DRAINAGE REPORT

FOR

AG MINOR SUBDIVISION PHASE II

5550 E Virginia St

OWNER: INDUSTRIAL CONTRACTORS, INC.
P.O. BOX 208
EVANSVILLE, IN 47702

BY

ENGINEER: MORLEY AND ASSOCIATES, INC.
605 S.E. MARTIN LUTHER KING, JR. BLVD.
EVANSVILLE, INDIANA 47713-1797
(812) 464-9585

OCTOBER 10, 1994

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AG MINOR SUBDIVISION

NARRATIVE

Existing conditions at this + 6.5 acre site include flat cultivated farm land which has been recently cultivated, a parking lot, and a building. Because this area is extremely flat, it is not anticipated that drainage will enter the development from off sight areas. Therefore, the drainage calculations were developed using the area bounded by the development plan which is 6.51 acres. 2.31 acres are shown as undeveloped on the master plan for this site. Refer to the original master plan drainage report prepared by Morley and Associates, Inc. on December 10, 1993. This undeveloped portion of the site is now being addressed in this report. Currently, the storm water from the undeveloped portion of the site is conveyed by a swale along the north line of the property to Stockfleth Ditch approximately 1000 feet east of the proposed Phase II site.

The proposed development will detain all storm water on site, while at the same time releasing storage at a controlled rate not to exceed the undeveloped run-off rate. The retention basin covers approximately 12,500 square feet of land at normal pool elevation. The capacity required for storing developed run-off generated by a 25 year storm is 27,347 cubic feet. The existing retention basin has enough capacity to store the developed runoff of Phase II. Undeveloped storm run-off rates were calculated using a runoff coefficient of 0.20 which corresponds with the

Vanderburgh County Drainage Board approved master plan of the entire basin.

The preliminary design for the interior storm drainage system followed criteria for a reoccurrence interval of 10 years. Due to grade restrictions, pipes large enough to convey a 10 year storm where not always possible. Because of this, the largest pipes that could be used with respect to the grade restrictions were used. This will create brief ponding around the inlets during a 10 year event. The site will be graded to allow a maximum ponding depth of 0.3 feet. Water above this depth will leave the site by means of overland sheet flow. The 25 year storm will be conveyed by pipes in a surcharged condition.

Calculation for an orifice on the outlet pipe were also performed. If the outlet pipe were allowed to flow into a system without any tailwater, it would need an orifice. However, with a 25 year storm event, it is believed that the storm sewer system would be filled with tailwater. Therefore, freeflow conditions would not exist in the system. The tailwater in the system will in effect act as an orifice and limit the flow discharge. On a site inspection during a 25 year storm event, we observed that Stockfleth Ditch flows full at an elevation = 386.5 feet. With the ditch at this level the maximum outflow discharge will be 2.42 cfs. Therefore, the tailwater would allow less discharge than what is actually allowed by the system. This amount being the difference between undeveloped and developed discharge rates. For this reason no orifice was placed on the outlet pipe.

SECTION I

Undeveloped

Formulas

Kerby Formula (1959) $\Rightarrow$ $t_c = K(LNS^{-.5})^{.467}$ p. 3-8 HERPICC Manual

$K = 0.83$ L = length of flow = 440'

S = average slope (ft/ft) = 0.0009 = 0.09%

N = retardance roughness coeff. = .20

p. 3-8 HERPICC Manual

$$t_c = 0.83((440')(.20)(.0009)^{-.5})^{.467} = 34.54 \text{ minutes}$$

Vanderburgh County Drainage Ordinance

Rainfall IDF Table for Evansville $\Rightarrow$ Table 807

$$\text{Intensity} = 3.226 - (4.54/30)(3.226 - 1.819) = 3.013 \text{ in/hr}$$

Rational Method for runoff:

Section 802 County Ordinance

$Q = ciA$

c = Table 803 = 0.20 i = 3.013 in/hr A = 2.31 acres

$$Q_{10} = 0.20(3.013 \text{ in/hr})(2.31 \text{ acres}) = 1.39 \text{ cfs}$$

Developed

C_d = parking lot = .92 C_d = areas outside subbasins = .15

Composite $C_d = \dfrac{.92(1.88) + .15(.43)}{2.31} = .78$

Areas

\# 4A = 36,095 ft^2 \# 4B = 27,360 ft^2 \# 4C = 18,293 ft^2

Total = 1.88 acres

$Q_d = c\,i\,A = .15(3.013)(.43) = .20$ cfs ⇐ runoff outside of basins

therefore, total site runoff = 1.39 − .20 = 1.19 cfs

SECTION II

PROJECT: AG MINOR SUBDIVISION PHASE II DATE: 95 06 21
ENGINEER:MORLEY AND ASSOCIATES,INC

DESIGN RETURN PERIOD:	5|25d
RELEASE RATE PERIOD:	5|25d
WATERSHED AREA (ACRES):	1.8687
TIME OF CONCENTRATION(UNDEVELOPED):	34.54
RAINFALL INTENSITY (INCHES/HR):	3.013
UNDEVELOPED RUNOFF COEFFICIENT:	.2
UNDEVELOPED RUNOFF RATE (CFS):	1.19
DEVELOPED RUNOFF COEFFICIENT:	.92

25 YEAR STORM

STORM DURATION (HRS)	RAINFALL INTENSITY (INCH/HR)	INFLOW RATE (CFS)	OUTFLOW RATE (CFS)	STORAGE RATE (CFS)	REQUIRED STORAGE (ACRE-FT)
.08	6.85	11.78	1.19	10.59	.071
.17	5.45	9.37	1.19	8.18	.116
.25	4.65	7.99	1.19	6.80	.142
.33	4.15	7.13	1.19	5.94	.163
.42	3.80	6.53	1.19	5.34	.187
.50	3.40	5.85	1.19	4.66	.194
.58	3.20	5.50	1.19	4.31	.208
.67	2.85	4.90	1.19	3.71	.207
.75	2.75	4.73	1.19	3.54	.221
.83	2.60	4.47	1.19	3.28	.227
.92	2.45	4.21	1.19	3.02	.232
1.00	2.30	3.95	1.19	2.76	.230
1.25	2.05	3.52	1.19	2.33	.243
1.50	1.85	3.18	1.19	1.99	.249
1.75	1.60	2.75	1.19	1.56	.228
2.00	1.40	2.41	1.19	1.22	.203
2.50	1.25	2.15	1.19	.96	.200
3.00	1.10	1.89	1.19	.70	.175
4.00	.84	1.44	1.19	.25	.085

PEAK STORAGE (ACRE/FT): .25
PEAK STORAGE (CUBIC FT): 10,838

Storage

Required volume from spreadsheet = 10,838 ft^3

Volume available in phase I detention basin = 12,160 ft^3

Therefore, adequate storage is provided and no new pond is needed

Outflow

$$Q = A_p \left[\frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.87\, n^2 L}{D^{4/3}}} \right]^{1/2} = .785 \left[\frac{2}{\left(\frac{.15+1}{2(32.2)}\right) + \left(\frac{2.87(.012)^2 70}{(1)^{4/3}}\right)} \right]^{1/2}$$

$Q = 4.88$ cfs

Phase I $Q = 1.72$ cfs   Phase II = 1.19 cfs

Combined = 1.72 + 1.19 = 2.91 cfs

Outflows exceed maximum, therefore an orifice is needed if there is no tailwater.

Orifice calculation ⇒ $Q = C_d A \sqrt{2g(h-a)}$

Q = flow (cfs) C_d = loss coefficient A = area of pipe (ft^2)
g = gravity h = dist. from I.E. to pool a = half the outlet ht.

$Q = .6(.4096673)\sqrt{2(32.2)(2.5-.3611)} = 2.88$ cfs < 2.91 cfs. ✓

Use $8\frac{2}{3}$" diameter steel plate orifice, if no tailwater

Morley and Associates

American General Subdivision Phase 2 94–2940–1
Calculations of Retention Basin Outflow at Different Elevation Scenarios

Elevation of Retention Basin
387.00

Elevation of Stockfleth Ditch	Outflow (cfs)
384.00	5.93
384.50	5.41
385.00	4.84
385.50	4.19
386.00	3.42
386.50	2.42

Elevation of Retention Basin
386.50

Elevation of Stockfleth Ditch	Outflow (cfs)
384.00	5.41
384.50	4.84
385.00	4.19
385.50	3.42
386.00	2.42
386.50	0.00

Elevation of Retention Basin
386.00

Elevation of Stockfleth Ditch	Outflow (cfs)
384.00	4.84
384.50	4.19
385.00	3.42
385.50	2.42
386.00	0.00
386.50	ERR

Elevation of Retention Basin
385.50

Elevation of Stockfleth Ditch	Outflow (cfs)
384.00	4.19
384.50	3.42
385.00	2.42
385.50	0.00
386.00	ERR
386.50	ERR

SECTION III

Sub-basins

#4C 18,293 ft^2 = 0.4120 acres C = .92 10 yr. storm

Length = 144' N = 0.02 Slope = .0069 = .69%

$t_c = .83(144(.02)(.0069)^{-.5})^{.467} = 4.34$ min ∴ use 5 min minimum

$i_{10} = 6.625$ in/hr $Q_{@C} = CiA = .92(6.625)(.4120) = \underline{\underline{2.51 \text{ cfs}}}$

#4B 27,360 ft^2 = 0.6281 acres C = .92 10 yr. storm

Length = 140' N = 0.02 Slope = .0084 = .84%

$t_c = .83(140'(.02)(.0084)^{-.5})^{.467} = 4.09$ min ∴ use 5 min minimum

travel time from STR # 710 to # 711 = $^{120'}/_{3.25}\left(^{1 m}/_{60 sec}\right)$ = .62 min.

Compare #4B t_c to (#4C t_c + travel time)

⇒ 5 min < (5 min + .667 min) ∴ use t_c = 5.62 min

$i_{10} = 6.625 - (^{.62}/_5)(6.625 - 5.380) = 6.471$ in/hr

$Q_{@B} = .92(6.458)(.6281 + .4120) = \underline{\underline{6.19 \text{ cfs}}}$

4A

36, 095 ft^2 = 0.8286 acres   C = .92   10 yr storm

Length = 150'   N = .02   slope = .00783 = 0.783%

$t_c = .83\left(150(.02)(.00783)^{-.5}\right)^{.467} = 4.30$ min ∴ use 5 min minimum

travel time from structure # 710 to # 712 = $^{240'}/_{3.25}\left(^{1 min}/_{60 sec}\right)$ = 1.29. min

Compare # 4A to (# 4C + travel time)

⇒ 5 min < (5 min + 1.33 min) ∴ use t_c = 6.29 min

$i_{10} = 6.625 - (^{1.29}/_5)(6.625 - 5.380) = 6.30$ in/hr

$Q_{@A} = .92(6.30)(.6281 + .4120 + .8286) =$ 10.81 acres

SECTION IV

Morley and Associates

Project # 2718-1 Date

Engineer J.E.M. Line # 1

Upstream Str. # 710 Downstream Str. # 711

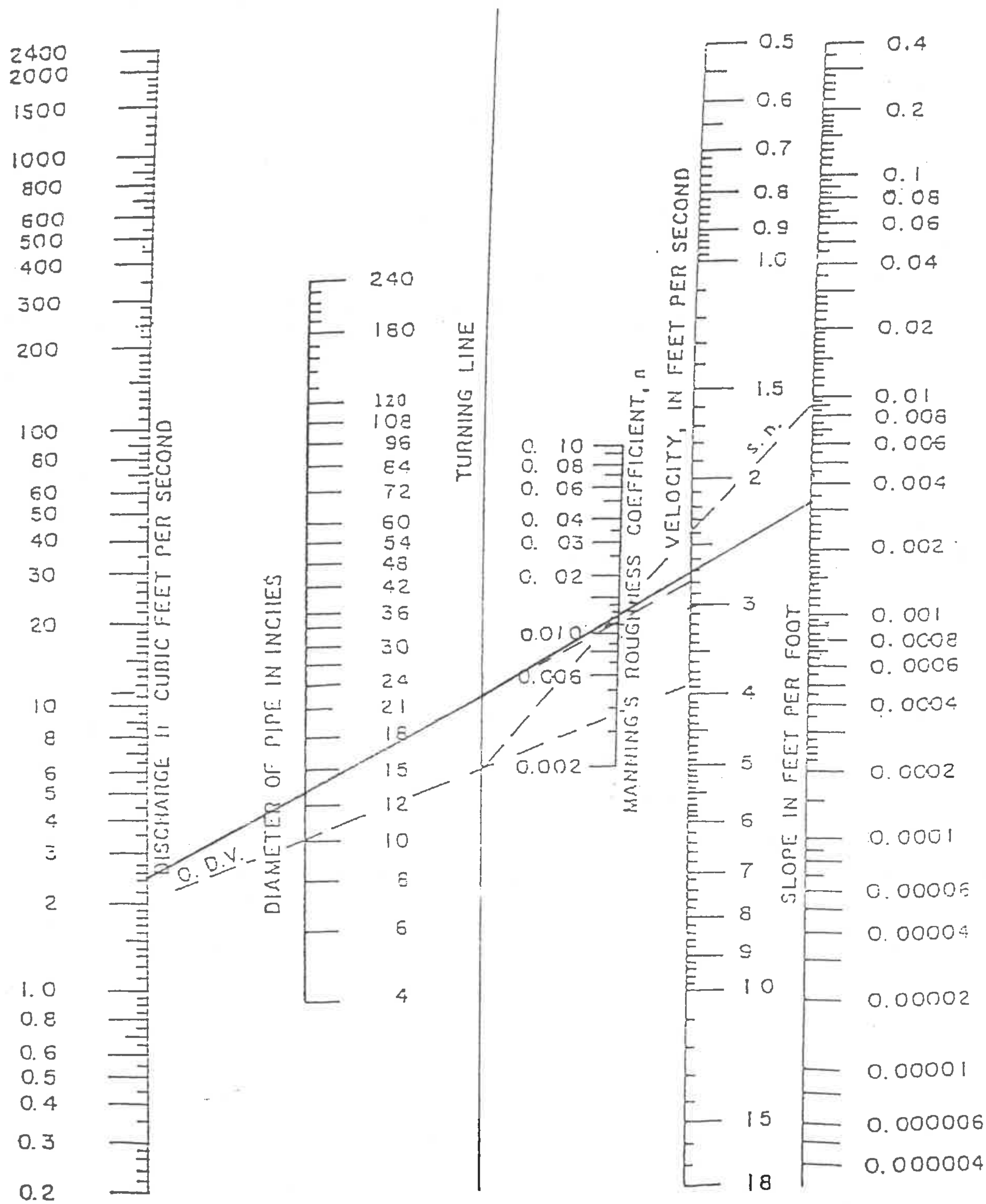
Type of Pipe Concrete Storm Duration 10 yr

7-415.04 0

JAN 1971

NOMOGRAPH FOR SOLUTION OF MANNING'S

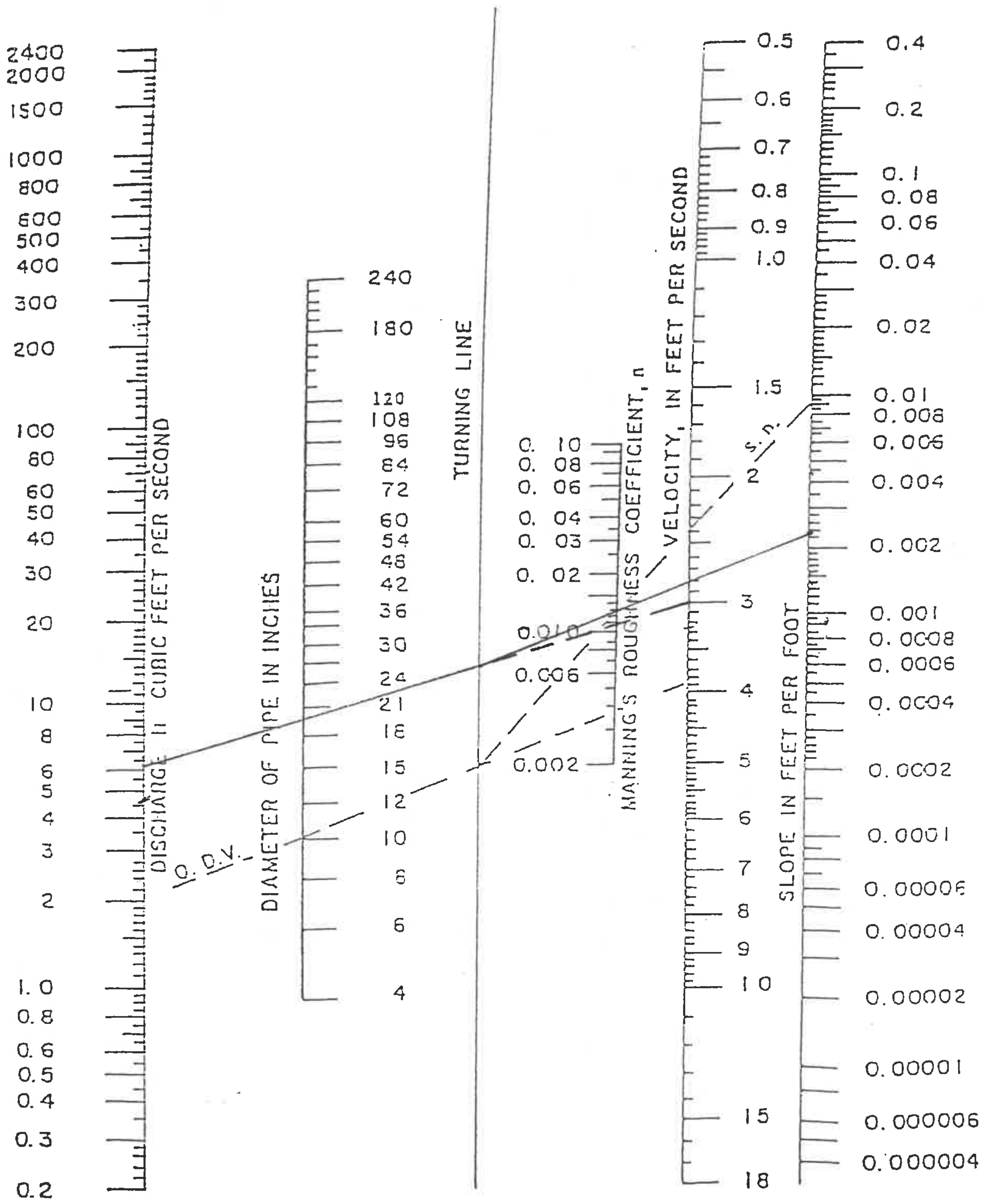
FORMULA FOR FLOW IN STORM SEWERS



Morley and Associates
Project # 2718-1 Date
Engineer JEM Line # 2
Upstream Str. # 711 Downstream Str. # 712
Type of Pipe Concrete Storm Duration 10

7-415.04 0
JAN 1971

NOMOGRAPH FOR SOLUTION OF MANNING'S
FORMULA FOR FLOW IN STORM SEWERS



Morley and Associates

Project # 2718-1 Date

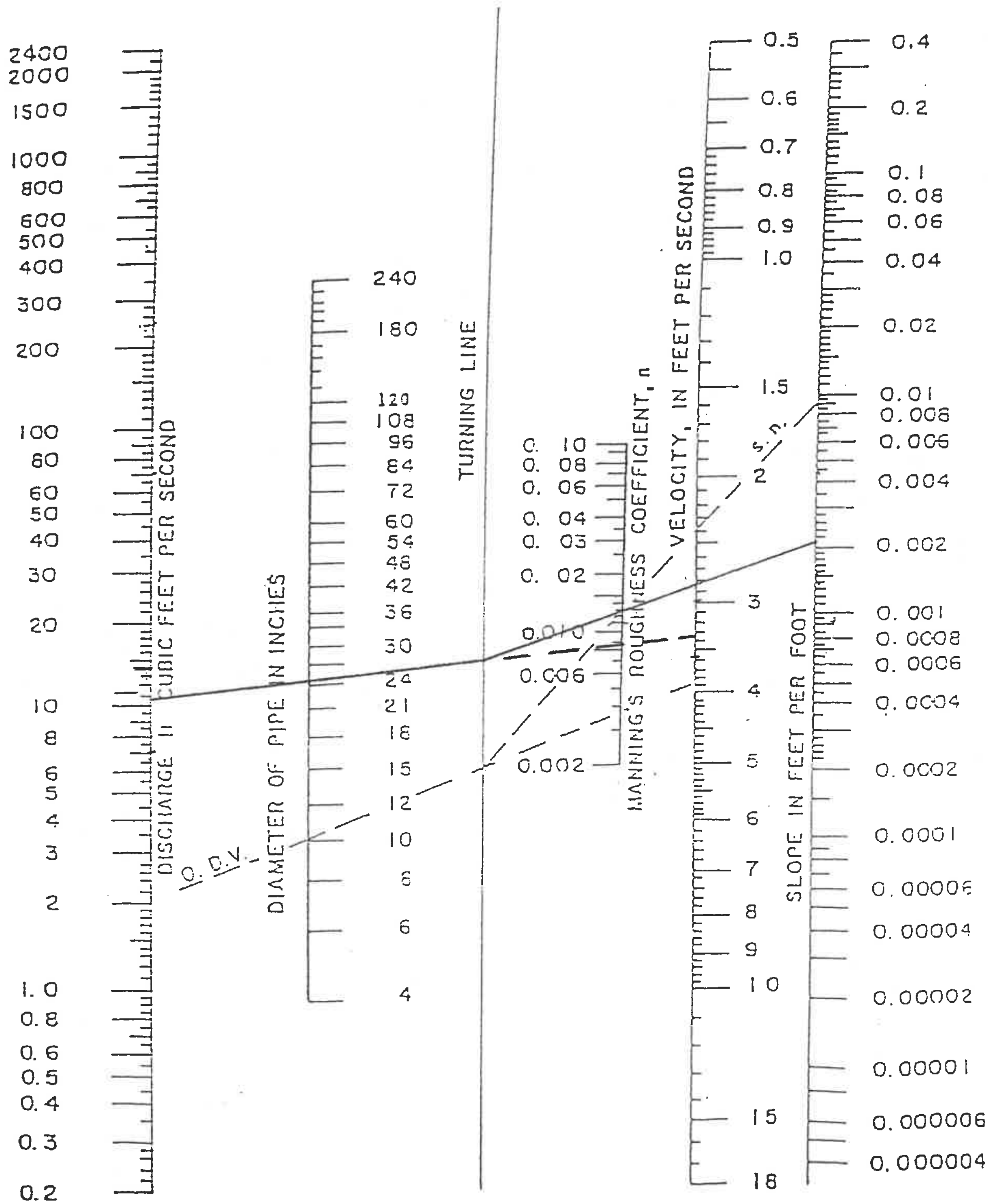
Engineer JEM Line # 3

Upstream Str. # 712 Downstream Str. # 713

Type of Pipe Concrete Storm Duration 10 yr

7-415.04 0
JAN 1971

NOMOGRAPH FOR SOLUTION OF MANNING'S
FORMULA FOR FLOW IN STORM SEWERS



STORM SEWER DESIGN SHEET – RATIONAL METHOD

PROJECT AG Minor Subdivision DATE ______ SHEET 1 OF 1

ENGINEER Morley + Assoc. DESIGN STORM 10 MANNINGS n 0.13

Line Number	Upstream Manhole	Downstream Manhole	Length (Ft)	C_j	A_j (Acres)	C_jA_j	ΣA_jC_j	t_j (min)	t_{cum} (min)	i [inches/hr]	Q (CFS)	Pipe Diameter (inches)	Pipe Slope (%)	Pipe Capacity (CFS)	Velocity (Ft/Sec)	Travel Time (min)	Rim Elevation Upstream	Rim Elevation Downstream	Invert Elevation Upstream	Invert Elevation Downstream	Pipe Cover Upstream	Pipe Cover Downstream
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	710	711	120	.92	.4120	.38	—	5	—	6.625	2.52	15	.325	3.3	3.0	.67	387.5	387.5	385.45	385.06	.72	1.11
2	711	712	120	.92	.6281	.58	.96	5	5.67	6.471	6.21	21	.255	7.5	3.25	.62	387.5	387.5	385.06	384.75	.61	.92
3	712	713	120	.92	.8286	.76	1.722	5	6.33	6.30	10.85	27	.208	14.0	3.6	.56	387.5	—	384.75	384.50	.42	—

℄ Pipes installed due to grade restrictions

Line Number	Upstream Manhole	Downstream Manhole	Length (Ft)	C_j	A_j (Acres)	C_jA_j	ΣA_jC_j	t_j (min)	t_{cum} (min)	i [inches/hr]	Q (CFS)	Pipe Diameter (inches)	Pipe Slope (%)	Pipe Capacity (CFS)	Velocity (Ft/Sec)	Travel Time (min)	Rim Elevation Upstream	Rim Elevation Downstream	Invert Elevation Upstream	Invert Elevation Downstream	Pipe Cover Upstream	Pipe Cover Downstream
1	710	711	120	.92	.4120	.38	—	5	5	6.625	2.52	15	.325	3.3	3.0	.67	387.5	387.5	385.45	385.06		
2	711	712	120	.92	.6281	.58	.96	5	5.67	6.471	6.21	18	.255	4.6	2.82	.71	387.5	387.5	385.06	384.75		
3	712	713	120	.92	.8286	.76	1.722	5	6.33	6.30	10.85	21	.208	7.1	3	.67	387.5	—	384.75	384.50		

Figure 7.1 Storm Sewer Design Sheet – Rational Method

APPENDIX

TABLE 807

RAINFALL INTENSITY-DURATION-FREQUENCY TABLE FOR EVANSVILLE

INTENSITY IN INCHES PER HOUR

STORM DURATION	STORM RETURN PERIOD IN YEARS				
	5	10	25	50	100
5 MIN	6.063	6.625	7.208	7.936	8.469
10 MIN	4.863	5.380	5.925	6.616	7.126
15 MIN	4.029	4.515	5.033	5.697	6.194
30 MIN	2.837	3.226	3.646	4.194	4.608
60 MIN	1.549	1.819	2.078	2.412	2.663
2.0 HRS	1.053	1.230	1.400	1.620	1.785
3.0 HRS	0.774	0.899	1.019	1.175	1.291
4.0 HRS	0.632	0.736	0.836	0.965	1.062
5.0 HRS	0.524	0.606	0.684	0.785	0.861
6.0 HRS	0.453	0.522	0.589	0.676	0.741
7.0 HRS	0.399	0.459	0.516	0.591	0.647
8.0 HRS	0.358	0.412	0.463	0.530	0.581
9.0 HRS	0.323	0.370	0.415	0.472	0.516
10 HRS	0.297	0.339	0.379	0.431	0.470
11 HRS	0.276	0.314	0.351	0.399	0.435
12 HRS	0.259	0.296	0.331	0.376	0.410
13 HRS	0.245	0.280	0.314	0.357	0.390
14 HRS	0.233	0.267	0.299	0.341	0.372
15 HRS	0.220	0.252	0.281	0.320	0.349
16 HRS	0.209	0.238	0.266	0.302	0.329
17 HRS	0.198	0.225	0.251	0.284	0.310