at point "B"	Area of bypass pipe at point "C"	Depth of water at point "D"	Depth of water at point "E"	Upstream weir discharge	Overflow weir discharge
	(mm)	(mm)	(mm^2)	(mm)	(mm)	m^3/s	m^3/s
	21	22	23	24	25	26	27
1	286	299	14264	182	148	**0.043977**	**0.028574**
2	348	362	14264	213	133	**0.092775**	**0.060865**
3	309	322	14264	193	142	**0.062689**	**0.040363**
4	610	562	14264	130	77	**0.090114**	**0.052418**
5	450	515	14264	107	154	**0.053599**	**0.031273**
6	511	533	14264	116	125	**0.067416**	**0.039865**
7	689	717	14264	134	132	**0.063390**	**0.032191**
8	827	860	14264	165	175	**0.057402**	**0.025621**
9	973	1012	14264	232	232	**0.092775**	**0.038382**
10	1030	1075	14264	245	205	**0.109301**	**0.052967**
11	595	620	14264	248	215	**0.092775**	**0.051325**
12	611	636	14264	102	145	**0.075970**	**0.041872**
13	644	670	14264	102	145	**0.044287**	**0.024689**

Model Results and Discussions

Model Observations

Run No.	Discharge through point "A"		Discharge through point "B"		Discharge through point "C"	Discharge through point "D"		Discharge through point "E"	
	m^3/s		m^3/s		m^3/s	m^3/s		m^3/s	
	at 0.6d	0.2d&0.8 d	at 0.6d	0.2d&0.8 d		at 0.6d	0.2d&0.8 d	at 0.6d	0.2d&0.8 d
	28	29	30	31	32	33	34	35	36
1	0.043187	0.042273	0.044358	0.043778	0.016184	0.028012	0.027155	0.045056	0.044129
2	0.092048	0.073490	0.076819	0.076053	0.031272	0.060194	0.059105	0.07854	0.076651
3	0.062003	0.062586	0.062682	0.062115	0.021098	0.04188	0.041315	0.061141	0.061525
4	0.090931	0.089972	0.090657	0.090991	0.037972	0.052495	0.052977	0.090182	0.088574
5	0.053124	0.052876	0.053349	0.054510	0.022801	0.031399	0.030807	0.053739	0.053215
6	0.067913	0.067789	0.066867	0.066867	0.027993	0.039089	0.038233	0.067425	0.065341
7	0.063652	0.063583	0.063716	0.063169	0.031609	0.032688	0.032688	0.063173	0.062734
8	0.057610	0.057255	0.056953	0.056631	0.032486	0.025136	0.024985	0.057146	0.056375
9	0.092331	0.091809	0.092177	0.091522	0.053767	0.038884	0.038367	0.092965	0.090369
10	0.109243	0.109243	0.109050	0.107937	0.054783	0.052212	0.05175	0.10884	0.107992
11	0.092261	0.090836	0.092596	0.091676	0.039249	0.051924	0.051251	0.092573	0.092
12	0.075712	0.075460	0.075335	0.075909	0.033036	0.041951	0.041951	0.075795	0.075215
13	0.044924	0.044708	0.044883	0.044370	0.018770	0.024882	0.024882	0.044143	0.043234

Model Results and Discussions

Model Observations

Run No.	Ce for Upstream Weir	Ce for Upstream Weir	Upstream weir discharge	Overflow weir discharge	Bypass pipe discharge
			L/s	L/s	L/s
	37	38	39	40	41
1	0.6076	0.61137	**43.98**	**28.57**	**16.18**
2	0.6122	0.61844	**92.77**	**60.87**	**31.27**
3	0.60951	0.61418	**62.69**	**40.36**	**21.1**
4	0.61198	0.61676	**90.11**	**52.42**	**37.97**
5	0.60861	0.61204	**53.6**	**31.27**	**22.8**
6	0.60996	0.61406	**67.42**	**39.86**	**27.99**
7	0.60958	0.61227	**63.39**	**32.19**	**31.61**
8	0.60899	0.61061	**57.4**	**25.62**	**32.49**
9	0.6122	0.61373	**92.77**	**38.38**	**53.77**
10	0.61355	0.61687	**109.3**	**52.97**	**54.78**
11	0.6122	0.61653	**92.77**	**51.33**	**39.25**
12	0.61074	0.61451	**75.97**	**41.87**	**33.04**
13	0.60764	0.61036	**44.29**	**24.69**	**18.77**

Model Results and Discussions

Model Observations

Run No.	Velocity at point "A" (m/s)		Velocity at point "B" (m/s)		Velocity at point "C" (m/s)	Velocity at point "D" (m/s)		Velocity at point "E" (m/s)	
	at 0.6d	0.2d&0.8 d	at 0.6d	0.2d&0.8 d	center	at 0.6d	0.2d&0.8 d	at 0.6d	0.2d&0.8 d
	42	43	44	45	46	47	48	49	50
1	0.2281	0.2233	0.3211	0.3169	1.1345	0.2989	0.2897	0.3928	0.3847
2	0.3996	0.319	0.4593	0.4547	2.1922	0.5487	0.5388	0.762	0.7436
3	0.3031	0.306	0.4214	0.4175	1.479	0.4214	0.4157	0.5556	0.5591
4	0.2252	0.2228	0.3492	0.3504	2.6619	0.7841	0.7913	1.5112	1.4843
5	0.1783	0.1775	0.2242	0.2291	1.5984	0.5698	0.5591	0.4503	0.4459
6	0.2008	0.2004	0.2715	0.2715	1.9623	0.6543	0.64	0.696	0.6745
7	0.1396	0.1394	0.1923	0.1907	2.2159	0.4737	0.4737	0.6175	0.6132
8	0.1052	0.1046	0.1433	0.1425	2.2773	0.2958	0.294	0.4214	0.4157
9	0.1433	0.1425	0.1972	0.1958	3.7692	0.3254	0.3211	0.517	0.5026
10	0.1602	0.1602	0.2196	0.2173	3.8404	0.4138	0.4101	0.6851	0.6797
11	0.2342	0.2306	0.3233	0.3201	2.7514	0.4065	0.4013	0.5556	0.5521
12	0.1872	0.1866	0.2564	0.2583	2.3159	0.7986	0.7986	0.6745	0.6693
13	0.1054	0.1049	0.145	0.1433	1.3158	0.4737	0.4737	0.3928	0.3847

Model Results and Discussions

Model Observations

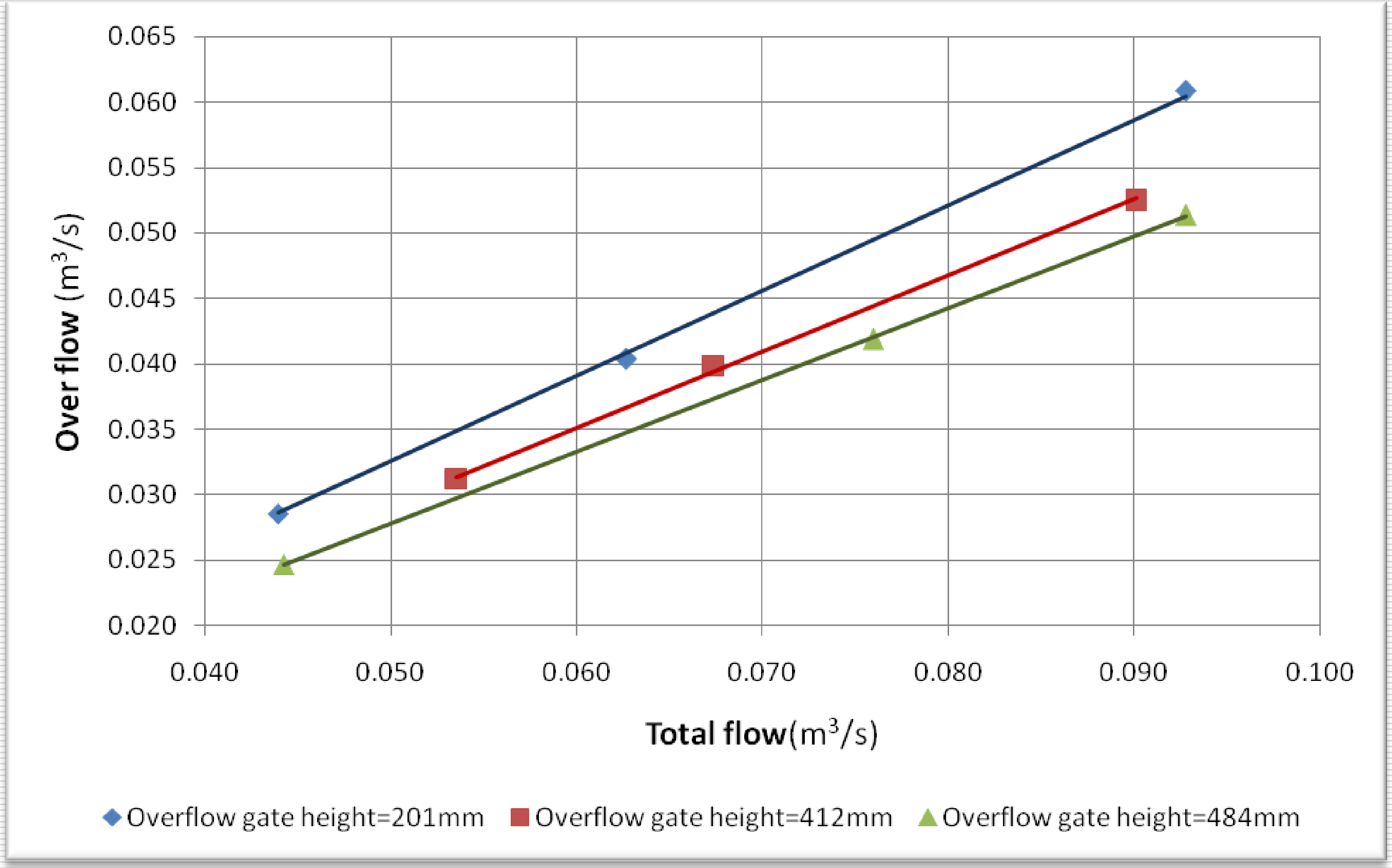


Figure: Relation between total flow versus overflow

Model Results and Discussions

Model Observations

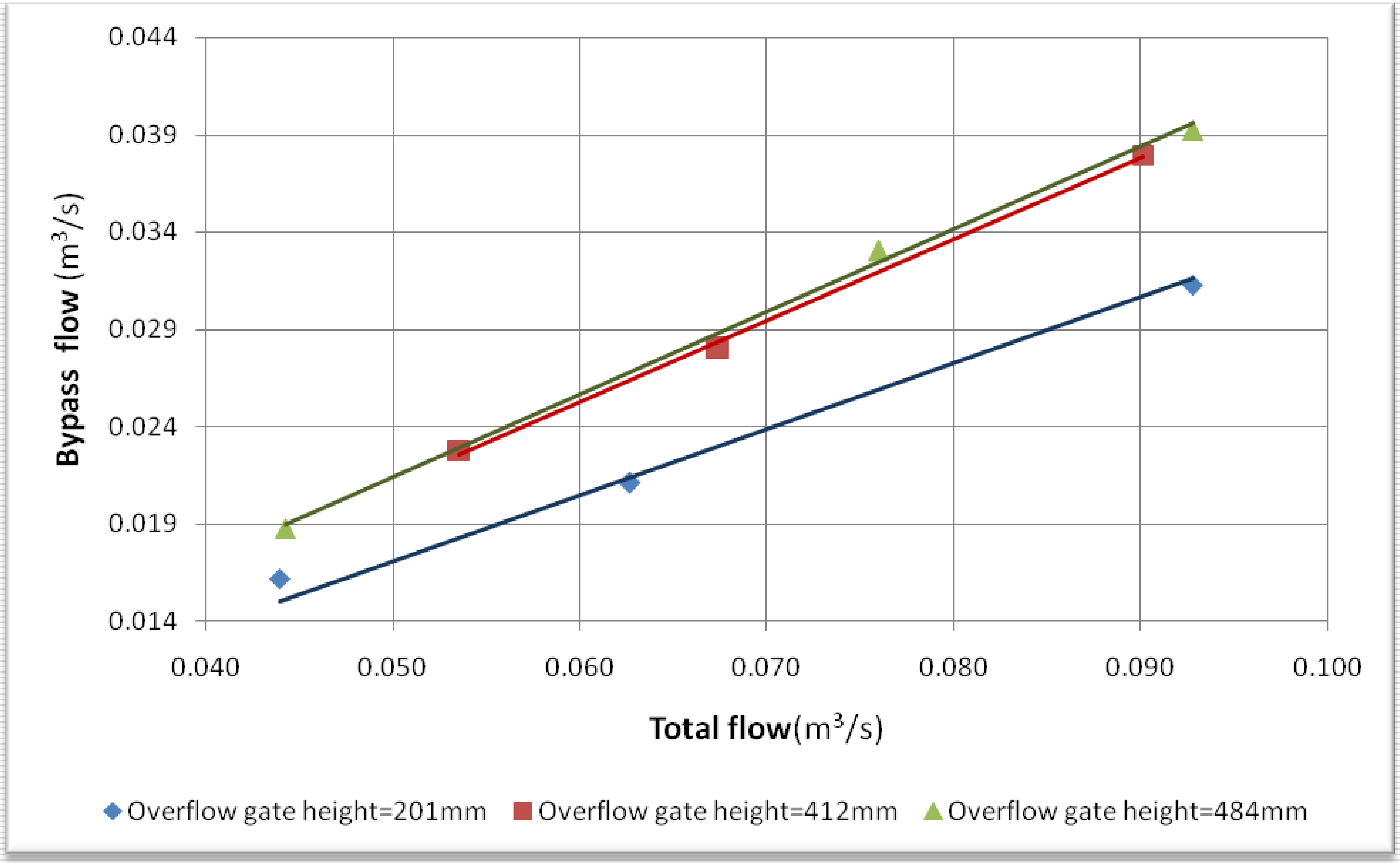


Figure: Relation between total flow versus bypass flow

Model Results and Discussions

Model Observations

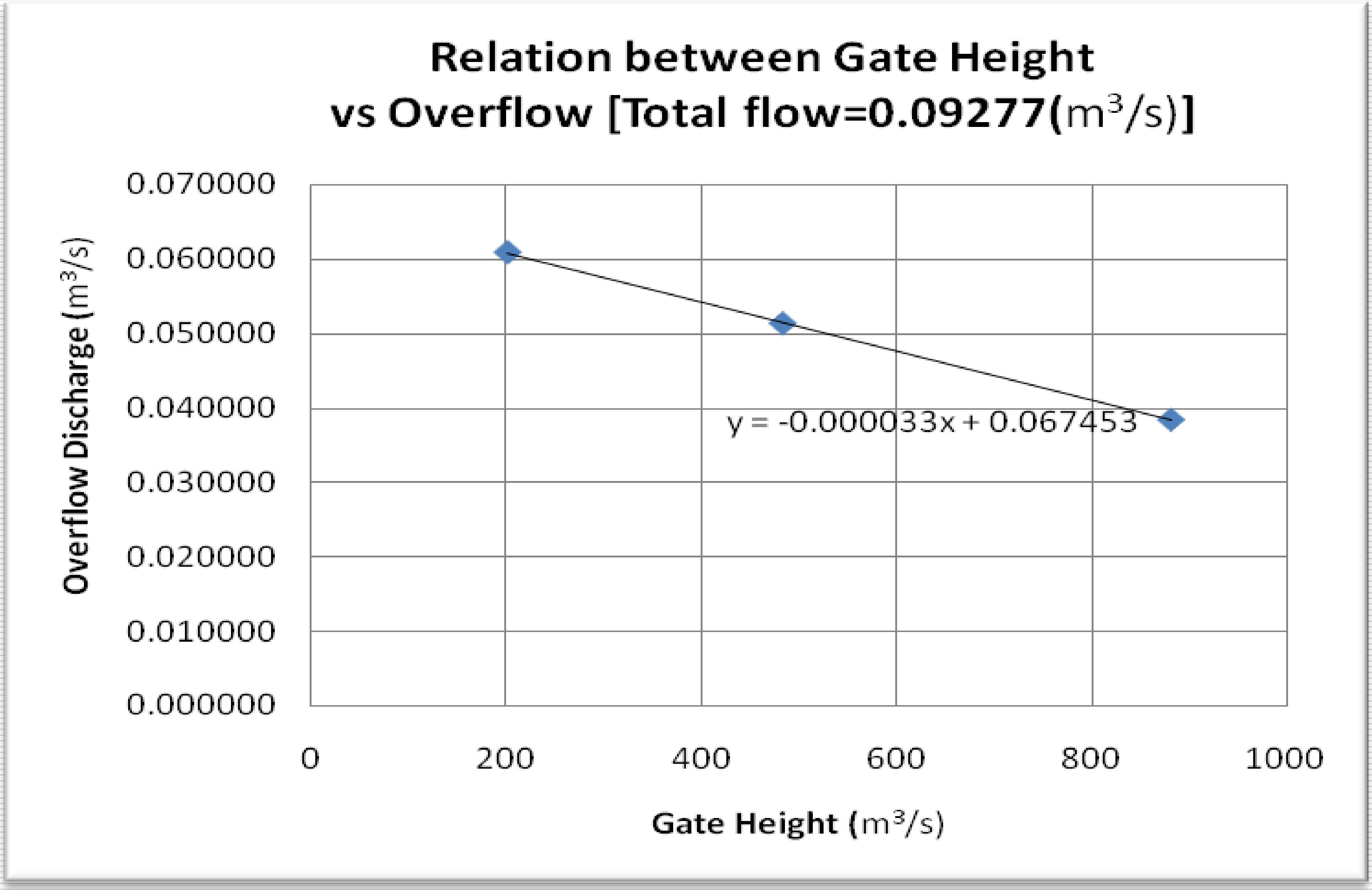


Figure: Relation between overflow gate height versus overflow

Model Results and Discussions

Model Observations

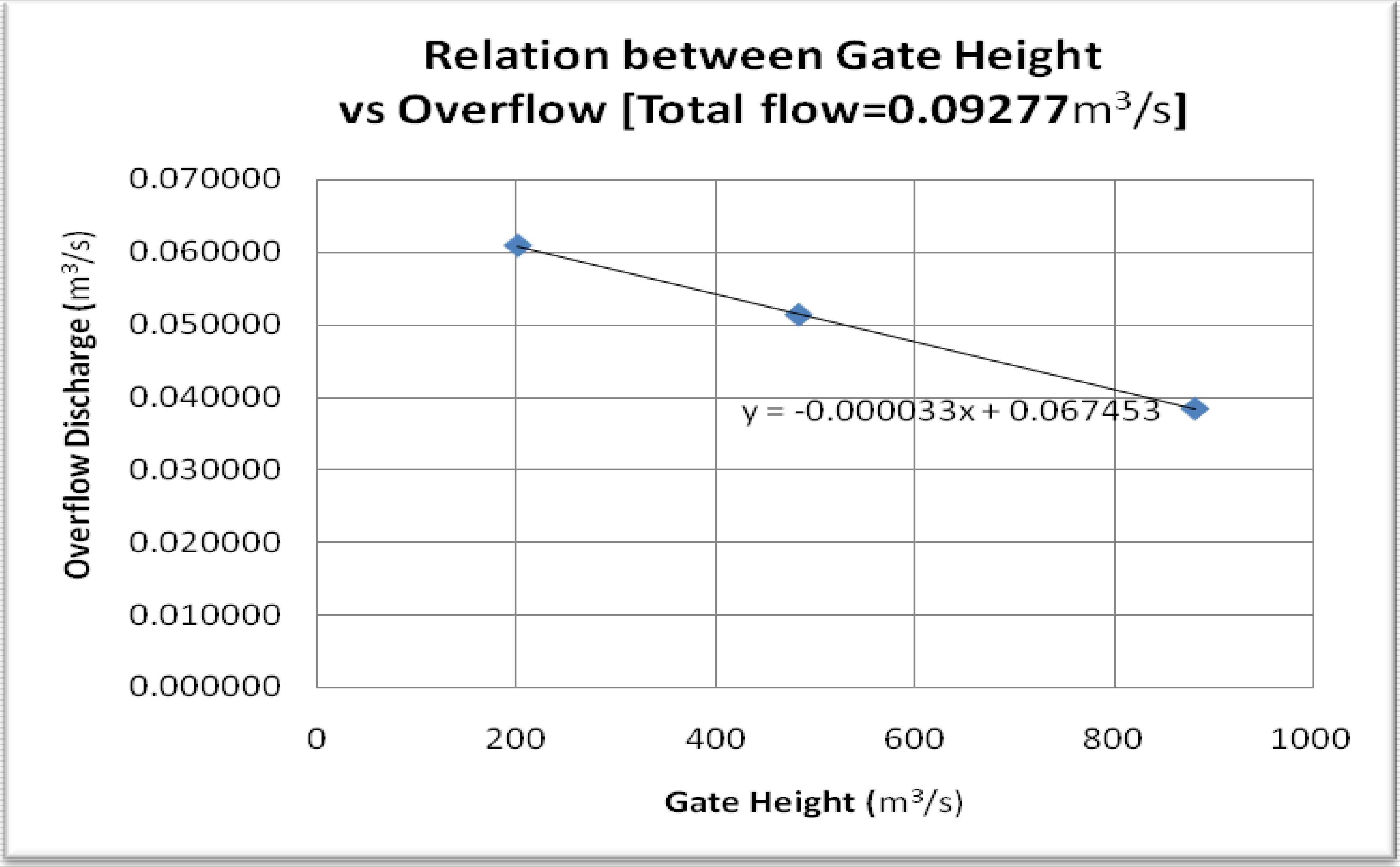


Figure: Relation between overflow gate height versus bypass flow

Model Results and Discussions

Model Observations

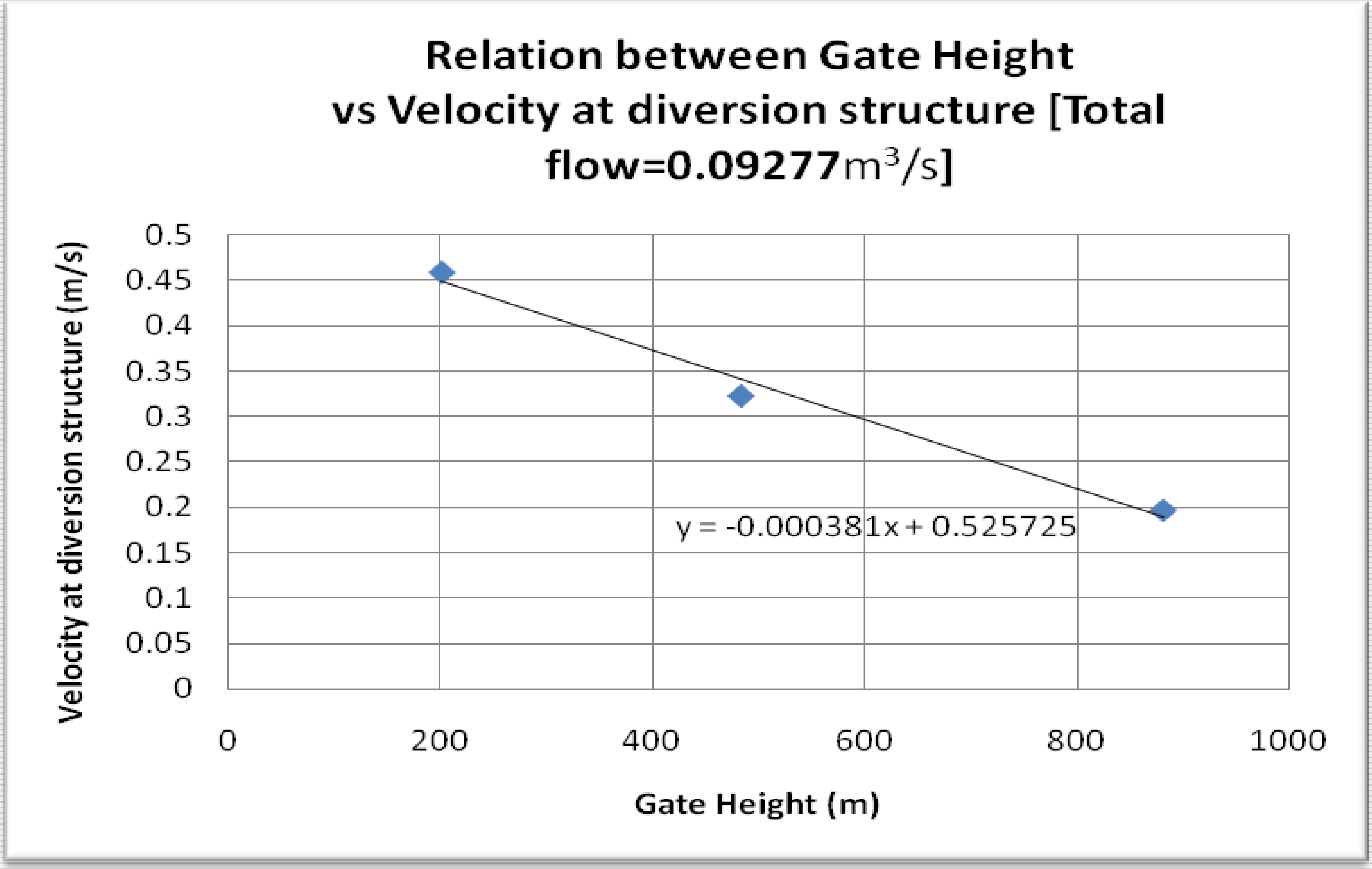


Figure: Relation between overflow gate height versus velocity at SDS chamber

Model Results and Discussions

Model Observations

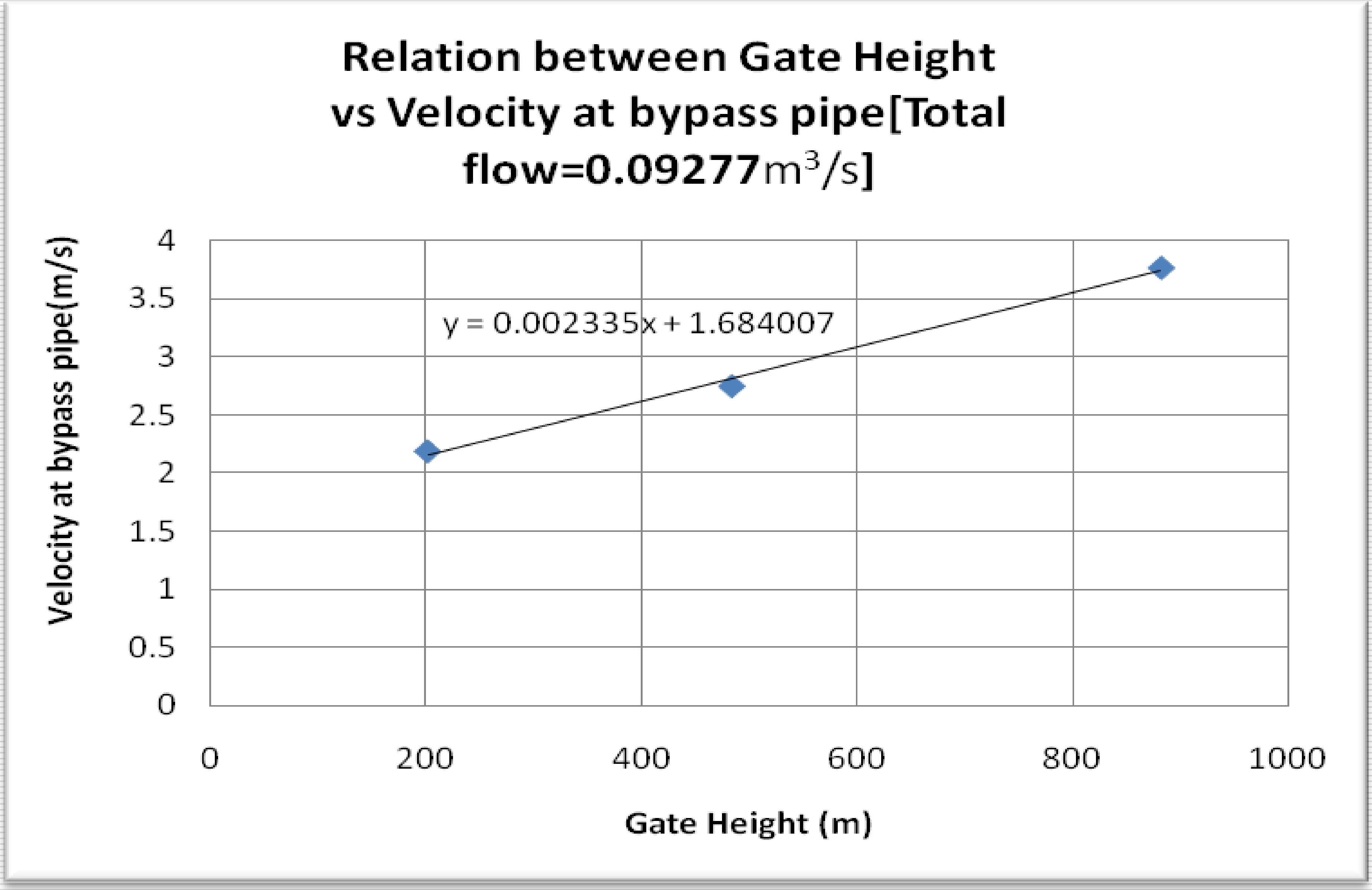


Figure: Relation between overflow gate height versus velocity at bypass pipe

Model Results and Discussions

Model Observations

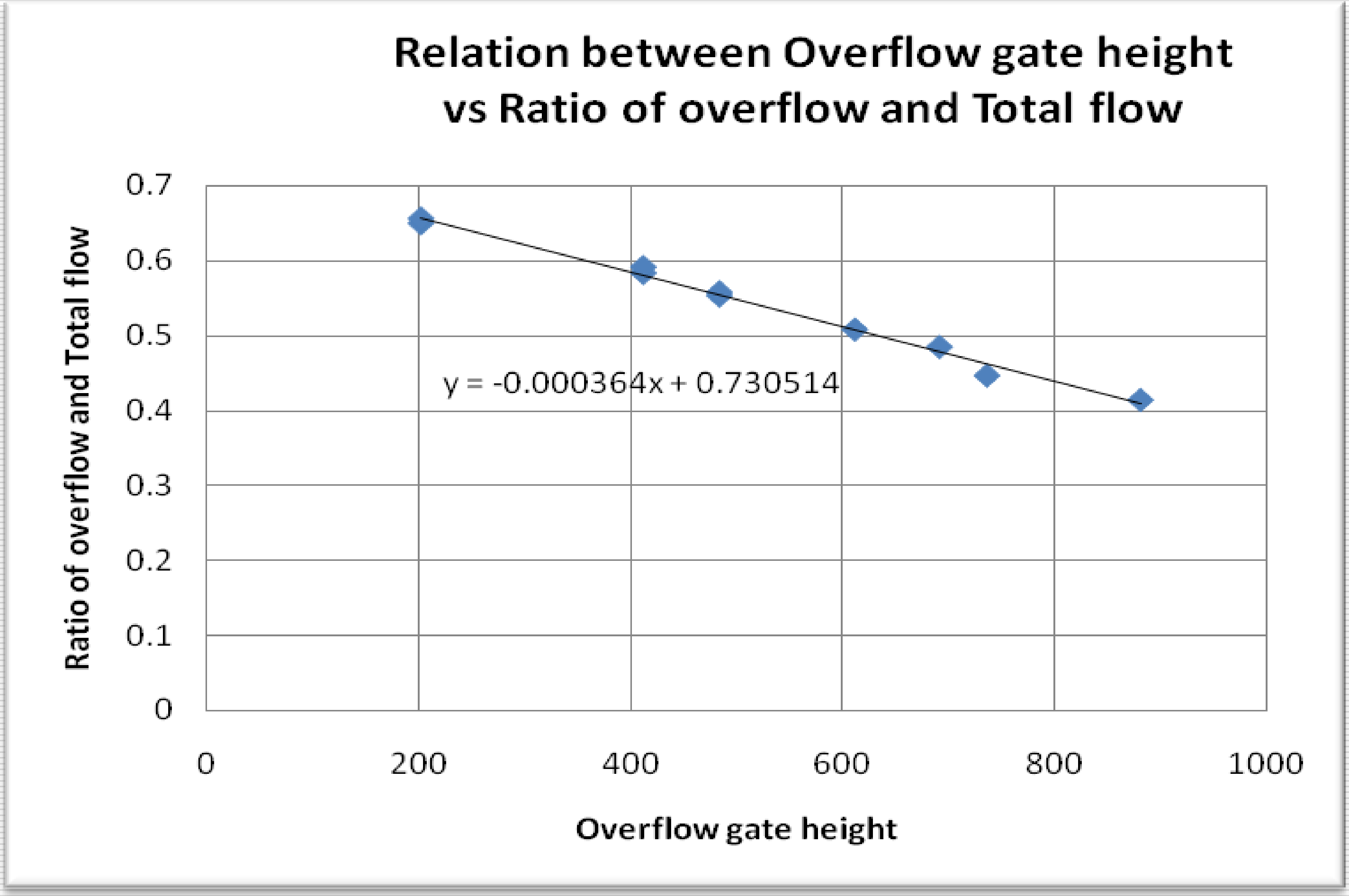


Figure: Relation between overflow gate height versus ratio of overflow and total flow

Model Results and Discussions

Model Observations

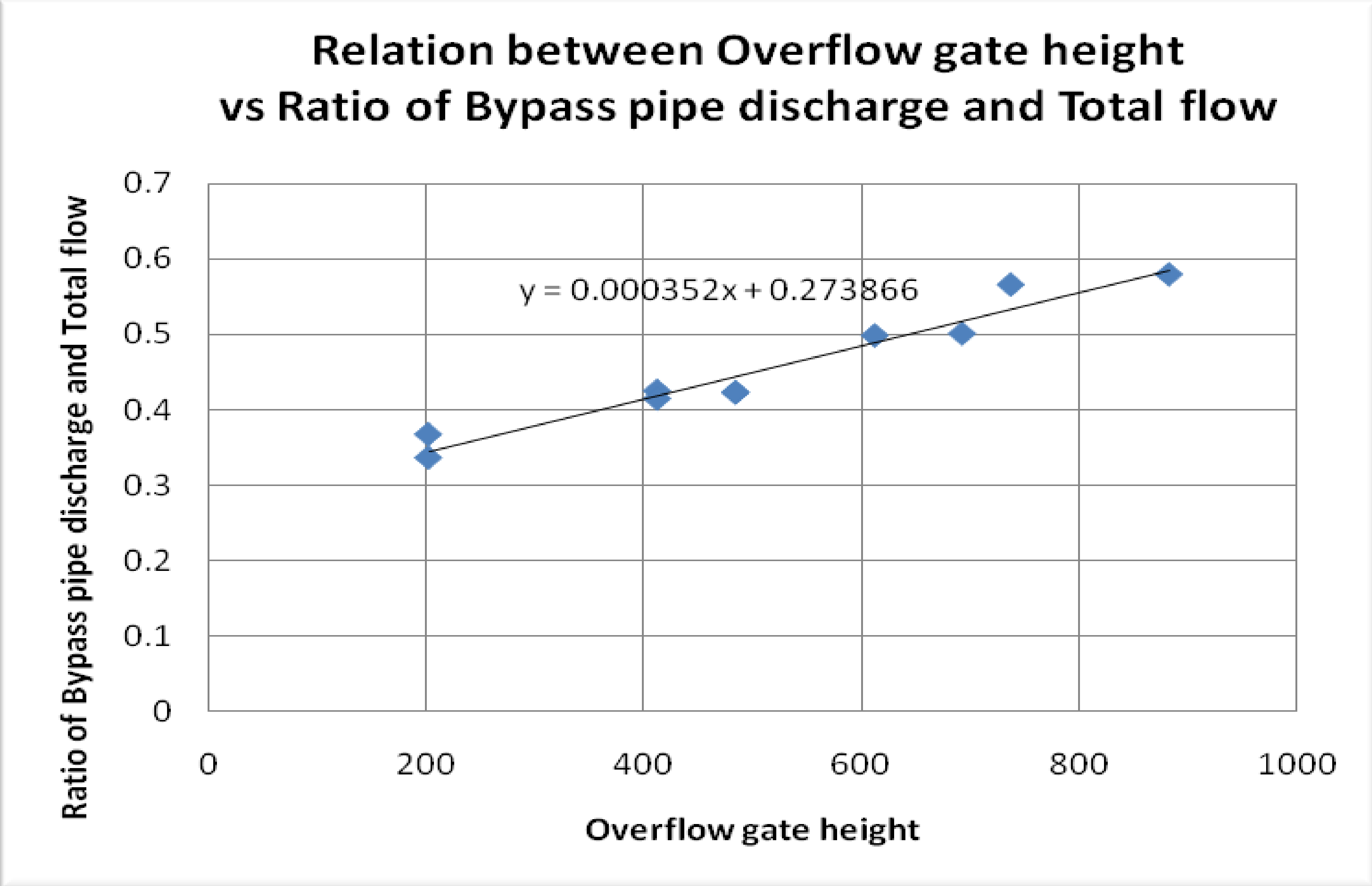


Figure: Relation between overflow gate height versus ratio of bypass pipe discharge

Model Results and Discussions

Model Observations

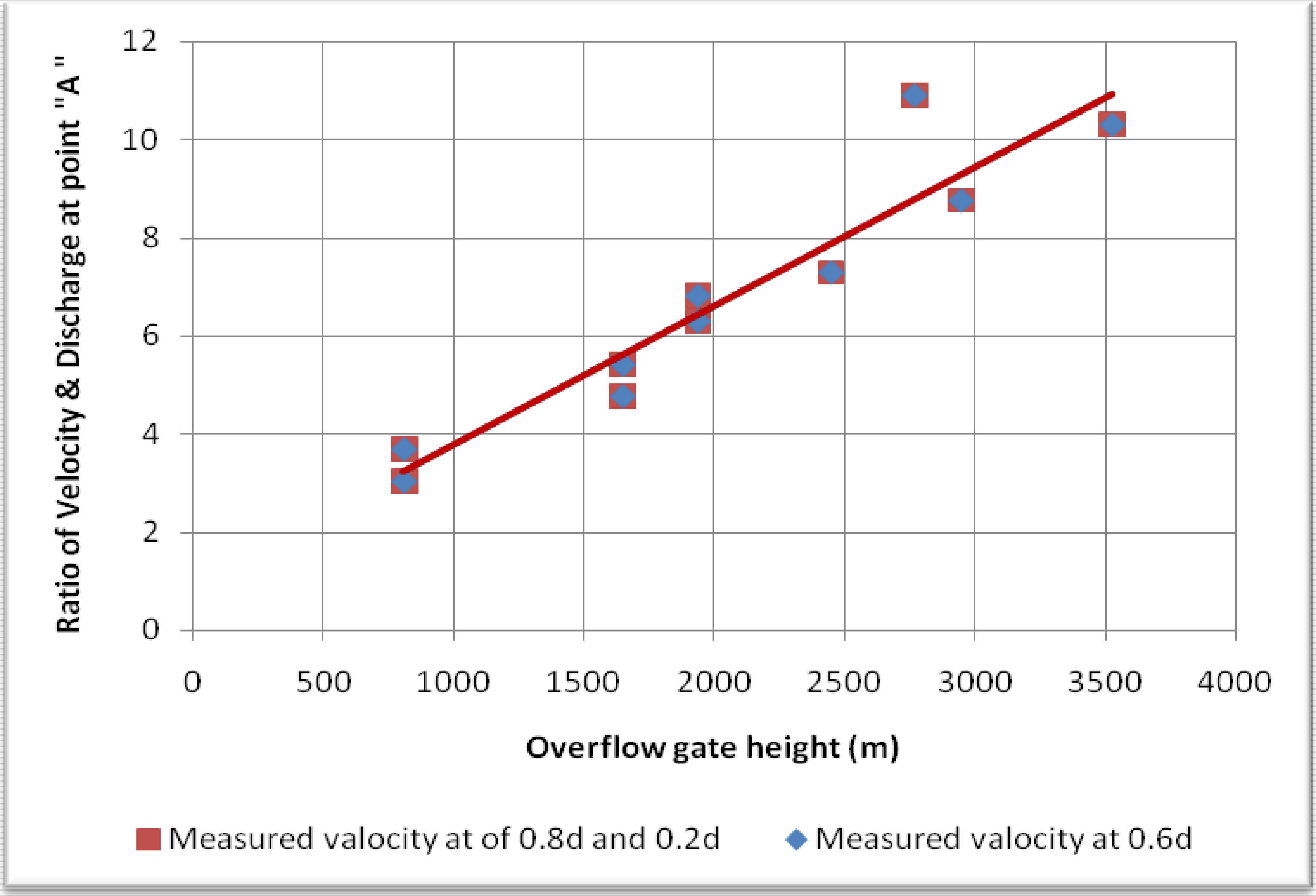


Figure: Relation between overflow gate height versus velocity & discharge

Model Results and Discussions

Model Observations

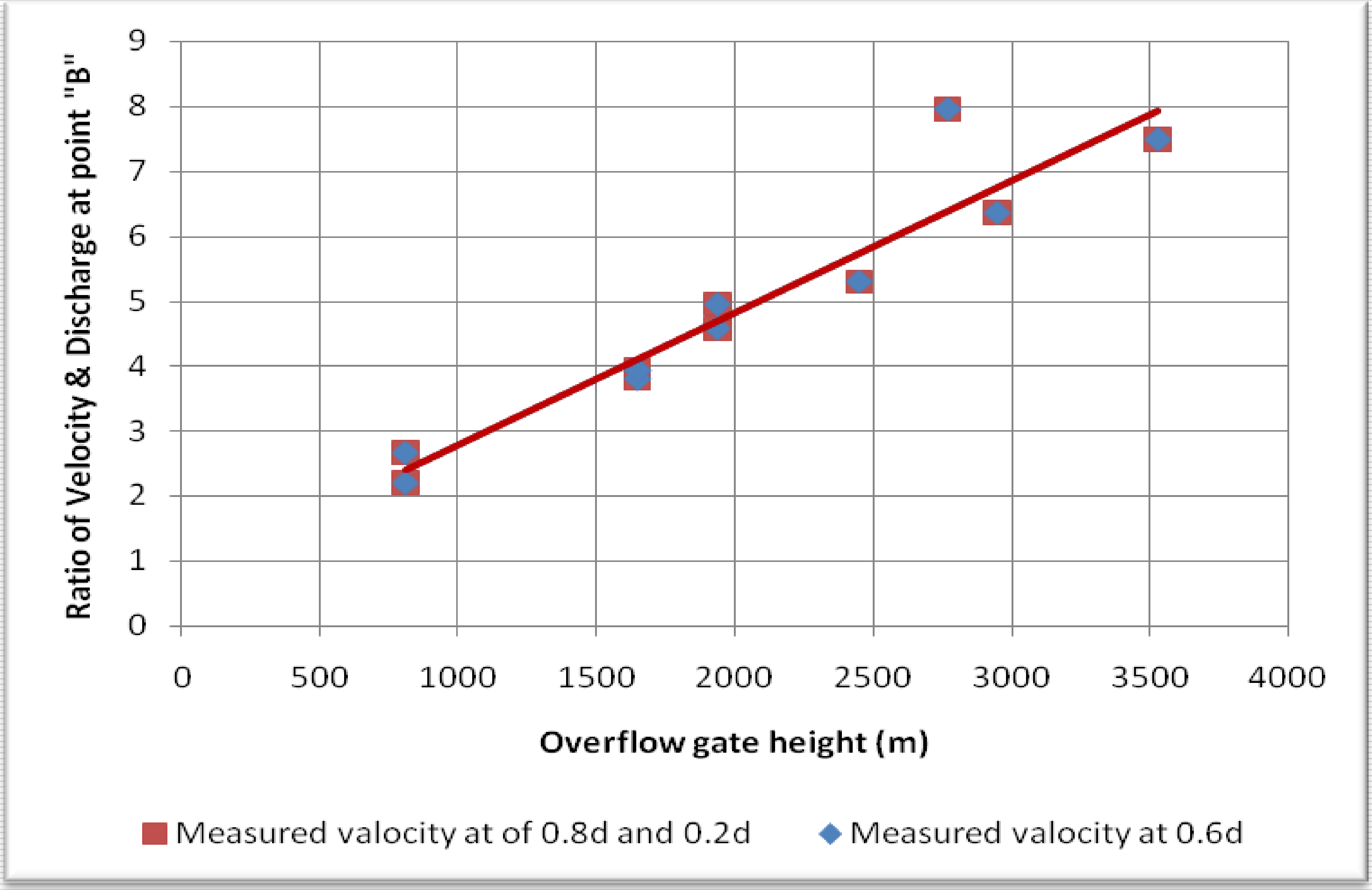


Figure: Relation between overflow gate height versus velocity & discharge

Model Results and Discussions

Model Observations

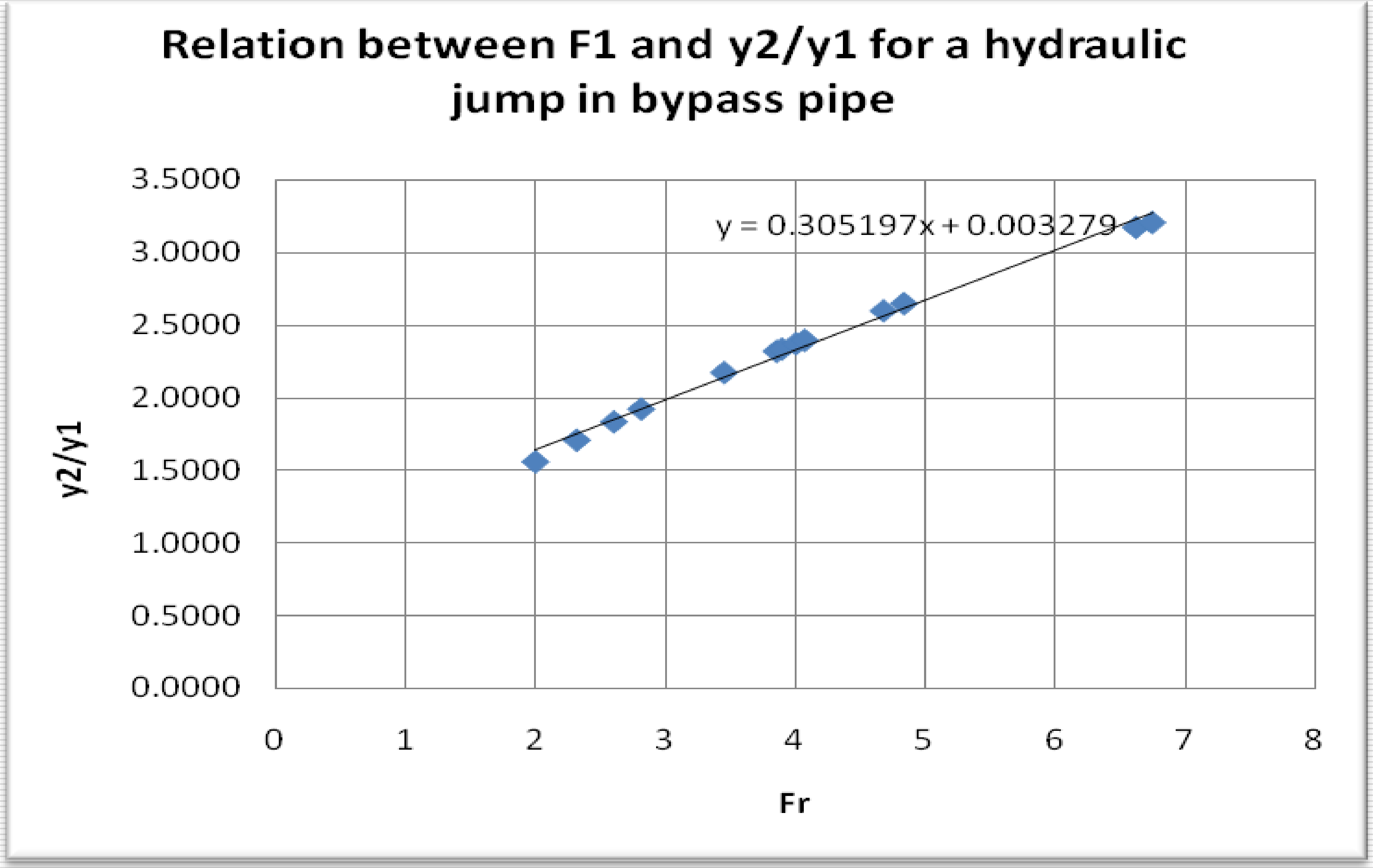


Figure: Relation between Froude Number and in bypass pipe

Model Results and Discussions

Prototype data table

Run No.	Overflow gate height (mm)	Upstream weir discharge m^3/s	Overflow weir discharge m^3/s	Discharge through point "A" m^3/s at 0.6d	Discharge through point "A" m^3/s 0.2d & 0.8 d	Discharge through point "B" m^3/s at 0.6d	Discharge through point "B" m^3/s 0.2d & 0.8 d	Discharge through point "C" m^3/s	Discharge through point "D" m^3/s at 0.6d	Discharge through point "D" m^3/s 0.2d & 0.8 d	Discharge through point "E" m^3/s at 0.6d	Discharge through point "E" m^3/s 0.2d & 0.8 d
1	804	1.407252	0.914381	1.381999	1.35272	1.41947	1.40089	0.517886	0.896384	0.868964	1.4418	1.412125
2	804	2.968792	1.947688	2.945536	2.351678	2.458206	2.433706	1.000711	1.92621	1.891349	2.513272	2.452832
3	804	2.006046	1.291617	1.984112	2.002737	2.005829	1.987695	0.675131	1.340172	1.322086	1.956527	1.968808
4	1648	2.883636	1.67736	2.909784	2.879094	2.901034	2.911717	1.215097	1.679844	1.695251	2.88583	2.834367
5	1648	1.715174	1.000728	1.699969	1.692045	1.707154	1.744312	0.72964	1.004782	0.985835	1.719645	1.702886
6	1648	2.157301	1.275664	2.173228	2.169247	2.139743	2.139743	0.895762	1.250836	1.223461	2.1576	2.0909
7	2448	2.028491	1.030111	2.036877	2.034651	2.038902	2.021412	1.011503	1.046008	1.046008	2.021544	2.007481
8	2947	1.836869	0.819856	1.843517	1.832167	1.822495	1.812185	1.039544	0.804339	0.799535	1.828672	1.803993
9	3528	2.968792	1.228228	2.954588	2.937872	2.94966	2.928696	1.720551	1.244291	1.227745	2.974892	2.891814
10	2768	3.49764	1.694931	3.495773	3.495773	3.489605	3.453973	1.753071	1.67079	1.655993	3.482873	3.455756
11	1936	2.968792	1.642415	2.952357	2.906745	2.96307	2.933646	1.255976	1.661575	1.640044	2.962347	2.943996
12	1936	2.431033	1.339892	2.422774	2.414713	2.410705	2.429083	1.057138	1.342441	1.342441	2.425444	2.406875
13	1936	1.417175	0.790051	1.437565	1.430652	1.436266	1.419851	0.600641	0.796215	0.796215	1.412574	1.383501

Model Results and Discussions

Prototype data table

Run No.	Overflow gate height	Upstream weir discharge	Overflow weir discharge	Velocity at point "A"		Velocity at point "B"		Velocity at point "C"	Velocity at point "D"		Velocity at point "E"	
	(mm)	m^3/s	m^3/s	(m/s)		(m/s)		(m/s)	(m/s)		(m/s)	
				at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d	center	at 0.6d	0.2d & 0.8 d	at 0.6d	0.2d & 0.8 d
1	804	1.407252	0.914381	0.45621	0.44654	0.6422	0.6338	2.269	0.5977	0.5794	0.7856	0.7695
2	804	2.968792	1.947688	0.79911	0.638	0.9186	0.9095	4.3845	1.0975	1.0776	1.5239	1.4873
3	804	2.006046	1.291617	0.60622	0.61191	0.8427	0.8351	2.958	0.8427	0.8313	1.1112	1.1181
4	1648	2.883636	1.67736	0.45035	0.4456	0.6983	0.7009	5.3238	1.5682	1.5826	3.0224	2.9685
5	1648	1.715174	1.000728	0.35666	0.35499	0.4484	0.4582	3.1968	1.1396	1.1181	0.9005	0.8918
6	1648	2.157301	1.275664	0.40152	0.40078	0.5431	0.5431	3.9247	1.3086	1.28	1.392	1.349
7	2448	2.028491	1.030111	0.2791	0.2788	0.3847	0.3814	4.4318	0.9473	0.9473	1.2351	1.2265
8	2947	1.836869	0.819856	0.21046	0.20916	0.2867	0.2851	4.5546	0.5916	0.5881	0.8427	0.8313
9	3528	2.968792	1.228228	0.28669	0.28506	0.3943	0.3915	7.5384	0.6509	0.6422	1.0341	1.0052
10	2768	3.49764	1.694931	0.32043	0.32043	0.4391	0.4347	7.6809	0.8276	0.8203	1.3701	1.3595
11	1936	2.968792	1.642415	0.46846	0.46122	0.6465	0.6401	5.5029	0.8131	0.8026	1.1112	1.1043
12	1936	2.431033	1.339892	0.37436	0.37312	0.5128	0.5167	4.6317	1.5972	1.5972	1.349	1.3386
13	1936	1.417175	0.790051	0.21075	0.20973	0.29	0.2867	2.6316	0.9473	0.9473	0.7856	0.7695

Model Results and Discussions

Prototype data table

Upstream weir discharge	Overflow gate height	Froude number at point "A"		Froude number at point "B"		Froude number at point "C"
		At 0.6d	avg. of .2d&.8d	At 0.6d	avg. of .2d&.8d	At center
0.043977	201	0.3676	0.3598	0.5175	0.5107	1.3122
0.044287	484	0.1698	0.169	0.2337	0.231	1.5219
0.053599	412	0.2874	0.286	0.3613	0.3692	1.8487
0.057402	736.8	0.1696	0.1685	0.231	0.2297	2.634
0.062689	201	0.4885	0.4931	0.679	0.6729	1.7106
0.063390	612	0.2249	0.2247	0.31	0.3073	2.5629
0.067416	412	0.3235	0.3229	0.4376	0.4376	2.2697
0.075970	484	0.3017	0.3007	0.4132	0.4163	2.6786
0.090114	412	0.3629	0.3591	0.5627	0.5648	3.0788
0.092775	201	0.6439	0.5141	0.7402	0.7329	2.5356
0.092775	882	0.231	0.2297	0.3177	0.3155	4.3595
0.092775	484	0.3775	0.3716	0.521	0.5158	3.1824
0.109301	692	0.2582	0.2582	0.3539	0.3502	**4.4419**

Model Results and Discussions

Prototype data table

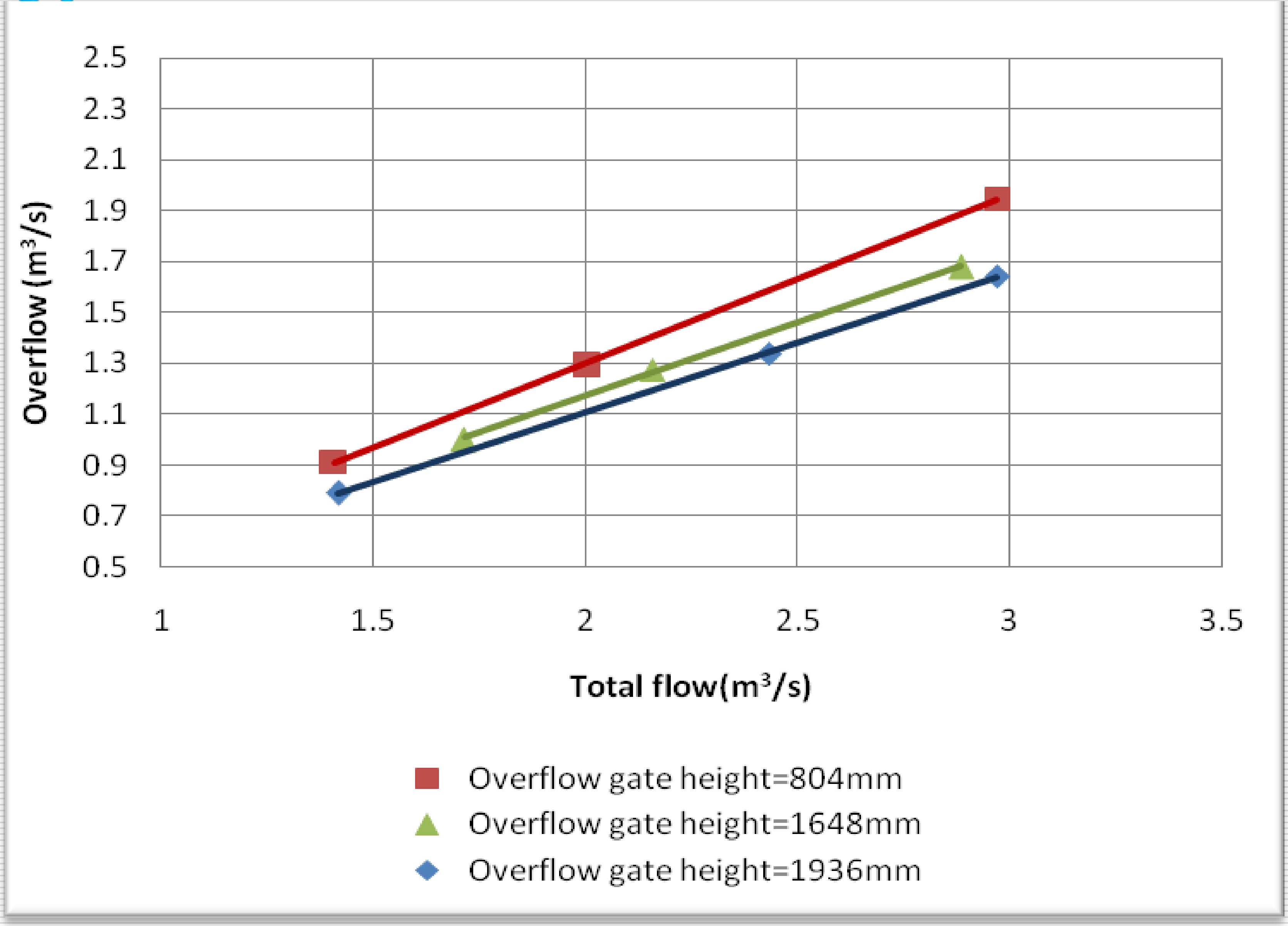


Figure: Relation between total flow versus overflow of prototype

Model Results and Discussions

Prototype data table

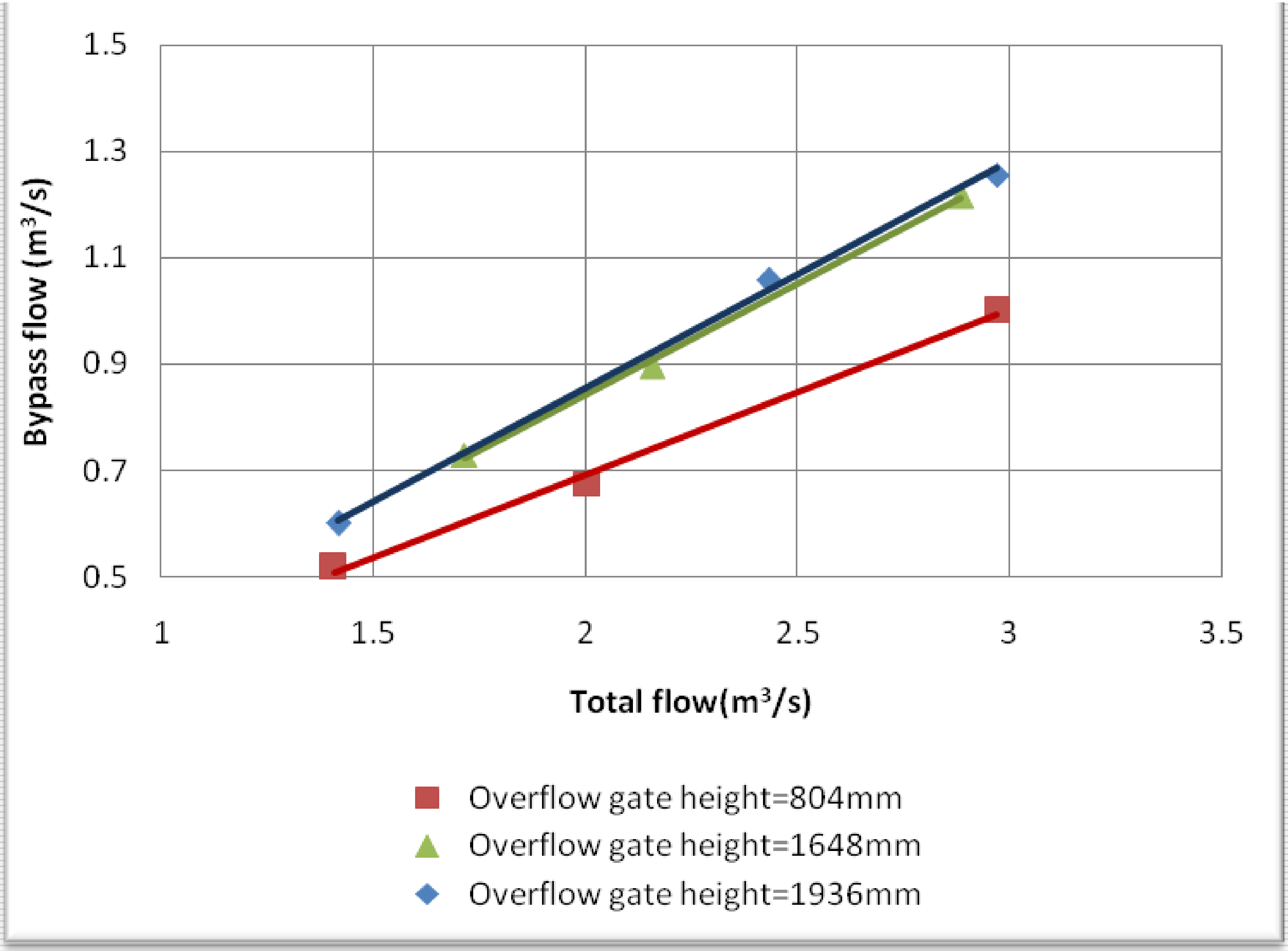


Figure: Relation between total flow versus bypass flow of prototype

Model Results and Discussions

Prototype data table

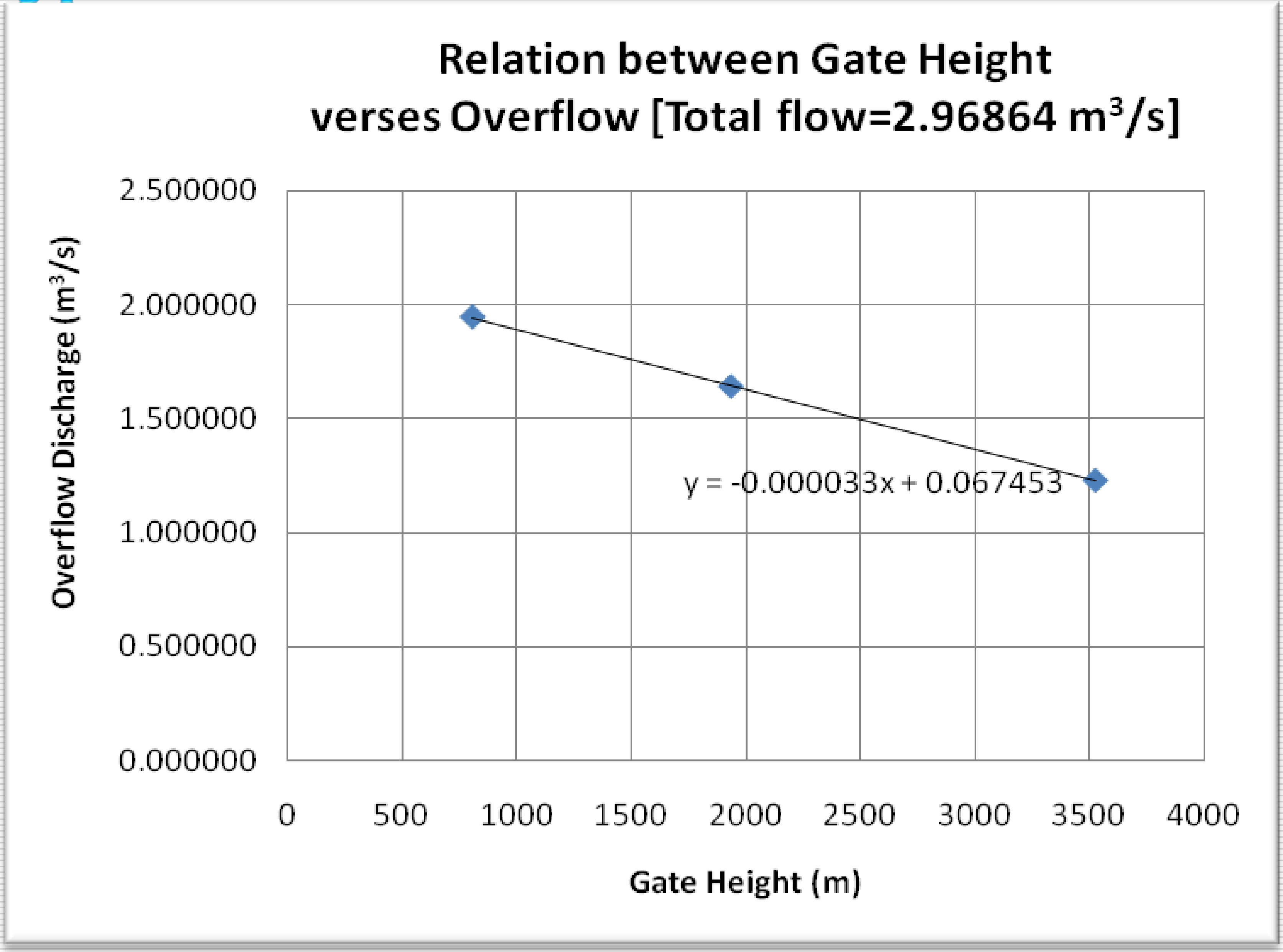


Figure: Relation between overflow gate height versus overflow of prototype

Model Results and Discussions

Prototype data table

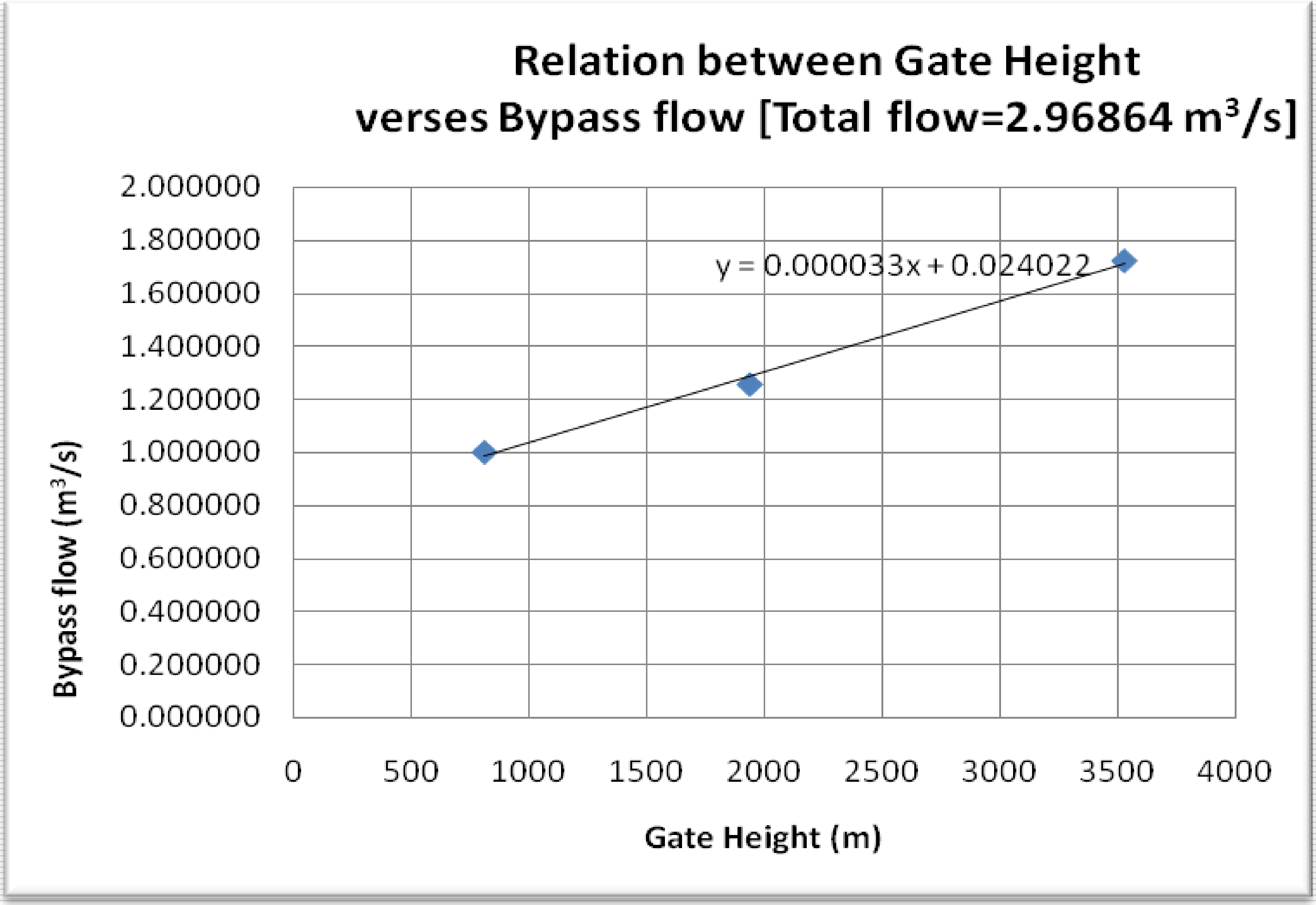


Figure: Relation between overflow gate height versus bypass flow of prototype

Model Results and Discussions

Prototype data table

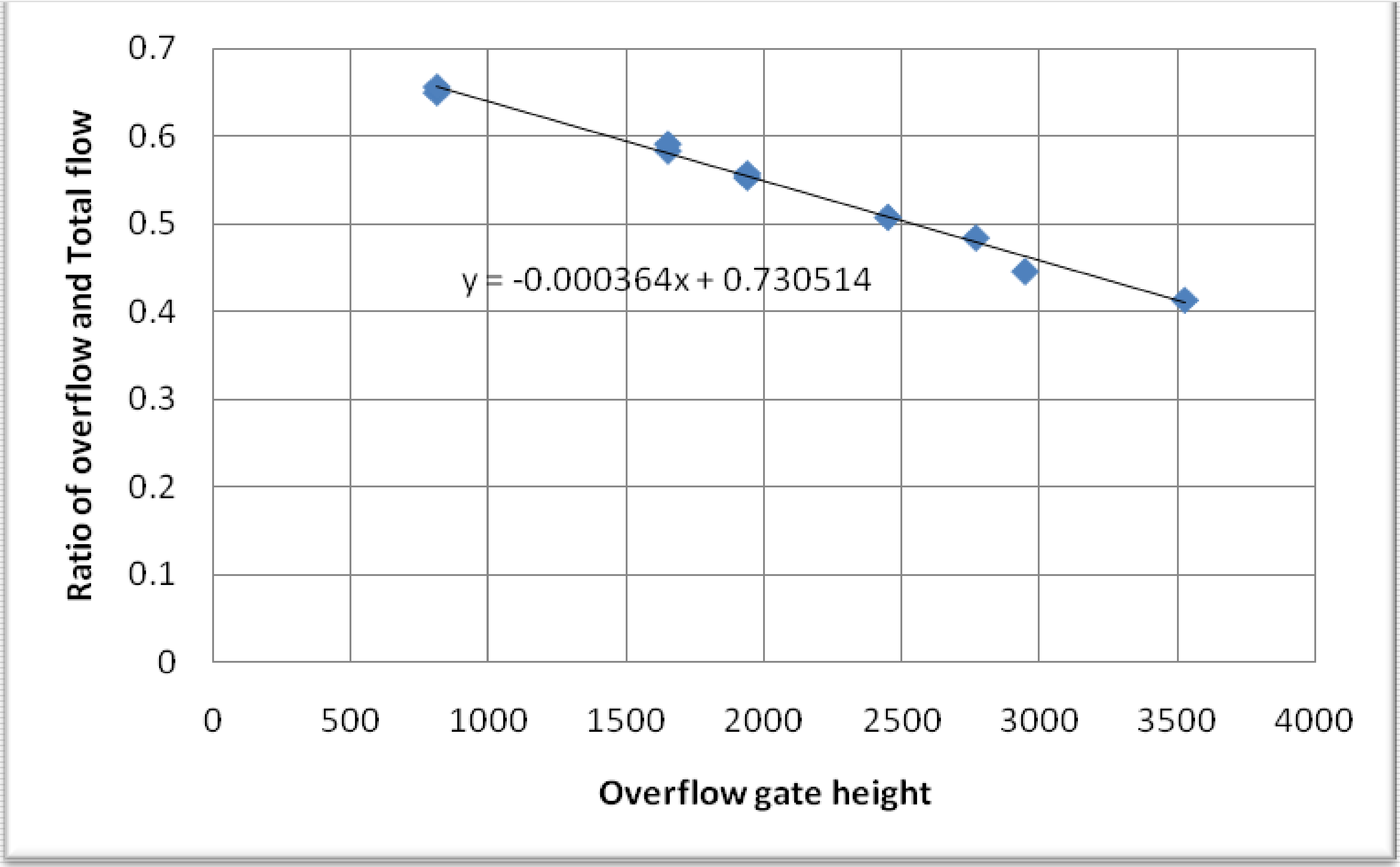


Figure: Relation between overflow gate height versus ratio of overflow and total flow of prototype

Model Results and Discussions

Prototype data table

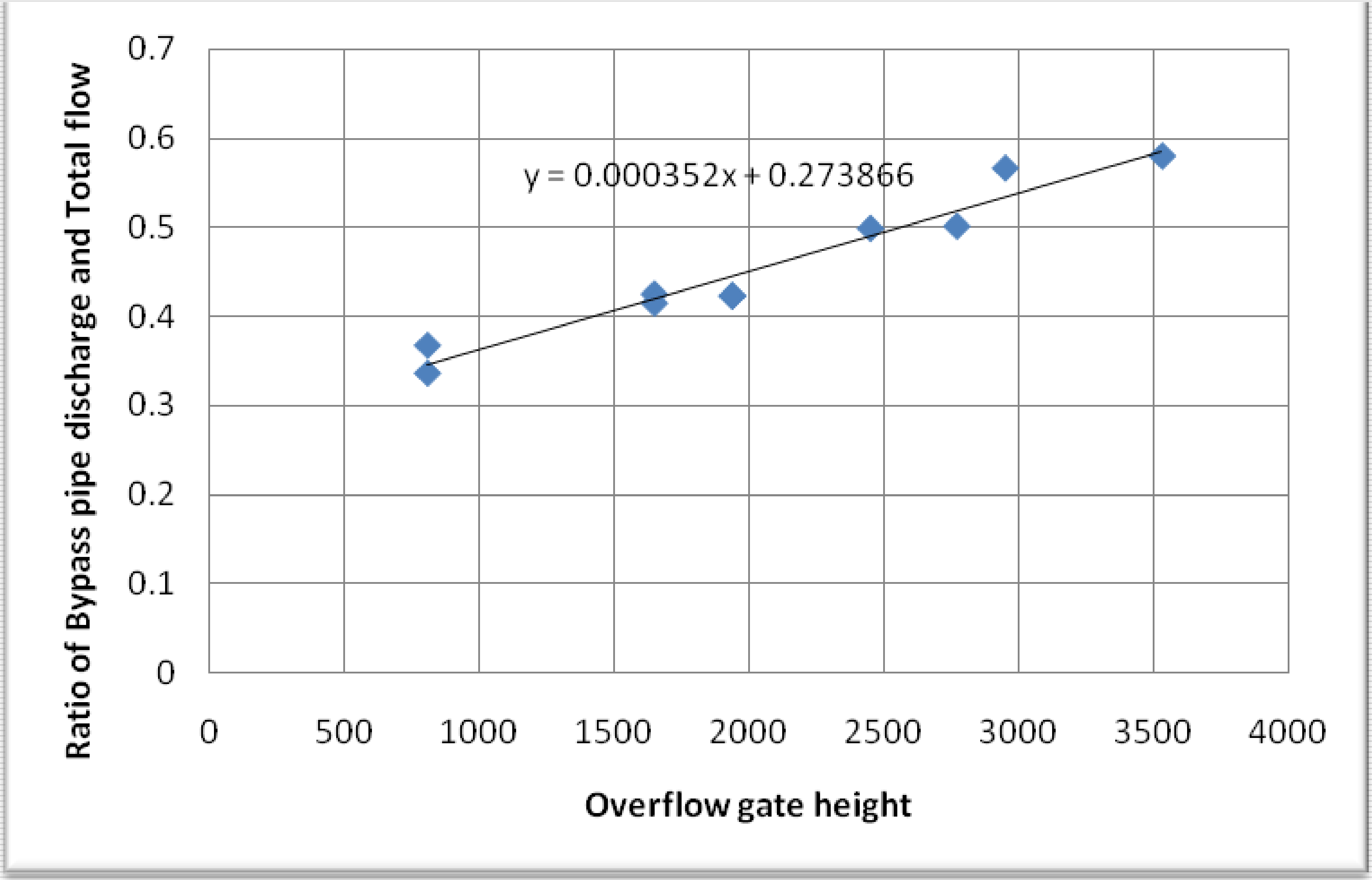


Figure: Relation between overflow gate height versus ratio of bypass pipe discharge of prototype

Model Results and Discussions

Prototype data table

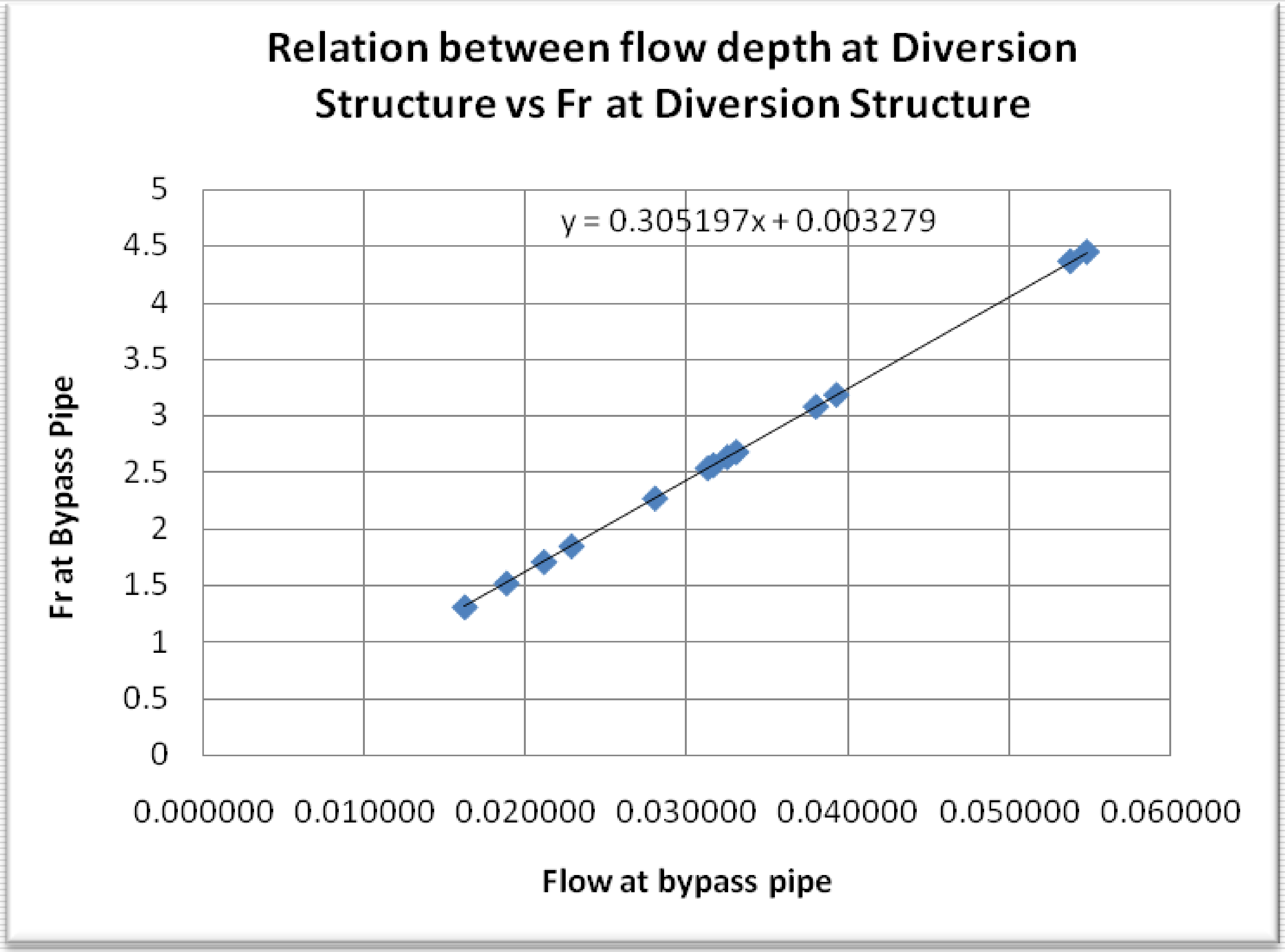


Figure: Relation between flow at bypass pipe versus F_r at bypass pipe

Model Results and Discussions

Some empirical relationship

From the graphical relationship of both model and prototype as discussed in the previous article, the following empirical relationship has been developed. This relationship can be used to predict overflow rate for given overflow gate height and dry weather flow is known. Froude number at different part of the Storm Diversion Structure (SDS) has also calculated and drawn relationship with discharge at that section.

Model Results and Discussions

Some empirical relationship

$$\frac{Q_O}{Q_T} = -0.000364 \times H_O + 0.73051 \qquad (5.1)$$

$$and \quad \frac{Q_B}{Q_T} = 0.000352 \times H_O + 0.273866 \qquad (5.2)$$

$$F_r = 0.305197 \times Q_B + 0.003297 \qquad (5.3)$$

Model Results and Discussions

Model data consistency

The continuity of inflow and outflow discharge rate has been assessed for the model run. It is found that, these two discharges are very close. The error is estimated and found only about 2% due to little leakage and instrumental measurement.

Model Results and Discussions

Model data consistency

The continuity of inflow and outflow discharge rate has been assessed for the model run. It is found that, these two discharges are very close. The error is estimated and found only about 2% due to little leakage and instrumental measurement.

Conclusions and recommendations

Conclusion

Based on the detail experimental investigation, analysis and discussion presented in forgoing chapters, following conclusion may be drawn

1. The ratio of overflow discharge and total inflow flow of SDS is inversely proportional to overflow gate height.

2. The ratio of bypass flow discharge and total inflow flow of SDS is directly proportional to overflow gate height

3. The overflow gate needs to be controlled depending on the storm water to be disposed to the lake. Equation 5.1 can be used to estimate the overflow discharge. The amount of flow to be disposed at hatirjhil can be calculated by deducting the known dry weather flow from the incoming flow of SDS.

Conclusions and recommendations

Conclusion

4. Equation 5.2 can be used to calculate bypass flow rate for respective overflow gate height.

5. The Froude number at different locations of SDS was evaluated. It is found that the flow is always subcritical in storm diversion chamber and that is supercritical at bypass pipe for flow rate grater then dry weather flow.

6. A hydraulic jump was observed in bypass pipe. The normal depth of the flow through bypass pipe can be calculated from the Froude number and depth of water at storm diversion structure. Froude number in the bypass pipe depends on combined sewer flow rate and overflow gate height. This Froude number can be calculated at different locations of SDS by using the equation 5.3

Conclusions and recommendations

Future recommendation

It is to be noted that the proposed SDS does not include the pollutant treatment facilities, thus only hydraulic condition of the diversion flow chamber and outlets are studied in the model. The model is run discharge 0.116 m^3/s which is commensurate with outfall for Tejgaon (Q4). Similar model study is necessary for other outfalls to investigate the flow behavior , especially culvert outfall (Q1) at panthapath.

THANK YOU