# COMPREHENSIVE DRAINAGE STUDY

## FOR THE CITY OF EVANS

## VOLUME II
## CITY OF EVANS DRAINAGE CRITERIA MANUAL

ROCKY MOUNTAIN CONSULTANTS, INC.
825 Delaware Avenue, Suite 500
Longmont, CO 80501

MARCH 1997

# SECTION 1.0
# GENERAL REQUIREMENTS

# SECTION 1 - GENERAL REQUIREMENTS

## 1.1 PURPOSE

The purpose of the "City of Evans Storm Drainage Criteria Manual" (ESDCM) is to provide minimum design and specification criteria for the analysis and design of future storm drainage facilities within the city of Evans and the areas within its Urban Growth Boundary. All subdivision, residential, commercial, and industrial developments shall include adequate storm drainage system design. All designs must meet the requirements set forth in the ESDCM.

## 1.2 RELATIONSHIP TO OTHER CRITERIA

The ESDCM is to be used in conjunction with the Urban Storm Drainage Criteria Manual (USDCM). Policies and technical criteria not specifically addressed in this document shall be in accordance with the most recent version of the USDCM. If the government imposes stricter criteria or standards, the city's criteria shall be amended to reflect the most restrictive standards.

The Weld County Subdivision Ordinance and City of Greeley Storm Drainage Design Criteria Manual (GSDCM) were used as a guideline for the ESDCM. Many of the illustrations and specifications were taken from the GSDCM. It was determined that Evans should adopt criteria similar to Greeley's to provide continuity in the control, treatment, and disposal of stormwater. If a discrepancy in criteria, regulations, or requirements develop, the most restrictive standard shall apply.

## 1.3 POLICY

### 1.3.1 Evans Town Ditch

It is the policy of the City of Evans that stormwater flows in excess of the historic flow rates will not be allowed to be discharged into the Evans Town Ditch. Use of the ditch as a stormwater conveyance system for historic flows is discouraged. As development occurs in the Urban Growth Area, the drainage channels should be modified so that stormwater bypasses the ditch and is discharged in the South Platte River floodplain. Developments within the city shall make necessary modifications to ensure that excess stormwater does not discharge into the ditch.

A plan for addressing storm drainage water quality has been required of large municipalities since 1991. Eventually, the same criteria will be applied to smaller cities. Point source discharges, such as the Evans Town Ditch, will be required to meet minimum water quality standards before release into the South Platte River. Therefore, it is the policy of the City of Evans to discourage inflow of any storm drainage into the Evans Town Ditch which may have a deleterious effect on the quality of water in the ditch. Provisions to improve the water quality before the storm drainage enters the ditch may be required if deemed appropriate by City staff.

Other discussions regarding the Evans Town Ditch are shown in Appendix 2.

### 1.3.2 Storm Drainage Fees

For the purpose of providing adequate stormwater conveyance systems, the fees shall be set by the Evans City Council. A discussion of fee options is set forth in Appendix 2.

# SECTION 2.0
# SUBMITTAL REQUIREMENTS

# SECTION 2 - SUBMITTAL REQUIREMENTS

## 2.1 REVIEW AND ACCEPTANCE

Drainage reports and plans, construction drawings, special provisions, and calculations submitted to the City of Evans for review must be prepared by or under the supervision of a Professional Engineer licensed in Colorado. The City's review and proposal will only be to determine if the submittals conform to the City's requirements. The City's approval does not relieve the design engineer or the contractor from responsibility or liability for the design or construction of a project.

Approval of the submittal information shall be valid for one year after the acceptance date. If construction of the project has not started within that period, the acceptance by the City will be invalid.

## 2.2 PRELIMINARY DRAINAGE REPORT

The purpose of the preliminary drainage report is to identify and define conceptual solutions to existing or future drainage problems that result from the proposed development. The preliminary drainage report shall be reviewed and signed by a professional engineer licensed in Colorado.

The preliminary drainage report will include a Project Information and Drainage Report and a Site Drainage Plan. The Project Information and Drainage Report shall be submitted on 8-1/2" x 11" paper and bound. The plans shall be on 24"x36" paper. Two copies of the report and plan will be submitted to the City for review. One copy will be returned with comments for revision. As a minimum, the preliminary drainage report will include the following information:

2.2.1 Project Information and Drainage Report

A. General location and Description

1. Location of the proposed development with respect to adjacent public and private roads.
2. Legal description of property location including Township, Range, Section and 1/4 Section.
3. Names of surrounding developments within ½ mile of the proposed development.
4. Area in acres.
5. Ground cover (type of trees, shrubs, vegetation).
6. General topography.
7. General soil conditions.

8. Major drainageways on the property.
9. Irrigation facilities and laterals within the property.

B. Drainage Basins and Sub-basins
   1. Reference of any major drainageway planning study, such as master drainage plans, flood hazard delineation reports, and flood insurance studies.
   2. Discussion of major basin drainage characteristics, including historical and planned land use and basin slope.
   3. Irrigation facilities and laterals that will affect or be affected by the local drainage.
   4. Discussion of the historic drainage pattern of the proposed development.
   5. Discussion of off-site flow patterns and their potential impact on the proposed development.

C. Drainage Criteria
   1. Refer to all criteria, master plans, and technical information used in support of the drainage facility design concept.
   2. Identify provisions by section number for which a waiver or variance is requested.
   3. Provide justification for each waiver or variance.
   4. Identify design rainfall and storm recurrence interval.
   5. Identify runoff calculation method.
   6. Identify detention storage and discharge calculation method.
   7. Identify various capacity references.

D. Drainage Facility Design
   1. Discussion of compliance with off-site runoff considerations.
   2. Discussion of anticipated and proposed drainage patterns.
   3. Discussion of the content of tables, charts, figures, plates, or drawings presented in the report.
   4. Presentation of existing and proposed hydrologic conditions with approximate flow rates entering and exiting the proposed development.
   5. Presentation of approach to accommodate drainage impacts of existing or proposed improvements and facilities.
   6. Presentation of proposed drainage facilities with respect to alignment, material, structure type, and size.
   7. Discussion of opportunities for integration of other services (recreational, natural resource) within drainage facilities.
   8. Discussion of stormwater quality control concepts.
   9. Discussion of maintenance access aspects of the design.

E. Wetland Determination and Review (if applicable).

F. Conclusions

1. Compliance with City criteria, City Comprehensive Drainage Plan, and USDCM.
2. Effectiveness of drainage concept to control storm runoff.

G. References

1. Reference all criteria and technical information.

H. Appendices

1. All hydrologic computations.
2. All hydraulic computations. (Optional for preliminary, but required in final submittal.)

2.2.2 Site Drainage Plan(s)

A. General Location Map

1. A 24"x36" map shall be provided in sufficient detail to identify general drainage patterns and drainage flows entering and exiting the proposed development for at least 100' from the project boundaries. The map shall identify any major facilities (e.g. development, irrigation ditches, existing detention facilities, culverts, and storm sewers) that will influence or be influenced by the proposed subdivision.

B. Drainage Plan (24"x36")

1. Property lines and easements with purposes noted.
2. Location and elevations of 100 year floodplain limits.
3. Existing and (if available)proposed contours at an interval not to exceed 2'.
4. Site flow arrows delineating the direction of site flows.
5. Paths chosen for times of concentration.
6. Design point designations.
7. Routing and accumulation of flows at various critical points for the minor and major storm runoffs associated with the development.
8. Overall area boundary and sub-basin boundaries, areas, and names.
9. Locations of off-site "inflow" and "outflow".
10. Location and type of pertinent facilities relevant to the proposed development including water features ( ponds, streams, irrigation ditches, etc.), existing buildings, streets, and roads.
11. Existing drainage facilities with all pertinent information such as material, size, shape, slope and locations included.
12. Proposed drainage facilities (e.g. manholes, storm pipes, detention ponds, inlets, culverts, open drainageways, rip-rap, and other appurtenances.)
13. Detention pond volume, grading, and outlet design (if applicable).
14. Proposed development features (e.g. building footprint, hard surfacing, etc.).

15. Streets indicating ROW width, flowline width, curb type, sidewalk, and approximate street slopes.
16. Proposed erosion control measures.
17. Detail sheet showing drainage control features proposed for the project.
18. Any off-site feature influencing the development.

## 2.3 FINAL DRAINAGE REPORT

The purpose of the final drainage report is to update the concepts and to present the design details for the drainage facilities presented in the preliminary drainage report. The final drainage report must address any changes to the preliminary design concept and any questions or comments made during the review of the preliminary submittal. The final drainage report shall be reviewed and signed by a professional engineer licensed in Colorado. The report shall be properly certified and signed by such engineer.

The final drainage report shall include all the requirements of the preliminary report. In addition, the final report shall include the following:

1. Presentation of an accurate, complete, and current estimate of cost for proposed facilities.
2. A statement by the owner/developer relieving the City of liability for drainage system design or construction and certifying that drainage facilities will be constructed according to the design in the final drainage report.

## 2.4 CONSTRUCTION PLANS

When drainage improvements are to be constructed, final construction plans (24"x36") shall be submitted with the final drainage plan. Construction plans shall be signed and registered by a professional engineer. Two copies of the report and plan will be submitted to the City for review. One copy will be returned with comments for revision. Once the revisions are made, the original set of plans, one set of plans on reproducible mylar and one additional copy will be submitted to the city for final acceptance and approval. The original set will be signed and returned to the originator. Issuance of the necessary construction permits is contingent on the approval of the construction plans by the City.

After approval of the final construction plans, any changes in plans or specifications must be approved by the City. These changes will be noted on the as-built drawings.

The construction plan set shall include, but is not limited to, the following:

A. General Details
   1. Title block.
   2. Scale and legend.
   3. Date and revisions block.
   4. Name of firm and professional engineer with the professional engineer's stamp.
   5. Approval block.

B. Master Utility Plan
   1. Proposed storm drain lines.
   2. Property lines.
   3. Existing and proposed easements and right-of-ways.
   4. Street and alley names.
   5. Proposed utilities.
   6. Existing utilities on and adjacent to the site.
   7. Topographic features (houses, curbs, water courses, etc.).
   8. North arrow.

C. Construction plans and profiles
   1. Key map.
   2. Existing utilities.
   3. Proposed and existing easements, right-of-ways, and property lines.
   4. Diameter, type, and length of pipe of proposed storm drain lines.
   5. Depth, elevation, slope, manhole invert, and rim elevations on proposed storm drain lines.
   6. Horizontal and vertical relationship of the storm drain to the other proposed and existing utilities.
   7. Existing and proposed ground profile.
   8. Matchlines indicating references to next sheets of design.
   9. Tie downs to the center of the street.
   10. Survey stations.
   11. North arrow.
   12. Horizontal and vertical scales.

D. Details Sheet
   1. Critical connections.
   2. Crossings.
   3. Special fittings and appurtenances.

# SECTION 3.0
# STORM DRAINAGE IMPROVEMENTS

# SECTION 3 - STORM DRAINAGE IMPROVEMENTS

## 3.1 GENERAL DESIGN CRITERIA

1. In general, the procedure, criteria, and standards set forth in the Denver Regional council of Governments' "Urban Storm Drainage Criteria Manual" (USDCM) will be used for storm drainage system analysis. The criteria outlined within shall supersede the USDCM.

2. The runoff analysis for a site shall be based on the zoned land use for that area. The analysis shall include contributing runoff from upstream areas. The contributing runoff shall be based on:
   a) the existing land use if the subject area is not anticipated for annexation by the City of Evans or;
   b) ultimate developed land use if the area is within the City of Evans or is anticipated for annexation into the City or;
   c) topographic characteristics of those areas.
   All runoff calculations shall be based on the master drainage plan.

3. Natural topographic features shall be used as the basis for locating drainage easements and runoff calculations. Average land slopes may be utilized in runoff computations (see the USDCM for detailed methods of computing runoff). Wherever existing drainage patterns and slopes are defined, such as by streets with curb and gutter, these patterns and slopes shall be used.

4. Streets shall not be used as a primary floodway for storm runoff.

5. Natural drainageways are to be used whenever feasible. Past experience has shown that stormwater drainage systems perform better and have fewer problems when they follow the existing natural drainageways. Alteration to natural drainage patterns will be considered if investigation and analysis can show no hazard or environmental degradation will result from the proposed construction.

6. Drainage systems shall not be designed to transfer the excess stormwater from one location to another. System design must not create a more hazardous condition downstream of the site. Each design shall include provisions for the 100-year storm to pass through the site at historic discharge levels. Detention facilities shall be designed to contain runoff in excess of the historical 100-year storm.

7. Drainage systems shall be designed to keep stormwater flows within the existing drainage basin.

8. All drainage improvements shall be made to appear as natural as possible. This includes the use of linear park systems complete with bike paths, park areas, and open space. Linear park systems shall be incorporated into the drainage system design whenever open channels or natural drainageways are used.

9. Irrigation ditches shall not be used as the future outfall of any drainage basin or sub-basin. Developments within a basin or sub-basin whose drainage currently outfalls into an irrigation ditch must design the drainage system to prevent increased discharge into that ditch. Future discharges into irrigation facilities shall not exceed the historic discharge rates for that drainage area.

10. Drainage system design should consider and not impair surface or subsurface drainage.

## 3.2 RAINFALL

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 1 "Rainfall" section.

1. Two design storms shall be investigated for each development. Both a minor storm and a major storm will be investigated. The minor storm occurs as fairly regular intervals. It is not typically the cause of excessive damage, but results in higher costs in maintenance, repair, and replacement of facilities if not handled adequately. Proper handling of the major storm can eliminate substantial property damage or loss of life.

2. The minor storm is based on the 2-year recurrence interval discharge for residential developments, up to the 10-year recurrence interval discharge for commercial developments. The major storm is based on the 100-year recurrence interval discharge for all developments. The minor storm shall be used for the design and analysis of proposed stormwater features. Major drainageways and detention facilities shall be designed based on the major storm. The minor and major recurrence intervals to be used based on land use are shown below.

DESIGN STORM FREQUENCIES

| Land Use | Design Storm Period |
|---|---|
| Residential | 5 years |
| Open Space | 5 years |
| Commercial | 10 years |
| Public Buildings | 10 years |
| Industrial | 10 years |
| Road Crossings Conducting Drainage | |
| Local Road | 10 years |
| Collector Road | 25 years |
| Arterial Road | 50 years |
| Natural Drainage | 25 years |

3. Rainfall intensities to be used in the rational method computation of runoff shall be obtained from the Greeley Intensity-Duration-Frequency Curves which are included in Figure 3.2-1, the corresponding tabulated values are included in Table 3.2-1.

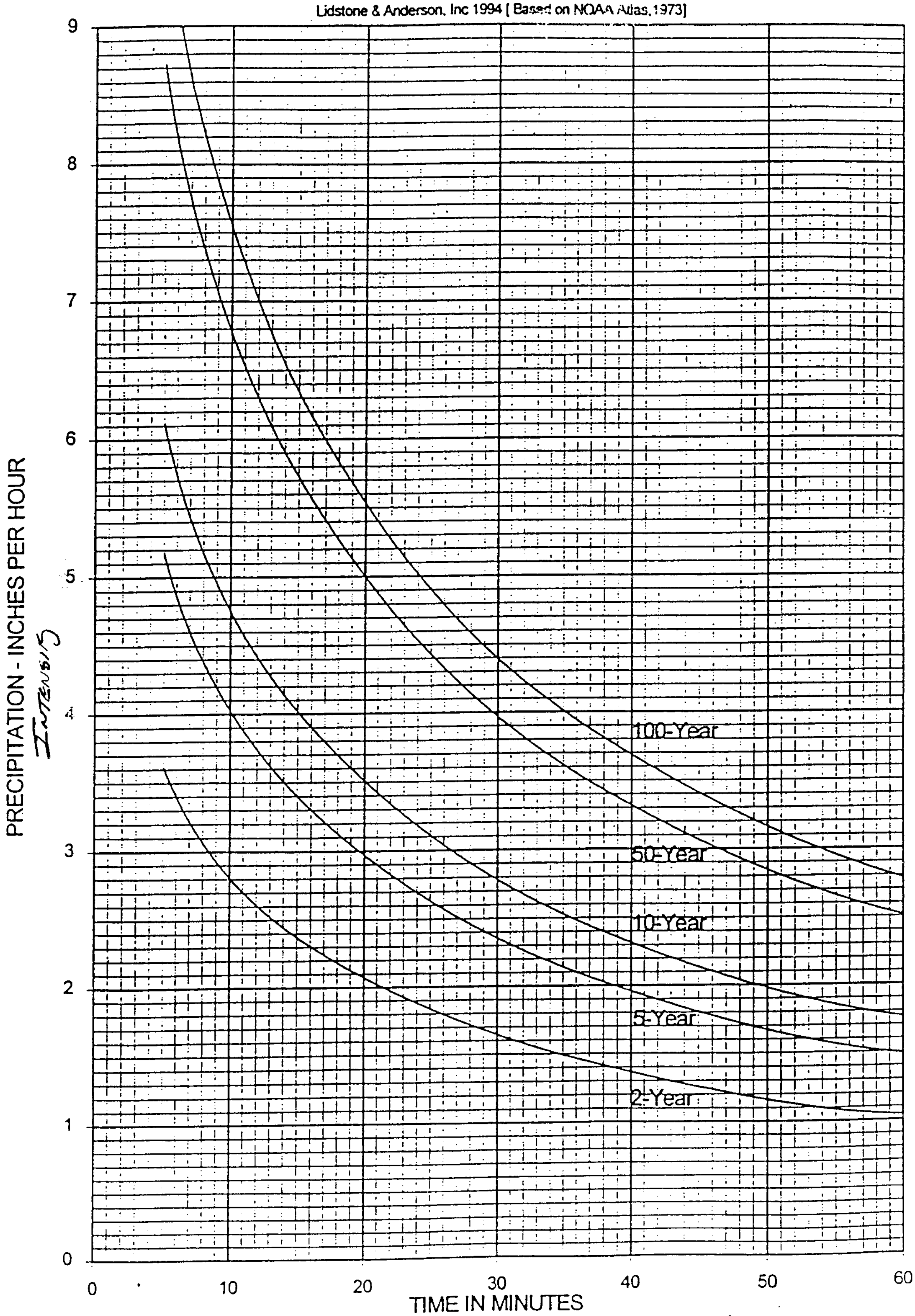
4. One-hour point rainfall values to be used with the CUHP method of analysis shall be obtained from the Greeley One-Hour Point Rainfall values shown below:

| ONE-HOUR POINT RAINFALL (INCHES) | | | | |
|---|---|---|---|---|
| 2-year | 5-year | 10-year | 50-year | 100-year |
| 1.04 | 1.49 | 1.76 | 2.51 | 2.78 |

5. For analysis of watersheds greater than 5 square miles, the design storm duration and rainfall values must be adjusted to account for the averaging effects of larger watersheds. The incremental rainfall distribution for all basin areas up to 20 square miles shall be based on the Greeley values and are included in Table 3.2-2.

Figure 3.2-1
INTENSITY - DURATION - FREQUENCY CURVES
EVANS, COLORADO

Lidstone & Anderson, Inc 1994 [ Based on NOAA Atlas, 1973]






$T_c$ → TIME OF CONCENTRATION

Table 3.2.1 Extended Duration - Intensity - Frequency Tabulation

| Storm Duration | Storm Frequency 2-year (in/hr) | 5-year (in/hr) | 10-year (in/hr) | 25-year (in/hr) | 50-year (in/hr) | 100-year (in/hr) |
|---|---|---|---|---|---|---|
| 5 min | 3.62 | 5.19 | 6.12 | 7.31 | 8.73 | 9.67 |
| 10 | 2.81 | 4.02 | 4.75 | 5.67 | 6.78 | 7.51 |
| 15 | 2.37 | 3.4 | 4.01 | 4.79 | 5.72 | 6.34 |
| 20 | 2 | 2.86 | 3.38 | 4.03 | 4.81 | 5.34 |
| 25 | 1.77 | 2.54 | 3 | 3.58 | 4.28 | 4.74 |
| 30 | 1.64 | 2.35 | 2.78 | 3.22 | 3.97 | 4.39 |
| 40 | 1.34 | 1.92 | 2.27 | 2.7 | 3.23 | 3.59 |
| 50 | 1.16 | 1.66 | 1.96 | 2.34 | 2.8 | 3.1 |
| 60 (1 hr) | 1.04 | 1.49 | 1.76 | 2.1 | 2.51 | 2.78 |
| 80 | 0.8 | 1.14 | 1.47 | 1.61 | 1.91 | 2.16 |
| 100 | 0.67 | 0.94 | 1.2 | 1.3 | 1.58 | 1.79 |
| 120 (2 hr) | 0.58 | 0.8 | 0.96 | 1.14 | 1.3 | 1.5 |
| 150 | 0.49 | 0.66 | 0.78 | 0.93 | 1.1 | 1.23 |
| 180 (3 hr) | 0.42 | 0.56 | 0.67 | 0.8 | 0.92 | 1.05 |
| 4 hr | 0.33 | 0.44 | 0.53 | 0.62 | 0.72 | 0.81 |
| 5 | 0.27 | 0.36 | 0.43 | 0.5 | 0.57 | 0.66 |
| 6 | 0.23 | 0.3 | 0.37 | 0.43 | 0.49 | 0.57 |
| 8 | 0.2 | 0.24 | 0.29 | 0.34 | 0.39 | 0.44 |
| 10 | 0.15 | 0.2 | 0.24 | 0.29 | 0.32 | 0.36 |
| 12 | 0.13 | 0.17 | 0.2 | 0.25 | 0.28 | 0.31 |
| 14 | 0.11 | 0.15 | 0.18 | 0.23 | 0.24 | 0.27 |
| 16 | 0.1 | 0.13 | 0.16 | 0.2 | 0.22 | 0.24 |
| 18 | 0.09 | 0.12 | 0.14 | 0.18 | 0.19 | 0.21 |
| 20 | 0.08 | 0.11 | 0.13 | 0.17 | 0.18 | 0.19 |
| 22 | 0.07 | 0.1 | 0.12 | 0.16 | 0.16 | 0.17 |
| 24 | 0.07 | 0.09 | 0.11 | 0.14 | 0.15 | 0.16 |

## DESIGN STORMS FOR EVANS

### INCREMENTAL RAINFALL DEPTH/RETURN PERIOD

| TIME (Min) | BASINS LESS THAN 5 SQ. MILES | | | | | BASINS BETWEEN 5 AND 10 SQ. MILES | | | | | BASINS BETWEEN 10 AND 20 SQ. MILES | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2-YR (in) | 5-YR (in) | 10-YR (in) | 50-YR (in) | 100-YR (in) | 2-YR (in) | 5-YR (in) | 10-YR (in) | 50-YR (in) | 100-YR (in) | 2-YR (in) | 5-YR (in) | 10-YR (in) | 50-YR (in) | 100-YR (in) |
| 5 | 0.02 | 0.03 | 0.04 | 0.03 | 0.03 | 0.02 | 0.03 | 0.04 | 0.03 | 0.03 | 0.02 | 0.03 | 0.04 | 0.03 | 0.03 |
| 10 | 0.04 | 0.06 | 0.07 | 0.09 | 0.08 | 0.04 | 0.06 | 0.07 | 0.09 | 0.08 | 0.04 | 0.06 | 0.07 | 0.09 | 0.08 |
| 15 | 0.09 | 0.13 | 0.14 | 0.13 | 0.13 | 0.09 | 0.13 | 0.14 | 0.13 | 0.13 | 0.09 | 0.13 | 0.14 | 0.13 | 0.13 |
| 20 | 0.17 | 0.23 | 0.26 | 0.20 | 0.22 | 0.16 | 0.22 | 0.25 | 0.20 | 0.22 | 0.15 | 0.21 | 0.24 | 0.20 | 0.22 |
| 25 | 0.26 | 0.37 | 0.44 | 0.38 | 0.39 | 0.25 | 0.36 | 0.42 | 0.36 | 0.37 | 0.23 | 0.34 | 0.40 | 0.34 | 0.35 |
| 30 | 0.15 | 0.19 | 0.21 | 0.63 | 0.70 | 0.14 | 0.19 | 0.20 | 0.60 | 0.67 | 0.13 | 0.17 | 0.19 | 0.57 | 0.63 |
| 35 | 0.07 | 0.09 | 0.10 | 0.30 | 0.39 | 0.07 | 0.09 | 0.01 | 0.29 | 0.37 | 0.07 | 0.09 | 0.10 | 0.27 | 0.35 |
| 40 | 0.05 | 0.07 | 0.08 | 0.20 | 0.22 | 0.05 | 0.07 | 0.08 | 0.20 | 0.22 | 0.05 | 0.07 | 0.08 | 0.20 | 0.22 |
| 45 | 0.03 | 0.05 | 0.07 | 0.13 | 0.17 | 0.03 | 0.05 | 0.07 | 0.13 | 0.17 | 0.03 | 0.05 | 0.07 | 0.13 | 0.17 |
| 50 | 0.03 | 0.05 | 0.06 | 0.08 | 0.14 | 0.03 | 0.05 | 0.06 | 0.08 | 0.14 | 0.03 | 0.05 | 0.06 | 0.08 | 0.14 |
| 55 | 0.03 | 0.05 | 0.06 | 0.08 | 0.11 | 0.03 | 0.05 | 0.06 | 0.08 | 0.11 | 0.03 | 0.05 | 0.06 | 0.08 | 0.11 |
| 60 | 0.03 | 0.05 | 0.06 | 0.08 | 0.11 | 0.03 | 0.05 | 0.06 | 0.08 | 0.11 | 0.03 | 0.05 | 0.06 | 0.08 | 0.11 |
| 65 | 0.03 | 0.05 | 0.06 | 0.06 | 0.11 | 0.03 | 0.05 | 0.06 | 0.06 | 0.11 | 0.03 | 0.05 | 0.06 | 0.06 | 0.11 |
| 70 | 0.02 | 0.05 | 0.06 | 0.06 | 0.06 | 0.02 | 0.05 | 0.06 | 0.06 | 0.06 | 0.02 | 0.05 | 0.06 | 0.06 | 0.06 |
| 75 | 0.02 | 0.04 | 0.06 | 0.05 | 0.06 | 0.02 | 0.04 | 0.06 | 0.05 | 0.06 | 0.02 | 0.04 | 0.06 | 0.05 | 0.06 |
| 80 | 0.02 | 0.03 | 0.04 | 0.05 | 0.03 | 0.02 | 0.03 | 0.04 | 0.05 | 0.03 | 0.02 | 0.03 | 0.04 | 0.05 | 0.03 |
| 85 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 |
| 90 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 |
| 95 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 | 0.02 | 0.03 | 0.03 | 0.04 | 0.03 |
| 100 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 |
| 105 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 |
| 110 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 | 0.02 | 0.02 | 0.03 | 0.04 | 0.03 |
| 115 | 0.01 | 0.02 | 0.03 | 0.04 | 0.03 | 0.01 | 0.02 | 0.03 | 0.04 | 0.03 | 0.01 | 0.02 | 0.03 | 0.04 | 0.03 |
| 120 | 0.01 | 0.02 | 0.02 | 0.04 | 0.03 | 0.01 | 0.02 | 0.02 | 0.04 | 0.03 | 0.01 | 0.02 | 0.02 | 0.04 | 0.03 |
| 125 | | | | | | | | | | | 0.01 | 0.01 | 0.02 | 0.02 | 0.02 |
| 130 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.02 | 0.02 |
| 135 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.02 |
| 140 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| 145 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| 150 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| 155 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| 160 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| 165 | | | | | | | | | | | 0.01 | 0.01 | 0.01 | 0.01 | 0.01 |
| 170 | | | | | | | | | | | 0.00 | 0.00 | 0.01 | 0.00 | 0.01 |
| 175 | | | | | | | | | | | 0.00 | 0.00 | 0.00 | 0.00 | 0.01 |
| 180 | | | | | | | | | | | 0.00 | 0.00 | 0.00 | 0.00 | 0.01 |
| TOTAL | 1.20 | 1.72 | 2.04 | 2.81 | 3.21 | 1.18 | 1.69 | 2.00 | 2.76 | 3.15 | 1.24 | 1.73 | 2.05 | 2.79 | 3.22 |

DATE: NOV. 21, 1994

REV:

REFERENCE: MILLER, J.F., AND TRACEY, R.J
PRECIPITATION-FREQUENCY ANALYSIS OF THE WESTERN UNITED STATES
(NOAA ATLAS) VOLUME III - COLORADO 1973

## 3.3 RUNOFF

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 1 "Runoff" section.

1. The Rational Method of runoff analysis may be used for basins less than 160 acres in size. Procedures for the Rational Method and applicable runoff coefficients are presented in the USDCM Volume 1. See Section 3.2 of this report for the required rainfall values.

2. For basins greater than 160 acres the Colorado Urban Hydrograph Procedure (CUHP) method of runoff analysis is required. The CUHP method is recommended for basins greater than 90 acres, but is not required. Detailed explanation of the CUHP procedures is presented in the USDCM Volume 1. The design storms to be used with CUHP are presented in Section 3.2 of this report.

3. For runoff analysis using CUHP, the computer versions, CUHPD and CUHP/PC, may be used to calculate hydrographs. These programs may be obtained by contacting the Urban Storm Drainage and Flood Control District.

4. When channel routing procedures are necessary, computer programs, such as the EPA Stormwater Management Model (SWMM), are recommended but not required. Channel routing methodology is explained in the USDCM Volume 1.

## 3.4 STORM SEWERS

The following design criteria are in additions to and clarifications of the requirements and recommendations set forth in the USDCM Volume 1 "Storm Sewers" section.

1. Storm sewers shall be designed to convey the initial storm peaks. All hydraulic losses shall be considered in the computations.

2. The Manning's "n" values to be used in the calculations of storm sewer capacity are presented in Table 3.4-1.

3. Losses due to changes in pipe size, branches, bends, junctions, expansions, and contractions shall be calculated using the following equation:

$$H_L = K\,(V^2/2g)$$

Where:
$H_L$ = head loss (feet)
$K$ = loss coefficient
$V^2/2g$ = velocity head (feet)
$g$ = gravitational acceleration (32.2 ft/sec$^2$)

Loss coefficients for various flow conditions are presented in Table 3.4-2.

4. The head loss due to expansions shall be calculated using the following equation:

$$H_L = K_e\,(V_1^2/2g)\,[\,1 - A_1/A_2]^2$$

Where:
$K_e$ = contraction/expansion coefficient
$V$ = average flow velocity
$A$ = cross section area
Subscripts 1 and 2 represent upstream and downstream sections, respectively.

Expansion loss coefficients for various flow conditions are shown in Table 3.4-2.

5. The head loss due to contractions shall be calculated using the following equation:

$$H_L = K_C\,(V_2^2/2g)\,[\,1-(A_2)^2/A_1]^2$$

Contraction loss coefficients for various flow conditions are shown in Table 3.4-2.

6. The head loss for bends shall be calculated using the following equation:

$$H_L = K_b (V^2/2g)$$

Where: $K_b$ = bend coefficient

Recommended bend coefficients are presented in Tables 3.4-3 thru 3.4-4.

7. The head loss through a junction or manhole shall be calculated using the following equation:

$$H_L = (V_2^2/2g) - K_J (V_1^2/2g)$$

Where: $K_J$ = junction loss coefficient
$V_2$ = outfall flow velocity
$V_1$ = inlet velocity

Junction loss coefficients are presented in Table 3.4-5.

8. Figures 3.4-1 thru 3.4-3 have been developed for calculating the hydraulic properties for various pipe shapes.

9. Sewer grade shall be such that a minimum cover is maintained to withstand AASHTO HS-20 loading on the pipe. Cover shall be no less than 12 inches at any point along the pipe.

10. The minimum clearance between storm sewer and water main shall be 12 inches. Concrete encasement of the water line will be required for clearances less than 12 inches.

11. The minimum clearance between storm sewer and sanitary sewer shall be 12 inches. When a sanitary sewer lies above the storm sewer or within 18 inches below the storm sewer, the sanitary sewer shall have an impervious encasement or be constructed of structural sewer pipe for a minimum of 10 feet on each side of where the storm sewer crosses.

12. Storm sewers may be constructed with curvilinear alignment for 48 inch diameter and larger pipe. The limitations and parameters for alignment are shown in Table 3.4-1.

13. Minimum pipe size for storm sewers (except for detention outlets) is shown in Table 3.4-1.

14. Maintenance and access easements shall be as follows:

| REQUIRED STORM SEWER MAINTENANCE AND ACCESS EASEMENTS | |
|---|---|
| Storm Sewer Diameter | Easement Width |
| Less than 36" | 20' |
| Equal to or greater than 36" | 25' (With sewer at the 1/3 point in the easement) |

15. Manholes or maintenance access ports shall be required at changes in size, direction, elevation, grade, or where there is a junction of two or more sewers. The maximum spacing between manholes is outlined in Table 3.4-1. Required manhole sizes are shown below:

| MANHOLE SIZE | |
|---|---|
| Sewer Diameter | Manhole Diameter |
| 15" to 18" | 4' |
| 21" to 42" | 5' |
| 48" to 54" | 6' |
| 60" or larger | CDOT Standard M-604-20 and -21 |

Table 3.4-1

| VERTICAL DIMENSION OF PIPE (INCHES) | MAXIMUM ALLOWABLE DISTANCE BETWEEN MANHOLES AND/OR CLEANOUTS |
|---|---|
| 15 TO 36 | 400 FEET |
| 42 AND LARGER | 500 FEET |

MINIMUM RADIUS FOR RADIUS PIPE

| DIAMETER OF PIPE | RADIUS OF CURVATURE |
|---|---|
| 48" TO 54" | 28.50 FEET |
| 57" TO 72" | 32.00 FEET |
| 78" TO 108" | 38.00 FEET |

SHORT RADIUS BENDS SHALL NOT BE USED ON SEWERS 42 INCHES OR LESS IN DIAMETER

MINIMUM PIPE DIAMETER

| TYPE | MINIMUM EQUIVALENT PIPE DIAMETER | MINIMUM CROSS-SECTIONAL AREA |
|---|---|---|
| MAIN TRUNK<br>LATERAL FROM INLET | 18 INCHES | 1.77 SQ. FT. |

*MINIMUM SIZE OF LATERAL SHALL ALSO BE BASED UPON A WATER SURFACE INSIDE THE INLET WITH A MINIMUM DISTANCE OF 1 FOOT BELOW THE GRATE OR THROAT

MANNING'S N-VALUE

| SEWER TYPE | CAPACITY CALCULATION | VELOCITY CALCULATION |
|---|---|---|
| CONCRETE (NEWER PIPE) | .013 | .011 |
| CONCRETE (OLDER PIPE) | .015 | .012 |
| CONCRETE (PRELIMINARY SIZING) | .015 | .012 |
| PLASTIC | .011 | .009 |



STORM SEWER ALIGNMENT AND SIZE CRITERIA

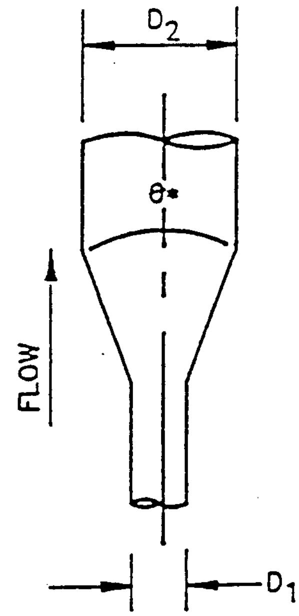
Table 3.4-2

# EXPANSION/CONTRACTION

(a) EXPANSION ($K_e$)

| θ* | $\frac{D_2}{D_1}=3$ | $\frac{D_2}{D_1}=1.5$ |
|---|---|---|
| 10 | 0.17 | 0.17 |
| 20 | 0.40 | 0.40 |
| 45 | 0.86 | 1.06 |
| 60 | 1.02 | 1.21 |
| 90 | 1.06 | 1.14 |
| 120 | 1.04 | 1.07 |
| 180 | 1.00 | 1.00 |

* THE ANGLE θ IS THE ANGLE IN DEGREES BETWEEN THE SIDES OF THE TAPERING SECTION




(b) PIPE ENTRANCE FROM RESERVOIR

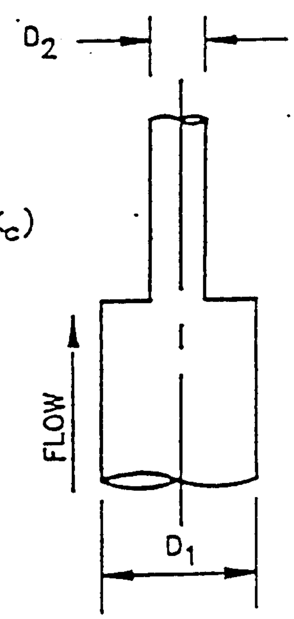
BELL–MOUTH $H_L=0.04\frac{V2}{2g}$

SQUARE EDGE $H_L=0.5\frac{V2}{2g}$

GROOVE END U/S FOR CONCRETE PIPE $H_L=0.2\frac{V2}{2g}$

(c) CONTRACTION ($K_c$)

| $\frac{D_2}{D_1}$ | $K_c$ |
|---|---|
| 0 | 0.5 |
| 0.4 | 0.4 |
| 0.6 | 0.3 |
| 0.8 | 0.1 |
| 1.0 | 0 |






## STORM SEWER ENERGY LOSS COEFFICIENT

# STORM SEWER ENERGY LOSS COEFFICIENT

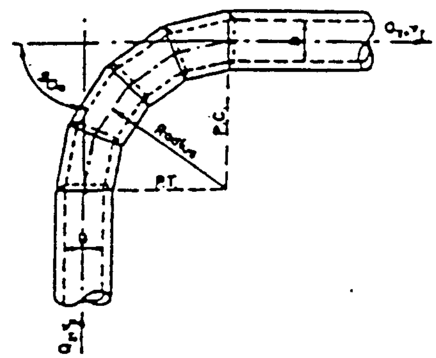
(BENDS)

## CASE I
## CONDUIT ON 90° CURVES

NOTE: Head loss applied at P.C. for length

$$K_b = 0.25\sqrt{\frac{\theta}{90}}$$

| θ | $K_b$ |
|---|---|
| 90 | 0.25 |
| 60 | 0.20 |
| 45 | 0.18 |
| 30 | 0.14 |




## CASE II
## BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at begining of bend

| θ° BEND | $K_b$ |
|---|---|
| 90 | 0.50 |
| 60 | 0.43 |
| 45 | 0.35 |
| 22-1/2 | 0.20 |

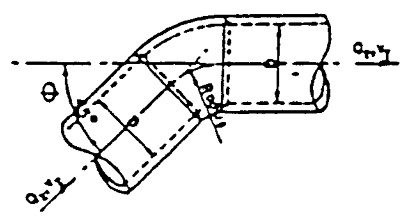
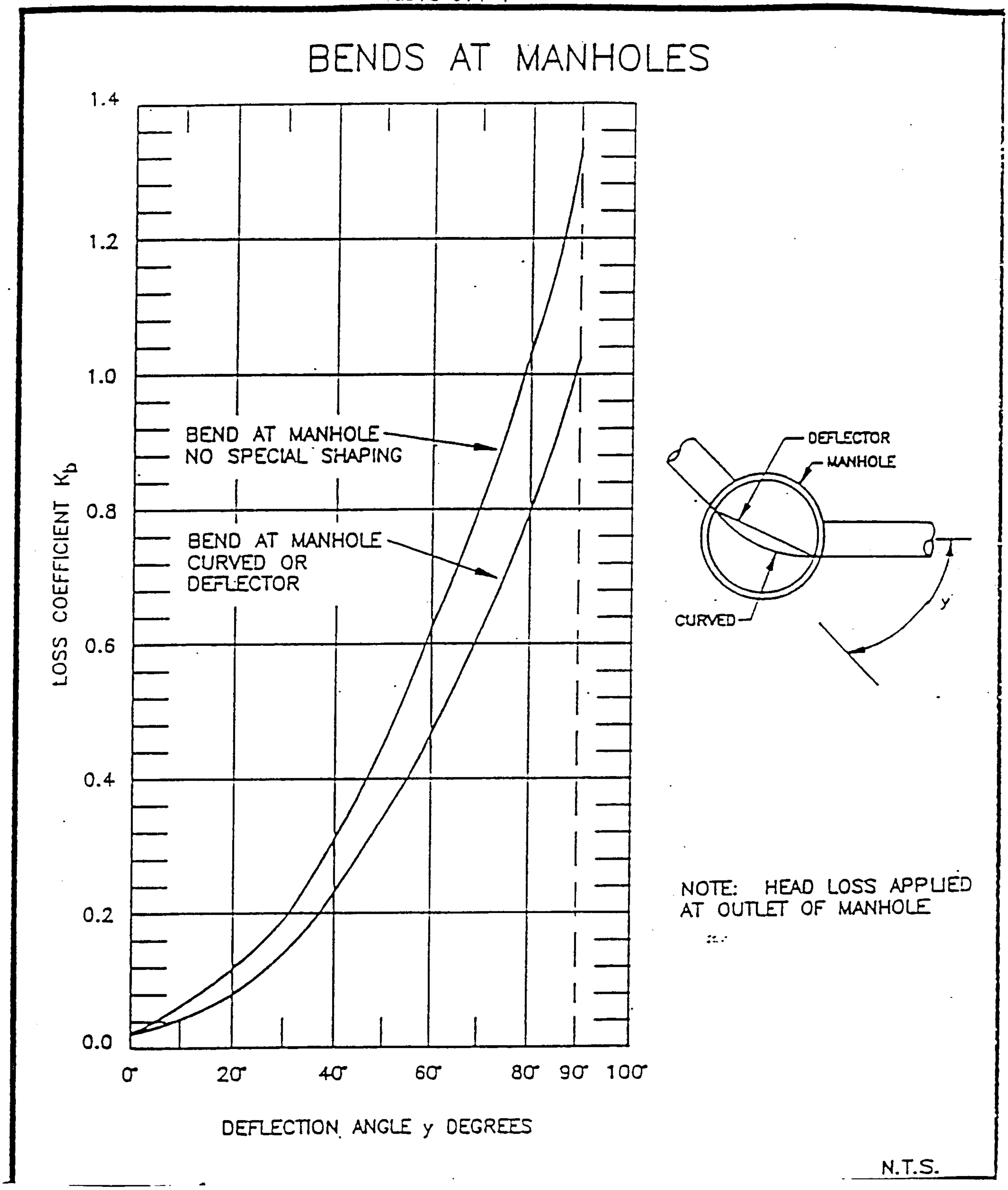


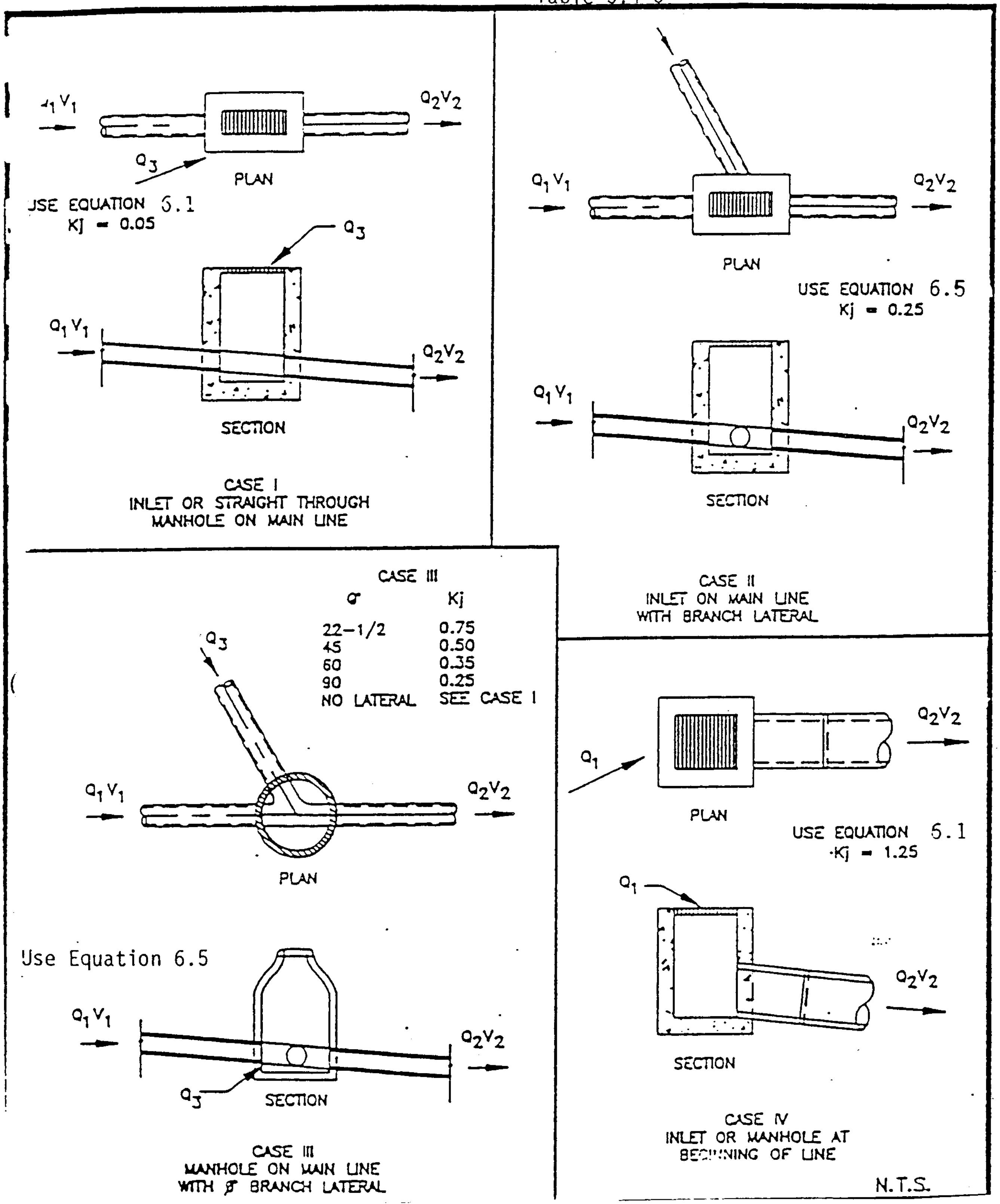
Table 3.4-4






# STORM SEWER ENERGY LOSS COEFFICIENT

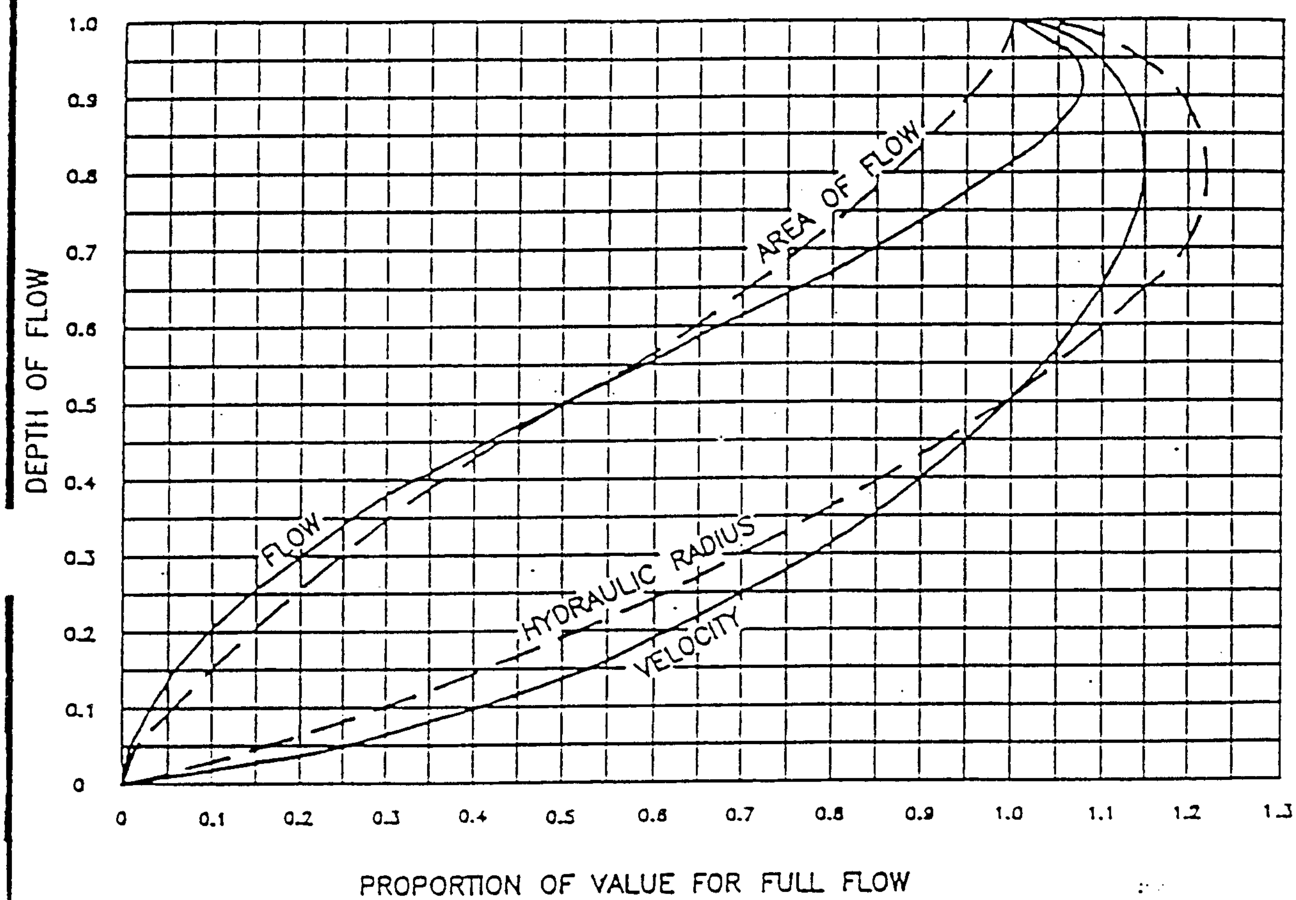
Table 3.4-5




**CITY OF EVANS** **STORM DRAINAGE CRITERIA**

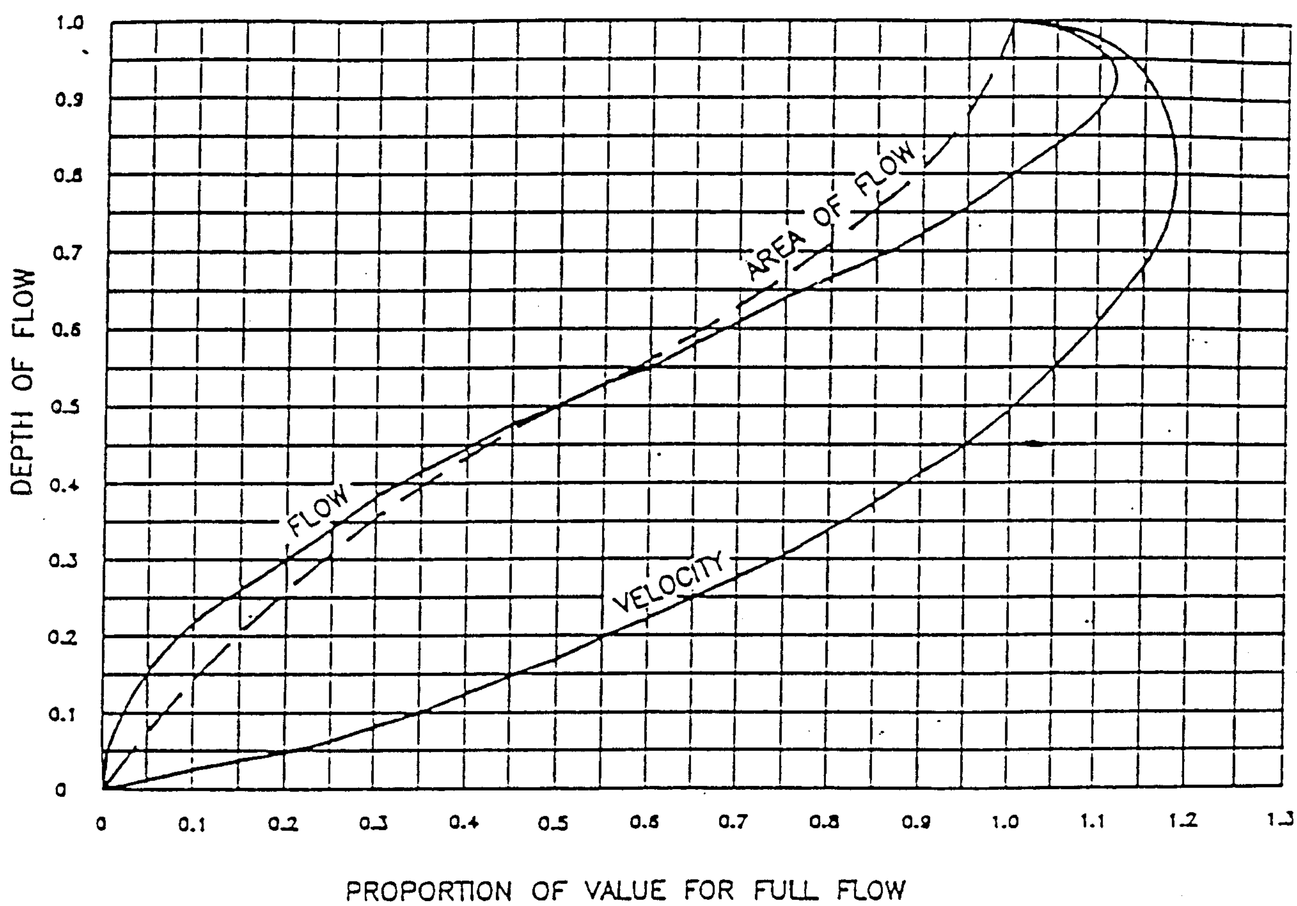
# MANHOLE JUNCTION LOSSES

Figure 3.4-1




**CITY OF EVANS** **STORM DRAINAGE CRITERIA**

# HYDRAULIC PROPERTIES CIRCULAR PIPE

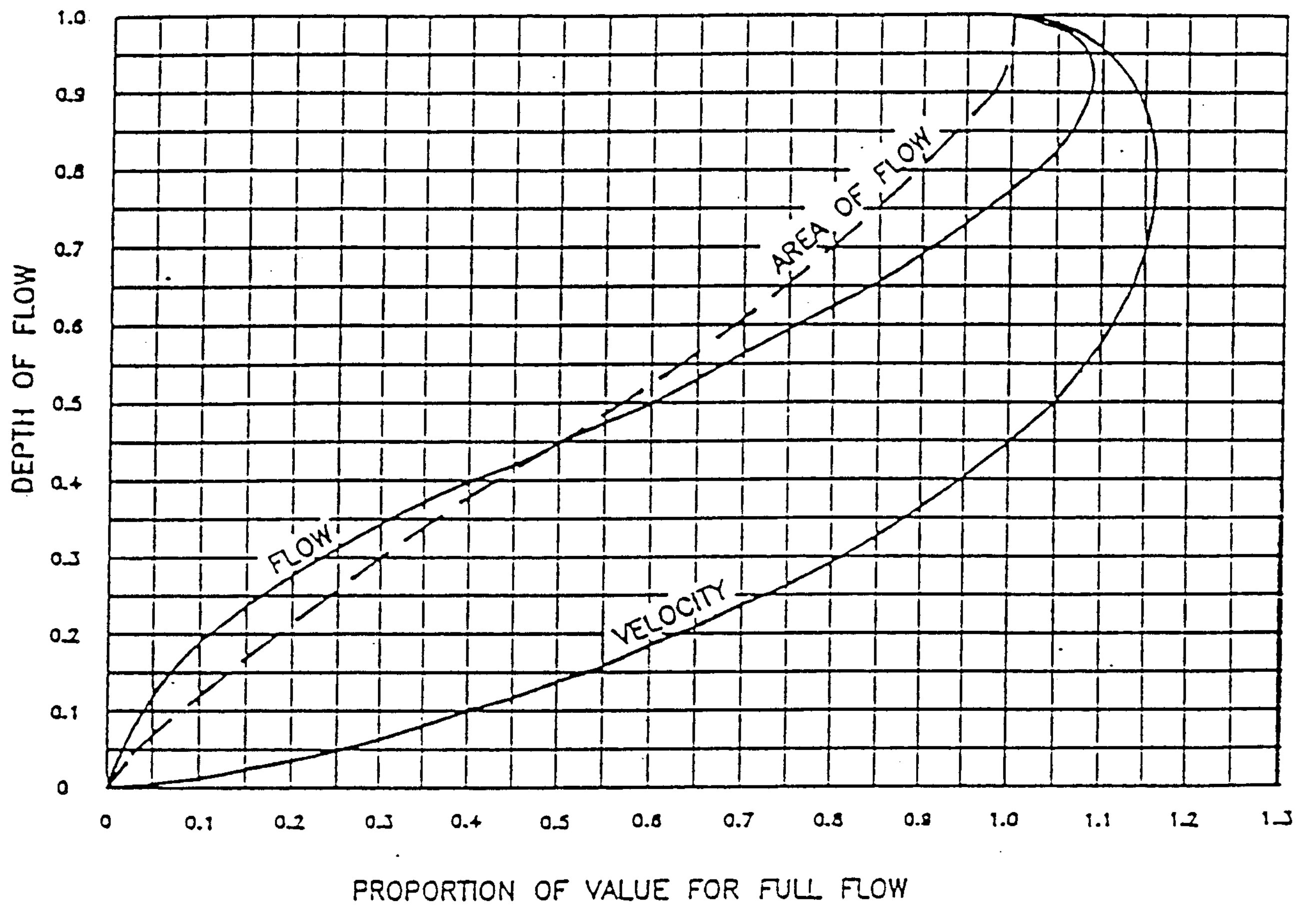



CITY OF EVANS

STORM DRAINAGE CRITERIA

# HYDRAULIC PROPERTIES
# HORIZONTAL ELLIPTICAL PIPE

Figure 3.4-3




CITY OF EVANS

STORM DRAINAGE CRITERIA

# HYDRAULIC PROPERTIES
# ARCH PIPE

CITY OF EVANS

STORM DRAINAGE CRITERIA

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STA | INVERT | D | W.S. | PIPE SHAPE | A | $f$ | V | Q | Hv | E.G. | Sf | AVG Sf | L | Hf | Hb | Hj | Hm | Ht | TOTAL LOSS |
| 0+00 | 94.50 | 66 | 100.00 | RND | 23.76 | 0.00492 | 6.1 | 145 | 0.58 | 100.58 | 0.0019 | 0.0019 | 110 | 0.21 | – | – | – | – | 0.21 |
| 1+10 | 94.71 | 66 | 100.21 | RND | 23.76 | 0.00492 | 6.1 | 145 | 0.58 | 100.79 | 0.0019 | 0.0019 | 42.4 | 0.08 | 0.12 | – | – | – | 0.20 |
| 1+52.4 | 94.91 | 66 | 100.41 | RND | 23.76 | 0.00492 | 6.1 | 145 | 0.58 | 100.99 | 0.0019 | 0.0019 | 95.6 | 0.18 | – | – | – | – | 0.18 |
| 2+48 | 95.08 | 66 | 100.59 | RND | 23.76 | 0.00492 | 6.1 | 145 | 0.58 | 101.17 | 0.0019 | 0.0048 | 7.5 | 0.04 | – | – | – | 0.15 | 0.19 |
| 2+55.5 | 96.06 | 54 | 100.07 | RND | 15.90 | 0.00492 | 9.1 | 145 | 1.29 | 101.36 | 0.0078 | 0.0078 | 100 | 0.76 | – | – | 0.06 | – | 0.82 |
| 3+55.5 | 96.90 | 54 | 100.89 | RND | 15.90 | 0.00492 | 9.1 | 145 | 1.29 | 102.18 | 0.0078 | 0.0078 | 100 | 0.76 | – | – | – | – | 0.76 |
| 4+55.5 | 97.69 | 54 | 101.65 | RND | 15.90 | 0.00492 | 9.1 | 145 | 1.29 | 102.94 | 0.0078 | 0.0063 | 10 | 0.06 | – | 0.68 | – | – | 0.74 |
| 4+65.5 | 98.40 | 48 | 102.69 | RND | 12.57 | 0.00492 | 8.0 | 100 | 0.99 | 103.68 | 0.0049 | 0.0049 | 100 | 0.49 | – | – | – | – | 0.49 |
| 5+65.5 | 98.89 | 48 | 103.18 | RND | 12.57 | 0.00492 | 8.0 | 100 | 0.99 | 104.17 | 0.0049 | 0.0064 | 10 | 0.06 | – | 1.56 | – | – | 1.62 |
| 5+75.5 | 100.69 | 24 | 105.15 | RND | 3.14 | 0.00492 | 6.4 | 20 | 0.64 | 105.79 | 0.0079 | 0.0079 | 100 | 0.79 | – | – | | – | 0.82 |
| 6+75.5 | 101.61 | 24 | 105.79 | RND | 3.14 | 0.00492 | 6.4 | 20 | 0.64 | 106.81 | 0.0079 | | | | | | | | |

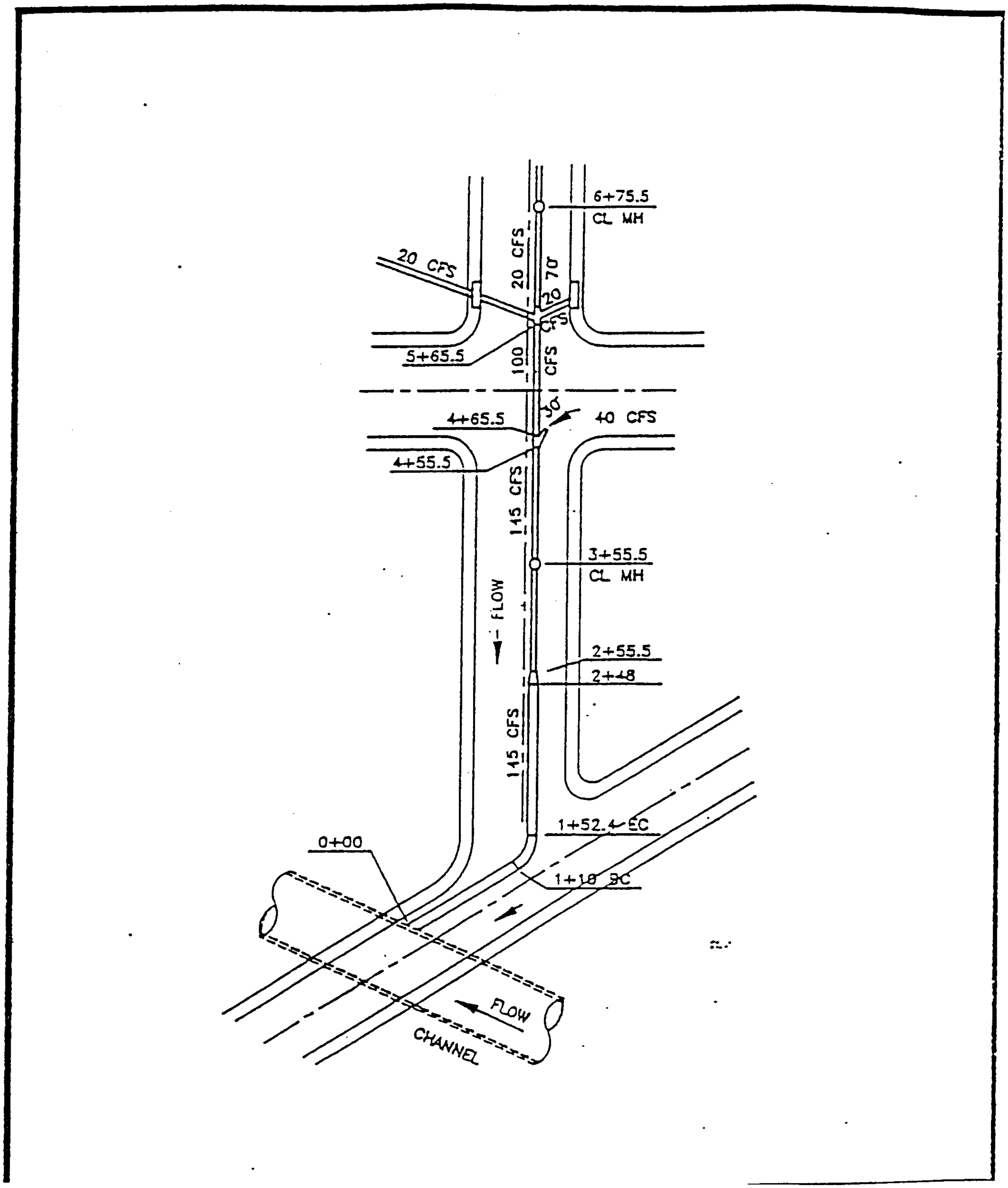
$$Sf = \frac{f\,Hv}{R^{1.33}}$$

$$f = \frac{2g(n^2)}{2.21}$$

TOTAL FRICTION LOSS = 3.43
TOTAL FORM LOSS = 2.75

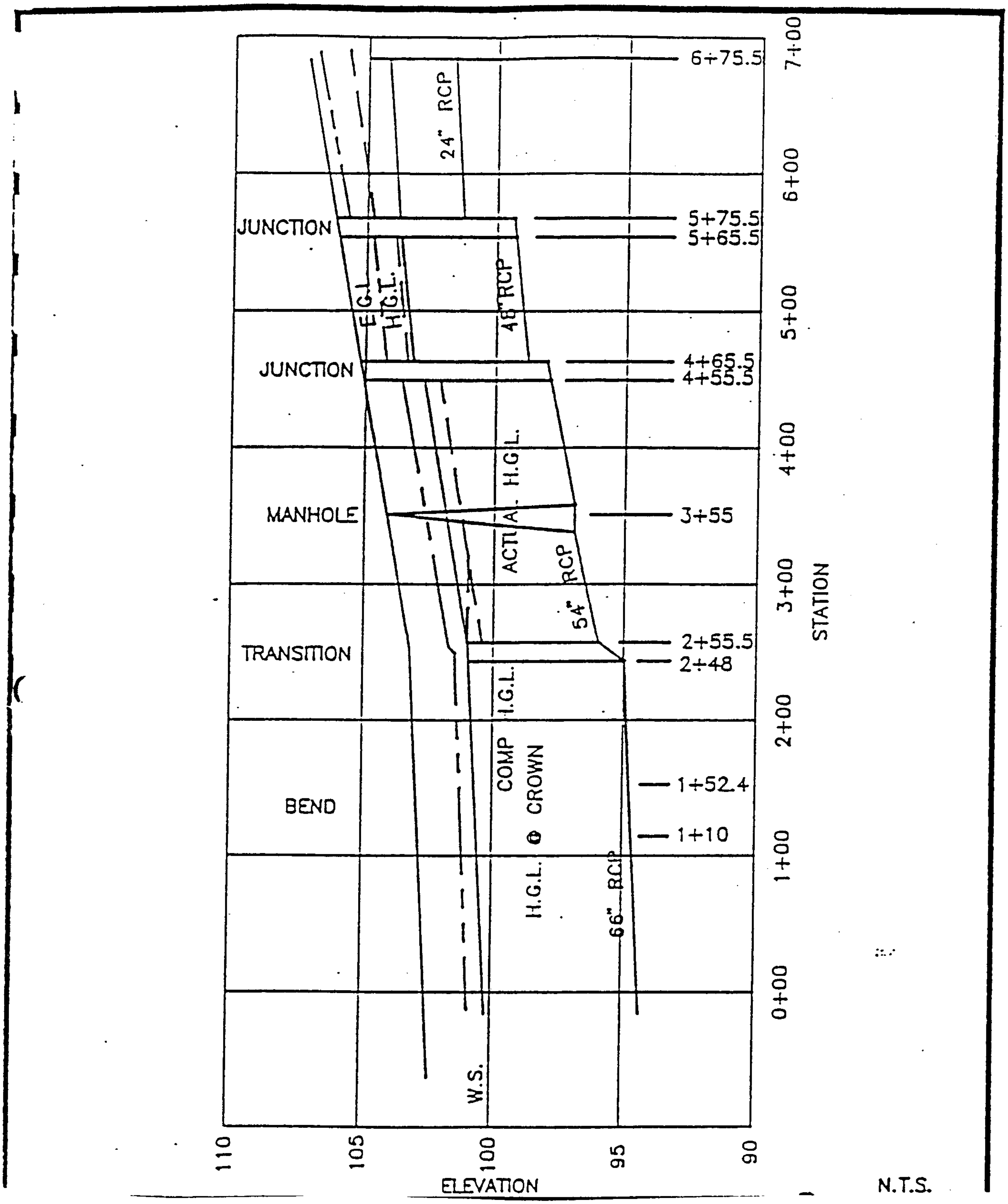
N.T.S.

# DESIGN EXAMPLE FOR STORM SEWERS




CITY OF EVANS STORM DRAINAGE CRITERIA

# DESIGN EXAMPLE FOR STORM SEWERS - PLAN




**CITY OF EVANS** **STORM DRAINAGE CRITERIA**

# DESIGN EXAMPLE FOR STORM SEWERS - PROFILE

## 3.5 STREETS

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 1 "Streets" section.

1. The procedures and requirements for storm drainage design for streets are explained in USDCM Volume 1.

2. The City of Greeley street classifications and their respective dimensions and cross sections shall be used for street capacity analysis. The adopted City classifications are summarized in Table 3.5-1.

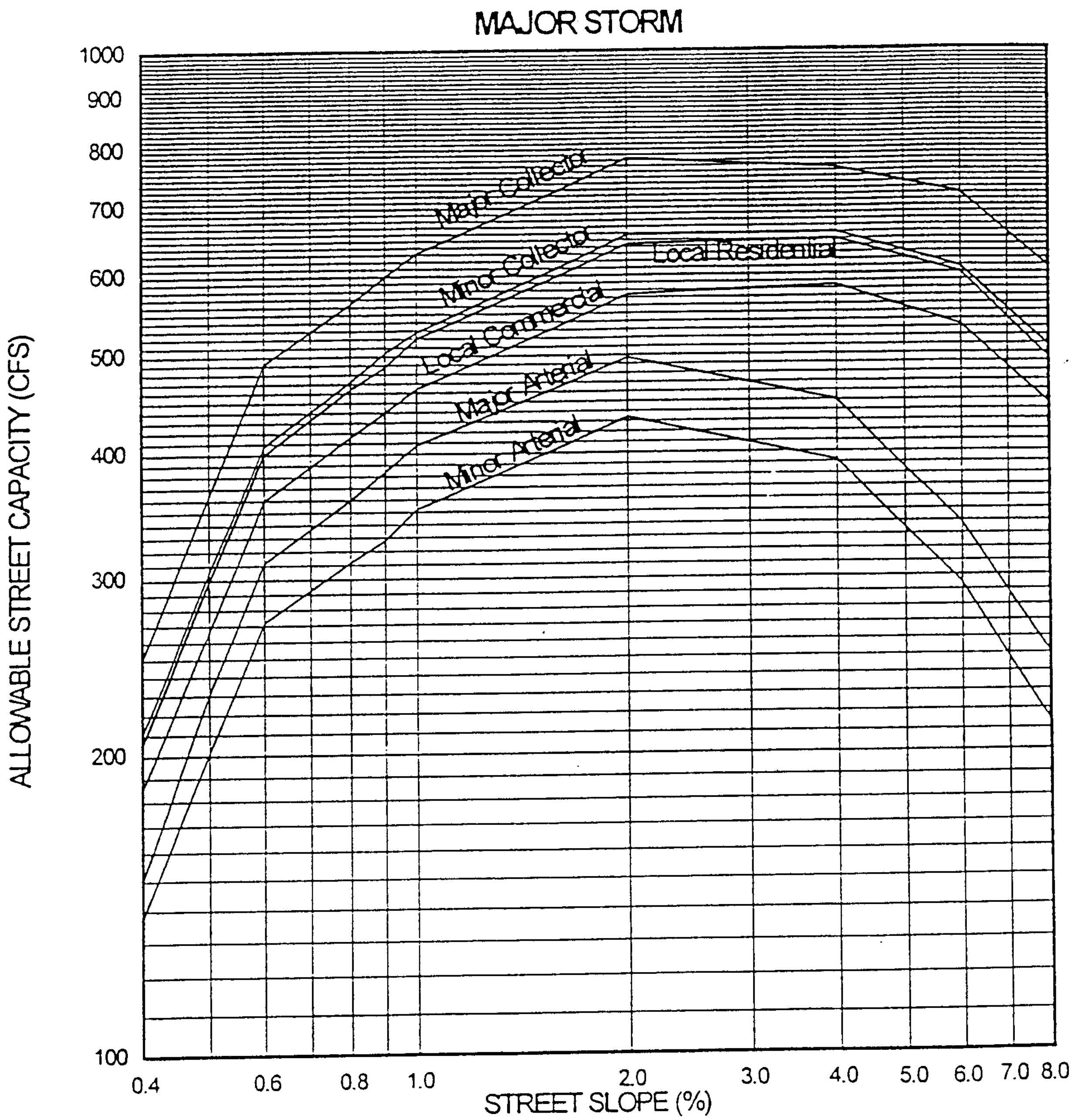
3. The City of Greeley street flow capacity standards have been adopted for the City of Evans. Hydraulic capacities for standard street sections are included in Figure 3.5-1 and in Table 3.5-2. Allowable gutter capacity for each standard street section has been calculated and in presented in Figures 3.5-2 thru 3.5-3.

TABLE 3.5-1 STANDARD STREET CLASSIFICATIONS

| CITY OF EVANS STREET CLASSIFICATIONS | | |
|---|---|---|
| Classification | Width<br>Flowline to Flowline | Standard Detail No.* |
| Local - Low Volume | No curb & gutter | S-1 |
| Local - Residential | 40' | S-2 |
| Local - Commercial/Industrial | 40' | S-2 |
| Minor Collector | 50' | S-3 |
| Major Collector | 60' | S-4 |
| Minor Arterial | 60' | S-5 |
| Major Arterial | 2 lanes @ 27' each | S-6 |

*Reference: City of Greeley Street Design Criteria ( Standard Detail Nos. S-2 to S-6 are are included for informational purposes at the end of Section 3-5).

Figure 3.5-1




NOTE: 1. Figure includes reduction factor for allowable capacity
2. The values in the figure indicate the flow capacity for a symmetrical street section (both gutters)

Table 3.5-2

# City of Evans - Standard Street Section Capacities

| Local-Residential (Std I) | | | | | | |
|---|---|---|---|---|---|---|
| | Reduction Factors [from Figure 8-2] | | Initial Storm (half street) | | Major Storm (full street) | |
| Gutter Slope (ft/ft) | Initial Storm | Major Storm | Theoretical Capacity (cfs) | Allowable Capacity (cfs) | Theoretical Capacity (cfs) | Allowable Capacity (cfs) |
| 0.004 | 0.500 | 0.500 | 5.5 | 2.8 | 410 | 205 |
| 0.005 | 0.650 | 0.650 | 6.0 | 3.9 | 460 | 299 |
| 0.006 | 0.800 | 0.800 | 6.5 | 5.2 | 500 | 400 |
| 0.008 | 0.800 | 0.800 | 7.5 | 6.0 | 580 | 464 |
| 0.009 | 0.800 | 0.800 | 8.0 | 6.4 | 610 | 488 |
| 0.010 | 0.800 | 0.800 | 8.5 | 6.8 | 650 | 520 |
| 0.020 | 0.800 | 0.700 | 11.5 | 9.2 | 920 | 644 |
| 0.040 | 0.610 | 0.500 | 16.5 | 10.1 | 1300 | 650 |
| 0.060 | 0.410 | 0.375 | 20.0 | 8.2 | 1600 | 600 |
| 0.080 | 0.280 | 0.270 | 24.0 | 6.7 | 1830 | 494 |

| Local-Commercial (Std II) | | | | | | |
|---|---|---|---|---|---|---|
| | Reduction Factors [from Figure 8-2] | | Initial Storm (half street) | | Major Storm (full street) | |
| Gutter Slope (ft/ft) | Initial Storm | Major Storm | Theoretical Capacity (cfs) | Allowable Capacity (cfs) | Theoretical Capacity (cfs) | Allowable Capacity (cfs) |
| 0.004 | 0.500 | 0.500 | 7.0 | 3.5 | 370 | 185 |
| 0.005 | 0.650 | 0.650 | 7.5 | 4.9 | 410 | 267 |
| 0.006 | 0.800 | 0.800 | 8.5 | 6.8 | 450 | 360 |
| 0.008 | 0.800 | 0.800 | 9.5 | 7.6 | 520 | 416 |
| 0.009 | 0.800 | 0.800 | 10.5 | 8.4 | 550 | 440 |
| 0.010 | 0.800 | 0.800 | 11.0 | 8.8 | 580 | 464 |
| 0.020 | 0.800 | 0.700 | 15.5 | 12.4 | 820 | 574 |
| 0.040 | 0.610 | 0.500 | 21.5 | 13.1 | 1170 | 585 |
| 0.060 | 0.410 | 0.375 | 27.0 | 11.1 | 1420 | 533 |
| 0.080 | 0.280 | 0.270 | 30.0 | 8.4 | 1650 | 446 |

| Minor Collector | | | | | | |
|---|---|---|---|---|---|---|
| | Reduction Factors [from Figure 8-2] | | Initial Storm (half street) | | Major Storm (full street) | |
| Gutter Slope (ft/ft) | Initial Storm | Major Storm | Theoretical Capacity (cfs) | Allowable Capacity (cfs) | Theoretical Capacity (cfs) | Allowable Capacity (cfs) |
| 0.004 | 0.500 | 0.500 | 7.0 | 3.5 | 420 | 210 |
| 0.005 | 0.650 | 0.650 | 7.5 | 4.9 | 470 | 306 |
| 0.006 | 0.800 | 0.800 | 8.5 | 6.8 | 510 | 408 |
| 0.008 | 0.800 | 0.800 | 9.5 | 7.6 | 590 | 472 |
| 0.009 | 0.800 | 0.800 | 10.5 | 8.4 | 630 | 504 |
| 0.010 | 0.800 | 0.800 | 11.0 | 8.8 | 660 | 528 |
| 0.020 | 0.800 | 0.700 | 15.5 | 12.4 | 940 | 658 |
| 0.040 | 0.610 | 0.500 | 21.5 | 13.1 | 1320 | 660 |
| 0.060 | 0.410 | 0.375 | 27.0 | 11.1 | 1620 | 608 |
| 0.080 | 0.280 | 0.270 | 30.0 | 8.4 | 1880 | 508 |

Table 3.5-2 (Cont.)

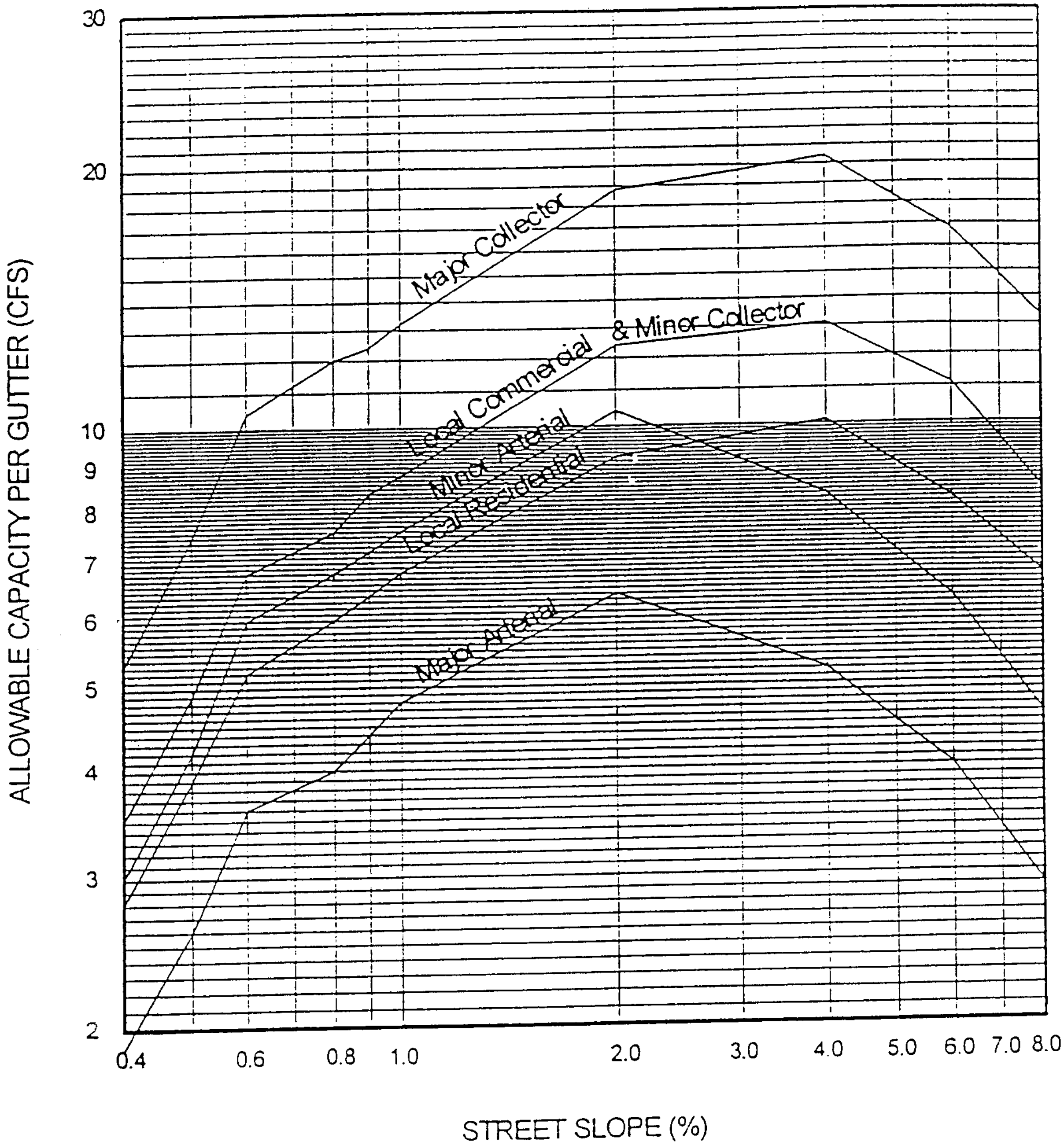
| Major Collector | | | | | | |
|---|---|---|---|---|---|---|
| | Reduction Factors [from Figure 8-2] | | Initial Storm (half street) | | Major Storm (full street) | |
| Gutter Slope (ft/ft) | Initial Storm | Major Storm | Theoretical Capacity (cfs) | Allowable Capacity (cfs) | Theoretical Capacity (cfs) | Allowable Capacity (cfs) |
| 0.004 | 0.500 | 0.500 | 10.5 | 5.3 | 500 | 250 |
| 0.005 | 0.650 | 0.650 | 11.5 | 7.5 | 560 | 364 |
| 0.006 | 0.800 | 0.800 | 13.0 | 10.4 | 615 | 492 |
| 0.008 | 0.800 | 0.800 | 15.0 | 12.0 | 705 | 564 |
| 0.009 | 0.800 | 0.800 | 15.5 | 12.4 | 750 | 600 |
| 0.010 | 0.800 | 0.800 | 16.5 | 13.2 | 790 | 632 |
| 0.020 | 0.800 | 0.700 | 23.5 | 18.8 | 1120 | 784 |
| 0.040 | 0.610 | 0.500 | 33.5 | 20.4 | 1530 | 765 |
| 0.060 | 0.410 | 0.375 | 41.0 | 16.8 | 1920 | 720 |
| 0.080 | 0.280 | 0.270 | 47.5 | 13.3 | 2250 | 608 |

| Minor Arterial | | | | | | |
|---|---|---|---|---|---|---|
| | Reduction Factors [from Figure 8-2] | | Initial Storm (half street) | | Major Storm (full street) | |
| Gutter Slope (ft/ft) | Initial Storm | Major Storm | Theoretical Capacity (cfs) | Allowable Capacity (cfs) | Theoretical Capacity (cfs) | Allowable Capacity (cfs) |
| 0.004 | 0.500 | 0.500 | 6.0 | 3.0 | 275 | 138 |
| 0.005 | 0.650 | 0.650 | 6.5 | 4.2 | 310 | 202 |
| 0.006 | 0.800 | 0.800 | 7.5 | 6.0 | 340 | 272 |
| 0.008 | 0.800 | 0.800 | 8.5 | 6.8 | 390 | 312 |
| 0.009 | 0.800 | 0.800 | 9.0 | 7.2 | 410 | 328 |
| 0.010 | 0.800 | 0.800 | 9.5 | 7.6 | 440 | 352 |
| 0.020 | 0.800 | 0.700 | 13.0 | 10.4 | 620 | 434 |
| 0.040 | 0.450 | 0.450 | 18.5 | 8.3 | 870 | 392 |
| 0.060 | 0.275 | 0.275 | 23.0 | 6.3 | 1070 | 294 |
| 0.080 | 0.175 | 0.175 | 26.0 | 4.6 | 1220 | 214 |

| Major Arterial | | | | | | |
|---|---|---|---|---|---|---|
| | Reduction Factors [from Figure 8-2] | | Initial Storm (half street) | | Major Storm (full street) | |
| Gutter Slope (ft/ft) | Initial Storm | Major Storm | Theoretical Capacity (cfs) | Allowable Capacity (cfs) | Theoretical Capacity (cfs) | Allowable Capacity (cfs) |
| 0.004 | 0.500 | 0.500 | 3.8 | 1.9 | 300 | 150 |
| 0.005 | 0.650 | 0.650 | 4.0 | 2.6 | 355 | 231 |
| 0.006 | 0.800 | 0.800 | 4.5 | 3.6 | 390 | 312 |
| 0.008 | 0.800 | 0.800 | 5.0 | 4.0 | 450 | 360 |
| 0.009 | 0.800 | 0.800 | 5.5 | 4.4 | 480 | 384 |
| 0.010 | 0.800 | 0.800 | 6.0 | 4.8 | 510 | 408 |
| 0.020 | 0.800 | 0.700 | 8.0 | 6.4 | 710 | 497 |
| 0.040 | 0.450 | 0.450 | 11.5 | 5.2 | 1000 | 450 |
| 0.060 | 0.275 | 0.275 | 14.5 | 4.0 | 1230 | 338 |
| 0.080 | 0.175 | 0.175 | 16.5 | 2.9 | 1430 | 250 |

Figure 3.5-2

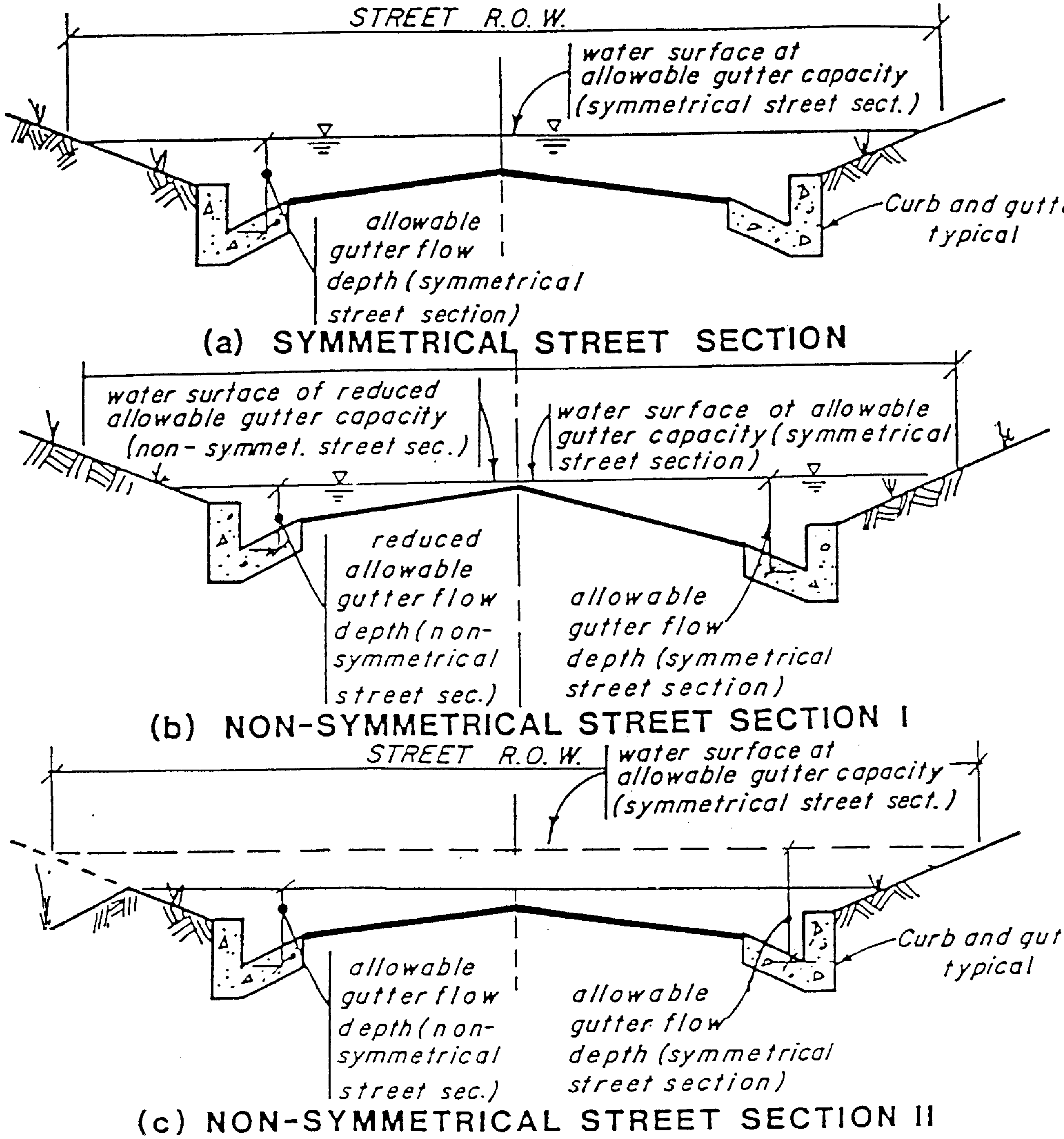
INITIAL STORM




NOTE: 1. Figure includes reduction factor for allowable capacity

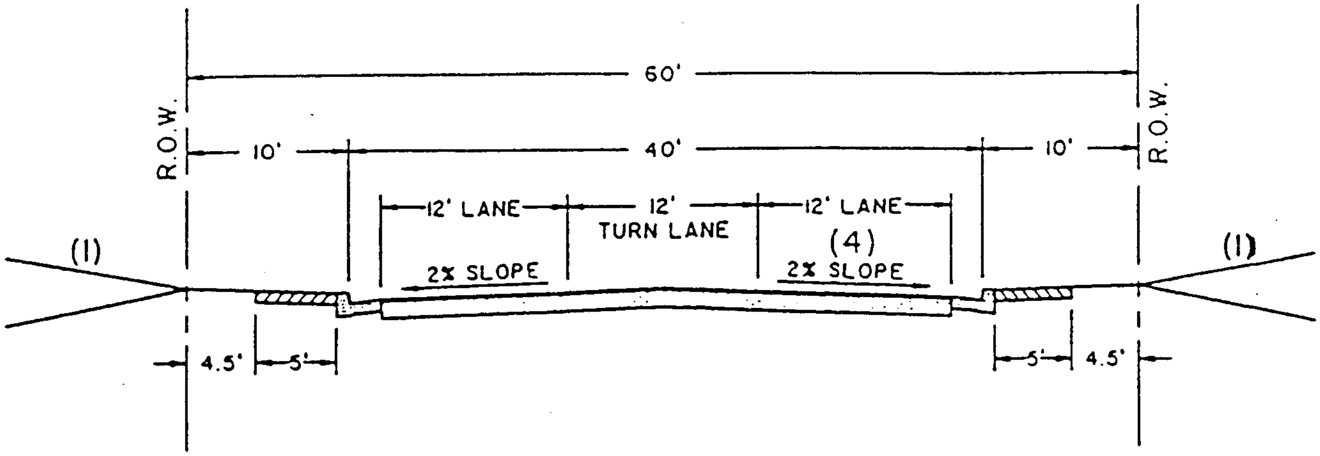
STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

# ADJUSTMENT FOR GUTTER CAPACITY WITH NON-SYMMETRICAL STREET SECTION MAJOR STORM



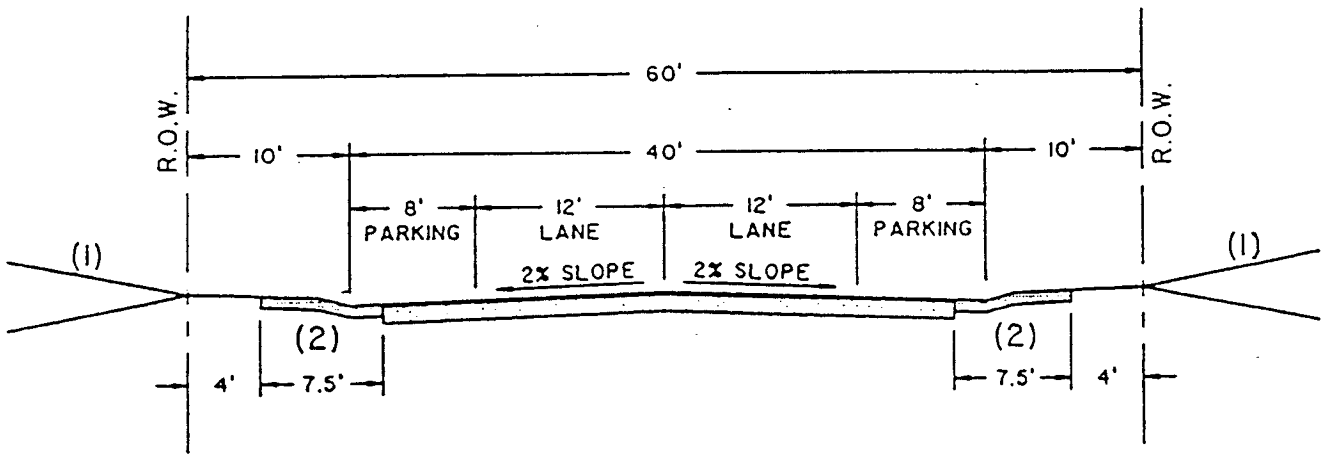

Note: For non-symmetrical street section, adjust the total gutter capacity by reducing the allowable gutter capacity for the gutter with the higher flowline or for the entire section when property line slopes are different.

REFERENCE:




## LOCAL-STANDARD -II

COMMERCIAL/INDUSTRIAL DESIGN




## LOCAL-STANDARD -I

RESIDENTIAL DESIGN

NOTES:

1. CUT AND FILL SLOPES SHALL BE A MAXIMUM OF 4:1.
2. VERTICAL CURB AND GUTTER MAY BE USED ON STANDARD ROADWAYS.
3. RIGHT OF WAY & EASEMENT AREAS SHALL BE GRADED (CUT & FILL) TO SUBGRADE (+/-0.5) AT UTILITY LOCATIONS INCLUDING SERVICES, PRIOR TO UTILITY INSTALLATION.
4. NORMAL CROWN SLOPE IS 2%. WITH SPECIAL DESIGN REVIEW, 1% TO 4% IS ALLOWABLE AT TRANSITION AND OTHER NON-NORMAL SECTIONS.

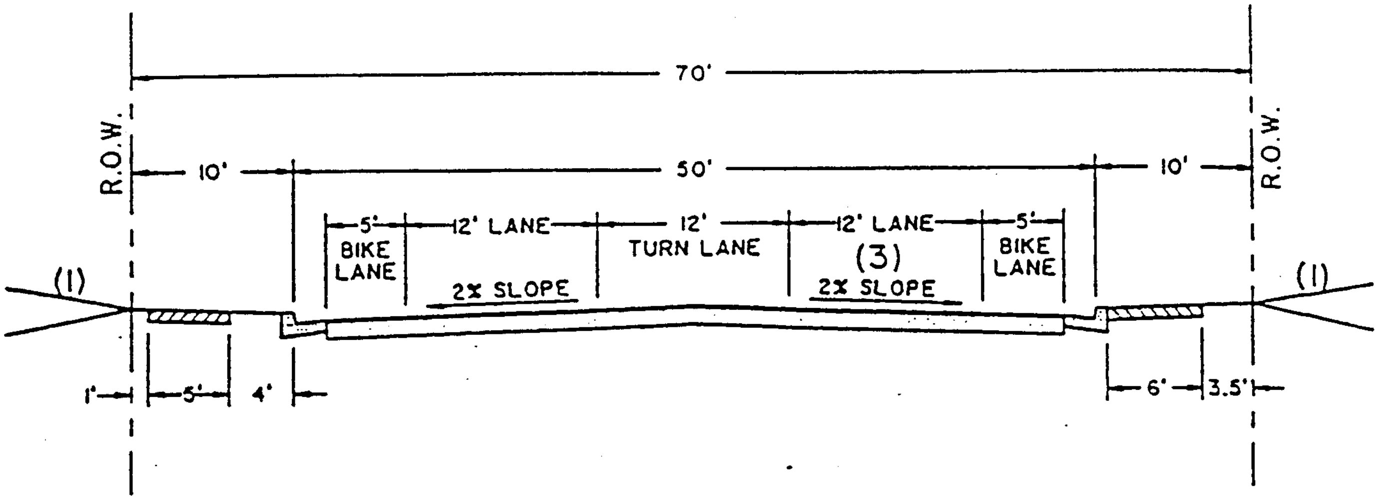
# STANDARD ROADWAY SECTION STANDARDS I & II

DETAIL NO. S-2

6-15

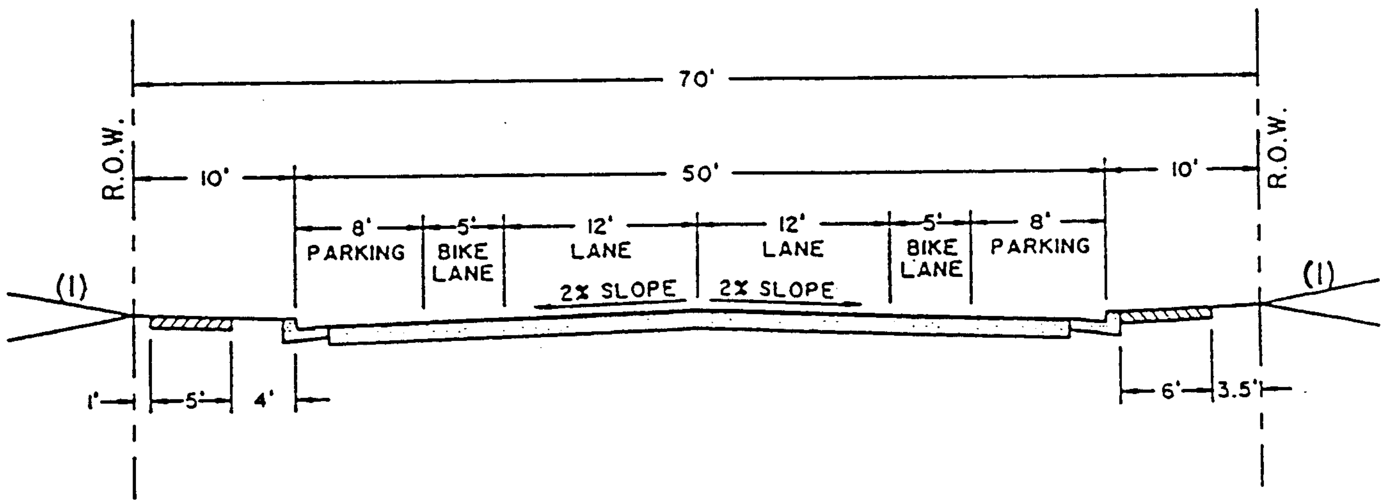


SCALE: N.T.S.




## AT INTERSECTIONS

TWO LANES WITH LEFT TURN LANE




## BETWEEN INTERSECTIONS

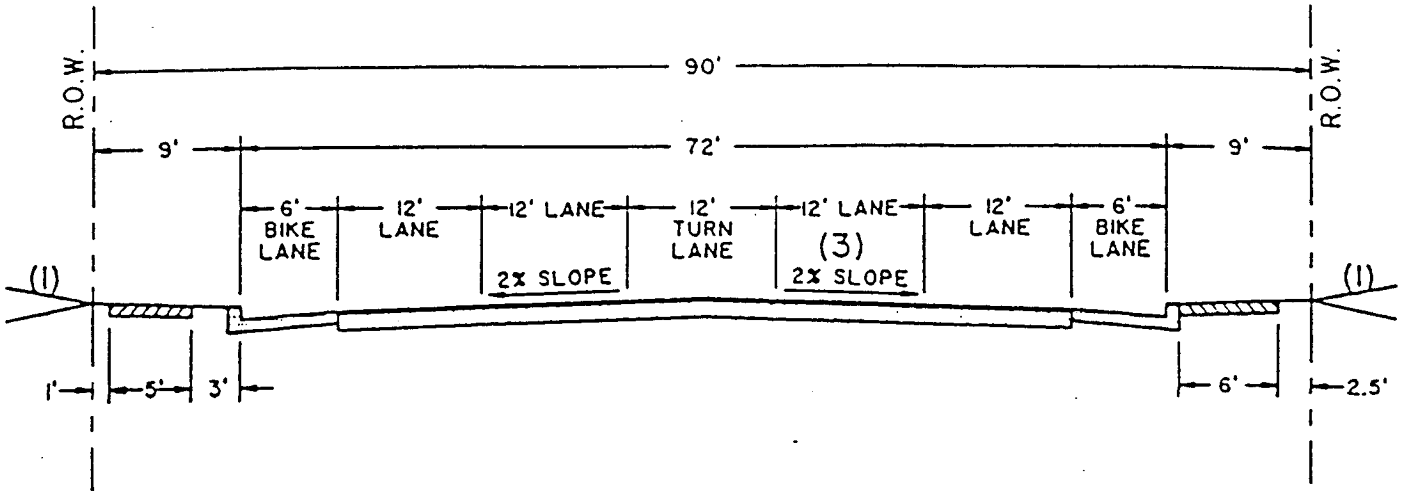
TWO LANES WITH PARKING

NOTES:

1. CUT AND FILL SLOPES SHALL BE A MAXIMUM OF 4:1.
2. RIGHT OF WAY & EASEMENT AREAS SHALL BE GRADED (CUT & FILL) TO SUBGRADE (+/-0.5) AT UTILITY LOCATIONS INCLUDING SERVICES, PRIOR TO UTILITY INSTALLATION.
3. NORMAL CROWN SLOPE IS 2%. WITH SPECIAL DESIGN REVIEW, 1% TO 4% IS ALLOWABLE AT TRANSITION AND OTHER NON-NORMAL SECTIONS.

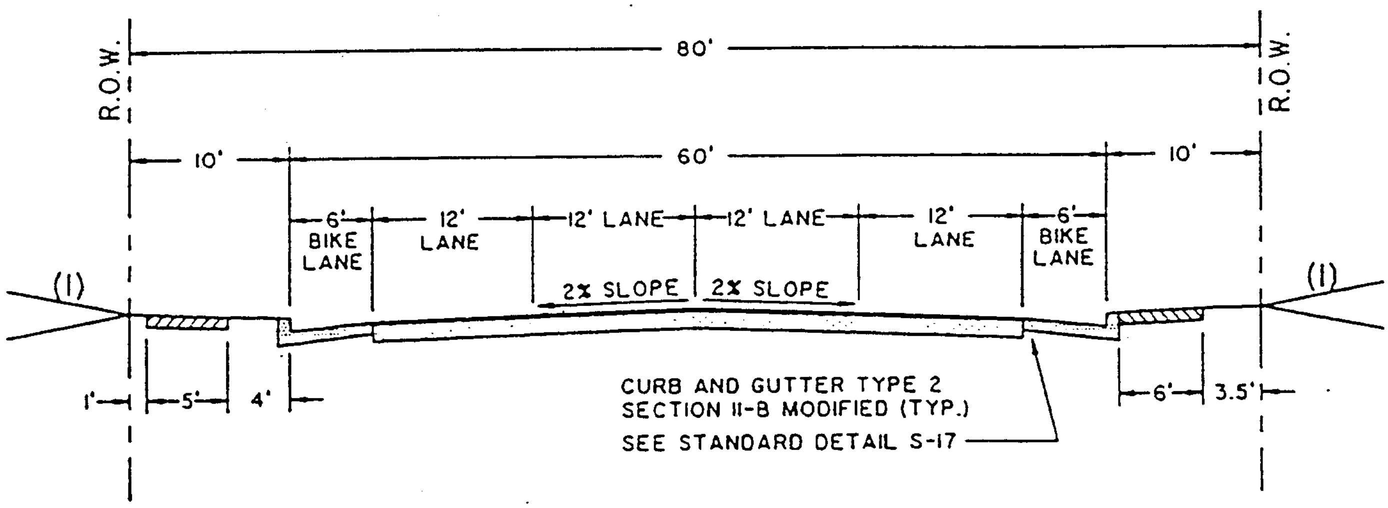
| | |
|---|---|
| | STANDARD ROADWAY SECTION<br>MINOR COLLECTOR<br>DETAIL NO. S-3 |
| DATE: JULY, 1994 | SCALE: N.T.S. |

6-15




## AT INTERSECTIONS

FOUR LANES WITH LEFT TURN LANE




## BETWEEN INTERSECTIONS

FOUR LANES

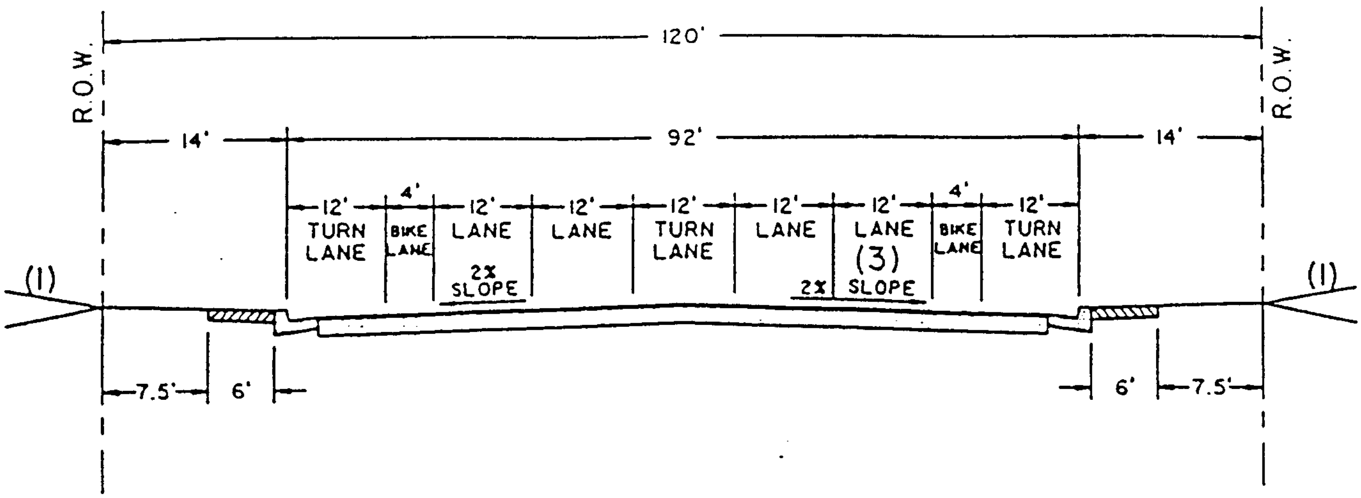
NOTES:

1. CUT AND FILL SLOPES SHALL BE A MAXIMUM OF 4:1.
2. RIGHT OF WAY & EASEMENT AREAS SHALL BE GRADED (CUT & FILL) TO SUBGRADE (+/-0.5) AT UTILITY LOCATIONS INCLUDING SERVICES, PRIOR TO UTILITY INSTALLATION.
3. NORMAL CROWN SLOPE IS 2%. WITH SPECIAL DESIGN REVIEW, 1% TO 4% IS ALLOWABLE AT TRANSITION AND OTHER NON-NORMAL SECTIONS.

# STANDARD ROADWAY SECTION
# MAJOR COLLECTOR

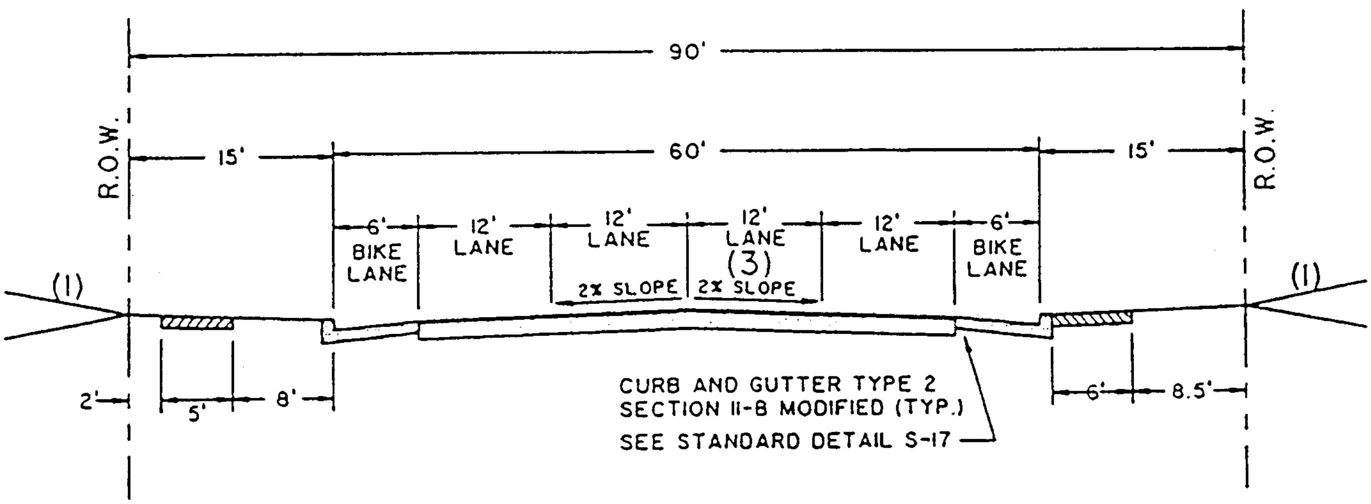
DETAIL NO. S-4

6-15




## AT INTERSECTIONS

FOUR LANES WITH LEFT TURN LANE




## BETWEEN INTERSECTIONS

FOUR LANES

NOTES:

1. CUT AND FILL SLOPES SHALL BE A MAXIMUM OF 4:1.
2. RIGHT OF WAY & EASEMENT AREAS SHALL BE GRADED (CUT & FILL) TO SUBGRADE (+/-0.5) AT UTILITY LOCATIONS INCLUDING SERVICES, PRIOR TO UTILITY INSTALLATION.
3. NORMAL CROWN SLOPE IS 2%. WITH SPECIAL DESIGN REVIEW, 1% TO 4% IS ALLOWABLE AT TRANSITION AND OTHER NON-NORMAL SECTIONS.

# STANDARD ROADWAY SECTION MINOR ARTERIAL

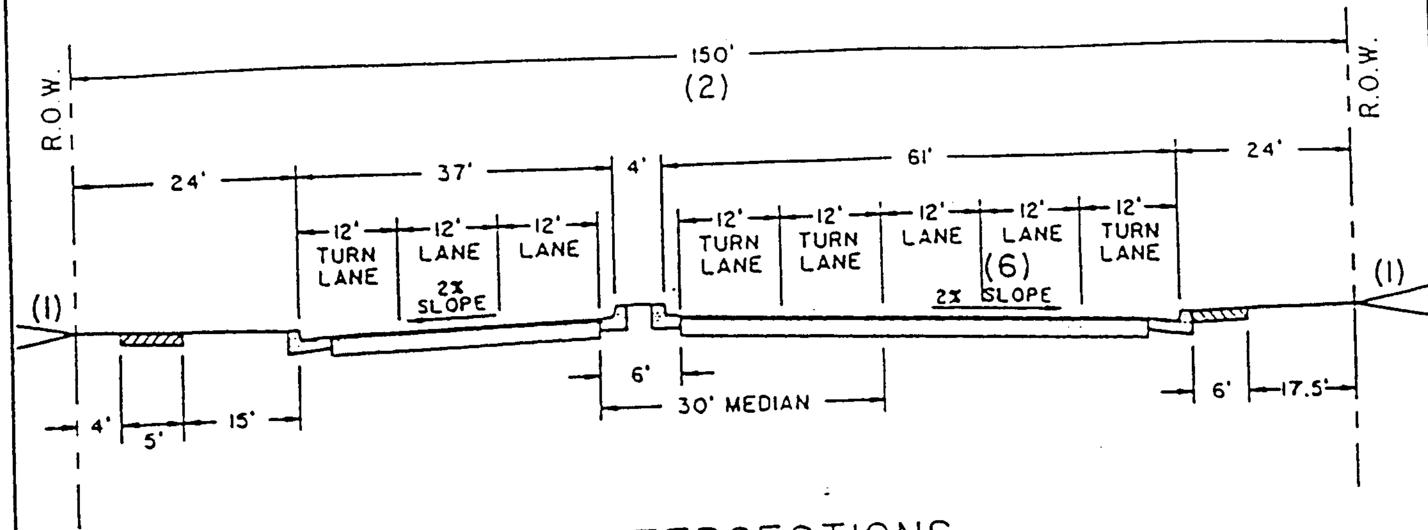
DETAIL NO. S-5

6-15

DATE: JULY, 1994

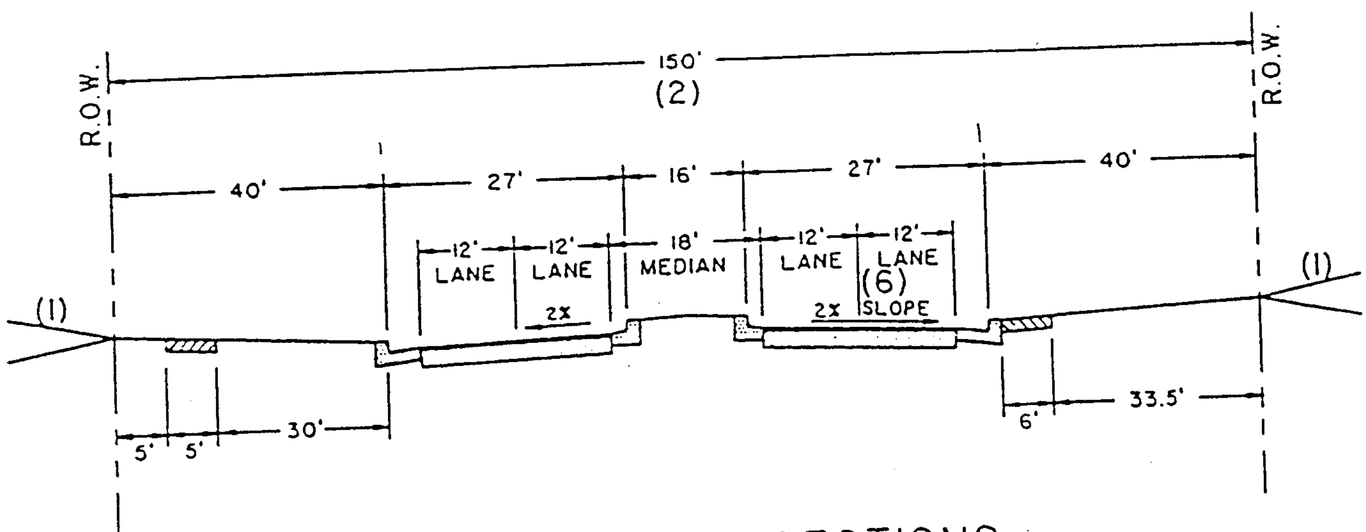


SCALE: N.T.S.




## AT INTERSECTIONS

FOUR LANES WITH TURN LANES




## BETWEEN INTERSECTIONS

FOUR LANES WITH MEDIAN

NOTES:

1. CUT AND FILL SLOPES SHALL BE A MAXIMUM OF 4:1.
2. R.O.W. WIDTHS ARE PROVIDED TO ACCOMMODATE POSSIBLE FUTURE THROUGH LANES.
3. RIGHT OF WAY & EASEMENT AREAS SHALL BE GRADED (CUT & FILL) TO SUBGRADE (+/-0.5) AT UTILITY LOCATIONS INCLUDING SERVICES, PRIOR TO UTILITY INSTALLATION.
4. R.O.W. CAN ACCOMMODATE A SIX LANE SECTION.
5. ACCEPTABLE BIKEWAYS ARE REQUIRED ON MAJOR ARTERIALS; TYPE AND DESIGN TO BE DETERMINED DURING THE DESIGN PROCESS.
6. NORMAL CROWN SLOPE IS 2%. WITH SPECIAL DESIGN REVIEW, 1% TO 4% IS ALLOWABLE AT TRANSITION AND OTHER NON-NORMAL SECTIONS.

STANDARD ROADWAY SECTION
MAJOR ARTERIAL

DETAIL NO. S-6

6-15

SCALE: N.T.S.

DATE: JULY, 1994

## 3.6 INLETS

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 1"Storm Inlets" section.

1. Inlets shall be located to intercept curb flow. Inlets and inlet transitions are prohibited in curb returns, driveways, and street/curb transitions.

2. Optimum inlet spacing will depend on traffic requirements, land use, street slope, and distance to the outfall system. The recommended sizing and spacing of the inlets is based upon the interception rate of 70 to 80 percent. However, due to variable street flow capacities, optimum street flow cannot always be achieved.

3. Standard inlets permitted for use in the City are shown below.

| Inlet Type | Detail | Permitted Use |
|---|---|---|
| Curb Opening Inlet, Type R | Figure 3.6-1 | All street types |
| Grated Inlet Type C | Figure 3.6-2 | All streets with roadside or median ditch |
| Grated Inlet Type 13 | Figure 3.6-3 | Alleys or private drives with a valley gutter |
| Combination Inlet Type 13 | Figure 3.6-4 | All street types |
| Note: Other combination inlet types may be requested as a variance and used with City approval. | | |

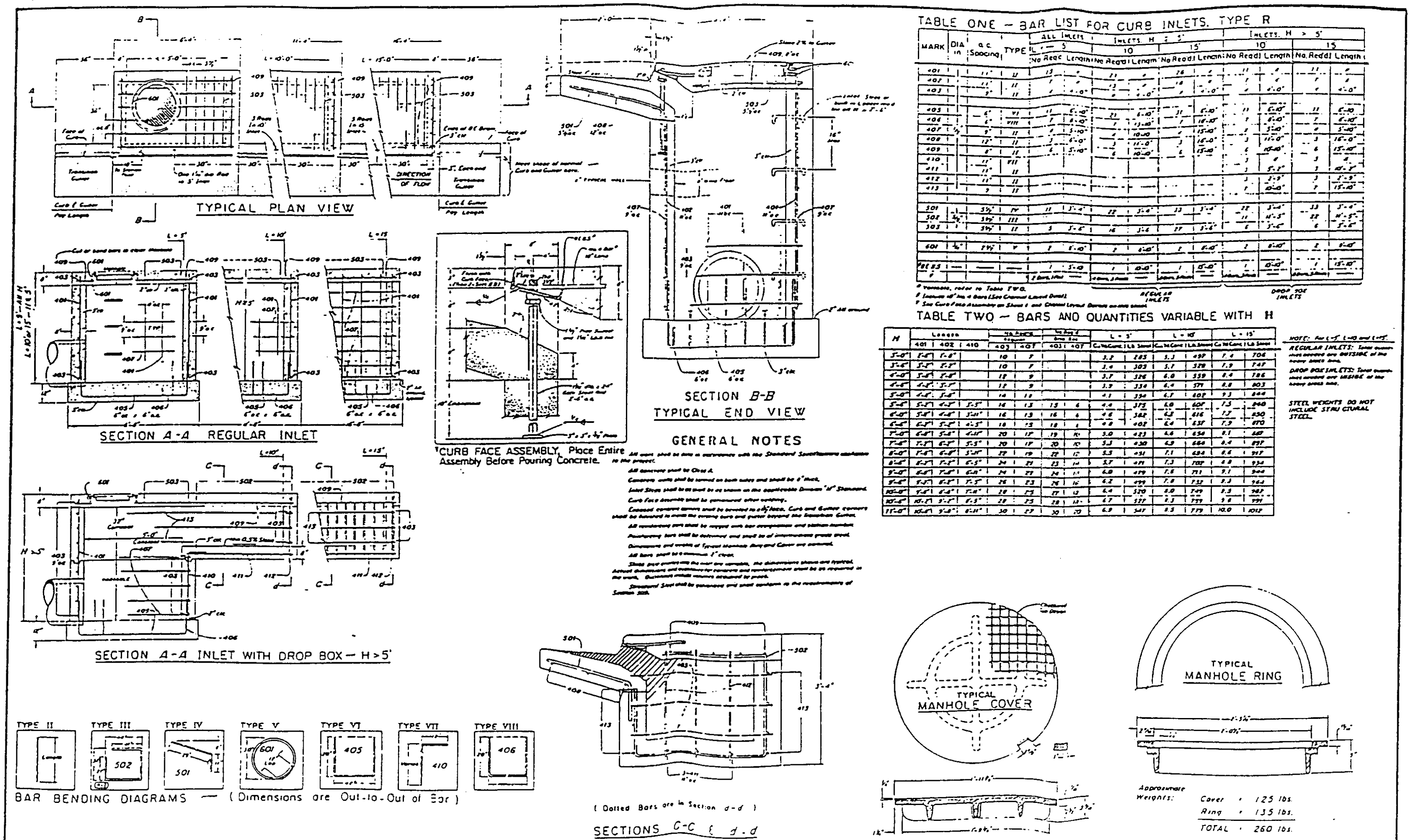
4. Allowable standard inlet capacities for the initial storm shall be in accordance with the City of Greeley standards and are presented in Figures 3.6-5 thru 3.6-7 for continuous grade and Figure 3.6-8 for sump conditions. These figures include the required reduction factors. The allowable inlet capacities are compatible with the allowable street capacities (see Section 3.5). The values shown were calculated on the basis of the maximum flow allowed in the street gutter, or maximum flow in the roadside ditch for Type C. For gutter flows less than the maximum the inlet capacity must be proportionately reduced.

5. The concept of inlet capacity reduction due to grate clogging, pavement overlaying, and variations in design assumptions, shall be used in design. Stated below are the allowable capacities after application of the minimum reduction factors to the theoretical capacities.

| ALLOWABLE INLET CAPACITY | | |
|---|---|---|
| Condition | Inlet Type | Percentage of Theoretical Capacity Allowed |
| Sump or Continuous Grade | Type R<br>5' Length<br>10' Length<br>15' Length | <br>88<br>92<br>95 |
| Sump or Continuous Grade | Grated Type 13 | 50 |
| Continuous Grade | Combination Type 13 | 66 |
| Sump | Grated Type C | 50 |
| Sump | Combination Type 13 | 65 |

6. Example design problems are included. They are taken from the City of Greeley Drainage Criteria and can be used as guidelines.

Figure 3.6-1

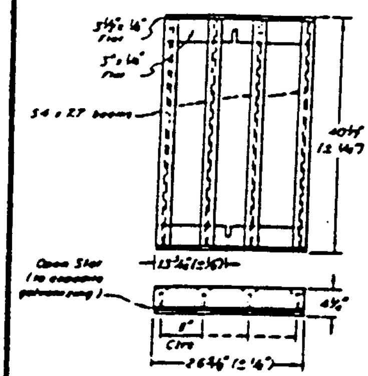



REFERENCE: Colorado Department of Highways
Standard M-604-R

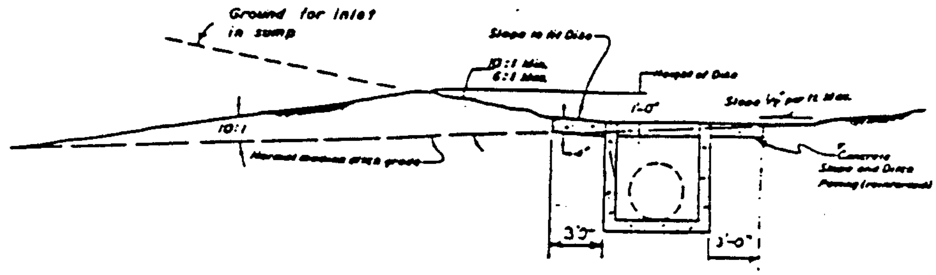
CURB INLET TYPE R

## STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

# GRATED INLET TYPE C



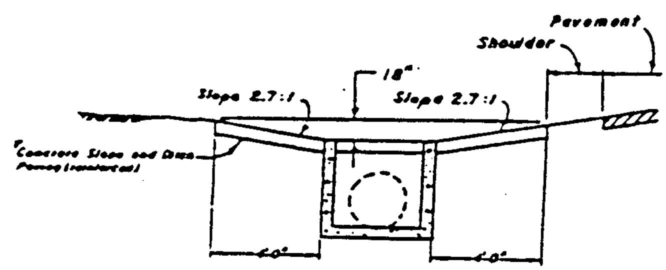
DETAIL OF GRATING



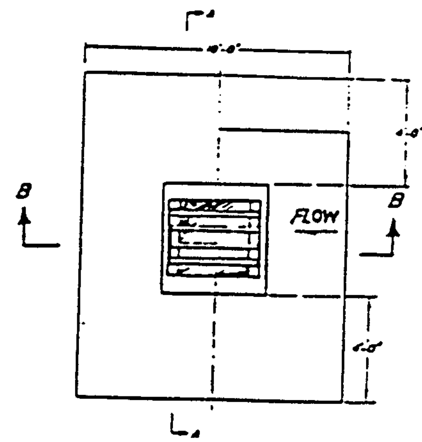

SECTION B - B

INLET FOR USE IN DITCH ON GRADE

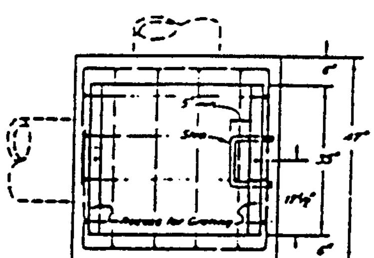
(FLOW FROM ONE DIRECTION)



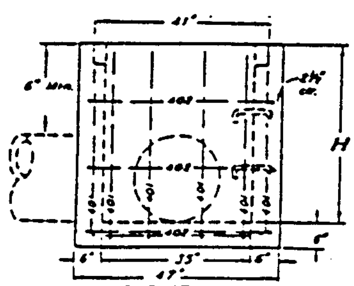

SECTION A — A




LAYOUT OF INLET IN ROADSIDE DITCH

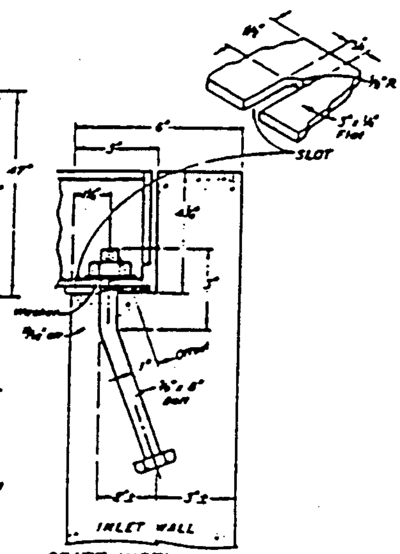



PLAN




ELEVATION

CONCRETE INLET



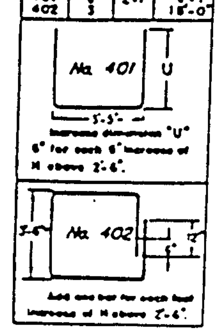

GRATE INSTALLATION DETAIL

QUANTITIES FOR ONE INLET

| H | CONCRETE *(CU. YDS.) | REINF. STEEL (LBS.) | NO. STEPS REQ'D. |
|---|---|---|---|
| 2'-6" | 0.9 | 73 | 0 |
| 3'-0" | 1.0 | 80 | 0 |
| 3'-6" | 1.2 | 96 | 0 |
| 4'-0" | 1.3 | 101 | 1 |
| 4'-6" | 1.4 | 116 | 2 |
| 5'-0" | 1.5 | 122 | 2 |
| 5'-6" | 1.7 | 137 | 2 |
| 6'-0" | 1.8 | 142 | 3 |
| 6'-6" | 1.9 | 158 | 3 |
| 7'-0" | 2.0 | 163 | 3 |
| 7'-6" | 2.2 | 179 | 4 |
| 8'-0" | 2.3 | 184 | 4 |
| 8'-6" | 2.4 | 199 | 4 |
| 9'-0" | 2.5 | 205 | 5 |
| 9'-6" | 2.7 | 220 | 5 |
| 10'-6" | 3.0 | 235 | 6 |
| 11'-6" | 3.4 | 251 | 6 |

BAR LIST FOR H=2'-6" AND BENDING DIAGRAM

| MARK | NO. REQ'D | HGT. "U" | LENGTH |
|---|---|---|---|
| 401 | 2 | 2'-3" | 7'-11" |
| 401 | 6 | 2'-7" | 6'-7" |
| 402 | 3 | | 15'-0" |

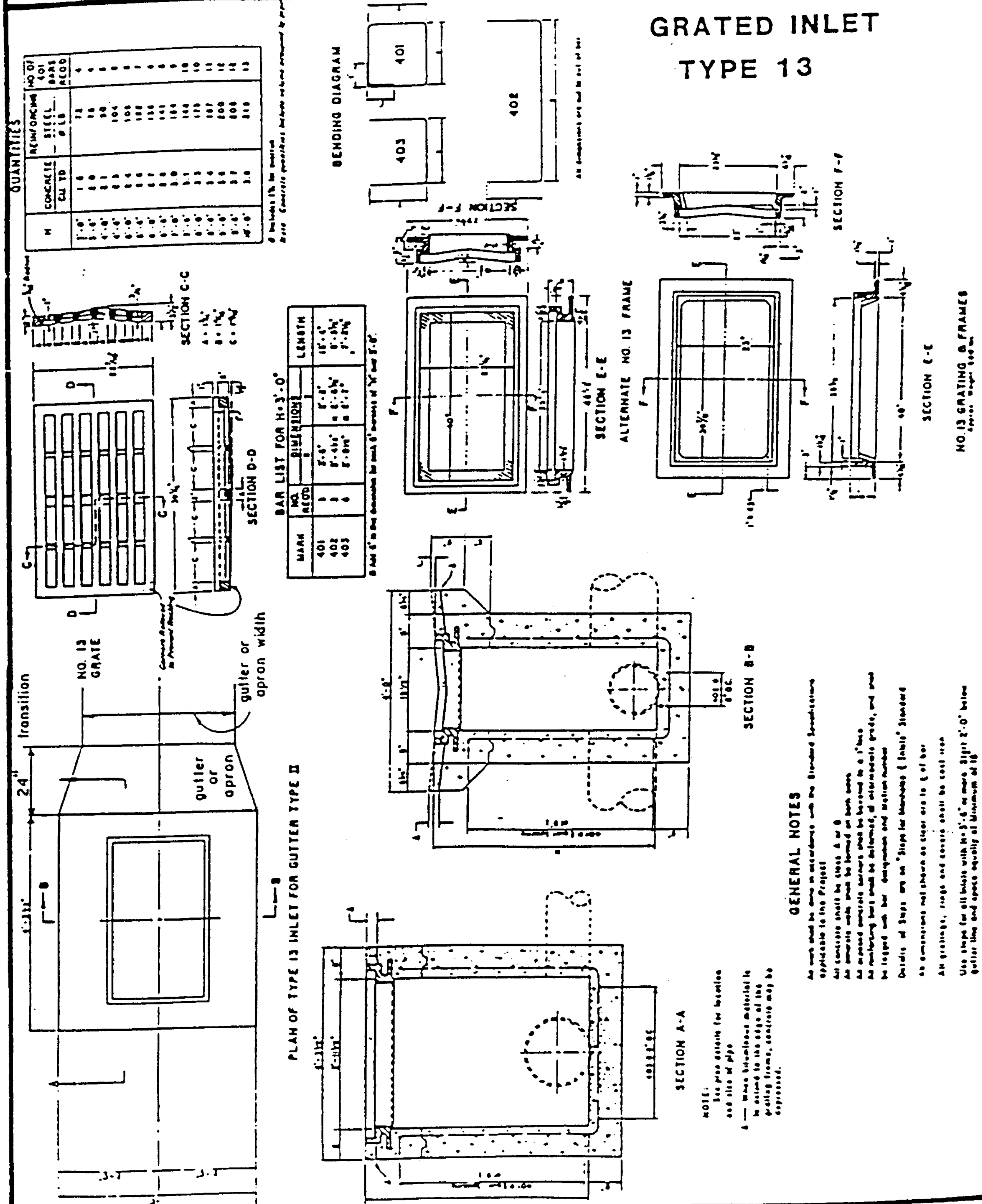



REFERENCE: Colorado Department of Highways
Standard M-604-8A (with modifications)

Figure 3.6-3

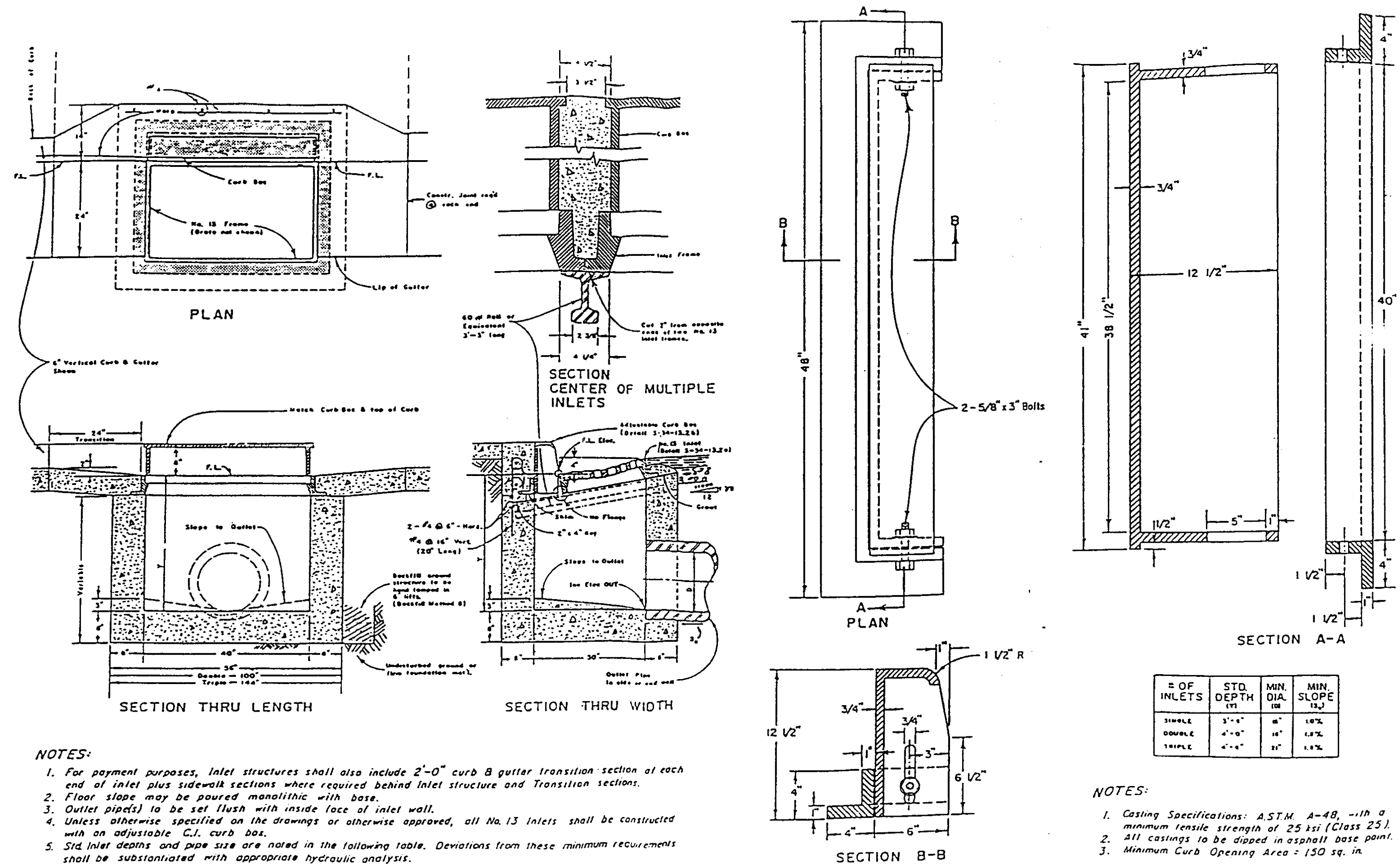
# STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

## GRATED INLET
## TYPE 13




REFERENCE:

Figure 3.6-4




NOTES:

1. For payment purposes, Inlet structures shall also include 2'-0" curb & gutter transition section at each end of inlet plus sidewalk sections where required behind Inlet structure and Transition sections.
2. Floor slope may be poured monolithic with base.
3. Outlet pipe(s) to be set flush with inside face of inlet wall.
4. Unless otherwise specified on the drawings or otherwise approved, all No. 13 Inlets shall be constructed with an adjustable C.I. curb box.
5. Std. Inlet depths and pipe size are noted in the following table. Deviations from these minimum requirements shall be substantiated with appropriate hydraulic analysis.

| # OF INLETS | STD. DEPTH (Y) | MIN. DIA. (D) | MIN. SLOPE (S) |
|---|---|---|---|
| SINGLE | 3'-6" | [illegible] | 1.0% |
| DOUBLE | 4'-0" | 18" | 1.5% |
| TRIPLE | 4'-6" | 21" | 1.5% |

NOTES:

1. Casting Specifications: A.S.T.M. A-48, with a minimum tensile strength of 25 ksi (Class 25).
2. All castings to be dipped in asphalt base paint.
3. Minimum Curb Opening Area = 150 sq. in.

REFERENCE:
City and County of Denver Standard Details S-34-13.1a and S-34-13.2b (with modifications)

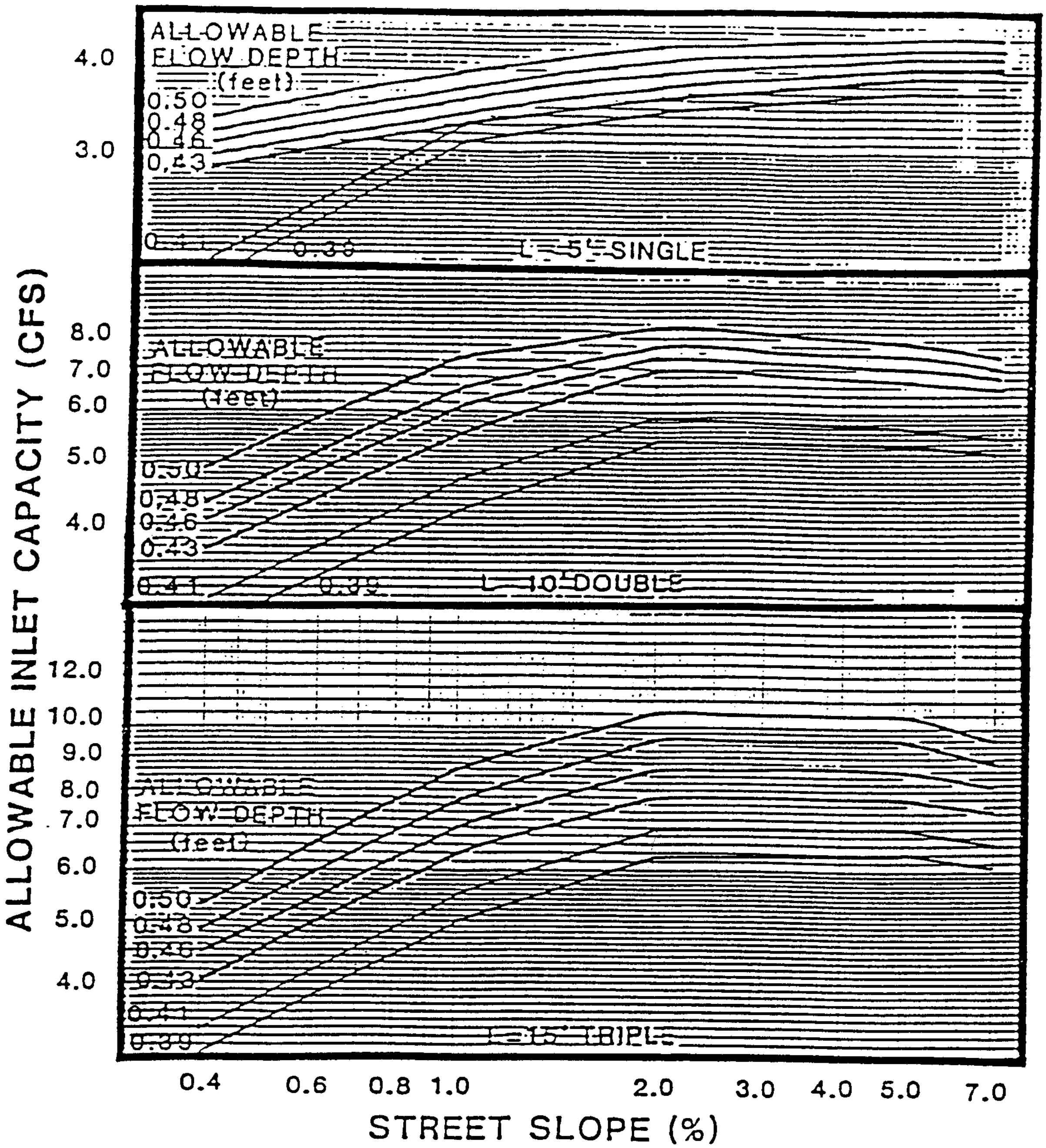
STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

COMBINATION INLET TYPE 13

STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

# ALLOWABLE INLET CAPACITY

## TYPE - R CURB OPENING ON A CONTINUOUS GRADE




NOTES: 1. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.

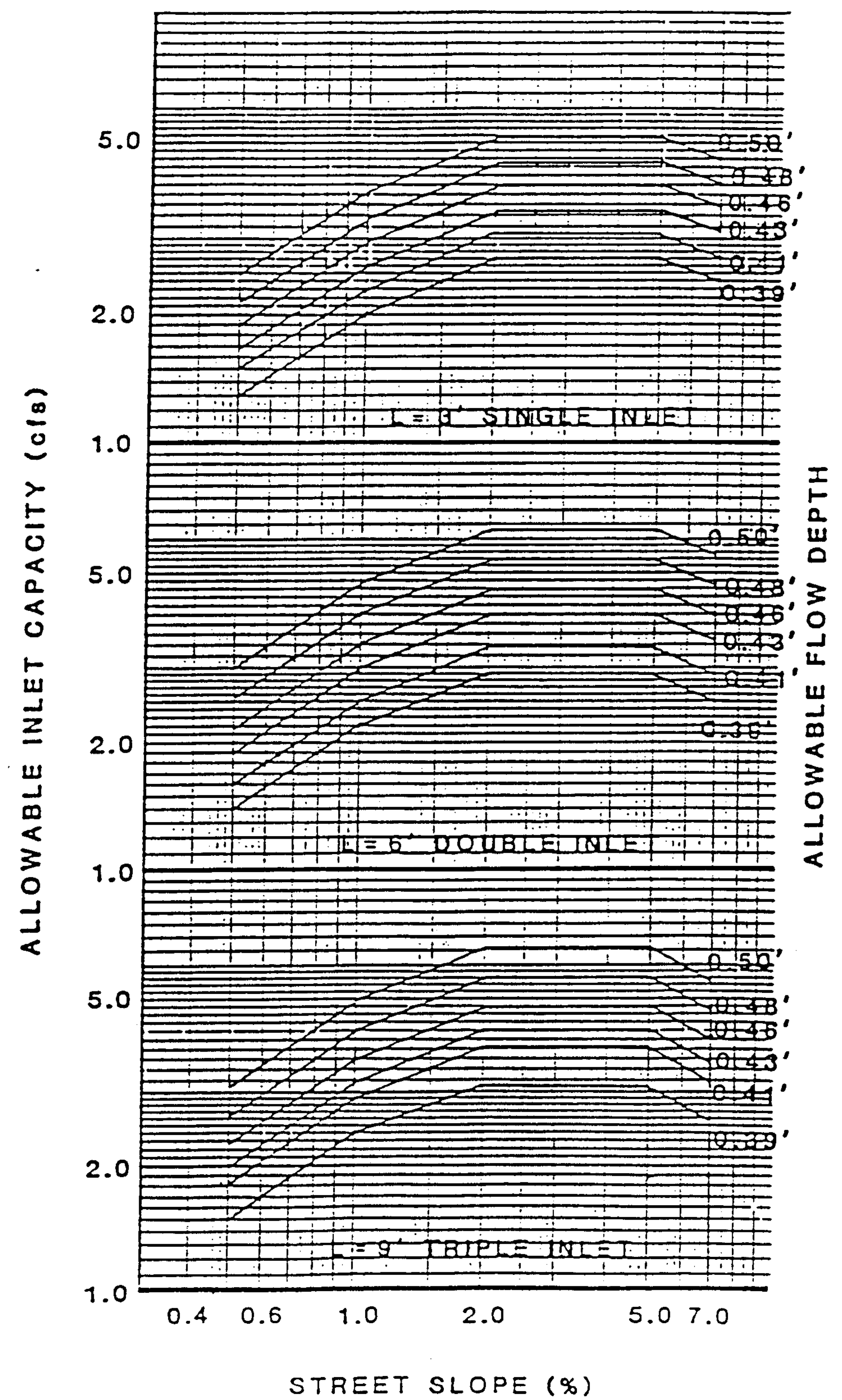
2. Allowable Capacity =
88% (L = 5')
92% (L = 10')
95% (L = 15')
of Theoretical Capacity

3. Interpolate for other inlet lengths.

STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

# ALLOWABLE INLET CAPACITY

## TYPE 13 GRATED ON A CONTINUOUS GRADE

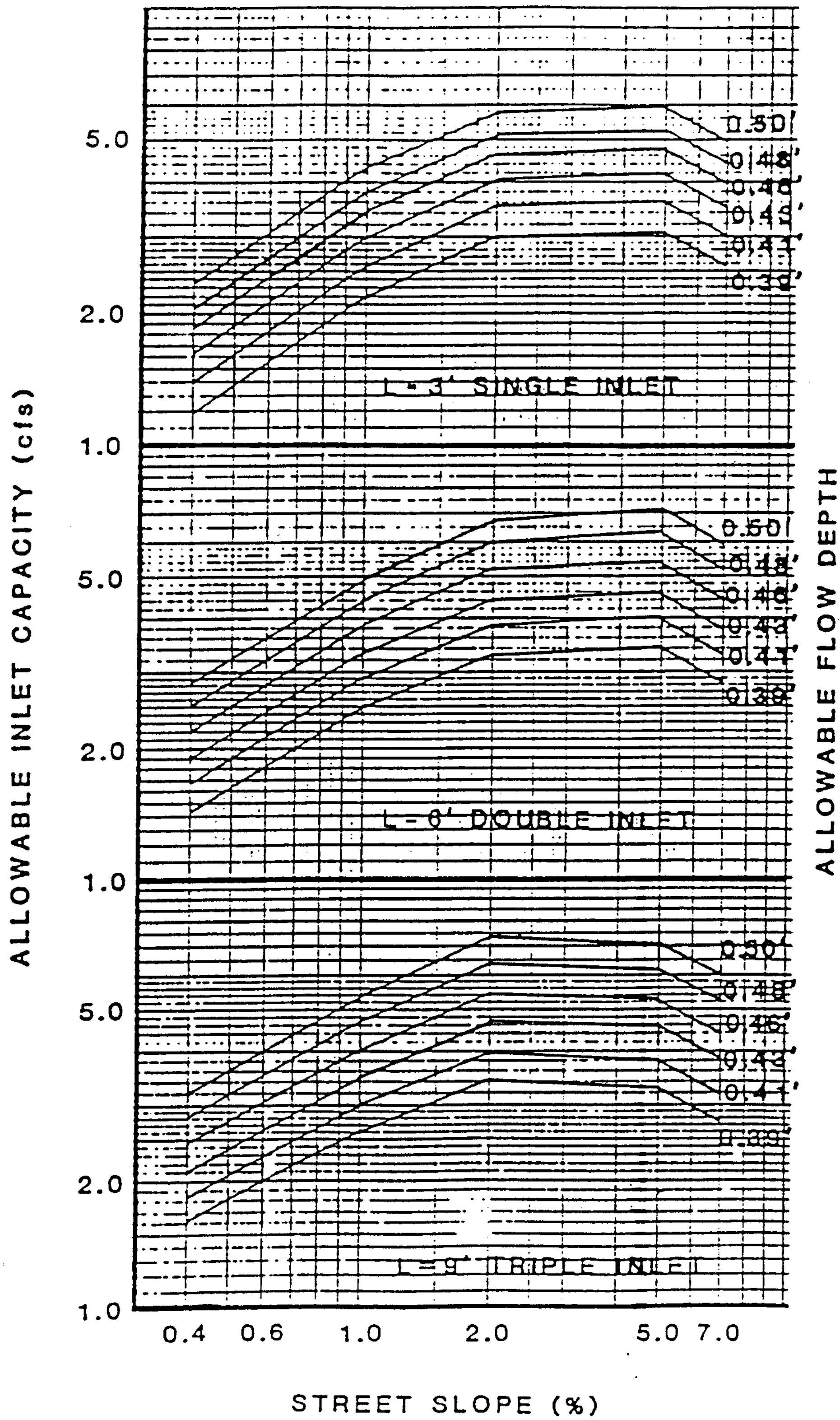



NOTES:
1. Allowable capacity = 60 % theoretical capacity
2. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.

STORM DRAINAGE DESIGN AND TECHNICAL CRITERIA

# ALLOWABLE INLET CAPACITY

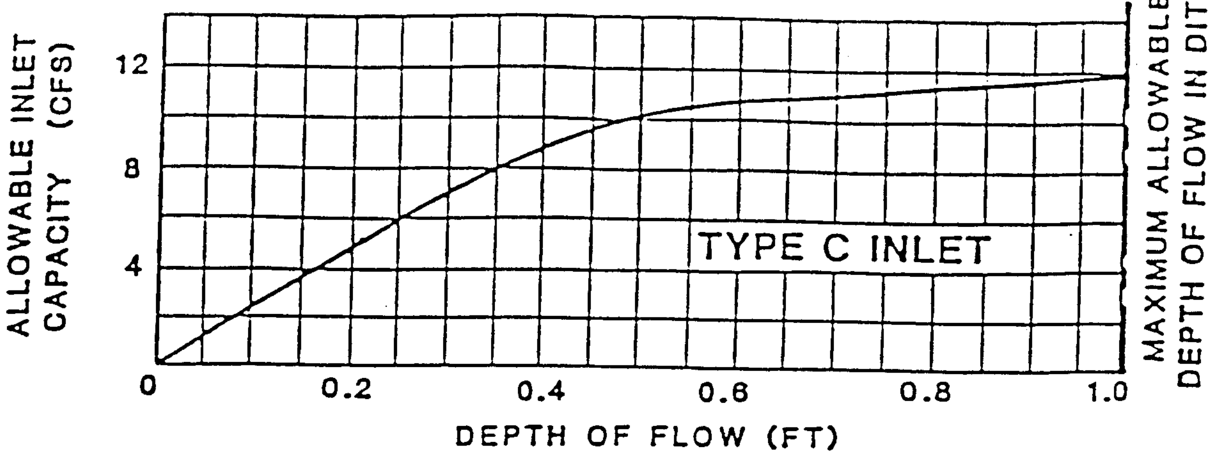
## TYPE 13 COMBINATION ON A CONTINUOUS GRADE

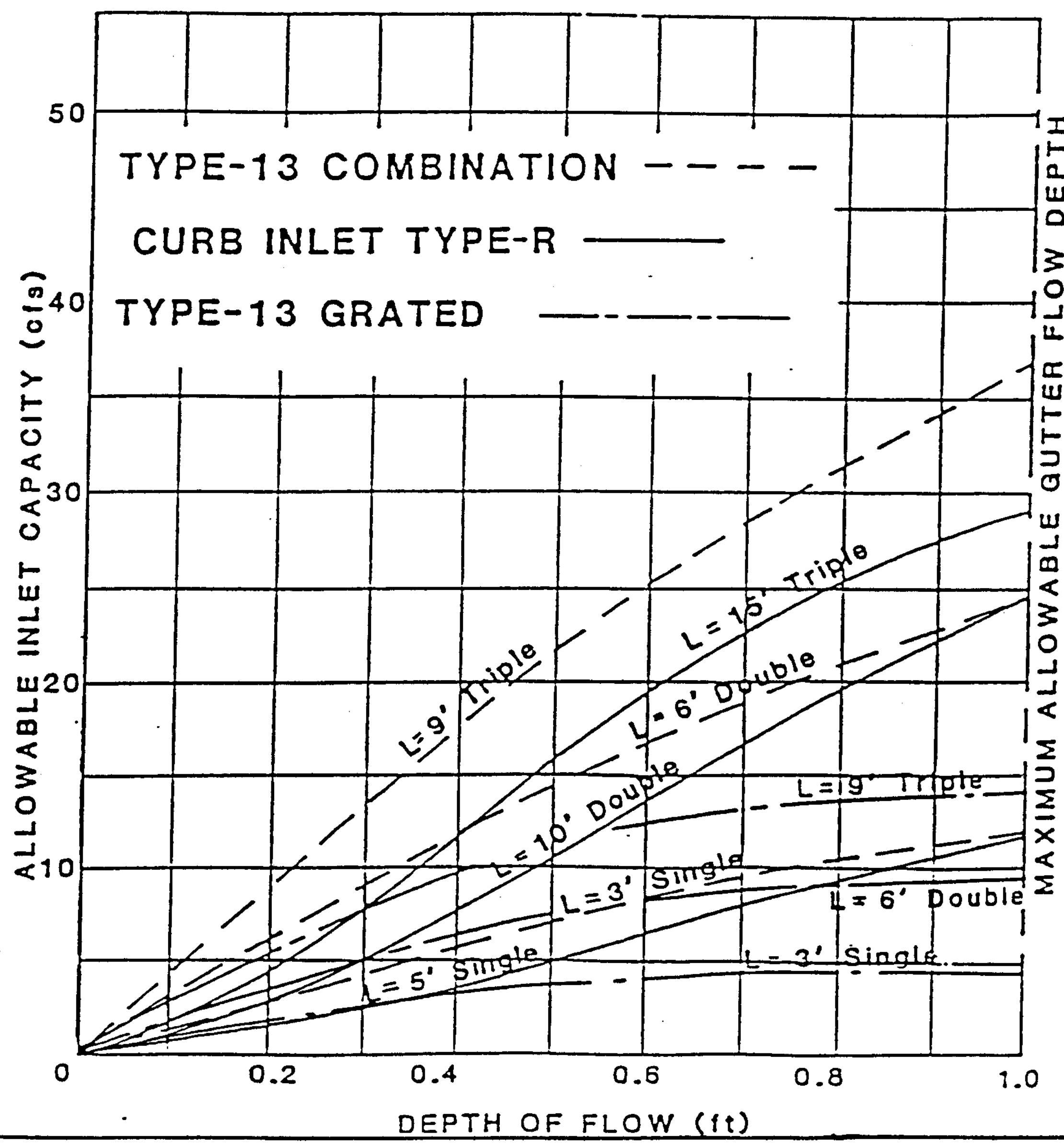



NOTES:
1. Allowable capacity = 66% theoretical capacity
2. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.

# ALLOWABLE INLET CAPACITY

## SUMP CONDITIONS - ALL INLETS



DESIGN OF TYPE R CURB OPENING INLETS (INITIAL STORM)

GIVEN: Street type = Arterial, 6 lane; S = 1.0 percent
Maximum flow depth = 0.5 feet (refer to Section 8)
Maximum allowable gutter capacity = 11.0 cfs
Starting gutter flow ($Q_o$) = 8.0 cfs

FIND: Interception and carryover amounts for the inlets and flow conditions illustrated on Figure 7-9.

SOLUTION: From Figure 7-9 we can see that inlets 1 and 2 are in a continuous grade condition and inlet 3 is in a sump condition. The first step is to calculate the interception ratio R, for the continuous grade inlets. This ratio is then applied to the actual gutter flow (local runoff plus carryover flow) to determine amount intercepted by the inlet and the carryover flow. The final step is to calculate the size of the inlet required for the sump condition, as discussed in the following section.

STEP 1: From Figure 7-6 for an allowable depth of 0.50 feet and a 15-foot inlet, read the value 8.6 cfs. Note that even though the gutter flow is less than maximum allowable, the maximum depth is used for Figure 7-6. The effect of the lower depth on the inlet capacity shall be accounted for in the following steps.

STEP 2: Compute the interception ratio R

$$R = \frac{\text{Allowable inlet capacity}}{\text{Allowable street capacity}} = \frac{8.6}{11.0}$$

$$R = 0.78$$

STEP 3: Compute the interception amount $Q_I$

$Q_I$ = R x Q street
= 0.78 x 8.0
= 6.2 cfs intercepted by inlet

STEP 4: Compute the carryover amount $Q_{co}$

$Q_{co}$ = Q street - $Q_I$
= 8.0 - 6.2
= 1.8 cfs

STEP 5: Compute the total flow at the next inlet, which is the sum of the carryover ($Q_{co}$) from inlet #1 plus the local to inlet #2

$Q_T$ (inlet #2) = $Q_{co}$ (inlet #1) + $Q_L$ (inlet #2)
= 1.8 cfs + 4 cfs
= 5.8 cfs

STEP 6: Compute the interception ratio, intercepted amount, and carryover flow for inlet #2 using the procedure described in Steps 1 through 4

Allowable inlet capacity = 7.2 cfs (Figure 7-6)

R = (7.2 cfs)/(11.0 cfs) = 0.65
$Q_I$ (inlet #2) = (0.65)(5.8 cfs) = 3.8 cfs
$Q_{co}$ (inlet #2) = 5.8 cfs - 3.8 cfs = 2.0 cfs

STEP 7: Compute the total flow at inlet #3 using the procedures described in Step 5

$Q_T$ (inlet #3) = 8 cfs + 2.0 cfs = 10.0 cfs

STEP 8: Size the inlet in the sump condition using the procedures described in the following section, for a sump condition. For this example, with an allowable maximum depth of flow of 0.5 ft, a 10-foot Type R inlet shall intercept more than the total gutter flow and is therefore acceptable.

## DESIGN EXAMPLE: ALLOWABLE CAPACITY FOR TYPE 13 INLET IN A SUMP (INITIAL STORM)

GIVEN: Flow = 8.0 cfs
Maximum allowable street depth = 0.50
Type 13 combination double inlet

FIND: Depth of ponding

SOLUTION:

STEP 1: From Figure 7-8 read the depth of ponding for a double Type 13 combination inlet as D = 0.28' at the gutter flow of 8.0 cfs (inlet capacity).

STEP 2: Compare computed to allowable depth. Since the computed depth is less than the allowable depth, the inlet is acceptable, otherwise the amount of inlets or the type of inlet would be changed and the procedure repeated.

## DESIGN EXAMPLE: INLET SPACING

GIVEN: Maximum allowable street flow depth = 0.50 ft.
Street slope = 1.0 percent
Maximum allowable gutter flow = 11.0 cfs
Gutter flow = 11.0 cfs

FIND: Size and type of inlet for 75 percent interception

SOLUTION:

STEP 1: Compute desired capacity

$$Q = 0.75\ (11.0\ \text{cfs}) = 8.3\ \text{cfs}$$

STEP 2: Read the allowable inlet capacities from Figures 7-5 and 7-6 for various inlets. The following values were obtained:

| Inlet Type | Capacity | % Interception |
|---|---|---|
| Triple Type 13 | 5.5 cfs | 50 |
| Triple Type R | 8.6 cfs | 78 |

Therefore, a curb opening inlet Type R, L = 15 feet is required and shall intercept 8.6 cfs. The remaining 2.4 cfs shall continue downstream and contribute to the next inlet. Spacing between such inlets shall depend on the local runoff, and the amount of flow bypassed at the upstream inlet.

A comparison of the inlet capacity with the allowable street capacity (refer to Section 8) shall show that the percent of street flow interception by the inlets varies from less than 50 percent to as much as 95 percent of the allowable street capacity. Therefore, the optimum inlet spacing cannot be achieved in all instances, and the spacing requirements should be analyzed by the design engineer.

## 3.7 OPEN CHANNELS

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 2 "Major Drainage" section.

### 3.7.1 Natural Channels

1. If supercritical flows are present, drop structures or other appropriate energy dissipation structures must be provided.

2. Segments which have a calculated Froude number greater than 0.8 for the 100-year storm runoff shall be protected from erosion.

3. A channel stability analysis will be completed to determine the impact of urbanization on the bank stabilization.

4. Structures constructed along the channel shall be elevated a minimum of one foot above the 100-year water surface level.

### 3.7.2 Grass Lined Channels

1. The maximum velocity for the 100-year flood peak shall not exceed 7.0 feet per second (fps), it shall not exceed 5.0 fps for sandy soils. (The City of Evans and its Urban Growth Area have predominantly sandy soils.) The minimum velocity shall be 2.0 for the initial storm runoff.

2. The Froude number should be kept below 0.8.

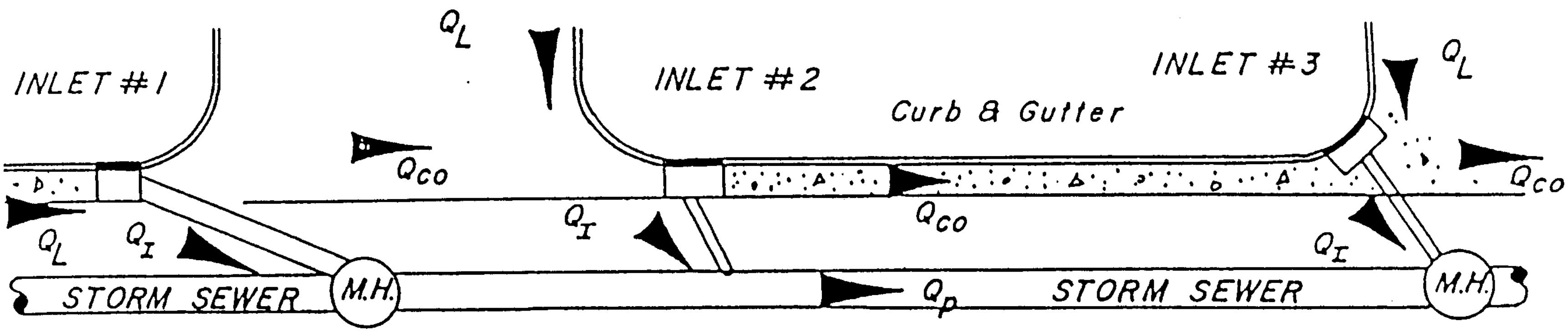
3. Minimum freeboard shall be 1.0 feet.

4. Variation of Manning's "n" with the retardance and the product of mean velocity and hydraulic radius shall be used in the capacity computations. The corresponding retardance curves are presented in Figure 3.7-1. Retardance curve C shall be used to determine channel capacity. Retardance curve D shall be used to determine the limiting velocity.

5. Representative cross-sections of suitable channels and their limitations for design are shown in Figures 3.7-2 thru 3.7-5

### 3.7.3 Concrete Lined Channels

1. The surface of the concrete lining shall be provided with a wood float finish. Excessive working or wetting of the finish shall be avoided.




LEGEND $Q_L$ = Local runoff for design storm tributary to designated inlet (cfs)

$Q_I$ = Runoff intercepted by inlet (cfs)

$Q_{CO}$ = Carry over runoff past inlet (cfs)

$Q_T$ = Total runoff at inlet = $Q_L + Q_{CO}$

$Q_P$ = Runoff in Pipe

SUMMARY OF FLOWS
FOR DESIGN EXAMPLE 3

| INLET | $Q^*$ ALLOW | $Q_L$ | $Q_{CO}$ | $Q_T$ | $Q_I$ | $Q_{CO}$ | $Q_P$ SEWER | COMMENTS |
|---|---|---|---|---|---|---|---|---|
| NO. 1, 15' TYPE R | 8.6 | 8 | 0 | 8 | 6.2 | 1.8 | 6.2 | Inlet on Grade |
| NO. 2, 10' TYPE R | 7.2 | 4 | 1.8 | 5.8 | 3.8 | 2.0 | 10.0 | Inlet on Grade |
| NO. 3, 10' TYPE R | 10.4 | 8 | 2.0 | 10.0 | 10.0 | 0 | 20.0 | Inlet in Sump Condition |

* Maximum allowable inlet capacity at maximum allowable gutter capacity, from Figures- 3.6-5

## INLET DESIGN EXAMPLES - INITIAL STORM

3.7.4 Riprap Channel Linings

1. Channels shall be designed for a Froude number (turbulence factor) less than 0.8.

2. Freeboard and maintenance access requirements shall be in accordance with the standards for grass lined channels.

3.7.5 Other Lining Types

1. The criteria for channel linings other than grass, rock, or concrete will be based on the manufacturer's recommendations for the specific product.

2. Technical data in support of the proposed material must be submitted with the drainage report
3. The Froude number shall be less than 0.8.

4. The freeboard requirements and cross section limitations will be the same as for grass lined channels.

5. The center line of curvature shall have a minimum radius twice the top width of the design flow but not less than 100 feet.

6. A Manning's "n" value range shall be determined by the manufacturer. The high value shall be used for depth/capacity requirements and the low value used for Froude number and velocity restriction calculations.

3.7.6 Roadside Ditches

1. Representative roadside ditch sections are shown in Figure 3.7-6.

2. Roadside ditches shall have adequate capacity for the initial storm runoff peaks. Where storm runoff exceed the ditch capacity, a storm sewer system shall be required.

3. The maximum velocity is 5 feet per second for a Type I ditch, and 7 feet per second for Type II and Type III ditches.

4. The longitudinal slope shall be limited by the average velocity.

5. Freeboard shall be equal to the velocity head, or a minimum of 6 inches.

6. The minimum radius of curvature shall be 25 feet.

7. The Manning's "n" values shall be the same as for grass lined channels.

8. The grass lining shall meet the requirements for grass lined channels.

9. Driveway culverts shall be sized to convey the initial storm flows without overtopping of the driveway. The minimum diameter shall be 12 inches (or equivalent) with flared end sections.

10. The capacity of the ditch for the major storm drainage shall be restricted by the allowable flow depth at the street crown.

3.7.7 Channel Rundowns

1. Typical cross sections for channel rundowns are shown in Figure 3.7-7.

2. The rundown shall be designed to carry the initial storm runoff or 1 cfs, whichever is greater.

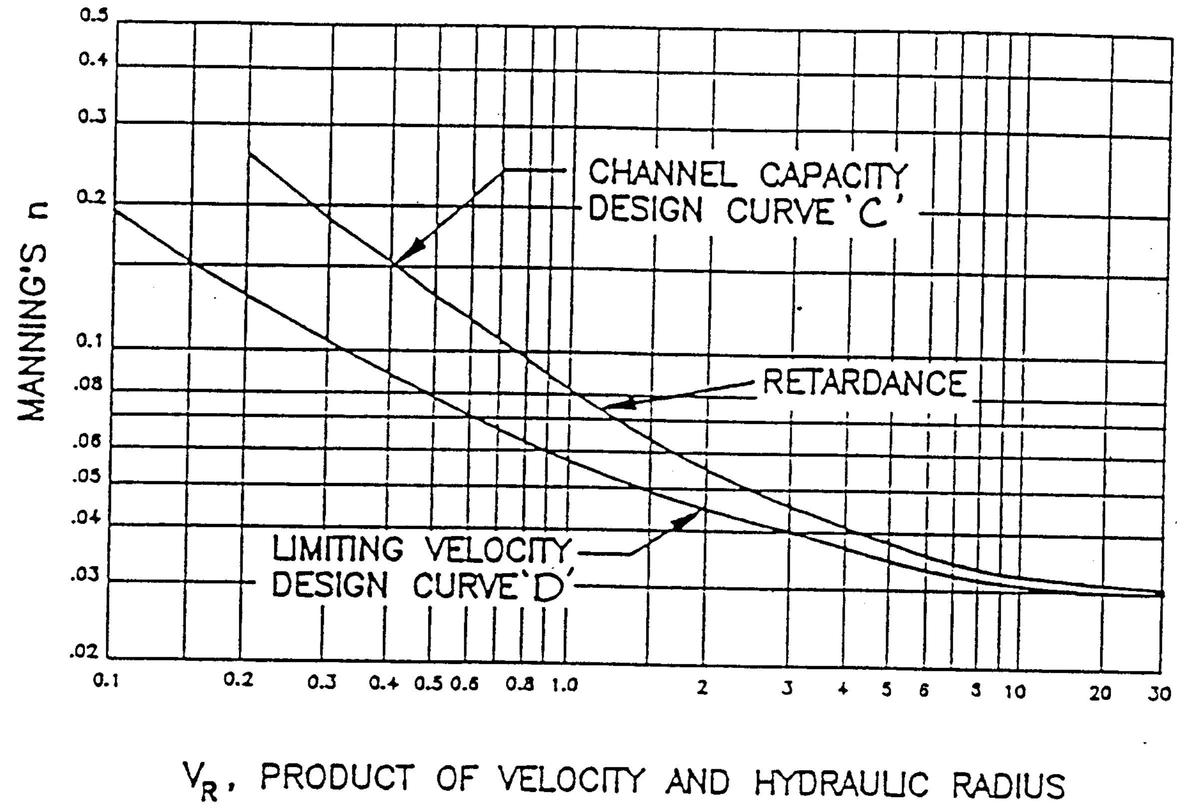
3. The maximum depth shall be 12 inches. The design depth of the rundown shall be the computed critical depth for the design flow.

4. The rundown outlet shall enter the drainageway at the trickle channel flowline. Erosion protection of the opposite channel bank shall be provided by a 24 inch layer of grouted Type L riprap. The width of the riprap shall be a minimum 3 times the channel rundown width or pipe diameter. Riprap shall extend up the opposite bank to the initial storm flow depth in the drainageway or 2 feet, whichever is greater.

3.7.8 Maintenance and Access Easements

1. Minimum maintenance and access easements shall be the same as those required by the City of Greeley criteria. The following easement widths are required:

| MINIMUM CHANNEL EASEMENT WIDTHS | |
|---|---|
| Channel Size | Total ROW or Easement Width |
| Q(100) < 20 cfs | 15 feet |
| Q(100) < 100 cfs | 25 feet |
| Q(100) > 100 cfs | Freeboard + 12 foot wide access road(s) Access may be required on both sides of the channel. |

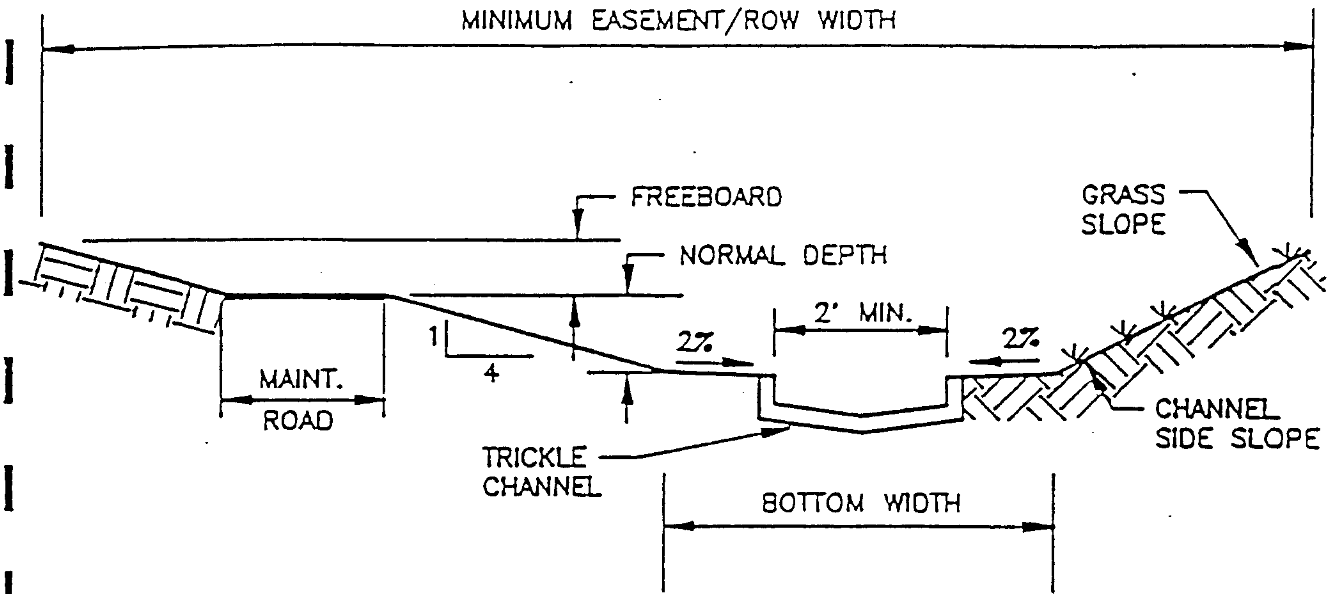



NOTE: FROM "HANDBOOK OF CHANNEL DESIGN FOR SOIL AND WATER CONSERVATION," U.S. DEPARTMENT OF AGRICULTURE. SOILS CONSERVATION SERVICE. NO. SCS-TP-61 MARCH, 1947 REVISED JUNE. 1954

N.T.S.

# ROUGHNESS COEFFICIENT FOR GRASS CHANNELS

# TYPE A




NOTES:

1. BOTTOM WIDTH: CONSISTENT WITH MAXIMUM ALLOWABLE DEPTH AND VELOCITY REQUIREMENTS, SHALL NOT BE LESS THAN TRICKLE CHANNEL WIDTH.

2. TRICKLE CHANNEL: MINIMUM CAPACITY TO BE 2% TO 4% OF 100-YEAR FLOW BUT NOT LESS THAN 1 CFS. CHANNEL TO BE CONSTRUCTED OF CONCRETE, GROUTED RIPRAP, OR OTHER APPROVED MATERIALS.

3. NORMAL DEPTH: NORMAL DEPTH AT 100-YEAR FLOW SHALL NOT EXCEED 5 FEET. MAXIMUM 100-YEAR FLOW VELOCITY AT NORMAL DEPTH SHALL NOT EXCEED 7 FPS.

4. FREEBOARD: FREEBOARD TO BE A MINIMUM OF 1 FOOT.

5. MAINTENANCE ACCESS ROAD: MINIMUM WIDTH TO BE 12 FEET. CITY MAY REQUIRE ALL OR PART OF THE ROAD TO BE SURFACED.

6. EASEMENT/ROW WIDTH: MINIMUM WIDTH TO INCLUDE FREEBOARD AND MAINTENANCE ACCESS ROAD.

7. CHANNEL SIDE SLOPE: MAXIMUM SIDE SLOPE FOR GRASSED CHANNELS TO BE 3:1.

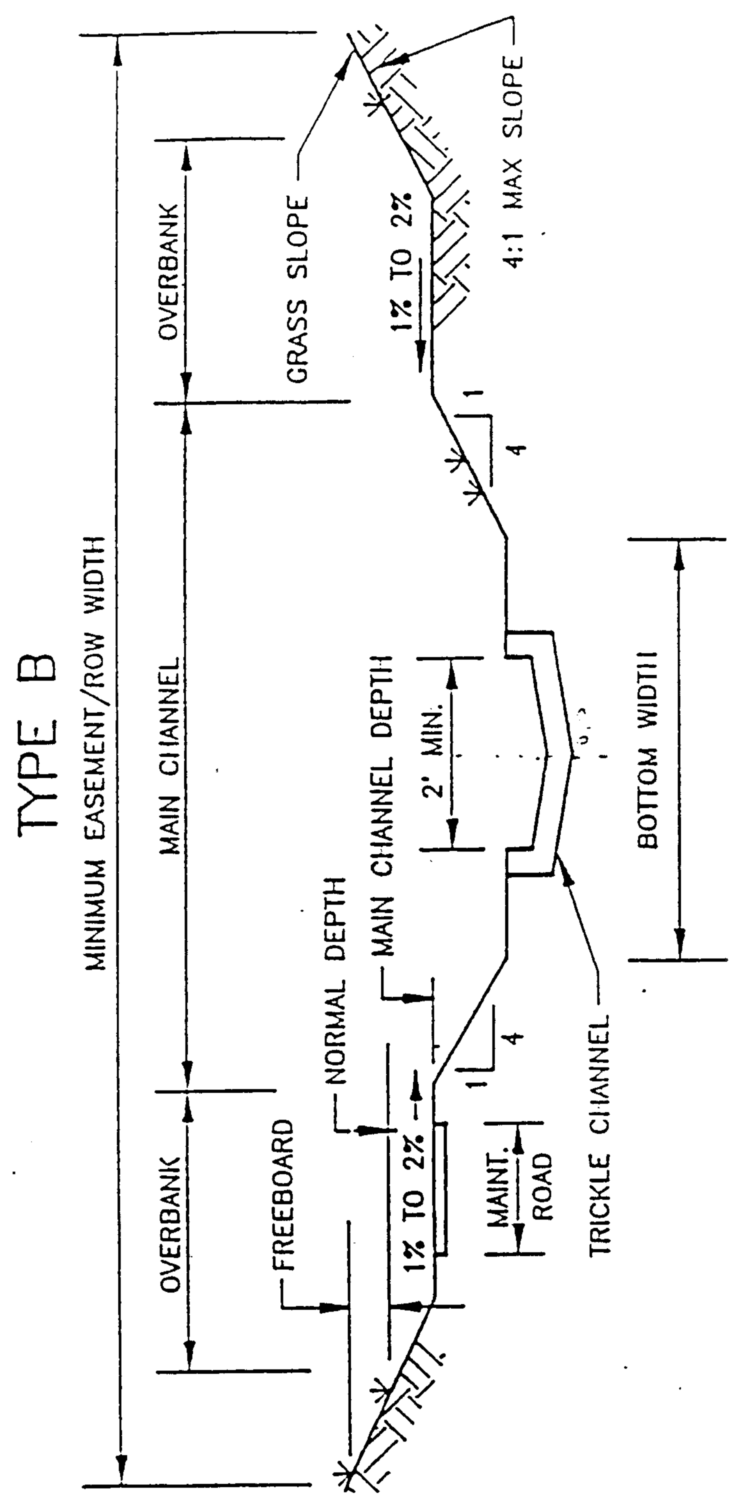
8. FROUDE NUMBER: MAXIMUM VALUE SHALL NOT EXCEED 0.8 FOR MINOR AND MAJOR FLOODS.

9. LOW FLOW PIPE MAY BE USED IN LIEU OF TRICKLE CHANNEL.

N.T.S.

# TYPICAL GRASS LINED CHANNEL SECTION

Figure 3.7-3




NOTE:

1. MAIN CHANNEL: CAPACITY TO BE NO LESS THAN 20% OF 100-YEAR AT MAIN CHANNEL DEPTH. MAXIMUM 100-YEAR FLOW VELOCITY IS 7 FPS.

2. TRICKLE CHANNEL: MINIMUM CAPACITY TO BE 2% TO 4% OF 100-YEAR FLOW BUT NOT LESS THAN 1 CFS. CHANNEL TO BE CONSTRUCTED OF CONCRETE, GROUTED RIPRAP OR OTHER APPROVED MATERIALS. SEE FIGURE 3.7-4 FOR REQUIREMENTS IN SANDY SOILS.

3. NORMAL DEPTH: FLOW DEPTH FOR 100-YEAR FLOW SHALL NOT EXCEED 5 FEET(Including main channel depth)

4. FREEBOARD: FREEBOARD TO BE A MINIMUM OF 1 FOOT.

5. MAINTENANCE ACCESS ROAD: MINIMUM WIDTH TO BE 12 FEET. CITY MAY REQUIRE ALL OR PART OF THE ROAD TO BE SURFACED.

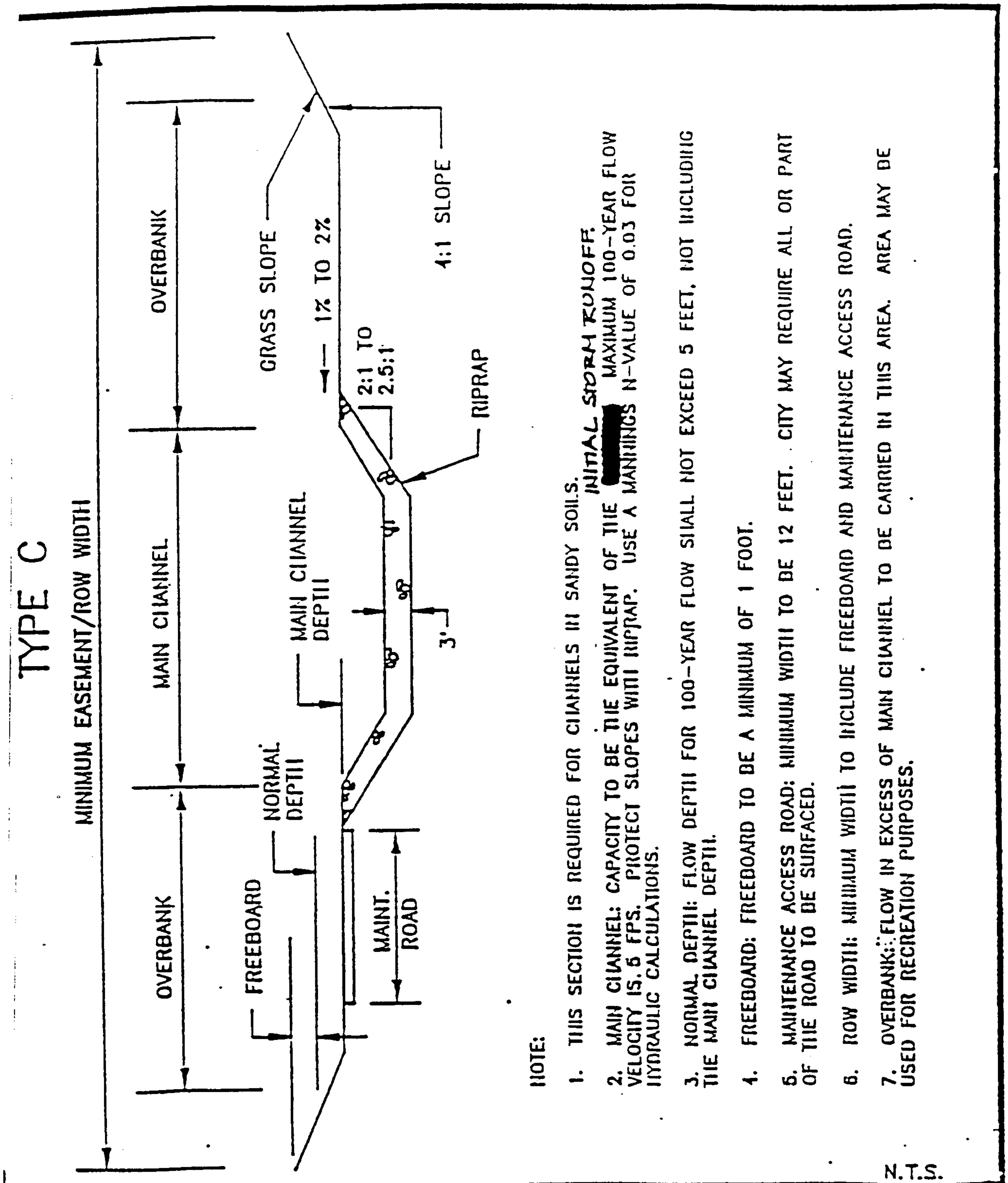
6. EASEMENT/ROW WIDTH: MINIMUM WIDTH TO INCLUDE FREEBOARD AND MAINTENANCE ACCESS ROAD.

7. OVERBANK: FLOW IN EXCESS OF MAIN CHANNEL TO BE CARRIED IN THIS AREA. AREA MAY BE USED FOR RECREATION PURPOSES.

N.T.S.

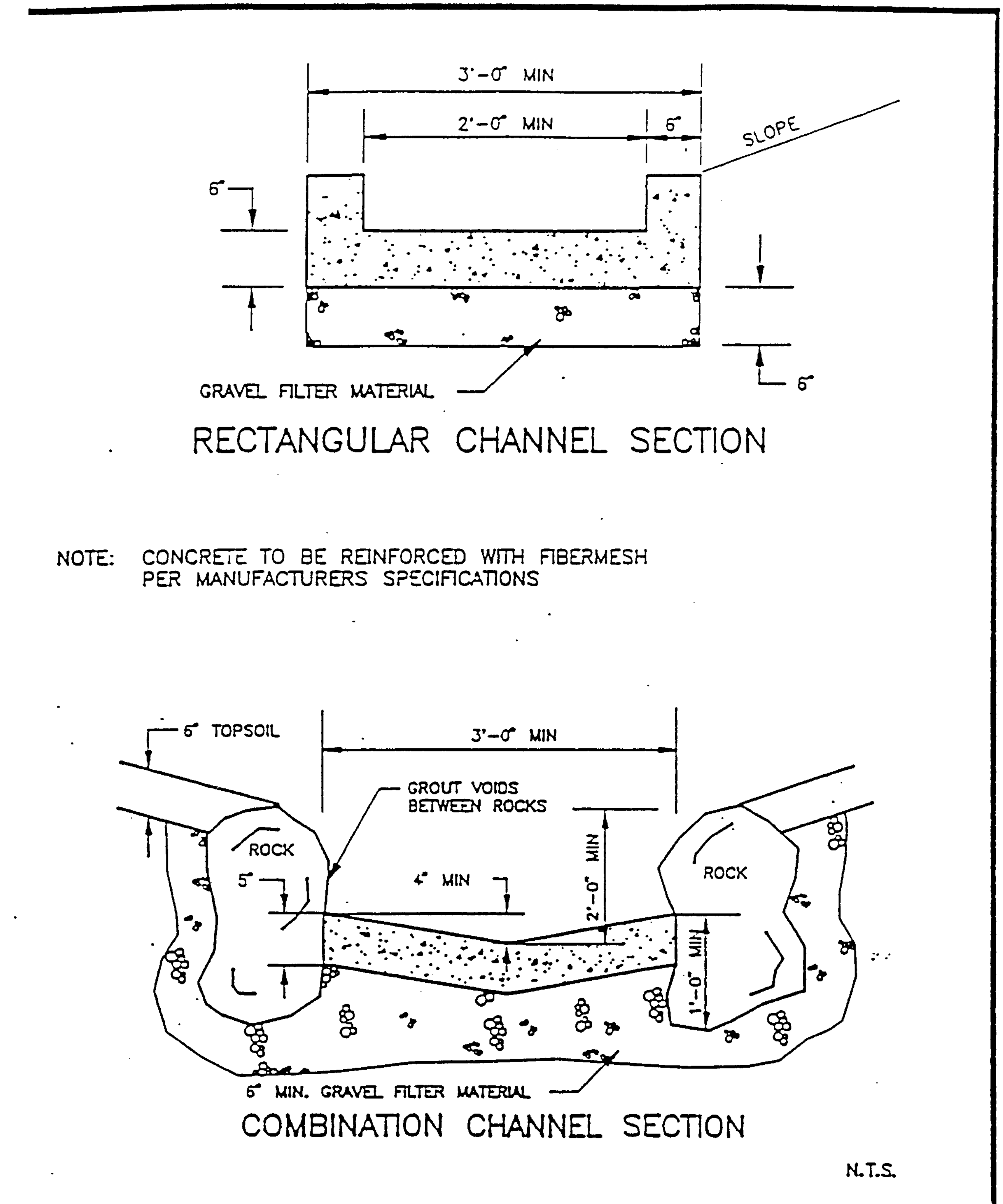
TYPICAL GRASS LINED CHANNEL SECTION

Figure 3.7-4




TYPICAL GRASS LINED
CHANNEL SECTION
FOR USE IN SANDY SOILS

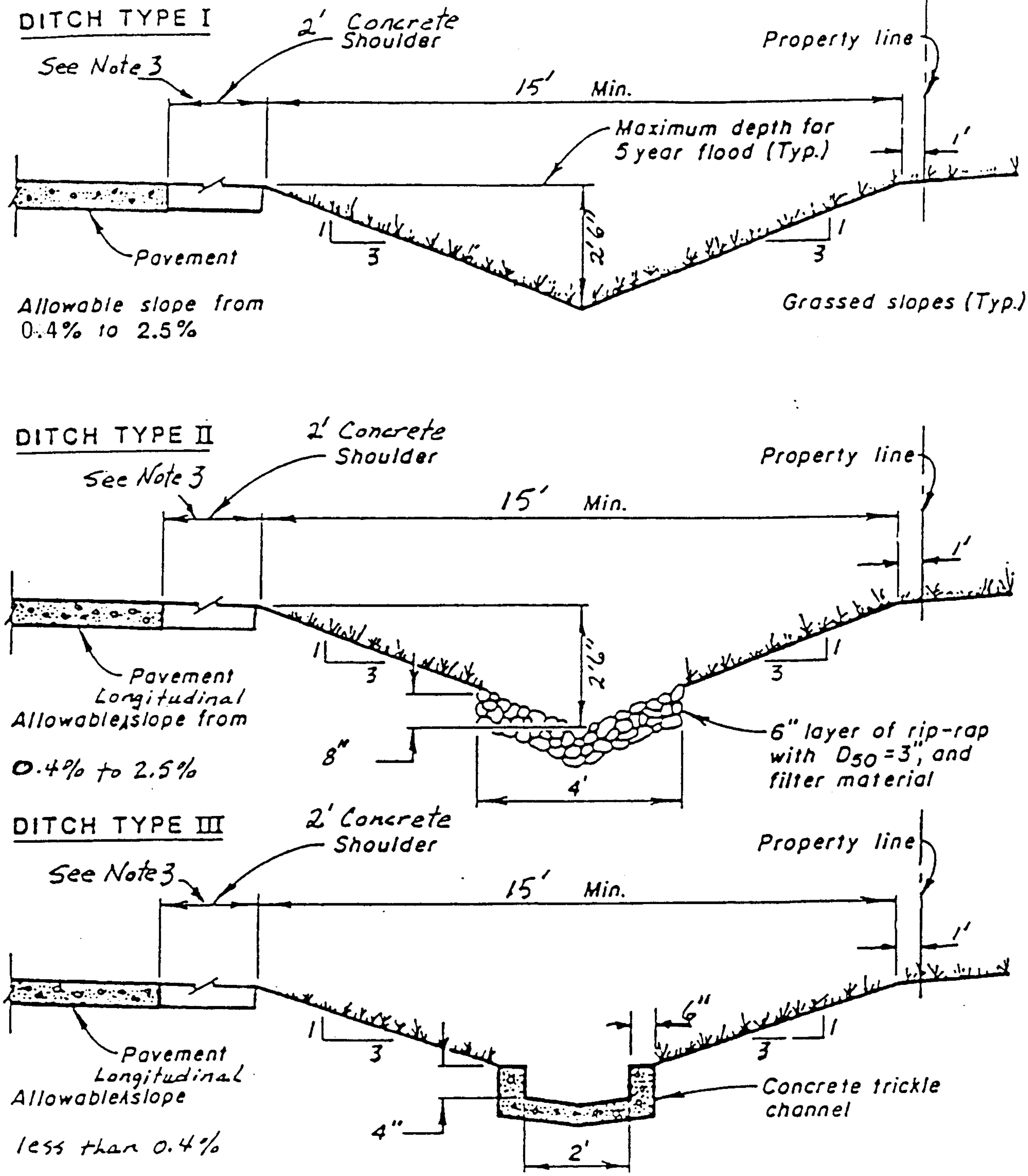
Figure 3.7-5




TRICKLE CHANNEL DETAILS

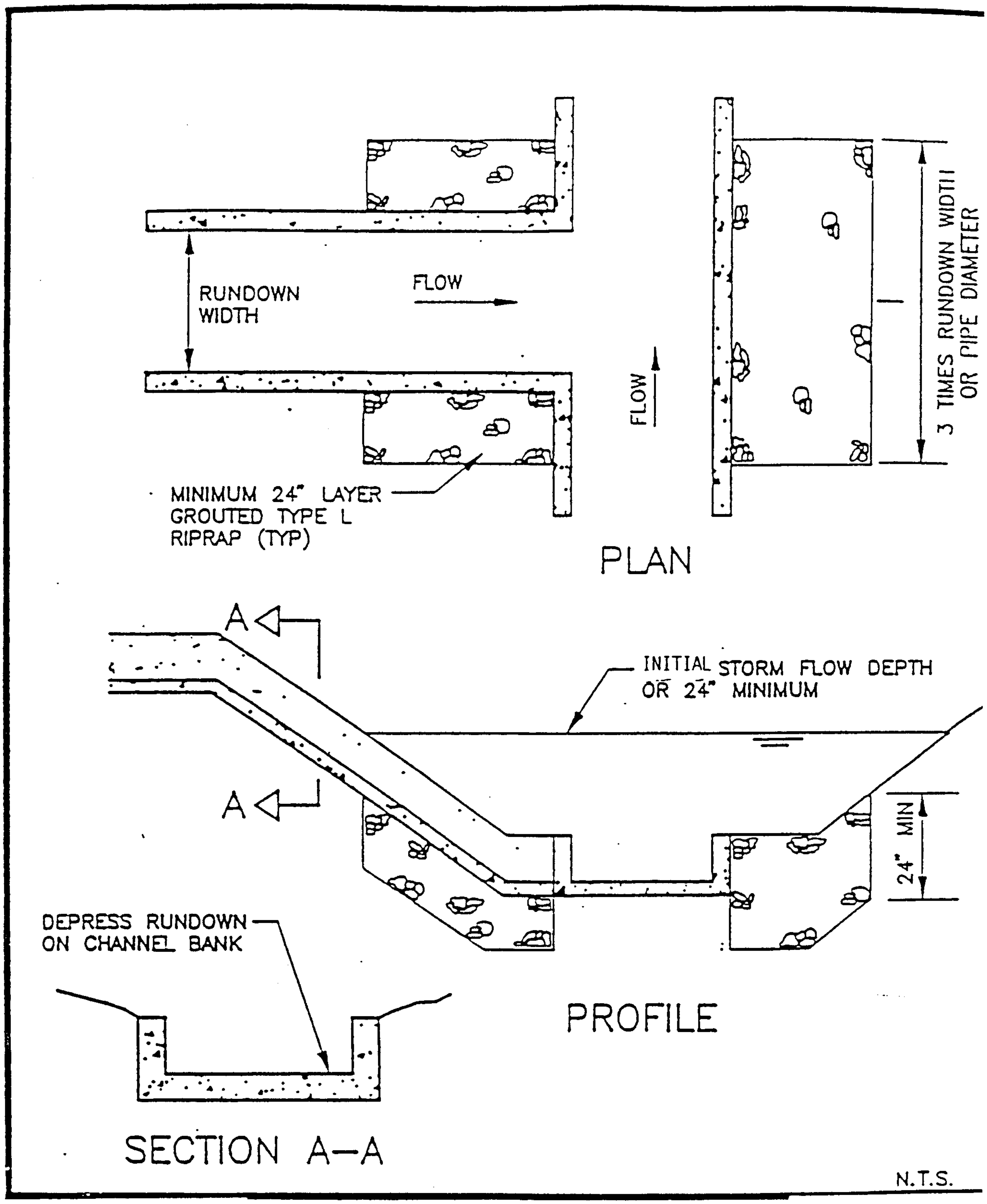
FIGURE

# ROADSIDE DITCH SECTIONS




NOTE: 1. For street slopes greater than maximum allowable, check drops (2' maximum height) will be required.
2. Street cross section may include concrete curb and gutter.
3. Provide 2' wide 8" thick concrete curb, see street standards Detail No. S-1.

Figure 3.7-7




CHANNEL RUNDOWN

## 3.8 HYDRAULIC STRUCTURES

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 2 "Major Drainage" and "Structures" sections.

1. For culverts or storm sewers where the Froude number at the outlet is in excess of 2.5, the USBR Type VI impact stilling basin shall be used.

2. The design capacity for bridges shall be determined by the method used for culvert sizing.

3. Any drainage plan in which surface drainage crosses or utilizes irrigation facilities shall have the plans approved by the controlling ditch company prior to acceptance by the City.

## 3.9 CULVERTS

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 2 "Inlets and Culverts" section.

1. The basic procedures and requirements to be used for the hydraulic evaluation of culverts shall be in accordance with USDCM Volume 2.

2. Culverts shall be designed with headwalls and wingwalls or with flared-end sections. Riprap or concrete shall also be required at the inlet and outlet due to potential scouring velocities.

3. Roughness coefficients and entrance loss coefficients to be used in analysis are presented in Table 3.9-1.

4. Capacity curves and nomographs to be used in analysis are found in USDCM Volume 2.

5. A minimum outlet velocity of 3 feet per second is required.

6. A maximum outlet velocity of 12 feet per second is recommended with erosion protection. Refer to Section 3.8 for protection requirements at culvert outlets.

7. The maximum headwater for the 100-year design flow will normally be 1.5 times the culvert diameter or the culvert rise dimension but may be dictated by the allowable street overtopping requirements in Section 3.5. For headwater depths greater than 1.5, the applicant shall submit detailed calculations of the outlet velocity. For velocities greater than 12 feet per second, an energy dissipator may be required.

8. Structural design of culverts shall be in accordance with AASHTO "Standard Specifications for Highway Bridges", and with pipe manufacturers' recommendations.

9. Trash racks may be required by the city at some culvert openings. If required, the hydraulic loss through the trashrack shall be computed using the following equation:

$$H = 0.11\ (TV/D^{2})(\sin A)$$

Where:

H = head loss through trashrack (feet)
T = thickness of trashrack bar (inches)
V = velocity normal to trashrack (fps)
D = center to center spacing of bars (inches)
A = angle of inclination of rack with horizontal

10. The culvert shall be sized for the difference between the 100-year runoff and the allowable street overtopping or the size required to convey the 10-year runoff without overtopping, whichever is greater.

## (A) MANNING'S n—VALUES FOR CORRUGATED STEEL PIPE

| CORRUGATIONS | ANNULAR 2-2/3"x1/2" | HELICAL | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 1-1/2"x1/4" #11.13 | | 2-2/3"x1/2" | | | | |
| | ALL DIAMETER | 8" | 10" | 12" | 18" | 24" | 36" | 48" |
| UNPAVED | .024 | .012 | .014 | .011 | .014 | .016 | .019 | .020 |
| 25% PAVED | .021 | | | | | .015 | .017 | .020 |
| FULLY PAVED | .012 | | | | | .012 | .012 | .012 |

| CORRUGATIONS | ANNULAR 3" x 1" | HELICAL—3" x 1" | | | | | |
|---|---|---|---|---|---|---|---|
| | ALL DIAMETER | 36" | 48" | 54" | 60" | 66" | 72" |
| UNPAVED | .027 | .021 | .023 | .023 | .024 | .025 | .026 |
| 25% PAVED | .023 | .019 | .020 | .020 | .021 | .022 | .022 |
| FULLY PAVED | .012 | .012 | .012 | .012 | .012 | .012 | .012 |

## (B) MANNING'S n—VALUES FOR STRUCTURAL PLATE METAL PIPE

| CORRUGATIONS 6" x 2" | DIAMETER | | | |
|---|---|---|---|---|
| | 5 FT | 7 FT | 10 FT | 15 FT |
| PLAIN—UNPAVED | .033 | .032 | .030 | .028 |
| 25% PAVED | .028 | .027 | .026 | .024 |

## (C) MANNING'S n—VALUES FOR CONCRETE PIPE/CULVERT

| | |
|---|---|
| PRE—CAST | 0.012 |
| CAST—IN—PLACE | ——— |
| WITH STEEL FORMS | 0.013 |
| WITH WOOD FORMS | 0.015 |

# HYDRAULIC DATA FOR CULVERTS

# (D) CULVERT ENTRANCE LOSSES

| TYPE OF ENTRANCE | ENTRANCE COEFFICIENT, Ke |
|---|---|
| **PIPE** | |
| HEADWALL | |
| GROOVED EDGE | 0.20 |
| ROUNDED EDGE (0.15D RADIUS) | 0.15 |
| ROUNDED EDGE (0.25D RADIUS) | 0.10 |
| SQUARE EDGE (CUT CONCRETE AND CMP) | 0.40 |
| HEADWALL & 45° WINGWALL | |
| GROOVED EDGE | 0.20 |
| SQUARE EDGE | 0.35 |
| HEADWALL WITH PARALLEL WINGWALLS SPACED 1.25D APART | |
| GROOVED EDGE | 0.30 |
| SQUARE EDGE | 0.40 |
| BEVELED EDGE | 0.25 |
| PROJECTING ENTRANCE | |
| GROOVED EDGE (RCP) | 0.25 |
| SQUARE EDGE (RCP) | 0.50 |
| SHARP EDGE, THIN WALL (CMP) | 0.90 |
| SLOPING ENTRANCE | |
| MITERED TO CONFORM TO SLOPE | 0.70 |
| FLARED-END SECTION | 0.50 |
| **BOX, REINFORCED CONCRETE** | |
| HEADWALL PARALLEL TO EMBANKMENT (NO WINGWALLS) | |
| SQUARE EDGE ON 3 EDGES | 0.50 |
| ROUNDED ON 3 EDGES TO RADIUS OF 1/12 BARREL DIMENSION | 0.20 |
| WINGWALLS AT 30° TO 75° TO BARREL | |
| SQUARE EDGE AT CROWN | 0.40 |
| CROWN EDGE ROUNDED TO RADIUS OF 1/12 BARREL DIMENSION | 0.20 |
| WINGWALLS AT 10° TO 30° TO BARREL | |
| SQUARE EDGE AT CROWN | 0.50 |
| WINGWALLS PARALLEL (EXTENSION OF SIDES) | |
| SQUARE EDGE AT CROWN | 0.70 |

NOTE: THE ENTRANCE LOSS COEFFICIENTS ARE USED TO EVALUATE THE CULVERT OR SEWER CAPACITY OPERATING UNDER OUTLET CONTROL.

## HYDRAULIC DATA FOR CULVERTS

## 3.10 DETENTION

The following design criteria are in addition to the requirements and recommendations set forth in the USDCM Volume 2 "Storage" section.

1. On site detention is required by all proposed residential and commercial developments unless specifically waived by the City of Evans. Examples of when the detention requirements may be waived are:
   a) development on the site decreases the percentage of impervious area already present
   b) the site is adjacent to a major outfall and runoff will not influence its time to peak or adversely impact downstream facilities
   c) the latter phase of a subdivision is submitted and the previous phases have already met the detention requirements for the entire site

2. The detention facility shall be designed such that the allowable detention release rate for the 5-year storm shall be the historic 5-year recurrence interval runoff discharge.

3. The detention facility shall be designed such that the allowable detention release rate for the 100-year storm shall be 1cfs per acre of contributing drainage area. (The historic 100-year runoff is approximately 1 cfs per acre.) If the development is within Weld County jurisdiction, Weld County's more restrictive detention criteria will apply. Weld County requires a 5-year historic release rate for the 100-year design storm.

4. The minimum required detention volume shall be determined using either the Rational Formula Method or the CUHP Method as outlined in USDCM Volume 2 or by the EPA SWMM computer program. For basins larger than 5 acres, the CUHP Method is recommended. An example of the use of the Rational Method is presented below.

5. The detention pond shall include a trickle channel for low flow conditions. The trickle channel shall meet the requirements for the trickle channel of a grass lined open channel.

6. The side slope of detention facilities shall be no greater than 4H:1V for earthen embankments. All earthen embankments shall be revegetated with grass or covered with riprap. Riprap covered embankments may have a maximum slope of 3H:1V. For embankments greater than 10 feet in height, the side slope shall be

such to maintain slope stability.

7. The minimum bottom slope shall be 0.5 percent, measured perpendicular to the trickle channel.

8. The minimum freeboard requirement is 1 foot above the computed 100-year water surface elevation.

9. Presented in Figures 3.10-1 thru 3.10-2 are examples of detention outlet configurations.

   Type I Outlet consists of a grated drop inlet, outlet pipe, and an overflow weir in the pond embankment. The control for the 5 year discharge shall be at the throat of the outlet pipe under the head of water as shown on Figure 3.10-1. The grate must be designed to convey the 5-year flow with 50 percent blockage. The difference between the 100-year and the 10-year discharge is released by the overflow weir or spillway.

   Type II Outlet consists of a drop inlet with an orifice controlled inlet for the 5-year discharge and a crest overflow and pipe inlet control for the 100-year discharge. The control for the 5-year discharge occurs at the orifice opening for the head as shown. The control for the 100-year discharge occurs at the throat of the outlet pipe as shown. The difference in the 5-year and 100-year discharges must pass over the weir.

   Type III Outlet is similar to the Type II outlet except the trashrack covers the entire outlet works.

10. When a detention facility uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Failure protection for the embankment may consist of a buried riprap layer on the downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. Structures shall not be permitted in the path of the emergency spillway. The invert of the emergency spillway should be set equal to or above the 100-year water surface elevation.

11. The general form of the weir flow equation for horizontal crested weirs to be used for detention outlet design is:

$$Q = CL(H)^{3/2}$$

Where: $Q$ = discharge (cfs)
$C$ = weir coefficient
$L$ = horizontal length (feet)
$H$ = total energy head (feet)

Or for v-notch weirs:

$$Q = 2.5 \tan (O/2)H^{5/2}$$

Where: $O$ = angle of the notch at the apex (degrees)

Weir flow requirements and coefficients are shown in Table 3.10-1.

12. The equation governing the orifice opening for detention outlet design is:

$$Q = C_d A (2gh)^{1/2}$$

Where: $Q$ = flow (cfs)
$C_d$ = orifice coefficient
$A$ = area ($ft^2$)
$g$ = gravitational constant = 32.2 ft/sec$^2$
$H$ = head on orifice measured from centerline (ft)

An orifice coefficient ($C_d$) value of 0.65 shall be used for sizing of orifice openings and plates.

13. The maximum allowable depth of ponding for parking lot detention is 18 inches for the 100-year flood and 6 inches for the 5-year storm.

14. For parking lot detention, the minimum outlet pipe diameter is 12 inches where a drop inlet is used and 3 inches where a weir and a small diameter outlet are used.

15. All parking lot detention areas shall post a minimum of two signs identifying the detention pond area, warning of periodic flooding, and noting the potential range

of water depth.

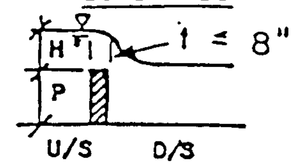
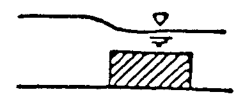
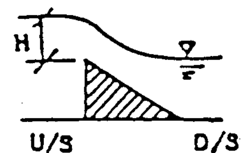
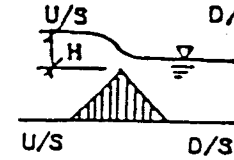
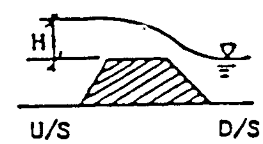
16. Underground detention facilities shall be constructed of corrugated aluminum pipe or reinforced concrete pipe with a minimum pipe diameter of 36 inches. See Figure 3.10-3 for an example underground detention design.

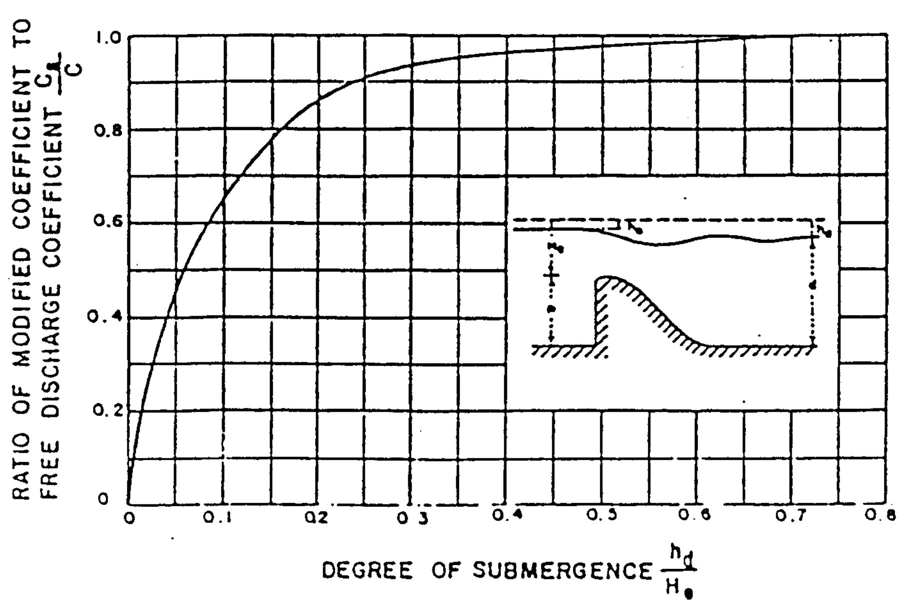
17. For underground detention, the minimum outlet pipe diameter is 12 inches. The outlet shall discharge into a standard manhole or drainageway.

18. Permanent buildings or structures shall not be placed above underground detention facilities.

19. Maintenance access shall be provided for all detention facilities to ensure the detention is performing as designed.

# WEIR FLOW COEFFICIENTS

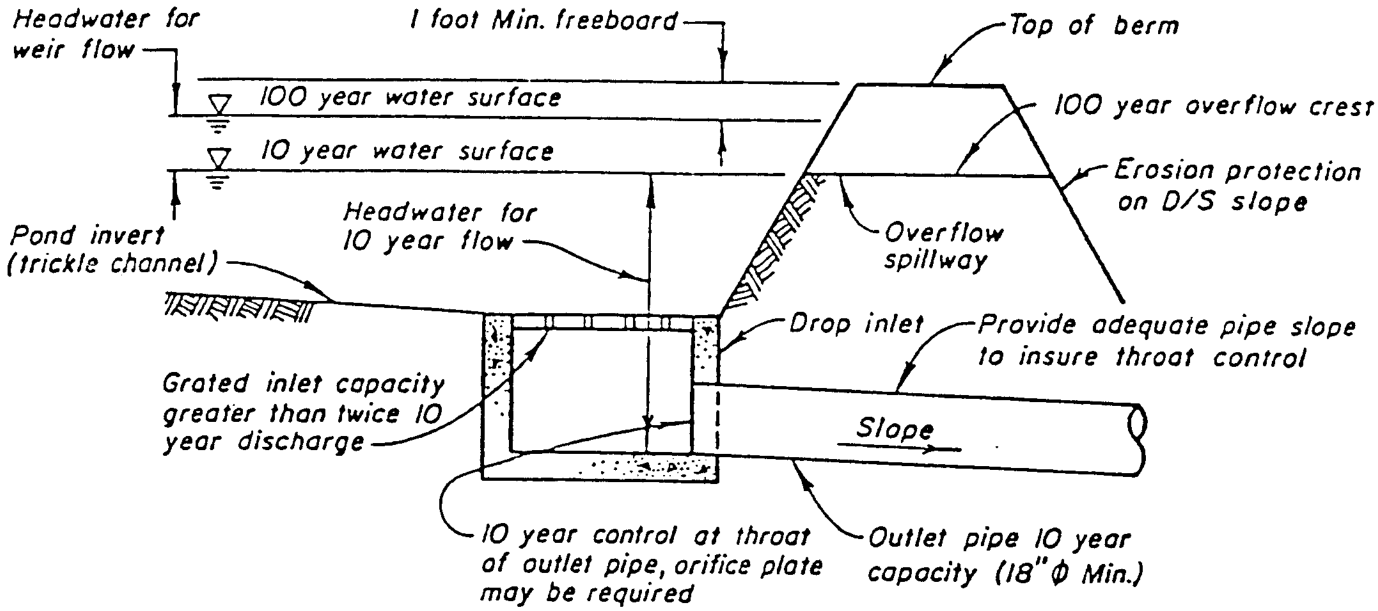
| SHAPE | COEFFICIENT | COMMENTS | SCHEMATIC |
|---|---|---|---|
| Sharp Crested | - | |  |
| Projection Ratio (H/P = 0.4) | 3.4 | H< 1.0 | |
| Projection Ratio (H/P = 2.0) | 4.0 | H> 1.0 | |
| Broad Crested | - | |  |
| W/Sharp U/S Corner | 2.6 | Minimum Value | |
| W/Rounded U/S Corner | 3.1 | Critical Depth | |
| Triangular Section | - | | |
| A) Vertical U/S Slope | - | |  |
| 1:1 D/S Slope | 3.8 | H > 0.7 | |
| 4:1 D/S Slope | 3.2 | H > 0.7 | |
| 10:1 D/S Slope | 2.9 | H > 0.7 | |
| B) 1:1 U/S Slope | - | |  |
| 1:1 D/S Slope | 3.8 | H >0.5 | |
| 3:1 D/S Slope | 3.5 | H >0.5 | |
| Trapezoidal Section | | |  |
| 1:1 U/S Slope, 2:1 D/S Slope | 3.4 | H > 1.0 | |
| 2:1 U/S Slope, 2:1 D/S Slope | 3.4 | H > 1.0 | |
| Road Crossings | | | |
| Gravel | 3.0 | H > 1.0 | |
| Paved | 3.1 | H > 1.0 | |




## ADJUSTMENT FOR TAILWATER

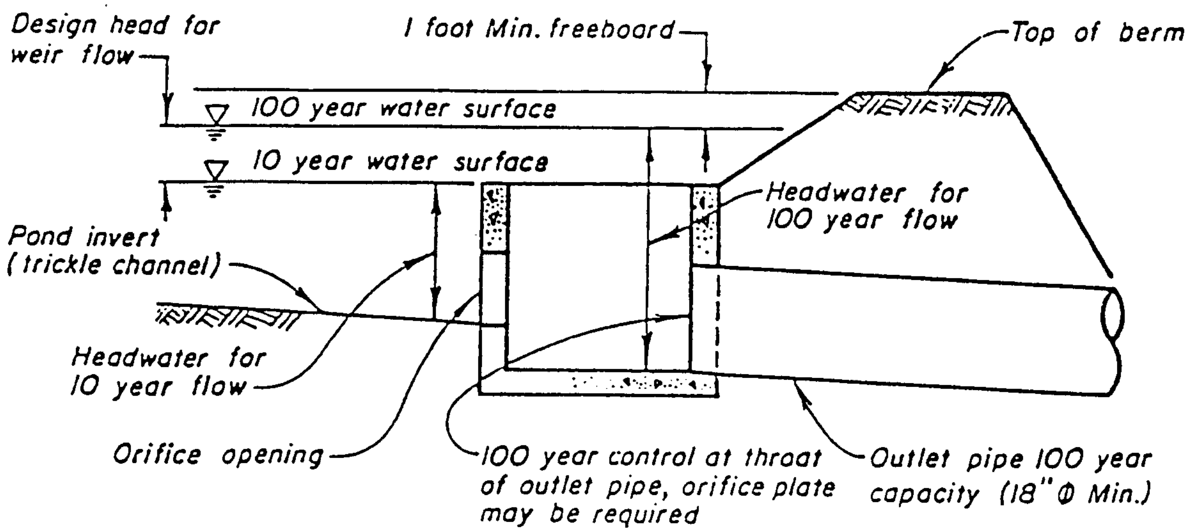
REFERENCE: King & Brater, Handbook of Hydraulics, McGraw Hill Book Company, 1963 - Design of Small Dams, Bureau of Reclam., 1977

# DETENTION POND OUTLET CONFIGURATIONS




TYPE 1 OUTLET

No Scale

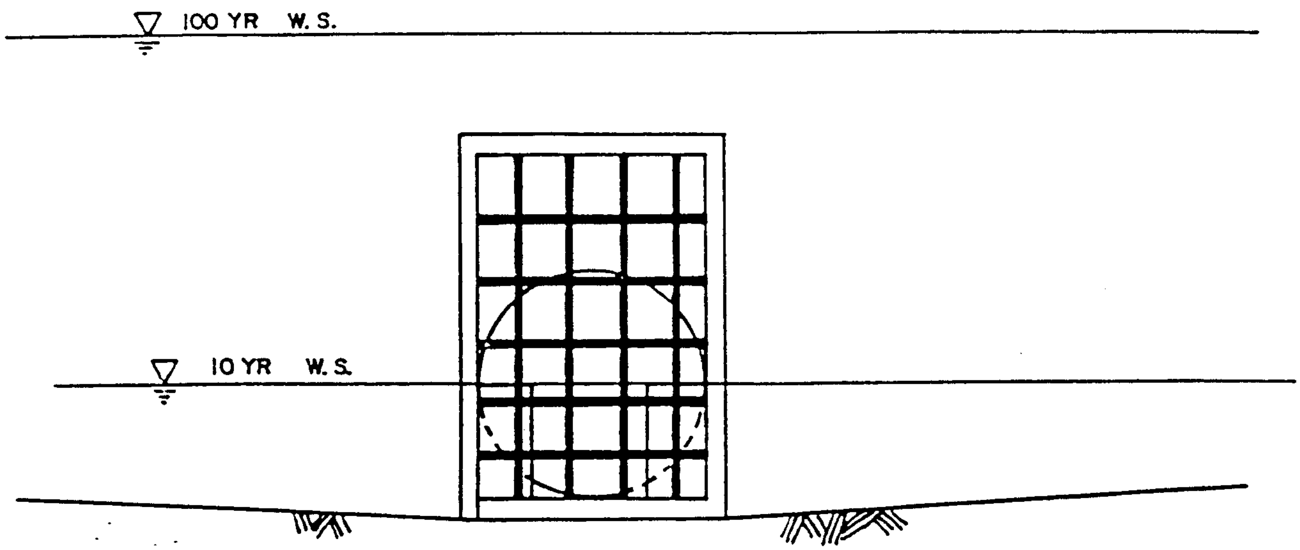



TYPE 2 OUTLET

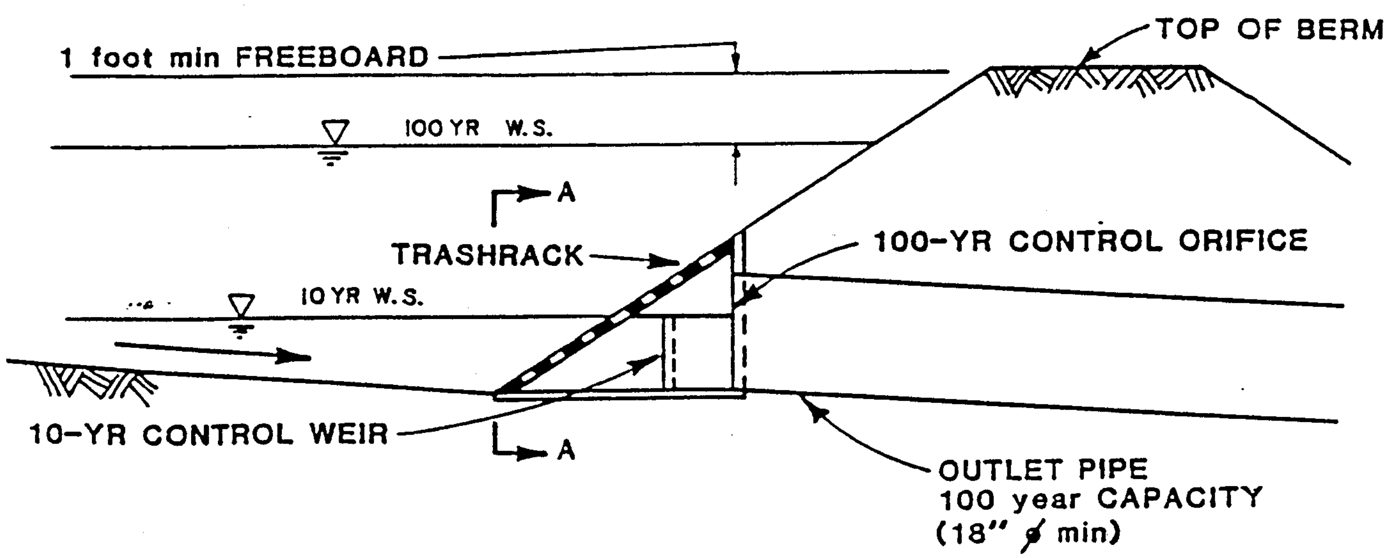
No Scale

REFERENCE: Boulder County Storm Drainage Criteria Manual

# DETENTION POND
# OUTLET CONFIGURATIONS



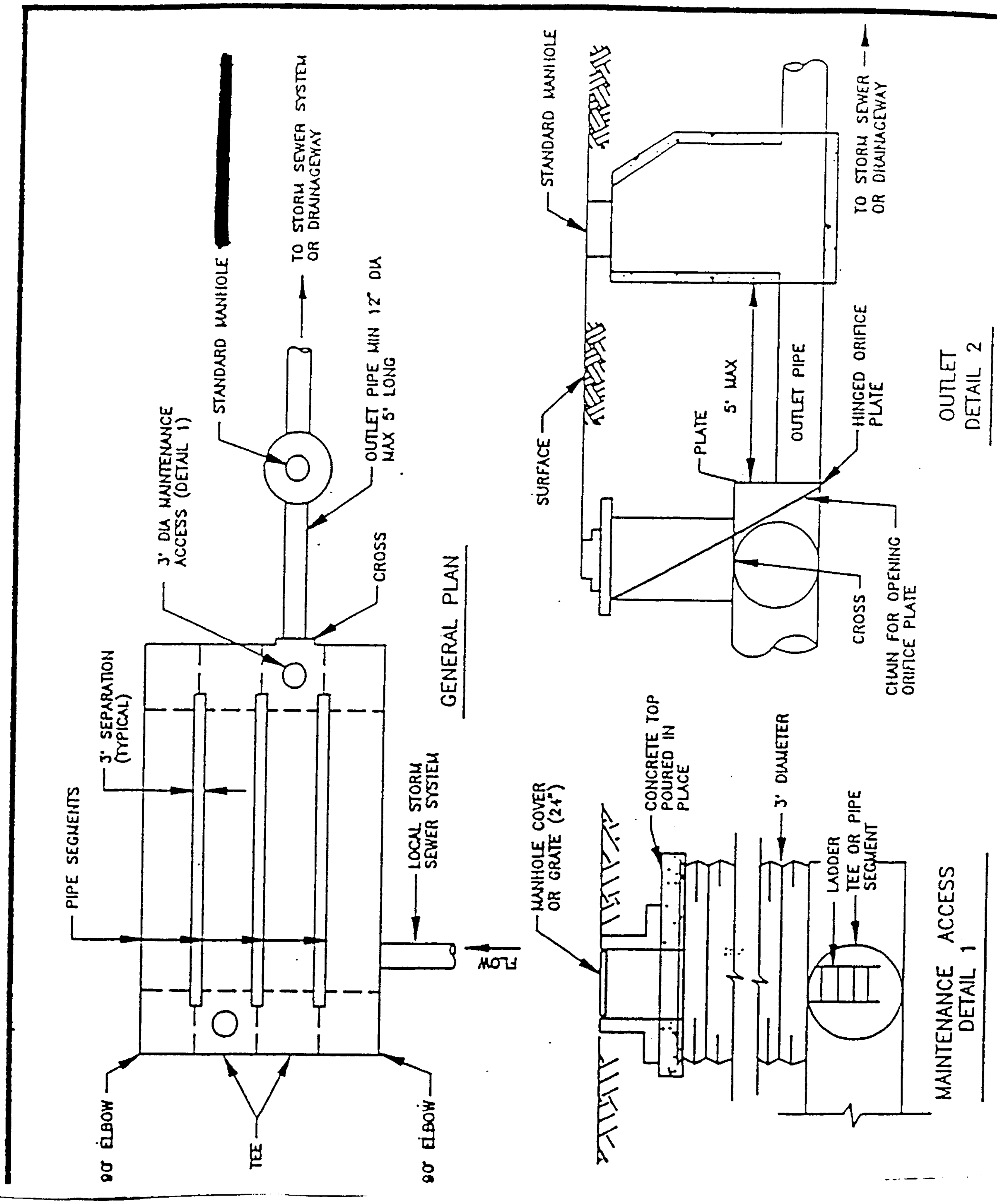

SECTION A-A




TYPE 3 OUTLET
NO SCALE

REFERENCE: Boulder County Storm Drainage Criteria Manual

Figure 3.10-3




UNDERGROUND DETENTION

## 3.11 WATER QUALITY AND EROSION

The City of Evans is committed to protecting and enhancing the environment. Consistent with this policy, the City believes non-degradation of water quality and erosion control are important aspects of all designs and construction. The City will adhere to the information and the design guidelines presented in the USDCM Volume 3 "Best Management Practices" for stormwater quality and erosion control.

# APPENDIX I

17TH AVENUE DETENTION POND & GROUNDWATER COMPLAINTS

A conversation with City of Evans staff indicated that the citizen complaints of groundwater problems were primarily from homeowners in Sundown Estates located east of the 17th Avenue Detention Pond, and from Platte Valley 3rd Filing, located south and east of the pond. One individual in Sundown Estates has had his sump pump operating continuously through the end of October when the water table apparently abated. The groundwater problems appear not to be directly related to precipitation events, although they may intensify during precipitation events. The soils report for Sundown Estates indicates groundwater was at a depth of seven feet on the test date in April of 1993. Since the homeowner with the most groundwater problems experienced a reduction in the amount of pumping which coincides with the end of the irrigation season, it is possible that the groundwater elevations may be more related to the irrigation season than to the 17th Avenue Pond.

In the RMC review of the 17th Avenue Detention Pond for previous studies (see Appendix 3 and 4 of the City of Evans Master Drainage Plan), it was determined that the capacity of the 17th Avenue Pond was adequate to allow drainage from Chappelow Village to flow into the detention pond in addition to the drainage for which it was originally designed. An under drain from Chappelow Village drains groundwater and appears to run continuously into the concrete channel of the pond. This indicates that there is constant groundwater in this area.

Although there is a concrete trickle channel through the detention pond, some site drainage water "ponds" rather than draining through the pond to the outlet. This water could be contributing to the groundwater elevation. Typically the 17th Avenue Detention Pond does not have water stored in it.

The 17th avenue Detention Pond reports noted above recommended that the outlet pipe be modified to limit the amount of water released from the pond in a storm event so that the water would not surge through the grated manhole onto the street. This modification would also avoid manual operating problems that may fill the pond and cause recharge of the groundwater.

Conversation with Evans City staff members indicated that lots in very close proximity to each other can exhibit extremely different groundwater characteristics. For example, one lot could have a shallow groundwater table requiring raising the floor elevation of the building and installing perimeter drains and a sump pump, while the lot next door would have a dry hole at the same depth. Another example is the Mini Mart at the northeast corner of 37th Street and 23rd Avenue. Gas tanks were buried at a depth of approximately 15 feet at the north end of the site, but 40 feet south of the gas tanks, groundwater was encountered at depths of 10 to 12 feet.

From the information gathered, it appears that lawn irrigation, precipitation, and water detained in the 17th Avenue detention pond could all be contributing to the water accumulating in adjacent basements. The bottom of the detention pond should be modified so that the groundwater draining from Chappelow Village stays in the trickle channel and is conveyed to the storm sewer and does not continuously saturate the pond bottom. Precipitation events which cause the detention pond to accumulate water as well as irrigation by residents will likely recharge the groundwater table.

The cost for lining the 17th Avenue detention pond with a slurry wall and with a polyvinyl liner material are estimated as follows:

A. Slurry wall lining - In estimating the costs of a slurry wall lining, the following assumptions were made:

Area of the pond approximately 550 x 550 = perimeter of 2200 lineal feet.
Depth to bedrock approximately 19 feet (Nearby drilling reports indicated depths ranged from 5 to 25 feet).
Southeast portion of the pond built up to form a dam, approximately 8 feet.
Slurry wall constructed 3 feet into bedrock.

Total for engineering and construction $128,700

B. Plastic liner

1. PVC - 20 mil @$.28/sq. ft. installed (laying and seams only, no dirt work) approximately 262,000 sq. ft. = $73,360
Preparation of sub-grade, bed and cover @$9.50/sq. yd. for approximately 29,000 sq. yds. = $275,000

Total $348,360

2. HDPE - 40 mil @$.45/sq. ft. installed for approximately 262,000 sq. ft. = $117,900
No cover required for UV protection, but is recommended for appearance, safety of people, and protection of liner 29,000 sq. yds. @ $950/sq. yd. = $275,000

Total $392,900

C. Bentonite Mat Liner - (Geosynthetic clay liner) - bentonite between fabric cover - $.40/sq. ft. installed for approximately 262,000 sq. ft. = $104,800
Less subsurface preparation needed, but still required rough subgrade and one foot cover, 29,000 sq. yds. @$5.00/sq. yd.= $245,000

Total $249,800

All the costs estimated above seem excessive relative to the potential for surcharging the groundwater that might be attributed to the detention pond. Before considering lining the pond, our recommendations would be:

1. Reconfigure the bottom of the pond so that flows other than storm water can enter and exit without being detained and seeping into the subsurface groundwater.

2. Develop a program to monitor the detention pond when storm flows are detained relative to the necessity for sump pumping in the adjacent subdivision. If visual monitoring is not conclusive, installation of monitoring wells may be necessary.

h\1958.044\ddrainage\17avedet.rpt

# APPENDIX II

BACKGROUND INFORMATION FOR POLICY FORMULATION

## 1. Evans Town Ditch

### a. Water Rights Considerations

Evans Town Ditch water rights issues are covered in detail in the Water/Wastewater Master Plan prepared in 1996 by HDR Engineering, Inc., thus will not be reiterated here. It is suggested that the recommendations presented therein be followed in order to preserve and optimize the value of the Evans Town Ditch to the City of Evans.

### b. Drainage Considerations

The Evans Town Ditch is currently used to receive historic drainage flows within Evans City limits and from a portion of the Urban Growth Area. In the operation of the Evans Town Ditch, the maximum available to the decree has been diverted through the ditch in order to preserve the decree. A preliminary report was prepared in 1985 by Western Technical Services which evaluated the ability of the Evans Town Ditch to carry storm flows in addition to its decreed right for 30 cfs. The report indicated that the ditch could carry the 10-year storm with the exception of a few culverts, but the drainage from a 100-year storm exceeded the capacity of several structures, and even the ditch capacity at one location.

With respect to water quality, co-mingling direct flow irrigation water with storm drainage flows which originate in developed areas will likely have a deleterious effect on the quality of Evans Town Ditch water. RMC's previous experience with similar circumstances has indicated that total coliforms, total dissolved solids, and other pollutants such as oil, grease, and heavy metals can increase when storm drainage from city streets is discharged into an irrigation ditch.

For these reasons, it is recommended that drainage into the Evans Town Ditch be restricted to historic rates, and wherever possible, storm water bypasses the ditch and is discharged into the South Platte River floodplain.

### c. Costs Attributable to Drainage

An updated analysis comparing the capacity of the Evans Town Ditch and its structures with the flows contributed by the direct flow decree compared with the flows contributed by storm drainage inflows would result in a cost splitting calculation. The Water/Wastewater Master Plan (HDR Engineering, Inc., 1996) estimated the annual costs associated with the Evans Town Ditch ranged from about $41,000 to $52,000 per year compared with an annual income of about $6,000 per year. If the deficit of $35,000 to $46,000 were made up by storm drainage fees, the fees would be $2.00 per month per tap (using the City's current 1924 taps).

The value of the Evans Town Ditch water being used by citizens for irrigation should be taken into consideration. For example, if 50 acres (individual yards, trailer parks, etc.,) are being irrigated by Evans Town Ditch, the demand for treated water is reduced by an estimated 100 acre-feet per year. If Evans had to purchase this raw water to turn over to Greeley for treatment, the cost at current market value of Colorado Big Thompson water would be about $391,000 (100 acre-feet + 15% shrink = 115 acre-feet x $3400 per acre-foot, assuming price of CBT at $2400 per unit).

**2. Development Fee and Monthly Fee Policy Considerations**

In order to provide guidance for the City Council in adopting a policy regarding development fees and/or monthly fees, some options were prepared for consideration. In evaluating the storm drainage costs for the options, growth over the next 20 years was estimated to extend to 35th Avenue, and would include the 23rd Avenue Basin and UGA East Basin (see Figure 1). The growth rate was based on Evans' Comprehensive Plan at a rate of 3.2%.

Option 1 assumes that the costs for drainage improvements are borne by development fees only, and that each basin must pay for the improvements required in that basin. Option 2 assumes that the costs for drainage improvements are borne by development fees only, and that the costs are averaged over both 23rd Avenue and UGA East Basins. Option 3 assumes that both development fees and monthly fees (ranging from $2.50 to $3.50 per month) are charged, and improvement costs include the four most important improvements within the existing city plus the improvements for the 23rd Avenue Basin and UGA East Basin. Option 4 includes the costs for 23rd Avenue, UGA East, and the top two improvements in the city to be paid for by development fees and monthly fees. Options 5 and 6 include the cost of the 23rd Avenue Drainage Basin improvements plus the top two improvements within the city. The difference between Options 5 and 6 is that Option 6 charges different development rates for residential areas versus commercial and industrial areas, which creates income slightly greater than anticipated expenditures. It should be noted that Options 4 and 5 do not include the cost of improvements for UGA East, which is quite expensive because of the lack of natural drainage conveyances in this basin. The options are intended to show the potential income compared with costs for improvements, and any combination of improvements can be made. For example, the income from development fees and monthly fees could be used to make some of the improvements needed in the City, a portion of those needed in the 23rd Avenue Basin, and possibly some improvements in the UGA East Basin. The pattern of growth will dictate where the money needs to be spent.

Table 1 summarizes the advantages and disadvantages of each of the options. Table 2 shows the costs of improvements versus the income generated by the various combinations of sources. Table 2 also notes the assumptions made in preparing these calculations. Table 3 shows the four improvements within the City ranked by importance. Table 4 shows a comparison of fees charged by surrounding municipalities, including Greeley, Longmont, Loveland and Ft.

Collins. Table 5 shows the amount of money that would be generated by applying Evans current fee structure for storm drainage (development fees only) and by combining development fees with monthly fees. As shown in Table 5, current development fees combined with monthly fees in the range of $2.50 to $3.50 would generate enough money to set aside $10,500 per year for operations and maintenance plus about $2.7 million to $3.3 million to pay for 23rd Avenue improvements and the top two improvements within the City, or any combination of improvements needed.

H\1958_044\DRAINAGE\VOL2APP2.RPT

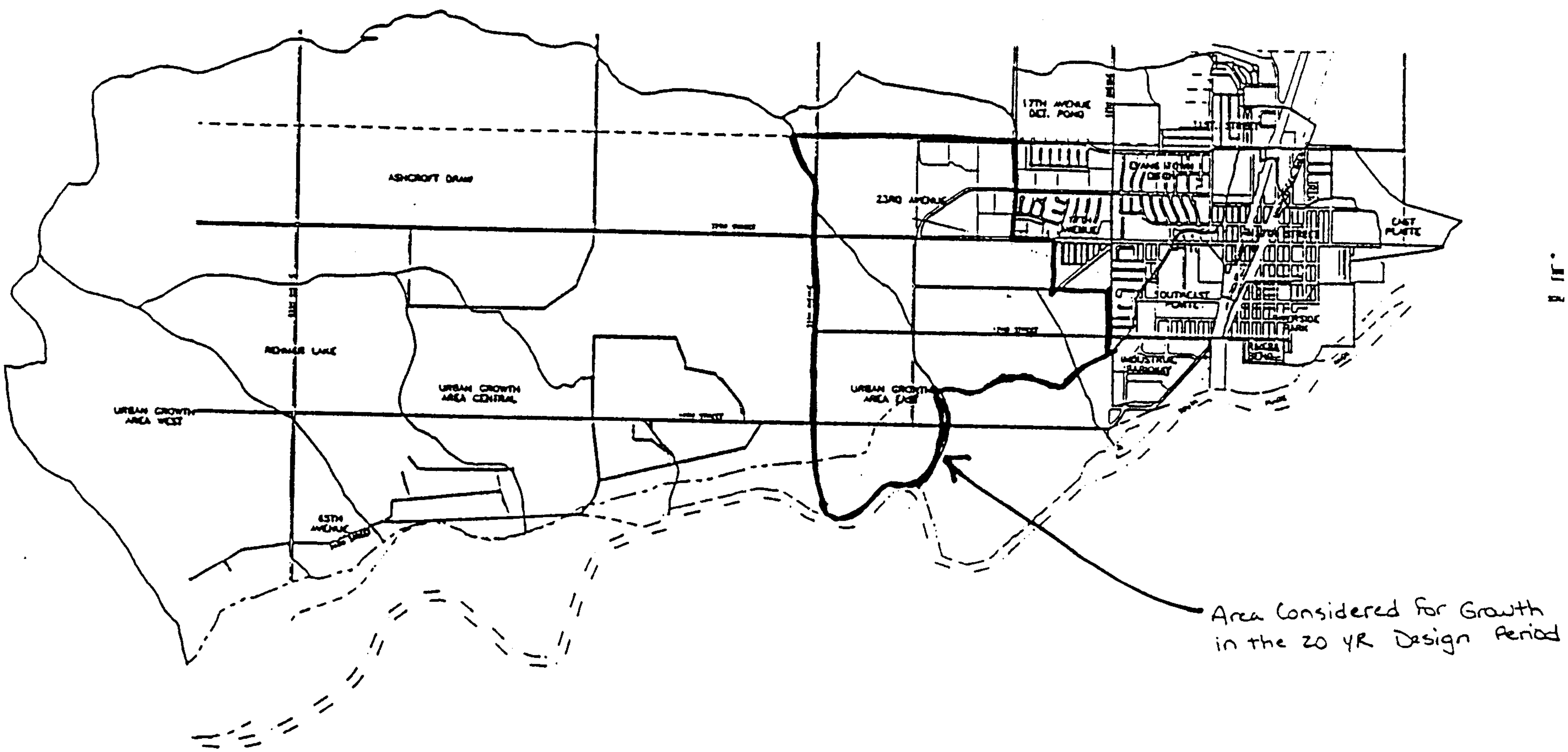

Area Considered for Growth
in the 20 YR Design Period
ASHCROFT DRAW
17TH AVENUE DET. POND
23RD AVENUE
EAST PLATTE
SOUTHEAST POINTE
URBAN GROWTH AREA CENTRAL
URBAN GROWTH AREA WEST
URBAN GROWTH AREA EAST
65TH AVENUE

FIGURE 1.

## TABLE 1. ADVANTAGES AND DISADVANTAGES OF VARIOUS STORM DRAINAGE FEE COLLECTION OPTIONS

**Option 1 - Each Basin Pays for Itself as it Grows (Development Fees Only)**

The developer is required to pay the amount per acre needed to construct the improvements required in that particular basin.

Advantages:
- No monthly fee to the citizens
- Over a long period of time, it creates a "pool" of money for improvments

Disadvantages:
- Improvements will not be paid for until the basin is completely developed
- Previously developed areas are not contributing to the storm drainage costs
- Some basins have much higher development costs than others
- No funding for basins that are already developed (ie: within the City)
- May discourage development, especially in the more expensive basins

**Option 2 - Average all the Growth Area (Development Fees Only)**

Calculate an average cost for all basins in the growth area. Developer pays the amount per acre required for improving all the drainage basins. Developers in basins with low drainage improvement costs will subsidize those in basins with high drainage improvement costs. The growth area over the next 20 years is considered to be the 23rd Ave. and the Urban Growth Area (UGA) East Basins only.

Advantages:
- No monthly fee to the citizens
- Over a long period of time, it creates a "pool" of money for improvments
- All developing basins have the same development costs

Disadvantages:
- Early improvements cannot be paid for up front
- Previously developed areas are not contributing to the storm drainage costs
- No funding for basins that are already developed (ie: within the City)
- May discourage development

**Options 3,4, 5& 6 - Development Fees & Monthly Drainage Fees to Incorporated Areas**

All improvement costs are averaged over total growth area as well as the existing City. A minimal monthly fee is charged to the citizens. The remainder of the costs are received through development fees.

Advantages:
- Provides a steady income for Oper. & Maint. as well as Capital Improvements
- Lower development costs
- Funding for all basins, even those already developed
- All citizens and developments are contributing

Disadvantages:
- Monthly fee to the citizens
- Does not include areas outside of the City of Evans' jurisdiction

H:\1958_044\DRAINAGE\POLICYOP.WPD

**TABLE 2. Development & Storm Drainage Fee Options**

| | Option 1 | | Option 2 | Option 3 | | | Option 4 | | | Option 5 | | | *Option 6 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 23rd Ave. | UGA East | | (1) | (2) | (3) | (1) | (2) | (3) | (1) | (2) | (3) | (1) | (2) | (3) |
| Development Fees (per acre | | | | | | | | | | | | | | | |
| Residential | $3,106 | $20,236 | $6,525 | $7,641 | $7,311 | $6,981 | $5,710 | $5,380 | $5,050 | $1,427 | $1,097 | $767 | $1,000 | $1,000 | $1,000 |
| Commercial | $3,106 | $20,236 | $6,525 | $7,641 | $7,311 | $6,981 | $5,710 | $5,380 | $5,050 | $1,427 | $1,097 | $767 | $2,500 | $2,500 | $2,500 |
| Industrial | $3,106 | $20,236 | $6,525 | $7,641 | $7,311 | $6,981 | $5,710 | $5,380 | $5,050 | $1,427 | $1,097 | $767 | $2,500 | $2,500 | $2,500 |
| Monthly Fees | | | | | | | | | | | | | | | |
| Residential | – | | – | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 |
| Commercial | – | | – | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 |
| Industrial | – | | – | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 | $2.50 | $3.00 | $3.50 |
| Projected Income Over 20 Years | $2,150,000 | $4,107,902 | $6,257,902 | $8,701,142 | | | $6,849,482 | | | $2,741,580 | | | $2,836,125 | $3,152,750 | $3,469,375 |

**Various Storm Drainage System Improvement Costs**

Option 1 - Development fees only. Each basin pays for itself.
- 23rd Ave. Storm Drainage System = $2,150,000
- UGA East Storm Drainage System = $4,107,902

Option 2 - Development fees only.
- 23rd Ave. and UGA East basins combined = $6,257,902

Option 3 - Development fees + varying monthly fees.
- 23rd Ave., UGA East & Top 4 Improvements w/in City = $8,701,142

Option 4 - Development fees + varying monthly fees.
- 23rd Ave., UGA East & Top 2 Improvements w/in City = $6,849,482

Option 5 - Development fees + varying monthly fees
- 23rd Ave. & Top 2 Improvements w/in City = $2,741,580

Option 6 - Varying development fees + varying monthly fees.
- 23rd Ave. & Top 2 Improvements w/in City = $2,741,580 + cushion

* RMC recommends Option 6 using the example numbers given or some variance of them.

Notes:

Amounts based on a 3.2% growth rate over a 20 year design period. Evans currently serves 1924 customers; which excludes Arrowhead and Hill-n-Park.
Area to be developed and types of development based on the City of Evans 1995 Comprehensive Plan.
Operations, Maintenance, & Overhead costs are estimated at $10,500 per year. This amount is deducted from the Projected Income amounts shown in the table.
The total area for development was reduced by 30% to account for roads, drainageways, and other additions that will not pay drainage fees.
Storm drainage system improvement costs based on the Evans Master Drainage Plan. These numbers are estimates only.

The options shown are to give you an idea of the type of funding needed and what it may take to get that funding. In actuality, some variance to these fees and charges may be used. Also, the funding does not have to be restricted to the exact projects noted. For example, partial improvements may be made in 23rd Ave. basin and partial in the UGA East basin to meet immediate drainage needs rather than funding only the 23rd Ave. improvements as given in Options 4 & 5.

H\1958_044\DRAINAGE\POLICYOP WB2

TABLE 3. PRIORITY OF DRAINAGE SYSTEM IMPROVEMENTS WITHIN THE CITY OF EVANS

| PRIORITY | LOCATION | DESCRIPTION | TOTAL COST | TOTAL PER ACRE COST | IMPORTANCE OF IMPROVEMENT |
|---|---|---|---|---|---|
| 1 | 37th St. & Hwy. 85 Interseciton | Install 2 detention ponds -<br>11.6 acre-ft on west side of Hwy.85<br>7.3 acre-ft on east side of Hwy. 85 | $235,875 | $874 | Alleviate flooding of the intersection which will hinder traffic flow on Highway 85.<br>Major inconvenience due to localized flooding.<br>No major threat to life or property exists under current conditions.<br>Fully developed. |
| 2 | SE corner of 11th Ave. and 31st St | Regional detention pond | $355,705 | $1,358 | Alleviate flooding of the 31st St. and Highway 85 intersection.<br>Eliminate 31st St. drainage system improveme<br>Reduce stormwater discharge into ETD.<br>Reduce ETD overflows therefore reducing the problems in the Southeast Platte basin.<br>No major threat to life or property exist under current conditions.<br>Fully developed. |
| 3 | Southeast Platte Basin Improvements | 78" diameter pipe | $1,577,070 | $12,225 | Alleviate flooding of Hwy. 85 west frontage roac and part of Hwy. 85.<br>Predominantly undeveloped. Flooding will becc a problem as the area develops.<br>No major threat to life. Potential threat to prope |
| 4 | Riverside Park Basin Improvements | 2-18", 24", 30" and 36" diameter pipe | $274,590 | $2,162 | Alleviate localized flooding of streets and lawns<br>Developed area with unpaved roads.<br>Implement drainage system along with the road improvements.<br>No major threat to life. Potential threat to privat property. |

H\1958_044\DRAINAGE\PRIORITY.WB2

## TABLE 4. LOCAL STORMWATER DRAINAGE FEE COLLECTION

| | Evans | Greeley | Longmont | Loveland | Ft. Collins |
|---|---|---|---|---|---|
| **Monthly Fees** | | | | | |
| Residential | 0 | 0 | $3.25 | $1.42-3.55 | $6-10 |
| Commercial | 0 | 0 | $3.25/20,000SF | $24.60 | ? |
| Industrial | 0 | 0 | $3.25/20,000SF | $23.25 | ? |
| Institutional | 0 | 0 | $3.25/20,000SF | $10.95 | ? |
| **Development Fees** | | | | | |
| Residential | | | | | |
| Area up to 10,000SF | $.045/SF | $300 | $10 / Bldg. | $220-1040/Acre | $3000-10,000/Acre |
| Area > 10,000SF | $.0225/SF | $300+$.04/SF> 10,000 | $10 / Bldg. | $220-1040/Acre | $3000-10,000/Acre |
| Commercial | Same | $300+$.04/SF> 10,000 | $10 / Bldg. | $2040 / Acre | $3000-10,000/Acre |
| Industrial | Same | $300+$.04/SF> 10,000 | $10 / Bldg. | $1920 / Acre | $3000-10,000/Acre |
| Institutional | Same | $300+$.04/SF> 10,000 | $10 / Bldg. | $900 / Acre | $3000-10,000/Acre |
| **Additional Notes:** | Based on lot size. Paid w/ bldg. permit. | Based on lot size. Paid w/ bldg. permit. Detention required. | Monthly based on lot area. Detention required. | Res. based on density. Credit for detention. | Fees based on drainag basin. Avg. dev. fee is $3000. Credit for detention. |

h\1958_044\drainage\drainfee.wb2

TABLE 5. PROJECTED 20 YEAR INCOME
USING THE CURRENT DEVELOPMENT CHARGES

| | Evans Current Policy | | Evans Current Policy Combined w/ Monthly Fees | | |
|---|---|---|---|---|---|
| | (per acre) | (per lot) | | | |
| Development Fees * | | | | | |
| Residential | $1,372 | $343 | | $1,372 | |
| Commercial | $1,372 | $343 | | $1,372 | |
| Industrial | $1,372 | $343 | | $1,372 | |
| Monthly Fees | | | | | |
| Residential | | | $2.50 | $3.00 | $3.50 |
| Commercial | | | $2.50 | $3.00 | $3.50 |
| Industrial | | | $2.50 | $3.00 | $3.50 |
| Projected Income Over 20 Years | $1,315,748 | | $2,688,873 | $3,005,498 | $3,322,123 |

* Fees were based on the assumption that 70% of the acreage is developed into lots, and lot sizes are less than 10,000 SF.
The per lot cost is based on the average 4 lots/acre.

h\1958_044\drainage\drainfe2.wb2