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**CE 365K Hydraulic Engineering Design** **Second Exam** **Spring 2015**

There are five questions on this exam. Please do all five questions.

**1. Hydraulic Engineering Design Methods**

**(a) Design Programs.** In the first part of the semester you used FlowMaster and Culvert Master to specify the size and performance of hydraulic design components. In the second part of the semester, you've used Pond Pack to design detention ponds. Describe and contrast these two types of design programs.

FlowMaster and CulvertMaster

(5) FlowMaster describes flow in open channels, pipes, wiers, orifices & street inlets
Culvert master describes flow in culverts and allows for determination of headwater elevation, discharge and size of culverts

Pond Pack

(5) Pond Pack describes flow in watersheds and detention ponds.
FlowMaster & Culvert Master have steady flows for single hydraulic components.
Pond Pack has unsteady flow through multiple components in sequence.

**(b) Hydrologic Modeling.** You have been shown how HEC-HMS can be used to do the same kinds of computations as Pond Pack for detention pond routing. How does HEC-HMS differ from Pond Pack? What are the advantages and limitations of HEC-HMS compared to Pond Pack?

(5) Pond Pack – enables design of individual elements of the outlet structure such as orifice or weir size

HEC-HMS – enables simulation of more complex watershed systems with many sub-basins & also direct linkage to GIS data support

**(b) GIS Application** You have seen various examples of how GIS can be used to provide input data for hydraulic design. Provide one example of this and describe what information is determined by GIS analysis of landscape data.

(5) Subwatershed BUT_060 in the Upper Brushy Creek watershed has land use, impervious cover, soils information to describe the character of the land surface, and length, slope and roughness information to describe flow along the longest flow path

**2. Risk and Hydraulic Design**

(a) At a particular location on a river, the annual maximum flood Q has a probability $P(Q \geq Q_T) = p$ of exceeding a value $Q_T$ in any year. If this event has a return period of T years, what is the relationship between p and T?

(4) $$P(Q \geq Q_T) = p = \frac{1}{T}$$

(b) Beginning with the equation stated in part (a), show that the probability, R, that at least one flood whose magnitude exceeds $Q_T$ will occur in N years is given by:

$$R = 1 - \left(1 - \frac{1}{T}\right)^N$$

$P(Q \geq Q_T \text{ in any year}) = \frac{1}{T}$

(8) $\therefore P(Q < Q_T \text{ in any year}) = 1 - \frac{1}{T}$ (complementary probability)

$\therefore P(Q < Q_T \text{ each year for N years}) = \left(1 - \frac{1}{T}\right)^N$ (independent events)

$\therefore P(Q \geq Q_T \text{ at least once in N yrs}) = 1 - \left(1 - \frac{1}{T}\right)^N$
(complementary probability)

(c) What is the probability that a 10 year flood magnitude will be exceeded at least once in the next 10 years? How many years would it take to achieve the same probability of exceeding a 100 year flood magnitude?

$$R = 1 - \left(1 - \frac{1}{T}\right)^N$$

for $T = N = 10$

$$R = 1 - \left(1 - \frac{1}{10}\right)^{10} = 0.6513$$

(8) if $T = 100$ & $R = 0.6513$

then $0.6513 = 1 - \left(1 - \frac{1}{100}\right)^N$

$\therefore (1 - 0.01)^N = 1 - 0.6513$

$0.99^N = 0.3487$

$N \log_{10}(0.99) = \log_{10}(0.3487)$

$N(-0.00436) = -0.45995$

or $N = 105$ years

If using natural logs

$N \ln(0.99) = \ln(0.3487)$

$N(-0.01005) = -1.05354$

$\therefore N = 105$ yrs

**3. Storm Runoff**

(a) A watershed of 3 square miles in Travis County has a time of concentration of 30 minutes. Suppose you wish to apply a 100 year design storm to this watershed and have the storm precipitation defined by two 15 minute rainfall periods with the heaviest rainfall occurring in the first 15 minutes and a lesser amount in the second 15 minutes. A table of depth-duration-frequency values for Travis County is attached at the end of the exam.

$3.04 - 2.29 = 0.75$

(i) What is the total depth of rainfall in the storm? (inches) 3.04"

(ii) What depth falls in the first 15 minutes (inches) 2.29" second 15 minutes 0.75"

(iii) If the watershed has an SCS curve number of 85, how much direct runoff will occur? (inches)

$$S = \frac{1000}{CN} - 10 = \frac{1000}{85} - 10 = 1.7647''$$

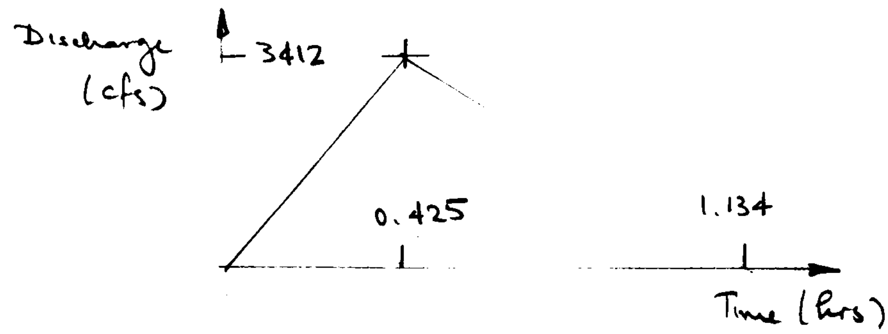
10

$$P_e = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(3.04 - 0.2 \times 1.7647)^2}{(3.04 + 0.8 \times 1.7647)} = \frac{2.687^2}{4.4517}$$

$$P_e = 1.62''$$

(iv) Determine the peak discharge (cfs), the time to the peak (hr) and the total duration (hr) of an SCS triangular unit hydrograph for this watershed, and draw a diagram of this unit hydrograph. Verify that the hydrograph contains 1 inch of direct runoff over the watershed. Take rainfall duration of unit hydrograph to be 15 mins.

10




$t_p = 0.6\, t_c = 0.6 \times 30 = 18 \text{ mins}$

$t_r = 15 \text{ mins}$

$\therefore T_p = t_r/2 + t_p = 15/2 + 18 = 25.5 \text{ min} = 0.425 \text{ hr}$

$t_b = 2.67\, T_p = 2.67 \times 0.425 = 1.134 \text{ hrs}$

$$Q_p = \frac{483.4\, A}{T_p} = \frac{483.4 \times 3}{0.425} = 3412 \text{ cfs}$$

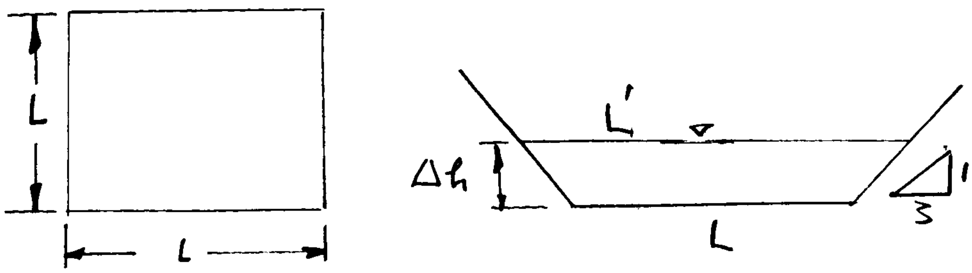
Check: $V_d = \frac{1}{2} Q_p t_b = \frac{1}{2} \times 3412 \times 3600 \times 1.134 = 6{,}964{,}574 \text{ ft}^3$

$$r_d = V_d / A = \frac{6{,}964{,}574}{3 \times 5280^2} = 0.0833' = 1.00''$$

### 4. Detention Pond Specification

(a) The base area of a square detention pond is 1 acre in area and has side slopes of horizontal: vertical of 3:1. If the pond is 10 ft deep, determine its elevation (ft) – area (acres) – storage (acre-ft) relationship in 2 foot increments from 0 to 10 feet. 1 acre = 43,560 ft$^2$.

| Elevation (ft) | Area (acres) | ΔS(acre-ft) | Storage (acre-ft) |
|---|---|---|---|
| 0 | 1.00 | — | 0 |
| 2 | 1.12 | 2.12 | 2.12 |
| 4 | 1.24 | 2.36 | 4.48 |
| 6 | 1.37 | 2.62 | 7.10 |
| 8 | 1.51 | 2.89 | 9.98 |
| 10 | 1.66 | 3.17 | 13.15 |




The base area of the pond is square,

$$\text{so } L = (43{,}560)^{1/2} = 208.7 \text{ ft}$$

for a height increment of water of $\Delta h$

$$L' = L + 3\Delta h * 2$$
$$= 208.7 + 6\Delta h$$

for $\Delta h = 2'$, $L' = 208.7 + 6 \times 2 = 220.7$

$$\therefore A' = 220.7^2 = 48708.49 \text{ ft}^2 = 1.12 \text{ acres}$$

$$\therefore \Delta S = \left(\frac{A + A'}{2}\right)\Delta h = \left(\frac{1 + 1.12}{2}\right) \times 2 = 2.12 \text{ acre-ft}$$

$\therefore S = 0 + 2.12 = 2.12$ ac-ft when depth is 2 ft and other values are similarly computed.

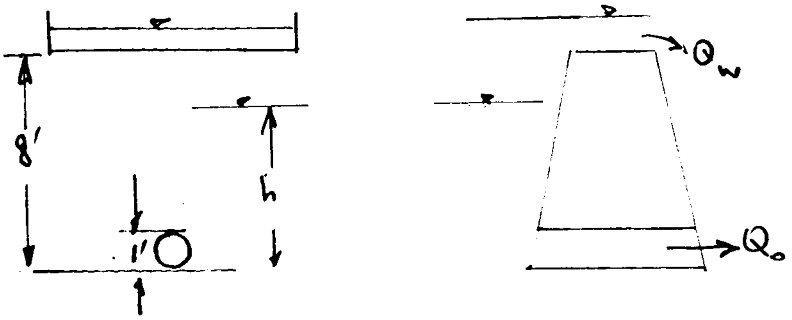
Alternatively:

$$\Delta S = \frac{h_2 - h_1}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$
$$= \frac{2-0}{3}\left(1.00 + 1.12 + \sqrt{1 \times 1.12}\right)$$
$$= 2.12 \text{ ac-ft}$$

same as by previous method

(b) The pond has a circular orifice outlet of 1 ft diameter whose invert is at elevation 0, and a rectangular weir of length 30 ft at elevation 8 ft. Determine the elevation (ft) – discharge (cfs) relation in two foot increments from 0 to 10 feet of depth of water in the pond. Take the orifice coefficient to be 0.6 and the weir coefficient to be 2.6.

| Elevation (ft) | Orifice Flow (cfs) | Weir Flow (cfs) | Total Discharge |
|---|---|---|---|
| 0 | 0 | 0 | 0 |
| 2 | 4.64 | 0 | 4.64 |
| 4 | 7.08 | 0 | 7.08 |
| 6 | 8.88 | 0 | 8.88 |
| 8 | 10.37 | 0 | 10.37 |
| 10 | 11.67 | 220.6 | 232.3 |




Orifice

$$Q_o = C_d A \sqrt{2gH} \quad ; C_d = 0.6 \; ; A = \frac{\pi}{4} \times 1^2 = 0.785 \text{ ft}^2$$

$$\therefore \; Q_o = 0.6 \times 0.785 \times \sqrt{2 \times 32.2 \times (h - 0.5)} \quad (h \text{ is measured from orifice center})$$

$$= 3.785 \, (h - 0.5)^{1/2}$$

eg for $h = 2$, $Q_o = 3.785 \, (2 - 0.5)^{1/2} = 4.64$ cfs

and others computed similarly

Weir

$$Q_w = C L H^{3/2} \qquad C = 2.6, \; L = 30, \; H = h - 8$$

$$\therefore \; Q_w = 2.6 \times 30 \, (h - 8)^{3/2} \qquad h \geq 8'$$

$$Q_w = 78 \, (h - 8)^{3/2}$$

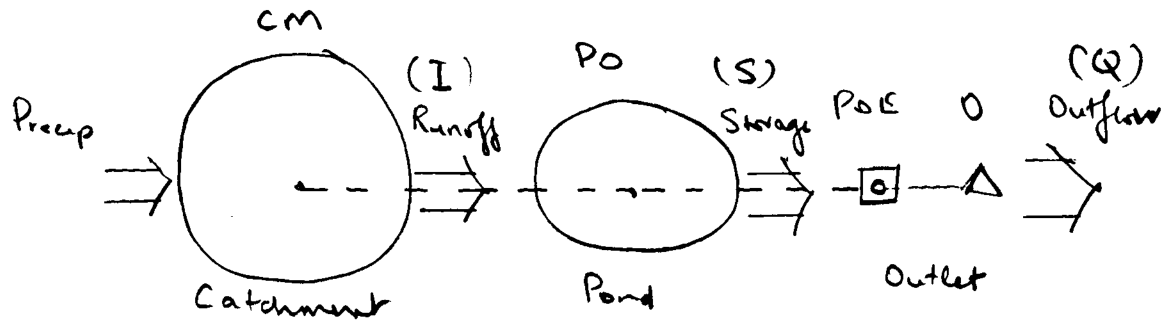
for $h = 10'$, $Q_w = 78 \, (10 - 8)^{3/2}$

$$Q_w = 220.6 \text{ cfs}$$

**5. Detention Pond Design**

(a) You have used the Pond Pack program for design of detention ponds. Draw a diagram that shows how the watershed in Question 3 and the pond and outlet in Question 4 would be symbolized in Pond Pack.

(6)




(b) Describe the steps in the computational process that Pond Pack would use to route the storm described in Question 3 through the watershed and detention pond in Question 4. Start with the storm rainfall hyetograph and end with the discharge hydrograph from the pond outlet.

There are two steps:

(8)

(1) Conversion of storm rainfall over watershed to direct runoff at outlet
- abstraction of losses to get effective precipitation
- application of unit hydrograph to get direct runoff hydrograph at outlet

(2) The direct runoff hydrograph (I) is routed through the pond using the storage-indication method to give the resulting storage S and outflow Q

(c) Suppose that as a designer you were looking at this watershed and detention pond combination and seeking to optimize its performance and functioning on a client's property. What are the things you would consider in looking at design variants? What alternatives to the project choices that have been made in Questions 3 and 4 would you consider in looking at ways to make a better design?

(6)

(1) Primary criteria
- make sure that outflow with pond ≤ outflow under natural conditions for a range of design return periods
- try to minimize amount of space the pond takes up on the property

(2) Alternatives - make pond smaller in area but deeper?
- change outlet structure characteristics to achieve best flow control.

**Table 2-3. Depth-Duration-Frequency Table for Austin and Travis County>**

**Depth of Precipitation (in inches)**

| Recurrence Interval (year) | 5 min* | 15 min | 30 min | 1-hr | 2-hr | 3-hr | 6-hr | 12-hr | 24-hr |
|---|---|---|---|---|---|---|---|---|---|
| 2 | 0.48 | 0.98 | 1.32 | 1.72 | 2.16 | 2.32 | 2.67 | 3.06 | 3.44 |
| 5 | 0.62 | 1.26 | 1.71 | 2.28 | 2.89 | 3.13 | 3.56 | 4.07 | 4.99 |
| 10 | 0.71 | 1.47 | 1.98 | 2.68 | 3.42 | 3.71 | 4.21 | 4.81 | 6.1 |
| 25 | 0.84 | 1.76 | 2.36 | 3.28 | 4.2 | 4.55 | 5.14 | 5.9 | 7.64 |
| 50 | 0.94 | 2.01 | 2.68 | 3.79 | 4.88 | 5.28 | 5.94 | 6.86 | 8.87 |
| 100 | 1.05 | 2.29 | 3.04 | 4.37 | 5.66 | 6.11 | 6.85 | 7.96 | 10.2 |
| 250 | 1.21 | 2.73 | 3.57 | 5.26 | 6.86 | 7.38 | 8.24 | 9.67 | 12 |
| 500 | 1.33 | 3.11 | 4.02 | 6.06 | 7.94 | 8.51 | 9.47 | 11.2 | 13.5 |

* The 5-min rainfall depths were calculated using the 5-min rainfall intensity values from Table 2-4.