THE

TOWN OF KERSEY STORM DRAINAGE DESIGN CRITERIA AND CONSTRUCTION SPECIFICATIONS

December 2004

Prepared for:
TOWN OF KERSEY
332 3rd Street
Kersey, CO 80644

Prepared by:

RG CONSULTING ENGINEERS, INC.
1331 17th Street, Suite 710
Denver, Colorado 80202
(303) 293-8107

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SECTION 1.0: GENERAL

1.1 PURPOSE

The purpose of the "Town of Kersey Storm Drainage Design Criteria" (hereafter referred to as the "Criteria") is to present the minimum design and technical criteria for the analysis and design of storm drainage facilities.

All subdivisions, residential, commercial, industrial development or any other proposed construction submitted for approval under the Town of Kersey regulations shall include adequate storm drainage system analysis and drainage system design. Such analysis and design shall conform to the criteria set forth herein. Options or alternatives to the provisions of these Criteria may be suggested by the applicant, and used only on the written approval of the Town. The applicant must demonstrate through adequate information and technical documentation that such options are equal to or better than the requirements of the Criteria.

Policies and technical criteria not specifically addressed in this document shall follow the provisions of the most recent edition and revisions of the Urban Drainage and Flood Control District (hereafter called the "District") "Urban Storm Drainage Criteria Manual" (hereafter abbreviated to USDCM), which is incorporated in these Criteria by reference. Copies of the USDCM can be obtained from the District, 2480 West 26th Avenue, Suite 156B, Denver, Colorado 80211-5500.

The applicant is also referred to the Colorado Department of Transportation's Standard Plans ("M-Standards") for additional design details not covered in these Criteria or the USDCM.

The Criteria shall apply to all land within the incorporated area of the Town of Kersey, and those areas outside the Town of Kersey covered by intergovernmental agreements with the appropriate jurisdiction, including any public lands. These Criteria shall apply to all facilities constructed on Town Right-of-Way; easements dedicated for public use, private roads, and to all privately and quasi-publicly owned and maintained stormwater detention facilities.

1.2 PRINCIPLES FOR STORM DRAINAGE PLANNING & DESIGN

The provision for adequate drainage in urban areas is necessary to preserve and promote the general health, welfare, and economic well being of the region. The Town of Kersey recognizes drainage as a sub-system of all development; and, as such, the planning and design of drainage facilities must be included in the development process.

Planning and design of stormwater drainage systems should not be based on the premise that problems can be transferred from one location to another. Colorado drainage law recognizes the inequitability of transferring the burden of managing storm drainage from one location or property to another. Liability questions also arise when historic drainage patterns are altered. The diversion of stormwater drainage from one basin to another should be avoided unless specific and prudent reasons justify and dictate such a transfer.

The subdivision process can significantly alter the historic or natural drainage paths. When these alterations result in a subdivision outfall system that discharges back into the natural drainage way at or near the historic location, then the alterations (interbasin transfer) are generally acceptable. However, when the subdivision outfall system does not return to the historic drainage way, then interbasin transfer may result. This interbasin transfer should be prevented since it violates a basic drainage law principle by discharging water into a subservient property in a manner or quantity to do more harm than formerly. If the development significantly increases the drainage area tributary to the subdivision outfall, then interbasin transfer into the property has occurred, which also must be prevented.

In addition to planning for the control of stormwater runoff flows, consideration for maintaining the quality of the urban stormwater runoff resource should be included in the evaluation and design of drainage facilities. Drainage facilities can fulfill other purposes in conjunction with primary stormwater considerations; recreational, water quality and open space values should be integrated where possible. Likewise, facilities not designed primarily for drainage, such as parks, open space areas, or other natural resource areas, can frequently be designed to utilize or enhance some aspects of the stormwater runoff resource or provide some drainage control benefits.

The Town requires on-site detention for all new development, expansion, and redevelopment for retention (see Section 11.3.3 of this manual).

Developments are typically responsible for runoff to the centerlines of all adjacent streets surrounding their site. This runoff must be routed to on-site detention facilities and released at a 5-year historic rate as required for other on-site runoff.

On-site detention facilities are not required for attenuation of off-site flows. However, the urbanization process must safely pass all off-site flows (including all irrigation and stormwater flows) through the development without creating any adverse impact to other upstream or downstream properties.

Detention facilities shall not be constructed within public rights-of-way. The design high water level of detention ponds shall not encroach upon public rights-of-way.

Adequate drainage easements must be provided for all detention and stormwater conveyance facilities including proper access for operation and maintenance (see various applicable sections of this criteria manual for more details).

The Town requires that maintenance access be provided to all storm drainage facilities to assure continuous operational capability of the system. The property owner shall be responsible for the maintenance of all privately owned drainage facilities including inlets, pipes, culverts, channels, ditches, hydraulic structures, and detention basins located on their land unless modified by the Developer's Agreement. Should the property owner fail to adequately maintain said facilities, the Town shall have the right to enter said land for the purposes of operations and maintenance. All such maintenance costs shall be assessed to the property owner.

Drainage easements shall be shown on the preliminary and final plats and any development plans and state that the Town has the right of access on the easements which shall be kept clear of obstructions to the flow and/or obstructions to maintenance access.

On-site erosion/sedimentation control programs are required for all development and redevelopment (see Section 13.0 Construction Site Erosion and Sediment control of this criteria manual).

The urbanization process may adversely impact downstream properties due to changes in the historic frequency and quantity of stormwater discharge. Even though on-site detention facilities are provided which reduces the historic peak runoff from development, the increase in impervious area will result in more frequent release of stormwater.

Drainage easements are sometimes provided within subdivisions along back property lines intended to drain runoff to street right-of-way. These easements, however, are often ineffective due to improper grading of lots or restrictions created by wood privacy fences or other types of obstructions. As a result, these easements shall not be relied upon for drainage of backyard areas.

Drainage easements through the back or sides of lots within subdivisions shall not be designated to convey off-site flows through the development. Right-of-ways dedicated to the Town must be provided when necessary to convey these off-site flows.

Sometimes drainage easements for on-site surface water flows must be designated within lots of subdivisions, when no other viable alternatives exist. When drainage easements of this type are necessary, Developers, Home Builders/Owner's must accept the responsibility to provide site grading in a manner consistent with development plans regarding site drainage. Drainage easements shall not be restricted. On-site and pass-through runoff shall be routed to streets, along property lines, and through easements in a manner, which controls surface runoff. To accommodate runoff, "V" shaped swales may need to be constructed at least one (1) foot deep and six (6) feet wide.

It is usually better to line swales with a weed barrier fabric and crushed rock as opposed to a grass swale. Grass swales sometimes fill in and become non-existent over time.

Drainage easements and drainage swales should not be blocked with wood privacy fences. A space should be left under the fence for water to pass. Steel rebar may be pounded into the ground at three (3) inch to four (4) inch spacing to keep dogs and small children from crawling under the fence.

1.3 Irrigation Facilities

There may be irrigation ditches in the Town area. The ditches have historically intercepted the storm runoff from the rural and agricultural type basins, generally without major problems. However, with urbanization of the basins, the storm runoff has increased in rate, quantity and frequency, and even the water quality has changed.

In evaluating the interaction of irrigation ditches with a major drainage way for the purpose of basin delineation, the ditch should not be used as a basin boundary. Irrigation ditches are designed with flat slopes and limited carrying capacity, which decreases in the downstream direction. As a general rule, irrigation ditches cannot be used as an outfall point for the storm drainage system because of these physical limitations. In addition, certain ditches are abandoned after urbanization and, therefore, could not be considered a permanent part of the storm drainage system. Due to these changes in the urban stormwater response, irrigation facilities should not be considered as part of the available drainage system for new development.

Irrigation facilities must be preserved through new development areas in order to maintain service to any upstream or downstream users of the ditch. Adequate easements must be provided for the ditch including access for operations and maintenance. Irrigation lines passing under roadways must be approved pipe materials by the Town Engineer.

1.4 Relationship to Other Standards

Whenever a provision of these Criteria, and any other provision in any law, ordinance, resolution, rule, or regulation of any kind, contains any restrictions covering any of the same subject matter, the most restrictive standard shall apply.

These Criteria are consistent with the Urban Drainage and Flood Control District's criteria. If the state or federal government imposes stricter criteria, standards, or requirements, these shall be incorporated into the Town's requirements after the appropriate due process needed to modify the Town's regulations and standards.

Adherence to these Criteria does not remove the applicant's responsibility to investigate and obtain any other regulatory permits or approvals, from local, regional, state or federal agencies, that may be required for a particular project.

1.5 VARIANCES

It is the responsibility of the Owner, or Owner's selected Design Engineer, to request any variances from Town Standards during the early stages of planning or design development. Variances from these Criteria shall be considered on a case-by-case basis.

1.6 REVIEW AND ACCEPTANCE

The Town Engineer shall review all drainage submittals for general compliance with these specific Criteria. An acceptance by the Town does not relieve the Owner, Engineer, or Designer from the responsibility of ensuring that the calculations, plans, specifications, construction, and as-built drawings are in compliance with the Criteria.

Approval of the submittal information shall remain valid for one year after the acceptance date. If construction of the project has not been initiated within that period, the acceptance by the Town shall become invalid.

SECTION 2.0: SUBMITTAL REQUIREMENTS

2.1 REVIEW PROCESS

Drainage reports and plans, construction drawings, specifications, and as-built information shall be submitted and approved as required by the Town of Kersey Subdivision Regulations and the Project Development Process. A pre-application consultation is suggested of all applicants for all processing steps of the Subdivision Regulations. The applicant should consult with the Town Engineer for general information regarding subdivision regulations, the Project Development Process, required procedures, possible drainage problems, stormwater requirements and issues associated with the pertinent master drainage plan, and specific submittal requirements.

All topographic mapping is to be based on NGVD 29 survey datum. The Town Of Kersey will not accept any other datum nor will an adjustment from some other datum to NGVD 29 be acceptable. All contour mapping and construction details must be on NGVD 29 datum.

All reports shall be typewritten on 8½" x 11" paper and bound. A cover letter shall be included, identifying the project and the type of information submitted (conceptual, preliminary, or final). The report shall be prepared (or supervised), signed and stamped by a Professional Engineer licensed to practice in the State of Colorado, and possessing adequate experience in the fields of hydrology and hydraulics. The report shall contain a certification sheet with the following statement, and appropriate signatures:

"I hereby attest that this report for the (Conceptual, Preliminary or Final) drainage design of (Name of Development) was prepared by me, or under my direct supervision, in accordance with the provisions of the Town of Kersey Storm Drainage Design Criteria for the responsible parties thereof. I understand that the Town of Kersey does not and shall not assume liability for drainage facilities designed by others.

____
Registered Professional Engineer

State of Colorado No._______________
(Affix Seal)

For project areas less than five acres, the applicant may propose that the conceptual, preliminary, and/or final reports be consolidated; written approval for this consolidation is required from the Town prior to any submittal.

The drawings, figures, plates, and tables shall be bound within the report or included in a pocket attached to the report. Photo static copies of charts, tables, nomographs, calculations, or any other referenced material should be legible and contain the origin of the reference. Washed out, blurred, or unreadable portions of the report are unacceptable, as is incomplete or absent information. The information presented in technical appendices should be in sufficient detail to allow replication of the results presented in the report. Any unacceptable conditions could warrant a requirement for re-submittal of the report, and subsequent delay of the project review.

2.2 CONCEPTUAL DRAINAGE REPORT

The Conceptual Drainage Report is required as a component of the Conceptual Planned Unit Development (P.U.D.) submittal. This report will review at a conceptual level the feasibility and design characteristics of the proposed development. The Conceptual Drainage Report shall be in accordance with the following outline, and as a minimum, shall contain the applicable information listed below. Two (2) copies of the Conceptual Drainage Report shall be submitted to the Town.

2.2.1 Conceptual Report Contents

I. General Location and Description

A. Location

1. Town, County, State Highway and local streets within and adjacent to the site, or the area to be served by the improvements
2. Township, range, section, ¼ section
3. Major drainage ways and facilities
4. Names of surrounding developments

B. Description of Property

1. Area in acres
2. Ground cover (type of ground cover and vegetation)
3. Major drainage ways on property
4. Existing major irrigation facilities such as ditches and canals
5. Proposed land use
6. Floodplain status

II. Drainage Basins and Sub-basins

A. Major Basin Description

1. Reference to major drainage way planning studies, flood hazard delineation reports, flood insurance rate maps
2. Major basin drainage characteristics, existing and planned land uses within the basin as defined by the Town
3. Identification of all nearby irrigation facilities within 100 feet of the property boundary, which will influence or be influenced by the local drainage

B. Sub-basin Description

1. Discussion of historic drainage patterns of the property
2. Discussion of off-site drainage flow patterns and impact on development under existing and fully developed basin conditions as defined by the Town

III. Drainage Facility Design

A. General Concept

1. Discussion of concept and typical drainage patterns
2. Discussion of compliance with off-site runoff considerations
3. Discussion of anticipated and proposed drainage patterns
4. Discussion of the content of tables, charts, figures, plates, or drawings presented in the report
5. Discussion of the Storm Water Quality Control concepts for the site

B. Specific Details (Optional Information)

1. Discussion of drainage problems encountered and solutions at specific design points
2. Discussion of detention storage and outlet design
3. Discussion of opportunities for integration of other functions (recreational, natural resource) within drainage facilities
4. Discussion of maintenance and access aspects of the design
5. Discussion of impacts of concentrating the flow on the downstream properties

IV. Wetland Determination and Review (If Applicable)

V. References

Provide references to all criteria, master plans, and technical information used in support of the drainage concept for the development.

2.2.2 Conceptual Report: Plan Contents

A. General Location Map

All drawings shall be 24" x 36". A map shall be provided in sufficient detail to identify general drainage patterns and drainage flows entering and leaving the development for at least 100' to 200' from the project boundaries. The map should be at a scale adequate to show the path of all drainage from the upper end of any off-site basins to a major drainage way. The map shall identify any major facilities from the property (i.e., development, irrigation ditches, existing detention facilities, culverts, storm sewers) along the flow path to the nearest major drainage way. Basins and divides are to be identified and labeled topographic contours are to be included.

B. Floodplain Information

The location of any defined floodplains on the property shall be shown.

C. Drainage Plan

Map(s) of the proposed development at a scale of 1" = 20' to 1" = 200' on a 24" x 36" drawing shall be included. The plan shall show the following:

1. Existing topographic contours at 2 feet maximum intervals. The contours shall extend a minimum of 100 feet beyond the property lines and be labeled as to elevation.

2. All existing drainage facilities.

3. Locations of all existing and proposed utilities to assure conflicts with proposed drainage facilities are being addressed.

4. Approximate flooding limits based on available information, such as previously defined floodplains and estimated floodplains.

5. Conceptual major drainage facilities including detention basins, storm sewers, swales, riprap, and outlet structures in the detail consistent with the proposed development plan.

6. Location and type of pertinent major drainage facilities identified by the Town relevant to the proposed development.

7. Major drainage boundaries and sub-boundaries.

8. Any off-site feature influencing development.

9. Proposed flow directions and, if available, proposed contours.

10. Legend to define map symbols.

11. Title block in lower right corner.

2.3 Preliminary Drainage Report

The purpose of the Preliminary Drainage Report is to identify and/ or refine the conceptual solutions to the problems, which may occur on-site and off-site as a result of the development. Any problems that existed on the site prior to development must also be addressed during the preliminary phase. The Preliminary Drainage Report shall be submitted with the Preliminary Subdivision Plat submittal. Two (2) copies of the Preliminary Drainage Report shall be submitted to the Town. Number all pages consecutively in the Preliminary Drainage Report including the Appendix for easy reference.

In addition to the information listed below, the requirements for submitting a Preliminary Stormwater Quality Control Plan and a Preliminary Erosion and Sediment Control Plan are detailed in Sections 12 and 13 (respectively).

2.3.1 Preliminary Report Contents

The report shall be in accordance with the following outline and contain the applicable information listed:

I. General Location and Description

A. Location

1. Township, range, section, ¼ section
2. Local streets within and adjacent to the subdivision with ROW width shown
3. Major drainage ways, facilities, and easements within and adjacent to the site
4. Locations of other utilities
5. Names of surrounding developments

B. Description of Property

1. Area in acres
2. Ground cover (type of trees, shrubs, vegetation, general soil conditions, topography, and slope)
3. Major drainage ways
4. General project description
5. Irrigation facilities
6. Proposed land use

II. Drainage Basins and Sub-basins

A. Major Basin Description

1. Reference to major drainage way planning studies, flood hazard delineation reports, flood insurance rate maps
2. Major basin drainage characteristics, existing and planned land uses
3. Identification of all irrigation facilities within the basin, which will influence or be influenced by the local drainage

B. Sub-basin Description

1. Discussion of historic drainage patterns of the property
2. Discussion of off-site drainage flow patterns and impact on development under existing and fully developed basin conditions as defined by the Town

III. Drainage Design Criteria

A. Regulations: Discussion of the optional provisions selected or the deviation from the Criteria, if any, and its justification.

B. Development Criteria Reference and Constraints

1. Discussion of previous drainage studies and/or pertinent master plans for the site in question that influence or are influenced by the drainage design and how the plan will affect drainage design for the site
2. Discussion of the effects of adjacent drainage studies
3. Discussion of the drainage impact of site constraints such as streets and transportation facilities, utilities, existing structures, and the development or site plan

C. Hydrological Criteria

1. Identify design rainfall
2. Identify runoff calculation method
3. Identify detention discharge and storage calculation method
4. Identify design storm recurrence interval
5. Discussion and justification of other criteria or calculation methods used that are not presented in or referenced by the Criteria

D. Hydraulic Criteria

1. Identify various capacity references
2. Discussion of other drainage facility design criteria used that are not presented in the Criteria

E. Waiver/Variance from Criteria

1. Identify provisions by section number for which a waiver or variance is requested
2. Provide justification for each waiver or variance requested

F. Stormwater Quality Considerations

1. See Section 12 for submittal requirements associated with the Stormwater Quality Control Plan
2. See Section 13 for Erosion Control Plan submittal requirements

IV. Wetland Preservation and Mitigation (If Applicable)

V. Drainage Facility Design

A. General Concept

1. Facility design concept and typical drainage patterns
2. Compliance with off-site runoff considerations
3. Discussion of the content of tables, charts, figures, plates, or drawings presented in the report
4. Anticipated and proposed drainage patterns
5. Water quality considerations
6. Opportunities for multi-functional use of drainage facilities.

B. Specific Details

1. Detail drainage problems encountered and solutions at specific design points
2. Detention storage and outlet design
3. Provision of stormwater quality facilities
4. Maintenance access and aspects of facility design
5. Provision of easements and tracts for drainage purposes, including conditions and limitation for use

VI. Conclusions

A. Compliance with Standards

1. Town Criteria
2. USDCM

B. Drainage Concept

1. Effectiveness of drainage design to control damage from storm runoff
2. Influence of proposed development

VII. References

Reference all criteria and technical information used.

VIII. Appendices

A. Hydrologic Computations

1. Land use assumptions regarding adjacent properties
2. Initial and major storm runoff at specific design points
3. Historic and fully developed runoff computations at specific design points
4. Hydrographs at critical design points
5. Time of concentration and runoff coefficients for each basin
6. A computer disk of all hydrologic modeling (CUHP, EPA SWMM, etc.) necessary to support analysis and conclusions in the report; documentation of modeling efforts will be in sufficient detail to allow replication of results.

B. Hydraulic computations

1. Open channel design. Check structure and/or channel drop design
2. Detention area/volume capacity and outlet capacity calculations; depths of detention basins
3. Downstream/outfall system capacity to the Major Drainage way system

C. Hydraulic computations (optional for preliminary)

1. Culvert capacities
2. Storm sewer capacity, including energy grade line (EGL) and hydraulic grade line (HGL) elevations for 30" or larger
3. Gutter and street capacity as compared to allowable
4. Storm inlet capacity including inlet control rating at connection to the storm sewer

2.3.2 Preliminary Drainage Report Checklist

Project: ______________________________ Date: ______________

Refer to Town of Kersey Storm Drainage Design Criteria (SDDC) Manual for requirements.

		Yes	No
1.	Is report signed and sealed by a licensed P.E.?	____	____
2.	Is general location and description in accordance with SDDC manual?	____	____
3.	Are existing contours based on NGVD 29 datum?	____	____
4.	Do contours extend a minimum of 100 feet outside property boundary and are they labeled as to elevation?	____	____
5.	Are basin boundaries to centerline of adjacent streets surrounding the development	____	____
6.	Are drainage areas close to those determined by designer?	____	____
7.	Is offsite water safely passed through the site?	____	____
8.	Are drainage design criteria in accordance with SDDC manual?	____	____
9.	If over 5 acres, has CUHP and SWMM hydraulic analysis been used to size the detention pond?	____	____
10.	Are runoff coefficients reasonable? I.e., 5-year historic average = 0.1, 100-year historic = 0.35.	____	____
11.	Has Water Quality Capture Volume (WQCV) been determined and added to total detention pond volume requirements?	____	____
12.	Is detention pond release rate equal to 5-year historic flow?	____	____
13.	Are pond side slopes no greater than 4H:1V?	____	____
14.	Does volume calculated from pond contours approximately equal designer's volume?	____	____
15.	Is wetland preservation and mitigation required and if so have provisions been made to address these issues?	____	____

____
Signed – Design Engineer

2.3.3 Preliminary Submittal: Plan Contents

A. General Location Map

All drawings shall be 24" x 36". A map shall be provided in sufficient detail to identify drainage flows entering and leaving the development and general drainage patterns. The map should be at a scale adequate to show the path of all drainage from the upper end of any off-site basins to a major drainage way. The map shall identify any major facilities from the property (i.e., development, irrigation ditches, existing detention facilities, culverts, storm sewers) along the entire path of drainage. Basins and divides are to be identified and topographic contours are to be included.

B. Floodplain Information

The location of any defined floodplains on the property shall be shown.

C. Drainage Plan

Map(s) of the proposed development at a scale of 1" = 20' to 1" = 200' on a 24" x 36" drawing shall be included. The plan shall show the following:

1. Existing and (if available) proposed topographic contours at a 2-foot maximum interval. The contours shall extend a minimum of 100 feet beyond the property lines and be labeled as to elevation.

2. Property lines and easements with purposes noted.

3. Streets, indicating ROW width, flow line width, curb type, sidewalk, and approximate street slopes.

4. Existing drainage facilities and structures, including irrigation ditches, roadside ditches, cross-pans, drainage ways, gutter flow directions, and culverts. All pertinent information such as material, size, shape, slope, and location shall also be noted.
5. Location and type of pertinent major drainage facilities identified in the Comprehensive Drainage Plan relevant to the proposed development.

6. Locations of other utilities.

7. Overall drainage area boundary and drainage sub-area boundaries.

8. Proposed type of street flow (i.e., vertical or combination curb and gutter), roadside ditch, gutter, slope and flow directions, and cross-pans.

9. Proposed storm sewers and open drainage ways, including inlets, manholes, culverts, and other appurtenances, including riprap protection.
10. Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties.
11. Routing and accumulation of flows at various critical points for the initial storm runoff listed on the drawing using the format shown in Table 2-1.
12. Routing and accumulation of flows at various critical points for the major storm runoff listed on the drawing using the format shown in Table 2-1.
13. Volumes and release rates for detention storage facilities and information on outlet works.
14. Location and elevations of all existing floodplains affecting the property.
15. Location and (if known) elevations of all existing and proposed utilities affected by or affecting the drainage design.
16. Routing of off-site drainage flow through the development.
17. Definition of flow path leaving the development through the downstream properties ending at a major drainage way.
18. Legend to define map symbols.
19. Title block in lower right corner.

2.4 FINAL DRAINAGE REPORT

The purpose of the Final Drainage Report is to update the concepts and present the design details for the drainage facilities discussed in the Preliminary or Conceptual Drainage Reports. Also, any changes to the Preliminary Drainage concept must be presented and supported with the same level of information as originally required in the Preliminary Drainage Report. Number all pages of the Final Drainage Report including the Appendix for easy reference.

The Final Drainage Report, which shall accompany the Final Subdivision submittal or the final Planned Unit Development (P.U.D.) submittal, must address comments made during the review of the Preliminary or Conceptual submittals. The Final Drainage Report shall be prepared in accordance with the outline for the Preliminary Drainage Report (Section 2.3.1); the Final Drainage Report drawings and plans shall fulfill the requirements for the contents of the Preliminary plans (Section 2.3.2). See Section 12 of these Criteria for the Final

Stormwater Quality Control Plan submittal requirements, and Section 13 for the Final Erosion Control Plan submittal requirements. Two (2) copies of the Final Drainage Report shall be submitted to the Town.

Include in the final drainage report bound 11"X17" copies of the drainage exhibits as well as the full size sheets. If smaller copies of the exhibit are bound into the report they shouldn't become separated.

In addition to the report format and submittal requirements presented in Section 2.1, the Final Drainage Report shall include a page with the following certification language, and the appropriate signature:

(Name of Developer/Owner) hereby certifies that the drainage facilities for (Name of Project) shall be constructed according to the design presented in this report. I understand that the Town of Kersey does not and will not assume liability for drainage facilities designed and/or certified by my Engineer. I also understand that the Town of Kersey relies on the representations of others to establish that drainage facilities are designed and constructed in compliance with Town guidelines, standards, or specifications. Review by the Town of Kersey can therefore in no way limit or diminish any liability, which I or any other party may have with respect to the design or construction of such facilities.

Attest: ______________________________
(Name of Responsible Party)

_________________________ _________________________
Notary Public Authorized Signature

2.4.1 Final Drainage Report Checklist

Project:______________________________Date:____________________

Refer to Town Of Kersey Storm Drainage Design Criteria Manual for requirements.

		Yes	No
1.	Is report signed and sealed by a licensed P.E.?	_____	_____
2.	Is report certified by owner and notarized?	_____	_____
3.	Are all pages of the report numbered consecutively for easier referencing?	_____	_____
4.	Are there any infringements on drainage easements?	_____	_____
5.	Are sub-basin areas and total area of site close to those of designer?	_____	_____
6.	Time of concentration, Check if (L/180) + 10 verified?	_____	_____
7.	Are runoff coefficients reasonable, 5 year historic average = 0.1, 100 year historic = 0.35 in Kersey?	_____	_____
8.	Do offsite flows - pass through pond or route around site?	_____	_____
9.	Has Water Quality Capture Volume (WQCV) been determined and added to total detention pond volume requirements?	_____	_____
10.	Is detention pond release rate equal to 5 year historic flow?	_____	_____
11.	Are drainage channel and pond side slopes no greater than 4H:1V?	_____	_____
12.	Have detention pond top of dike elevations (freeboard) been set 1 foot above the elevation of water passing over the spillway during a plugged orifice condition?	_____	_____
13.	Does volume calculated from pond contours approximately equal designer's volume?	_____	_____
14.	Is the spillway riprapped, or concreted and have sufficient grading to pass the overflow downstream to a suitable conveyance?	_____	_____
15.	Orifice calculations - Is head on orifice to center of opening?	_____	_____
16.	Is backwater from outlet being flooded possible and if so has it been considered?	_____	_____
17.	On the WQCV drawing details - are the holes per row and the spacing of holes correctly determined?	_____	_____
18.	Has a trash rack and orifice plate been properly designed and detailed?	_____	_____

	Yes	No
19. Are outlet metal components minimum 1/4" thick metal hot dipped galvanized?	____	____
20. Are bolts a minimum 3/8" diameter, by 1 ½" minimum length plus 3" for concrete embedment and stainless steel material?	____	____
21. Has the street capacity at critical locations for both minor and major storm events been determined?	____	____
22. Have inlet capacities using interception ratios been determined, if applicable?	____	____
23. Do grading contours around perimeter of site match existing terrain?	____	____
24. Are they labeled for elevation?	____	____
25. Do spot elevations match grading contours?	____	____
26. Do all areas drain to the detention/retention pond?	____	____
27. Are details for detention pond retaining wall construction shown?	____	____
28. Is "A" and "B" lot designation with diagram shown on grading plan?	____	____
29. Are there any "B" lots adjoining other "B" lots along back lot lines?	____	____
30. If so, is there an Out lot separating them for drainage or a concrete trickle pan off set to one side of the property line been provided?	____	____
31. Are finished top of foundation and lot corner elevations shown on the grading plan?	____	____
32. Have hydraulic calculations with energy grade line (EGL) and hydraulic grade line (HGL) been shown for the 100 year storm event on the storm sewer profiles?	____	____
33. Have backwater affects been considered?	____	____
34. Do pipe sizes and slopes agree between the drawings and calculations?	____	____
35. Have adequate sump depths been provided at sump inlets to accommodate sump design capacity before allowing water to overflow curb?	____	____
36. Are storm sewers sized so that there is no surcharge during a 2 year storm event?	____	____
37. Do the drawings contain standard Town of Kersey details for storm sewer pipe bedding, inlets, manholes, inlet protection vehicle tracking control, etc?	____	____
38. Is there at least 1 foot of coverage between the top of all RCP storm sewer pipes and top of pavement?	____	____

	Yes	No
39. Are storm sewer manholes placed no more than 400 feet apart and are they accessible from the street?	____	____
40. On drop structures - has the location of the hydraulic jump and seepage distance been determined, is there adequate distance for protection from short circuiting below and around the ends of the structures?	____	____
41. On culverts - has both inlet and outlet control been taken into account?	____	____
42. Is there erosion protection at the discharge of storm sewer pipes, if needed?	____	____
43. Have toe walls been provided at storm sewer outlets?	____	____
44. If turf reinforcement mat (TRM) is proposed have provisions for irrigation also been provided?	____	____
45. Is riprap placed on geotextile fabric and bedding or been called out as grouted?	____	____
46. In any channel designs; is there adequate freeboard (1ft.), velocity (<5 ft/s sandy soils, <7 ft/s clay soils, Froude Number <.8)?	____	____
47. On sidewalk chases - 2 year storm should pass under and 100 year storm should pass over the sidewalk?	____	____
48. Is a minimum opening elevation (M.O.) shown on all buildings next to ponds and major drainage swales? The M.O. elevation shall be 1' above the top of the stormwater elevation as it overtops the spillway. This elevation would be the minimum allowed elevation for all windowsill elevations, doorway threshold elevations, top of window well elevations, garage door threshold elevations, and any other building opening water could penetrate. Include an explanation in the key of the drawing for the M.O. elevation designation.	____	____

____
Signed - Design Engineer

2.5 CONSTRUCTION PLANS

Where drainage improvements are to be constructed, the final construction plans (24" x 36") shall be submitted with the Final Drainage Report. One (1) set of the construction plans shall be submitted for review and comment. After the changes have been made, the original set, one set of reproducible plans and four additional sets of plans shall be submitted for final approval and acceptance. The original set shall be signed and returned to the originator. Approval of the construction plans by the Town is a condition of the Town for issuing all construction permits, except for the grading permit which may be issued prior to approval of the construction plans.

Should circumstance warrant changes from the approved plans or specifications, a written approval must be obtained from the Town. Copies shall be given to the Contractor and the Developer. Project as built record drawings which record changes in construction are the Developer/Owners responsibility. These record drawings shall consist of detailed drawings which have been prepared by the Developer/Owner's Design Engineer, upon completion and at the time of the Certificate of Completion, and show actual construction and contain field dimensions, elevations, details, changes made to the construction drawings by modification, details which were not included on the construction drawings, and horizontal and vertical locations of underground utilities which have been impacted by the utility installation.

The plans for the drainage improvements shall include:

1. General Details

   a. Title Block (lower right hand corner preferred)
   b. Scale
   c. Date and Revisions Block
   d. Name of Professional Engineer and Firm
   e. Statement: All work must be in accordance with applicable Town Of Kersey construction standards. The Town's acceptance allows for plan distribution and permit application. The Town's acceptance shall not relieve the design engineer's responsibility for errors, omissions, or design deficiencies for which the Town is held harmless.

Accepted By: ______________________________ ____________________
Town Engineer Date

2. Plan Portion

   a. North Arrow
   b. Property lines
   c. Easement Limits with Dimensions
   d. Ownership of Subdivision Information
   e. Street Names

f. All Existing Utilities
g. All Topographic Features (houses, curbs, water courses, etc.)

3. Proposed Facilities

a. Storm sewers, inlets, outlets, and manholes with pertinent elevations, dimensions, type, and horizontal control indicated.
b. Culverts, end sections, and inlet/outlet protection with dimensions, type, elevations, and horizontal control indicated.
c. Channels, ditches, and swales (including side and/ or back yard swales) with lengths, widths, crosssections, and erosion control (i.e. riprap, concrete, grout) indicated.
d. Check structures, channel drops, erosion control facilities.
e. Detention pond grading (elevations and horizontal control), trickle channel, and outlets.
f. Other drainage related structures and facilities (including under drains and sump pump lines).
g. Maintenance access considerations.
h. Over lot grading.
i. Lot corner elevations for all residential lots.
j. Top of foundation (TOF) elevations (which are set two (2) feet higher than the highest curb elevation fronted by the property).

4. Profile Information

a. Stationing
b. Elevations shall be National Geodetic Vertical Datum (NGVD) 29
c. Length Between Structures and Connections
d. Slope of Pipe
e. Existing Ground Profile
f. Proposed Ground or Street Profile
g. HGL and EGL of Storm Sewers

The information required for the plans shall be in accordance with sound engineering principles, these Criteria, and the Town requirements for sub-structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the storm drainage facilities. The approved Final Plan shall be included as part of the construction documents for all facilities affected by the drainage plan. Construction plans shall be signed by a Registered Professional Engineer as being in accordance with the Town approved drainage reports/drawings. Construction plans along with the Town's Design Criteria and Construction Specifications Manuals shall be provided at the construction site by the Contractor at all times.

2.6 Construction Certification & Drawings of Record

Record drawings for all improvements are to be submitted to the Town with the request for Certificate of Completion. Record drawings shall be furnished in the following manner: one complete set on Mylar and one complete set as blue lines. The Mylar set shall state the date signed, and that the blueprints are on file with the Town; the blue line set shall include the signature and stamp of the Professional Engineer. Certification of the record drawings is required as follows:

a) Registered Land Surveyor: A registered land surveyor in the State of Colorado shall certify the as-built detention pond volumes and surface areas at the design depths, outlet structure sizes and elevations, storm sewer sizes and invert elevations at inlets, manholes, and discharge location, and representative open channel cross-sections, and dimensions of all the drainage structures.
b) Registered Professional Engineer: The responsible engineer shall state that "to the best of my knowledge, belief, and opinion, the drainage facilities were constructed in accordance with the design intent of the approved drainage report and construction drawings" in accordance with State Law.

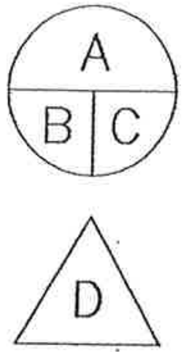
The Town shall compare the certified record drawing information with the construction drawings. A Certificate of Completion shall be issued only if:

1. The record drawing information demonstrates that the construction is in compliance with the design intent.

2. The record drawings are certified by both a Professional Land Surveyor and a Professional Engineer, both registered in the State of Colorado.

A summary of the required certifications and approvals is presented below:

Certifications & Approvals Required		
Item	**Certification Required**	**Town Approval Required**
Conceptual Report	Professional Engineer	Yes
Preliminary Report	Professional Engineer	Yes
Final Report	Professional Engineer & Responsible Party	Yes
Construction Drawings	Professional Engineer	Yes
Detention Pond Easement Agreement	Professional Engineer & Notary Public	Yes
Subdivision Grading Certificate	Professional Engineer or Professional Land Surveyor	Yes
Record Drawings	Professional Engineer Professional Land Surveyor	Yes (Certificate of Completion)



A = BASIN DESIGNATION

B = AREA IN ACRES

C = COMPOSITE RUNOFF COEFFICIENTS

D = DESIGN POINT DESIGNATION

SUMMARY RUNOFF TABLE
(TO BE PLACED ON DRAINAGE PLAN)

DESIGN POINT	CONTRIBUTING AREA (ACRES)	RUNOFF 5YR. (CFS)	PEAK 100YR. (CFS)
XX	XX.XX	XX.X	XX.X

TOWN OF KERSEY	
DRAWING SYMBOL CRITERIA AND HYDROLOGY REVIEW TABLE	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET TABLE 2-1

SECTION 3.0: RAINFALL

3.1 INTRODUCTION

Presented in this section is the design rainfall data to be used with the Colorado Urban Hydrograph Procedure (CUHP) and the Rational Method. All hydrological analysis within the jurisdiction of the Criteria shall use the rainfall data presented herein for calculating storm runoff.

The design storms and intensity-duration-frequency curves for the Town were developed using the rainfall data as presented in the NOAA Atlas for Colorado and the procedures presented in the Urban Storm Drainage Criteria Manual (USDCM).

3.2 SELECTION OF DESIGN STORM FREQUENCIES

All drainage systems for new development have to take into consideration two separate and distinct drainage situations. The first is the initial storm, which occurs at fairly regular intervals and is based on the two- to ten-year storm event, depending on land use. The runoff from the initial storm is usually not the cause of extensive damage, but can represent higher costs in maintenance, repair and replacement of public facilities if not handled correctly.

The second drainage situation that must be considered is the planning and design of facilities to convey the major storm, which occurs at less frequent intervals, and is based on the 100-year storm event. The runoff from this type of storm event can cause catastrophic property damage and personal injury or loss of life.

The purpose of the major drainage path criteria is to establish guidelines to protect the major paths from encroachment. No development shall be allowed along the major drainage path except by specific approval of the Town. The major drainage paths are shown on exhibits in the Comprehensive Drainage Plans for each major drainage basin within the Town. All development that occurs along the major paths will be required to meet the guidelines herein. The guidelines herein are intended to be supplements to the specific recommendations made for each drainage basin analyzed in the Comprehensive Drainage Plans.

The major drainage paths are a part of the major drainage system and all facilities along said paths will be required to be designed using a 100-year design storm frequency.

The 100-year runoff shall be computed at the point where the major drainage path enters the proposed development and again where the major drainage path leaves the development. Intermediate points of runoff shall be computed where deemed critical and to better define the major drainage path. Critical points are generally identified by the intersection of a drainage path with the major drainage path, rapid changes in grades or channel widths. The runoff shall be based on the total area tributary to the runoff point regardless of the ratio of developed area to the drainage basin.

The width of the channel and maintenance access to the channel shall be as defined in Section 5.0-Open Channels. No development will be allowed within this designated area. Said area shall be called a drainage right-of-way and identified as such on all plans and plats. The drainage right-of-way shall be dedicated to the Town for public use. Access roads shall be dedicated to the Town where necessary.

Where possible and as required by the Town, the major drainage channels shall be incorporated into greenbelt and recreational areas. Natural drainage channels lend themselves to greenbelt areas. Where the width can be extended, hiking trails, pedestrian walkways and bicycle paths should be located in the major drainage path. Due consideration should be given to street and utility layout to gain maximum aesthetic use of the drainage path.

The initial and major storm frequencies to be used for runoff analysis and the subsequent design of stormwater control facilities in the Town of Kersey are presented below:

Design Storm Frequencies		
Land Use	Initial Storm Frequency	Major Storm Frequency
Residential	2-year	100-year
Commercial, Business & Industrial	5-year	100-year

(1) Any industrial development within or adjacent to the downtown business area shall have an initial storm frequency of 10-years.

3.3 Colorado Urban Hydrograph Procedure (CUHP) Design Storms

For drainage basins less than five square miles, a two-hour storm distribution without area adjustments of the point rainfall values shall be used for CUHP. For drainage basins between five and ten square miles, a two-hour storm distribution is used but the incremental rainfall values are adjusted for the large basin area in accordance with suggested procedures in the NOAA Atlas for Colorado. The adjustment is an attempt to relate the average of all point values for a given duration and frequency within a basin to the average depth over the basin for the same duration and frequency.

For drainage basins between ten and twenty square miles, a threehour storm distribution with adjustment for area shall be used. The distribution for the last hour was obtained by uniformly distributing the difference between the two- and three-hour point rainfall values. The adjustment for area was obtained from the NOAA Atlas for

Colorado. The incremental rainfall distribution for all basin areas up to 20 square miles is presented in Table 3-1.

3.4 INTENSITY-DURATION-FREQUENCY (IDF) CURVES

Intensity-Duration-Frequency (IDF) curves are necessary to utilize the Rational Method for runoff analysis. The one-hour design point rainfall values obtained from the NOAA Atlas for Colorado are required for the development of the IDF curves. The one-hour point rainfall values applicable for the Town of Kersey are presented 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

The IDF curves were developed by distributing the one-hour point rainfall values using the factors obtained from the NOAA Atlas as presented below.

Factors for Durations of Less than One Hour				
Duration	5	10	15	20
Ratio to 1-hour Depth	1.49	1.76	2.51	2.78

The point values were then converted to intensities and plotted on Figure 3-1. The data are also presented in Tables 3-1, 3-2, and 3.3.

Table 3-1 Design Storms for Kersey Incremental Rainfall Depth/Return Period

Time (min)	Basins Less Than 5 Sq. Miles					Basins Between 5 & 10 Sq. Miles					Basins Between 10 & 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	0.11	0.03	0.05	0.06	0.08	0.14
55	0.03	0.05	0.06	0.008	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	0.03	0.04	0.03	0.02	0.03	0.03	0.04	0.03	0.02	0.03	0.03	0.04	0.03
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											0.0	0.00	0.00	0.00	0.01
TTL	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

Table 3-2 Intensity-Duration-Frequency Tabulation

DURATION:	5 MIN	10 MIN	15 MIN	30 MIN	60 MIN	120 MIN	180 MIN
FACTOR:	0.29	0.45	0.57	0.79	1.00		
2-YEAR							
1-Hr Depth (In):	1.04	1.04	1.04	1.04	1.04		
Depth at Duration (In):	0.30	0.47	0.59	0.82	1.04	1.16	1.25
Intensity (In/Hr):	3.62	2.81	2.37	1.64	1.04	0.58	0.42
5-YEAR							
1-Hr Depth (In):	1.49	1.49	1.49	1.49	1.49		
Depth at Duration (In):	0.43	0.67	0.85	1.18	1.49	1.60	1.69
Intensity (In/Hr):	5.19	4.02	3.40	2.35	1.49	0.80	0.56
10-YEAR							
1-Hr Depth (In):	1.76	1.76	1.76	1.76	1.76		
Depth at Duration (In):	0.51	0.79	1.00	1.39	1.76	1.91	2.02
Intensity (In/Hr):	6.12	4.75	4.01	2.78	1.76	0.96	0.67
50-YEAR							
1-HR Depth (In):	2.51	2.51	2.51	2.51	2.51		
Depth at Duration (In):	0.73	1.13	1.43	1.98	2.51	2.65	2.75
Intensity (In/Hr):	8.73	6.78	5.72	3.97	2.51	1.33	0.92
100-YEAR							
1-HR Depth (In):	2.78	2.78	2.78	2.78	2.78		
Depth at Duration (In):	0.81	1.25	1.58	2.20	2.78	2.99	3.15
Intensity (In/Hr):	9.67	7.51	6.34	4.39	2.78	1.50	1.05

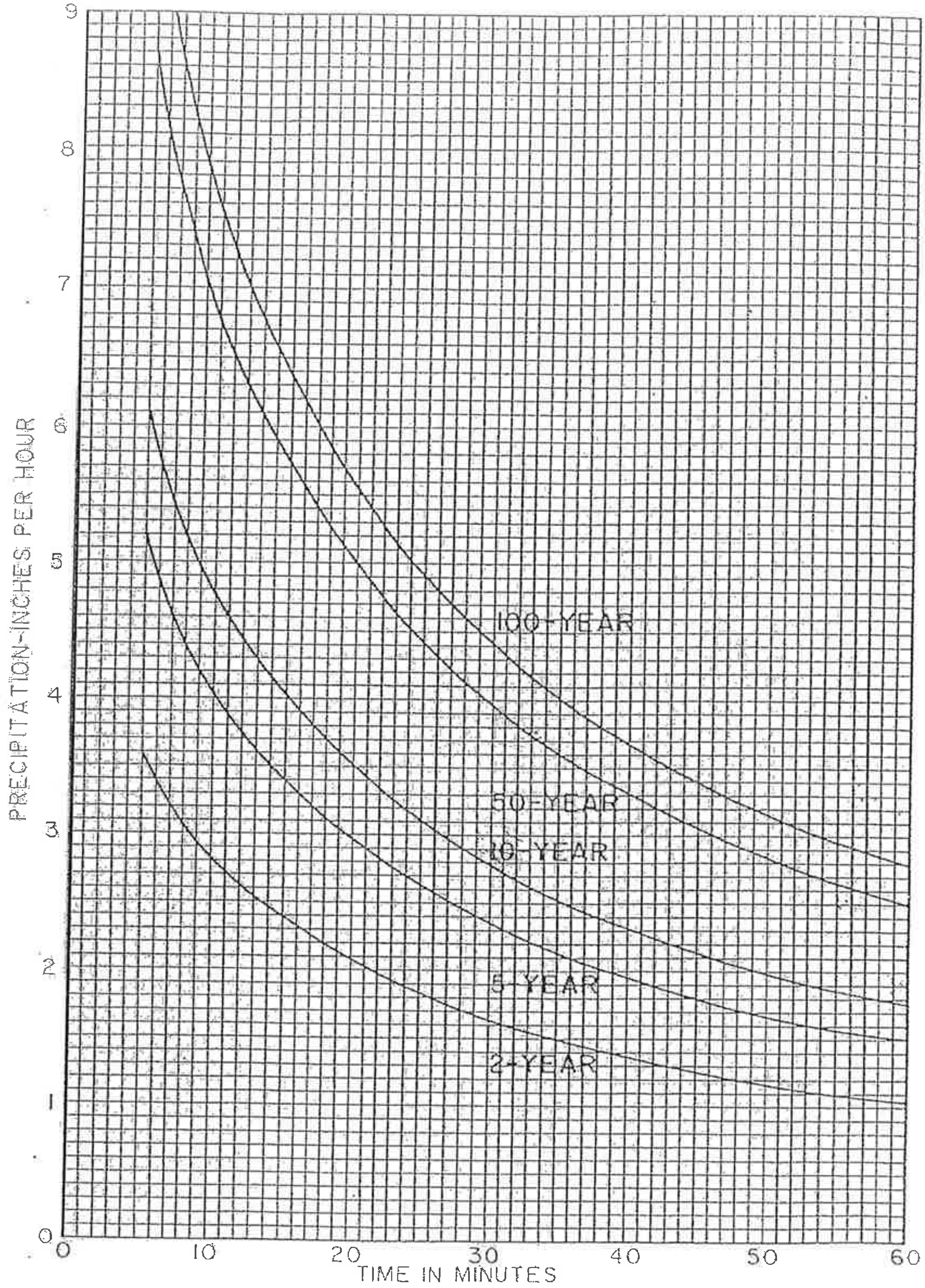
Note: Depth at each duration = 1-Hour of Rainfal Depth x Respective Duration Factor

Example: 5 Minute Duration

Depth (In) = 2-Year, 1-Hour Depth (In) x Factor
= 1.04 x 0.29
= 0.30 Inches

Table 3.3 Extended Duration-Intensity-Frequency Tabulation, Kersey, Colorado

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



TOWN OF KERSEY

INTENSITY—DURATION FREQUENCY CURVES

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 3-1

SECTION 4.0: RUNOFF ANALYSIS

4.1 INTRODUCTION

This section presents the criteria and methodology for determining the storm runoff design peaks and volumes to be used in the Town of Kersey in the preparation of storm drainage studies, plans, and facility design. Further details and discussion of each of these rainfall/runoff models are presented in the Urban Storm Drainage Criteria Manual (USDCM).

4.2 RATIONAL METHOD

The Rational Method may be utilized for the sizing of storm sewers and for determining the amount of runoff from undeveloped areas. The limit of application of the Rational Method is approximately 160 acres. It has been concluded that, for tributary basins in excess of 160 acres, the cost of the drainage works justifies significantly more study, thought, and judgment on the part of the Engineer, than is permitted by the Rational Method. When the urban drainage basin exceeds 160 acres, the CUHP Method represents better practice and should be used.

The procedures for the Rational Method, as explained in the USDCM, Volume 1 "Runoff" shall be followed in the preparation of drainage reports and storm drainage facility designs in the Town.

Standard forms for the calculation of Time of Concentration and Storm Drainage System Design are provided in Tables 4-1 and 4-2.

4.3 COLORADO URBAN HYDROGRAPH PROCEDURE (CUHP)

The application of the CUHP is generally applicable to basins greater than 90 acres, and is required for drainage basins larger than 160 acres. The procedures for the CUHP, as explained in the USDCM, Volume 1 "Runoff", shall be followed in the preparation of drainage reports and storm drainage facility designs in the Town. The design storms to be used with the CUHP method are presented in Section 3 of these Criteria. Applicable infiltration depths are outlined in Table 2-3, Section 2.4 of the USDCM, Volume 1, "Runoff".

A computer program has been developed to calculate hydrographs using the CUHP Method. This program is available for use on both large mainframe computers and on the smaller IBM-PC XT or IBM-PC AT personal computers. In addition, the personal computer version has an added capability of using CUHP to compute runoff hydrographs for basins as small as 10 acres. For the Town, only the personal computer version with the capability to model basins as small as 10 acres shall be allowed. Specific details of the required input for this version are available in the CUHPE/PC

Version Users Manual. Both the CUHPD (large mainframe) and CUHPE/PC (personal computer) computer programs can be obtained by contacting the Urban Storm Drainage and Flood Control District.

The CUHPE/PC computer program was modified to provide the capability of estimating hydrographs for small drainage basins (generally less than 90 acres) for which the original CUHP method is not applicable. The resulting flood peaks in many cases are generally comparable, but not identical to those estimated by the Rational Formula as specified in the USDCM. To estimate a hydrograph for small basins requires the input of the time of concentration as computed by the Rational Method described in the USDCM. It is often advantageous to generate a storm hydrograph to facilitate the routing of flows through detention facilities or channels.

4.4 Storm Flow Analysis

When determining the design storm flows, the Engineer shall follow particular criteria and guidelines to assure that minimum design standards and uniformity of drainage solutions are maintained throughout the Town. The information presented herein shall be used by the Design Engineer in the development of design storm runoff. Runoff coefficients used for analysis shall be as given in Tables RO-3 and RO-5 of Section "Runoff", Volume 1 of the Urban Storm Drainage Design Criteria Manual (USDCM) latest edition (June 2001) or "C" factors can be as determined in Table 3-1 of the 1990 Volume 1 USDCM. An acceptable percentage imperviousness for soils found in the Kersey area typically should be 0 percent as given in Table RO-3.

4.4.1 On-site Flow Analysis

When analyzing the flood peaks and volumes, the Design Engineer shall use the proposed fully developed land use plan to determine runoff coefficients. In addition, the Engineer shall take into consideration the changes in flow patterns (from the undeveloped site conditions) caused by the proposed street alignments. When evaluating surface flow times, the proposed site grading shall be used to calculate the time of concentration or the CUHP parameters.

4.4.2 Off-site Flow Analysis

The analysis of off-site runoff is dependent on the development status and whether the tributary off-site area lies within a major drainage way basin. In some cases, credit may be given for detention as defined below.

4.4.3 Tributary Area Within a Major Drainage Way Basin

Where the off-site area is undeveloped, the runoff shall be calculated assuming a fully developed basin as defined by the Town. If this information is not available, then the runoff shall be calculated using the coefficients defined in Table RO-3 and RO-5 of

Section "Runoff," Volume 1 of the USDCM, "Undeveloped Areas – Offsite Flow Analysis."

Where the off-site area is fully or partially developed, the storm runoff shall be based upon the existing platted land uses and topographic features. Credit may be allowed for on-site detention in the undeveloped off-site area for any design frequency.

4.4.4 Tributary Area Not Within A Major Drainage Way Basin

Where the off-site area is undeveloped, storm runoff shall be calculated assuming a fully developed basin as defined by the Town. If this information is not available, then the runoff will be calculated as stated in Section 4.4.3, without credit for on-site detention in the off-site area.

Where the off-site area is fully or partially developed, storm runoff for the developed area shall be based on the existing platted land uses and topographic features. Credit may be allowed for on-site detention in the developed off-site areas provided it has been constructed and accepted by the Town. Storm runoff for the undeveloped areas shall be calculated assuming full development as defined by the Town. No credit will be given for on-site detention in the undeveloped off-site area.

4.5 Channel Routing

Whenever a larger or non-homogeneous watershed is being investigated, it is necessary to segment the watershed into smaller and somewhat homogeneous sub-basins. The storm hydrograph for each sub-basin can then be calculated by the CUHP methodology as explained in the USDCM, Volume 1 "Runoff". It is up to the Engineer to route and combine the individual sub-basin hydrographs to calculate a storm hydrograph for the entire watershed. There are several methods commonly used in channel routing that include:

- Direct Translation
- Convex
- Muskingum
- Storage-Discharge (Modified Puls)
- Kinematic Wave
- Diffusion Wave
- Dynamic Wave

The Direct Translation and Convex methods are presented in the USDCM, Volume 1 "Runoff". The last three methods are more accurate; however these methods require the utilization of a highspeed computer. Computer programs, such as the EPA Stormwater Management Model (SWMM) which incorporates the Kinematic wave method, are available to route flows through channels, pipes and detention ponds and are recommended for utilization within the Town. Of the remaining methods, the

Muskingum method is similar to the Convex method and the Storage-Discharge method is less convenient for hand calculations than the Direct Translation or Convex methods. Other computer programs capable of routing flows through stormwater conveyance channels and detention ponds will be reviewed by the Town and written approval must be obtained prior to utilization.

CALCULATED BY ______
DATE ______
CHECKED BY ______

JOB NO. ______
PROJECT ______
DESIGN STORM ______

		DIRECT RUNOFF							TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			
STREET	DESIGN POINT	DESIGN AREA	AREA (AC)	RUNOFF COEFF	tc (MIN.)	CA (AC)	I IN/HR	Q (CFS)	tc (MIN.)	Σ(CA) (AC)	I IN/HR	Q (CFS)	SLOPE %	STREET FLOW CFS	DESIGN FLOW CFS	SLOPE %	PIPE SIZE	LENGTH (FT)	VELOCITY (FPS)	tt (MIN.)	REMARKS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
1																					
2																					
3																					
4																					
5																					
6																					
7																					
8																					
9																					
10																					
11																					

TOWN OF KERSEY

STORM DRAINAGE SYSTEM DESIGN

SCALE: NONE | DATE: 12/04

APPROVED:

FJB

TOWN ENGINEER

SHEET TABLE 4-1

TIME OF CONCENTRATION

SUBDIVISION ______________________________

CALCULATED BY ________________ DATE ________________

SUB-BASIN DATA			INITIAL/OVERLAND TIME (t_i)			TRAVEL TIME (t_t)				t_c CHECK (URBANIZED BASINS)			FINAL t_c	REMARKS
DESIGN (1)	AREA AC (2)	C_S (3)	LENGTH FT (4)	SLOPE % (5)	t_i MIN (6)	LENGTH FT (7)	SLOPE % (8)	VEL FPS (9)	t_t MIN (10)	COMP tc (11)	TOTAL LENGTH FT (12)	t_c = (l/180) + 10 MIN (13)	MIN (14)	

TOWN OF KERSEY

TIME OF CONCENTRATION

SCALE: NONE

DATE: 12/04

APPROVED:

FJB

TOWN ENGINEER

SECTION 5.0 OPEN CHANNELS

5.1 INTRODUCTION

This section addresses the technical criteria for the hydraulic evaluation and hydraulic design of open channels in the Town. The information presented herein is considered to be a minimum standard. In many instances, special design or evaluation techniques will be required. Except as modified herein, all open channel criteria shall be in accordance with the most current edition and/or revisions of the Urban Storm Drainage Criteria Manual (USDCM).

5.2 CHANNEL TYPES

The channels in the Town are defined as natural or artificial, and either major drainage ways or small drainage ways. Natural channels include all watercourses that have occurred naturally. Artificial channels are those constructed or developed by human effort, large designated floodways, irrigation canals and flumes, roadside ditches, and grassed or lined channels to convey runoff.

Major drainage ways, as defined by these criteria, will be identified and classified in conjunction with the Town. All remaining drainage ways shall be classified as small drainage ways. Generally, channels conveying 100 cfs or greater flow shall be considered a major channel, however, there may be some instances where a channel conveying less than 100 cfs may be considered a major drain way.

5.2.1 Natural Channels

The hydraulic properties of natural channels vary along the channel reach and can be either controlled to the extent desired or altered to meet given requirements. The initial decision to be made regarding natural channels is whether or not the channel is to be protected from erosion due to high velocity flows, or protected from excessive silt deposition due to low velocities.

Many natural channels in urbanized and developing areas have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggradation. However, if a natural channel is to be used for carrying storm runoff from an urbanized area, the altered nature of the runoff peaks and volumes from urban development will cause erosion. Detailed hydraulic and channel stability analysis will be required for natural channels in order to identify the erosion tendencies and the impact of the storm runoff on channel stability. Some on-site modifications of the natural channel may be required to assure a stabilized condition.

The investigations necessary to assure that the natural channels will be adequate are different for every waterway. The Engineer must prepare cross sections of the channel, define the water surface profile for the initial and major design flood, investigate the bed and bank material to determine erosion tendencies, and study the stability of the channel bed and bank under future conditions of flow. Super critical flow does not normally occur in natural channels, but calculations must be made to assure that the results do not reflect super critical flow. If super critical flow is present, drop structures or other appropriate energy dissipation structures must be provided.

5.2.2 Grass Lined Channels

Grass lined channels are the most desirable of the artificial channels. The grass will stabilize the body of the channel, consolidate the soil mass of the bed, check the erosion on the channel surface, and control the movement of soil particles along the channel bottom. The channel storage, the lower velocities, and the greenbelt multiple-use benefits obtained create significant advantages over other artificial channels.

The presence of grass in channels creates turbulence, which results in loss of energy and increased flow retardance. Therefore, the designer must give full consideration to sediment deposition and to scour, as well as hydraulics. Unless existing development within the Town restricts the availability of right-of-way, only channels lined with grass will be considered acceptable for major drainage ways. Grass lined channels may require an acceptable trickle channel, as defined in Section 5.4.2.

For the purpose of these Criteria, sandy soils are defined as noncohesive sands classified as SW, SP or SM in accordance with the Unified Soil Classification System.

5.2.3 Concrete Lined Channels

Concrete lined channels for major drainage ways will be permitted only where right-of-way restrictions within existing development prohibit grass lined channels. The lining must be designed to withstand the various forces and actions, which tend to overtop the bank, deteriorate the lining, erode the soil beneath the lining, and erode unlined areas, especially for the super critical flow conditions.

If the project constraints suggest the use of a concrete channel for a major drainage way, the Town shall allow such use only upon approval. The applicant shall present the justification and design to the Town for consideration of a variance from these Criteria. A Design Report is required for approval of a concrete lined channel. The Town shall determine the contents of such report.

5.2.4 Rock Lined Channels

Riprap lined channels are generally accepted. The advantage of rock lining a channel is that a steeper channel grade can be used due to the higher friction coefficient of the rock and a higher allowable shear stress. Also, steeper side slopes are permitted. Rock linings, or revetments, are permitted as a means of controlling erosion for natural channels.

If the project constraints dictate the use of riprap lining for a major drainage way, then the Engineer must present the concept, with justification, to the Town for consideration of a variance from these Criteria. The design of rock-lined channels shall be in accordance with the most current revision of the USDCM, Volume 1, "Major Drainage, Section 4.4 – Riprap – Lined Channels."

5.2.5 Other Lining Types

The use of synthetic fabrics and slope revetment mats for major drainage ways in the Town is restricted to areas of existing development where the ROW constraints prohibit the use of a grass lined section. A synthetic lining, such as a soil stabilization fabric, in combination with grass lining may be acceptable in some situations. If a soil stabilization fabric also referred to as turf reinforcement mat (TRM) is used in combination with grass, a permanent irrigation system must be included. Grass shall always be planted prior to installing the fabric. Provide details on construction drawings for proper installation of the fabric, according to manufacture's recommendations. Such use shall be allowed only upon written approval from the Town. The linings shall be restricted to channels with a Froude Number of 0.8 or less.

5.2.6 Wetlands Vegetation Bottom Channels

The selection of a particular channel can be based on many factors, including hydraulic practice, environmental design, sociological impact, and basic project requirements. However, prior to choosing the channel type, the need or desire for channelization should be established.

Once a decision is made to channelize, then investigations into the status of the present drainage way are necessary to define the constraints on the channel design. For instance, if the channel presently has wetland characteristics, then the Section 404 requirements of the Clean Water Act may require that the design maintain a wetland area. The Engineer should contact the Corps of Engineers for additional information.

The process of choosing a channel configuration and the design criteria for a wetlands bottom channel (if this type is selected) shall follow the latest revision of the USDCM. The engineer is referred to these interim criteria (Section 2 of the USDCM under Major Drainage) for the procedures and criteria for all channel design.

5.3 FLOW COMPUTATION

Uniform flow and critical flow computations shall be in accordance with the USDCM, Sections 2.2.3 and 2.2.4, "Major Drainage", and shall use the Manning's equation as follows:

$$Q = \frac{1.49}{n} (AR^{2/3}S^{1/2}) \quad \text{(Equation 5.1)}$$

Where:
- Q = flow rate (cfs)
- n = Manning's roughness coefficient
- A = area (ft^2)
- R = A/P = hydraulic radius (ft)
- P = wetted perimeter (ft)
- S = energy grade line slope (ft/ft)

5.4 Design Standards for Major Drainage Ways

These standards cover the design of major drainage ways. The design standards for open channels cannot be presented in a step-by-step fashion because of the wide range of design options available to the Design Engineer. Certain planning and conceptual design criteria are particularly useful in the preliminary design of a channel. These criteria, which have the greatest effect on the performance and cost of the channel, are discussed below.

5.4.1 Natural Channels (Major Drainage Ways)

The design criteria and evaluation techniques for natural channels are:

1. The channel and over bank areas shall have adequate capacity for the 100-year storm runoff.
2. Natural channel segments, which have a calculated Froude Number greater than 0.8 for the 100-year flood peak shall be protected from erosion.
3. The water surface profile shall be defined so that the floodplain can be zoned and protected.
4. Roughness factors (Manning's n), which are representative of unmaintained channel conditions, shall be used for the analysis of water surface profiles.
5. Roughness factors (Manning's n), which are representative of maintained channel conditions, shall be used to determine velocity limitations.
6. Erosion control structures, such as drop structures or check dams, may be required to control flow velocities, including the initial storm runoff.
7. If a natural channel is to be utilized as a major drainage way for a developed area, then the applicant shall meet with the Town to discuss the concept and to obtain the requirements for planning and design documentation including the completion of a detailed channel stability analysis.

8. Plan and profile drawings of the floodplain shall be prepared. Appropriate allowances for known future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis. The applicant shall contact the Town for information on future bridges, culverts or other planned improvements.

With most natural waterways, erosion control structures should be constructed at regular intervals to decrease the thalweg (flowline) slope and to control erosion. However, these channels should be left in as near a natural condition as possible. For that reason, extensive modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent deposition downstream.

The usual rules of freeboard depth, curvature, and other rules, which are applicable to artificial channels, do not apply for natural channels. All structures constructed along the channel shall be elevated a minimum of one foot above the 100-year water surface level. There are significant advantages, which may occur if the Designer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas which are laid out and developed for the purpose of being inundated during the major runoff peak.

5.4.2 Grass Lined Channels (Major Drainage Way)

Key parameters in grass lined channel design include velocity, slope, roughness coefficients, depth, freeboard, curvature, crosssection shape, and lining materials. Other factors such as water surface profile computation, erosion control, drop structures, and transitions also play an important role. A discussion of these parameters is presented below.

1. Flow Velocity (Major Drainage Way)

   The maximum normal depth velocity for the 100-year flood peak shall not exceed 7.0 feet per second (fps) for grass-lined channels, except in sandy soil, where the maximum velocity shall not exceed 5.0 fps. The Froude Number shall be less than 0.8 for grass-lined channels. The minimum velocity, wherever possible, shall be greater than 2.0 fps for the initial storm runoff.

2. Longitudinal Channel Slope (Major Drainage Way)

   Grass-lined channel slopes are dictated by velocity and Froude Number requirements. Where the natural topography is steeper than desirable, drop structures shall be utilized to maintain design velocities and Froude Numbers.

3. Freeboard (Major Drainage Way)

Except where localized overflow in certain areas is desirable for additional ponding benefits or other reasons, the freeboard for the 100-year flow shall be as follows. The minimum freeboard shall be 1.0 foot.

$$H_{FB} = 2.0 + 0.025V(yo)^{1/3} + \Delta y \qquad \text{(Equation 5.2)}$$

Where: H_{FB} = freeboard height (feet)
V = average channel velocity (fps)
yo = depth of flow (feet)
Δy = increase in water surface elevation due to super elevation at bends (see equation MD-9) (no bends allowed in supercritical channels)

4. Horizontal Curvature (Major Drainage Way)

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

5. Roughness Coefficient (Major Drainage Way)

The variation of Manning's "n" with the retardance and the product of mean velocity and hydraulic radius, as presented in Figure 5-1, shall be used in the capacity computation.

Retardance curve C shall be used to determine the channel capacity, since a mature channel (substantial vegetation with minimal maintenance) will have a higher Manning's "n" value. However, a recently constructed channel will have minimal vegetation and the retardance will be less than the mature channel. Therefore, retardance curve D shall be used to determine the limiting velocity in a channel.

For the purpose of floodplain definition, only the higher Manning's "n" values need to be considered in the hydraulic analysis.

6. Cross Sections (Major Drainage Way)

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational functions. Representative cross sections are presented in Figures 5-2, 5-3, and 5-4. The limitations within which the design must fall for the major storm design flow include:

a. Trickle Channel - The base flow shall be carried in a trickle channel except for sandy soils. The minimum capacity shall be 2.0 percent to 4.0 percent of the 100-year flow but not less than 1 cfs. Trickle channels shall be grass-lined with a perforated under drain for water quality enhancement wherever practical. Where the water quality trickle channel is impractical, construct the channel with concrete or other approved materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils. Recommended trickle channel sections are presented in Figure 5-5. The minimum trickle channel width shall be three feet.

An alternative configuration for a trickle channel may consist of a subsurface storm drainpipe. If used, this alternative would consist of a minimum 24" diameter pipe, provided with access manholes, and sloped to maintain a minimum pipe flow velocity of 3 feet per second at onehalf of full pipe depth.

b. Main Channel - Representative main channels, and limitations for these channels, are shown in Figures 5-2, 5-3, and 5.4. Figure 5.4 indicates channel configurations for sandy soils.

c. Bottom Width - The minimum bottom width shall be consistent with the maximum depth and velocity criteria. The minimum width shall be four (4) feet or the trickle channel width, where required.

d. Right-Of-Way Width - The minimum ROW width shall include freeboard and a twelve (12) foot wide maintenance access. In some situations, the Town may require a twelve (12) maintenance access on both sides of the channel.

e. Flow Depth - The maximum design depth of flow (outside the trickle channel area and main channel area for sandy soils) for the 100-year flood shall be limited to five (5) feet in grass-lined channels.

f. Maintenance/Access Road - Continuous maintenance access shall be provided for all major drainage ways at a minimum width of twelve (12) feet. The Town may require six (6) inches of Class 2 road base or a concrete slab.

g. Side Slopes - Side slopes shall be 4H (horizontal) to 1V (vertical) or flatter. Slopes as steep as 3H:1V may be used in existing developed areas where ROW availability is restricted, provided prior written approval is obtained from the Town.

7. Grass lining (Major Drainage Way)

The grass lining for channels shall be in accordance with the USDCM, Section 4.1, "Major Drainage - Grass lined Channels - Grass."

8. Erosion Control (Major Drainage Way)

The requirements for erosion control for grass-lined channels shall be as defined in the USDCM, Section 2.3.6, "Major Drainage Erosion Control." The design of riprap and erosion control devices shall be in accordance with Sections 10.1 and 10.2 of these Criteria.

9. Water Surface Profiles (Major Drainage Way)

Computation of the water surface profile shall be presented for all open channels utilizing standard backwater methods, taking into consideration losses due to changes in velocity or channel cross section, drops, waterway openings, or obstructions. The energy gradient shall be shown on all design drawings.

5.4.3 Concrete Lined Channels (Major Drainage Way)

The criteria for the design and construction of concrete lined channels is presented below:

1. Hydraulics (Major Drainage Way)

a. Freeboard - Adequate channel freeboard above the designed water surface shall be provided and shall not be less than determined by the following:

$$H_{FB} = 2.0 + 0.025\,V\,(d)^{1/3} \qquad \text{(Equation 5.3)}$$

Where: H_{FB} = freeboard height (feet)
V = velocity (fps)
d = depth of flow (feet)

Freeboard shall be in addition to super-elevation, standing waves, and/or other water surface disturbances. These special situations should be addressed in the Final Drainage Report.

Concrete side slopes shall be extended to include the freeboard height.

b. Super-elevation - Super-elevation of the water surface shall be determined at all horizontal curves, and design of the channel section adjusted accordingly.

c. Velocities - Flow velocities shall be such that critical or super critical flow conditions are not created. In no case shall the velocity exceed 18 fps.

d. Critical or super critical flow conditions are not allowed. Drop structures or other appropriate energy dissipation facilities may be required to maintain a sub critical flow regime.

2. Concrete Materials (Major Drainage Way)

   a. Cement type: Sulphate resistant (Type V).

   b. Minimum cement content: 550 lbs/CY

   c. Maximum water content: 0.50 (six gallons per sack)

   d. Maximum aggregate size: 1½ inch

   e. Air content range: 4% to 7%

   f. Slump: 2 to 4 inches

   g. Minimum compressive strength (f'_c): 3,750 psi @ 28 days

3. Concrete Lining Section (Major Drainage Way)

   a. All concrete lining shall have a minimum thickness of seven (7) inches.

   b. The side slopes shall be a maximum of 2 vertical to 1 horizontal, or be designed as a structurally reinforced retaining wall, if steeper.

4. Concrete Joints (Major Drainage Way)

   a. Concrete channels shall be continuously reinforced and contain transverse joints. Expansion joints shall be installed where new concrete lining is connected to a rigid structure or to an existing concrete lining which is not continuously reinforced.

   b. Longitudinal joints, where required, shall be constructed on the sidewalls at least one foot vertically above the channel invert.

   c. All joints shall be designed to prevent differential movement.

   d. Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint.

5. Concrete Finish (Major Drainage Way)

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

6. Concrete Curing (Major Drainage Way)

All concrete shall be cured by the application of a liquid membrane forming curing compound (white pigmented) upon completion of the concrete finish.

7. Reinforcement Steel (Major Drainage Way)

a. Steel reinforcement shall be minimum grade 40 deformed bars. Fabric mesh may also be approved. Wire mesh shall not be used.

b. Ratio of longitudinal steel area to concrete cross sectional area shall be greater than 0.005.

c. Ratio of transverse steel area to concrete cross sectional area shall be greater than 0.0025.

d. Reinforcing steel shall be placed at the center of the section with a minimum clear cover of three inches adjacent to the earth.

e. Additional steel shall be placed as needed if a retaining wall structure is used.

8. Earthwork (Major Drainage Way)

The following areas shall be compacted to at least 95% of maximum density as determined by ASTM D-698 (Standard Proctor within 2% of the optimum moisture content):

a. The 12 inches of sub grade immediately beneath concrete lining (both channel bottom and side slopes).
b. Top 24 inches of maintenance road.
c. Top 12 inches of earth surface within 10 feet of concrete channel lip.
d. All fill material.

9. Bedding (Major Drainage Way)

Provide six inches of granular bedding equivalent in gradation to ¾" concrete aggregate, No. 67 (Standard Specifications for Road & Bridge Construction, CDOT, latest revision) under channel bottom and side slopes.

10. Under Drain (Major Drainage Way)

Longitudinal under drains shall be provided on 10-foot centers and shall daylight at the check drops. Weep holes shall be provided in vertical wall sections of the channel.

11. Safety Requirements (Major Drainage Way)

    a. A six-foot high vinyl coated chain link or comparable fence shall be installed to prevent access wherever the 100-year channel flow depths exceed three feet. Gates, with top latch, shall be placed at 250-foot intervals and staggered where fence is required on both sides of the channel.

    b. Ladder-type steps shall be installed not more than 400 feet apart on alternating sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above the channel invert.

12. Maintenance Access Road

    A maintenance access road shall be provided along the entire length of all major drainage ways with a minimum passage width of 12 feet. In some situations the Town may require maintenance access on both sides of the channel. The Town shall require the road to be surfaced with six inches of Class 2 road base or concrete slab.

5.4.4 Riprap Channel Linings (Major Drainage Way)

The criteria for the design and construction of riprap channel linings shall be in accordance with the USDCM, Volume 2, "Major Drainage - Riprap." Riprap lined channels shall be designed for a turbulence factor (Froude Number) less than 0.8 for the 100-year flood peaks.

The riprap shall be designed and constructed in accordance with Section 10.2 of these Criteria. Freeboard and maintenance access road requirements shall be in accordance with the standards for grass-lined channels as defined in Section 5.4.2 of these Criteria.

5.4.5 Other Lining Types (Major Drainage Way)

The criteria for the design of major drainage way channels with linings other than grass, rock, or concrete will be dependent on the manufacturer's recommendations for the specific product. The applicant will be required to submit the technical data in support of the proposed material. Additional information or calculations may be requested by the Town to verify assumptions or design criteria. The following minimum criteria will also apply.

1. Flow Velocity (Major Drainage Way)

   The maximum normal depth velocity will be dependent on the construction material used; however, the Froude Number shall be less than 0.8.

2. Freeboard (Major Drainage Way)

   Same as for grass lined channels (see Section 5.4.2).

3. Curvature (Major Drainage Way)

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

4. Roughness Coefficient (Major Drainage Way)

   A Manning's "n" value range shall be established by the manufacturer's data, with the high value used to determine depth/capacity requirements and the low value used to determine the Froude Number and velocity restrictions.

5. Cross Sections (Major Drainage Way)

   Same as for grass lined channels (see Section 5.4.2).

5.5 Design Standards for Small Drainage Ways

These standards cover the design of channels that are not classified as a major drainage way. Additional flexibility and less stringent standards are allowed for small drainage ways.

5.5.1 Natural Channels (Small Drainage Way)

The design criteria and evaluation techniques for natural channels are:

1. The channel and over bank areas shall have adequate capacity for the 100-year storm runoff.
2. Natural channel segments, which have a calculated Froude Number greater than 0.8 for the 100-year flood peak shall be protected from erosion.
3. Roughness factors (Manning's n), which are representative of unmaintained channel conditions, shall be used for the analysis of water surface profiles.
4. Roughness factors (Manning's n), which are representative of maintained channel conditions, shall be used to determine velocity limitations.
5. Erosion control structures, such as check drops or check dams, may be required to control flow velocities, including the initial storm runoff.

6. A channel stability analysis will be completed to determine the impact of urbanization on the stability of the channel bed and banks.

7. Plan and profile drawings shall be prepared showing the 100-year water surface profile, floodplain, and details of erosion protection, if required.

5.5.2 Grass Lined Channels (Small Drainage Way)

Key parameters in grass lined channel design include velocity, slope, roughness coefficients, depth, freeboard, curvature, cross section shape, and lining materials. Other factors such as water surface profile computation, erosion control, drop structures, and transitions also play an important role. A discussion of these parameters is presented below.

1. Flow Velocity (Small Drainage Way)

   The maximum normal depth velocity for the 100-year flood peak shall not exceed 7.0 feet per second (fps) for grass-lined channels, except in sandy soil, where the maximum velocity shall not exceed 5.0 fps. The Froude Number shall be less than 0.8 for grass-lined channels. The minimum velocity, wherever possible, shall be greater than 2.0 fps for the initial storm runoff.

2. Longitudinal Channel Slope (Small Drainage Way)

   Grass-lined channel slopes are dictated by velocity and Froude Number requirements. Where the natural topography is steeper than desirable, drop structures shall be utilized to maintain design velocities and Froude Numbers.

3. Freeboard (Small Drainage Way)

   A minimum freeboard of 1 foot shall be included in the design of the 100-year flow. For swales (i.e. small drainage ways with a 100-year flow less than 20 cfs), the minimum freeboard requirement is 6 inches.

4. Horizontal Curvature (Small Drainage Way)

   The centerline curvature shall have a minimum radius twice the top width of the design flow but not less than 50 feet. The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

5. Roughness Coefficient (Small Drainage Way)

   The variation of Manning's "n" with the retardance and the product of mean velocity and hydraulic radius, as presented in Figure 5-1, shall be used in the computation of capacity and velocity.

6. Cross Sections (Small Drainage Way)

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational functions. Representative cross sections are presented in Figures 5-2, 5-3, and 5-4. The limitations on the cross sections are as follows:

a. Trickle Channel - The base flow (except for swales) shall be carried in a trickle channel except for sandy soils. The minimum capacity shall be 1.0 percent to 3.0 percent of the 100-year flow but not less than 1 cfs. Trickle channels shall be constructed of concrete or other approved materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils. For sandy soils, a main channel is required in accordance with Section 5.4.2.6(b). For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property.

b. Right-Of-Way Width - The minimum ROW width shall include freeboard and a 12-foot wide maintenance access.

c. Flow Depth - The maximum design depth of flow (outside the trickle channel area and main channel area for sandy soils) for the 100-year flood shall be limited to 5.0 feet in grass-lined channels.

d. Side Slopes - Side slopes shall be 4 (horizontal) to 1 (vertical) or flatter. Side slopes for channels with 100-year runoff of 20 cfs or less shall be 3H:1V or flatter.

7. Grass Lining (Small Drainage Way)

The grass lining for channels shall be in accordance with the USDCM, Section 2.3.2, "Major Drainage - Grass lined Channels - Grass."

8. Erosion Control (Small Drainage Way) The requirements for erosion control for grass-lined channels shall be as defined in the USDCM, Volume 1, Section 4.1.6. The design of riprap and erosion control devices shall be in accordance with Section 4.4 of these Criteria.

9. Hydraulic Information (Small Drainage Way)

Calculations of the capacity, velocity, and Froude Numbers shall be submitted with the Final Drainage Report.

5.5.3 Concrete Lined Channels (Small Drainage Way)

The criteria for the design and construction of concrete lined channels is presented below:

1. Hydraulics (Small Drainage Way)

   a. Freeboard - Adequate channel freeboard above the designed water surface shall be provided and shall not be less than determined by Equation 5.3. Freeboard shall be in addition to super elevation, standing waves, and/or other water surface disturbances. These special situations should be addressed in the Final Drainage Report.

      Concrete side slopes shall be extended to include the freeboard height.

   b. Super-elevation - Super-elevation of the water surface shall be determined at all horizontal curves, and design of the channel section adjusted accordingly.

   c. Velocities - Flow velocities shall be such that critical or super critical flow conditions are not created. In no case shall the velocity exceed 18 fps.

2. Concrete Materials (Small Drainage Way)

   a. Cement type: Sulphate resistant.

   b. Minimum cement content: 550 lbs/CY

   c. Maximum water content: 0.50 (six gallons per sack)

   d. Maximum aggregate size: 3/4 inch

   e. Air content range: 4% to 7%

   f. Slump: 2 to 4 inches

   g. Minimum compressive strength (f'_c): 3,750 psi @ 28 days

3. Concrete Lining Section (Small Drainage Way)

   a. All concrete lining shall have a sufficient thickness to withstand the structural and hydraulic loads.

   b. The side slopes shall be a maximum of 2 vertical to 1 horizontal, or be designed as a structurally reinforced retaining wall, if steeper.

4. Concrete Joints (Small Drainage Way)

   a. Expansion joints shall be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.

   b. Longitudinal joints, where required, shall be constructed on the sidewalls at least one foot vertically above the channel invert.

   c. All joints shall be designed to prevent differential movement.

   d. Construction joints are required for all cold joints and where the lining thickness changes.

5. Concrete Finish (Small Drainage Way)

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

6. Concrete Curing (Small Drainage Way)

   Concrete shall be cured by the application of a liquid membrane forming curing compound (white pigmented) upon completion of the concrete finish. Curing compound shall not be placed on the face of joints where new concrete will adjoin.

7. Reinforcement Steel (Small Drainage Way, where appropriate)

   a. Steel reinforcement shall be minimum grade 40 deformed bars. Fabric mesh may also be approved. Wire mesh shall not be used.

   b. Ratio of longitudinal steel area to concrete cross sectional area shall be greater than 0.005.

   c. Ratio of transverse steel area to concrete cross sectional area shall be greater than 0.0025.

   d. Additional steel shall be placed as needed if a retaining wall structure is used.

8. Earthwork (Small Drainage Way)

   The following areas shall be compacted to at least 95% of maximum density as determined by ASTM D-698 (Standard Proctor within 2% of the optimum moisture content):

a. The 12 inches of sub grade immediately beneath concrete lining (both channel bottom and side slopes).

b. Top 24 inches of maintenance road.

c. Top 12 inches of earth surface within 10 feet of concrete channel lip.

d. All fill material.

9. Bedding (Small Drainage Way)

Provide six inches of granular bedding equivalent in gradation to ¾-inch concrete aggregate (Standard Specifications for Road & Bridge Construction, CDOT, latest revision) under channel bottom and side slopes.

10. Under Drain (Small Drainage Way)

Longitudinal under drains shall be provided and shall daylight at the check drops. Weep holes shall be provided in vertical wall sections of the channel.

11. Safety Requirements (Small Drainage Way)

a. A six-foot high vinyl coated chain link or comparable fence shall be installed to prevent access wherever the 100-year channel flow depths exceed three feet. Gates, with top latch, shall be placed at 250-foot intervals and staggered where fence is required on both sides of the channel.

b. Ladder-type steps shall be installed not more than 400 feet apart on alternating sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above the channel invert.

5.5.4 Riprap Channel Linings (Small Drainage Way)

The criteria for the design and construction of riprap channel linings shall be in accordance with the USDCM, Volume 2, "Major Drainage - Riprap." Riprap lined channel shall be designed for a turbulence factor (Froude Number) less than 0.8 for the 100-year flood peaks. The riprap shall be designed and constructed in accordance with Section 10.2 of these Criteria. Freeboard requirements shall be in accordance with the standards for grasslined channels (see Section 5.5.2).

5.5.5 Other Lining Types (Small Drainage Way)

The criteria for the design of small drainage way channels with linings other than grass, rock, or concrete will be dependent on the manufacturer's recommendations for the specific product. The applicant will be required to submit the technical data in support of the proposed material. Additional information or calculations may be requested by the Town to verify assumptions or design criteria. The following minimum criteria will also apply.

1. Flow Velocity (Small Drainage Way)

   The maximum normal depth velocity will be dependent on the construction material used; however, the Froude Number shall be less than 0.8.

2. Freeboard (Small Drainage Way)

   Same as for grass lined channels (see Section 5.5.2).

3. Curvature (Small Drainage Way)

   The centerline curvature shall have a minimum radius twice the top width of the design flow but not less than 50 feet. The minimum radius of curvature for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

4. Roughness Coefficient (Small Drainage Way)

   A Manning's "n" value range shall be established by the manufacturer's data, with the high value used to determine depth/capacity requirements and the low value used to determine the Froude Number and velocity restrictions.

5. Cross Sections (Small Drainage Way)

   Same as for grass lined channels (see Section 5.5.2).

5.6 ROADSIDE DITCHES

The criteria for the design of roadside ditches are similar to the criteria for grass-lined channels with modification for the special purpose of initial storm drainage. The criteria is as follows (refer to Figure 5-6):

1. Capcity - Roadside ditches shall have adequate capacity for the initial storm runoff peaks. During the initial storm runoff event, encroachment shall not extend beyond the street right-of-way. Where the storm runoff exceeds the capacity of the ditch, a storm sewer system shall be required.

2. Flow Velocity - The maximum velocity for the initial storm flood peak shall not exceed 5 feet per second for a type I ditch, and 7 fps for type II or type III ditches.

3. Longitudinal Slope - The slope shall be limited by the average velocity of the initial storm flood peaks. Check drops may be required where street slopes are in excess of 2%. Maximum permissible slope is 3%.

4. Freeboard - Freeboard shall be equal to the velocity head, or a minimum of six inches.

5. Curvature - The minimum radius of curvature shall be 25 feet.

6. Roughness Coefficient - Manning's "n" values presented in Figure 5-1 shall be used in the capacity computation for roadside ditches.

7. Grass Lining - The grass lining shall be in accordance with USDCM, Section 2.3.2, "Major Drainage - Grass Lined Channels - Grass."

8. Driveway Culverts - Driveway culverts shall be sized to pass the initial storm ditch flow capacity without overtopping the driveway. The minimum size culvert shall be a 12-inch diameter pipe (or equivalent) with flared end sections. More than one culvert may be required.

9. Major Drainage Capacity - The capacity of roadside ditches for major drainage flow is restricted by the maximum flow depth allowed at the street crown (see Section 8). However, the flow spread should not inundate the ground line of residential dwellings, or public, commercial, or industrial buildings.

5.7 CHANNEL RUNDOWNS

A channel rundown is used to convey storm runoff from the bank or side-slope of a channel or detention pond to the channel invert or to the bottom of a detention pond. The purpose of the structure is to minimize channel bank erosion from concentrated overland flow. The design criteria for channel rundowns is as follows:

1. Cross Sections - Typical cross sections for channel rundowns are presented in Figure

2. Design Flow - The channel rundown shall be designed to carry a minimum flow of a one hundred year frequency storm.

3. Flow Depth - The maximum depth at the design flow shall be 12 inches. Due to the typical profile of a channel rundown beginning with a flat slope and then dropping steeply into the channel, the design depth of flow shall be the computed critical depth for the design flow.

4. Outlet Configurations Into Channels - The channel rundown outlet shall enter the drainage-way at the trickle channel flow line. Erosion protection of the opposite

channel bank shall be provided by a 24-inch layer of grouted Type L riprap. The width of this riprap erosion protection shall be at least three times the channel rundown width or pipe diameter. Riprap protection shall extend up the opposite bank to the initial storm flow depth in the drainage way or 2 feet, whichever is greater. Riprap shall only be used with bedding or fabric on slopes less than or equal to 3:1.

5. Outlet Configurations Into Detention Ponds - the channel rundown outlet shall be constructed with a flared end section, a concrete cutoff wall and an adequately reinforced splash pad at the bottom of the rundown. Cut off walls shall be concrete, eight inches (8") thick, four-feet (4') deep and eight feet (8') to sixteen-feet (16') wide. Reinforce the cut off wall with #4 rebar at twelve inches (12") apart each way. Flared end sections shall be at least twice the width of the channel and the cut off wall twice the width of the flared end section.

   The splash pad may be constructed with eight-inch (8") thick concrete; a twenty-four (24") layer of grouted Type L riprap or may be bluegrass with Turf Reinforcement Mat (TRM). All components must be sized for a 100-year frequency storm. Consult the latest edition of the Urban Storm Drainage Design Criteria Manual (USDCM) Volume 2 for guidance in designing various components of the rundown.

6. Turf Reinforcement Mat (TRM) on soil with native grass is not acceptable for channel rundowns. Only TRM with irrigated bluegrass is allowed in conjunction with a cut off wall and TRM at the base of the slope, which will allow the flow to dissipate in different directions.

7. If concrete V-pan rundowns sized for 5-year frequency storm are used they must be constructed with an eighteen-inch (18") layer of grouted Type L riprap on both sides to accommodate for a 100-year frequency storm. This type of rundown must have a FES, an adequate concrete cutoff wall at the FES and an adequately reinforced splash pad at the bottom of the rundown.

8. Rundowns may be constructed with a 24-inch layer of Type L riprap, on bedding, on fabric that is completely grouted and sized for a 100-year frequency storm. A concrete cutoff wall must be provided with an adequate splash pad at the bottom.

5.8 MAINTENANCE AND ACCESS EASEMENTS

An important aspect of storm drainage facilities is continued maintenance. Maintenance of storm drainage channels and structures may include periodic removal of sediment and debris; repair of channel erosion; and repair of inlet, outlet, and drop structures. The Town requires the following minimum right-of-way or easement widths, which must be shown on the Final Plat:

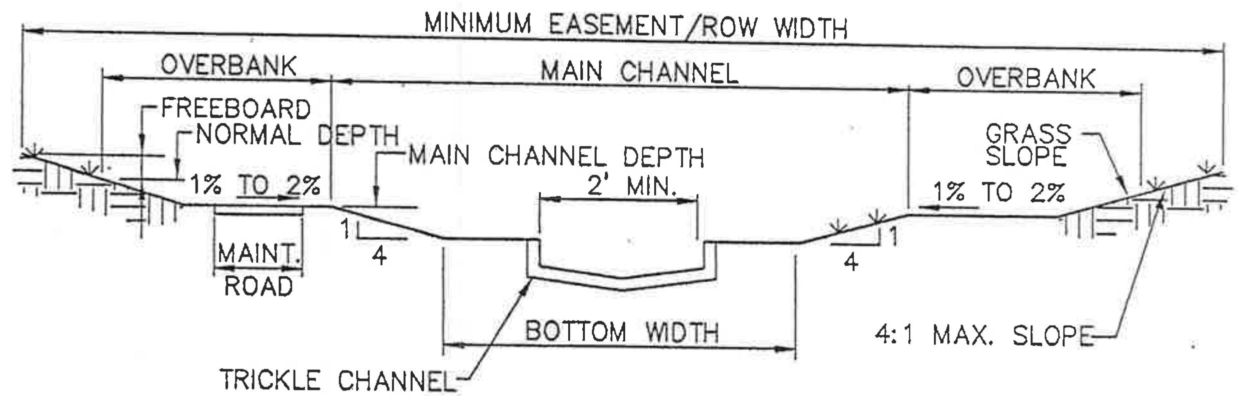
Minimum Channel Easements Widths	
Channel Size	Total R.O.W. or Easement Width
Q_{100} less than 20 cfs	15 feet
Q_{100} less than 100 cfs	25 feet
Q_{100} greater than 100 cfs	Minimum width calculated to include freeboard plus 12-foot wide access road. The Town shall determine if access is required on both sides of channel.

5.9 CHECKLIST

To aid the designer and reviewer, the following checklist has been prepared.

1. Check flow velocity with the low retardance factor and capacity with the high retardance factor.
2. Check the Froude Number and critical flow conditions.
3. Grass channel side slopes must be 4H:1V or flatter, unless approved by the Town.
4. Show the energy grade line and the hydraulic grade line on the design drawings.
5. Consider all backwater conditions at culverts when determining channel capacity.
6. Check the flow velocity for flood conditions without backwater effects.
7. Provide adequate freeboard.
8. Provide adequate right-of-way for the channel and continuous maintenance access.

TYPE B



NOTES:

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 MATERIAL. SEE FIGURE 5–4 FOR REQUIREMENTS IN SANDY SOILS.
3. NORMAL DEPTH: SHALL INCLUDE MAIN CHANNEL DEPTH. FLOW DEPTH FOR 100–YEAR SHALL NOT EXCEED 5 FEET.
4. FREEBOARD: FREEBOARD TO BE A MINIMUM OF 1 FOOT.
5. MAINTENANCE ACCESS ROAD: MINIMUM WIDTH TO BE 12 FEET. **TOWN** 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.

TOWN OF KERSEY

TYPICAL GRASS LINED CHANNEL SECTION

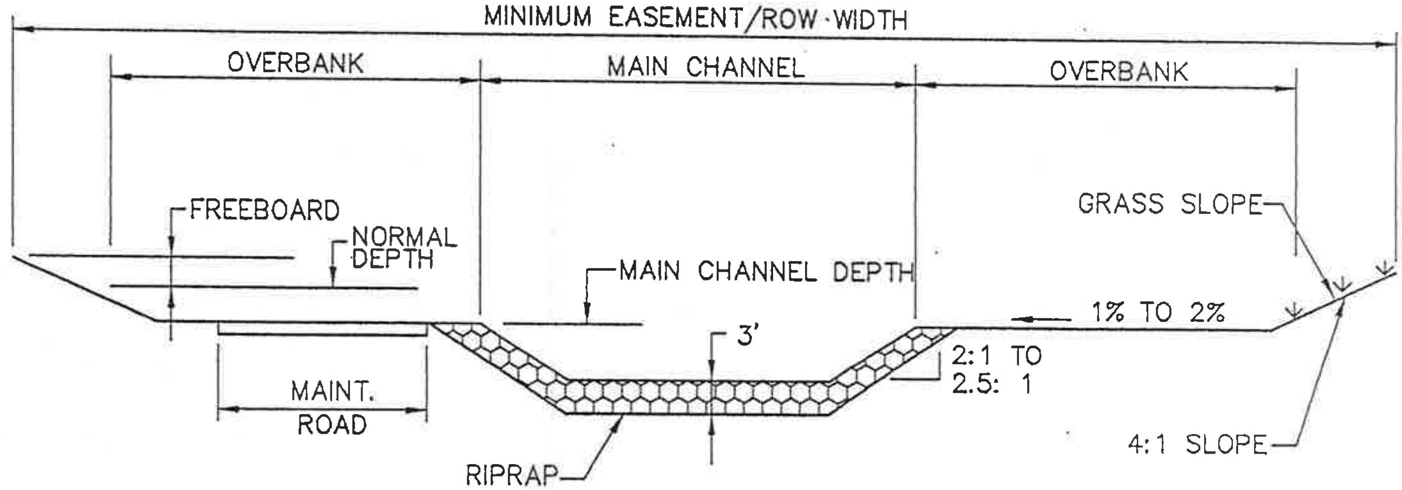
SCALE: NONE

DATE: 12/04

APPROVED: FJB

TOWN ENGINEER

TYPE C



NOTES:

1. THIS SECTION IS REQUIRED FOR CHANNELS IN SANDY SOILS.
2. MAIN CHANNEL: CAPACITY TO BE THE EQUIVALENT OF THE INITIAL STORM RUNOFF. MAXIMUM 100–YEAR FLOW VELOCITY IS 5 FPS. PROTECT SLOPES WITH RIPRAP. USE A MANNINGS N–VALUE OF 0.03 FOR HYDRAULIC CALCULATIONS.
3. NORMAL DEPTH: FLOW DEPTH FOR 100–YEAR FLOW SHALL NOT EXCEED 5 FEET, NOT INCLUDING THE MAIN CHANNEL DEPTH.
4. FREEBOARD: FREEBOARD TO BE A MINIMUM OF 1 FOOT.
5. MAINTENANCE ACCESS ROAD: MINIMUM WIDTH TO BE 12 FEET. **TOWN** MAY REQUIRE ALL OR PART OF THE ROAD TO BE SURFACED.
6. 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.

TOWN OF KERSEY

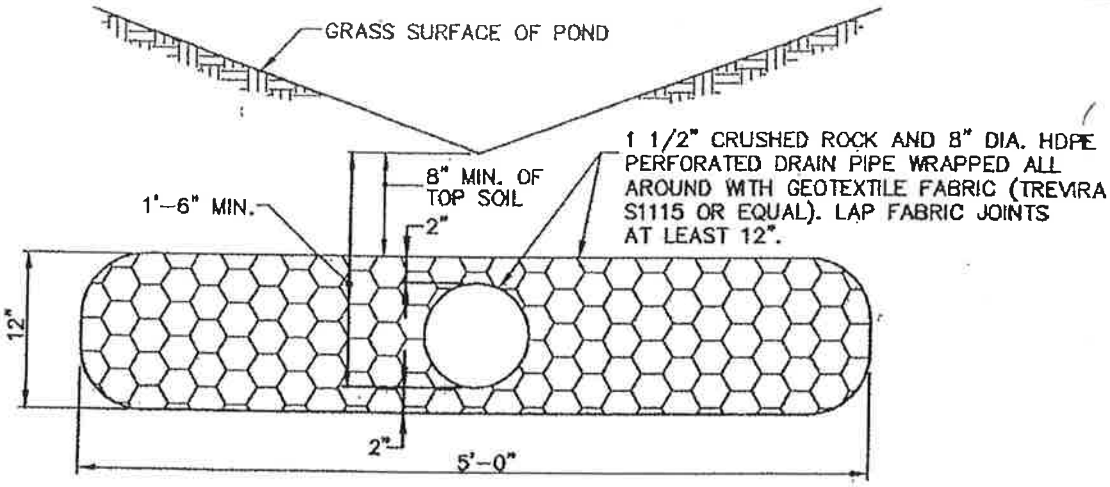
TYPICAL GRASS LINED CHANNEL SECTION FOR SANDY SOILS

SCALE: NONE

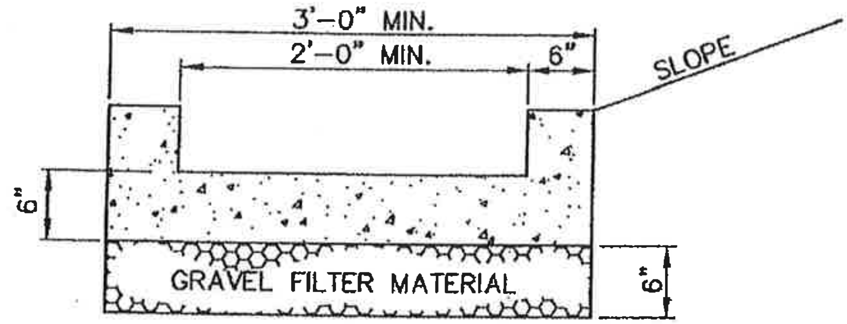
DATE: 12/04

APPROVED: FJB

TOWN ENGINEER



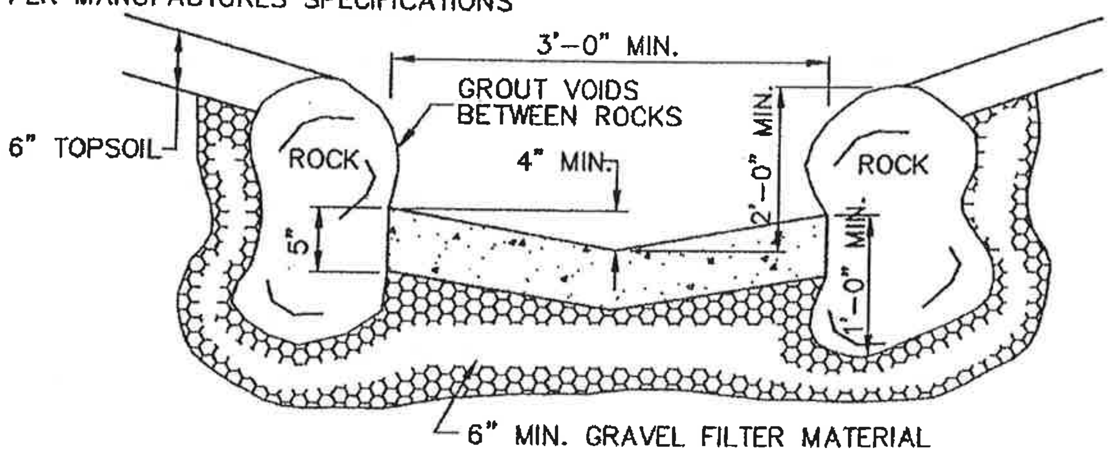
WATER QUALITY CHANNEL SECTION



RECTANGULAR CHANNEL SECTION

NOTES:

CONCRETE TO BE REINFORCED WITH FIBERMESH PER MANUFACTURES SPECIFICATIONS



COMBINATION CHANNEL SECTION

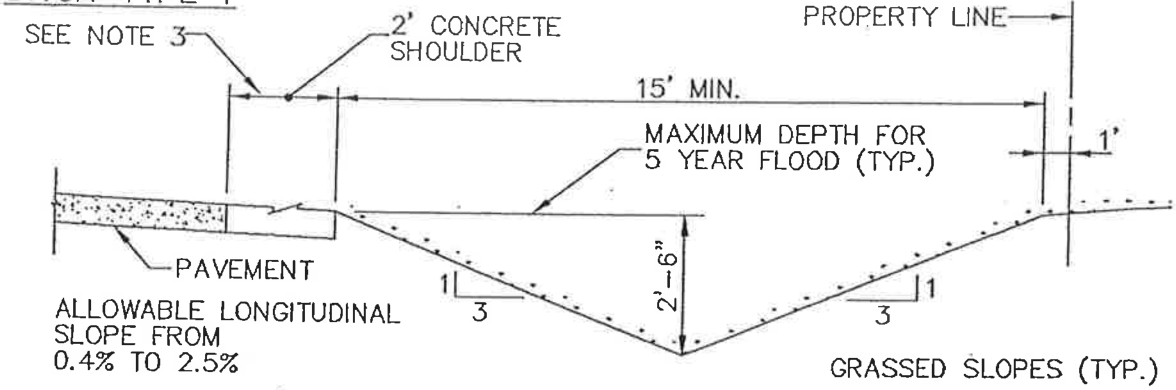
TOWN OF KERSEY

TRICKLE CHANNEL DETAILS

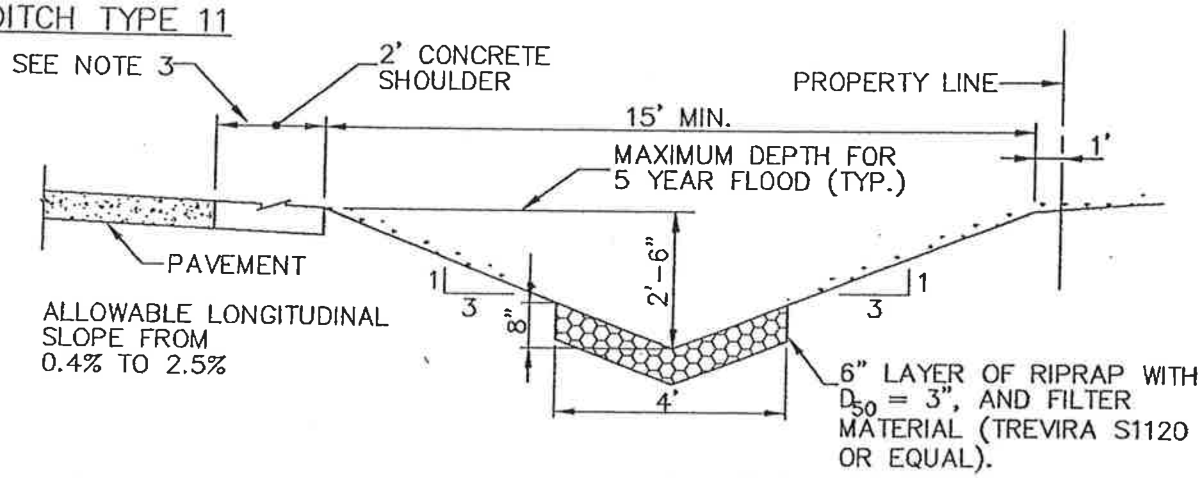
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG 5-5

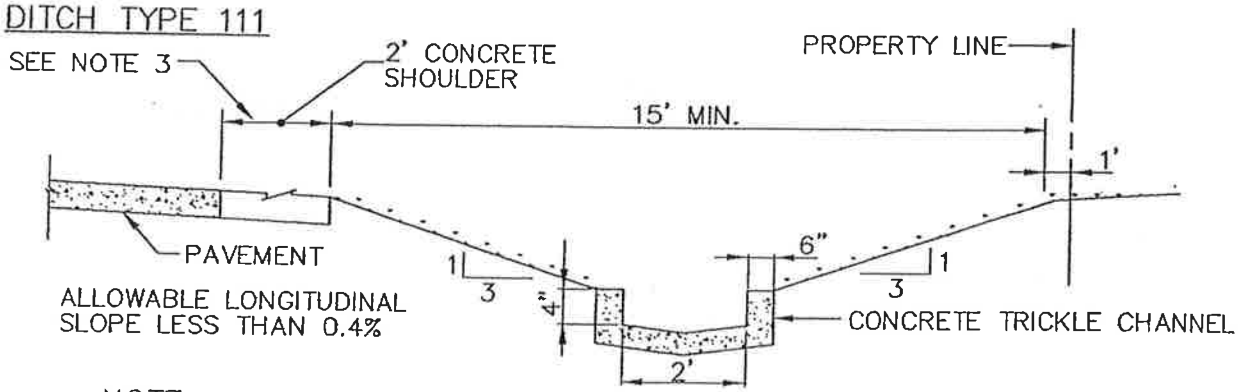
DITCH TYPE 1



DITCH TYPE 11



DITCH TYPE 111

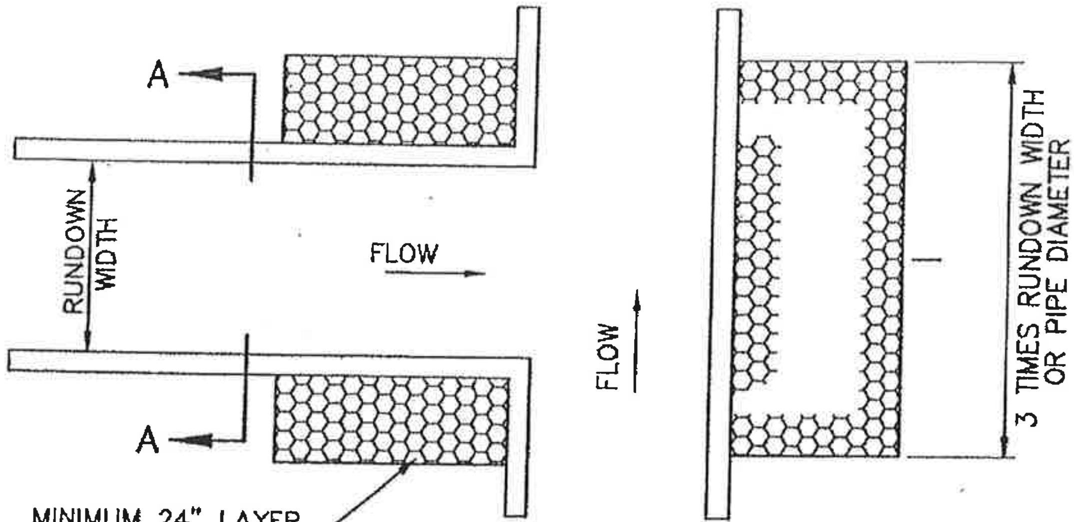


NOTE:

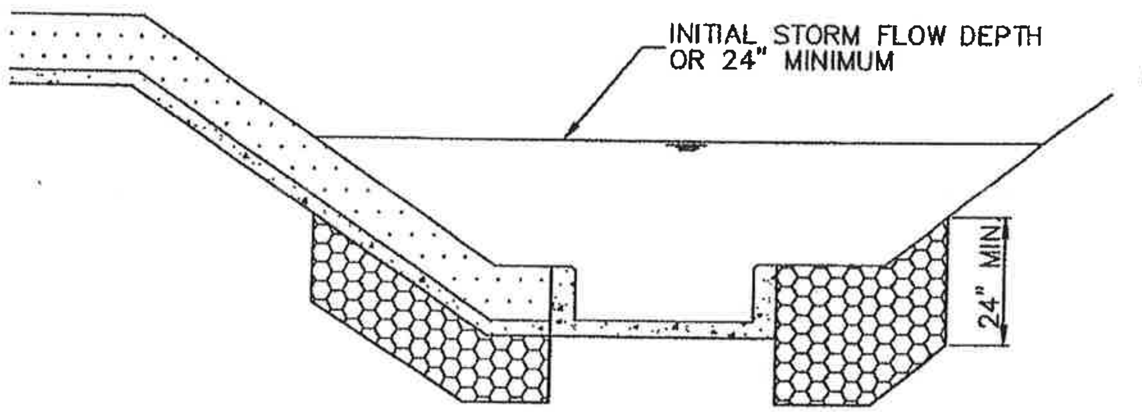
1. FOR STREET SLOPES GREATER THAN MAXIMUM ALLOWABLE, CHECK DROPS (2' MAXIMUM HEIGHT) WILL BE REQUIRED.
2. STREET CROSS SECTION MAY INCLUDE CONCRETE CURB AND GUTTER.
3. PROVIDE 2' WIDE 8" THICK CONCRETE CURB, SEE STREET STANARDS DETAIL NO. S-1.

TOWN OF KERSEY	
STORM DRAINAGE CRITERIA ROADSIDE DITCH SECTIONS	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

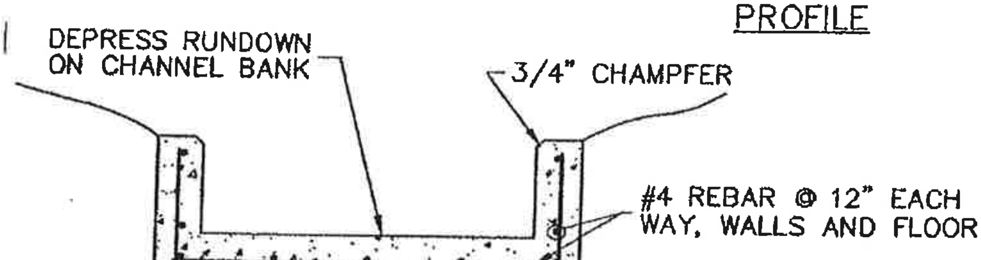
SHEET TABLE 5-6



PLAN



PROFILE



SECTION A–A

TOWN OF KERSEY

CHANNEL RUNDOWN

SCALE: NONE

DATE: 12/04

APPROVED: FJB

TOWN ENGINEER

SHEET FIG. 5–7

SECTION 6.0: STORM SEWERS

6.1 INTRODUCTION

Storm sewers are required when the other parts of the drainage system, primarily curb, gutter, and roadside ditches no longer have capacity for the additional stormwater runoff.

Except as modified herein, the design of storm sewers shall be in accordance with the USDCM section on "Storm Sewers". Reference is made to follow specific sections in the USDCM for clarity. The user is referred to the USDCM and any other references cited for additional discussion and basic design concepts.

6.2 CONSTRUCTION MATERIALS/INSTALLATION OF STORM SEWERS

6.2.1 Construction Materials

Storm Sewer Piping Materials

All storm sewers within the Town shall be constructed using one of the following materials and meet applicable standards as presented below:

Storm Sewer Standards	
Pipe Material	Standard
Reinforced Concrete Pipe (RCP)	ASTM C-33, 76, 150, 260, 361, 443, 494 (Type A or D), 497, & 655, ASTM E 329 and AASHTO M 170, and 242 and FED Specifications (FS): SS-S-00210
Plastic Pipe (PVC) only on approval	AASHTO M304M-911, ATM D-1784, 2122, 2321, 2412, & 3212, and ASTM D3034 DR35 or better, and with elastomeric gaskets per ASTM F477, and ASTM F679, 794, 949, & 1803
Aluminized Steel Pipe (ASP)	AASHTO M-36, 198 & 274 and ASTM A-760, 796, 798 & 891
Corrugated Steel for Culverts Only	AASHTO M-36, 167, 190, 218, 243, 245, 246, 264, 289 and ASTM A-444, 742, 760, 761, 762, 806, 819, 849, 885 and D-1056
High Density Polyethylene Pipe (HDPE)	AASHTO M-252, 294 (Type S), and Section 18 with rubber water-tight joints and ASTM D-1056 (Grade 2A2), 1248, 2321, 3212 & 3350 (cell class 324420C or higher) and F477 & 677, provide a long term design strength specification.

Note: Elliptical and arched pipe should be used only when conditions prevent the use of circular pipe.

At the option of the Contractor, non-reinforced concrete pipe conforming to ASTM-C14 and AASHTO M 86 may be used in lieu of reinforced concrete pipe for all sizes 36 inches in diameter and smaller, provided it meets the same D-load to produce the ultimate load under the three-edge bearing method as specified for reinforced concrete pipe in accordance with AASHTO M170, and the Contractor provides written certification that it does so. Wall thickness of pipe may be increased as required to meet D-load requirement.

All requirements for reinforced concrete pipe, except those referring to reinforcement shall apply to non-reinforced concrete pipe.

The minimum class for RCP pipe shall be Class-III with flexible gasket material (water tight rubber gaskets) meeting ASTM C443 and gasket bell and spigot joints. The required pipe strength shall be determined from the actual depth of cover, true load, and proposed field conditions. A typical design strength calculation shall be submitted to the Town for approval.

ASP and HDPE pipe may be used for storm sewer in parks, green belts, and other open space areas only upon approval of the Town.

Where corrugated pipe (CMP) is intended to be used as culvert material, the minimum gage for the pipe shall be determined from Colorado Department of Transportation (CDOT) Standard Plan M-603-1, November 1, 1992, or latest edition for actual depth of cover.

Site specific calculations may be submitted utilizing the following structural design specifications: "AASHTO - Standard Specifications for Highway Bridges - Division O, Section 1;, ASTM – Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications, ASTM A796. The minimum gage for corrugated metal pipe buried beneath railroad tracks shall be determined from American Railroad Engineers Association Specification, AREA - Section 4.9." Also, when CMP is intended to be used as culvert material, site soil pH and Rmin, minimum resistivity, tests shall be performed. pH shall be measured for soil and water. Minimum resistivity shall be for soil and water and shall be determined in the laboratory and not in the field. Corrugated aluminum pipe may be used when the pH is between 7.2 and 9.0 and Rmin is 1,000 ohm-cm or greater.
Galvanized steel corrugated pipe may be used when the pH is between 6.0 and 10.0 and Rmin is 2000 ohm-cm or greater. Supplemental corrosion protection coatings approved by the Town shall be applied to corrugated steel pipe when the pH is outside the above-stated range and/or Rmin is less than 2000 ohm-cm.

Also, when ASP pipe is intended to be used as the storm sewer pipe material, site soil pH and Rmin, minimum resistivity, tests shall be performed. PH shall be measured for soil and water. Minimum resistivity shall be for soil and water and shall be determined in the laboratory and not in the field. ASP pipe may be used when the pH is between 7.2 and 9.0 and Rmin is 1,000 ohm-cm or greater.

Bedding Materials

RCP pipe and other rigid pipe systems are less susceptible to failure because of improper bedding and backfill procedures. Bedding for RCP pipe shall be 3/8-inch minus squeegee, six to eight inches deep under the pipe and backfilled at least to the spring line. The pipe embedment, including bedding, haunching, and initial backfill zones, is shown on the pipe bedding details presented in each of the pipe material (see Figure 6-6 & 6-7). Proper bedding provides uniform support under the pipe for rigid pipe materials, as well as structural integrity for flexible pipe materials.

Backfill of ASP, HDPE, CMP pipe or any flexible pipe is very critical. Non-structural (flexible) pipe materials shall include bedding requirements as specified in Urban Drainage and Flood Control District (UDFCD) Design Manual "March, 1998 update to storm sewer pipe technical memorandum", or latest edition, and storm sewer bedding details in this manual, Figure 6-6 & 6-7. The backfill is a major component to the structural integrity of the pipe system. Bedding materials used and compaction must have good quality control for a successful pipe installation. Bedding for flexible pipe shall be as follows:

The bedding material for a particular installation is an important factor for satisfactory long-term performance. For coarse-grained native trench material, the bedding should be selected to prevent migration of fines from the bedding into the native soil. When the native soil is fine-grained, the bedding material should be selected to prevent migration of native fines into the pipe-bedding zone, which could result in pavement failure outside of the trench.

Bedding and embedment material gradations for flexible pipe materials and native soils are presented in the following table:

Bedding & Embedment Material Guide for Flexible Pipe	
Native Soul	Embedment for Flexible Pipe
Course grained sand and gravel (50% or more by weight retained on #40 sieve)	CDOH Class A filter material (Section 703.09 or No. 67aggregate)
Fine grained soil (less than 50% by weight retained on #40 sieve)	Type 1 UD&FCD filter material or CDOH concrete sand AASHTOM6 (Section 703.01)

Embedment includes bedding below pipe as shown in pipe bedding details (Figure 6-6 & 6-7) extending to 1-foot above top of pipe and is required for all flexible pipe storm sewer installations.

Colorado Department of Highway (CDOH) specifications are contained in the Colorado Department of Highways Standard Specifications for Road and Bridge Construction. The bedding material shall consist of a well-graded mineral aggregate mixture, which will provide good stability. The size range of the aggregate shall be from 1/4-inch minimum to 3/4-inch maximum, with a maximum amount of fines passing a No. 8 screen not to exceed 5 percent by weight and shall conform to ASTM C33 or ASTM D448, gradation size No. 67, Class B. In the event over excavation for bedding is below the water table, the sub bedding material shall consist of 3/4 to 1-1/2-inch rock (or larger if approved) and shall be placed as specified.

ASTM C-33 No. 67 Aggregate Gradation	
Normal Size	Percent Passing by Weight
3/4"	90-100
3/8"	20-55
No. 4	0-10
No. 8	0-5

The bedding material shall be a minimum of 6-inches thick, with greater depth required for larger pipe. See pipe-bedding details (Figure 6-6 & 6-7) for recommended depth of bedding pipe for various pipe diameters.

CDOH Class A Filter Material (Section 703.09)	
Sieve	Mass Percent Passing Square Mesh Sieves
3"	100
3/4"	20-90
No. 4	0-20
No. 200	0-3

Type 1 UD&FCD Filter Material which is the Same Gradation as CDOH Sand AASHTO M6 (Section 703.01)	
Sieve	Mass Percent Passing Square Mesh Sieves
3/8"	100
No. 4	95-100
No. 16	45-80
No. 50	10-30
No. 100	2-10

Concrete Materials

Concrete materials for manholes, junction boxes, vaults, headwalls, cutoff walls, and other miscellaneous concrete structures within the storm sewer system shall be Class B concrete and shall conform to the requirements of the latest edition of Section 601-Structural Concrete and 602-Reinforcing Steel CDOT Standard Specifications for Road and Bridge Construction.

6.2.2 Installation of Storm Sewers

1. Excavation
   (See Town of Kersey, Street Design Standards and Construction Specifications).

2. Pipe Installation and Testing

   All pipe shall be installed in accordance with the manufacturer's recommendations and/or standard installation specifications required by AASHTO and others. In addition, installation shall be as follows:

I. Installation-General

   A. Use equipment, methods, and materials ensuring installation to lines and grades indicated.

      1. Maintain within tolerances specified or acceptable laying schedule.

         a. Alignment: ± 1 inch per 100 feet in open cut or tunnel.
         b. Grade: ± 1 inch per 100 feet.

      2. Do not lay pipe on blocks unless pipe is to receive total concrete encasement.

      3. Accomplish horizontal and vertical curve alignments with bends, bevels, and joint deflections.

a. Limit interior joint opening in concrete pipe except for open side on deflected joints to:
(1) 3/8-inch in laying schedule.
(2) ½-inch in actual installation.

B. Install pipe of size, materials, strength class, and joint type with embedment indicated for plan location.

C. Insofar as possible, (commence laying at downstream end of line and) install pipe with spigot or tongue ends in direction of flow. Obtain Engineer approval for deviations there from.

D. Clean interior of all pipe, fittings, and joints prior to installation. Exclude entrance of foreign matter during installation and at discontinuance of installation.

1. Close open ends of pipe with snug-fitting closures.

2. Do not let water fill trench. Include provisions to prevent flotation should water control measures prove inadequate.

3. Remove water, sand, mud, and other foreign materials from trench before removal of end cap.

E. Brace or anchor as required preventing displacement after establishing final position.

F. Perform only when weather and trench conditions are suitable. Do not lay pipe in water.

G. Observe extra precaution when hazardous atmospheres might be encountered.

II. Jointing

A. General Requirements

1. Locate joint to provide for differential movement at changes in type of pipe embedment, impervious trench checks, and structures. Provide one of the following:

a. Not more than 8-inches from structure wall.
b. Support pipe from wall to first joint with concrete cradle structurally continuous with base slab or footing.
c. As indicated.

2. Perform conforming to manufacturer's recommendations.

3. Clean and lubricate all joint and gasket surfaces with lubricant recommended.

4. Use methods and equipment capable of fully seating or making up joints without damage.

5. Check joint opening and deflection for specification limits.

6. Excavate bell holes at each joint or coupling to provide full-length barrel support of the pipe and to prevent point loading at the bells or couplings.

B. Special Provision for Jointing Concrete Pipe

1. With rubber gaskets:

a. Check gasket position and condition with feeler gauge prior to installation of next section.

C. Special Provisions for Jointing PVC Pipe and HDPE Pipe

1. Conform to ASTM D2321.

2. Connect pipe to new or existing rigid structures or manhole tie-ins with manhole couplings and a standard boot.

D. Special Provisions for Jointing ASP

1. Connect pipe to new or existing rigid structures or manhole tie-ins with manhole couplings and a standard boot.

III. Cutting

A. Cut in neat manner without damage to pipe.

IV. Temporary Plugs

A. Furnish and install temporary plugs at each end of Work for removal by others when completed ahead of adjacent contract.

B. Remove from pipe laid under adjacent contract in order to complete pipe connection when work by other contractor is finished prior to work at connection point under this Contract.

C. Plugs

1. Test plugs as manufactured by pipe supplier.

2. Fabricated by Contractor of substantial construction.

3. Watertight against heads up to 20 feet of water.

4. Secured in place in a manner to facilitate removal when required to connect pipe.

V. Connections to Existing Structures

A. Connect pipe to existing structures and pipelines where indicted.

B. Prepare structure by making an opening with at least 3-inches clearance all around fitting to be inserted.

C. Observe pertinent articles of Specifications pertaining to joint locations and closures.

D. Repair wall opening with non-shrinking grout.

VI. Field Testing

A. Acceptance Tests

1. Alignment

a. Sewer shall be inspected by flashing a light between manholes or by physical passage where space permits
b. Contractor shall clean pipe of excess mortar, joint sealant, and other dirt and debris prior to inspection.
c. Determine from Illumination or Physical Inspection:
(1) Presence of any misaligned, displaced, or broken pipe.
(2) Presence of visible infiltration or other defects.

B. Deflection Testing

1. Maximum installed deflections of flexible pipe if required shall be as follows:

Type of Pipe		Defection – Percent of Mean Internal Diameter
ASP & CMP		5
PVC		5
HDPE		5

2. Engineer may require Contractor to test flexible pipe after backfill has been in place 30 days and again after eleven (11) months if deemed necessary.

   a. Provide rigid ball or mandrel deflection testing equipment and labor.
   b. Obtain approval of equipment and acceptance of method proposed for use. Test shall be performed without mechanical pulling devices.
   c. Remove and replace pipe exceeding deflection limits.

3. Backfill and Compaction

   The backfill is the area above the pipe embedment zone. The pipe trench shall be backfilled and compacted in accordance with Town of Kersey, Street Design Standards and Construction Specifications.

   Haunching is the zone above the bedding up to the pipe spring line. Granular material as outlined for the bedding shall be placed and consolidated evenly on each side of the pipe. The embedment materials shall be consolidated under the lower haunch of the pipe with shovel slicing and tamping. Care shall be taken to see that conduit alignment and cross-sectional areas are maintained.

   Initial backfill extends from the spring line to 12-inches over the top of the pipe. Select bedding material is required for all flexible pipes to 12-inches over the top of the pipe. Backfill material may be local site material that is wellgraded, non-cohesive granular material free of rocks, frozen lumps, foreign material or stones greater than 3" in any dimension, or otherwise select bedding material. Remove all debris including soda cans, rags, pipe banding material, etc. from the pipe trench before backfilling. Compaction for native material shall be compacted to at least 95 percent of maximum density as determined by ASTM D698 (Standard Proctor) or AASHTO T99. Compaction machinery should not be used around flexible pipes until the select bedding is placed 12-inches over the top of the pipe.

4. Inspection and Testing

Installation of the pipe bedding, haunching, and initial backfill up to a point 12-inches above the top of the pipe, shall be observed by the Engineer. The Contractor/Developer will provide acceptance testing during backfill operations of trenches. The Contractor shall take Quality Control tests in the pipe haunch area for pipe diameters 36-inches and larger. For pipes smaller than 36-inch diameter, begin tests at one foot above the pipe. Quality Control tests shall be taken as followed:

Schedule for Quality Control Sampling & Testing		
Identification	**Type of Test Required**	**Minimum Sampling/Testing Frequency**
Storm Sewer Compaction	Moisture/Density Curve	One per soils type
	% Compaction	**Mainline:** one test every 200 LF every other lift
	% Moisture	**Lateral:** one test, per later; every other lift. Note: lifts are usually 1' thick

After backfill and compaction of the trench is completed, an inspection of the pipe shall be made to detect any deformations, sags, or joint displacements. Rigid pipe shall be visually inspected for sags or displaced joints.

Upon completion of storm sewer installation and prior to paving, the contractor shall notify the Engineer. The Contractor may be required to perform a pipe deflection test for flexible pipes in the presence of the Engineer.

Flexible pipe 48-inch diameter and smaller may be tested with a "Go/No Go" deflection test gage, which shall be pulled through the pipe. The maximum allowable deflection is 5 percent. The horizontal diameter shall not differ from the design diameter by more that 5 percent. Similarly, for pipes other than circular, the field-installed dimensions shall not vary more than 5 percent of the design dimensions. Any pipe that exceeds the maximum allowable deflection is to be removed and replaced.

The tests and inspection reports shall be submitted to the Engineer prior to proceeding to the next phase of construction and prior to paving. The Contractor shall provide the Engineer a Letter of Certification, prior to the issuance of building permits. The Letter of Certification shall state that the class, gage, or stiffness of pipe is in accordance with the Engineer's design for installation conditions encountered.

Inspection checklists for handling, storing, installing, and testing pipe are included at the end of this Section.

5. Connections to Existing Storm Sewer Systems

During construction of connections to existing storm sewer systems, extreme care shall be taken to ensure that there is adequate compaction of embedment material around existing pipe and new pipe.

Connection to different pipe materials shall be made using manholes or with transition sleeves, if available. Details for connection to different materials shall be provided.

6. Pipe Inspection Checklist

- Before unloading, inspect pipe and fittings for any obvious transportation damage.
- Check each pipe section and fitting for proper markings on interior of pipe.
- ASTM or AASHTO Specification.
- Pipe diameter, class or strength designation.
- Manufacturer or trade name.
- Date of manufacture.
- Number assigned to each pipe corresponding to laying diagram.
- Check each pipe section for external and internal damage.
- Check gaskets for damage and proper markings or identifications.
- Check that all pre-inserted gaskets are in place.

- Check lubricants, cleaners, or adhesives for conformance.
- Check flexible pipe for axial or longitudinal deformation.
- Mark each pipe that is rejected or needs to be repaired to prevent usage.
- Compare field repair procedures with manufacturer's requirements.
- Document repairs with photos, names of personnel, dates, equipment, and supplies.
- Pipe stored in accordance with manufacturer's instructions.
- Pipe stored on flat area, with joints supported.
- Pipe not stacked higher than allowed by manufacturer.
- Procedures followed that will not allow the pipe to become deformed during storage.
- All blocks, chocks, wedges are intact and firmly in place.
- PVC/HDPE pipe is protected from long-term (greater than 30 days) exposure to sunlight.
- Pipe is protected from adverse weather, harmful chemicals, dirt or debris accumulating on the interior of the pipe.
- Gaskets are protected from dust and grit, solvents, and petroleum-based greases and oils, and other agents having a harmful effect on the gasket.
- Stringing of pipe is in accordance with manufacturer's recommendations.
- Pipes are blocked to prevent movement due to wind or accidental bumping.
- Pipe joining surfaces shall be cleaned of any dust, dirt, and debris accumulation prior to installing gasket and joining.
- Interior of pipe is free of dirt and debris.
- Access to roads, driveways, fire hydrants, meters, etc., maintained.

- If stringing of pipe is required along roadway, is pipe orientated (angular rotation) properly, is pipe a safe distance from traffic, and is proper flasher signage present to protect traveling public.

7. Pipe Installation Checklist

- Pipe is correct type, diameter, strength, (class, SDR, etc).
- Pipe numbers and stationing checked against lay schedule.
- Pipe re-inspected for damage.
- Pipe cleaned of debris in interior and on gasket sealing surfaces.
- Pipe laid uphill on grades that exceed 10% (or less if specified).
- Pipe with marked field top laid with top up.
- Contractor continually checks alignment and grade of pipeline.
- Ends of pipe sealed at close of work or for shut-down periods.
- Bedding soils meet specifications.
- Compaction requirements met.
- Frequency of testing the bedding soils conforms to specifications.
- Bedding material checked for compatibility with other trench materials to prevent soil migration in groundwater areas.
- Trench bottom is free from loose rocks, large dirt clots, and debris.
- Bedding material does not contain organic matter, stumps or limbs, frozen earth, debris, refuse, or other unsuitable material.
- Minimum bedding thickness places. Required thickness = ________ ____________.
- Bedding surface is at the proper elevation so that pipe will be placed on grade.
- Bedding is placed so that barrel of pipe has uniform support.
- Blocking or mounding not used to bring pipe to grade.

- Bell holes and/or sling holes excavated.
- Clearance between bell and bedding checked.
- If high groundwater table present, floating may become a problem during installation of flexible pipelines. Trench must be dewatered during installation.
- Special attention given to HDPE pipe during times of high temperature to ensure increased pipe flexibility does not cause excessive deflection.
- PVC and HDPE may become brittle during cold weather. Avoid impact damage.

8. As Built Record Drawings

   Detailed drawings which have been prepared by the Design Engineer, upon completion and at the time of the Certificate of Completion, and show actual construction and contain field dimensions, elevations, details, changes made to the construction drawings by modification, details which were not included on the construction drawings, and horizontal and vertical locations of underground utilities which have been impacted by the utility installation.

   Maintain record drawings in clean, dry, legible conditions and in good order. Do not use record documents for construction purposes. Record as-built information concurrently with construction progress. Do not backfill work until required information is recorded.

   As-built record drawings shall be submitted to the Town for approval on 24"x36" black line form.

6.3 HYDRAULIC DESIGN

Storm sewers shall be designed to convey initial storm peaks without surcharging the sewer. To ensure that this objective is achieved, the hydraulic grade line shall be calculated by accounting for pipe friction losses and pipe form losses. Total hydraulic losses will include friction, expansion, contraction, bend, and junction losses. The final energy grade line shall be at or below the proposed ground surface. The methods for estimating the hydraulic losses are presented in the following sections.

6.3.1 Pipe Friction Losses

The Manning's "n" values to be used in the calculation of storm sewer capacity are presented in Table 6-1 for concrete and plastic pipe and Table 9-1 for corrugated metal pipe.

6.3.2 Pipe Form Losses

Generally, between the inlet and outlet the flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Form losses are the result of fully developed turbulence and can be expresses as follows:

$$H_L = K\,(V^2/2g) \qquad \text{(Equation 6.1)}$$

Where: H_L = head loss (feet)
K = loss coefficient
$\frac{V^2}{2g}$ = velocity head (feet)
g = gravitational acceleration (32.2 ft/sec2)

The following is a discussion of a few of the common types of form losses encountered in sewer system design. The user is referred to the USDCM or other professional manuals for additional discussion.

1. Expansion Losses

Expansion in a storm sewer conduit will result in a shearing action between the incoming high velocity jet and the surrounding sewer boundary. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence. The loss of head can be expressed as:

$$H_L = K_e\,(V_2^2/2g)\,[\,1 - (A_2)^2/A_1\,]^2 \qquad \text{(Equation 6.2)}$$

In which A is the cross section area, V is the average flow velocity, and Ke is the loss coefficient. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. The value of Ke is about 1.0 for a sudden expansion, and about 0.2 for a well-designed expansion transition. Table 6-2 presents the expansion loss coefficients for various flow conditions.

2. Contraction Losses

The form loss due to contraction is:

$$H_L = K_c\,(V22/2g)\,[\,1 - (A2)2/A1\,]2 \qquad \text{(Equation 6.3)}$$

Where K_c is the contraction coefficient. K_c is equal to 0.5 for a sudden contraction and about 0.1 for a well-designed transition. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. Table 6-2 presents the contraction loss coefficients for various flow conditions.

3. Bend Losses

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, may be expressed by the relation:

$$H_L = K_b (V^2/2g) \qquad \text{(Equation 6.4)}$$

In which K_b is the bend coefficient. The bend coefficient has been found to be a function of: (a) the ratio of the radius of curvature of the bend to the width of the conduit, (b) deflection angle of the conduit, © geometry of the cross section of flow, and (d) the Reynolds Number and relative roughness. A tabulation of the recommended bend loss coefficients for standard bends, radius pipe, and bend through manholes is presented in Tables 6-3 and 6-4.

4. Junction and Manhole Losses

A junction occurs where one or more branch sewers enter a main sewer, usually at manholes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances should be made for head loss due to the impact at junctions. The head loss for a straight through manhole or at an inlet entering the sewer is calculated from the original equation for form losses (Equation 6.1). The head loss at a junction can be calculated from:

$$H_L = (V_2^2/2g) - K_j (V_1^2/2g) \qquad \text{(Equation 6.5)}$$

Where V_2 is the outfall flow velocity and V_1 is the inlet velocity. The loss coefficient, K_j, for various junctions is presented in Table 6-5.

6.3.3 Storm Sewer Outlets

When the storm sewer system discharges into the Major Drainage way system (usually an open channel), additional losses occur at the outlet in the form of expansion losses (refer to Section 6.3.2). For a headwall and no wing walls, the loss coefficient K_e = 1.0 (refer to Table 6-2), and for a flared-end section the loss coefficient is approximately 0.5 or less.

6.3.4 Partially Full Pipe Flow

When a storm sewer is not flowing full, the sewer acts like an open channel, and the hydraulic properties can be calculated using open channel techniques (refer to Section

5). For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Figures 6-1, 6-2, and 6-3). The data presented assumes that the friction coefficient, Manning's "n" value, does not vary throughout the depth.

6.3.5 Hydraulic Research

The American Public Works Association (APWA) has conducted research into the head losses at various junctions and manholes. The work consisted of experimentally modeling three types of sewer junctions: junctions with a 90 degree bend, junctions of a main with a perpendicular lateral, and junctions of two opposed laterals. The work was primarily directed at sanitary sewers because the sizes investigated (i.e., manhole diameter to sewer diameter ratio of 2.3 to 4.6) and the flow conditions (i.e., pressure flow) were typical for sanitary sewers. However, several trends were observed that are considered to be suitable for storm sewers, including specific energy loss coefficients that have been adopted for these Criteria. The trends observed from the test results are as follows:

1. For manhole diameter to sewer diameter ratios between two (2) and six (6), the variation in head loss was insignificant.

2. The most significant reduction in head loss occurs when the manhole is shaped by benching the bottom of the manhole up to the top of the pipe. This appears to provide a better channelization of the flow, which reduces the losses.

3. Some testing was also performed for open channel flow conditions. These tests generally showed that the energy losses were less than for pressure flow. Since most storm sewers are not pressurized, the use of the coefficients should be conservative.

   This information is presented to aid the Designer in selecting suitable energy loss coefficients for situations not covered by these Criteria.

6.4 VERTICAL ALIGNMENT

The sewer grade shall be such that a minimum cover is maintained to withstand AASHTO HS-20 (or as designated by the Town Engineer loading on the pipe. The minimum cover depends upon the pipe size, type and class, and soil bedding condition, but shall not be less than 12 inches on any point along the pipe.

The minimum clearance between storm sewer and water main, either above or below, shall be 1 foot. Concrete encasement of the water line will be required for clearances of less than or equal to 1 foot.

The minimum clearance between storm sewer and sanitary sewer, either above or below, shall also be 1 foot. In addition, when a sanitary sewer main lies above a storm sewer, or within 18 inches below, 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.

6.5 HORIZONTAL ALIGNMENT

Storm sewer alignment between manholes shall be straight for storm sewers less than 48-inches in diameter. Storm sewers may be constructed with curvilinear alignment for 48-inch diameter and larger pipe by either the pulled joint method or by radius pipe in accordance with Table 6-1. The limitations on the radius for pulled joint pipe is dependent on the pipe length and diameter, and amount of opening permitted in the joint. The maximum allowable joint pull shall be ¾ of an inch. The minimum parameters for radius type pipe are shown in Table 6-1. The radius requirement for pipe bends is dependent upon the manufacturer's specifications. The minimum horizontal separation between storm sewers or sanitary sewers and water lines will be 10 feet.

6.6 PIPE SIZE

The minimum allowable pipe size for storm sewers, except for detention outlets, shall be 18 inches in diameter and shall be round pipe. Table 6-1 presents the minimum pipe size for storm sewers.

6.7 MANHOLES

6.7.1 General Information

Manholes or maintenance access ports shall be required whenever there is a change in size, direction, elevation, grade, or where there is a junction of two or more sewers. A manhole may be required at the beginning and/or at the end of the curved section of storm sewer. The maximum spacing between manholes for various pipe sizes shall be in accordance with Table 6-1.

The required manhole size shall be as follows:

Manhole Size	
Sewer Diameter	Manhole Diameter
15" to 18"	4'
21" to 30"	5'
36" to 54"	6'
60" and larger	CDOT Standard M-604-20

Larger manhole diameters or a junction structure may be required when sewer alignments are not straight through or more than one sewer line goes through the manhole.

6.7.2 Manhole Materials

1. Manhole

All materials, manufacturing operations, testing and inspection of manholes shall conform to the requirements of:

ASTM C 478M(C478) Precast Reinforced Concrete Manhole Risers (AASHTO M199) and Tops.

All precast concrete materials shall conform to Section 712.05 - Precast concrete units and all poured in place concrete manholes or vaults shall be Class B concrete and shall conform to Section 601 - Structural Concrete and 602 - Reinforcing Steel, CDOT Standard Specifications for Road and Bridge Construction.

Manholes shall consist of precast riser sections, top or cone section, precast adjusting rings, precast or field poured base, steps and rings and covers. Manholes shall be constructed in accordance with the Standard Details or as shown on the Plans.

Precast concrete manholes shall be of the eccentric, concentric or flat top type as described in the Standard Details. Manholes shall be of the diameter and depth shown on the Plans. Manholes in excess of 20 feet depth shall have an intermediate platform located at the approximate center of the depth (See Figure 6-11).

Riser and top sections shall be precast reinforced concrete. Adjusting rings shall be reinforced with the same percentage of steel as the riser and top.

2. Steps

Steps shall be required when the manhole depth exceeds 3'-6" and shall be in accordance with AASHTO M 199. Steps shall be firmly embedded in the wall of each manhole riser and cone section. Steps shall withstand vertical loads of 400 pounds. Steps shall be placed in a straight line and be uniformly spaced. Steps shall be positioned to allow 20 to 26 inches spacing from the rim to the first step, and spacing thereafter shall be not less than 12 inches nor more than 15 inches center to center.

Steps shall be comprised of a minimum 3/8-inch diameter grade 60 steel reinforcing rod completely encapsulated in polypropylene, as manufactured by M.A. Industries, Inc. or equal. M.A. steps shall be either type PS-2PF or PS-2-PFS. Plastic manhole steps shall conform to ASTM C-478. Steps in riser sections shall project from the wall not less than 6-5/8 inches.

Steps in cone sections shall project from the wall not less than 4-7/8 inches. All steps shall penetrate the wall not less than 3-3/8 inches.

3. Rings and Covers

Iron manhole rings and covers shall be the best quality gray iron, tough and even grain, and when cast, shall be free from faults, blowholes, or other defects, and shall posses a tensile strength of not less than 18,000 psi. Rings and covers shall be designed to withstand the traffic loads that will be imposed upon them. Rings and covers shall be manufactured for current CDOT Standards 712.06 and meet the requirements shown on Figure 6-8.

The horizontal bearing surfaces of the ring and cover shall be machined so that they will not rock under traffic. Covers, which do not rest solidly in the frames, will not be accepted.

Manhole rings and covers shall be in accordance with the Town of Kersey Standards. Covers shall be non-perforated, and shall show the lettering as indicated on Figure 6-8.

When a manhole is located in a pavement area, it shall not be brought to final grade until the pavement has been completed.

4. Manhole Gaskets

Where preformed, flexible plastic gaskets shall be used to seal joints between precast manhole sections, they shall conform to Federal Specifications SS-S-00210 (6SA-FSS), Type I, Rope Form, and shall have a minimum diameter of 1-1/2 inches. Gaskets shall be applied to the tongue and shoulder lips of the precast section, providing two (2) gaskets per joint. "RUB'R NEK" or "KENT SEAL" or approved equal products shall be used.

6.7.3 Manhole Construction

The work covered by this Subsection consists of constructing precast, preassembled or field assembled manholes for storm sewer construction. Construction consists of excavation; shoring; dewatering; subgrade preparation; construction of base; placement and assembly of risers, cone, or tops; installation of ring, cover and adjusting rings; backfilling; surface restoration and other related work. The following quality standards shall apply:

ASTM C 891: Installation of Underground Precast Concrete Utility Structures.

1. Materials

The Contractor shall install manholes of the dimensions shown on the Plans. All materials used shall conform to the requirements of Section 6.7.2 above.

2. Surface Preparation, Excavation, De-watering

Surface preparation, excavation and dewatering shall conform to the requirements cited in Volume I - Streets - Design Criteria and Construction Specifications.

3. Manhole Base

Manhole bases shall be precast or field poured as detailed on standard plates but never less than 8" thick below the invert. Concrete shall be Class B and shall conform to the requirements of the latest edition of Section 601-Structural Concrete and 602-Reinforcing Steel (for larger manholes, See Figure 6-9), CDOT Standard Specifications for Road and Bridge Construction. Concrete shall be consolidated and struck-off to a horizontal surface within the forms or pouring rings.

Invert channels shall be smooth and semi-circular in shape conforming to the inside of the adjacent sewer section. Changes in direction of flow in manholes with only one entering sewer shall be made per applicable (see details shown on Figure 6-9 & 6-10) standard plate. Changes in size and grade of the channels shall be made gradually and evenly. Channels in bases of manholes at intersections of sewers shall follow the alignment of the sewers. The invert channels may be formed directly in the concrete of the manhole base or may be half-pipe laid in concrete. Flow entering into and passing through the manhole shall be unobstructed. Liquid or solids shall not be retained by the manhole or adjoining pipe. The floor of the manhole outside the channel shall be broomed and shall slope toward the channel not less than one inch per foot, nor more than two inches per foot.

Field poured concrete bases for 6' dia. or larger manholes shall be reinforced as detailed on the Plans or as shown on Figure 6-9 & 6-10.

Precast reinforced concrete bases shall be of the size and shape detailed on the Plans as shown on Figure 6-9 & 6-10. Precast sections which appear to be porous, honeycombed, cracked, out-ofround, chipped, have exposed rebar, or are otherwise defective shall be rejected.

4. Manhole Barrels

Manhole barrels shall be assembled of precast riser sections. Riser sections shall be placed vertically with tongues and grooves properly keyed. Ladder rungs shall be vertically aligned and equidistant in the finished manhole. The top step shall be placed so that it is between 20 and 26 inches below the finished rim elevation. Barrel sections that appear to be porous, honeycombed, out-of-round, cracked, chipped, have exposed rebar, or are otherwise deformed or damaged shall be rejected.

Intermediate platforms shall be assembled into the manholes that are over 20 feet in depth (see Figure 6-11). Free drop inside the manhole shall not exceed two feet measured from the invert of the inlet pipe to the invert of the outlet pipe. Where the drop exceeds two feet, inside drop manholes shall be constructed as detailed on the Plans of as approved by the Town.

All connections between the riser or base sections and the sewer pipe shall be joined in such a manner as to make the manhole watertight. Preformed rubber water stop gaskets cast into the riser or base section is an acceptable joining method.

Preformed flexible plastic sealing compounds similar or equal to "Rub'R Nek" or "Kent Seal" are acceptable, provided acceptable water tightness is achieved.

5. Top or Cone Sections

Flat top sections may be used on a shallow manhole. Otherwise, cone sections shall be installed for heights exceeding 8 feet (see Figure 6-9 for more detail).

Cone shaped top sections shall be assembled on top of the manhole barrel with tongues and grooves properly keyed. Ladder rungs shall be equidistant and vertically aligned in the finished manhole. The top ladder rung shall be installed to an elevation in accordance with Figure 6-9.

Concrete grade rings may be used for adjusting the manhole lid elevation. The total height from the top of cone to top of frame shall not exceed 16 inches. Metal adjusting grade rings are not allowed. Broken concrete grade rings shall be rejected.

Non-shrink grout, shall be placed under and between the metal ring and adjusting grade rings, and between adjusting grade rings and the cone section. For manholes in open areas, grass or on gravel roads, concrete grout shall surround the metal ring and adjusting grade rings; the grout shall be formed horizontally to the outside diameter of the cone section and vertically to the rim elevation, in accordance with Figure 6-9. Install manhole lid to the ground surface in an open area or grass area. In gravel roads, leave the lid 6-inches below the road surface. For manholes in paved (asphalt or concrete) streets or other paved areas, install a concrete collar in accordance with Standards.

6. Watertightness

The finished manhole is expected to be as watertight as the pipe system it is incorporated into. Observed leaks shall be cause for rejection.

All connections between riser sections, bases, tops, and rings shall be sealed with preformed flexible plastic joint sealing compound. Application of sealing compound shall be accomplished in conformance with the manufacturer's recommendations. Grade of materials, quantity of materials and application temperatures recommended by the manufacturer shall govern. Sealing compound similar or equal to "Rub 'R Nek" or "Kent Seal" shall be used.

7. Connections

All connections of pipe to manhole shall be made with a proper water stop. Mains tapped into manholes shall be constructed so that the flow entering the manhole is channeled through the bench into the invert of the manhole under all flow conditions.

8. Bedding and Backfilling

Bedding, backfilling, and surface restoration around manholes shall conform to the requirements of the applicable section. Bedding material shall be placed up to a point equal to that required for the adjacent pipe.

9. Quality Control

Inspection, testing, approval and acceptance shall conform to the requirements of Town of Kersey - Streets - Design Criteria and Construction Specifications.

Materials not inspected by the Town's Inspector or damaged by an action of the Contractor may be subsequently rejected and replaced at the Contractor's expense.

Materials inspected by the Town's Inspector, installed by the Contractor and found to be damaged through no fault of the Contractor shall be repaired or rejected and replaced at the Developer's expense.

10. Clean Up

All rubbish, unused materials and other non-native materials shall be removed from the job site. All excess excavation shall be disposed of as specified, and the right-of-way shall be left in a state of order and cleanliness. Manholes and pipelines shall be free of dirt, scum, gravel, grout, and other foreign material.

6.8 Maintenance and Access Easements

An important aspect of storm sewer operation is the continued maintenance of the facilities to ensure that they will continue to function as designed. Maintenance may include cleaning of trash racks, inlet and outlet structures, and the removal of sediment and debris from the storm sewers. The Town requires that maintenance access be provided to all storm drainage facilities. The following minimum easement widths, which must be shown on the Final Plat, are required:

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 placed at the 1/3 point in the easement)

6.9 DESIGN EXAMPLE

The following calculation example, including the calculation Table 6-6, and Figures 6-4 and 6-5, were obtained from Modern Sewer Design, AISI, Washington D.C., 1980 and edited for the calculation of manhole and junction losses in accordance with this Section.

Given: (a) Plan and Profile of storm sewer (Figures 6-4 and 6-5), example calculation form (Table 6-6).
(b) Station 0+00 (outfall) data as follows:

Design Discharge	Q	= 145 cfs	(Column 9)
Invert of Pipe		= 94.50 feet	(Column 2)
Diameter	D	= 66-inch RCP	(Column 3)
Starting Water Surface	W.S.	= 100 feet	(Column 4)
Area of Pipe	A	= 23.76 sq. ft.	(Column 6)
Velocity = Q/A	V	= 6.1 fps	(Column 8)
Pipe Roughness	n	= 0.013	

Notes:

(i) Numbers in brackets refers to the Columns on Table 6-6

(ii) Sizes of the storm sewer were determined during the preliminary design phases.

Find: Hydraulic Grade Line and Energy Grade Line for storm sewer.

Discussion: The following procedure is based on full-flow pipe conditions. If the pipe is flowing substantially full (i.e., greater than 80 percent), the following procedures can be used with minimal loss of accuracy. However, the Designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.

STEP 1: The normal depth is greater than critical depth, dn > dc; therefore, calculations to begin at outfall, working upstream. Compute the following parameters:

$$\varphi = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times (.013)^2}{2.21}$$

This equation is derived from the Manning's equation by solving for velocity and converting to velocity head.

$\varphi = 0.00492$

This value remains constant for this design since the n-value does not change.

STEP 2: Velocity Head (Column 10):

$$H_v = \frac{V^2}{2g} = \frac{(6.1)^2}{(2)(32.2)}$$

$H_v = 0.58$

STEP 3: Energy Grade Point, E.G. (Column 11):

E.G. = W.S. + Hv = 100 + 0.58

E.G. = 100.58

For the initial calculation, the Energy Grade Line is computed as described above. For subsequent calculations, the equation is reversed, and the water surface is calculated as follows (see Step 12):

W.S = E.G. - Hv

This equation is used since the losses computed in Step 8 are energy losses which are added to the downstream energy grade elevation as the new starting point from which the velocity head is subtracted as shown above.

STEP 4: $$S_f = \frac{\varphi H v}{R^{4/3}} = \frac{(.00492)(0.58)}{(1.375)4/3}$$

$S_f = .0019$

Note: R = the hydraulic radius of the pipe

STEP 5: Avg S_f (Column 13):

Average skin friction: This is the average value between Sf of the station being calculated and the previous station. For the first station, Avg Sf = Sf. Beginning with Column 13, the entrees are placed in the next row since they represent the calculated losses between two stations.

STEP 6: Enter sewer length, L, in Column 14.

STEP 7: Friction loss Hf (Column 15):

$H_f = (\text{Avg Sf})(L)$
$H_f = (.0019)(110)$
$H_f = 0.21$

STEP 8: Calculate the form losses for bends, junction, manholes, and transition losses (expansion or contraction) using the appropriate equations. The calculation of these losses is presented below for the various sewer segments since all the losses do not occur for one sewer segment.

(a) Station 1 + 10 to 1 + 52.4 (bend)

$H_b = K_b H_v$, where the degree of bend is 60 degrees
$K_b = 0.20$ (Table 6-3, Case I)
$H_b = (0.20)(0.58) = 0.12$ (Enter in Column 16)

(b) Station 2 + 48 to 2 + 55.5 (transition, expansion)

$H_L = K_e\ Hv1\ [1\text{-}A_1/A_2]$
$K_e = 1.06$ (Table 6-2) for $D_2/D_1 = 1.5$, and $\varphi = 45$
$H_L = (1.06)\ 1.29\ [1 - 15.9/23.76]2 = 0.15$ (Enter in Column 19)

(c) Station 3 + 55.5 (manhole, straight through)

$H_m = K_m H_v$
$K_m = (0.05)$(Table 6-5, Case I)
$H_m = (0.05)(1.29) = 0.06$ (Enter in Column 18)

(d) Station 4 + 55.5 to 4 + 65.5 (junction)

$H_j = H_{v2} - K_j H_{v1}$
$K_j = 0.62$ (Table 6-5, Case III) $\varphi = 30$ degrees
$H_j = 1.29 - (.66)(0.99) = 0.64$ (Enter in Column 17)

(e) Station 5 + 65.5 to 5 + 75.5 (junction) - since there are two laterals, the loss is estimated as twice the loss for one lateral

K_j = 0.33 (Table 6-5, Case III) φ = 70 degrees
H_j = 0.99 - (0.33)(0.64) = 0.78 for one lateral

STEP 9: Sum all the form losses from Columns 15 through 19 and enter in Column 20. For the reach between Station 0+00 and 1+10, the total loss is 0.21

STEP 10: Add the total loss in Column 20 to the energy grade at the downstream end (Sta 0+00) to compute the energy grade at the upstream end (Sta 1+10) for this example.

E.G. (U/S) = E.G. (D/S) + TOTAL LOSS
= 100.58 + 0.21
= 100.79 (Column 11)

Where: E.G. = energy grade
U/S = upstream
D/S = downstream

STEP 11: Enter the new invert (Column 2), pipe diameter D (Column 3), pipe shape (Column 5), pipe area A (Column 6), then compute the constant φ from Step 1 in Column 7, the computed velocity V in Column 8, the new Q (Column 9), and the computed velocity head Hv (Column 10).

STEP 12: Compute the new water surface, W.S., for the upstream station (Station 1+10 for this example).

W.S. = E.G. - H_v
= 100.79 -0.58
= 100.21 (Column 4)

STEP 13: Repeat Steps 1 through 12 until the design is complete. The hydraulic grade line and the energy grade line are plotted on the profile (see Figure 6-5).

Discussion of Results

The hydraulic grade line (HGL) is at the crown of the pipe from Station 0+00 to 2+48. Upstream of the transition (Station 2+55.5), the 54-inch RCP has a greater capacity (approximately 175 cfs) at that slope than the design flow (145 cfs). The pipe is therefore not flowing full but is substantially full (i.e., 145/175 = 0.84, greater than 0.80).

The computed HGL is below the crown of the pipe. However, at the outlet, the actual HGL is higher, since the outlet of the 54-inch RCP is submerged by the headwater for the 66-inch RCP. To compute the actual profile, a backwater calculation would be required; however this accuracy is not necessary for storm sewer design in most cases.

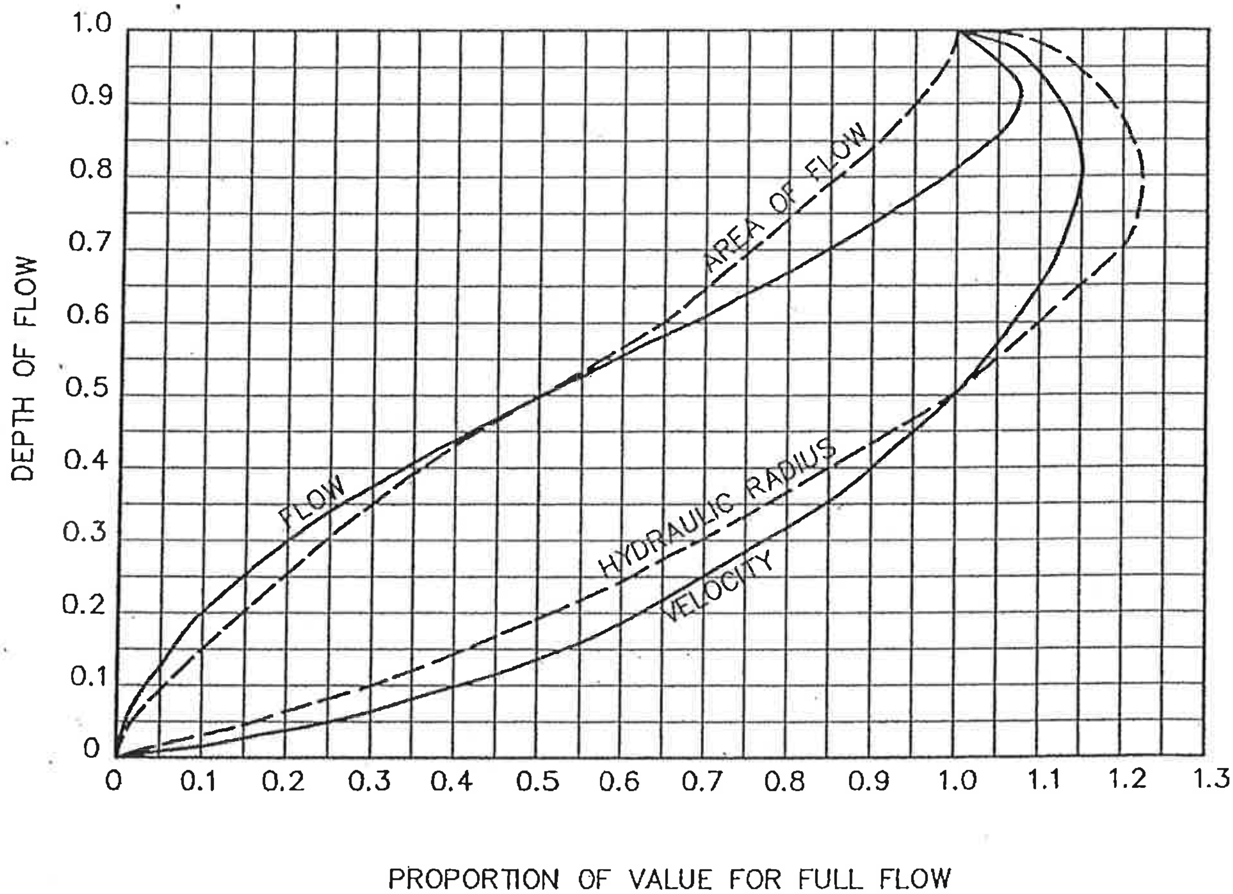
At the junction (Station 4+55.5), the HGL is below the top of the pipe. However, in this case, the full flow capacity (100 cfs) is the same as the design capacity, and the HGL remains parallel to the top of the pipe. A similar situation occurs at the junction at Station 5+65.5 except that the HGL remains above and parallel to the top of the pipe.

If the pipe entering a manhole or junction is at an elevation significantly above the manhole invert, a discontinuity in the EGL may occur. If the EGL of the incoming pipe for the design flow condition is higher then the EGL in the manhole, then a discontinuity exists, and the higher EGL is used for the incoming pipe.

6.10 CHECKLIST

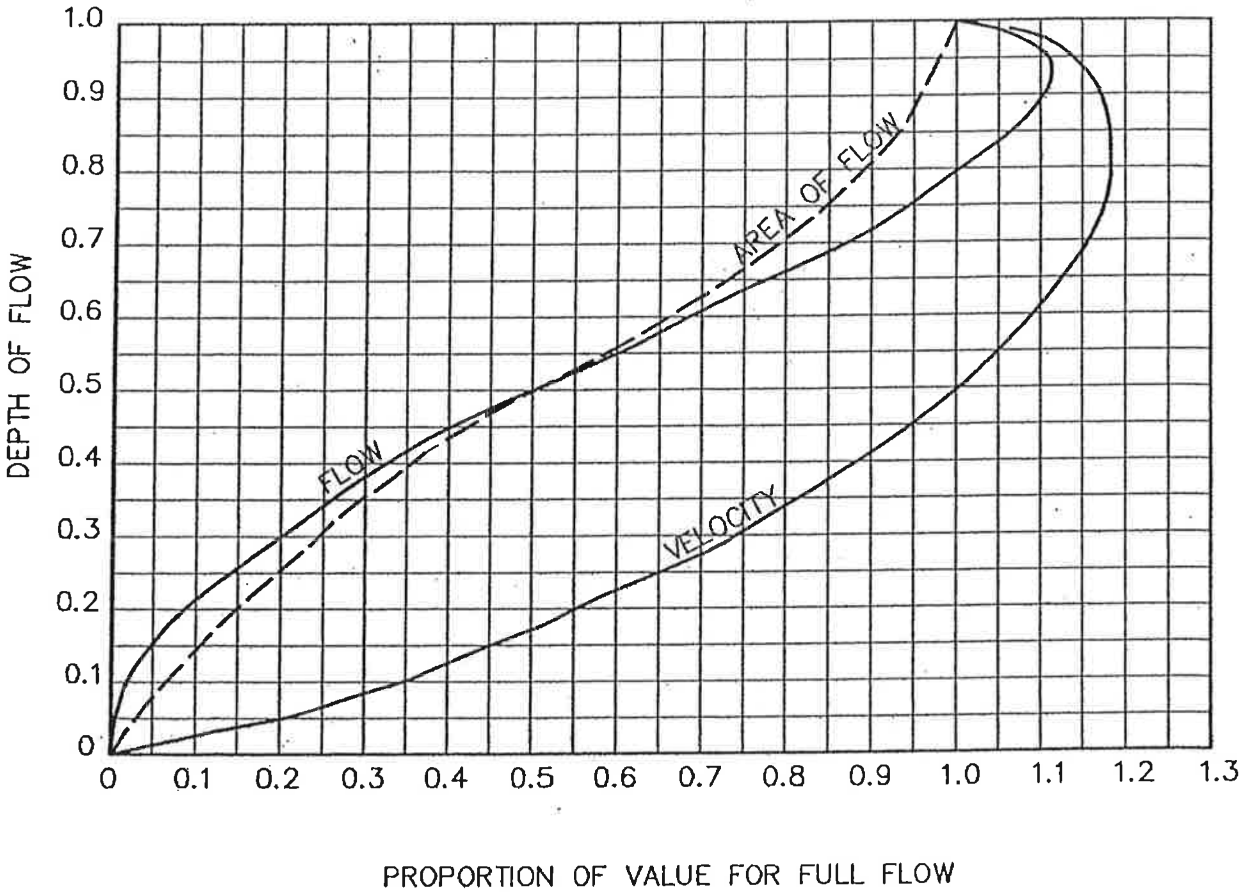
To aid the Designer and Reviewer, the following checklist has been prepared:

1. Calculate the energy grade line (EGL) and hydraulic grade line (HGL) for all sewers and show on the plan and profile drawings.
2. Account for all losses in the EGL calculation including outlet, form, bend, manhole, and junction losses.
3. Provide adequate protection at the outlet of all sewers into open channels.
4. Check for minimum pipe cover and clearance with all utilities.



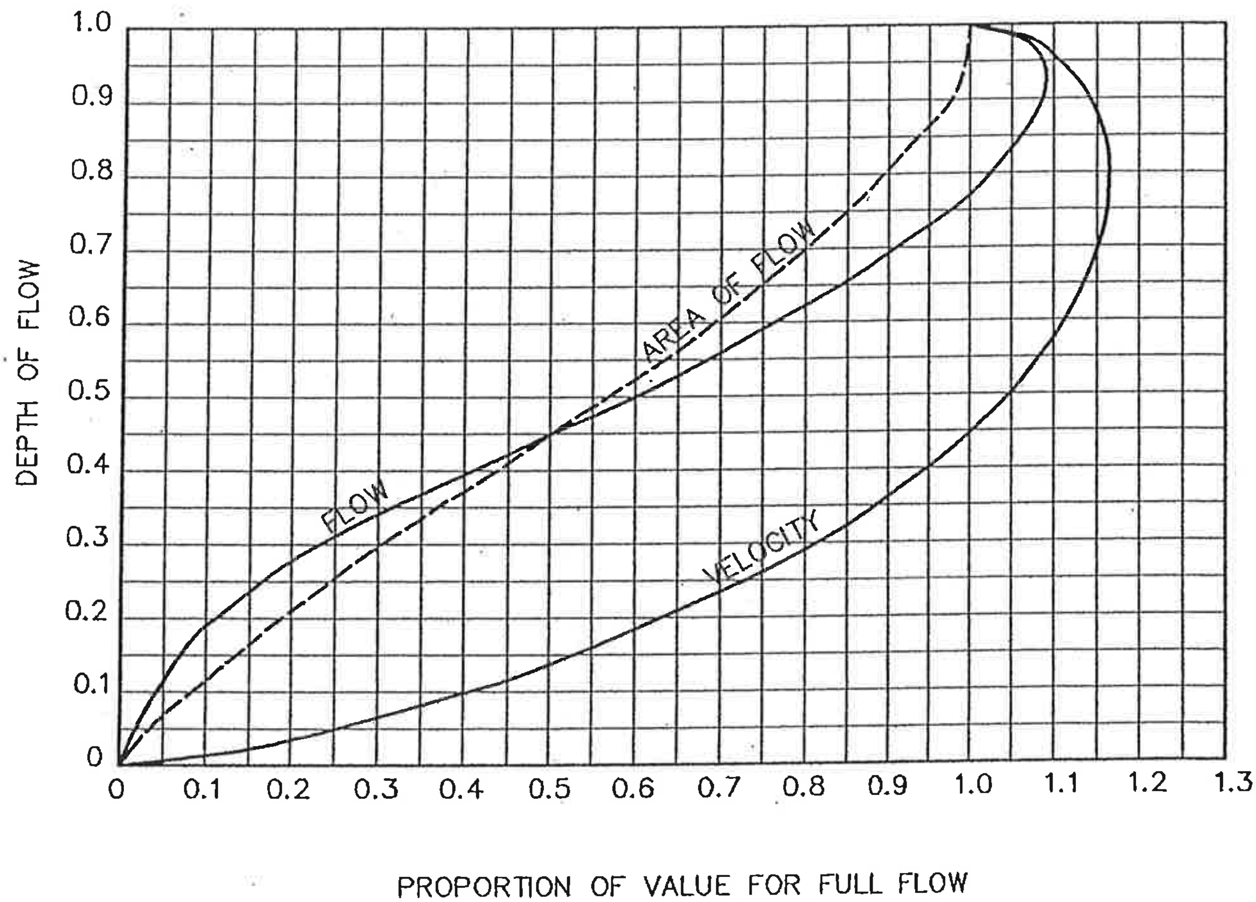
TOWN OF KERSEY	
HYDRAULIC PROPERTIES CIRCULAR PIPE	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 6–1

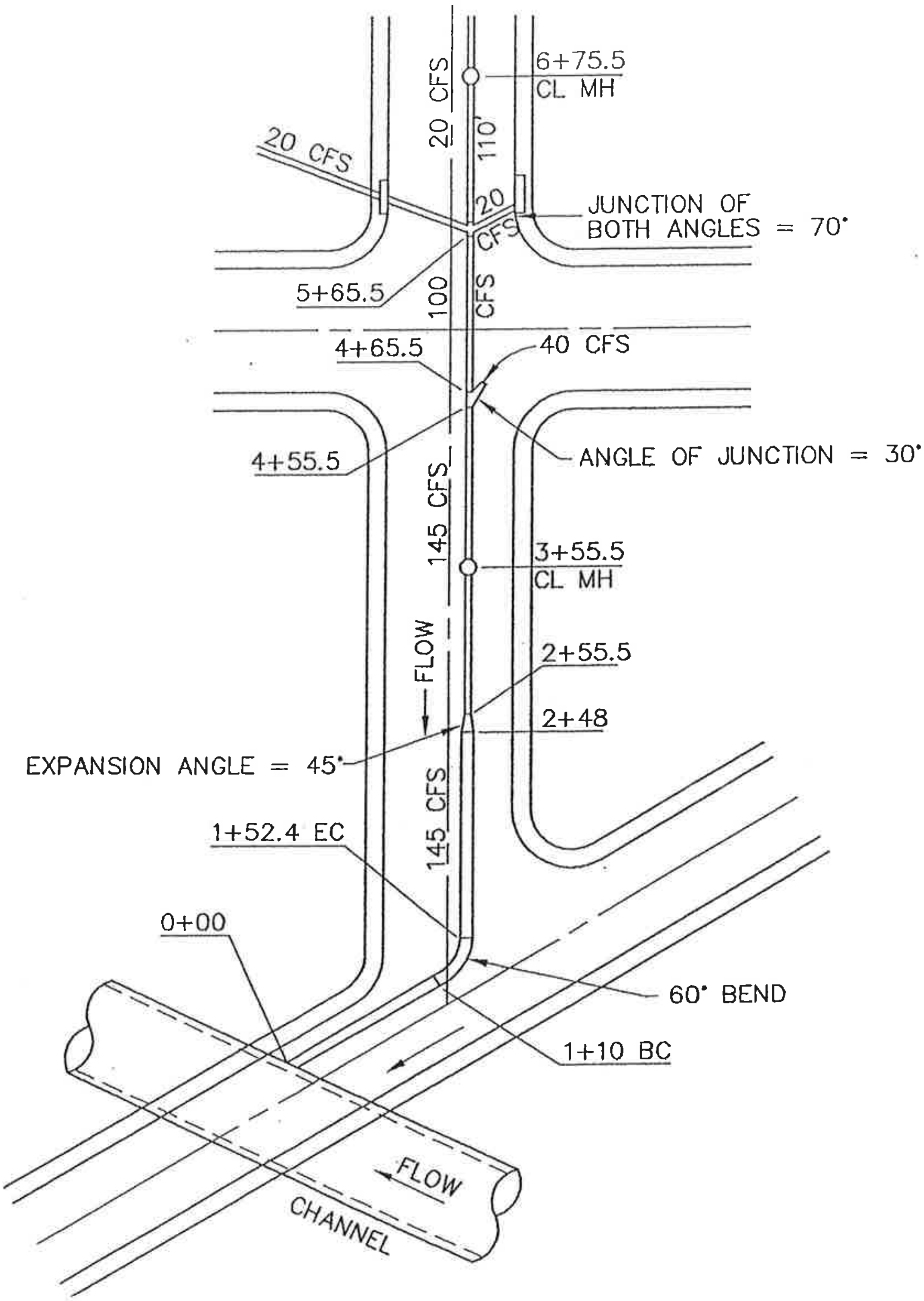


TOWN OF KERSEY	
HYDRAULIC PROPERTIES HORIZONTAL ELLEPTICAL PIPE	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 6–2



TOWN OF KERSEY	
HYDRAULIC PROPERTIES ARCH PIPE	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

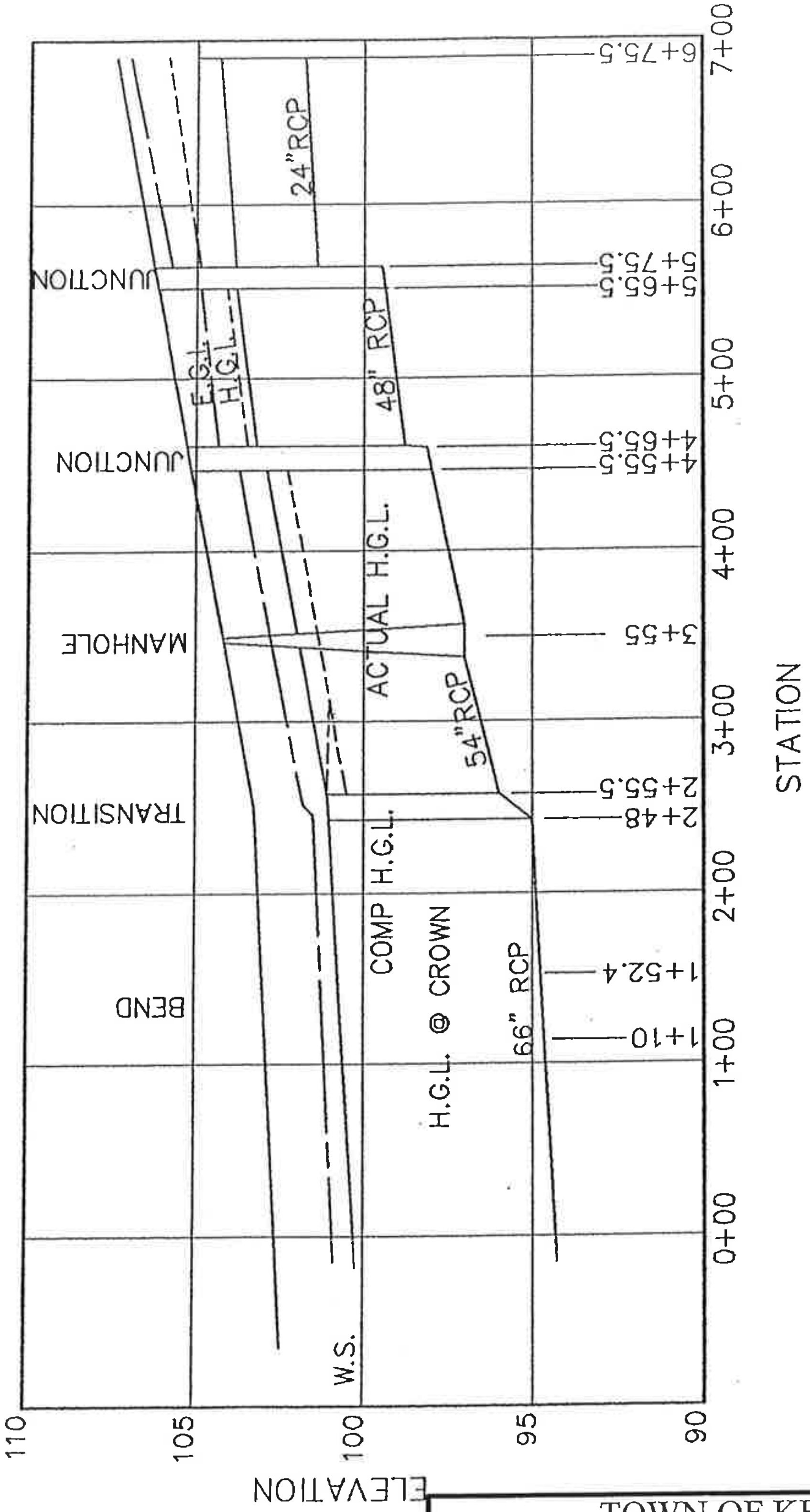


TOWN OF KERSEY

DESIGN EXAMPLE FOR STORM DRAINS – PLAN

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 6–4



TOWN OF KERSEY

DESIGN EXAMPLE FOR STORM DRAINS – PROFILE

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 6–5

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	¢	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	0.0019	7.5	0.01	–	–	–	0.15	0.16
2+55.5	96.08	54	100.04	RND	15.90	0.00492	9.1	145	1.29	101.33	0.0054	0.0037	100	0.37	–	–	–	–	0.37
3+55.5	96.90	54	100.41	RND	15.90	0.00492	9.1	145	1.29	101.70	0.0054	0.0054	100	0.54	–	–	0.06	–	0.61
4+55.5	97.66	54	101.01	RND	15.90	0.00492	9.1	145	1.29	102.30	0.0054	0.0054	10	0.05	–	0.64	–	–	0.69
4+65.5	98.40	48	102.01	RND	12.57	0.00492	8.0	100	0.99	103.00	0.0049	0.0052	100	0.52	–	–	–	–	0.52
5+65.5	98.89	48	102.52	RND	12.57	0.00492	8.0	100	0.99	103.51	0.0049	0.0049	10	0.05	–	1.56	–	–	1.61
5+75.5	100.89	24	104.49	RND	3.14	0.00492	6.4	20	0.64	105.12	0.0079	0.0063	100	0.63	–	–	–	–	0.63
6+75.5	101.61	24	105.13	RND	3.14	0.00492	6.4	20	0.64	105.76	0.0079	0.0078							

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

$$\text{¢} = \frac{2g(n^2)}{2.21}$$

TOTAL FRICTION LOSS = 2.64
TOTAL FORM LOSS = 2.54

NOTE:
SEE FIGURE 6–4 AND 6–5

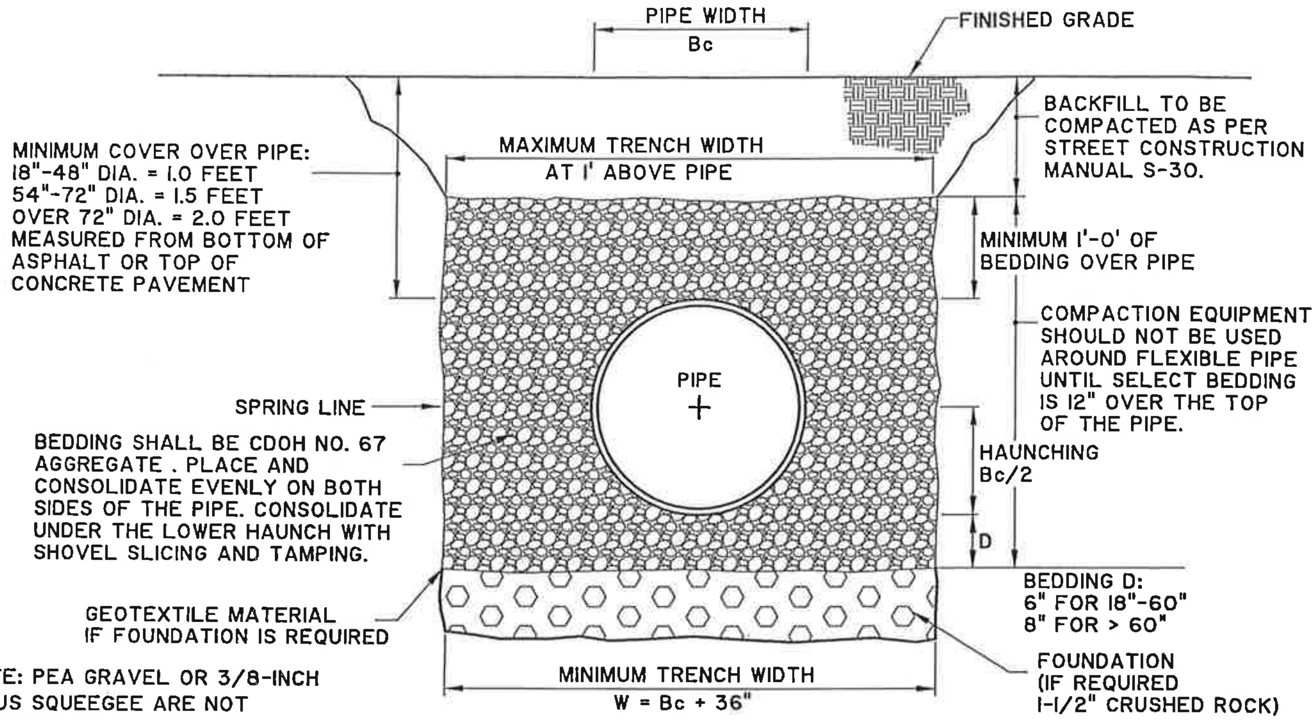
TOWN OF KERSEY

DESIGN EXAMPLE FOR STORM DRAINS

SCALE: NONE
DATE: 12/04

APPROVED:
FJB
TOWN ENGINEER

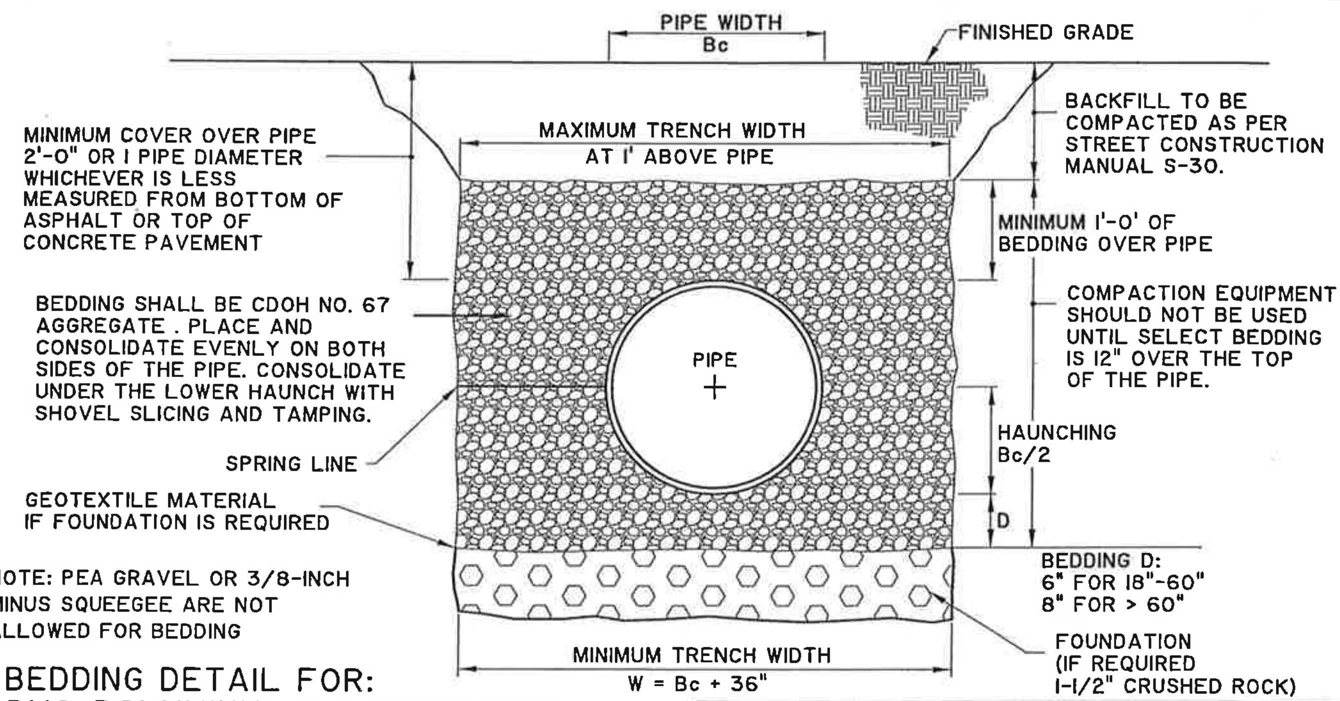
SHEET TABLE 6–6



NOTE: PEA GRAVEL OR 3/8-INCH MINUS SQUEEGEE ARE NOT ALLOWED FOR BEDDING

NOTE: FOR TRENCH BACKFILL GUIDELINE SPECIFICATIONS SEE DETAIL IN THE TOWN OF KERSEY DESIGN CRITERIA CONSTRUCTION SPECIFICATIONS - STREETS

BEDDING DETAIL FOR:
CMP-CORRUGATED METAL PIPE



NOTE: PEA GRAVEL OR 3/8-INCH MINUS SQUEEGEE ARE NOT ALLOWED FOR BEDDING

BEDDING DETAIL FOR:
PVC-POLYVINYL CHLORIDE PIPE

NOTE: FOR TRENCH BACKFILL GUIDELINE SPECIFICATIONS SEE DETAIL IN THE TOWN OF KERSEY DESIGN CRITERIA CONSTRUCTION SPECIFICATIONS - STREETS

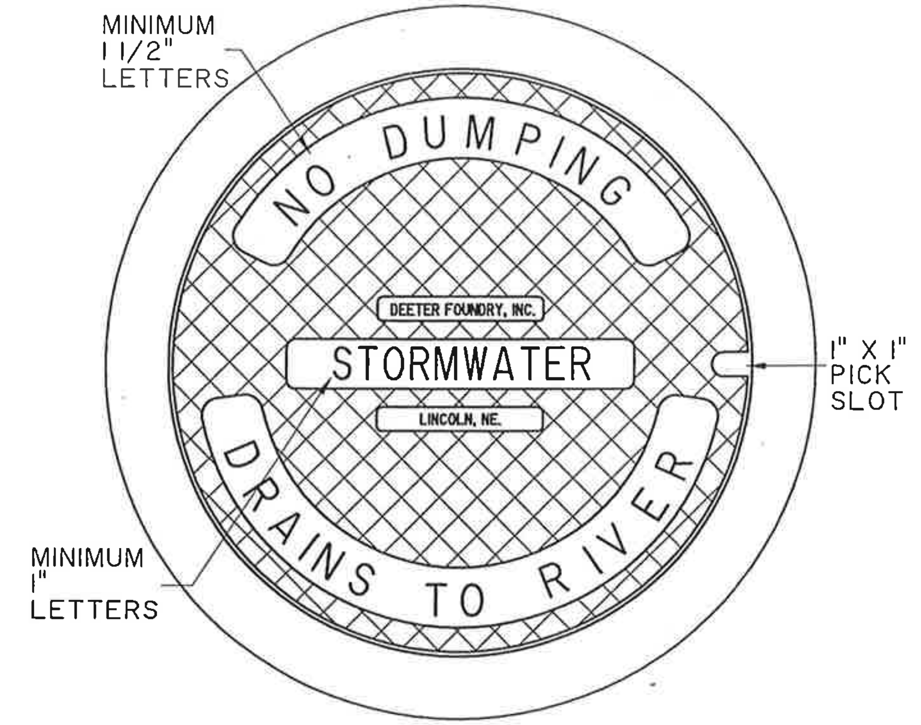
TOWN OF KERSEY

STANDARD STROMWATER PIPING BEDDING DETAIL

SCALE: NONE | DATE: 12/04

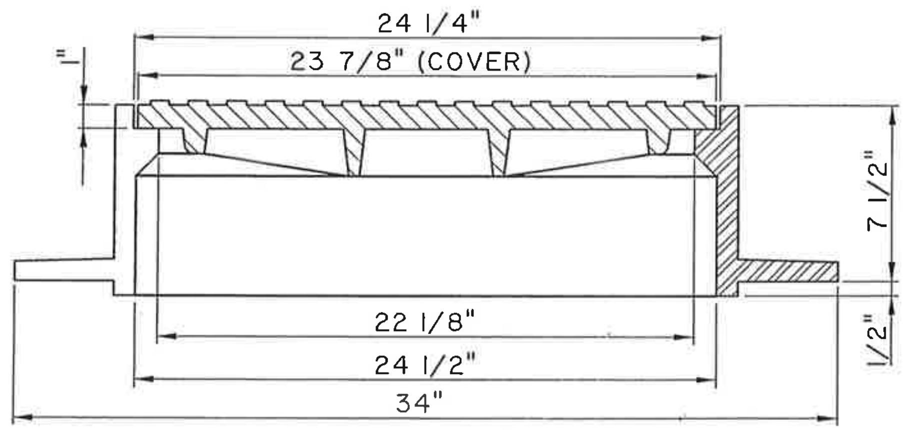
APPROVED: FJB
TOWN ENGINEER

SHEET FIG. 6-7



TOTAL WEIGHT - MINIMUM OF 365 LBS.

PLAN



SECTION

GENERAL NOTES

1. RING AND COVER SHALL BE GRAY IRON, MANUFACTURED PER ASTM A-48, AASHTO M 105, CLASS 35B.
2. COVER SHALL BE NONPERFORATED, WITH LETTERING AS SHOWN, CAST ON THE TOP OF THE LID FOR STORMWATER MANHOLES.
3. COVER SHALL BE BOLTED IF SPECIFIED BY THE TOWN. BOLTS SHALL BE STAINLESS STEEL.
4. NEENAH NO. R-1706 MANHOLE RING AND SOLID COVER, OR DEETER NO. 1258, OR APPROVED EQUAL. TOTAL WEIGHT SHALL BE A MINIMUM OF 365 LBS AND MUST EXCEED PROOF LOADS OF 16,000 LBS.
5. LID SHALL READ "STORMWATER". "STORM SEWER" IS UNACCEPTABLE.
6. FISH SYMBOL ON LID IS PREFERRED, BUT NOT REQUIRED.
7. COVER AND RING OPENING SHALL BE SQUARE, NOT TAPERED.

TOWN OF KERSEY

STANDARD STORMWATER MANHOLE RING AND COVER

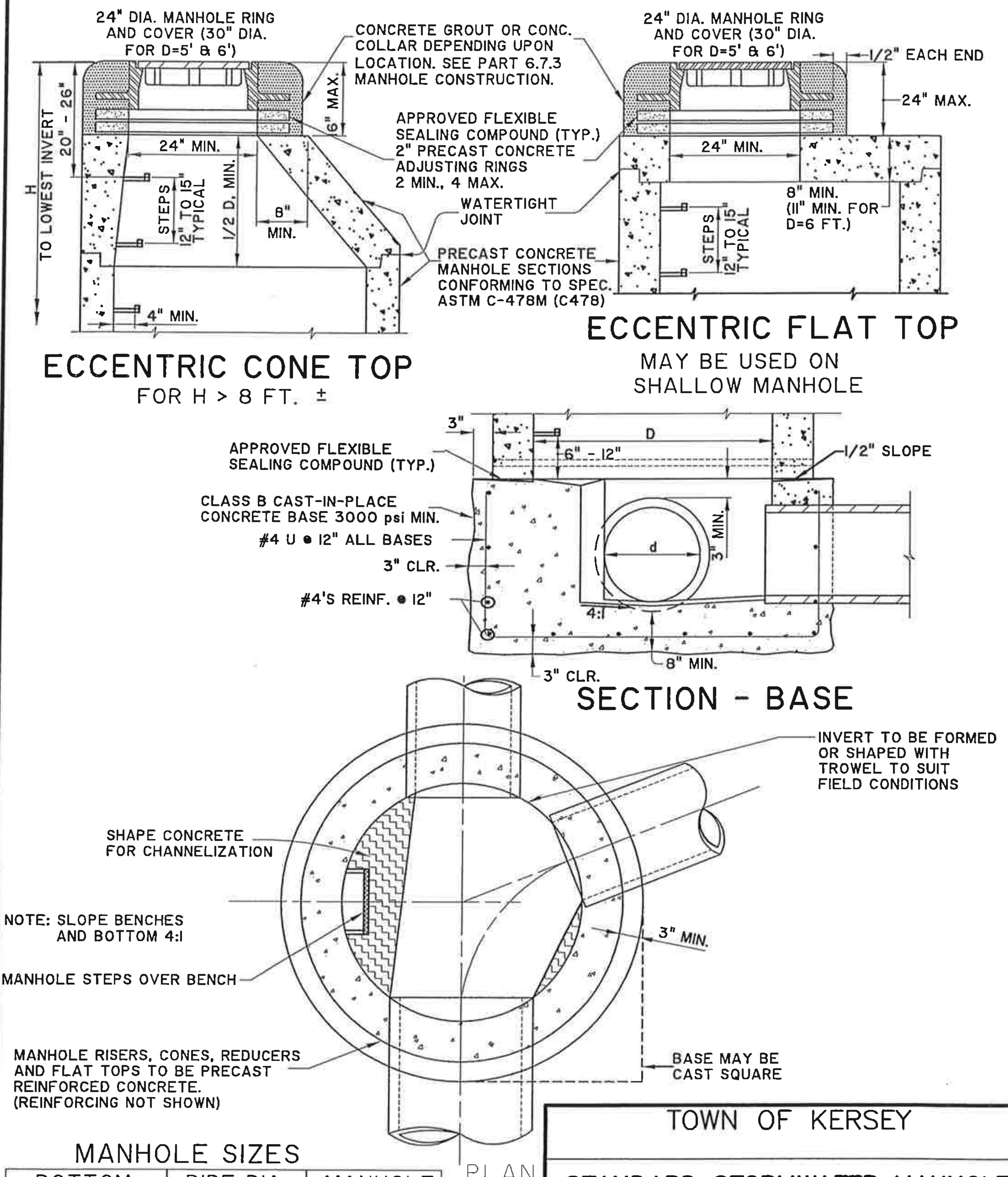
SCALE: NONE

DATE: 12/04

APPROVED:

FJB

TOWN ENGINEER



MANHOLE SIZES

BOTTOM REINF.	PIPE DIA. (d)	MANHOLE DIAMETER
#4 @ 12	15" TO 18"	4 FT.
#4 @ 12	21" TO 30"	5 FT.
#4 @ 12	36" TO 54"	6 FT.
60" & Larger	CDOT Std. M-604-20	

TOWN OF KERSEY

STANDARD STORMWATER MANHOLE

SCALE: NONE

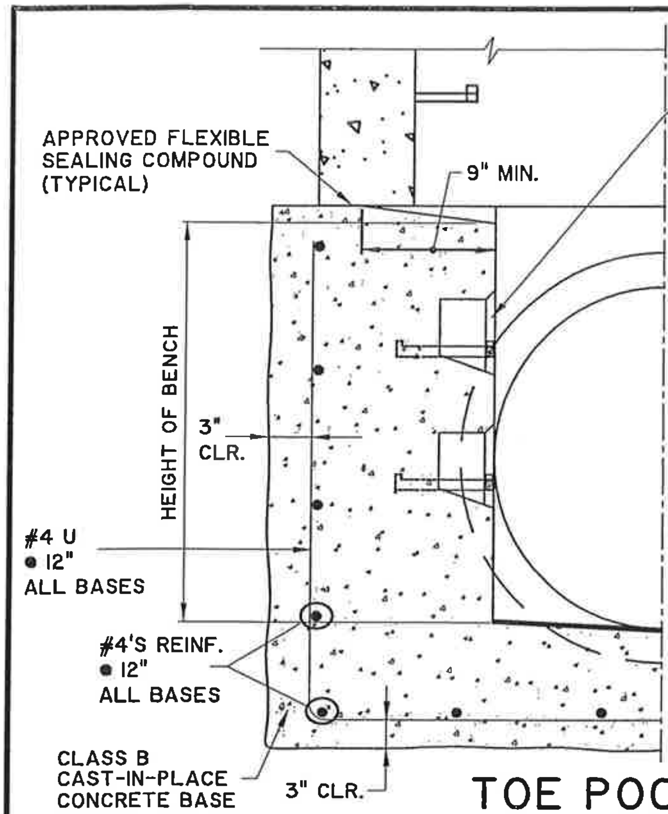
DATE: 12/04

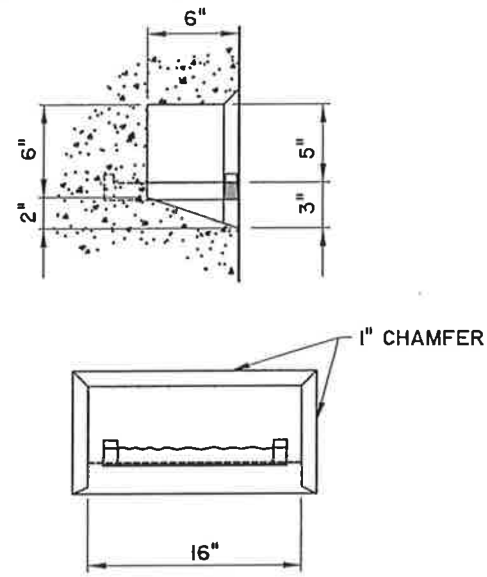
APPROVED:

FJB

TOWN ENGINEER

SHEET FIG. 6-9

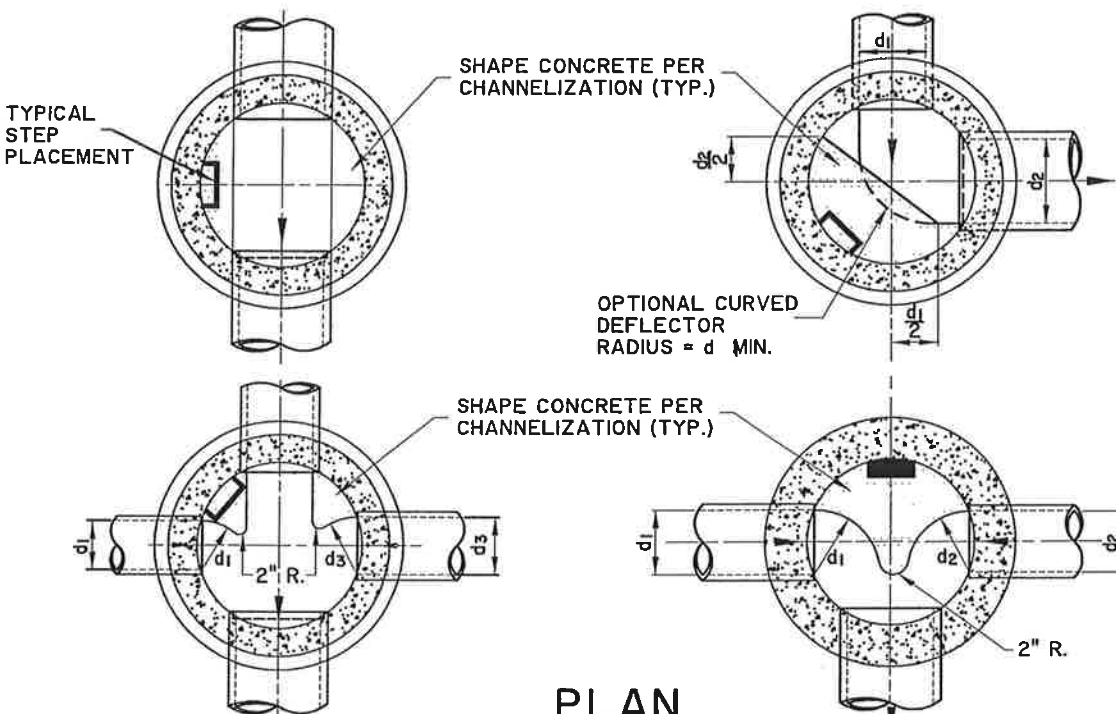




TOE SLOT

TOE POCKET DETAILS

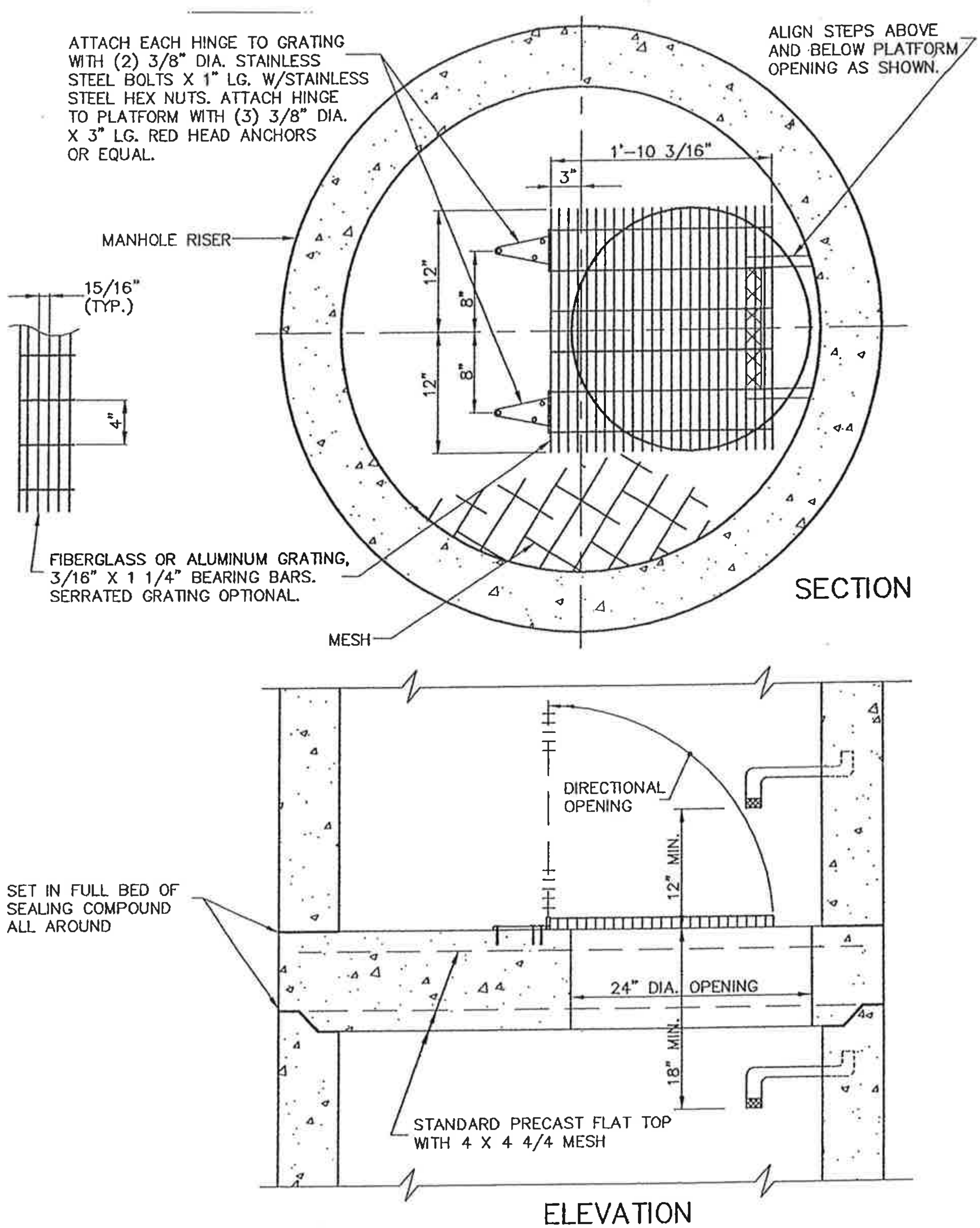
FOR HEIGHT OF BENCH GREATER THAN 30"



PLAN

TOWN OF KERSEY	
STORMWATER MANHOLE TYPICAL BASE CHANNEL DETAILS	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 6–10



TOWN OF KERSEY

INTERMEDIATE PLATFORM FOR MANHOLES OVER 20' IN DEPTH

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG 6–11

7.2 STANDARD INLETS

The standard inlets permitted for use in the Town are:

Inlet Type	Detail	Permitted Use
Curb Opening Inlet, Type R	Reference CDOT M Standards	All street types (See note below) Discourage on rollover curb
Grated Inlet, Type C	Reference CDOT M Standards	All streets with a roadside or median ditch
Grated Inlet, Type 13	Reference CDOT M Standards	Alleys, parking lots or private drives with a valley gutter
Combination Inlet, Type 3	Reference CDOT M Standards	All street types (See note below)

Note: Type R inlets are considered undesirable in standard rollover curb and gutter sections of Local Standard I, Residential Street for the following reasons:

* Greater vertical depression and deeper opening is less desirable for parking cars, children's safety while playing and appearance.
* Rollover to vertical transition disrupts a considerable length of frontage of property and less than attractive at corner radius points.

Type 3 inlets are considered safer and more attractive in residential areas with rollover curb and gutter.

In areas where vertical curbs are installed, particularly where there is no on-street parking, Type R inlets may be acceptable. In areas where large storm flows need to be captured by inlets, particularly with flat street grades, it may be desirable to install Type R inlets. These situations shall be reviewed on a case by case basis.

Other types of combination inlets may be requested as a variance and used only with Town Approval.

Inlets and inlet transitions are prohibited in curb, and street/curb transitions.

7.3 INLET HYDRAULICS

The procedures and basic data used to define the capacities of the standard inlets under various flow conditions were obtained from the USDCM, Volume 1, Section on "Storm Inlets", for curb opening inlets. The procedure consists of defining the amount and depth of flow in the gutter and determining the theoretical flow interception by the inlet. To account for effects which decrease the capacity of the various types of inlets, such as debris plugging, pavement overlaying, and variations in design assumptions, the theoretical capacity calculated for the inlets is reduced to the allowed capacity by the factors presented below for the standard inlets.

Allowable Inlety Capacity		
Condition	Inlet Type	Percentage of Theoretical Capacity Allowed
Sump or Continuous Grade	Type R 5' Length 10' Length 15' Length	 88 92 95
Sump or Continuous Grade	Grated Type 13	50
Continuous Grade	Combination Type 3	66
Sump	Grated Type C	50
Sump	Combination Type 3	65

Allowable standard inlet capacities for the initial storm have been developed and are presented in Figures 7-2, 7-3, and 7-4 for continuous grade and Figure 7-8 for sump conditions. These figures include the reduction factors in the above table. The allowable inlet capacity is compatible with the allowable street capacity (see Section 8). The values shown were calculated on the basis of the maximum flow allowed in the street gutter (or roadside ditch for Type C). For the gutter flow amounts less than the maximum, the allowable inlet capacity must be proportionately reduced.

7.3.1 Continuous Grade Condition

For the continuous grade condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted and some flow will continue past the inlet area (called "inlet carryover"). The amount of carryover must be included in the drainage facility evaluation as well as in the design of the inlet.

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 (Qt) = 8.0 cfs

Find: Interception and carryover amounts for the inlets and flow conditions illustrated on Figure 7-6.

Solution: From Figure 7-6 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 achieved if the inlets are located in the sumps created by street intersections. The following example illustrates how inlet sizing and interception capacity may be analyzed:

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 cfs) = 8.3 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 3	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. In situations where local runoff is not the governing factor, inlets placed on a continuous grade must be spaced at least 50 feet apart in order to pick up carry over flow as indicated by allowable inlet capacity as shown on Figures 7-2, 7-3, and 7-4.

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 Design Engineer should analyze the spacing requirements.

7.5 CHECKLIST

To aid the Designer and Reviewer, the following checklist has been prepared:

1. Check the inlet capacity to determine the carryover flow, and account for this flow plus the local runoff in the sizing of the downstream inlet.

2. Place inlets at the flattest grade or in sump conditions where possible to increase capacity.

3. Space inlets based upon the interception rate of 70 to 80% of the gutter flow to optimize inlet capacity.

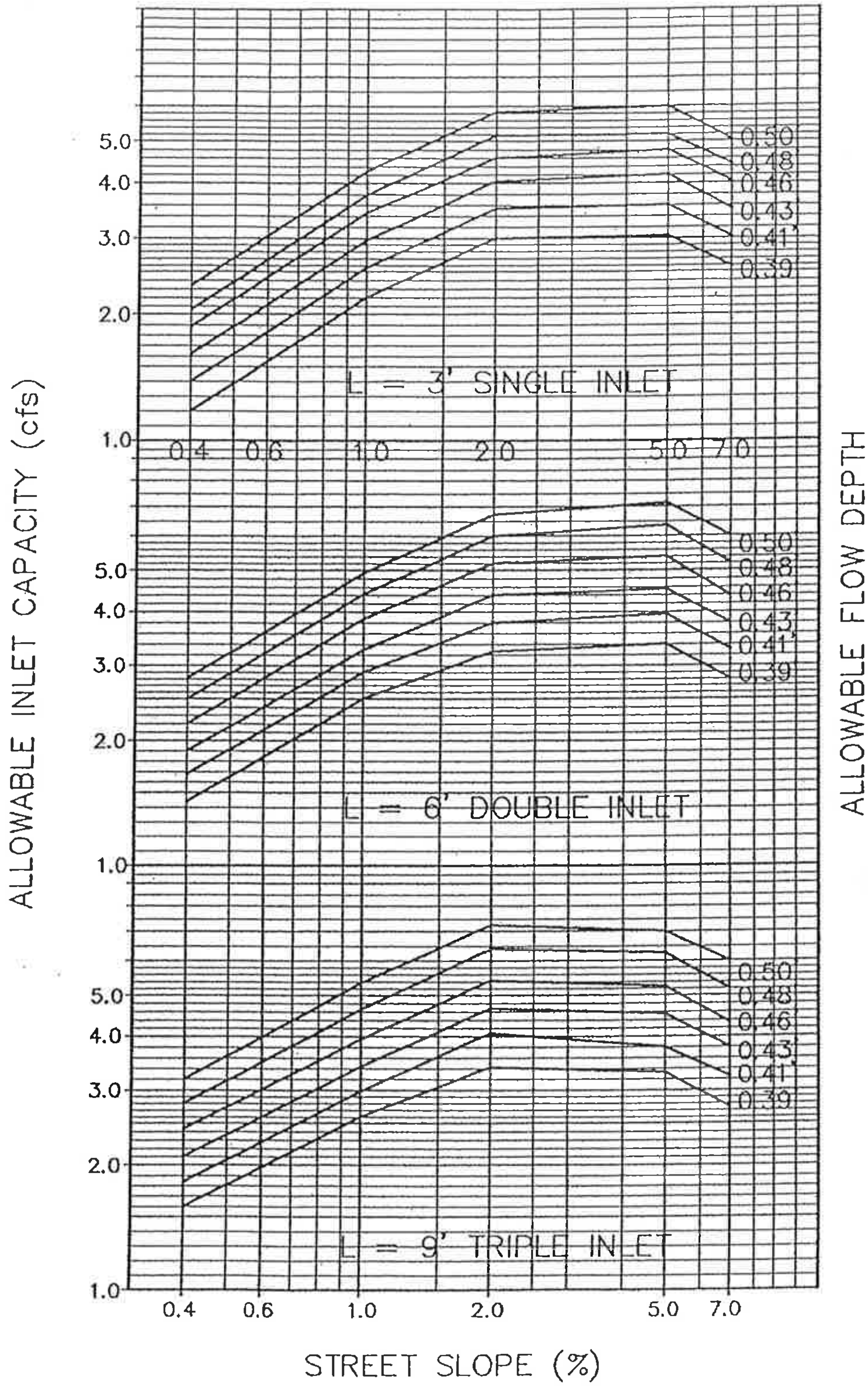
4. Inlet structures shall not be constructed until the curb and gutter has been installed. The Town may allow the inlet structures to be constructed if the curb and gutter has been staked and the stakes can be used to set the inlet structures for line and grade 100 feet in each direction.

FOR THE FOLLOWING INLETS REFERENCE COLORADO DEPARTMENT OF TRANSPORTANTION M&S STANDARDS

1. CURB INLET TYPE R..M-604-12
2. GRATED INLET TYPE C..................................M-604-10
3. GRATED INLET TYPE 13 (GENERAL)................M-604-13
4. GRATED INLET TYPE 13 (FOR VERTICAL FACE CURB...................
5. GRATED INLET TYPE 13 (FOR ROLLOVER CURB)...................
5. COMBINATION INLET TYPE 3 (FOR VERICAL FACE CURB)...................
5. COMBINATION INLET TYPE 3 (FOR ROLLOVER CURB)...................

TOWN OF KERSEY	
INLETS (REFERENCE TO CDOT)	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG 7-1



NOTES:

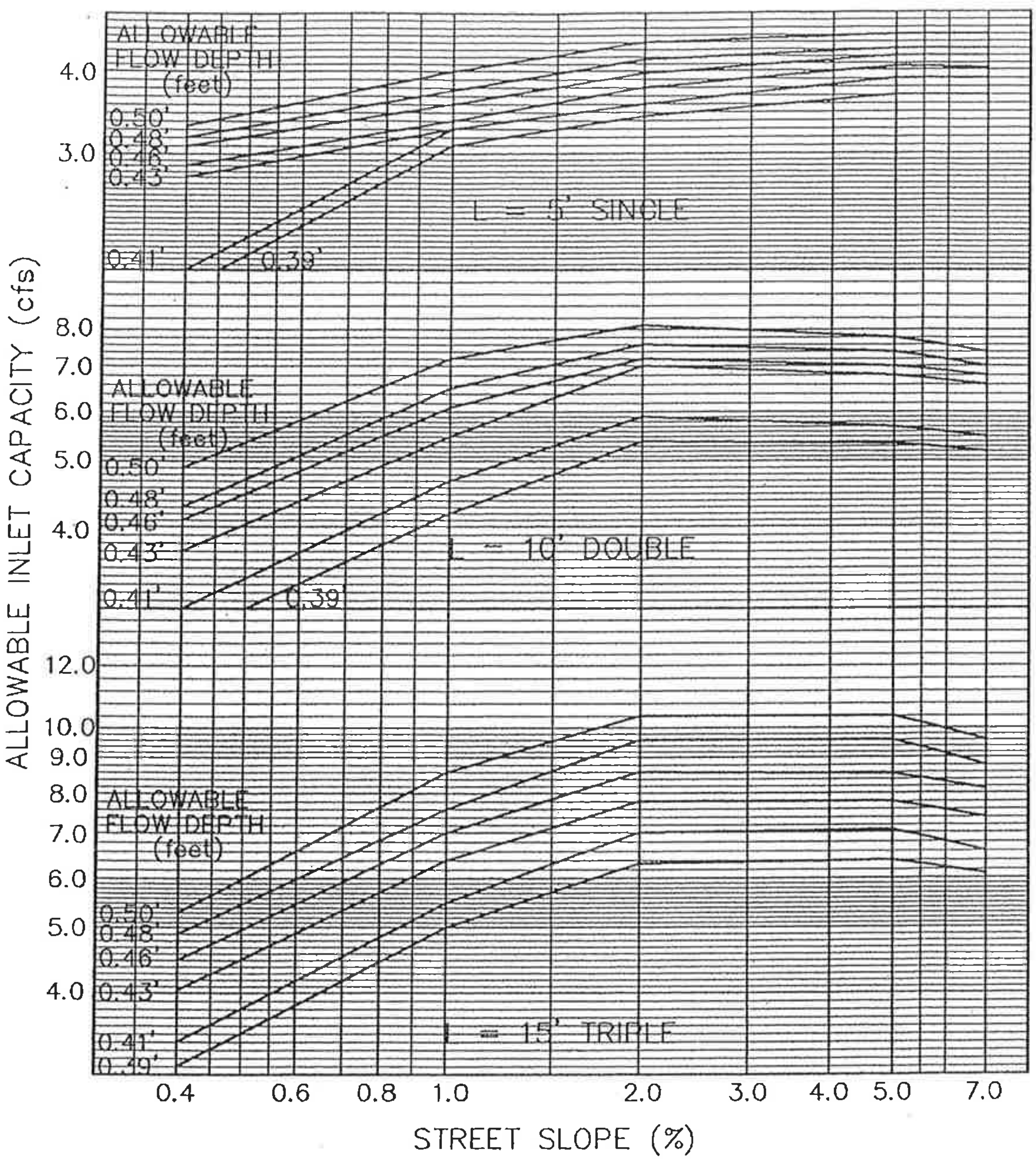
1. ALLOWABLE CAPACITY = 66% THEORETICAL CAPACITY
2. MAXIMUM INLET CAPACITY AT MAXIMUM ALLOWABLE FLOW DEPTH. PROPORTIONALLY REDUCE FOR OTHER DEPTHS.

TOWN OF KERSEY

ALLOWABLE INLET CAPACITY TYPE 3 COMBINATION ON A CONTINUOUS GRADE

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

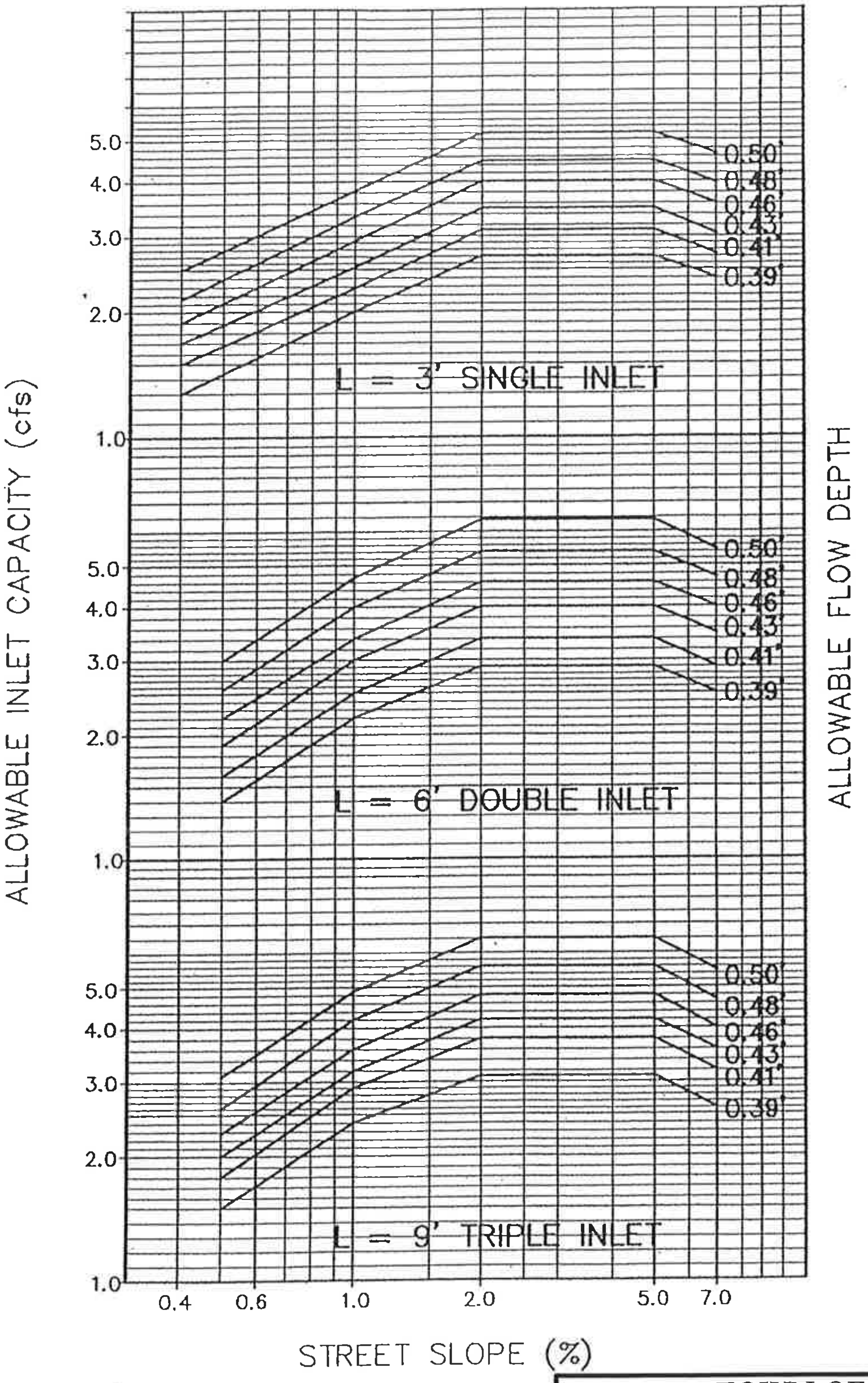
SHEET FIG. 7-2



NOTES:

1. MAXIMUM INLET CAPACITY AT MAXIMUM ALLOWABLE FLOW DEPTH. PROPORTIONALLY REDUCE FOR OTHER DEPTHS.
2. ALLOWABLE CAPACITY =
   88% (L = 5')
   92% (L = 10') OF THEORETICAL CAPACITY
   95% (L = 15')
3. INTERPOLATE FOR OTHER INLET LENGTHS.

TOWN OF KERSEY	
ALLOWABLE INLET CAPACITY TYPE R CURB OPENING ON A CONTINUOUS GRADE	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	



NOTES:
1. ALLOWABLE CAPACITY = 60% THEORETICAL CAPACITY
2. MAXIMUM INLET CAPACITY AT MAXIMUM ALLOWABLE FLOW DEPTH. PROPORTIONALLY REDUCE FOR OTHER DEPTHS.

TOWN OF KERSEY

ALLOWABLE INLET CAPACITY TYPE 13 GRATED ON A CONTINUOUS GRADE

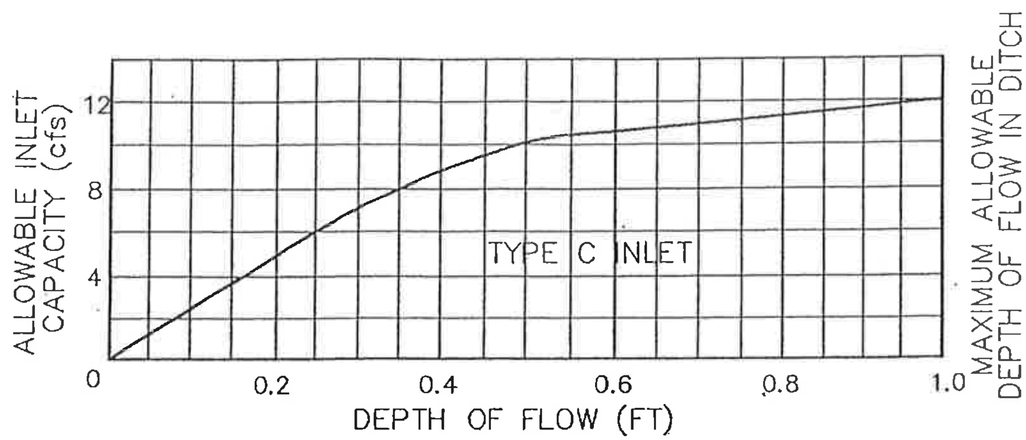
SCALE: NONE

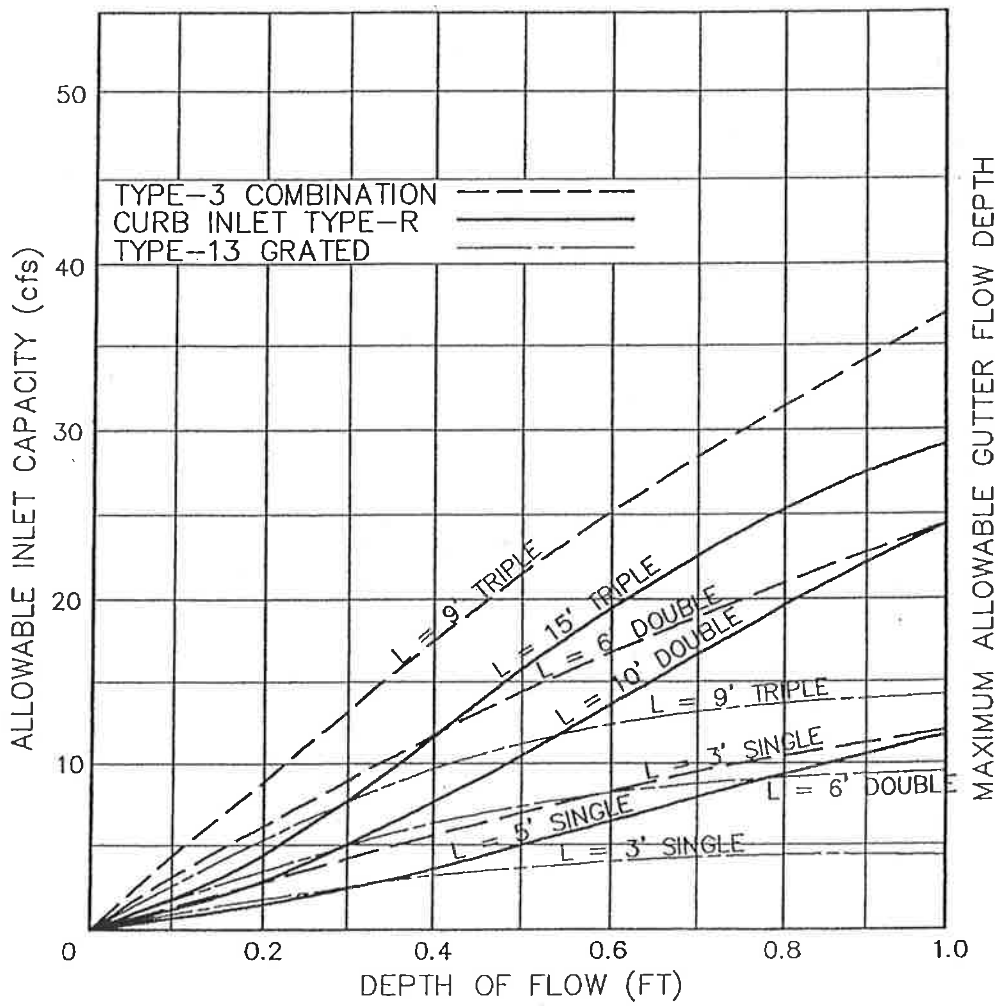
DATE: 12/04

APPROVED: FJB

TOWN ENGINEER

SHEET FIG. 7-4

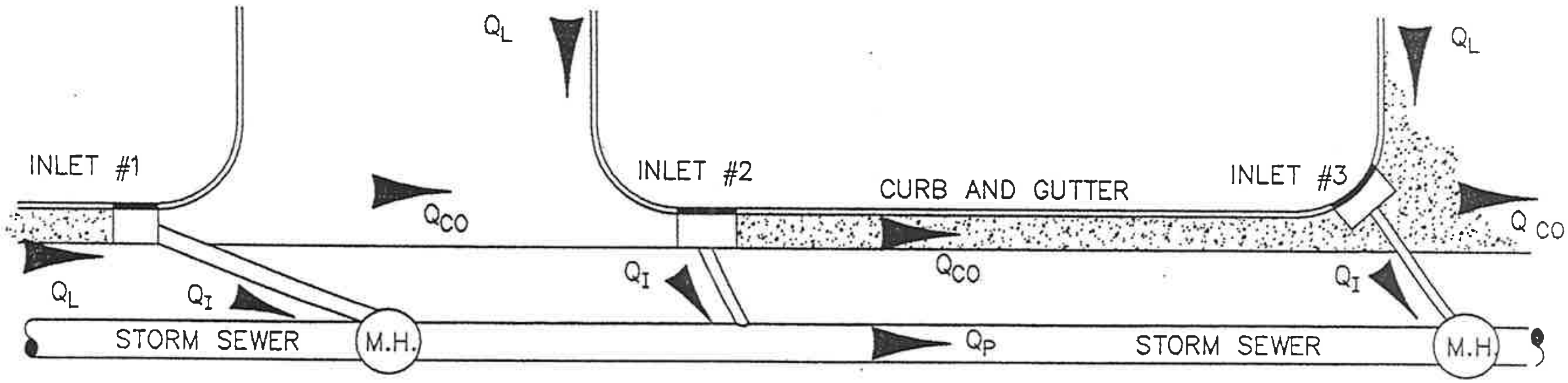




TOWN OF KERSEY

ALLOWABLE INLET CAPACITY SUMP CONDITIONS – ALL INLETS

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	



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 – 7-6 AND 7-8

TOWN OF KERSEY

INLET DESIGN EXAMPLES INITIAL STORM

SCALE: NONE

DATE: 12/04

APPROVED: FJB

TOWN ENGINEER

SHEET FIG. 7-6

SECTION 8.0 STREETS

8.1 INTRODUCTION

The criteria presented in this section shall be used in the evaluation of the allowable drainage encroachment within public streets. The review of all submittals shall be based on the criteria herein.

8.2 FUNCTION OF STREETS IN THE DRAINAGE SYSTEM

Urban and rural streets, specifically the curb and gutter or the roadside ditches, are part of the storm drainage system. When the drainage in the street exceeds allowable limits, a storm sewer system or an open channel is required to convey the excess flows. However, the primary function of the urban street system is for traffic movement, and, therefore, the drainage function is secondary and must not interfere with the traffic function of the street.

Design criteria for the collection and moving of runoff water on public streets are based on a reasonable frequency and magnitude of traffic interference. Depending on the character of the street, certain traffic lanes can or cannot be inundated during specific design storm runoff events. The primary function of the streets during the initial storm runoff event is to convey the nuisance flows quickly and efficiently to the storm sewer or open channel drainage without interference with traffic movement. During the major storm runoff event the function of the streets is to provide a passageway for the flood flows with minimal damage to the urban environment, and passage of emergency vehicles.

8.3 STREET CLASSIFICATIONS AND CAPACITY LIMITATIONS

8.3.1 Street Classifications

The streets in the Town are classified for drainage use according to the average daily traffic (ADT) for which the street is designed. The larger the ADT, the more restrictive the allowable drainage encroachment into the driving lanes is.

Town of Kersey Street Classifications		
Classification	Width (Flowline to Flowline)	Town of Kersey Standard Detail No.
Local – Low Volume	No Curb & Gutter Shoulder Only w/Ditch	Figure 8-4
Local Street	38'	Figure 8-5
Residential Collector	40'	Figure 8-6
Major Collector	40'	Figure 8-7
Arterial	60'	Figure 8-8
Major Arterial	68'	Figure 8-9

8.3.2 Street Capacity - Initial Storm

The street capacity for initial storm runoff events is determined by the limitations set forth below:

Street Capacity for Initial Storm Runoff		
Street Classification	Curb Overtopping Allowed	Maximum Pavement Encroachment
Local[1] Minor Collector	No	Flow may spread to crown of street
Major Collector	No	Flow must leave a minimum 10' wide center lane open
Minor Arterial	No	Flow must leave a minimum of one traffic lane open each direction
Major Arterial	No	Flow must leave a minimum of one traffic lane open each direction

*Note:
[1]For Local – Low Volume Streets see Section 5.6 of these Criteria for the design and capacity of roadside ditches.

8.3.3 Street Capacity - Major Storm

The street capacity during major storm events is determined by the limitations set forth below:

Street Capacity for Major Storm Runoff[1]			
Street Classification	Maximum Depth at Gutter Flowline	Maximum Depth at Crown	Allowable Inundation
Local[2] & Collector	18"	N/A	No inundation at groundline[3]
Arterial	18"	6"	No inundation at groundline[3]

*Note:

[1]Most restrictive condition shall control design.

[2]For Local – Low Volume Streets see Section 5.6 of these Criteria for the design and capacity of roadside ditches.

[3]Includes inundation of residential dwellings, public, commercial and industrial buildings

8.4 HYDRAULIC EVALUATION FOR STREET CAPACITY

8.4.1 Allowable Street Flow - Initial Storms

The determination of the allowable street capacity shall be based on the following procedure: determine the theoretical capacity based on the street cross section; compute the street flow; then, apply the appropriate reduction factor to calculate the allowable street capacity.

Based on the allowable encroachment for the various street classifications (as presented in Section 8.3) the theoretical capacity of each street section (except for local, low volume streets with borrow ditches) is calculated using the Modified Manning's formula shown below:

$$Q = (0.56)\ (Z/n) S^{1/2} d^{8/3} \qquad \text{(Equation 8.1)}$$

Where:

Q = discharge in cfs
$Z = 1/S_x$, where S_x is the cross slope of the pavement (ft/ft)
d = depth of water at face of curb (feet)
S = longitudinal grade of street (ft/ft)
n = Manning's roughness coefficient

If Equation 8.1 is used, the allowable gutter capacity shall be calculated by multiplying the theoretical capacity obtained by the appropriate reduction factor found in Figure 8-2 and 8-3. The purpose of the reduction factor is to account for various street conditions, which decrease the street capacity. These conditions may include street overlays, parked vehicles, debris and hail accumulation, and deteriorated pavement.

The allowable gutter capacity for each standard, symmetrical street section has been calculated, and is presented in Figure 8-2 and Figure 8-3. The calculations were performed for various allowable flow depths and street slopes. A Manning's n-value of 0.016 was used for the calculations at all street slopes. The allowable gutter capacity was computed by multiplying the theoretical capacity by the appropriate reduction factor from Figure 8-2 and 8-3.

The allowable gutter capacity will need to be reduced if nonsymmetrical street sections are encountered. Street capacity calculations at critical locations of non-symmetrical street sections shall be submitted to the Town for review.

8.4.2 Allowable Street Flow - Major Storms

The street capacity for the major storm is determined by the depth and inundation limits set forth in Section 8.3.3. To determine the allowable street capacity the maximum theoretical capacity for the street section is calculated using the Manning's formula, and then the appropriate reduction factors from Figure 8-2 and 8-3 is applied. The Manning's formula is utilized by dividing the street cross section into the pavement and the sidewalk/grass areas and then summing the individual flow contributions.

A Manning's value of 0.016 for the pavement areas and 0.025 for the sidewalk/grass areas was used to determine capacity. The maximum allowable depth at the gutter is 18 inches. The street capacity criteria for both the initial and major storms is graphically displayed on Figures 8-2 and 8-3.

8.4.3 Rural Streets (Local, Low-volume Streets without Curb and Gutter)

Rural streets are characterized by the use of roadside ditches instead of curb and gutters. The capacity is limited by the depth in the ditch and the maximum flow velocity. Refer to Section 5.6 for the design and capacity of roadside ditches.

8.5 ALLOWABLE STREET CROSS-FLOW CONDITIONS

8.5.1 Cross Street Flow at Intersections

Cross street flow normally occurs at converging street intersections where the flow must cross from one side to the other, in either a cross pan (where allowed) or across the street crown. The restrictions for flow depth at intersections are set forth below:

Allowable Cross Street Flow at Intersections		
Street Classification	Initial Storm Runoff	Major Storm Runoff
Local	Maximum 6" Depth at Street Crown or in Cross Pan	Maximum 18" Depth Above Gutter Flowline
Collector	Maximum 6" Depth Above Cross Pan Flowline (Where Cross Pan is Allowed)	Maximum 18" Depth Above Gutter Flowline
Minor Arterial	None Allowed	Maximum 6" Depth Above Crown
Major Arterial	None Allowed	Maximum 6" Depth Above Crown

8.5.2 Street Overtopping

In locations of culvert crossings, the opportunity for the flow in the drainage way to exceed the road culvert capacity and subsequently overtop the crown of the street must be investigated. The restrictions for street overtopping are set forth below:

Allowable Culvert Overtopping		
Street Classification	10-Year Storm Maximum Depth	Major Storm Maximum Depth[1]
Local	None	18" at the Gutter Flowline
Local w/Roadside Ditch	None	6" at the Street Crown
Collector & Arterial	None	6" at the Street Crown
Major Arterial	None	No Overtopping Allowed For bridges, the minimum clearance between the low chord and the EGL shall be 6"

Note:
[1]The maximum headwater for the 100-year design flows shall be 1.5 times the culvert diameter or 1.5 times the rise dimension for pipe shapes other than round.

8.6 DESIGN EXAMPLE

Determination of Street Capacity

Given: Street with a traffic classification of "Minor Collector" and a slope of 1.0 percent.

Find: Maximum allowable capacity for initial and major storm.

Solution:

STEP 1: Determine the allowable depth:

From Section 8.3, for a Minor Collector, the maximum depth at the curb (without overtopping) would be 6" for the initial storm.

STEP 2: Determine the allowable initial storm gutter capacity:

From Figure 8-3, for a "Minor Collector" with an allowable depth of 0.50 feet and a slope of 1.0 percent, read the allowable gutter capacity of 8.8 cfs.

STEP 3: Determine the allowable major storm street capacity:

From Figure 8-4 for a "Minor Collector" with a slope of 1.0 percent, read the allowable capacity of 528 cfs for the full street section, assuming the street is symmetrical.

8.7 CHECKLIST

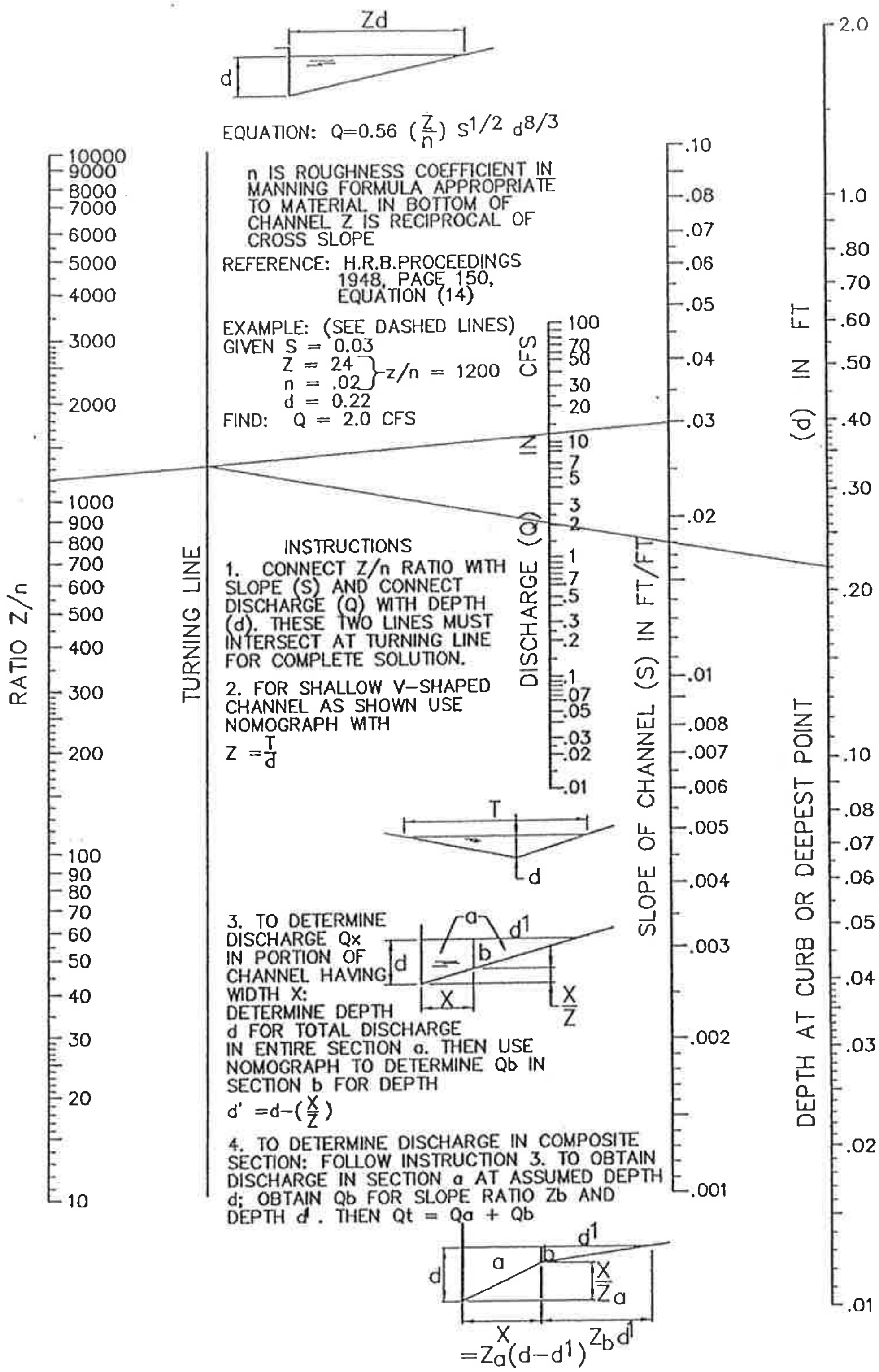
To aid the Designer and Reviewer, the following checklist has been prepared:

1. Determine the street classification first and then the allowable flow depth and gutter capacity.
2. Use the flattest street slope to determine the gutter capacity.
3. If the Modified Manning's equation is used to calculate the allowable street flow, use the appropriate reduction factor (F) to calculate the allowable gutter capacity.
4. Check for non-symmetrical street evaluation.

5. Check for cross-flow conditions at intersections and allowable culvert overtopping depths.

6. Storm sewer required for gutter capacity exceedance.

7. Check adequacy of downstream facilities.



EQUATION: $Q=0.56\ (\frac{Z}{n})\ S^{1/2}\ d^{8/3}$

n IS ROUGHNESS COEFFICIENT IN MANNING FORMULA APPROPRIATE TO MATERIAL IN BOTTOM OF CHANNEL Z IS RECIPROCAL OF CROSS SLOPE

REFERENCE: H.R.B.PROCEEDINGS 1948, PAGE 150, EQUATION (14)

EXAMPLE: (SEE DASHED LINES)
GIVEN S = 0.03
Z = 24 } z/n = 1200
n = .02 }
d = 0.22
FIND: Q = 2.0 CFS

INSTRUCTIONS

1. CONNECT Z/n RATIO WITH SLOPE (S) AND CONNECT DISCHARGE (Q) WITH DEPTH (d). THESE TWO LINES MUST INTERSECT AT TURNING LINE FOR COMPLETE SOLUTION.

2. FOR SHALLOW V-SHAPED CHANNEL AS SHOWN USE NOMOGRAPH WITH $Z = \frac{T}{d}$

3. TO DETERMINE DISCHARGE Qx IN PORTION OF CHANNEL HAVING WIDTH X: DETERMINE DEPTH d FOR TOTAL DISCHARGE IN ENTIRE SECTION a. THEN USE NOMOGRAPH TO DETERMINE Qb IN SECTION b FOR DEPTH $d' = d - (\frac{X}{Z})$

4. TO DETERMINE DISCHARGE IN COMPOSITE SECTION: FOLLOW INSTRUCTION 3. TO OBTAIN DISCHARGE IN SECTION a AT ASSUMED DEPTH d; OBTAIN Qb FOR SLOPE RATIO Zb AND DEPTH d'. THEN Qt = Qa + Qb

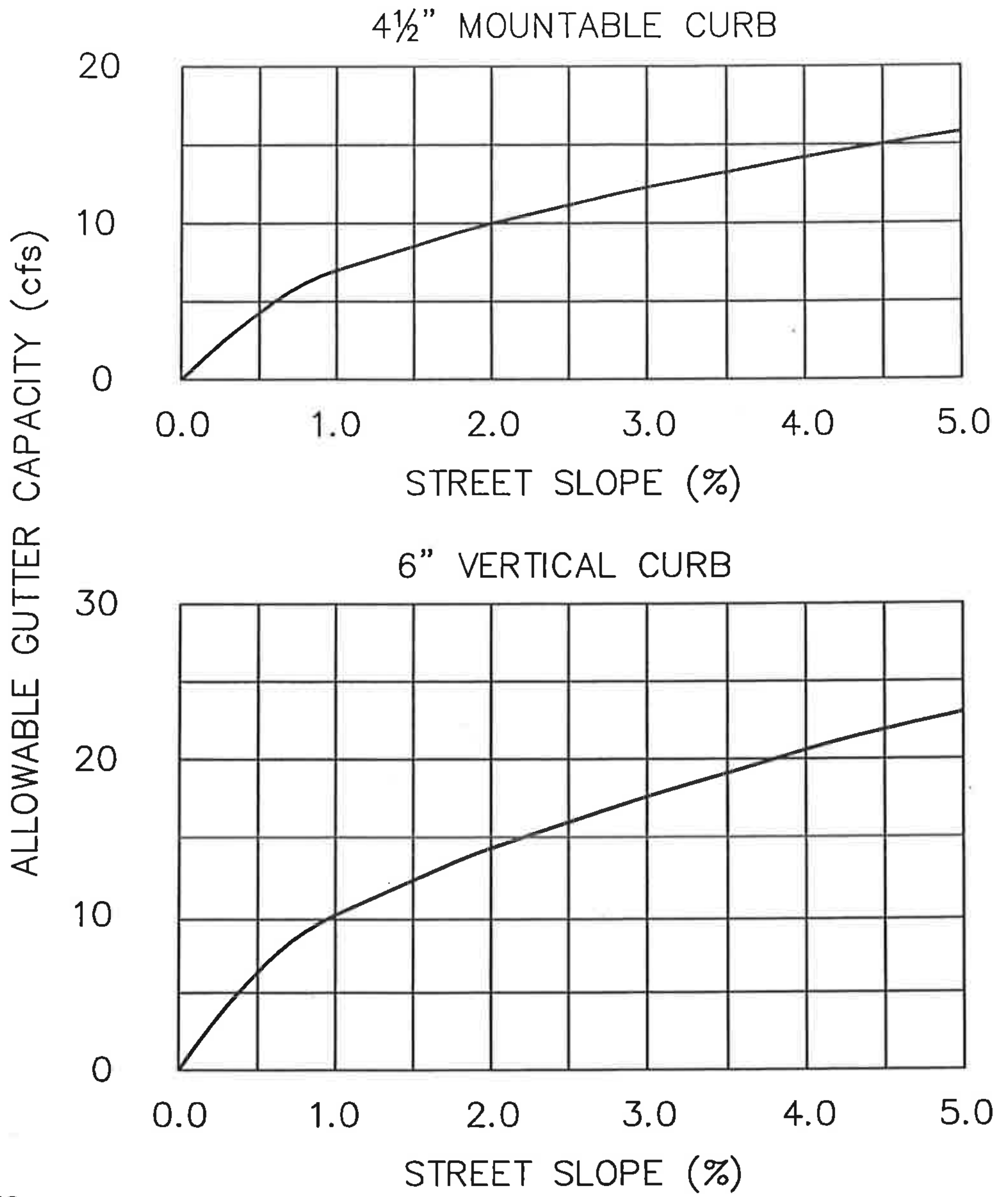
TOWN OF KERSEY

NOMOGRAPH FOR FLOW IN TRIANGULAR GUTTERS

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8-1

STREET CAPACITY - MINOR STORM



NOTES:

1. MORE RESTRICTIVE CRITERIA MAY BE PLACED UPON STREETS DESIGNATED AS ARTERIAL ROADWAYS AT THE DISCRESSION OF THE TOWN ENGINEER.
2. MAXIMUM DEPTH AT FLOWLINE = 6' INCHES (for 6" vertical curb).
   MAXIMUM DEPTH AT FLOWLINE = 4½" INCHES (for 4½" mountable curb).
3. n = 0.016 (streets)
   n = 0.025 (grass)

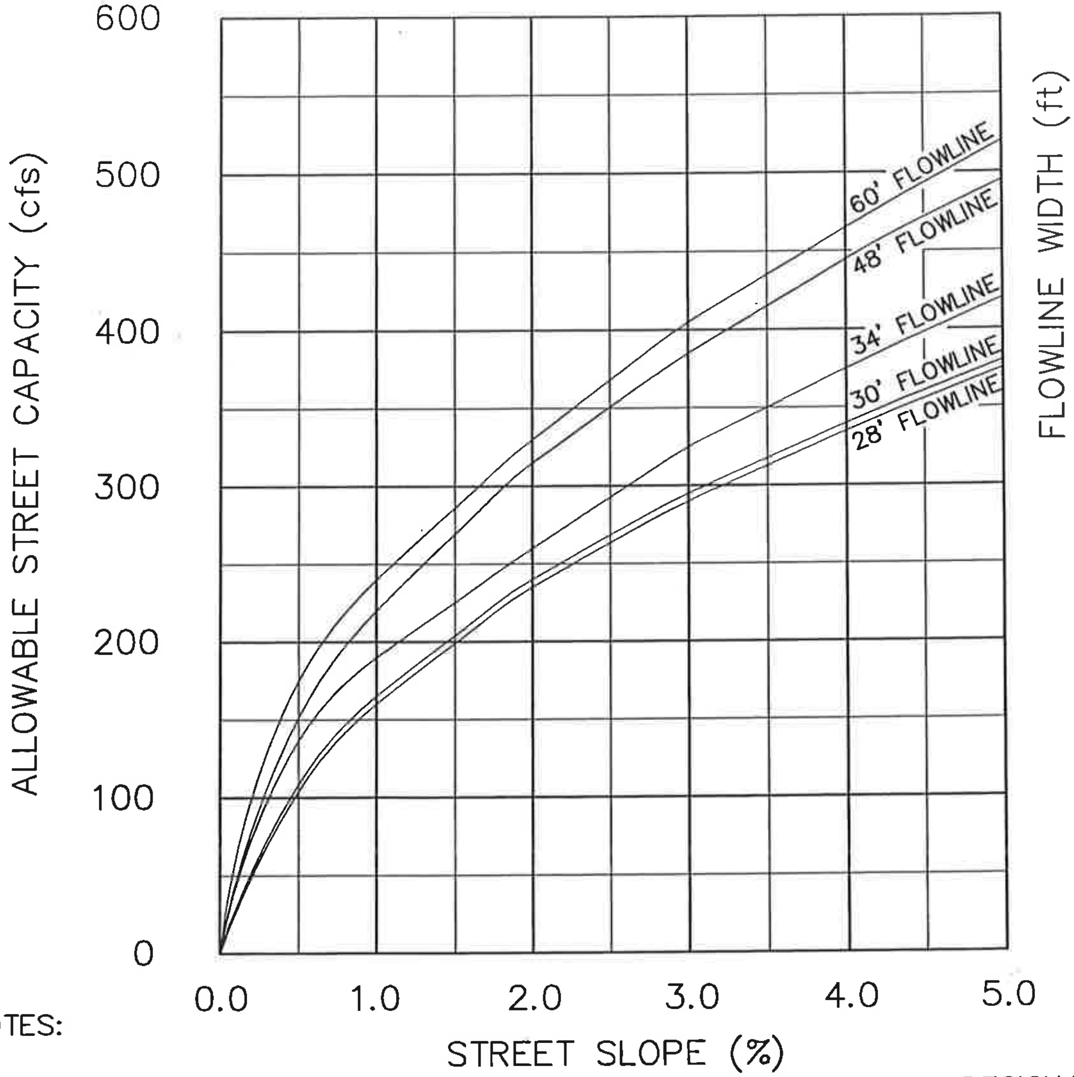
TOWN OF KERSEY

STREET CAPACITY

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8–2

STREET CAPACITY - MAJOR STORM



NOTES:

1. MORE RESTRICTIVE CRITERIA MAY BE PLACED UPON STREETS DESIGNATED AS ARTERIAL ROADWAYS AT THE DISCRESSION OF THE TOWN ENGINEER.

2. MAXIMUM DEPTH AT FLOWLINE = 12 INCHES.

3. FOR FLOWLINE WIDTHS OTHER THAN THOSE LISTED HERE INTERPOLATE TO FIND THE ALLOWABLE STREET CAPACITY.

4. FOR STREETS WITH FLOWLINE WIDTHS GREATER THAN 60 FEET USE A 60 FOOT FLOWLINE WIDTH. FOR STREETS WITH SLOPES GREATER THAN 5.0% USE A 5.0% STREET SLOPE.

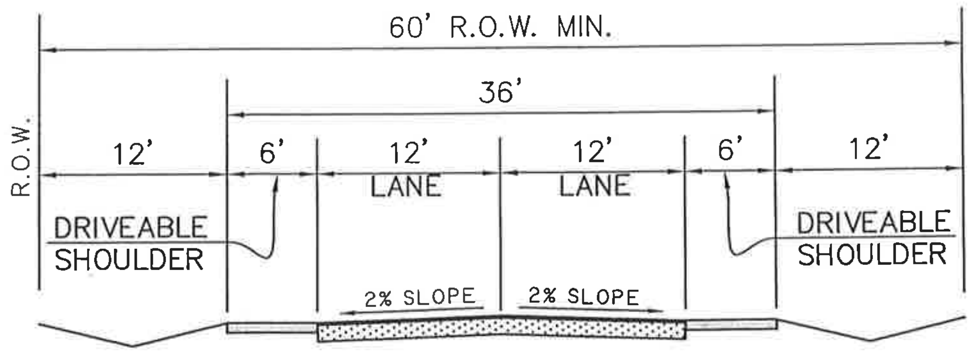
$n = 0.016$ (streets)
$n = 0.025$ (grass)

TOWN OF KERSEY

STREET CAPACITY

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8–3



RURAL LOCAL STREET

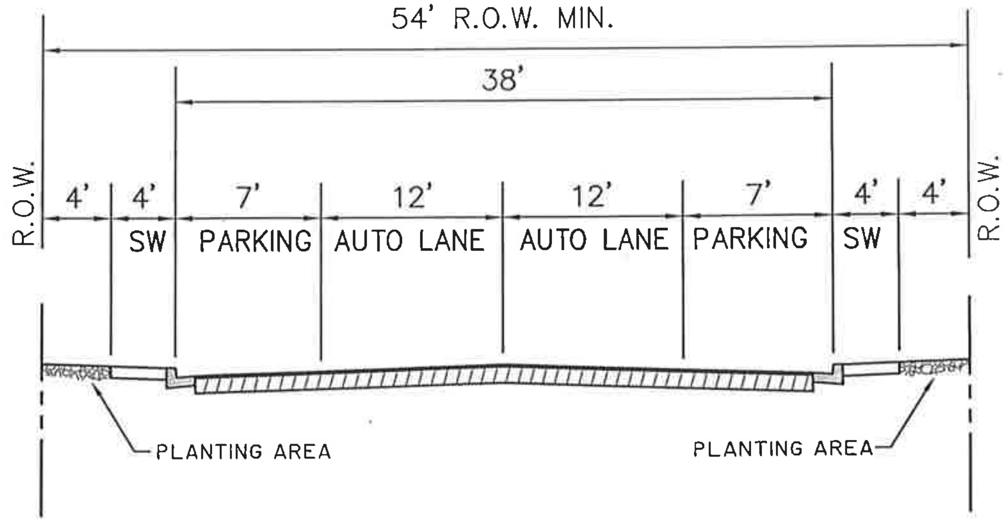
SINGLE FAMILY (LARGE LOT)
RESIDENTIAL

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN INDICATED.
3. UTILITY EASEMENTS ARE LOCATED OUTSIDE THE RIGHT-OF-WAY.

TOWN OF KERSEY	
STANDARD ROADWAY SECTION RURAL LOCAL STREET	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8-4



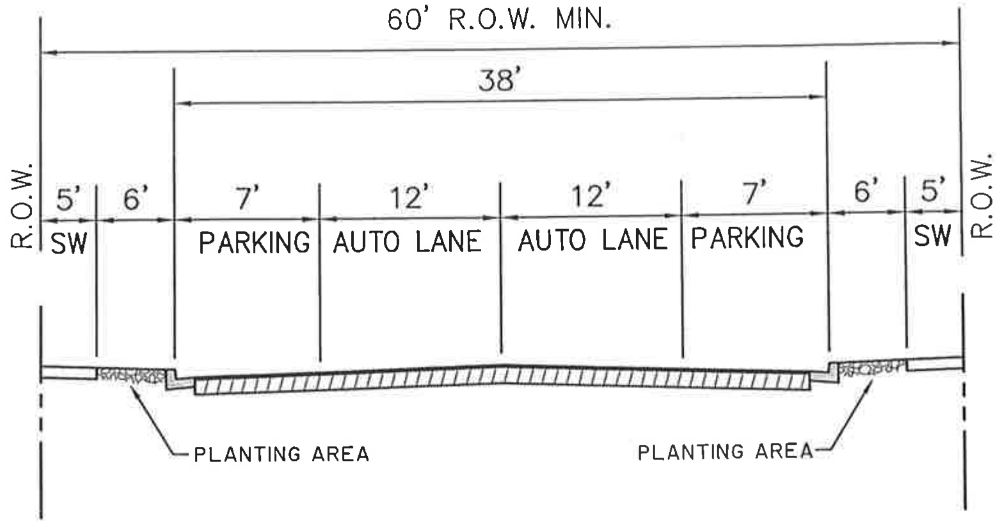
LOCAL STREET

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER.
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN INDICATED.

TOWN OF KERSEY	
STANDARD ROADWAY SECTION LOCAL STREET	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8-5-A



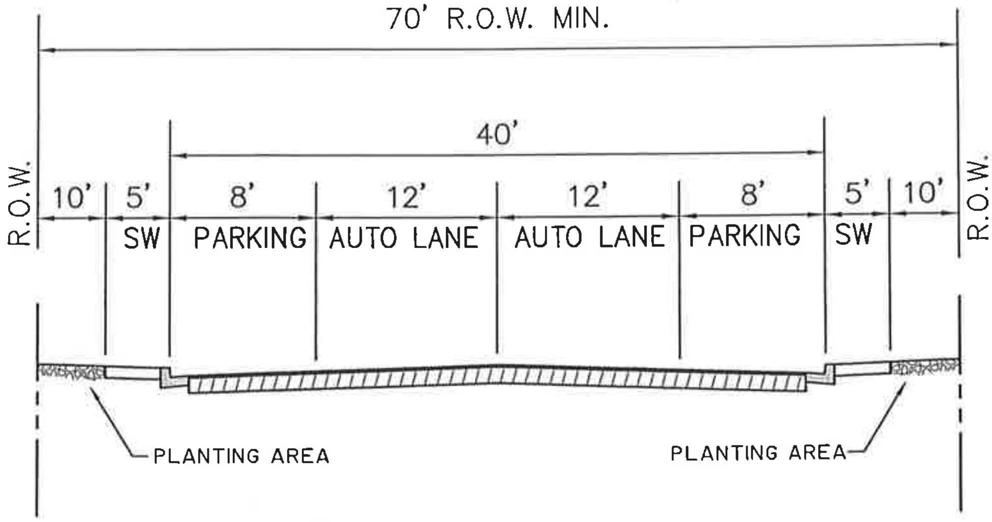
LOCAL STREET
WITH DETACHED SIDEWALK

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER.
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN INDICATED.

TOWN OF KERSEY	
STANDARD ROADWAY SECTION LOCAL STREET	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8-5-B

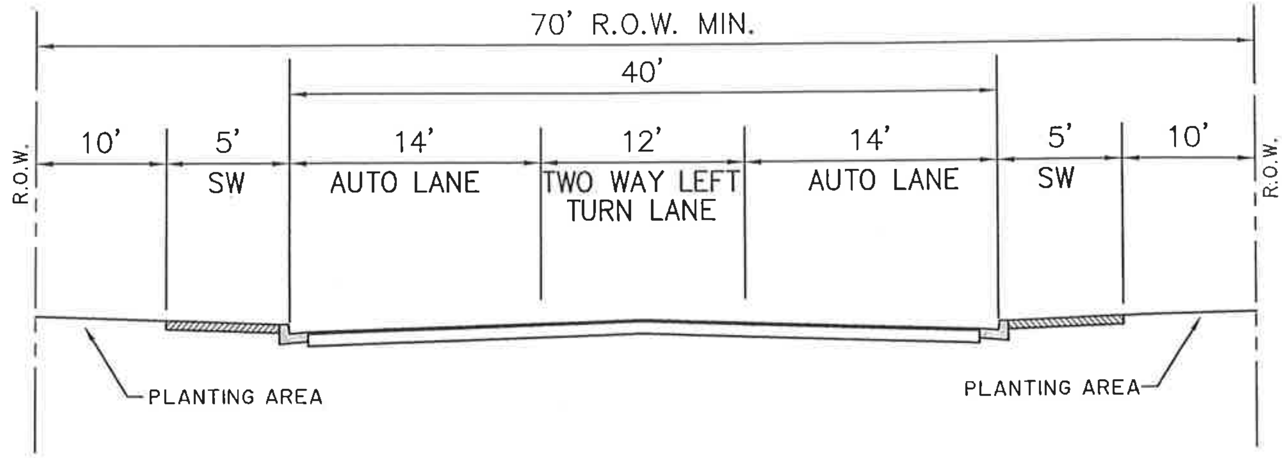


RESIDENTIAL COLLECTOR

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER.
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN INDICATED.

TOWN OF KERSEY	
STANDARD ROADWAY SECTION RESIDENTIAL COLLECTOR	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	



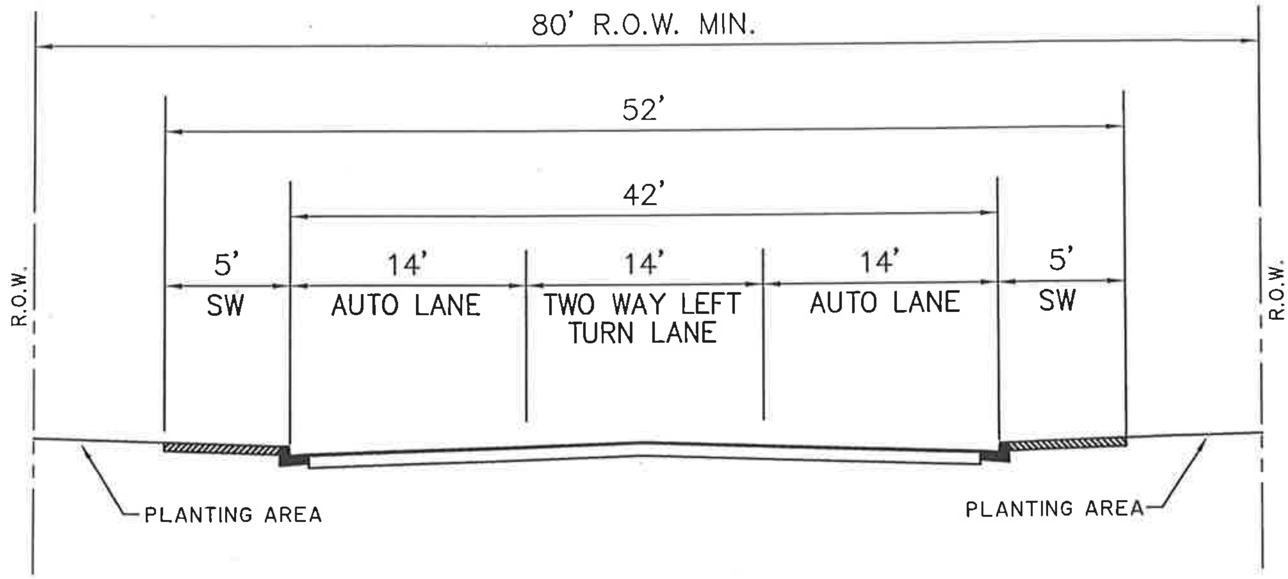
MAJOR COLLECTOR

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN INDICATED.
3. TURN LANES MAY BE NEEEDED AT INTERSECTIONS
4. BICYCLE LANES MAY BE REQUIRED FOR DESIGNATED ROUTES.

TOWN OF KERSEY	
STANDARD ROADWAY SECTION MAJOR COLLECTOR	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8-7



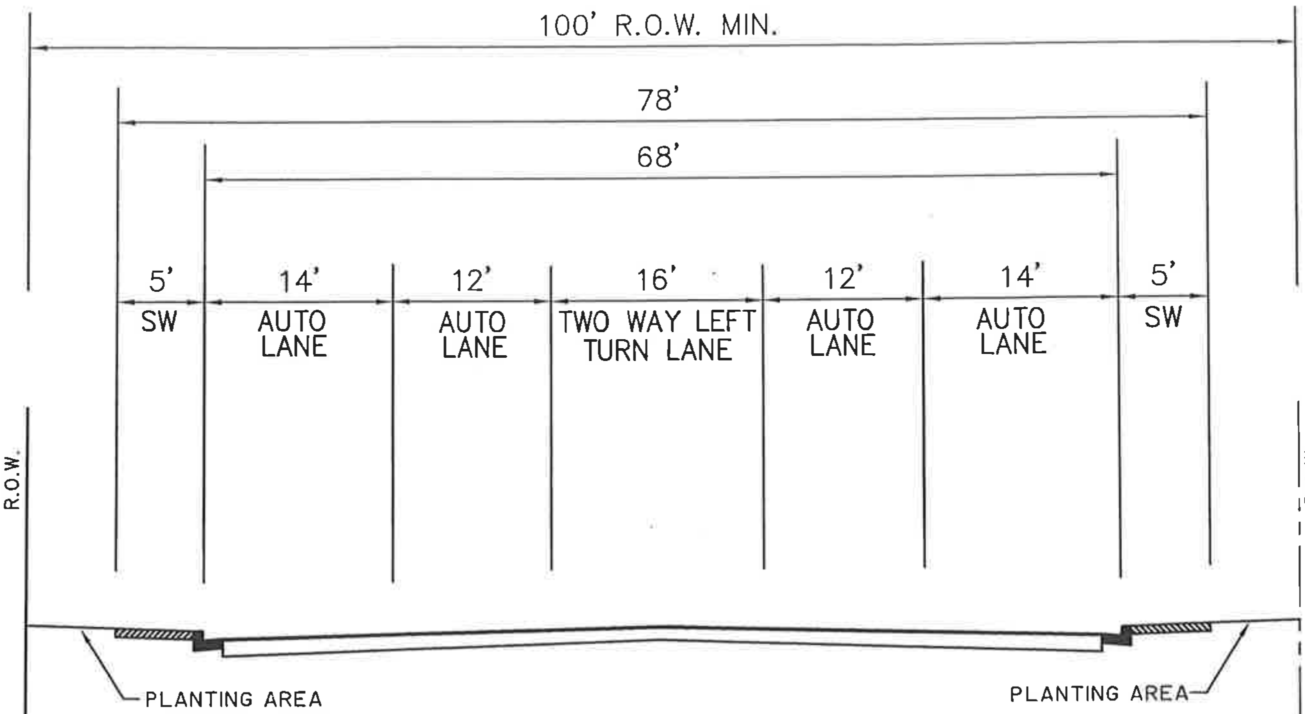
ARTERIAL

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN THE MAXIMUM DIMENSION INDICATED.
3. ADDITIONAL TURN LANES MAY BE NEEDED AT INTERSECTIONS.
4. BICYCLE LANES MAY BE REQUIRED ON DESIGNATED ROUTES.
5. PRIVATE DRIVEWAYS SHALL BE LIMITED TO THOSE LOCATIONS WHERE NO ACCPETABLE ALTERNATE ACCESS IS AVAILABLE.

TOWN OF KERSEY	
STANDARD ROADWAY SECTION ARTERIAL	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8–8



MAJOR ARTERIAL

NOTES:

1. THIS ROADWAY SECTION MAY BE MODIFIED IF APPROVED BY THE TOWN ENGINEER
2. RIGHT-OF-WAY WIDTH MAY BE GREATER THAN INDICATED.
3. ADDITIONAL TURN LANES MAY BE NEEDED AT INTERSECTIONS.
4. BICYCLE LANES MAY BE REQUIRED ON DESIGNATED ROUTES.
5. PRIVATE DRIVEWAYS SHALL BE LIMITED TO THOSE LOCATIONS WHERE NO ACCPETABLE ALTERNATE ACCESS IS AVAILABLE.
6. AN INTERIM ROADWAY SECTION CONSISTENT WITH AN ARTERIAL STREET MAY BE APPROVED IF DETERMINED ACCEPTABLE TO SERVE TRAFFIC DEMANDS.

TOWN OF KERSEY

STANDARD ROADWAY SECTION
MAJOR ARTERIAL

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 8-9

SECTION 9.0 CULVERTS

9.1 INTRODUCTION

A culvert is defined as a conduit for the passage of surface drainage water under a highway, railroad, canal, or other embankment (except detention outlets). Culverts may be constructed with many shapes and materials. Reinforced concrete pipe (RCP) is available in round, elliptical, or arch cross sections, in sizes typically ranging from 12 inches to 108 inches in diameter.

Corrugated metal pipe (CMP), Aluminized Steel (ASP) and aluminum pipe culverts are available in round or arch cross sections. Sections of corrugated metal can also be bolted together to form several other cross sectional shapes, such as elliptical and pear shapes, forming structural plate pipe (SPP). Corrugations also come in various dimensions, which affect the hydraulics of the pipe flow.

Reinforced concrete box culverts (RCBC) can be constructed with generally any rectangular cross section, the only limitations being the physical site constraints and the structural requirements. Precast box culverts are also available in several standard dimensions.

9.2 CULVERT HYDRAULICS

The procedures and basic data to be used for the hydraulic evaluation of culverts in the Town shall be in accordance with the USDCM, Volume 2, "Inlets and Culverts", Section 2, Hydraulics, except as modified herein. The reader is also referred to the many texts and publications covering the subject for additional information.

9.3 CULVERT DESIGN STANDARDS

9.3.1 Construction Material and Pipe Size

Within the Town, culverts shall be constructed from corrugated steel, Aluminized Steel (ASP), Plastic Pipe (PVC) or aluminum, concrete, corrugated metal, or high-density polyethylene pipe (HDPE). Standards for the use of these materials are presented in Section 6.2 of these Criteria. Other materials for construction shall be subject to the approval of the Town Engineer.

The minimum pipe size for culverts within a public ROW shall be a 24-inch diameter round culvert, or shall have a minimum cross sectional area of 2.8 ft2 for arch shapes, and 3.3 ft2 for elliptical shapes. Roadside ditch culverts for driveways shall be a minimum of 12-inch diameter round, or have a minimum cross sectional area of 0.79 ft2.

9.3.2 Inlet and Outlet Configuration

Within the Town, all culverts are to be designed with headwalls and wing walls or with flared-end sections at the inlet and outlet. Flared-end sections are only allowed on pipes with diameters of 42 inches (or equivalent) or less.

Additional protection in the form of riprap or concrete will also be required at the inlet and outlet due to the potential scouring velocities. Refer to Sections 10.2 and 10.3.

Headwalls, wing walls, and flared-end sections should be designed and constructed to complement the existing landforms of the site and blend with the natural surrounding environment, to the greatest extent possible.

9.3.3 Hydraulic Data

When evaluating the capacity of a culvert, the following data shall be used:

1. Roughness Coefficient: see Table 9-1.

2. Entrance Loss Coefficients: see Table 9-1.

3. Capacity Curves: There are many charts, tables, and curves in the literature for the computation of culvert hydraulic capacity. To assist in the review of the culvert design computations and to obtain uniformity of analysis, the following data shall be used:

   All culverts: USDCM, Vol.2, "Inlets and Culverts" Section.

   Concrete Pipe: Concrete Pipe Design Manual, ACPA, Arlington, Virginia, latest edition.

   Corrugated Metal Pipe: "Handbook of Steel Drainage and Highway Construction Products", AISI, Washington, DC, latest editions.

   Copies of product manuals may frequently be obtained through local pipe suppliers.

4. Table 9.2 is to be used for determining culvert capacities. A design example is presented in Section 9-5 and shown on Table 9-3.

9.3.4 Velocity Considerations

In the design of culverts, both the minimum and maximum velocities must be considered. A minimum velocity of three feet per second at the outlet is required.

The maximum velocity is dictated by the channel conditions at the outlet. If the outlet velocities are less than 7 feet per second for grassed channels, then the minimum amount of protection is required due to the eddy currents generated by the flow transition. Higher outlet velocities will require substantially more protection. A maximum outlet velocity of 12 feet per second is recommended with erosion protection. Refer to Sections 10.2 and 10.3 for protection requirements at culvert outlets.

9.3.5 Headwater Considerations

The maximum headwater for the 100-year design flow will normally be 1.5 times the culvert diameter, or 1.5 times the culvert rise dimension for non-round shapes. Also, the headwater depth may be limited by the street overtopping requirements in Section 8. For headwater depths greater than 1.5, the applicant shall submit detailed calculations determining the outlet velocity. If the outlet velocity is greater than 12 fps, an energy dissipator will be required.

9.3.6 Structural Design

As a minimum loading, all culverts shall be designed to withstand an HS-20 loading (unless designated otherwise by the Town) in accordance with the design procedures of AASHTO, "Standard Specifications for Highway Bridges," and with the pipe manufacturers' recommendations.

9.3.7 Trash Racks

Trash racks may be required at the entrance of culverts for some installations as designated by the Town. Installation of trash racks prevents debris from entering culverts, thereby protecting the culverts from internal blocking. Routine cleaning of the trash racks is required to remove the collection of debris. Trash racks typically are not required at entrances to culverts crossing local streets or culverts within the right-of-way, which cross-driveways unless a safety hazard is identified. Trash racks typically are required at entrances to culverts in all other situations for purposes of safety, water quality, and/or maintenance.

The following criteria shall be used for design of trash racks for storm drainage applications:

1. Materials: All trash racks shall be constructed with smooth steel pipe with a minimum 1.25 inches outside diameter. The trash rack ends and bracing should be constructed with steel angle sections. All trash rack components shall have a corrosion protective finish.

2. Trash Rack Design: The trash racks shall be constructed without cross braces (if possible) in order to minimize debris clogging. The trash rack shall be designed to withstand the full hydraulic load of a completely plugged trash rack based on

the highest anticipated depth of ponding at the trash rack. The trash rack shall also be hinged and removable for maintenance purposes.

3. Bar Spacing: The steel pipe bars shall be spaced with a maximum clear opening of six inches. In addition, the entire trash rack shall have a minimum clear opening area (normal to the rack) at the design flow depth of four times the culvert opening area.

4. Trash Rack Slope: The trash rack shall have a longitudinal slope of no steeper than 2.5 horizontal to 1 vertical for maintenance purposes.

5. Hydraulics: Hydraulic losses through trash racks shall be computed using the following equation:

$$H_T = 0.11\,(TV/D^2)(\sin A)$$

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

This equation shall apply to all racks constructed normal to the approach flow direction. The velocity normal to the trash rack shall be computed considering the rack to be 50 percent plugged.

9.4 CULVERT SIZING CRITERIA

The sizing of a culvert is dependent upon two factors, the drainage classification of the street (arterial, collector, local, etc.) and the allowable street overtopping. The allowable street overtopping is set forth in Section 8. In addition to the allowable street overtopping, no street overtopping shall occur for any street classification at a 10-year frequency design storm event.

Therefore, as the minimum design standard for street crossings, the following procedure shall be used.

1. Using the future developed conditions 100-year runoff, the allowable street overtopping shall be determined from overflow rating curves developed from the street profile crossing the waterway.

2. The culvert is then sized for the difference between the 100-year runoff and the allowable overtopping.

3. If the resulting culvert is smaller than that required to pass the 10-year flood peak without overtopping, the culvert shall be increased in size to pass the 10-year flow.

The criteria are considered a minimum design standard and must be modified where other factors are considered more important. For instance, if the procedure still results in certain structures remaining in the 100-year floodplain, the design frequency may be increased to lower the floodplain elevation. Also, if only a small increase in culvert size is required to prevent any overtopping, then the larger culvert is recommended.

9.5 Design Example

The procedure recommended to evaluate existing and proposed culverts is based on the procedures presented in the Hydraulic Design Series, No. 5 (See Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, Report No. FHWA IP-85-15, USDOT, FHWA, September 1985). The methodology consists of evaluating the culvert headwater requirements, assuming both inlet control and outlet control. The rating that results in the larger headwater requirements is the governing flow condition.

Culvert Rating

A sample calculation for rating an existing culvert is presented in Table 9-3. The required data are as follows:

Culvert size, length, and type: 48" CMP, L = 150', n = 0.024
Inlet, outlet elevation, and slope: 5540.00, 5535.5, S_o = 0.03 ft/ft

Inlet treatment: flared-end section
Low point elevation of embankment: El = 5551.9
Tail water-rating curve: see Table 9-3, Column 5

From the data above, the entrance loss coefficient, Ke, and the "n" value are determined. The full flow Q and the velocity are calculated for comparison. The rating then proceeds in the following sequence:

STEP 1: Headwater values are selected and entered in Column 3. The headwater to pipe diameter ratio (H_w/D) is calculated and entered in Column 2. If the culvert is other than circular, the height of the culvert is used.

STEP 2: For the H_w/D ratios, the culvert capacity is read from the rating curves (refer to Section 9.3.3) and entered into Column 1. This completes the inlet condition rating.

STEP 3: For outlet conditions, the Q values in Column 1 are used to determine the head values (H) in Column 4 from the appropriate outlet control rating curves (refer to Section 9.3.3).

STEP 4: The tail water depths (T_w) are entered into Column 5 for the corresponding Q values in Column 1 according to the tail water-rating curve (i.e., downstream channel rating computations). If a tail water-rating curve is not available, then the tail water can be approximated by calculating the normal depth for each flow value using the trapezoidal section (noted on Table 9-3). If the tail water depth (T_w) is less than the diameter of the culvert (D), Columns 6 and 7 are to be calculated (go to Step 5). If T_w is more than D, the tail water values in Column 5 are entered into Column 8 for the h_o values, and proceed to Step 6.

STEP 5: The critical depth (dc) for the corresponding Q values in Column 1 is entered into Column 6. The average of the critical depth and the culvert diameter is calculated and entered into Column 7 as the h_o values.

STEP 6: The headwater values (H_w) are calculated according to the equation:

$$H_w = H + h_o - LS_o \qquad \text{(Equation 9.1)}$$

Where H is from Column 4, and ho is from Column 8 (for $T_w > D$) or the larger value between Column 5 and Column 7 (for $T_w < D$). The values are entered into Column 9.

STEP 7: The final step is to compare the headwater requirements (Columns 9 and 3) and to record the higher of the two values in Column 10. The type of control is recorded in Column 11, depending upon which case gives the higher headwater requirements. The headwater elevation is calculated by adding the controlling H_w (Column 10) to the upstream invert elevation. A culvert-rating curve can then be platted from the values in Columns 12 and 1.

To size a culvert crossing, the same form can be used with some variations in the basic procedures. First, a design capacity is selected and the maximum allowable headwater is determined. An inlet type (i.e., headwall) is selected, and the invert elevations and culvert slope are estimated based upon site constraints. A culvert type is then selected and first rated for inlet control and then for outlet control.

If the controlling headwater exceeds the maximum allowable headwater, a different culvert configuration is selected and the procedure repeated until the desired results are achieved.

9.6 CHECKLIST

To aid the Designer and Reviewer the following checklist has been prepared:

1. Minimum culvert size within the public ROW is 24-inch diameter round or equivalent for other shapes.
2. Minimum culvert size for roadside ditches at driveways is 12-inch diameter round or equivalent for other shapes.
3. Headwalls, wing walls, or flared-end sections required for all culverts.
4. Check outlet velocity and provide adequate protection.
5. Check maximum headwater for design condition.
6. Check structural requirements.

(A) VALUES OF COEFFICIENT OF ROUGHNESS (n) FOR STANDARD CORRUGATED STEEL PIPE (MANNING'S FORMULA)

CORRUGATIONS	ANNULAR 2 2/3 X 1/2	HELICAL 1in.1/2X1/4in.		HELICAL 2 2/3" X 1/2"								
Flowing:	DIAMETERS	8 in.	10 in.	12 in	15 in.	18 in.	24 in.	30 in.	36 in.	42 in.	48 in.	54 in. and Larger
Full-Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021
Full-25% Paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019
Part Full-Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023
Flowing:	PIPE ARCH				17x13	21x15	28x20	35x24	42x29	49x33	57x38	64 x 43 and Larger
Full Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022
Part Full	0.029				0.018	0.019	0.021	0.023	0.024	0.025	0.025	0.026

	ANNULAR	HELICAL 3 X 1 in.							
Flowing:	3 x 1 in.	36 in.	42 in.	48 in.	54 in.	60 in.	66 in.	72 in.	78 in. & Larger
Full Unpaved	0.027	0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
25% Paved	0.023	0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023

	ANNULAR	HELICAL 5 X 1 in.					
Flowing:	5 x 1 in.	48 in.	54 in.	60 in.	66 in.	72 in.	78 in. & Larger
Full Unpaved	0.025	0.022	0.022	0.023	0.024	0.024	0.025
25% Paved	0.022	0.019	0.019	0.020	0.021	0.021	0.022

All Pipe With Smooth Interior*	ALL DIAMETERS 0.012

* INCLUDES FULL PAVED, CONCRETE LINED, SPIRAL RIB AND DOUBLE WALL PIPE. REFERENCE 13 MODIFIED FOR LOWER VALUES OF n FOR HELICAL PIPE. SEE HANDBOOK OF STEEL DRAINAGE AND HIGHWAY CONSTRUCTION PRODUCTS BY THE AMERICAN IRON AND STEEL INSTITUTE, 1994 OR LATEST EDITION.

NOTE: COEFFICIENT VALUES SHALL CONFORM TO THE HANDBOOK OF STEEL DRAINAGE AND HIGHWAY CONSTRUCTION PRODUCTS, 1994 OR LATEST EDITION PUBLISHED BY THE AMERICAN IRON AND STEEL INSTITUTE.

TOWN OF KERSEY

HYDRAULIC DATA FOR CULVERTS

SCALE: NONE

DATE: 12/04

APPROVED: FJB

TOWN ENGINEER

(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

TOWN OF KERSEY

HYDRAULIC DATA FOR CULVERTS

SCALE: NONE

DATE: 12/04

APPROVED:

FJB

TOWN ENGINEER

SHEET FIG. TABLE 9-1A

(D) CULVERT ENTRANCE LOSSES

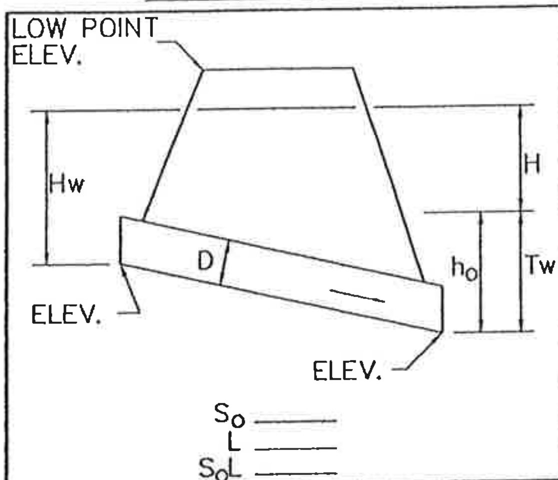
TYPE OF ENTRANCE	ENTRANCE COEFFICIENT, Ke
PIPE	
HEADWALL	
GROOVED EDGE	0.20
ROUNDED EDGE (0.15D RADIUS)	0.15
ROUNDED EDGE (0.25D RADIUS)	0.10
SQUARE EDGE (CUT CONCRETE AND CMP)	0.40
HEADWALL AND 45° WINGWALL	
GROOVED EDGE	0.20
SQUARE EDGE	0.35
HEADWALL WITH PARALLEL WINGWALLS SPACED 1.25D APART	
GROOVED 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.

TOWN OF KERSEY	
HYDRAULIC DATA FOR CULVERTS	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

CULVERT RATING

PROJECT ____________ LOCATION ____________ STATION ____________



CULVERT DATA

TYPE ____________ n ____________

INLET ____________ Q_{FULL} ____________

Ke ____________ V_{FULL} ____________

OUTLET CONTROL EQUATIONS

(1) $Hw = H + h_o - LS_o$

(2) FOR $Tw < D$; $h_o = \frac{d_c + D}{2}$ OR Tw (WHICHEVER IS GREATER)

$Tw > D$; $h_o = Tw$

(3) FOR BOX CULVERT: $d_c = 0.315(Q/B)^{2/3} \leq D$

	INLET CONTROL		OUTLET CONTROL						CONT	CONTROL	ELEV
					Tw < D		Tw > D				
Q	$\frac{Hw}{D}$	Hw	H	Tw	d_c	$\frac{d_c+D}{2} = h_o$	h_o	Hw	Hw		
1	2	3	4	5	6	7	8	9	10	11	12

TOWN OF KERSEY

CULVERT RATING

SCALE: NONE

DATE: 12/04

APPROVED:

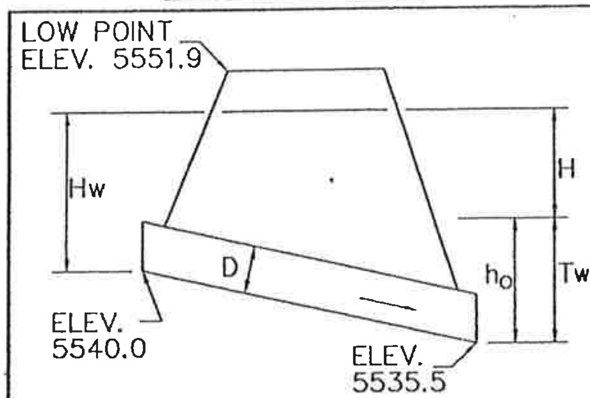
FJB

TOWN ENGINEER

SHEET TABLE 9–2

CULVERT RATING

PROJECT EXAMPLE LOCATION KERSEY, CO. STATION 2+00



S_o 0.030
L 150
S_oL 4.5

CULVERT DATA

TYPE 48" CMP n 0.024
INLET FLARED END SEC Q_{FILL} 135
Ke 0.5 V_{FILL} 10.7

OUTLET CONTROL EQUATIONS

(1) $Hw = H + h_o - LS_o$

(2) FOR $Tw < D$; $h_o = \frac{d_c+D}{2}$ OR Tw (WHICHEVER IS GREATER)

$Tw > D$; $h_o = Tw$

(3) FOR BOX CULVERT: $d_c = 0.315(Q/B)^{2/3} \leq D$

Q	INLET CONTROL		OUTLET CONTROL						CONT	CONTROL	ELEV
					Tw < D		Tw > D				
Q	$\frac{Hw}{D}$	Hw	H	Tw	d_c	$\frac{d_c+D}{2}=h_o$	h_o	Hw	Hw		
1	2	3	4	5	6	7	8	9	10	11	12
70	1.0	4	1.9	1.5	2.5	3.3		0.7	4	INLET	5544.0
115	1.5	6	5.5	2.0	3.0	3.5		4.5	6	INLET	5546.0
145	2.0	8	8.9	2.5	3.4	3.7		8.1	8.1	OUTLET	5548.8
170(1)	2.5	10	12.5	3.0	3.7	3.9		11.9	11.9	OUTLET	5551.9
195(2)	3.0	12	16.0	3.5	4.0	4.0		15.5	15.5	OUTLET	5555.5

OUTLET VELOCITY, V=Q/A=170 cfs /12.6 ft^2= 13.5 fps

TOWN OF KERSEY

EXAMPLE OF STANDARD FORM 400-SF4

SCALE: NONE DATE: 12/04

APPROVED: FJB
TOWN ENGINEER

SHEET TABLE 9-3

SECTION 10.0 HYDRAULIC STRUCTURES

10.1 EROSION CONTROL

Hydraulic structures are used in storm drainage design to control the flow of the runoff. The energy associated with flowing water has the potential to create damage to the drainage works, especially in the form of erosion. Hydraulic structures, which include rock riprap revetment, energy dissipators, check structures, bridges, and irrigation ditch crossings, all control the energy and minimize the damage potential of storm runoff.

The criteria to be used in the design of hydraulic structures shall be in accordance with the USDCM, Volume 2, in the "Major Drainage" and "Structures" sections. The specific criteria to be used within the Town of Kersey are summarized in the following information.

10.2 ROCK RIPRAP REVETMENT

The design of the riprap protection for culverts, channel bottom and banks, check drops, bridges, gabions, or other areas subject to erosion, shall be in accordance with the latest revisions of the USDCM, Volume 2, "Major Drainage Section 5 - Riprap."

10.3 ENERGY DISSIPATORS

Where riprap structures are insufficient or uneconomical to control the storm runoff, concrete energy dissipator structures (stilling basins) shall be provided in accordance with the USDCM, Volume 2 "Structures, Section 2,2 - Energy Dissipators."

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.

10.4 CHECK STRUCTURES AND DROP STRUCTURES

As discussed in Section 5-Open Channels, there is a maximum permissible velocity for major design storm runoff in grass-lined channels. One of the more common methods of controlling the flow velocity is to reduce the channel invert slope, which requires a drop structure to make up for the elevation difference occurring when the channel slope is reduced.

The design criteria for the drop structures shall be in accordance with the USDCM, Volume 2, "Structures, Section 3 - Channel Drops."

10.5 BRIDGES

The design of bridges within the Town shall be in accordance with the USDCM, Volume 2, "Structures, Section 4 - Bridges." The design capacity of the bridge shall be determined by the method presented in Section 9.4 of these Criteria.

10.6 IRRIGATION DITCH CROSSINGS

Any proposed developments in the vicinity of irrigation ditches and canals that cross or utilize the ditches or canals for surface drainage shall have the plans approved by the controlling ditch company prior to acceptance by the Town.

SECTION 11.0 DETENTION

11.1 INTRODUCTION

The criteria presented in this section shall be used in the design and evaluation of all detention facilities for the Town. The review of all planning submittals (refer to Section 2) will be based on the criteria presented in this section.

The main purposes of a detention facility are to store the excess runoff associated with an increase in basin imperviousness and discharge this excess at a rate similar to the rate experienced from the basin without development. It is intended that the detention facility protect downstream property and improvements, and avoid the overloading of storm drainage facilities located further downstream. The Town before proceeding with design shall review any special design conditions that cannot be defined by these Criteria.

The various detention methods are defined on the basis of where the facility is constructed, such as open space detention, parking lot, underground, or rooftop. The Town permits all methods of detention except for rooftop detention.

Detention facilities shall not be constructed within public right-of-way. The design high water level of detention ponds shall not encroach upon public right-of-way.

11.2 WATER QUALITY ENHANCEMENT

Guidelines for incorporating water quality considerations within the design and construction of detention ponds are presented within these Criteria in Section 12 - Stormwater Quality Enhancement.

11.3 STORAGE REQUIREMENTS

11.3.1 Areas Without Master Drainage Plans

In basins where a master drainage plan has not been approved, the Town may require detention storage in accordance with this section to protect irrigation structures or downstream development. The stormwater runoff shall not be released from developments at a rate greater than the 5-year historical runoff (prior to any development). The amount of runoff to be detained on-site shall be the difference between the 100-year runoff under developed conditions and the 5-year historical runoff. In all cases, it should be assumed that detention is required unless proven otherwise.

11.3.3 Variances

For any release rate greater than specified in either Sections 11.3 or 11.4, the Design Engineer requesting the variance shall analyze the downstream conditions in detail and show that no adverse effects will occur. This analysis shall include any and all information required by the Town and all calculations pertaining to the analysis shall be submitted for review (such as the volume and peak discharge calculations). This analysis shall be submitted for review and approval before the detention facility for that site is designed.

Examples of when detention requirements may be varied are: (a) if development occurring on the site decreases the percentage of impervious area already present, (b) if the site is adjacent to a major outfall and runoff will not influence its time to peak or adversely impact downstream facilities, or (c) if the latter phase of a subdivision is submitted and the previous phases have already met the detention requirements for the entire site. Any variances shall be approved by the Town and these areas must be thoroughly analyzed to show that no hazards will be created downstream. Retention ponds are not acceptable unless there is no feasible method for draining the pond by gravity. If a retention pond is found to be acceptable, the minimum volume of the facility must be adequate to retain the storm runoff from the 100-year developed storm event. For any storm event, if the retention pond cannot be drained within 3 days, either temporary or permanent pumping facilities will be required to drain the pond. Percolation/evaporation alone may be an accepted method for draining the pond over a prolonged period of time only if a storage capacity of at least two times the 100-year developed storm event is provided. Percolation will not be acceptable in situations where an increase in groundwater flow as a result of pond percolation could cause an adverse impact upon structures with basements down gradient of the pond site.

11.4 DESIGN CRITERIA

11.4.1 Volume and Release Rate

The minimum required volume shall be determined using either: (a) the Rational Formula Method; or (b) the CUHP method as documented in the USDCM Volume 2 "Storage" or the EPA SWMM computer program. Utilization of the Rational Formula Method is restricted to basins less than or equal to 5 acres. For basins larger than 5 acres, detention volumes should be determined using the CUHP method or the EPA SWMM computer program. Alternative computer programs for routing flows through detention facilities must be reviewed and approved by the Town prior to utilization.

For all detention facilities within the Town, the minimum volume of the facility must be adequate to detain the storm runoff from the 100-year developed storm event to an amount not greater than the peak runoff associated with the 5-year storm event for the historical (prior to any development) conditions. Off-site flows may be passed through the detention facility. Provide adequate spillway/outlet capacity to safely pass these flows to downstream conveyance elements. Additional information related to minimum storage volumes and maximum release rates from detention ponds can be obtained from the master drainage plans for each major drainage basin.

Procedures for the determination of the required detention volume using the Rational Formula Method are presented below.

Rational Formula Method

The cumulative runoff volume entering a detention pond is estimated by

$$V_{in} = CiAT \qquad \text{(Equation 11.1)}$$

Where V_{in} = cumulative runoff volume, ft3
C = runoff coefficient
I = storm's intensity taken from IDF curve at time T, inches per hour
A = tributary area, acres
T = storm duration, seconds

The cumulative volume leaving the basin is estimated by

$$V_{out} = kQ_{out}T \qquad \text{(Equation 11.2)}$$

in which T is defined above and

V_{out} = cumulative volume of outflow, ft^3

Q_{out} = maximum outflow rate, cfs
k = outflow adjustment coefficient from Figure 11-1

In Figure 11-1, Q_{pin} (peak inflow rate) is determined from the Rational Method Formula (Q=CiA).

The required detention volume is the maximum difference between the cumulative inflow and the cumulative outflow volumes or

$$V = \max (V_{in} - V_{out}) \qquad \text{(Equation 11.3)}$$

As per Urban Drainage Design Guidelines, if the procedure results in an increasing storage volume at the end of two hours, use the 2-hour storage volume.

This procedure assumes a constant outflow rate which is the rate of discharge when the detention pond is full. Discharge varies, however, with the depth of water. This fact is partially compensated for by the outflow adjustment factor k.

11.4.2 Design Frequency

All detention facilities are to be designed to release not greater than the 5-year historical (prior to any development) peak runoff during the 100-year storm event.

11.4.3 Hydraulic Design

Hydraulic design procedures for sizing of detention facilities outlet works are described below.

1. Weir Flow

The general form of the equation for horizontal crested weirs is:

$$Q = CL(H)^{3/2} \quad \text{(Equation 11.4)}$$

Where Q = discharge (cfs)
C = weir coefficient (see Table 11-1)
L = horizontal length (feet)
H = total energy head (feet)

Another common weir is the v-notch, whose equation is as follows:

$$Q = 2.5 \tan(\Theta/2)H^{5/2} \quad \text{(Equation 11.5)}$$

Where Θ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow, the effects of submergence must be considered. A single check on submergence can be made by comparing the tail water to the headwater depth. The example calculation for a weir design on Figure 11-2 illustrates the submergence check.

2. Orifice Flow

The equation governing the orifice opening and plate is the orifice flow equation:

$$Q = C_dA(2gh)^{1/2} \quad \text{(Equation 11.6)}$$

Where Q = flow (cfs)
C_d = orifice coefficient
A = area (ft^2)
g = gravitational constant = 32.2 ft/sec2
h = head on orifice measured from centerline (ft)

An orifice coefficient (C_d) value of 0.65 shall be used for sizing of all square edged orifice openings and plates (see Figure 11-4).

11.5 Design Standards for Open Space Detention

11.5.1 State Engineer's Office

Any dam constructed for the purpose of storing water, with a surface area, volume, or dam height as specified in Colorado Revised Statues 37-87-105 as amended, shall require the approval of the plans by the State Engineer's Office. Those detention storage facilities subject to state statutes shall be designed and constructed in accordance with the criteria of the State.

11.5.2 Grading Requirements

Slopes on earthen embankments shall not be steeper than 4H:1V. All earthen slopes shall be covered with topsoil and re-vegetated with grass. For grassed detention facilities, the minimum bottom slope shall be 0.5 percent, measured perpendicular to the trickle channel.

11.5.3 Freeboard Requirements

The minimum required freeboard for open space detention facilities is 1 foot above the computed 100-year water surface hydraulic grade line elevation. The invert of the emergency spillway shall be placed at or above the computed on-site 100-year water surface elevation. The computed 100-year water surface hydraulic grade line elevation is defined as follows:

A flow situation for which the emergency spillway is passing the 100-year developed on-site flow plus the 100-year off-site flows (if they exist) based on an assumed plugged orifice condition.

All state dam safety criteria must be carefully considered when determining the freeboard capacity of an impoundment, especially in ponds, which incorporate high embankments or large pond areas and storage volumes.

11.5.4 Trickle Flow Channels

All grassed bottom detention ponds shall include a grass lined trickle channel with perforated under drain wherever practical for water quality enhancement. Where the water quality channel is impractical, construct a concrete channel. Trickle flow channel criteria are presented in Section 5.4.2.

11.5.5 Outlet Configuration

Presented in Figure 11-3 are two examples of a detention pond outlet configuration (see Section 12 for outlet requirements for stormwater quality enhancement). A Type 1 outlet consists of a grated drop inlet, outlet pipe, and an overflow weir in the pond embankment. The outlet will be designed to release the peak discharge associated with the 5-year historical runoff during the 100-year runoff under developed conditions. The control for the 100-year peak discharge shall be at the throat of the outlet pipe under the head of water as defined on Figure 11-3. The grate must be designed to pass the 5-year historical storm flow with a minimum of 50 percent blockage (i.e. twice the initial storm calculated flow). Since the minimum size of the outlet pipe is 12 inches, then a control orifice plate at the entrance of the pipe may be required to control the discharge of the design flow. An example orifice plate is shown in Figure 11-4.

A Type 2 outlet consists of a depressed inlet with an outlet pipe and an overflow weir in the pond embankment. Again, an orifice plate at the entrance may be required to control the release rate to the 5-year historical peak flow. The control for the 100-year developed condition discharge occurs at the throat of the outlet pipe as shown on the Figure 11-3. The outlet pipe must have an adequate slope to ensure throat control in the pipe.

For both outlet types, flows in excess of the developed condition 100-year discharge are released through the overflow weir or spillway. The control orifice plate shall not be may be oversized to pass release flows from off-site (upstream) detention facilities. The overflow weir or spillway may be oversized to pass other off-site flows through the pond. Size the spillway to pass the 100-flow developed on-site flow plus the 100-year off-site flows based on an assumed plugged orifice condition. The maximum trash rack-opening dimension shall be equal to half the minimum opening in the orifice plate.

Other outlet configurations will be allowed provided they meet the requirements of the permitted release rates at the required volume and include proper provisions for maintenance and reliability. The outlet shall be designed to minimize unauthorized modifications which effect proper function.

Additional storage will be required to incorporate water quality considerations into the design and construction of a detention pond. Minimum requirements for the implementation and use of detention ponds for stormwater quality control are provided in Section 12.0-Stormwater Quality Enhancement. For information and design guidelines related to the construction of detention ponds that promote stormwater quality control, the reader is referred to the Urban Storm Drainage Criteria Manual (USDCM), Volume 3, "Best Management Practices".

11.5.6 Embankment Protection

Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a larger than 100-year storm occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity equal to the 100-year historical storm event for the basin. Detention facilities in major drainage channels shall have a spillway capacity equal to the 100-year peak discharge associated with fully developed conditions. Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway should be set equal to or above the 100-year water surface elevation.

11.5.7 Vegetation Requirements

All open space detention ponds shall be re-vegetated by either irrigated sod or natural dry land grasses.

11.5.8 Maintenance Access

To assure that the detention facility performs as designed, maintenance access shall be provided, and shown on the Final Plat. For subdivisions, the detention facility is usually dedicated to the Town for operation and maintenance. For privately maintained facilities such as P.U.D.'s and commercial or industrial sites, an easement should be granted to the Town to allow access and to assure that the facility continues to function as intended.

11.6 Design Standards for Parking Lot Detention

The requirements for parking lot detention are presented below.

11.6.1 Depth Limitation

The maximum allowable design depth of the ponding is 18 inches for the 100-year flood and 6 inches for the initial storm runoff event. Also, provide calculations indicating the depth of ponding for the 10-year storm event. If the depth of ponding is greater than 8 inches for the 10-year event, signs must be posted warning users of high water during significant storm events.

11.6.2 Outlet Configuration

The minimum pipe size for the outlet is 12 inches in diameter where a drop inlet is used to discharge to a storm sewer or drainage way. Where a weir and a small diameter outlet through a curb are used, the size and shape are dependent on the discharge and storage requirements. In this case, a minimum pipe diameter of 3 inches is recommended.

11.6.3 Maintenance Access

To assure that the detention facility performs as designed, maintenance access, especially to outlet structures, shall be provided. The outlet shall be designed to minimize unauthorized modifications, which effect function. Any repaving of the parking lot shall be evaluated for impact on volume and release rates and is subject to approval by the Town prior to proceeding with the repaving. A sign shall be attached or posted in accordance with Section 11.6.4.

11.6.4 Flood Hazard Warning

All parking lot detention areas shall post a minimum of two signs identifying the detention pond area. The sign shall have a minimum area of 1.5 square feet and contain the following message:

"WARNING"

This area is a detention pond and is subject to periodic flooding depths ranging from a depth of __ (provide the design depth) for the initial storm runoff event to (provide the design depth) for the 100-year storm."

Any suitable materials and geometry of the sign are permissible, subject to approval of the Town.

11.7 DESIGN STANDARDS FOR UNDERGROUND DETENTION

The requirements for underground detention are presented in the following paragraphs.

11.7.1 Materials

Underground detention shall be constructed using corrugated aluminum pipe (CAP) or reinforced concrete pipe (RCP). The pipe thickness cover, bedding, and backfill shall be designed to withstand HS-20 loading.

11.7.2 Configuration

Pipe segments shall be sufficient in number, diameter, and length to provide the required minimum storage volume for the 100-year design. As an option, the initial design storm runoff volume can be stored in the pipe segments and the difference for the 100-year runoff volume stored above the pipe in an open space detention pond, or in parking lot detention. The minimum diameter of the pipe segments shall be 36 inches.

The pipe segments shall be placed side by side and connected at both ends by elbow tee fittings and across the fitting at the outlet (see Figure 11-5). The pipe segments shall be continuously sloped at a minimum of 0.25 percent to the outlet. Manholes for maintenance access (see Section 11.7.4) shall be placed in the tee fittings and in the straight segments of the pipe, when required.

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

11.7.3 Inlet and Outlet Design

The outlet from the underground detention shall consist of a short (maximum 25 feet) length of CAP or RCP with a 12-inch minimum diameter. The outlet pipe(s) shall discharge into a standard manhole or into a drainage way with erosion protection provided per Sections 9.3.2, 10.2, and 10.3. If an orifice plate is required to control the release rates, the plate shall be hinged to open into the detention pipes to facilitate back flushing of the outlet pipe.

Inlet to the detention pipes can be by way of surface inlets and/or by a local private storm sewer system. Channel rundowns down the slope of detention ponds shall be designed as per Section 5.0, Open Channels, Part 5.7 Channel Rundowns.

11.7.4 Maintenance Access

Access easements to the detention site shall be provided. To facilitate cleaning of the pipe segments, 3-foot diameter maintenance access ports shall be placed according to the following schedule:

Maintenance Access Requirements		
Detention Pipe Size	Maximum Spacing	Minimum Frequency
36” to 54”	150”	Every pipe segment
60” to 66”	200’	Every other pipe segment
Greater than 66”	200’	One at each end of the battery of pipes

The manholes shall be constructed in accordance with the detail on Figure 11-5.

11.8 Design Example

Detention Volume

GIVEN: Basin area = 5 acres
Single family residential site, Percent Impervious = 40%
Historical 5-year release rate = 2 cfs
Developed 100-year peak inflow rate = 16.5 cfs
100-year Runoff Coefficient = 0.6

FIND: Determine size of detention pond to limit developed 100-year peak inflow rate to historical 5-year release rate of 2 cfs.

SOLUTION:

STEP 1: Determine Qout/Qpin to be 0.12

STEP 2: From Figure 11-1, k = 0.96

STEP 3: At intervals of 5 minutes, determine the

rainfall intensity (from IDF curves)
Runoff volume (V_{in} from Equation 11.1)
Outflow volume (V_{out} from Equation 11.2)
Storage volume (V_{in} - V_{out})

The calculating procedure is illustrated in Table 11-2. The required detention volume from Table 11-2 is determined to be 0.53 acre-feet.

Detention Outlet Structure

GIVEN: Detention pond with the following characteristics.

Maximum 100-year release rate = 23.0 cfs
Maximum 5-year release rate = 6.9 cfs
Type 2 outlet (refer to Figure 11-3)
100-year water surface elevation = 105.0
5-year outlet pipe invert elevation = 100.0
18-inch diameter outlet pipe

FIND: 5-year outlet sizing

SOLUTION: (see Figure 11-6)

STEP 1: Determine maximum discharge from the outlet pipe. Depth to centerline of the outlet pipe = 4.25 ft.

$Q = C_dA(2gh)1/2$ (Equation 11.6)

$= 0.65(1.76)[2(32.2)(4.25)]^{1/2}$
$= 18.9$ cfs

Q > Q5; requires orifice plate

STEP 2: Determine 5-year orifice opening size; estimate depth to centerline of orifice (try 4.5 ft)

$A = Q/[C_d\ (2gh)^{½}]$ (Rearranged Equation 11.6)

$= 6.9/\{0.65[2(32.2)(4.5)]^{½}\}$
$= 0.62\ ft^2$

STEP 3: Determine 5-year orifice diameter

Diameter $= [4A/\pi]^{½}$
$= [4(0.62)/\pi]^{½}$
= 0.9 feet (10.7 inches)

Therefore, use an orifice opening with a 10.5-inch diameter hole at the entrance to the outlet box.

STEP 4: Check discharge through 5-year outlet for 100-year headwater and 10.5-inch orifice opening (h= 4.56 feet).

$Q = C_dA(2gh)^{½}$ (Equation 11.6)
$= 0.65(0.60)[2(32.2)(4.56)]^{½}$
= 6.7 cfs

Q = 6.7 cfs < Q5 = 6.9 cfs

STEP 5: Check trash rack design area given orifice diameter of 10.5 inches.

Orifice area = $A_o = 0.6\ ft^2$
$A/A_o = 10$ (from Figure 11-4 for 10.5-inch orifice diameter)
$A = 10(A_o) = 10(0.6) = 6\ ft^2$

Maximum opening = 0.5(10.5) = 5.25 inches

11.9 CHECKLIST

To aid the Designer and Reviewer, the following checklist has been prepared:

1. Earth slopes are to be 4:1 or flatter.

2. Minimum freeboard of 1 foot for the 100-year detention is required.

3. Open space detention areas shall include trickle channels. For the purpose of enhancing storm water quality, where practical, the trickle channel shall be grass lined with a buried perforated under drainpipe, surrounded by filter fabric and rock filter material. See Section 5.0 Open Channels Figure 5-5 Trickle Channel Details.

4. Channel rundowns have been designed as per Section 5.0 Open Channels, Part 5.7 Channel Rundown and Figure 5-7 Channel Rundown.

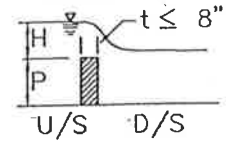
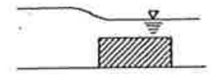
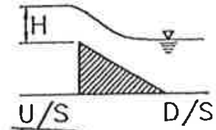
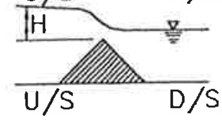
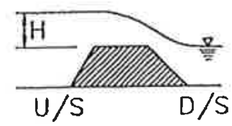
5. Protect embankment for overtopping condition by adding riprap or other acceptable erosion protection.

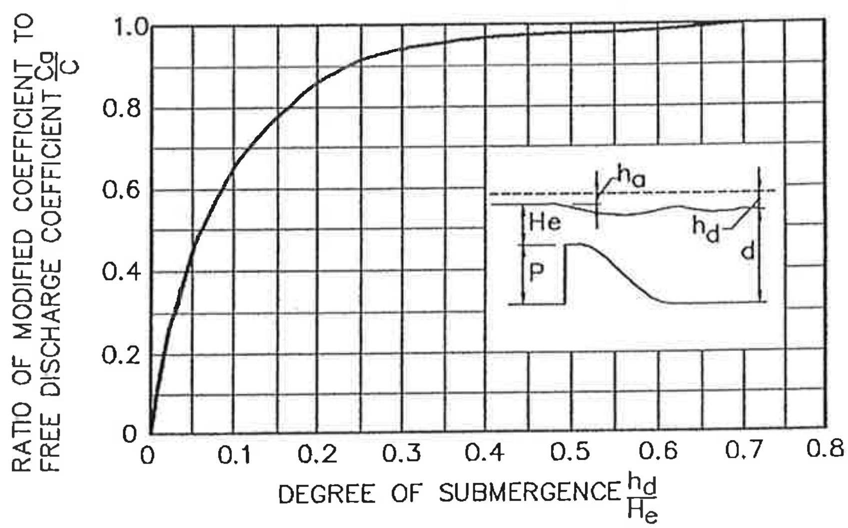
6. Provide trash racks where required.

7. Provide signs as required.

8. Provide maintenance access.

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

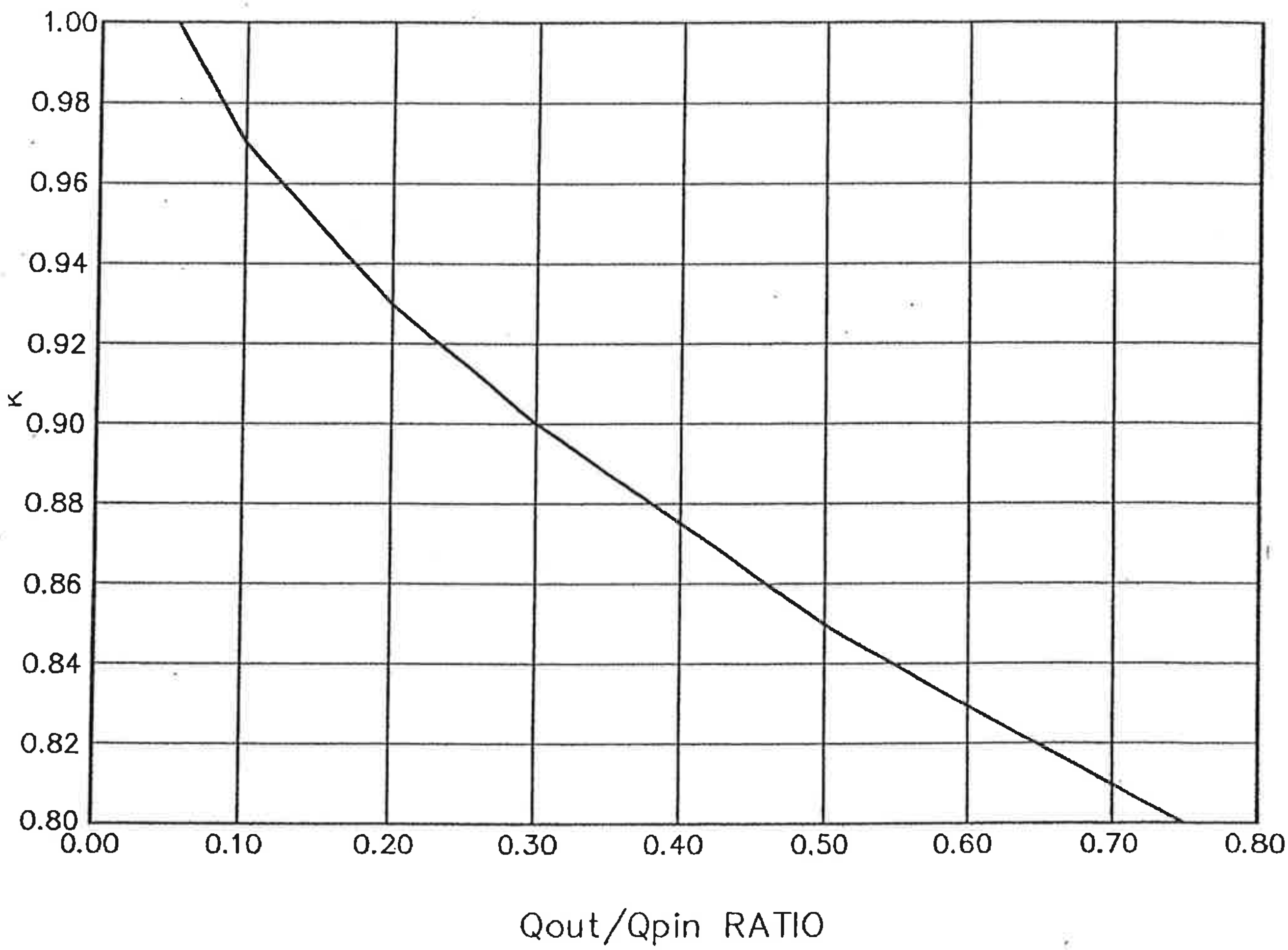
REFERENCE: KING & BRATER, HANDBOOK OF HYDRAULICS, McGRAW HILL BOOK COMPANY, 1963 – DESIGN OF SMALL DAMS, BUREAU OF RECLAMATION, 1977

TOWN OF KERSEY	
WEIR FLOW COEFFICIENTS	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

Table 11-2 Rational Formula Method for Detention Pond Sizing

Storm Duration T (min) [1]	Rainfall Intensity *I* (in/hr) [2]	Runoff Volume C*i*AT (ft³) [3]	Outflow Volume kQT (ft³) [4]	Storage Volume* (ft³) [5]	Storage Volume (AF) [6]
5	9.67	8,703	576	8,127	0.19
10	7.51	13,518	1,152	12,366	0.28
15	6.34	17,118	1,728	15,390	0.35
20	5.34	19,224	2,304	16,920	0.39
25	4.74	21,330	2,880	18,450	0.42
30	4.40	23,706	3,456	20,304	0.47
35	4.0	25,200	4,032	21,168	0.49
40	3.59	25,848	4,608	21,240	0.49
45	3.35	27,135	5,184	21,951	0.50
50	3.1	27,900	5,960	21,940	0.50
55	2.94	29,106	6,336	22,770	0.52
60	2.78	30,024	6,912	23,112	0.53**
70	2.47	31,122	8,064	23,058	0.53
80	2.16	31,104	9,216	21,888	0.50
90	2.0	32,400	10,008	22,392	0.51

* Column [3] – Column [4]
** Required Detention Volume

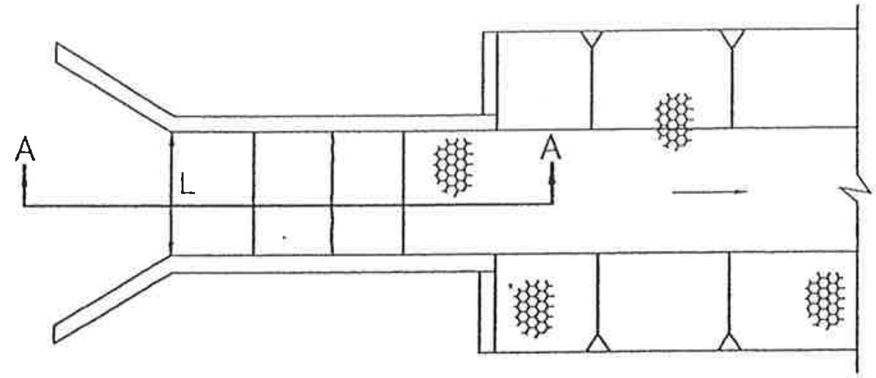


TOWN OF KERSEY

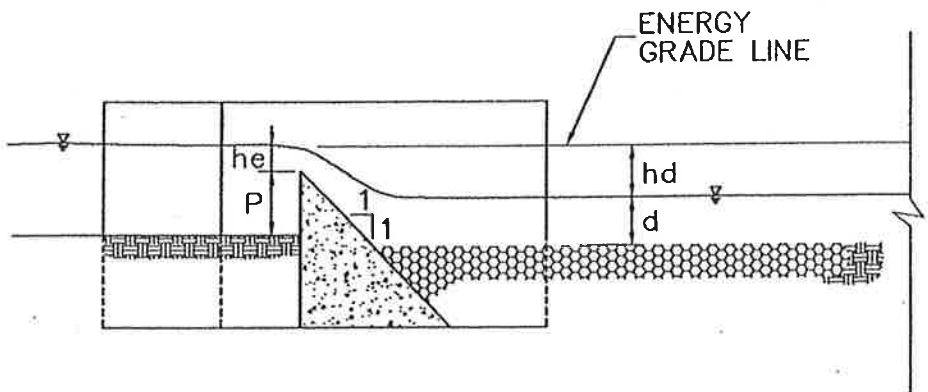
OUTFLOW ADJUSTMENT FACTOR VERSUS OUTFLOW RATE/INFLOW PEAK RATIO
(UNIVERSITY OF COLORADO–DENVER, 1991)

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG 11–1



PLAN



SECTION A–A

GIVEN: Q=100 CFS, TRIANGULAR WEIR WITH VERTICAL FACE, AND 1:1 DOWNSTREAM SLOPE, P=2', h_e =z', TAILWATER DEPTH = 4.5, hd = 1.5

FIND: L, AND CHECK SUBMERGENCE

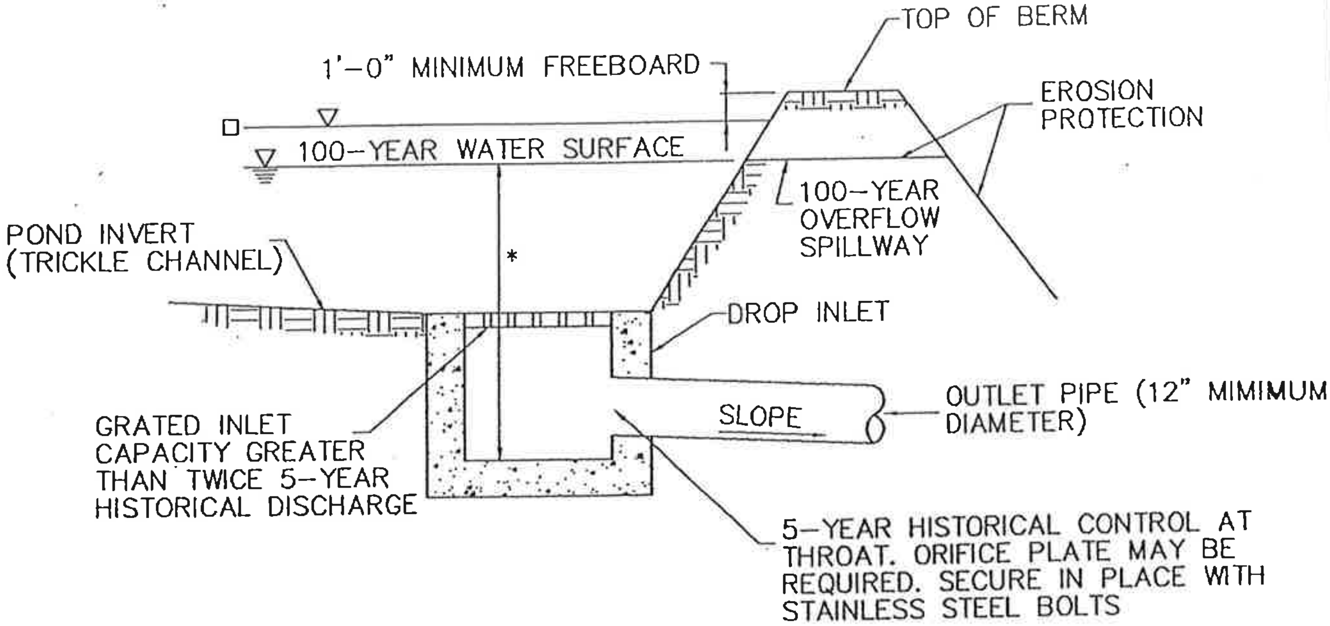
SOLUTION: Cw = 3.8 (TABLE 11–1)

$L = Q/CH^{3/2} = (100)/[(3.8)(2)^{3/2}] = 9.3$ FT

SUBMERGENCE CHECK

$\frac{hd}{he} = \frac{1.5}{2.0} = 0.75$, THEN Ca/C = 1.0, (TABLE 11–1)
THEREFORE NO SUBMERGENCE ADJUSTMENT IS REQUIRED

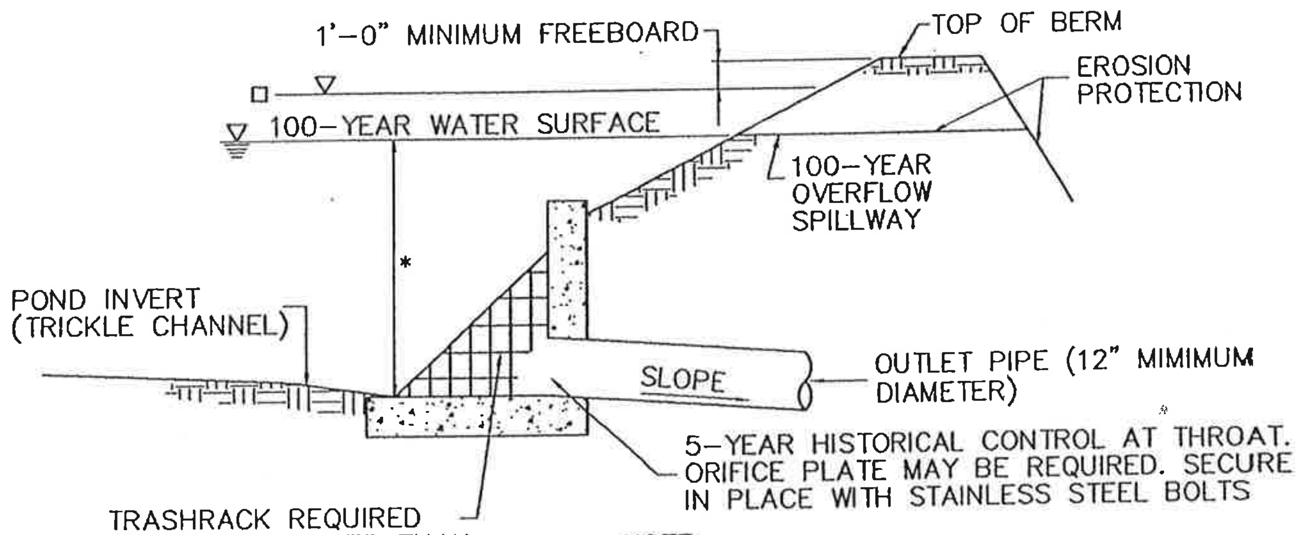
TOWN OF KERSEY	
WEIR DESIGN EXAMPLE	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	



TYPE I OUTLET

* HEADWATER FOR 5-YEAR HISTORICAL DISCHARGE

□ WATER SURFACE ELEVATION WHEN WATER IS FLOWING OVER SPILLWAY DURING A 100 YEAR STORM EVENT WITH A PLUGGED ORIFICE CONDITION



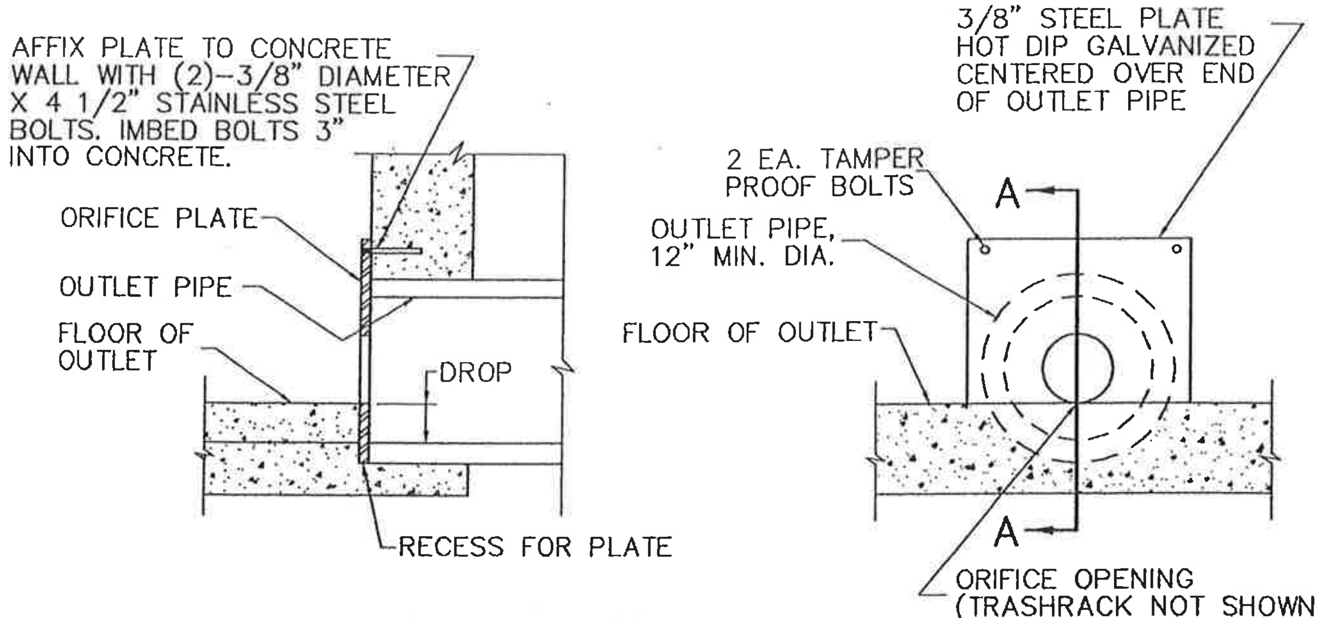
NOTE:
PROVIDE AT LEAST 1/4" STEEL ORIFICE PLATE AND 1/4" STEEL TRASHRACK COATED WITH INDUSTRIAL GRADE RUST INHIBITOR PAINT.

TYPE II OUTLET

TOWN OF KERSEY	
TYPE I AND TYPE II OUTLET	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

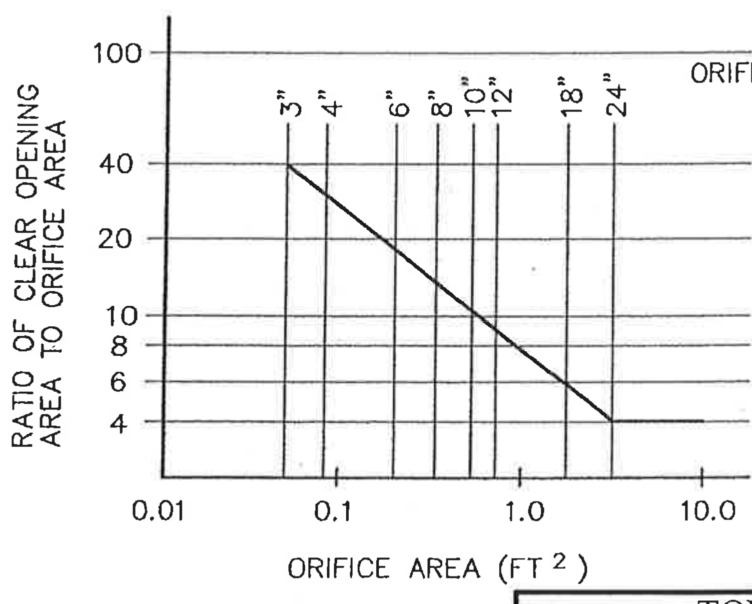
SHEET FIG. 11-3

ORIFICE PLATE DETAIL



SECTION A – A

(B) TRASHRACK AREA REQUIREMENTS



NOTE:
FOR ORIFICE DIAMETER LESS THAN 3", USE A MINIMUM CLEAR OPENING OF 2 FT^2

TOWN OF KERSEY

DETENTION POND DETAILS

SCALE: NONE

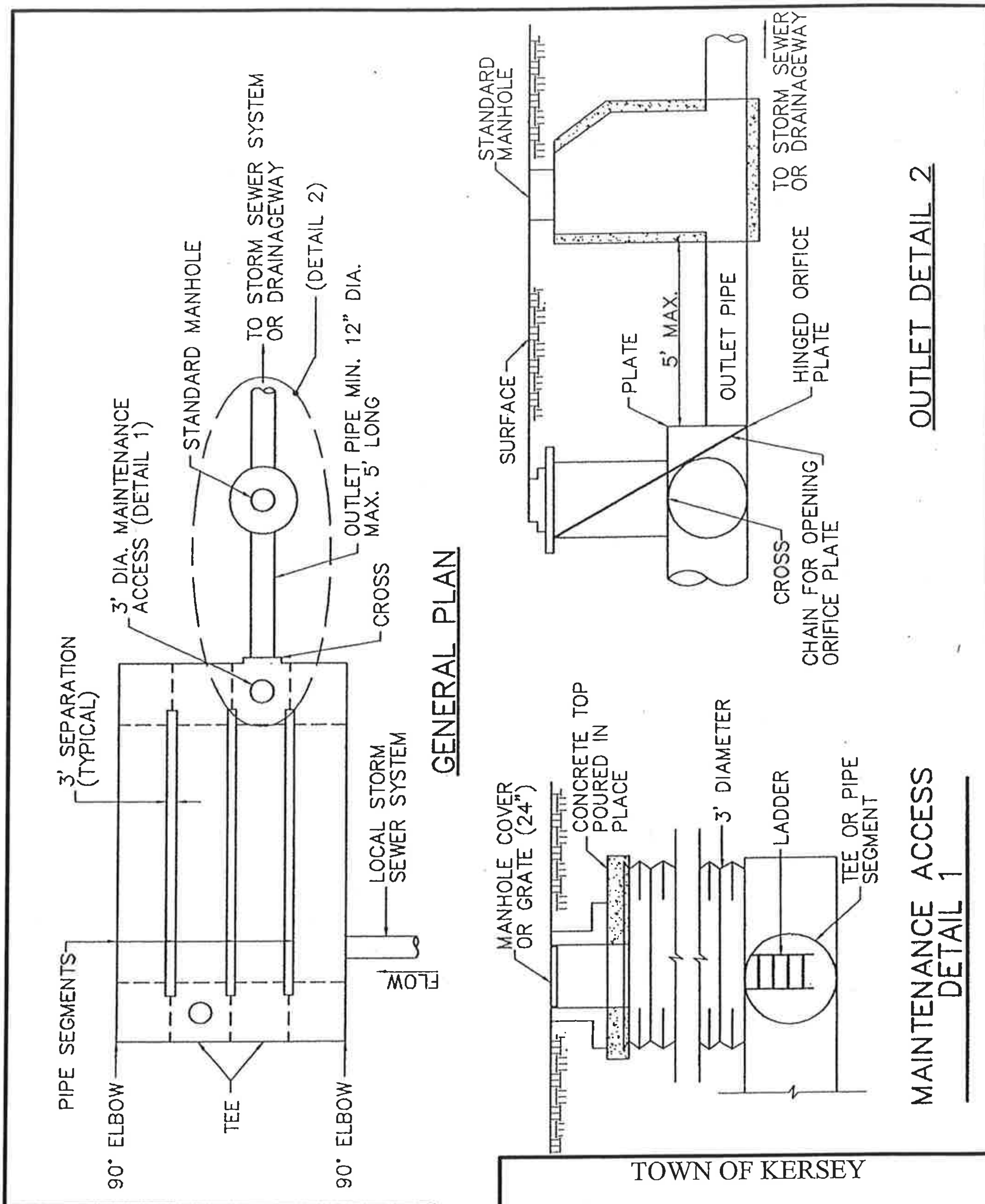
DATE: 12/04

APPROVED:

FJB

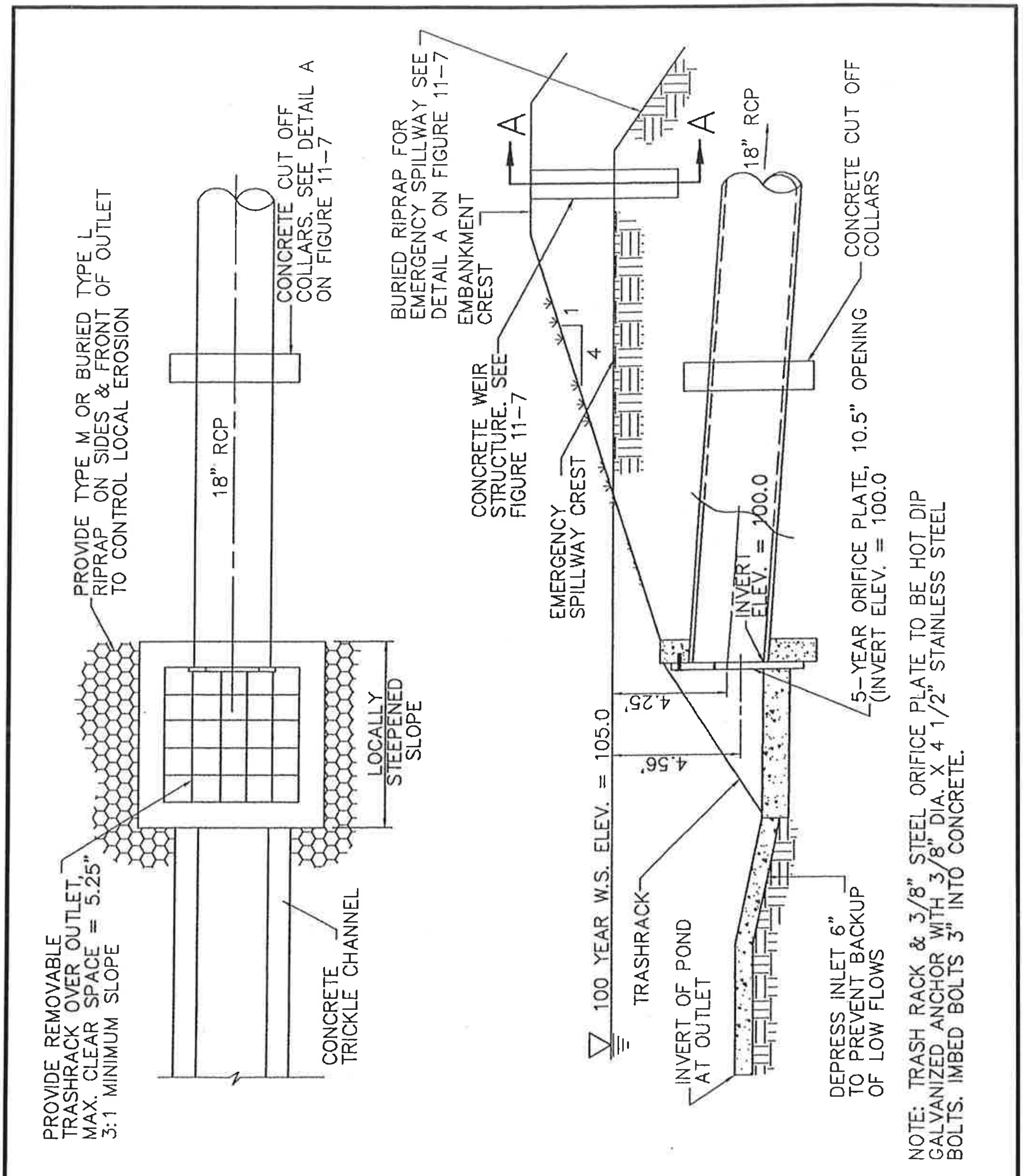
TOWN ENGINEER

SHEET FIG. 11–4



TOWN OF KERSEY	
UNDERGROUND DETENTION	
SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 11-5

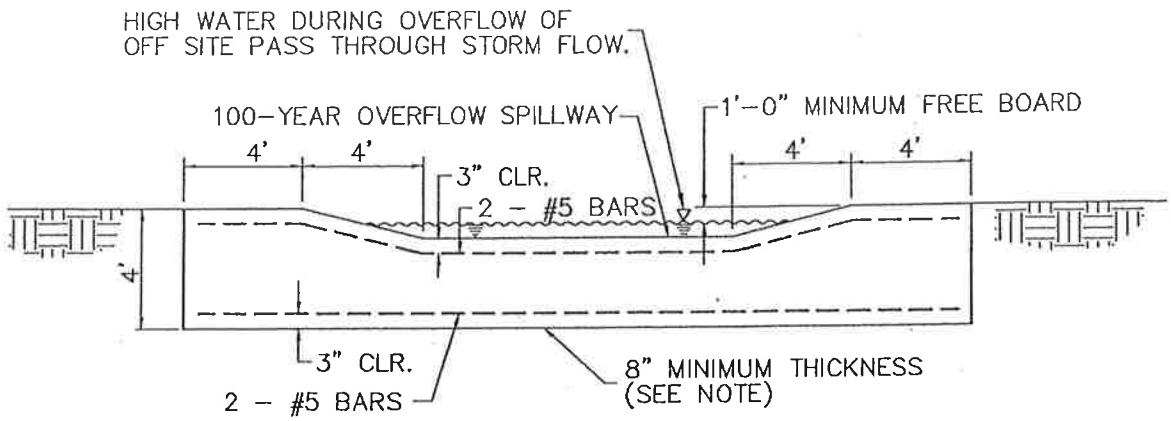


TOWN OF KERSEY

OUTLET DESIGN EXAMPLE
DETENTION POND SIZING

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

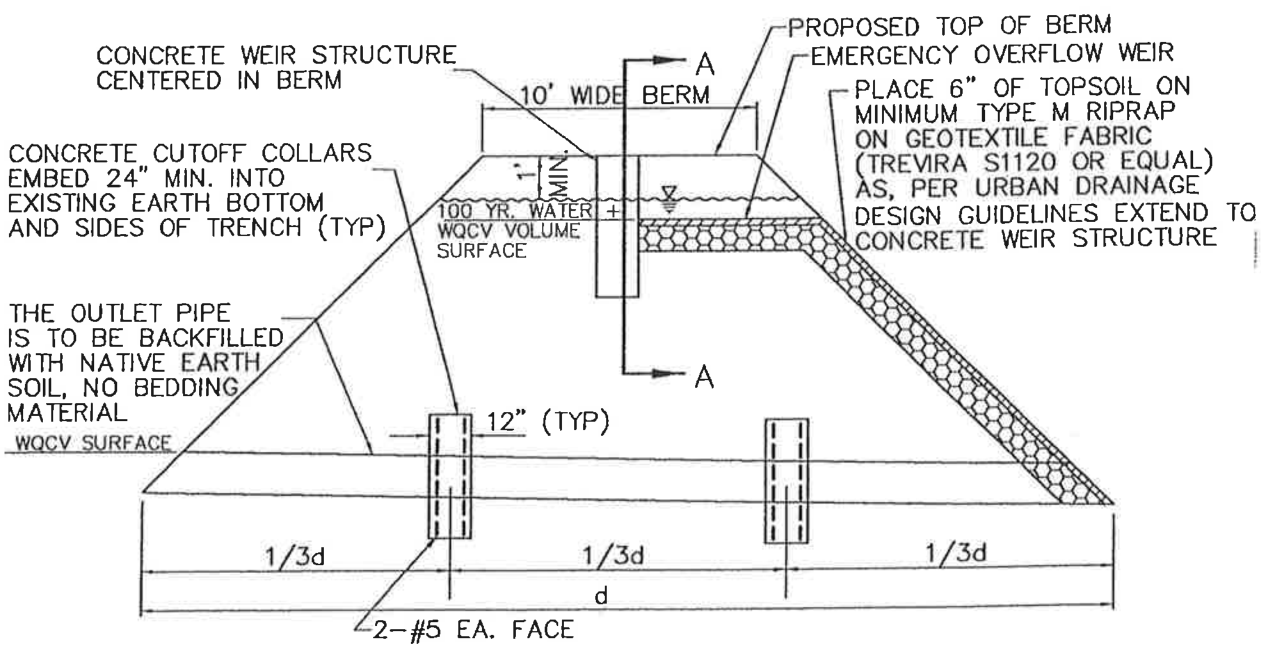
SHEET FIG. 11-6



NOTE:
TRENCH FOR WEIR OUTLET STRUCTURE USING NATIVE GROUND AS FORM WORK. CONSTRUCT WEIR 8" MINIMUM THICKNESS. UPON COMPLETION OF TRENCHING, PLACE TEMPERATURE STEEL AND CONCRETE IMMEDIATELY. FORM TOP 4".

SECTION A – A

CONCRETE WEIR OVERFLOW STRUCTURE



DETAIL A

OUTLET AND SPILLWAY DETAILS

TOWN OF KERSEY

OUTLET AND SPILLWAY DETAILS

SCALE: NONE

DATE: 12/04

APPROVED:

FJB

TOWN ENGINEER

SHEET FIG 11–7

SECTION 12.0 STORMWATER QUALITY ENHANCEMENT

12.1 INTRODUCTION

The character of the urban landscape affects both the quantity and the quality of stormwater discharged to receiving waters during and after each runoff event. The quality of stormwater runoff from developed lands and urbanized areas can be impacted by some or all of the sources and contaminants shown below. The increase in impermeable areas such as rooftops, parking lots and paved surfaces acts directly to impact stormwater quality by decreasing the opportunity for stormwater to infiltrate and percolate into the ground, and the absence of natural surfaces and vegetation allows for increased runoff velocity and pollutant carrying forces.

Possible Sources of Pollutants in Stormwater	
Source	Contaminant
Vehicles, Machinery and Industry	Metals, Lubricants, Solvents, Paints
Lawn Care, Gardening	Pesticides, Herbicies, Fertilizers
Household Chemicals Cleaners, Chlorine	Paints, Solvents, Detergents, Disinfectants
Pets and Animals	Fecal Material, Organic Wastes
Parking Lots	Oil, Grease, Automotive Fluids, Sediments

The intent of this section of the Criteria is to present minimum requirements for the implementation and use of Best Management Practices (BMP's) for stormwater quality control within the Town of Kersey. Compliance with these Criteria does not require water quality monitoring, or quantitative descriptions of pollutant load removal. Instead, a performance-based approach is described, whereby the existing principles and objectives of pollutant transport control are addressed in a general manner. Individual methods must be selected and implemented to best fit the conditions and requirements of each site.

These Criteria are developed from information and design guidelines presented in the Urban Storm Drainage Criteria Manual (USDCM), Volume 3, "Best Management Practices." The reader is referred to the USDCM for an extensive discussion of the development of stormwater quality controls and regionally acceptable BMP's. Section 13 of these Criteria discusses the Town's requirements for stormwater quality control due to erosion and sedimentation during the construction period.

12.2 REGULATION CONTROLLING DISCHARGES TO STORM SEWERS

The following regulations shall apply to discharges to storm sewers:

1. No person shall discharge non-stormwater wastewater, which contains pollutants from industrial, commercial or sanitary point sources to a storm sewer unless a

Colorado Discharge Permit System (CDPS) permit has been obtained for the discharge.

2. No person shall connect a system for drainage of industrial, commercial or sanitary wastewater, which contains pollutants, other than to convey stormwater runoff to a storm sewer unless a CDPS permit has been obtained.

3. All connections between industrial, commercial, or sanitary non-stormwater wastewater sewers or other drainage conveyances and storm sewers which are not the subject of a CDPS permit shall be disconnected even though the connection is unused.

12.3 OBJECTIVES FOR STORMWATER QUALITY CONTROL

The following principles and objectives for stormwater quality control shall be used by the Town to determine if adequate Best Management Practices have been proposed for a site during the design and development process:

1. Minimize, to the maximum extent practicable, impacts of stormwater on receiving waters. An effective level of urban pollutant removal should be accomplished by the selected BMP's.

2. Consider the sites physical constraints. The Town realizes that each site presents different topography, area limitations, and land use requirements. Select and design BMP's to work within the conditions on the site.

3. Evaluate the economic impacts of the selected BMP's. Controls must be evaluated for installation (construction) costs and for future operation and/or maintenance costs.

4. Recognize and incorporate multi-use benefits within stormwater quality features whenever possible. Land intensive BMP's such as detention/retention ponds and vegetative strips should be designed to incorporate recreational and aesthetic features such as open space and landscape values whenever possible.

12.4 Performance and Design Criteria

The Town of Kersey shall require that all land undergoing development incorporate BMP's to achieve, to the maximum extent practicable, the objectives of stormwater quality control. Due to the variability of factors such as land use, extent of development, existing improvements, and the physical characteristics of the site (including soils, slope, and runoff) it is expected that the BMP's designed for each site may vary considerably.

The Town of Kersey recommends the use of the following BMP's, as presented in the USDCM, Volume 3, "Best Management Practices":

- Minimization of Directly Connected Impervious Areas (DCIA)
- Irrigated grass buffer strips
- Grass lined swales
- Extended detention basins (dry basins)
- Retention ponds (w/permanent pool)
- Constructed wetlands
- Modular block porous pavement (as defined in the USDCM,
- Volume 3)

The design of these structural BMP's should be incorporated within the provision for flood control facilities whenever possible. In many cases adequate planning and design would allow for the inclusion of stormwater quality BMP's, such as detention, retention, or wetland areas within planned flood storage areas.

The Town shall evaluate the adequacy and appropriateness of the proposed BMP's based on their fulfillment of the previously stated objectives, as well as the satisfaction of the following minimum design criteria:

1. A site specific Stormwater Quality Control Plan describing the type of BMP's selected, a construction and implementation schedule, and a description of long-term maintenance requirements is approved by the Town.

2. The site is designed to minimize the extent of Directly Connected Impervious Areas (DCIA's) to at least 50% of Level 1 (as described below). The USDCM, Volume 3, "Structural Best Management Practices" identifies three levels of minimization as follows:

   Level 1: All developed impervious surfaces (rooftops, parking lots, sidewalks, etc.) are directed to drain over grass buffer strips before out letting onto public property.

   Level 2: In addition to the requirements of Level 1, all street curb and gutter systems are replaced with low velocity grass lined swales.

   Level 3: In addition to the measures required in Levels 1 and 2 above, all swales are oversized and elongated to provide increased detention benefits. Culvert crossings, such as for driveway intersections, are sized to control detention storage.

3. The maximum allowable slope for developed land surfaces utilizing Level 1 minimization of DCIA is 4%. Terracing and retaining wall construction may be required to maintain allowable slopes.

4. The design of developing sites shall incorporate one or more BMP's designed to capture and treat the calculated runoff equal to the 80th percentile rainfall event (see USDCM, Volume 3, "Best Management Practices, Section 5 - Stormwater Quality Hydrology). A variance may be allowed for development of small sites, such as the construction of small parking lot type detention ponds. Alternatives for stormwater quality treatment include extended detention basins (dry), retention ponds with a permanent pool, or constructed wetlands.

5. The evaluation and design for permanent erosion protection and stabilization measures shall be provided for all detention pond outlets, conveyance, outfall and channel facilities constructed on the site.

Detailed information on the development, application, design, and construction details for the BMP's required by the Town of Kersey can be found in the USDCM, Volume 3, "Best Management Practices". All updates and revisions to the USDCM shall be included in these Criteria.

The Town of Kersey encourages the innovative use and application of measures to insure stormwater quality control. The methods and applications of BMP's designed to meet the objectives of stormwater quality control are expected to increase and improve as the industry's experience and technology evolve. Applicants are encouraged to utilize the newest technology available, and incorporate the design data for these new methods in the Stormwater Quality Control Plan.

12.5 THE STORMWATER QUALITY CONTROL PLAN

A site specific Stormwater Quality Control Plan (SQCP) shall be submitted to the Town for review and approval. The SQCP should be consistent with the site's drainage report and could be included within the required drainage report for the project.

12.5.1 Preliminary Stormwater Quality Control Plan

The following information must be included within the Preliminary SQCP, which shall be submitted along with the Preliminary Drainage Report (refer to Section 2.3 of these Criteria) for the site:

1. Name, address and telephone number of the applicant and the Professional Engineer preparing the report.

2. Project description; briefly describing the nature and purpose of the development, the total area of the site, the area of disturbance involved, and the project location, including township, section and range.

3. Existing site conditions should be described, including existing topography, vegetation, and drainage. If wetlands are present on the site they must be

described: location, aerial extent, and type. It is the applicant's responsibility to determine and comply with all other federal or state regulations regarding the impact of development on wetlands.

4. A vicinity map indicating the general area and property lines for the site should be included. Acceptable scales range from 1" = 1000' to 1" = 8000'.

5. An exhibit or map of existing and proposed drainage features or facilities, and basin boundaries (existing and proposed) for the site. Complete basin boundaries shall be shown for all basins extending off of the site.

6. Neighboring areas must be described as to land use and existing features, such as adjacent streams, lakes, structures, roads, etc.

7. A description of the stormwater quality management-planning concept for the site.

8. Preliminary sizing and location of the selected BMP's.

9. A discussion of the maintenance requirements for all proposed BMP's, including suggested schedules, costs and designation of responsible party.

12.5.2 Final Stormwater Quality Control Plan

In addition to items numbered 1 through 7 as required in the Preliminary SQCP, the following information must be included within the Final SQCP, which shall be submitted along with the Final Drainage Report (refer to Section 2.4 of these Criteria):

1. A discussion of the final design, sizing and location of the selected BMP's.

2. Hydrologic, hydraulic and all other calculations used to size and design the selected BMP's.

3. A final site and grading plan indicating the path of all stormwater flow and the location of stormwater control and stormwater quality facilities.

4. Final construction drawings of the proposed stormwater quality improvements, if appropriate.

SECTION 13.0 CONSTRUCTION SITE EROSION AND SEDIMENT CONTROL

13.1 INTRODUCTION

Construction activities that disturb the natural soil and vegetation have the potential to increase soil erosion and sediment movement. The forces of rainfall, concentrated runoff, and even strong winds easily erode the disturbed, loose soil. Erosion and sediment control practices, also known as Best Management Practices (BMP's), shall be required to the maximum extent practicable, on all developing or redeveloping lands within the Town of Kersey. The implementation of these BMP's shall be designed to prevent disturbed soils from entering stormwater runoff and maintain stormwater quality at a level comparable to the historic runoff conditions, which existed prior to the construction activities.

These Criteria are developed from information and design guidelines presented in the Urban Storm Drainage Criteria Manual (USDCM), Volume 3, "Best Management Practices." The reader is referred to the USDCM for an extensive discussion of the development of erosion and sediment control BMP's.

The Town requires that a grading permit be obtained for construction activities on sites equal to or greater than 1 (one) acre, or a site that is a part of a larger common plan of development (which would include phased development of sites less than 1 (one) acre).

Other sites smaller than one acre but, due to the nature of their topography or location, provide a potential for significant negative impact on the Town's stormwater facilities, streets, or receiving waters, also must use BMP's during construction activity. The Town Engineer, or designee, shall identify such sites and provide written notice to the property owner with instructions to obtain a grading permit and prepare a SWMP plan prior to beginning or continuing grading activities.

In accordance with the requirements of the Federal Clean Water Act, the State of Colorado requires that a stormwater discharge permit be obtained for construction activities on sites equal to or greater than 1 (one) acre, or a site that is part of a larger common plan of development (which would include phased development of sites less than 1 (one) acre). Information on the requirements and forms necessary for obtaining a stormwater discharge permit from the State of Colorado may be obtained from the Stormwater Unit, Water Quality Control Division, of the Colorado Department of Public Health and Environment (303-692-3590).

13.2 OBJECTIVES FOR EROSION AND SEDIMENT CONTROL PRACTICES

The following objectives and principles of erosion and sediment control shall be used by the Town to determine if a site undergoing land disturbing activities has proposed and implemented adequate BMP's.

1. Fit development to the existing terrain and retain existing vegetation. The goal of this objective is to minimize the overall land disturbance, and maintain stormwater quality in a condition more similar to natural historic levels.

2. Schedule construction, grading, and land disturbing activities to minimize soil exposure, and avoid heavy runoff seasons. The best protection is prevention; therefore, effective scheduling should be used to minimize soil exposure between initial grading and completion of final grading or installation of improvements.

3. Manage stormwater flows to minimize erosion and sediment movement. This objective would include diverting concentrated flows from disturbed slopes, minimizing the length and steepness of disturbed slopes, keeping runoff velocities low, and preparing or reinforcing drainage ways and outlets to receive runoff flows.

4. Do not allow increased sediment movement off of the site. All sediment disturbed on site should be contained and either redeposited in a more stable location, or removed from the site.

5. Inspect, maintain, and remove all measures when appropriate. Scheduling will be highly dependent on the selection of BMP's, the rainfall/runoff events occurring during the land disturbance period, and the establishment of permanent stabilization.

13.3 Performance and Design Criteria

The Town shall require that BMP's for construction site or land disturbing activities be designed and implemented for each site in a manner that addresses the objectives and principles of erosion and sediment control (see Section 13.2). Given that the land use, topography, soils, and runoff flows will vary from site to site, it is expected that the proposed BMP's for each site will also vary considerably.

The Town of Kersey recommends the use of the following BMP's for erosion and sediment control, as presented in the USDCM, Volume 3, "Best Management Practices":

Stabilization and Erosion Control	Sediment Control
Surface Roughening	Sediment Basins and Traps
Mulching	Vehicle Tracking
Tackifier	Slope Drains and Dikes
Re-vegetation	Straw Bale Barriers
Erosion Blankets/Mats	Silt Fence
Drainage Way Protection	Inlet Filters

Design criteria and construction details for the selected BMP's are presented in the USDCM.

The Town encourages the innovative use and application of measures to adequately and efficiently control erosion and sediment movement due to land disturbing activities. Methods and applications of BMP's designed to meet the objectives for erosion and sediment control are expected to grow, improve, and expand. Owners of land undergoing land-disturbing activities are encouraged to utilize the newest technology available and incorporate the design data for these new methods in the Erosion and Sediment Control Plan (ESCP).

The Town shall evaluate the adequacy and appropriateness of the proposed BMP's based on their fulfillment of the previously stated objectives, as well as the satisfaction of the following minimum performance and design criteria:

1. An Erosion and Sediment Control Plan and Site Plan are approved by the Town.

2. Adjacent properties are protected from increased erosion and/or sediment deposition.

3. Construction access routes protect adjacent properties from sediment and mud tracking through either immediate placement of street base or construction of mud pads.

4. Timing and stabilization of sediment trapping practices is scheduled before site grading and construction.

5. Sediment traps/basins must be constructed if one (1) acre, or greater, of disturbed land drains to a common outfall.

6. All disturbed areas shall be adequately stabilized as defined in the UNDCM, Volume 3, Section 3.2 "Mulching". Permanent or temporary soil stabilization shall be required within 7 days after final grade is reached. If disturbed areas or stockpiles are not brought to final grade within 30 days following the initial disturbance, or re-disturbance, temporary stabilization measures shall be required.

7. All storm sewer inlets shall be protected from the entry of sediment-laden water.

8. The landowner shall be held responsible for the long-term stability of cut and fill slopes and the successful establishment of permanent vegetative cover on exposed soil as defined in the UNDCM, Volume 3, Section 3.2 "Mulching".

9. Inspection of all erosion and sediment control BMP's shall be required at the end of each day's work, with necessary maintenance and repairs provided immediately.

10. All temporary erosion and sediment control measures shall be removed as soon as their function has been fulfilled. Sediment traps/basins shall be cleaned and removed, or stabilized, when all upstream areas are permanently stabilized.

11. Construction work in or directly adjacent to a watercourse shall require adequate bed and bank stabilization as defined in the UNDCM, Volume 3, Section 3.2 "Mulching". Construction work within a defined channel shall require a stream crossing structure for bed and bank protection.

12. Construction work in flowing channels is prohibited in the months of May and June.

13. The construction of underground utilities shall be included as a land disturbing activity. All excavated material shall be placed where sediment will erode back into the trench. All trenches shall be backfilled by the end of the days work; backfill shall be permanently stabilized before construction is considered complete.

13.4 Erosion and Sediment Control Plan

A site specific Erosion and Sediment Control Plan (ESCP) shall be submitted to the Town for review and approval. The ESCP shall consist of two components: the first component shall be a narrative report describing the site, the proposed land disturbing activities, and the recommended BMP's for erosion and sediment control; the second component of the ESCP shall be a site plan.

The ESCP should be consistent with the site's drainage report, and could be included within the required drainage report for the project.

13.4.1 Preliminary Erosion and Sediment Control Plan

The submittal for the Preliminary ESCP Report (which shall be submitted with the Preliminary Drainage Report as described in Section 2.3 of these Criteria) shall consist of the following information:

Narrative Report (Preliminary)

1. Name, address, and telephone number of the applicant and the Professional Engineer preparing the report.

2. A project description briefly describing the nature and purpose of the land disturbing activity, the total area of the site, and the project location including township, range, and section.

3. The existing site conditions should be described, including existing topography, vegetation, and drainage. If wetlands are present on the site they must be described: location, aerial extent, and type. It is the applicant's responsibility to determine and comply with all other federal or state regulations regarding the disturbance of wetlands.

4. A vicinity map indicating the general area and property lines for the site should be included. Acceptable scales range from 1" = 1000' to 1" = 8000'.

5. Neighboring areas must be described as to land use and existing features such as streams, lakes, structures, roads, etc.

6. Soils information for the site should include soil type and names, mapping unit, erodibility, permeability, hydrologic soil group, depth, texture, and soil structure. This information may be obtained from the soil report for the site, from soil reports available for adjacent sites, or from Soil Conservation Service information. The source of information must be indicated.

7. Area and volume (in cubic yards) of the estimated quantity of excavation and fill on the site, and the surface area (acres) of the proposed disturbance.

8. A discussion of the approach to stormwater management on the site, including the erosion and sediment control measures to be used during construction. Briefly indicate the postconstruction stormwater quality control measures to be included in the site development, or refer to the site's Stormwater Quality Control Plan, if applicable.

Site Plan (Preliminary)

1. The site plan shall be presented on a 24" x 36" drawing, at scales ranging from 1" = 20' to 1" = 200'. The information required on this site plan may be placed on the site drainage plan, if it can be clearly presented.

2. Existing and if available proposed topography shall be shown at one or two-foot contour intervals. Topography information shall extend at least 100 feet beyond the property line.

3. Show the location of all on-site existing structures and hydrologic features on the site. All off-site existing structures or hydrologic features within 100 feet of the property boundaries shall also be shown. The path of both existing and proposed developed stormwater runoff flows leaving the site shall be identified.

4. Indicate the preliminary location of the proposed structures and development of the site.

5. Indicate the proposed limits of clearing and grading.

6. If required for the proposed construction or development activity, indicate preliminary locations of the following: temporary roads, soil stockpiles, and construction storage areas.

13.4.2 Final Erosion and Sediment Control Plan

The Final Erosion and Sediment Control Plan (which shall be submitted with the Final Drainage Report, as described in Section 2.4 of these Criteria) shall be based on the comments and review of the preliminary submittal, and the final construction plans for the project site. In addition to presenting all of the information included in the Preliminary ESCP, the submittal for the Final ESCP Report shall also include the following:

Narrative Report (Final)

1. Final summaries of the areas (acres) and volumes (cubic yards) of the excavation and fill on the site, and the total surface area of the proposed disturbance.

2. A description of erosion and sediment control measures which will be used on the site.

3. A construction schedule for all proposed site grading or other construction activities must indicate:

   - Start and completion dates for all construction.
   - Construction sequence, including the installation and removal time periods of erosion and sediment control measures.
   - Period and length of exposure of each area prior to the completion of temporary erosion and sediment control measures, as well as permanent stabilization.

4. Maintenance and inspection schedules for all erosion and sediment control measures during construction should be described.

5. A technical appendix should include all design calculations for determining rainfall and runoff, and sizing any basins, diversions or other conveyance or retention/detention facilities.

Site Plan (Final)

1. The final grading for the site, shown at one- or two-foot contour intervals, including elevations, dimensions, location, extent, and slope of all grading, including building site and driveway grades.

2. Final location of any soil stockpiles, storage areas (including equipment, fuel, lubricants and waste storage) and temporary roads designated for use during the construction period.

3. Plans of all drainage features, paved areas, retaining walls, cribbing, planting, temporary or permanent soil erosion control measures, or other features to be constructed in connection with, or as a part of, the proposed work. The drainage area of land tributary to the site in general, and isolated areas of disturbance, if applicable, should be shown. Tributary areas to all existing or proposed drain inlets should also be shown. All erosion measures should be depicted using the standard map symbols shown in Figure 13-1.

4. Design/detail drawings for any practices or measures not referenced in these Criteria should be included.

5. The following note shall be included on the plan:

   "This Erosion and Sediment Control Plan has been submitted to the Town of Kersey in fulfillment of the Town Criteria. Additional erosion and sediment control measures may be needed if unforeseen problems occur or if the submitted plan does not function as intended. The requirements of this plan shall run with the land and be the obligation of the land owner until such time as the plan is properly completed, modified, or voided."

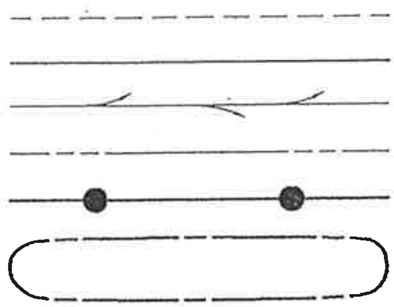
6. A signature block shall be placed below the note. The landowner and/or their legal agent shall affix their signature beneath the above note to acknowledge their review and acceptance of responsibility. The Professional Engineer responsible for the preparation of the Erosion and Sediment Control Plan shall also affix their signature and seal.

13.5 Review and Approval

The Town must issue a written approval of the Erosion and Sediment Control Plan prior to the issuance of a grading permit, subdivision plat approval, or site plan approval. The ESCP must be consistent with the Drainage Report submitted in accordance with the Town of Kersey Criteria. The Drainage Report and ESCP can be combined in one submittal package. Approval of the ESCP does not imply acceptance or approval of Drainage Plans, Utility Plans, Street Plans or any other aspect of site development.

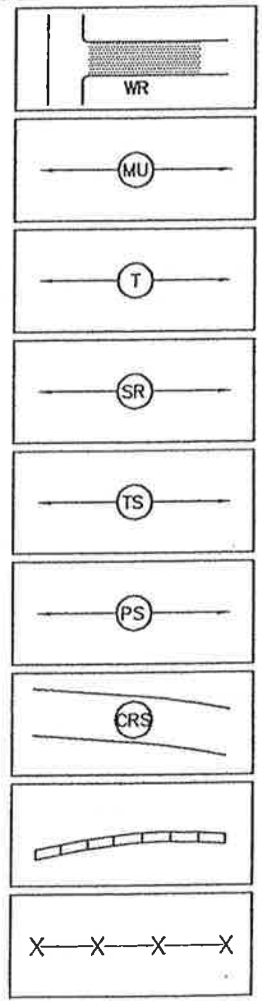
RECOMMENDED PLAN SYMBOLS:

EXISTING CONTOUR
FINISHED CONTOUR
DRAINAGE DIVIDE
LIMIT OF GRADING
STORMWATER
BOUNDARY OF A CONTROL MEASURE



TITLE	KEY
VEHICLE TRACKING CONTROL WITH WASH RACK	WR
MULCHING	MU
TACKIFIER	T
SURFACE ROUGHENING	SR
TEMPORARY SEEDING	TS
PERMANENT SEEDING	PS
CONSTRUCTION ROAD STABILIZATION	CRS
STRAW BALE BARRIER	STB
SILT FENCE	SF

SYMBOL

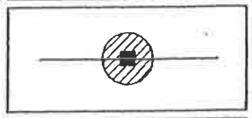
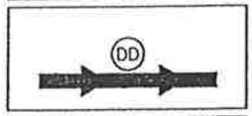
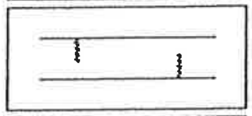
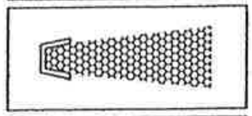
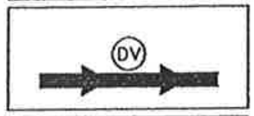
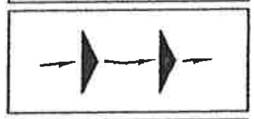
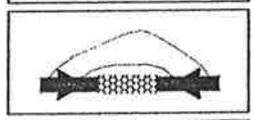
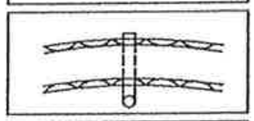
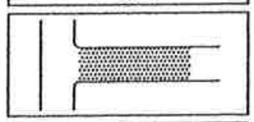
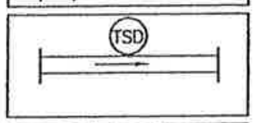
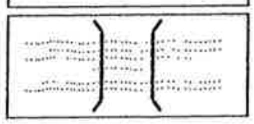


TOWN OF KERSEY

MAP SYMBOLS

SCALE: NONE
DATE: 12/04
APPROVED: FJB
TOWN ENGINEER

SHEET FIG 13–1

TITLE	KEY	SYMBOL
STORMWATER INLET PROTECTION	IP	
TEMPORARY DIVERSION DIKE	DD	
ROUGH CUT STREET CONTROL	RCS	
OUTLET PROTECTION	OP	
TEMPORARY CHANNEL DIVERSION	DV	
CHECK DAM	CD	
TEMPORARY SEDIMENT TRAP	ST	
TEMPORARY SEDIMENT BASIN	SB	
VEHICLE TRACKING CONTROL	VTC	
TEMPORARY SLOPE DRAIN	TSD	
TEMPORARY STREAM CROSSING	SC	

TOWN OF KERSEY

MAP SYMBOLS

SCALE: NONE	DATE: 12/04
APPROVED: FJB TOWN ENGINEER	

SHEET FIG. 13-1 CONT.