

DRAINAGE REPORT
FOR CITY OF STEAMBOAT SPRINGS
FINAL DEVELOPMENT PLAN APPROVAL

RIVERFRONT INDUSTRIAL PARK (LOT 2 PETRILLO SUBDIVISION)

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PURPOSE

The purpose of this report is to support the applicant's Riverfront Industrial Park Final Development Plan (FDP) submittal and respond to Technical Advisory Committee (TAC) comments dated June 22, 2005. Specifically, the TAC comments call for providing a drainage study or letter for "...managing and treating storm water run off." This report is intended to satisfy this requirement by identifying drainage problems for the project site and recommending that standard engineering practices be used to incorporate solutions to these in the site improvements design. This report does not analyze floodplain mapping nor does it address issues related to flooding of the Yampa River, which will be handled under a separate review process.

Survey boundary, existing conditions information, flood zone boundaries and topography have been provided by Emerald Mountain Surveys, Inc. Project site layout information has been provided by Archistructure One. Off-site drainage basin boundaries have been determined from City of Steamboat Springs topographic mapping dated 1994.

PROPERTY CHARACTERISTICS

The 4.50-acre, triangular-shaped project site is within the city limits of Steamboat Springs and consists of Lot 2 of the Petrillo Subdivision. The property is bounded by the City's twin sewer line easement to the northeast, the Yampa River to the south and City property to the northwest, which is adjacent to Shield Drive.

The subject property is very flat but generally slopes to the south and west, with existing grades exceeding 20% at limited areas of the Yampa River bank. With limited evidence of existing on-site drainage patterns indicated from site investigations, a review of the floodplain and topographic mapping for the project site was also made. This showed that the same areas potentially inundated by flood waters in the pre-development condition also allow on-site drainage to generally sheet flow to either the south or the west before reaching the Yampa River. That portion draining to the west enters an existing swale, which also drains to the south, through an area of wetlands and to the Yampa River (see FDP Existing Conditions Plan).

PROPOSED IMPROVEMENTS

The proposal includes the development of 106,752 total square feet of warehouses, office space and employee housing, along with associated parking and infrastructure (see FDP Site Plan). Following development, approximately 80% of the 4.50-acre site will be covered with impervious surfacing, including asphalt pavement, concrete walkways and building roofs. Site grading and storm sewer systems will allow for surface runoff to the south and west to be maintained (see FDP Grading/Drainage Plan). Stormwater quality features will provide for treatment of runoff before entering the Yampa River. We recommended that, in the final design, individual drainage system elements be specified and sized to accommodate the anticipated flows and water quality identified in this report.

METHODOLOGY FOR HYDROLOGY ANALYSIS

The Rational Method of analysis was used to determine peak runoff values. Rainfall intensity was determined from rainfall intensity-duration frequency curves developed for Steamboat Springs from precipitation data (see Attachment 1). For estimating 25-year peak runoff values, a standardized rainfall intensity of 2.7 inches per hour was used, based on a typical small-site time of concentration of 15 minutes. Actual concentration times will vary for individual drainage

basins, resulting in potentially higher rainfall intensity and peak runoff rates for short (less than 15-minute) periods of time, but standard engineering practices call for the selection of a reasonable minimum such as this. Runoff coefficients consisted of values of 0.2 for the existing, unimproved condition as well as future naturally-landscaped areas, 0.7 for existing gravel-surfaced areas, and 0.9 for the existing and future impervious surface areas (see Attachment 2).

SUMMARY OF HYDROLOGY AND HYDRAULIC CALCULATIONS

To determine the off-site drainage and relative change in anticipated on-site runoff resulting from the proposed development, drainage sub-basins were delineated for both the pre- and post-development condition for the purpose of calculating composite site runoff coefficients, C , and the rational formula, $Q = CIA$, was then applied for the storm event identified above.

Off-site storm drainage systems adjacent to the project site consist of a ditch along the northeast side of the City's sewer line berm and a series of culverts, allowing surface runoff to flow to the northwest and under Shield Drive through an existing 43" X 27" CMP arch culvert. The off-site runoff contribution from areas to the north has been recently reviewed for other development projects and the vast majority of this, including that from the north side of US 40 has subsequently been re-routed along Shield Drive and directly through the existing 43" X 27" CMP. This leaves a much smaller area south of US 40 contributing runoff to the existing culverts north of the project site. These consist of a 15"-diameter CMP, an 18"-diameter CMP and another 43" X 27" CMP arch culvert.

The smaller, 15" and 18", pipes of above have previously been determined to be undersized for the runoff that once entered from the north side of US 40, resulting in ponding during the spring. However, this runoff has now been greatly reduced and the adequacy of these pipes does not impact the project site since any overflow is intercepted by the existing ditch along the northeast side of the sewer berm and exits through the larger culverts. An analysis of the larger, downstream culverts was made, however, to confirm that the existing culverts and ditch are adequately sized for the continued off-site drainage anticipated at the project site boundaries (see Attachment 3).

For the purpose of identifying the affect of the proposed development upon adjacent storm drainage transmission facilities, pre-development runoff was first analyzed to determine flow rates and drainage patterns (see Attachment 4). In summary, calculations of peak runoffs resulting from the selected storm event show that 0.5 cfs currently flows to the existing swale and wetlands to the west of the project site and 2.1 cfs flows to the Yampa River, to the south, for a total pre-development figure of 2.6 cfs. For comparison, the post-development condition was similarly analyzed for each drainage sub-basin and topographic data was used to determine the hydraulic capacity of the affected transmission facilities, both existing and proposed (see Attachment 5). Peak runoff calculations resulting from the selected storm event range from 0.2 to 3.3 cfs, for an increased total of 9.7 cfs for all post-development sub-basins. As with the pre-development condition, all on-site drainage from the development will be directed toward existing receiving waters. On-site storm water detention is not recommended as the increase in runoff rates is not anticipated to significantly impact receiving waters (Yampa River).

In all cases, the capacity of the adjacent transmission facilities exceeds the calculated peak flows of the contributing sub-basin(s), assuming that proposed storm sewers are designed for the sizes and configurations selected as a part of this analysis. In summary, we recommend the proposed facilities include a minimum 12"-diameter culvert under the project site entrance,

allowing drainage from the northeast side of the site to reach the existing swale along the northwest side (or a final site grading design that would allow this same drainage to flow in a southeasterly direction, toward the Yampa River), and minimum 12" and 18" storm sewers with respective 0.9-square foot and 2.0-square foot open area catch basin grates as indicated.

STORMWATER QUALITY MANAGEMENT

We recommend that the final drainage design include elements and controls intended to reduce erosion, collect sediment and minimize water quality impacts to receiving waters. A stormwater management plan incorporating "Best Management Practices" (BMPs) should be prepared and employed during construction.

In order to meet the City of Steamboat Springs' requirements for designing of treatment of storm water runoff from parking areas, proposed swales and sedimentation ponds should be designed for adequate sizing and configuration, according to the peak runoff rates determined herein (see Attachment 6). It is recommended that the final design of the proposed sedimentation pond and swale system, shown as a part of the project's drainage plan, be designed to maximize detention times and treatment potential by sizing these structures in accordance with the assumptions and recommendations included in the analysis for each structural BMP in Attachment 6.

In addition, the final design of the systems shown, or equivalent facilities, should meet standard design criteria for inlet and outlet capacities, soil surfacing, vegetation establishment and other construction details necessary to properly functioning at the design flows and for the treatment of anticipated stormwater constituents of the proposed development. The easternmost swale shown discharging to the Yampa River should be developed as a Grass Swale Sedimentation Facility (per Urban Drainage and Flood Control District manual referenced in Attachment 6) running parallel to and north of the proposed soft trail to treat stormwater before either entering the sedimentation pond shown or discharging directly to the river. The sedimentation pond should be developed as an Extended Detention Basin Sedimentation Facility (per Urban Drainage and Flood Control District manual referenced in Attachment 6), also north of the soft trail and for treatment prior to discharge and, if also receiving the discharge of the grass swale above, should be increased in size according to the design criteria. Adequate space exists on the site plan for the development of each of these. Alternatively, final design measures may include mechanical treatment of runoff prior to entering receiving waters of the Yampa River.

The existing swale and wetlands system along the northwestern edge of the site should continue to serve as a natural buffer for treatment of runoff reaching this area and is adequately sized and configured for the peak runoff rates. Because the wetlands serve as a mitigation site for the adjacent James Brown Bridge, flows between the wetlands and the Yampa River are required to be maintained (see Attachment 6). The existing sheet flow outlet from the project site, along the Yampa River bank, will continue to serve as a grass buffer due to the reduced flow rates identified herein and should be maintained with vegetation similar to the pre-development condition in order to adequately do so.

Construction Mitigation (Temporary Measures):

The following practices are recommended to lessen water quality impacts from runoff during construction.

Silt fence or erosion logs should be installed along downhill limits of all earth stockpiles, at the base of fill slopes and at the downhill limits of site grading, and particularly along the existing

wetlands and Yampa River, which are to remain undisturbed.

Measures to control surface drainage and trap sediment should be employed to prevent sediment laden site runoff to discharge directly to the receiving waters and adjacent properties. All controls should be installed prior to any significant stripping and grading activities. Temporary sediment checks should be constructed along all roadway ditches and drainage channels until vegetation is established.

All controls and measures should be monitored regularly by the Contractor and modified as necessary to improve effectiveness. All perimeter controls should be cleaned or replaced when accumulated 1/2 full. Perimeter controls should be relocated as the areas of construction move and as the disturbed areas stabilize. Topsoil replacement and revegetation should occur as soon as practical after construction and finished grading activities are completed in any area of the site.

We recommend that all drainage channels be revegetated early enough in the growing season so that grasses can become established prior to the first winter, or that other suitable controls be installed prior to the first winter.

Post Construction Mitigation (Permanent Controls):

The following practices are recommended to lessen water quality impacts from site runoff following completion of development.

The entire disturbed portion of the site not covered by impervious surfaces should receive topsoil replacement, landscaping and/or revegetation. After final shaping, channel improvements should be revegetated to create grass-lined channels where erosive flow velocities are not anticipated and channels should be armored where slopes exceed 5% or where erosion is anticipated.

LIMITATIONS

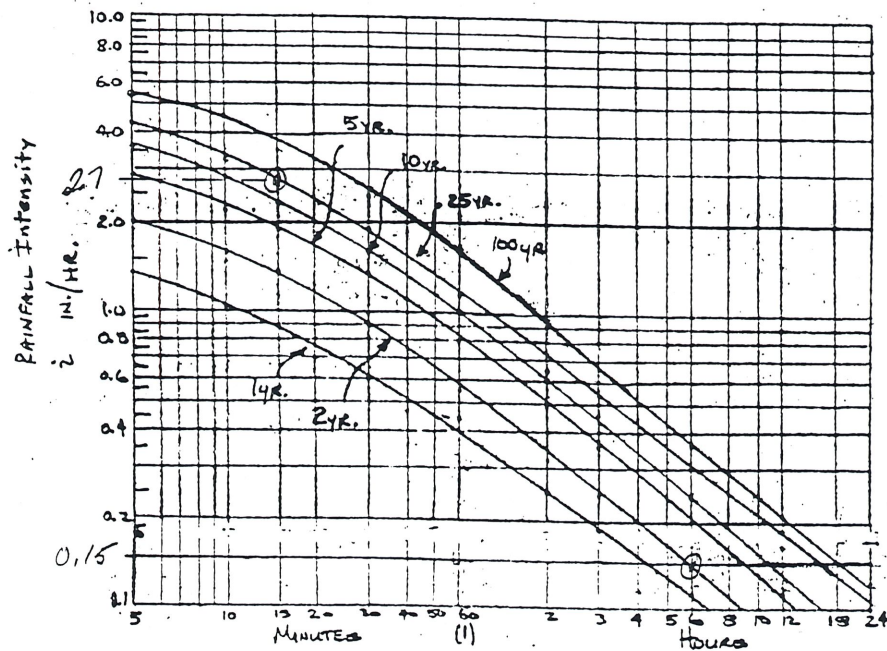
The recommendations presented in this report are based on topographic and existing conditions information provided by others and our knowledge and review of the site. We believe the recommendations presented in this report are suitable for planning and design. We recommend the final design be reviewed for consistency with these recommendations.

ATTACHMENTS

- 1 - Rainfall Intensity – Duration Curve
- 2 - Rational Method Runoff Coefficients
- 3 - Off-Site Runoff Calculations
- 4 - On-Site Runoff Calculations, Pre-Development
- 5 - On-Site Runoff Calculations, Post-Development
- 6 - Stormwater Quality Management Calculations

ATTACHMENT 1

RAINFALL INTENSITY-DURATION CURVE



RAINFALL INTENSITY/DURATION/FREQUENCY CURVE

FOR

STEAMBOAT SPRINGS, COLORADO

ATTACHMENT 2

RATIONAL METHOD RUNOFF COEFFICIENTS

CIVIL ENGINEERING REFERENCE MANUAL

Appendix A: Rational Method Runoff Coefficients

categorized by surface

forested	0.059-0.2
asphalt	0.7-0.95 <i>all imp. surf. $\Rightarrow$ 0.9</i>
brick	0.7-0.85
concrete	0.8-0.95
shingle roof	0.75-0.95
lawns, well drained (sandy soil)	
up to 2% slope	0.05-0.1
2% to 7% slope	0.10-0.15
over 7% slope	0.15-0.2
lawns, poor drainage (clay soil)	
up to 2% slope	0.13-0.17
2% to 7% slope	0.18-0.22
over 7% slope	0.25-0.35
driveways, walkways	0.75-0.85 <i>GRAVEL $\Rightarrow$ 0.7</i>

categorized by use

farmland	0.05-0.3
pasture	0.05-0.3
unimproved	0.1-0.3 <i>AVG. $\Rightarrow$ 0.2</i>
parks	0.1-0.25
cemeteries	0.1-0.25
railroad yard	0.2-0.40
playgrounds (except asphalt or concrete)	0.2-0.35
business districts	
neighborhood	0.5-0.7
city (downtown)	0.7-0.95
residential	
single family	0.3-0.5
multi-plexes, detached	0.4-0.6
multi-plexes, attached	0.6-0.75
suburban	0.25-0.4
apartments, condominiums	0.5-0.7
industrial	
light	0.5-0.8 <i>AVG. $\Rightarrow$ 0.6</i>
heavy	0.6-0.9

ATTACHMENT 3
OFF-SITE RUNOFF CALCULATIONS

RIVERFRONT INDUSTRIAL PARK
OFF-SITE DRAINAGE

RWS 1/4
4135.001A
9/22/05



RIVERFRONT INDUSTRIAL PARK
DRAINAGE REPORT
OFF-SITE DRAINAGE

RWS #4
4135,001A
9/22/05

$$A = 8.8 \text{ AC.}$$

ASSUME: $C = 0.6$ (AVG. LIGHT INDUSTRIAL)
 $I = 2.7 \text{ IN./HR.}$ (25-YR., 15-MIN. EVENT)

$$\begin{aligned}\therefore Q &= CIA \\ &= (0.6)(2.7)(8.8) \\ &= 14.3 \text{ CFS}\end{aligned}$$

1. LOOK @ EXISTING 36" ϕ CMP, WHICH CARRIES ENTIRE FLOW FROM DRAINAGE BASIN (EXISTING 15" ϕ AND 18" ϕ CMP DO NOT AFFECT PROJECT SITE AS ANY OVERFLOW ALSO REACHES 36" ϕ CMP):

FROM FIG. 3.28, STEEL DRAINAGE AND HIGHWAY CONSTRUCTION PRODUCTS, AISI, FOURTH EDITION:

$$\frac{HW}{D} = 0.58$$

$$\Rightarrow \text{DEPTH} = 21 \text{ IN.}$$

$\therefore$ CULVERT AND UPSTREAM DITCH, WHICH IS CONSIDERABLY DEEPER AND WIDER, HAVE CAPACITIES GREATER THAN RUNOFF FOR 25-YR., 15-MIN. STORM EVENT.

2. LOOK @ EXISTING 43" X 27" CMP ARCH:

SIMILARLY, FROM FIG. 3.30, AISI:

$$\frac{HW}{D} = 0.62$$

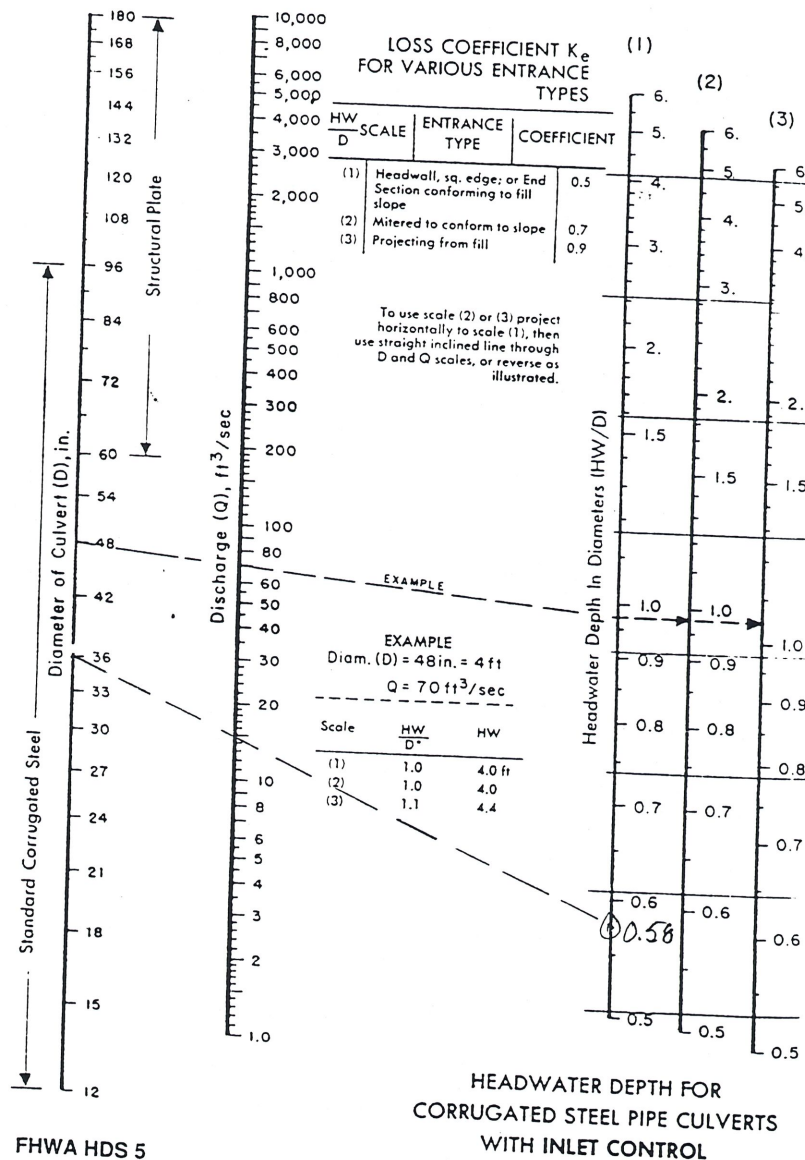
$$\Rightarrow \text{DEPTH} = 17 \text{ IN.}$$

$\therefore$ CULVERT CAPACITY ALSO EXCESSIVE

RWS 3/4
 135.001A
 9/22/05

Improved Inlets

Culvert capacity may be increased through the use of special inlet designs. The Federal Highway Administration has developed extensive data^{19,20} on these. While these designs increase the flow, their use has not been as expected. The increased costs of the special treatments is apparently responsible.



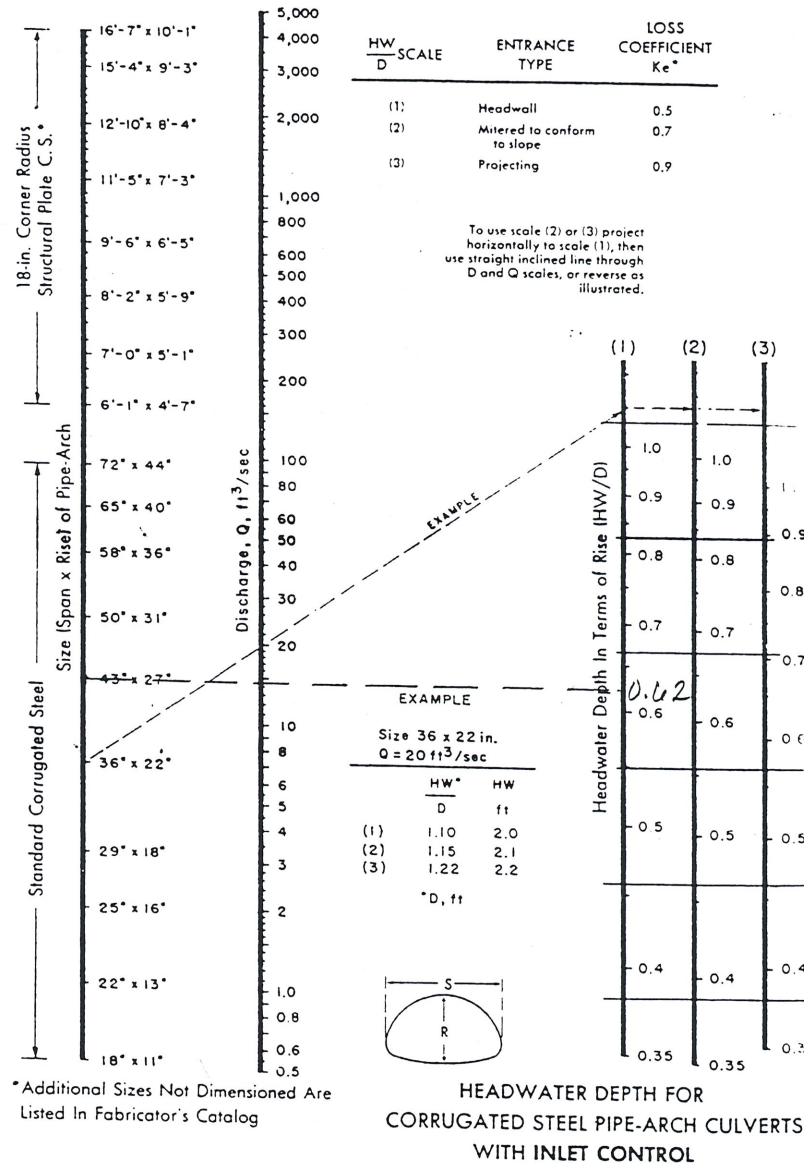
FHWA HDS 5

Figure 3.28 Inlet control nomograph for corrugated steel pipe culverts. The manufacturers recommend keeping HW/D to a maximum of 1.5 and preferably no more than 1.0 for diameters greater than 4 to 5 feet.

FHV

Fig

RWS 4/4
4/35.001A
9/22/05



FHWA HDS 5

Figure 3.30 Inlet control and headwater depths for corrugated steel pipe-arch culverts. Headwater depth should be kept low because pipe-arches are generally used where headroom is limited.

ATTACHMENT 4

ON-SITE RUNOFF CALCULATIONS, PRE-DEVELOPMENT

PWS 1/2
 4135.001 A
 10/12/05



AL NOTES

1. MINIMUM, ALL CONCRETE SLAB
 SHALL BE REINFORCED WITH REBAR
 1/2" DIA. AND ARE DESIGNED TO BE
 4" THICK OR MORE. DO NOT
 USE AT PACE OR MORE THIN.

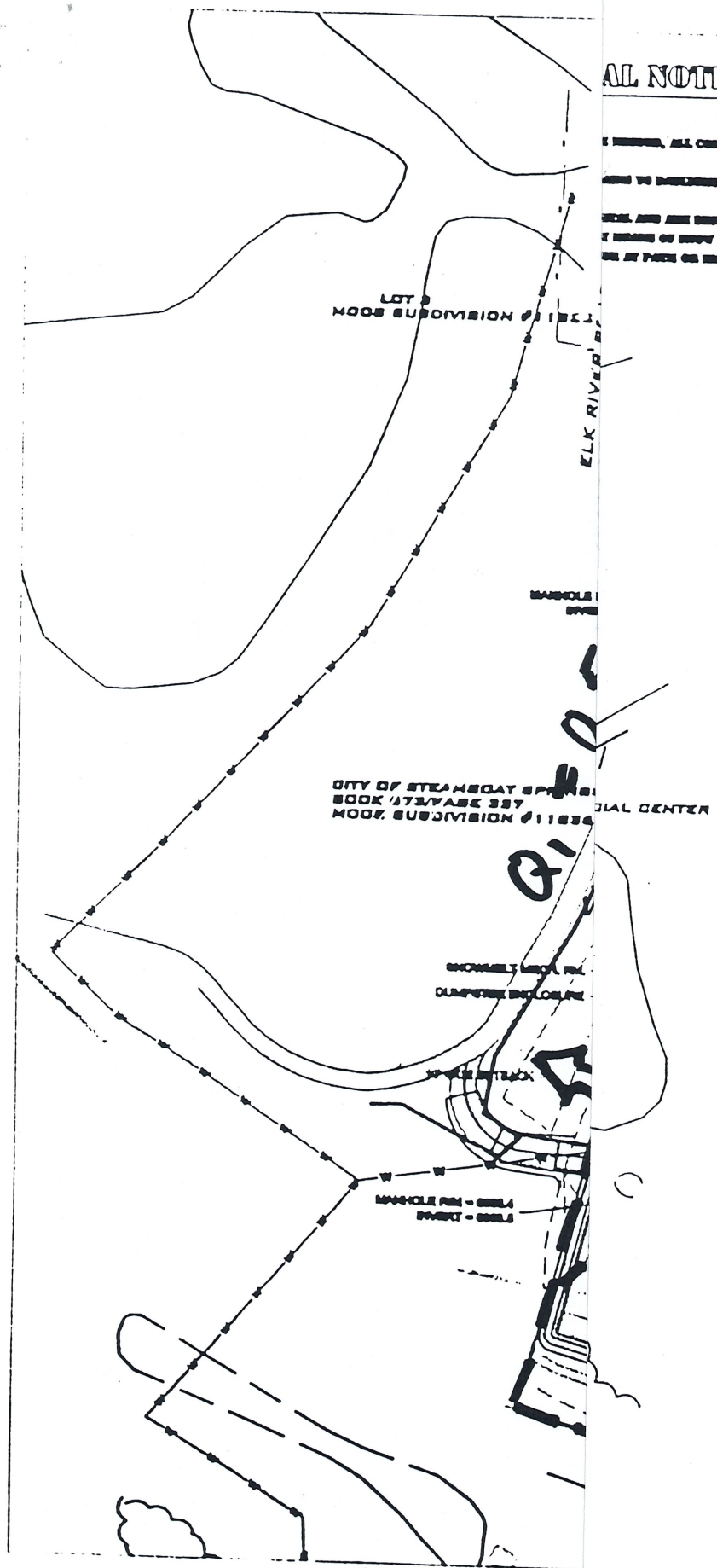
Architectural Co.
 P.O. Box 7000
 Steamboat Springs, CO 80427
 PHONE: 970.861.1111

RIVERFRONT
 INDUSTRIAL PARK
 1522 SHIELD DRIVE
 STEAMBOAT SPRINGS, CO

PROJECT #00004

DATE	ITEM
6.1.05	FDP ORIENTAL
6.15.05	FDP REVISIONS

FDP2



N
 1 SITE PL
 1/15

RIVERFRONT INDUSTRIAL PARK
DRAINAGE REPORT
ON-SITE DRAINAGE

RMS 2/2
4135.001 A
9/22/05

PRE-DEVELOPMENT

1. LOOK @ WESTERN EDGE OF PROJECT SITE,
DRAINING TOWARD EXISTING SWALE ALONG
PROPERTY BOUNDARY:

$$A_1 = 0.67 \text{ AC.}$$

ASSUME: FOR 25% OF AREA, $C = 0.7$ (GRAVEL)
FOR 75% OF AREA, $C = 0.2$ (UNIMPROVED)

$$\Rightarrow C_1 = (0.7)(0.25) + (0.2)(0.75) \approx 0.3$$

$$I = 2.7 \text{ IN./HR. (25-YR., 15-MIN. EVENT)}$$

$$\begin{aligned}\therefore Q_1 &= C_1 I A_1 \\ &= (0.3)(2.7)(0.67) \\ &= 0.5 \text{ CFS}\end{aligned}$$

2. LOOK @ REMAINING PORTION OF PROJECT
SITE, DRAINING TOWARD YAMPA RIVER:

$$\begin{aligned}A_2 &= 4.50 \text{ AC.} - A_1 \\ &= 3.83 \text{ AC.}\end{aligned}$$

ASSUME: $C_2 = 0.2$ (UNIMPROVED)

$$I = 2.7 \text{ IN./HR. (25-YR., 15-MIN. EVENT)}$$

$$\begin{aligned}\therefore Q_2 &= C_2 I A_2 \\ &= (0.2)(2.7)(3.83) \\ &= 2.1 \text{ CFS}\end{aligned}$$

ATTACHMENT 5

ON-SITE RUNOFF CALCULATIONS, POST-DEVELOPMENT

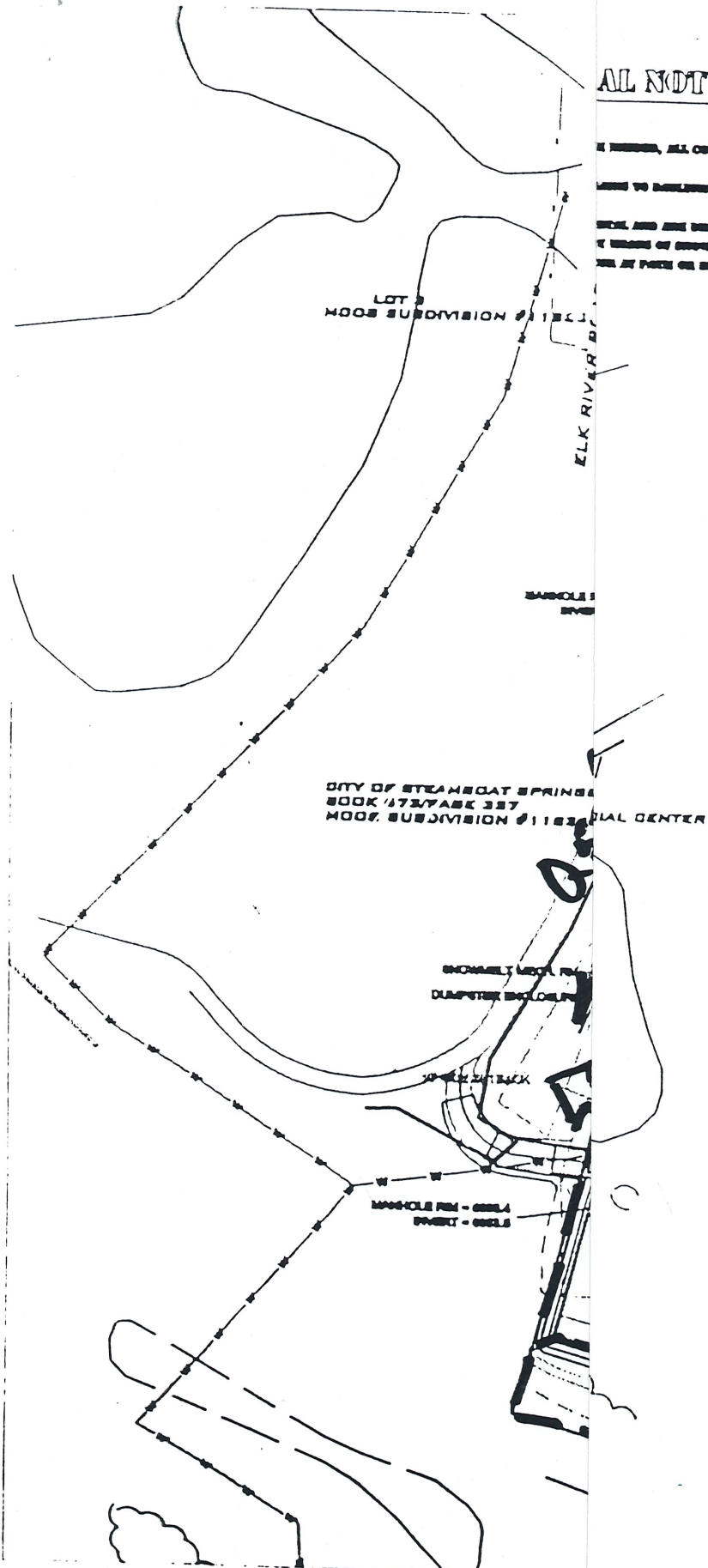
RWS 1/15
 4135.001A
 10/12/05



AL NOTES

ALL NOTES, ALL CHANGES AND
 TO BE MADE TO THE PLAN
 AND ARE DESIGNED TO HOLD
 THE WORKER TO THE PLAN, DO NOT
 AT THE END OF THE WORK.

Architectural Office
 P.O. Box 17000
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RIVERFRONT
 INDUSTRIAL PARK
 1322 SHIELD DRIVE
 STEAMBOAT SPRINGS, CO

PROJECT #03004

DATE	ITEM
6.16.05	POP SUBMITTAL
6.16.05	POP REVISIONS

FIDP2

N
 ① SITE PLAN
 1175

RWS 2/15
4135.00/A
9/22/05

POST-DEVELOPMENT

1. LOOK @ NORTHEASTERN EDGE OF PROJECT SITE, DRAINING TOWARD CITY SEWER 138 PM TO NORTHEAST;

$$A_1 = 0.83 \text{ AC.}$$

ASSUME: $C_1 = 0.9$ (MOSTLY IMPERVIOUS SURFACES)
 $I = 2.7 \text{ IN./HR.}$

$$\therefore Q_1 = C_1 I A_1 = (0.9)(2.7)(0.83) = 2.0 \text{ CFS}$$

CAPACITIES:

- A. EXISTING DITCH SECTION, FLOWING TOWARD NORTHWEST:

DEPTH, $D \approx 1 \text{ FT.}$
SIDE SLOPES $H:V \approx 2:1$
SLOPE $S \approx 0.01$
MANNING'S $n = 0.035$ (NATURAL CHANNELS w/ STONES, WEEDS)

FROM TRIANGULAR CHANNEL ANALYSIS & DESIGN MODEL, HASTAD METHOD:

$$\text{ACTUAL DEPTH @ } 2.0 \text{ CFS} = 0.71 \text{ FT.} < 1 \text{ FT.}$$

$\therefore$ DITCH IS ADEQUATE

- B. PROPOSED CULVERT UNDER PROJECT SITE ENTRANCE, FLOWING TOWARD EXISTING SWALE ALONG WESTERN PROPERTY BOUNDARY.

ASSUME: 12" ϕ CMP OR EQUIVALENT, MIN.

FROM FIG. 3.20, AISI:

$$\frac{H_W}{D} = 0.9$$

$$\Rightarrow \text{DEPTH} = 11 \text{ IN.}$$

$\therefore$ CULVERT CAPACITY IS ADEQUATE

Triangular Channel Analysis & Design
Open Channel - Uniform flow

PWS 3/15
4135.001A
9/22/05

Worksheet Name: Riverfront Ind. Park

Comment: Existing Northeastern Ditch

Solve For Depth

Given Input Data:

Left Side Slope..	2.00:1 (H:V)
Right Side Slope.	2.00:1 (H:V)
Manning's n.....	0.035
Channel Slope....	0.0100 ft/ft
Discharge.....	2.00 cfs

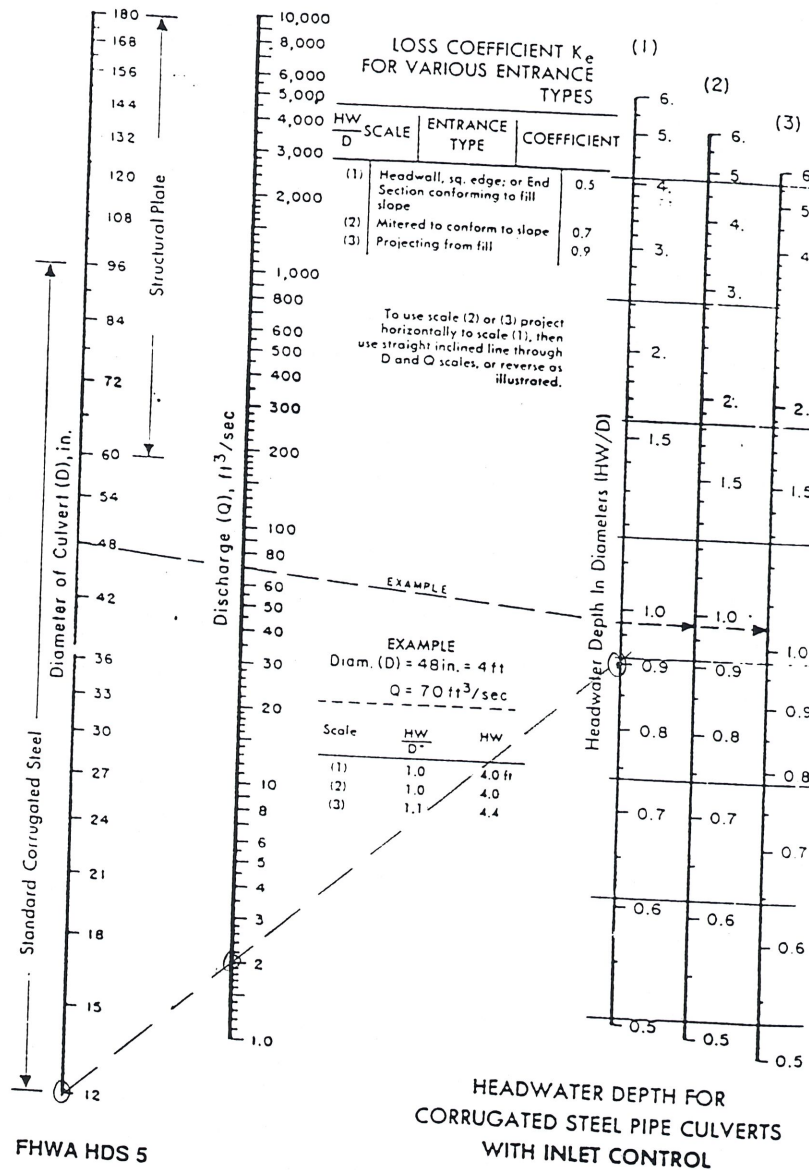
Computed Results:

Depth.....	0.71 ft
Velocity.....	1.98 fps
Flow Area.....	1.01 sf
Flow Top Width...	2.84 ft
Wetted Perimeter.	3.18 ft
Critical Depth...	0.57 ft
Critical Slope...	0.0314 ft/ft
Froude Number....	0.58 (flow is Subcritical)

RWS 4/15
4/135.001A
9/22/05

Improved Inlets

Culvert capacity may be increased through the use of special inlet designs. The Federal Highway Administration has developed extensive data^{19,20} on these. While these designs increase the flow, their use has not been as expected. The increased costs of the special treatments is apparently responsible.



FHWA HDS 5

Figure 3.28 Inlet control nomograph for corrugated steel pipe culverts. The manufacturers recommend keeping HWD to a maximum of 1.5 and preferably to no more than 1.0 for diameters greater than 4 to 5 feet.

FHV

Fig

2. LOOK @ SOUTHEASTERN CORNER OF PROJECT SITE,
DRAINING TO PROPOSED CATCH BASIN AND
STORM SEWER TOWARD YAMPA RIVER:

$$A_2 = 0.36 \text{ AC.}$$

ASSUME: $C_2 = 0.9$ (MOSTLY IMPERVIOUS SURFACES)
 $I = 2.7 \text{ IN./HR.}$

$$\therefore Q_2 = C_2 I A_2 = (0.9)(2.7)(0.36) = 0.9 \text{ CFS}$$

CAPACITIES:

A. PROPOSED CATCH BASIN GRATE INLET:

ASSUME: 0.9-SQUARE FOOT OPEN AREA, MIN.

FROM NEEDHAM FOUNDRY CO. DISCHARGE VS.
DEPTH ON GRATE NOMOGRAM, MAX.
0.3-FOOT DEPTH:

$$Q_{\text{MAX.}} = 2.3 \text{ CFS} > Q_2$$

$\therefore$ GRATE CAPACITY IS ADEQUATE

B. PROPOSED STORM SEWER:

ASSUME: 12" ϕ POLY PIPE OR EQUIVALENT,

$$S \approx \frac{1.0}{140} = 0.007$$

$$n = 0.015 \text{ (PVC, SMOOTH)}$$

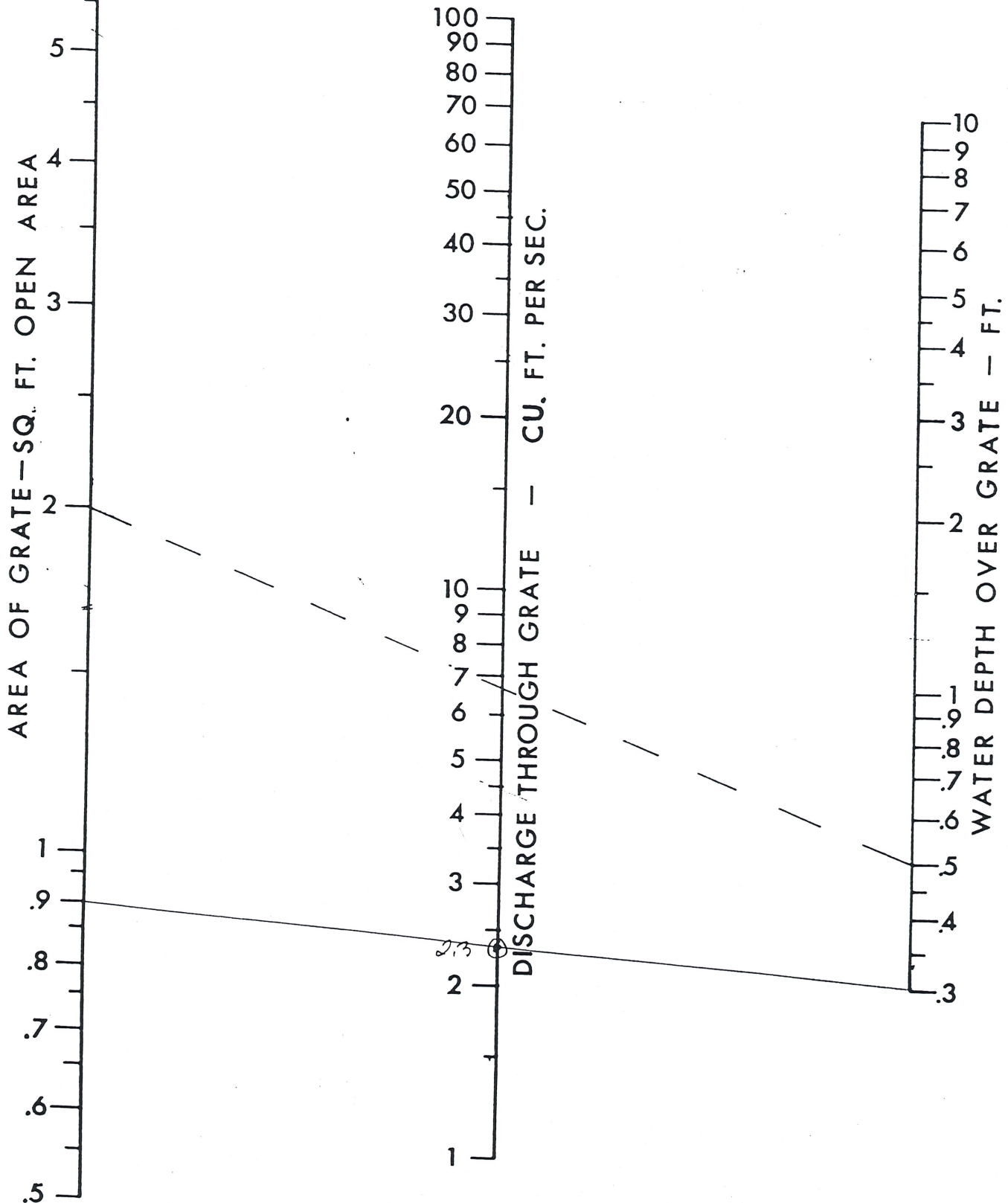
FROM CIRCULAR CHANNEL ANALYSIS &
DESIGN MANUAL, HYDRAULIC METHODS:

$$Q_{\text{MAX.}} = 2.50 \text{ CFS} > Q_2$$

$\therefore$ STORM SEWER CAPACITY IS ADEQUATE

Discharge vs Depth On Grate

PWS 6/15
4135.00/A
9/22/05



2ms 7/15
4135.001A
9/22/05

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Riverfront Ind. Park

Comment: Proposed Southeastern Storm Sewer

Solve For Full Flow Capacity

Given Input Data:

Diameter.....	1.00 ft
Slope.....	0.0070 ft/ft
Manning's n.....	0.015
Discharge.....	2.58 cfs

Computed Results:

Full Flow Capacity.....	2.58 cfs
Full Flow Depth.....	1.00 ft
Velocity.....	3.29 fps
Flow Area.....	0.79 sf
Critical Depth....	0.69 ft
Critical Slope....	0.0104 ft/ft
Percent Full.....	100.00 %
Full Capacity.....	2.58 cfs
QMAX @.94D.....	2.78 cfs
Froude Number.....	FULL

RWS 6/15

3. LOOK @ SOUTHERN EDGE OF PROJECT SITE,
DRAINING TO YAMPA RIVER:

$$A_3 = 0.61 \text{ AC.}$$

ASSUME: FOR 20% OF AREA, $C = 0.7$ (GRAVEL TRAIL)
FOR 80% OF AREA, $C = 0.2$ (UNIMPROVED)

$$\Rightarrow C_3 = (0.7)(0.2) + (0.2)(0.8) = 0.3$$

$$I = 2.7 \text{ IN./HR.}$$

$$\therefore Q_3 = C_3 I A_3 = (0.3)(2.7)(0.61) = 0.7 \text{ CFS}$$

CAPACITIES: THIS REPRESENTS A REDUCTION
IN SHEET FLOW ONLY (NOT TOTAL FLOW)
REACHING THE YAMPA RIVER HERE, COMPARED
TO THE PRE-DEVELOPMENT CONDITION AS FOLLOWS:

$$Q = 2.1 \text{ CFS} - 0.7 \text{ CFS} = 1.4 \text{ CFS}$$

$$\frac{Q_{\text{POST-DEVELOPMENT}}}{Q_{\text{PRE-DEVELOPMENT}}} = \frac{0.7}{2.1} = 0.33, \text{ OR } 33\%$$

4. LOOK @ CENTRAL AREA OF PROJECT SITE,
DRAINING TO PROPOSED CATCH BASIN AND
STORM SEWER TOWARDS YAMPA RIVER:

$$A_4 = 1.36 \text{ AC.}$$

ASSUME: $C_4 = 0.9$ (MOSTLY IMPERVIOUS SURFACES)
 $I = 2.7 \text{ IN./HR.}$

$$\therefore Q_4 = C_4 I A_4 = (0.9)(2.7)(1.36) = 3.3 \text{ CFS}$$

CAPACITIES:

A. PROPOSED CATCH BASIN GRATE INLET:

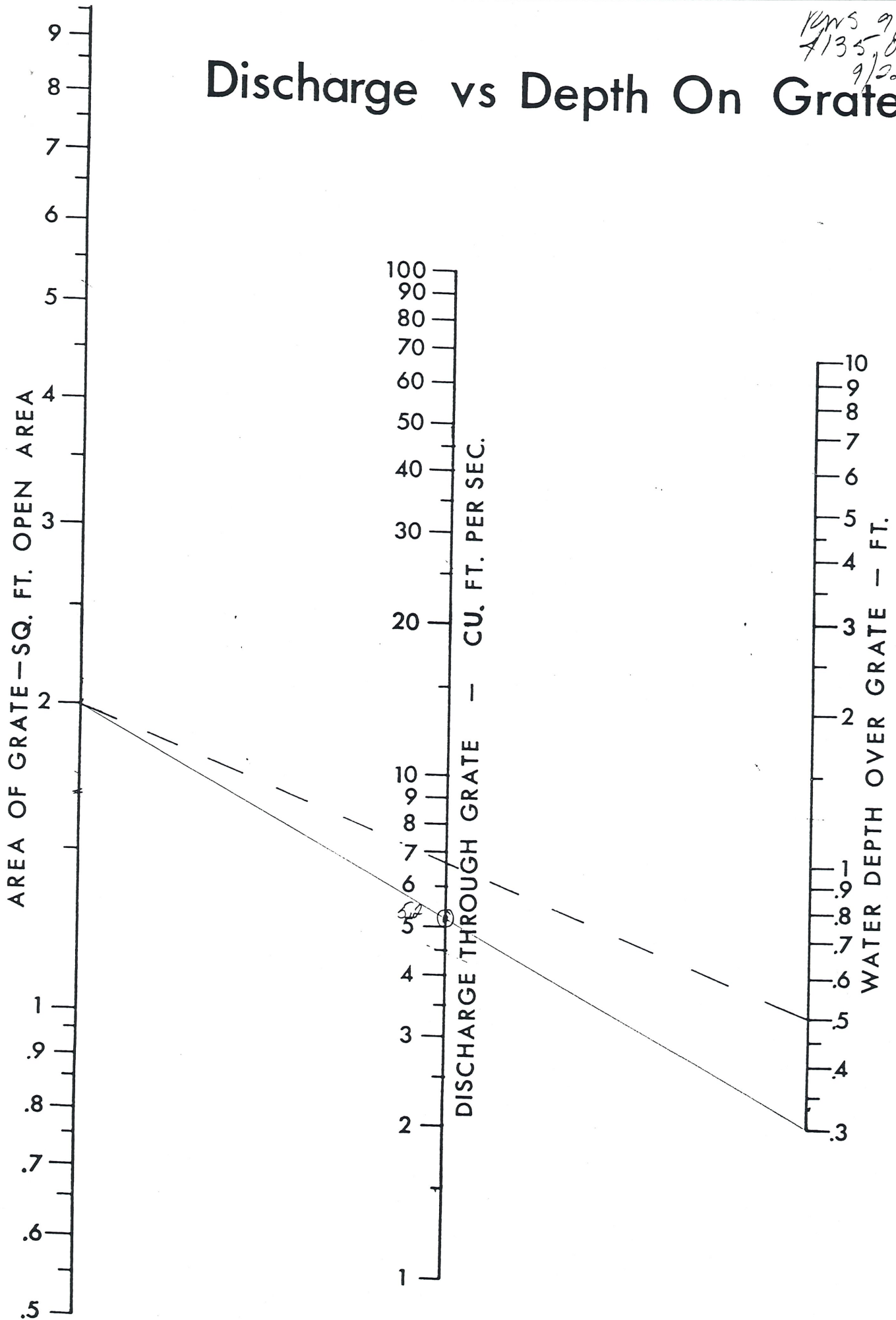
ASSUME: 2.0 - SQUARE-FOOT OPEN AREA, MIN.
FROM NEENAH FOUNDRY CO. DISCHARGE VS.
DEPTH ON GRATE NOMOGRAPH, MAX.
0.3 - FOOT DEPTH:

$$Q_{\text{MAX.}} = 5.2 \text{ CFS} > Q_4$$

$\therefore$ GRATE CAPACITY IS ADEQUATE

KWS 9/15
#135,001A
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Discharge vs Depth On Grate



B. PROPOSED STORM SEWER:

ASSUME: 18" d POLY PIPE OR EQUIVALENT,
MIN.
 $S = \frac{0.7}{196} = 0.004$

$$n = 0.015 \text{ (PVC, SMOOTH)}$$

NEED TO ADD CONTRIBUTION FROM
A₆ (SEE ITEM C.; BELOW):

$$Q_6 = C_6 I A_6 = (0.9)(2.7)(0.09) = 0.2 \text{ CFS}$$

$$\Rightarrow Q_{\text{TOTAL}} = Q_4 + Q_6 = 3.5 \text{ CFS}$$

FROM CIRCULAR CHANNEL ANALYSIS &
DESIGN MODEL, HASTEN METHODS:

$$Q_{\text{MAX.}} = 5.76 \text{ CFS} > Q_{\text{TOTAL}}$$

$\therefore$ STORM SEWER CAPACITY IS ADEQUATE

5. LOOK @ WESTERN EDGE OF PROJECT SITE,
DRAINING TOWARD EXISTING SWALE ALONG
PROPERTY BOUNDARY:

$$A_5 = 1.05 \text{ AC.}$$

ASSUME: $C_5 = 0.9$ (MOSTLY IMPERVIOUS SURFACES)
 $I = 2.7 \text{ IN./HR.}$

$$\therefore Q_5 = C_5 I A_5 = (0.9)(2.7)(1.05) = 2.6 \text{ CFS}$$

CAPACITIES:

EXISTING SWALE, FLOWING TOWARD
EXISTING WETLANDS TO SOUTH:

$$\text{DEPTH, } D \approx 2 \text{ FT.}$$

$$\text{SIDE SLOPES, } H:V \approx 20:2 = 10:1$$

$$\text{SLOPE, } S \approx \frac{6657.1 - 6655.4}{290} = 0.006$$

$$n = 0.035 \text{ (NATURAL CHANNELS W/
STONES, WEEDS)}$$

NEED TO ADD CONTRIBUTION FROM D (SEE

RWS 11/15
4135.001A
9/22/05

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Riverfront Ind. Park

Comment: Proposed Central Area Storm Sewer

Solve For Full Flow Capacity

Given Input Data:

Diameter.....	1.50 ft
Slope.....	0.0040 ft/ft
Manning's n.....	0.015
Discharge.....	5.76 cfs

Computed Results:

Full Flow Capacity.....	5.76 cfs
Full Flow Depth.....	1.50 ft
Velocity.....	3.26 fps
Flow Area.....	1.77 sf
Critical Depth....	0.93 ft
Critical Slope....	0.0081 ft/ft
Percent Full.....	100.00 %
Full Capacity.....	5.76 cfs
QMAX @.94D.....	6.19 cfs
Froude Number.....	FULL

ITEM 1., ABOVE):

RWS 12/15

$$\Rightarrow Q_{TOTAL} = Q_1 + Q_5 = 4.6 \text{ CFS}$$

FROM TRIANGULAR CHANNEL ANALYSIS & DESIGN MODEL, HAESTRA METHODS:

$$\text{ACTUAL DEPTH @ 4.6 CFS} = 0.57 \text{ FT.} < 2 \text{ FT.}$$

$\therefore$ SWALE IS ADEQUATE

6. LOOK @ NORTHERN AREA OF PROJECT SITE DRAINING TO PROPOSED CATCH BASIN AND STORM SEWER OF 4., ABOVE, TOWARD YAMPA RIVER:

$$A_6 = 0.09 \text{ AC.}$$

ASSUME: $C_6 = 0.9$ (MOSTLY IMPERVIOUS SURFACES)
 $I = 2.7 \text{ IN./HR.}$

$$\therefore Q_6 = C_6 I A_6 = (0.9)(2.7)(0.09) = 0.2 \text{ CFS}$$

CAPACITIES:

A. PROPOSED CATCH BASIN GRATE INLET:

ASSUME: 0.9-SQUARE-FOOT OPEN AREA, MIN.

FROM NEEDHAM FOUNDRY CO. DISCHARGE VS. DEPTH ON GRATE NOMOGRAPH, MAX.

0.3-FOOT DEPTH (SEE ITEM 2.A., ABOVE):

$$Q_{MAX.} = 2.3 \text{ CFS} > Q_6$$

$\therefore$ GRATE CAPACITY IS ADEQUATE

B. PROPOSED STORM SEWER:

ASSUME: 12" ϕ POLY PIPE OR EQUIVALENT:

$$S \approx \frac{6665.17 - 6664.5}{202} = 0.003$$

$$n = 0.015 \text{ (PVC, SMOOTH)}$$

Triangular Channel Analysis & Design
Open Channel - Uniform flow

PN 5 13/15
4135.091A
9/22/05

Worksheet Name: Riverfront Ind. Park

Comment: Existing Western Swale

Solve For Depth

Given Input Data:

Left Side Slope..	10.00:1 (H:V)
Right Side Slope.	10.00:1 (H:V)
Manning's n.....	0.035
Channel Slope....	0.0060 ft/ft
Discharge.....	4.60 cfs

Computed Results:

Depth.....	0.57 ft
Velocity.....	1.42 fps
Flow Area.....	3.24 sf
Flow Top Width...	11.39 ft
Wetted Perimeter.	11.45 ft
Critical Depth...	0.42 ft
Critical Slope...	0.0302 ft/ft
Froude Number....	FULL

FROM CIRCULAR CHANNEL ANALYSIS &
DESIGN MODEL, HAESTAD METHODS:

$$Q_{MAX.} = 1.69 \text{ CFS} > Q_6$$

∴ STORM SEWER CAPACITY IS ADEQUATE

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

RWA 15/15
7135.001A
9/22/05

Worksheet Name: Riverfront Ind. Park

Comment: Proposed Northern Area Storm Sewer

Solve For Full Flow Capacity

Given Input Data:

Diameter.....	1.00 ft
Slope.....	0.0030 ft/ft
Manning's n.....	0.015
Discharge.....	1.69 cfs

Computed Results:

Full Flow Capacity.....	1.69 cfs
Full Flow Depth.....	1.00 ft
Velocity.....	2.15 fps
Flow Area.....	0.79 sf
Critical Depth....	0.55 ft
Critical Slope....	.00086 ft/ft
Percent Full.....	100.00 %
Full Capacity.....	1.69 cfs
QMAX @.94D.....	1.82 cfs
Froude Number.....	FULL

ATTACHMENT 6

STORMWATER QUALITY MANAGEMENT CALCULATIONS

RIVERFRONT INDUSTRIAL PARK
DRAINAGE REPORT - STORM-
WATER QUALITY MANAGEMENT

RWS 1/11
4135.001/A
10/12/05

1. LOOK @ OUTLET 1, SERVING DRAINAGE
BASIN A₂ THROUGH PROPOSED SWALE:

FROM URBAN STORM DRAINAGE CRITERIA
MANUAL, VOLUME 3 - BEST MANAGEMENT
PRACTICES, 1999 URBAN DRAINAGE
AND FLOOD CONTROL DISTRICT (UDFCD),
GRASS SWALE SEDIMENTATION FACILITY,
SUITABLE FOR SMALL SITE,

ASSUME: 1-YEAR 15-MIN. RAINFALL EVENT
AS CONSERVATIVE "INITIAL STORM" FLUSHING
EVENT PER UDFCD GUIDELINES AND AVAIL.
STEAMBOAT SPRINGS RAINFALL DATA.

FROM RAINFALL INTENSITY/DURATION/
FREQUENCY CURVE FOR STEAMBOAT
SPRINGS (ATTACHMENT 1), $I = 0.88 \text{ in/hr.}$

$$\therefore Q = CIA = (0.9)(0.88)(0.36) = 0.29 \text{ CFS}$$

ASSUME: TRIANGULAR-SHAPED SWALE
CHANNEL W/ DRYLAND GRASSES ($n = 0.025$),
 $s = 0.004 \text{ ft./ft.}$, $z = 4.0 (H/V)$, $D \approx 1 \text{ ft.}$

FROM TRIANGULAR CHANNEL ANALYSIS
& DESIGN MODEL, HASTAD METHODS:

ACTUAL DEPTH @ 0.29 CFS = 0.27 FT. < 1 FT.
< 2 FT. MAX.

ALSO: FROUDE NUMBER = 0.47
< 0.5 MAX.

VELOCITY = 0.98 FT/SEC.
< 1.5 FPS MAX.

$\therefore$ SWALE IS ADEQUATE FOR DESIGN FLOW.

Triangular Channel Analysis & Design
Open Channel - Uniform flow

RWS 2/11
4135.0014
10/12/05

Worksheet Name: Riverfront Ind. Park

Comment: Proposed Grass Swale

Solve For Depth

Given Input Data:

Left Side Slope..	4.00:1 (H:V)
Right Side Slope.	4.00:1 (H:V)
Manning's n.....	0.025
Channel Slope....	0.0040 ft/ft
Discharge.....	0.29 cfs

Computed Results:

Depth.....	0.27 ft
Velocity.....	0.98 fps
Flow Area.....	0.30 sf
Flow Top Width...	2.18 ft
Wetted Perimeter.	2.25 ft
Critical Depth...	0.20 ft
Critical Slope...	0.0204 ft/ft
Froude Number....	0.47 (flow is Subcritical)

CHECK FOR PEAK RUNOFF
DESIGN FLOW OF 25-YEAR,
15-MIN. STORM EVENT:

RMS 3/11
4/35,001A
10/12/05

FROM ATTACHMENT 5, $Q_2 = 0.9$ CFS

FROM TRIANGULAR CHANNEL ANALYSIS
& DESIGN MODEL, HASTAN METHODS:

ACTUAL DEPTH @ 0.9 CFS = 0.4 FT. < 1 FT.

$\therefore$ WALL IS ADEQUATE FOR DESIGN FLOW

Triangular Channel Analysis & Design
Open Channel - Uniform flow

PWS 4/11
4135.00/A
10/12/05

Worksheet Name: Riverfront Ind. Park

Comment: Proposed Grass Swale

Solve For Depth

Given Input Data:

Left Side Slope..	4.00:1 (H:V)
Right Side Slope.	4.00:1 (H:V)
Manning's n.....	0.025
Channel Slope....	0.0050 ft/ft
Discharge.....	0.90 cfs

Computed Results:

Depth.....	0.40 ft
Velocity.....	1.41 fps
Flow Area.....	0.64 sf
Flow Top Width...	3.20 ft
Wetted Perimeter.	3.30 ft
Critical Depth...	0.32 ft
Critical Slope...	0.0176 ft/ft
Froude Number....	0.56 (flow is Subcritical)

PNW 5/11
4135.001A
10/12/05

2. LOOK @ OUTLET 2 SERVING DRAINAGE BASINS A4 AND A6 THROUGH PROPOSED EXTENDED DETENTION BASIN, SUITABLE FOR SITE, 0.5 - 2% OF TRIBUTARY AREA,

$$\text{ASSUME: DESIGN VOLUME} = \left(\frac{WQCV}{12} \right) (A_4 + A_6) (1.2)$$

WQCV BASED ON 40-MIN. DRAIN TIME

$$WQCV = d_u \frac{WQCV_{\text{DENVER}}}{0.43}$$

$d_u \approx 0.35$ FOR STEAMBOAT SPRINGS, CO

FROM UDFCD FOR $i = I_{WK}/100 = 0.9$
(TOTAL IMPERVIOUS RATIO)

$$WQCV_{\text{DENVER}} = 0.4 \text{ WATERSHED - INCHES}$$

$$\Rightarrow WQCV = (0.35) \left(\frac{0.4}{0.43} \right) = 0.33 \text{ WATERSHED - IN.}$$

$$\therefore V = \left(\frac{0.33}{12} \right) (1.36 + 0.09) (1.2) = 0.05 \text{ AC-FT}$$

ASSUME: 3:1 LENGTH TO WIDTH RATIO

$$\text{AVG. DEPTH} = 4 \text{ FT.}$$

$$\Rightarrow A = \frac{(0.05 \text{ AC-FT}) (43,520 \text{ SQ-FT/AC})}{4 \text{ FT.}} = 545 \text{ SQ. FT.}$$

ASSUME: WIDTH = 15 FT.

$$\Rightarrow \text{LENGTH} = 36 \text{ FT.}$$

$\therefore$ BASIN IS ADEQUATE FOR DESIGN FLOW

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4135.00/A
10/12/05

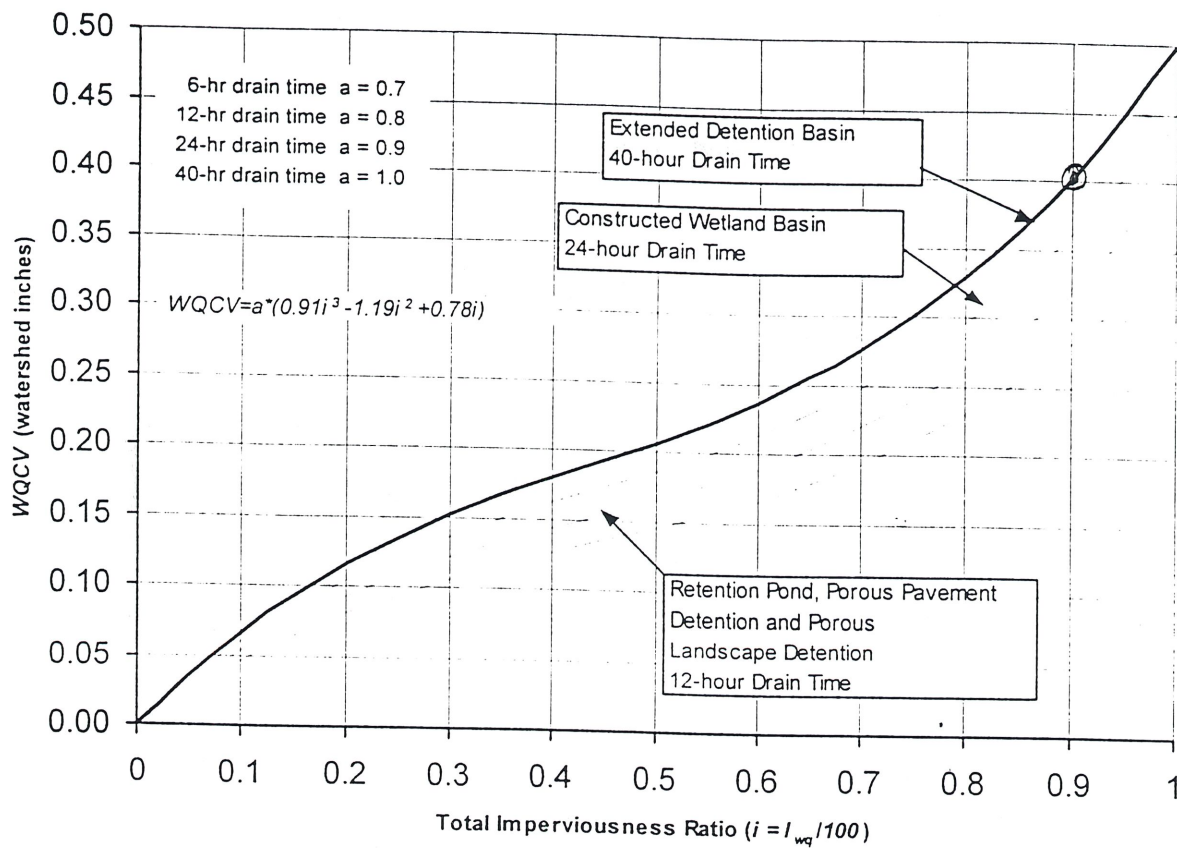


Figure EDB-2—Water Quality Capture Volume (WQCV), 80th Percentile Runoff Event

RWS 7/11
4135.001A
10/12/05

3. LOOK @ OUTLET 3 SERVING DRAINAGE
BASINS A, AND A5 THROUGH EXISTING
WETLANDS CHANNEL

ASSUME: TOTAL CONTRIBUTORY DRAINAGE
BASIN AREA = $\frac{31,500 \text{ SQ. FT.}}{43,560 \text{ AC/SF}} + A_1 + A_5$

$$= 0.72 + 0.83 + 1.05 \text{ AC.}$$

$$= 2.6 \text{ AC.}$$

(INCLUDES AREA BETWEEN
PROJECT SITE AND ELK
RIVER RD. WHERE $C = 0.3$
PER ATTACHMENT 2)

2-YEAR 6-HOUR RAINFALL EVENT,
 $I = 0.15 \text{ IN/HR.}$ PER 1, ABOVE

$$\therefore Q = (0.3)(0.72)(0.15) + (0.9)(0.83 + 1.05)(0.15) = 0.3 \text{ CFS}$$

ASSUME: TRAPEZOIDAL CHANNEL CROSS-
SECTION WITH $Z = 5.0 (H/V)$
 $D \approx 3 \text{ FT.}$, $S = 0.01$, 'BOTTOM'
WIDTH $\approx 20 \text{ FT.}$

FROM TRAPEZOIDAL CHANNEL ANALYSIS
& DESIGN MODEL, MAESTRO METHOD:

ACTUAL DEPTH @ 0.3 CFS = 0.06 FT. < 3 FT.,
OK 2-4 FT. MAX.

ALSO: VELOCITY = 0.27 FT./SEC.
OK 2.0 FPS MAX.

$\therefore$ WETLANDS CHANNEL IS ADEQUATE
FOR DESIGN FLOW.

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

RWS 8/11
\$135.00/A
10/12/05

Worksheet Name: Riverfront Ind. Park

Comment: Existing Wetlands Channel

Solve For Depth

Given Input Data:

Bottom Width.....	20.00 ft
Left Side Slope..	5.00:1 (H:V)
Right Side Slope.	5.00:1 (H:V)
Manning's n.....	0.080
Channel Slope....	0.0100 ft/ft
Discharge.....	0.30 cfs

Computed Results:

Depth.....	0.06 ft
Velocity.....	0.27 fps
Flow Area.....	1.12 sf
Flow Top Width...	20.55 ft
Wetted Perimeter.	20.56 ft
Critical Depth...	0.02 ft
Critical Slope...	0.3498 ft/ft
Froude Number....	0.20 (flow is Subcritical)

CHECK FOR PEAK RUNOFF
DESIGN FLOW OF 25-YEAR,
15-MIN. STORM EVENT:

RWS 9/11
\$135.00/A
10/12/05

FROM ATTACHMENT 5, $Q_1 + Q_5 = 2.0 + 2.6 \text{ CFS}$
 $= 4.6 \text{ CFS}$

$$\Rightarrow \text{TOTAL FLOW} = (0.3)(2.7)(0.72) + 4.6 \text{ CFS} \\ = 5.2 \text{ CFS}$$

(INCLUDES AREA BETWEEN PROJECT
SITE AND ELK RIVER RD., $C = 0.3$)

FROM TRAPEZOIDAL CHANNEL ANALYSIS
& DESIGN MODEL, HAESTRA METHODS:

ACTUAL DEPTH @ 5.2 CFS = 0.3 FT. < 3 FT.
 < 2-FT. MAX.

ALSO: VELOCITY = 0.8 FT./SEC
 < 2.0 FPS MAX.

$\therefore$ WETLANDS CHANNEL IS ADEQUATE
FOR DESIGN FLOW.

ASSUME: TRIBUTARY FLOW OF ABOVE
REQUIRED TO BE MAINTAINED OFF-SITE,
FROM WETLANDS TO YAMPA RIVER

18" ϕ CMP OR EQUIVALENT MIN.
TO BE INSTALLED UNDER EMERGENCY
ACCESS CONNECTOR AT SOUTHWEST
CORNER OF PROJECT SITE.

FROM FIG. 3.26, AISI:

$$\frac{H_w}{D} = 0.62$$

$$\Rightarrow \text{DEPTH} \approx 15 \text{ IN.}$$

$\therefore$ CULVERT CAPACITY IS ADEQUATE

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

RWS 10/11
4135.001A
10/12/05

Worksheet Name: Riverfront Ind. Park

Comment: Existing Wetlands Channel

Solve For Depth

Given Input Data:

Bottom Width.....	20.00 ft
Left Side Slope..	5.00:1 (H:V)
Right Side Slope.	5.00:1 (H:V)
Manning's n.....	0.080
Channel Slope....	0.0100 ft/ft
Discharge.....	5.20 cfs

Computed Results:

Depth.....	0.30 ft
Velocity.....	0.80 fps
Flow Area.....	6.51 sf
Flow Top Width...	23.03 ft
Wetted Perimeter.	23.09 ft
Critical Depth...	0.13 ft
Critical Slope...	0.1880 ft/ft
Froude Number....	0.26 (flow is Subcritical)