Cornet Creek Drainage Maintenance and Flood Mitigation Study, Colorado

Submitted to:

Town of Telluride
Dept. of Public Works
P.O. Box 397
Telluride, Colorado 81435

Submitted by:

Mussetter Engineering, Inc.
1730 S. College Avenue, Suite 100
Fort Collins, Colorado 80525

MEI Project 07-14

March 11, 2008

CORNET CREEK DRAINAGE MAINTENANCE AND FLOOD MITIGATION STUDY EXECUTIVE SUMMARY

ES.1. INTRODUCTION

The Town of Telluride (Town), located in southwestern Colorado in the San Juan Mountains, lies primarily on the alluvial fan of Cornet Creek, which drains to the San Miguel River. Cornet Creek has been responsible for the majority of Telluride's historic flooding problems. On average, under existing conditions, overbank flows occur along reaches of Cornet Creek one in every two years. Additionally, two destructive debris flows occurred on July 27, 1914, and August 1, 1969, and caused deposits of mud and rock with widespread depths of about 2 feet ranging to as much as 6 feet in localized areas (Mears et al., 1974). The most recent flood event occurred on the fan on July 23, 2007, blocked culvert and bridge crossings and damaged property on the north side of town. Most of the significant debris flow events have been caused by heavy rainfall following saturation of the soils in the basin, while most of the flood events have been caused by localized, high-intensity summer rainstorms.

In addition to delivering large amounts of debris during flood events, Cornet Creek conveys a significant amount of sediment on an annual basis. In recent years, the bed of the creek has aggraded by up to 3 feet in certain areas over the period of a single year. As a result, the Town's Public Works Department has routinely removed sediment from the channel under permits obtained from the U.S. Army Corps of Engineers (USACE).

This study was designed to evaluate the geologic, geomorphic, hydrologic, hydraulic, and sediment-transport characteristics of Cornet Creek in order to:

1. Develop an appropriate channel grade to maintain or improve capacity without inducing channel instability,

2. Assess the hydraulic and sediment-transport impacts of the recently replaced North Townsend Street Bridge (November 2005) and of replacing/improving low capacity culverts at Dakota and Pacific Avenues, and

3. Evaluate the potential for debris flows to occur along Cornet Creek and to investigate potential debris-flow mitigation techniques and state-of-the-art early warning systems.

ES.2. WATERSHED CHARACTERISTICS

Cornet Creek is an approximately 2.7-mile long, south-draining tributary to the San Miguel River that has a drainage area of about 2.4 square miles. The highest elevation in the basin is about 13,300 feet, and the relief from the apex of the Cornet Creek alluvial fan to the upper elevations of the basin is about 4,400 feet, giving an overall basin slope of approximately 3.3 percent. Glacial erosion of the south and southwest facing slopes of the basin has over-steepened the above-timberline slopes (40 to 50 degrees), which along with the continuing freeze-thaw erosion of the exposed relatively weak and erodible bedrock, provides an upper basin source of sediment.

The annual precipitation for the region is approximately 27 inches, 17 inches of which generally falls as snow between October and April, with the remainder falling as summer rainfall and thunderstorms (Dibble and Associates, 1983). Annual snowmelt flows, because of their relatively long duration, are capable of transporting available in-channel sediment supplies from the basin to the alluvial fan, but the magnitudes of both the flows and the sediment transport are relatively low. The highest peak flows are generated by high-intensity rainstorms in the mid-to-late summer. Recently, thunderstorm generated floods have occurred in August 2003 and July 2007.

Historic glacial erosion of the upper elevations of the Cornet Creek basin has produced extensive glacial till deposits in the basin. Saturation of these glacial deposits prior to the occurrence of intense summer thunderstorm precipitation resulted in their failure and the generation of a significant debris flow in 1969, which had an estimated bulked peak flow of between 10,000 and 14,000 cfs (Mears et al., 1974). The magnitude of a large debris flow that occurred in 1914 was estimated as being similar to the 1969 event, but the primary sediment source area has not been identified with any certainty (Mears et al., 1974). The lower reach of Cornet Creek above the alluvial fan apex contains at least three very large mass failure scarps within glacial till deposits on the east side of the creek both up- and downstream of Cornet Falls. Saturation-induced mass failure of the till deposits in this location are very likely to have been associated with historic debris flows on the alluvial fan. The presence of the mass failure scarps in the lower portion of the basin suggests that debris flows are not just generated in the upper basin.

ES.3. CHANNEL CHARACTERISTICS

The portion of Cornet Creek within the Town of Telluride is located on an alluvial fan, and extends about half a mile upstream from the San Miguel River to the mouth of the canyon. Average channel gradients range from almost 20 percent in the upper portion of the fan to only about 3 percent just upstream from the San Miguel River, which is typical of an alluvial fan longitudinal profile. Typical dimensions of the channel are approximately 20 feet wide by 3 feet deep, and the sinuosity (ratio of stream length to valley length) of the creek is very low. The creek does not have a defined floodplain adjacent to the channel because of the convex across-fan profile of the alluvial fan. As a result, flood flows that overtop the channel banks are not contained along the stream corridor, and will follow the natural topography which generally slopes away from the creek, impacting numerous public and private properties. Under existing conditions, there is a 50-percent probability, on an annual basis, of the channel capacity being exceeded at various low-channel capacity locations along the creek.

Surface sediment samples collected in June 2007, indicate that the median size (D_{50}) of the bed material ranges from about 37 mm (1.5 in.) to 54 mm (2.1 in.), and coarsens slightly in the upstream direction. Two bulk sediment samples collected in local depositional areas indicate that a considerable amount of sand-sized material is also transported by the creek. Typical of steep, low sinuosity, gravel/cobble-bed streams, the upper portion of Cornet Creek on the alluvial fan is characterized by a boulder step-pool morphology, which is not as evident in the downstream portion of the creek where the gradient is much flatter and the bed material is finer. The boulders were probably delivered to the upper portion of the fan by historic debris flows, but the natural morphology of the creek is somewhat masked due to maintenance activities. The primary hydraulic controls along the project reach are man-made and consist of 10 bridges and culverts.

ES.4. HYDROLOGY

Due to the lack of historical flow data and the discrepancies between previously estimated peak flows, the flood hydrology of the study reach and upstream drainage basin was developed from a rainfall-runoff model using the USACE HEC-HMS computer software (USACE, 2006). The purpose of the model was to estimate the runoff hydrographs at the mouth of the canyon, where Cornet Creek enters the Town of Telluride, for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval (RI) storms. The predicted peak discharges on Cornet Creek are based on rainfall driven events, and range from about 290 cubic feet per second (cfs) (30 acre-feet (ac-ft)) during the 2-year storm to about 1,490 cfs (121 ac-ft) during the 100-year storm (**Table ES-1**).

Table ES-1. Summary of computed peak discharges and flow volumes entering the Town of Telluride from the Cornet Creek watershed, based on the HEC-HMS model results.

Recurrence Interval (yrs)	Annual Probability of Occurrence (%)	Peak Discharge (cfs)	Storm Event Volume (ac-ft)
2	50	287	30
5	20	482	47
10	10	659	65
25	4	915	84
50	2	1,176	106
100	1	1,491	121

ES.5. HYDRAULICS

An existing conditions (June 2007) hydraulic analysis of Cornet Creek between the confluence with the San Miguel River and the apex of the alluvial fan was performed for a range of flows up to, and including the 100-year peak discharge using the U.S. Army Corps of Engineers HEC-RAS v.3.1 computer software (USACE, 2005). The purpose of the analysis was to evaluate the existing channel capacity and to estimate hydraulic conditions (e.g., velocity, depth, shear stress) in the creek to facilitate incipient motion and bed-material transport capacity calculations throughout the project reach.

Based on computed water-surface elevations, the existing (June 2007) capacity of the channel (i.e., elevation at which the flows would begin to break out of the channel and impact adjacent property or infrastructure) is lower than the 2-year peak discharge (i.e., 50-percent annual probability of flooding) in many locations throughout the project reach. Reach-averaged hydraulics indicate that main channel velocities range from about 4 to 8 feet per second (fps) during the 2-year event and from about 5 to 12 fps during the 100-year event. Average flow depths range from 2 to 4 feet, and effective widths range from 17 to 30 feet over the entire range of modeled flows.

The hydraulic model results also indicate that the capacities of the bridge and culvert crossings are typically less than the adjacent channel segments, which is largely due to sediment deposition. The existing culverts at Dakota Avenue (located near the upstream end of the study reach) and Pacific Avenue (located near the downstream end of the study reach) have much less than the 2-year peak discharge (287 cfs) conveyance capacity.

ES.6. SEDIMENT TRANSPORT

Based on the results of the hydraulic analysis, a sediment-transport analysis was carried out to evaluate the vertical stability of the project reach. The investigation consisted of an incipient motion analysis to assess the range of flows over which the existing bed material is mobilized, and a sediment-continuity analysis to evaluate potential aggradation/degradation trends along study reach.

The incipient motion analysis indicates that extremely small discharges of less than 10 cfs are required to mobilize the bed material along the entire study reach because of the steepness of the channel. This result is consistent with field observations made during a June 2007 site visit, which indicated that the bed material in the channel was at or near incipient-motion conditions at a measured discharge of approximately 5 cfs.

An evaluation of the effective channel shear stress shows that, as expected, subreach-averaged bed shear stresses are greater in the upper portion of the study reach, where the channel gradients are significantly steeper. The upper portion of the creek also has a slightly higher channel capacity, which allows the shear stress to continue to increase with increases in discharge. In the lower portion of the creek, banks are overtopped at relatively low flows, which results in only minor increases in shear stress at higher discharges.

Consistent with the shear stress results, sediment transport volumes computed for the 2- through 100-year storm hydrographs show a general trend of decreasing transport capacity in the downstream direction. Transport volumes at the upstream end of the study reach range from about 280 tons during the 2-year event to about 2,200 tons during the 100-year event. The upstream subreach transport rates are much higher at the higher recurrence interval flows. As the channel bed gradient decreases in the downstream direction, transport capacities also decrease to about 25 tons during the 2-year peak flow to about 80 tons during the 100-year peak flow.

According to the sediment-continuity analysis, the majority of the channel is aggradational during all of the storms that were analyzed, which is consistent with field observations following the July 2007 flood and long-term maintenance requirements that the Town has encountered. Subreach-averaged aggradation depths typically range from less than 0.5 feet to 1 foot at the 2- and 100-year peak flows, respectively. Locally, significantly higher amounts of aggradation could occur within a number of the subreaches.

ES.7. CHANNEL DESIGN RECOMMENDATIONS

The limited conveyance capacity of Cornet Creek is exacerbated by the fact that it does not have an adjacent floodplain to help convey flood flows. As a result, improvements designed to increase flow capacity must be restricted to the channel itself. The recommended channel improvements were developed by initially modifying the existing longitudinal profile to a grade that will increase channel capacity within the limits imposed by the elevations of existing structures that are not going to be modified. To maximize capacity within the limits imposed by the narrow stream corridor, a cross-sectional channel template consisting of a bottom width of 4 feet with 2H:1V sideslopes was selected. Ideally, a slightly wider channel width would be preferred. However, a 4-foot channel bottom width was selected as being the narrowest practical width that existing equipment available to the Town could realistically construct while also providing a reasonable increase in channel capacity.

The proposed improvements should increase channel capacity by fully containing flows up to at least the 5-year peak discharge, and in many location, up to the 10-year peak as well (not accounting for potential aggradation associated with these flow events). The proposed channel improvements will reduce the annual probability of flooding to about 20 percent (1 in 5 years). In general, the combination of reducing flow losses due to overtopping and narrowing of the channel at low discharges results in slight increases in channel velocities and hydraulic depths. Increases in reach-averaged velocities range from less than 0.1 fps at the 2-year event to about 3 fps at the higher recurrence interval flows. Estimated hydraulic depths under the improved design conditions increase by almost 2 feet during the 100-year event.

An evaluation of the effects of the proposed channel improvements on sediment transport indicated that sediment-transport capacities are more uniform up to the 5-year event from upstream to downstream, and that the increased hydraulic capacity of the channel allows the associated sediment-transport capacity to continue to rise during larger flood events (at least until flow begins to exceed the channel capacity). At the higher magnitude, lower frequency events, the proposed channel improvements do not significantly affect the existing aggradational/degradational patterns along Cornet Creek.

Because of the very limited capacity of the existing channel, any measures taken to improve channel capacity along the creek will require a considerable amount of excavation. Assuming that the channel can be excavated to the recommended elevations, with a bottom width of 4 feet, and stable 2H:1V sideslopes wherever corridor widths allow, the estimated volume of material that would need to be excavated is approximately 3,800 cubic yards. Based on the sediment-transport modeling, annual maintenance of the channel could potentially require the removal of up to an estimated 300 cubic yards of material. A channel profile for Cornet Creek within the project reach was developed to increase channel capacity while maintaining a reasonable level of stability (Appendix B). The profile provides a vertical limit to excavation during future channel maintenance operations.

ES.8. DEBRIS FLOW PREDICTION AND MITIGATION

Prediction of, and mitigation for, debris flows are dependent to a great extent on the ability to identify sediment source areas, triggering causes (hydro-meteorological), in-channel or channel-margin sediment sources that cause bulking of floods to develop mud and debris flows, run-out geometries on the downstream alluvial fans and event frequencies. Managing the risks of these natural hazards can include land use planning, installation of preventative measures, stabilization of slopes, implementation of early warning systems, installation of protective structures and development of measures and procedures to restore normal conditions after the event (Greminger, 2003).

The magnitude of the bulked flows during the 1914 and 1969 debris flows (9,000 to 14,000 cfs) that have recurrence intervals of about 50 years (2-percent annually) have so greatly exceeded the capacity of the Cornet Creek channel, even if the recommended improvements are implemented (about 500 cfs), it is apparent that there is a very high risk of damages on the fan if a similar magnitude event were to occur in the future. Because existing constraints and encroachments along the creek eliminate the possibility of constructing a channel that would safely convey a 14,000-cfs debris flow across the Cornet Creek fan as recommended by Mears et al. (1974), damages are very likely to be extensive on the fan in similar types of events.

Given that it is not possible to convey significant debris flows across the fan in a constructed channel (e.g., the debris conveyance flumes in the Town of Ouray), the only other measures available are to reduce the volume of the debris flow by trapping a portion of it upstream of the fan, or to provide an early-warning system that would reduce the risk to persons, but would not reduce damages to structures. Numerous types of debris-trapping structures have been used in regions of the world where debris flows are a problem (Romang et al., 2003). Open-type or slit-type check dams using beam, grid or column structures have been constructed more recently to allow some sediment bypass (Fiebiger, 1997; VanDine et al., 1997). In general, any open-type check dam should serve one or both of two purposes: debris-flow breaking and debris-flow retention (Wu and Chang, 2003). A functional debris-flow breaker should separate solid debris from the transporting fluid, whereas the debris-flow retention function should selectively retain harmful debris and allow the finer sediment to return to the river. Physical model and field prototype results in Taiwan have shown that crossing-truss dams that are composed of two rows of overlapping triangular trusses with suitable spacing within the impact row serve as debris-flow breakers and the spacing produced by the overlapping of impact and outlet rows creates solid-fluid separation (Wu and Chang, 2003). Given the space constraints in the lower part of the canyon, it is highly unlikely that a debris trapping structure could be built that would detain the debris-flow volume. However, a crossing-truss dam structure might be able to detain enough of the coarse material to reduce the downstream damages. Further research into the performance and costs of this type of structure is required.

Early-warning systems for debris-flow generation are generally based on an analysis of antecedent meteorological conditions that include both precipitation and temperature as well as real-time monitoring of precipitation event intensity. These systems require a relatively dense network of automated weather stations and have not been shown to be very reliable for debris-flow prediction. The primary advantage of these systems is that they provide sufficient time for dealing with the threat, and they are most applicable to residential areas where there is a high potential for loss of life. The primary disadvantage of the prior-warning systems is that the forecast accuracy is usually poor, and thus there is a high potential for false alarms.

Real-time warning systems, consisting of trip wire sensors, infrared photo beams, ground-vibration sensors, and acoustic sensors, on the other hand, tend to have a much higher accuracy, but they generally provide a much shorter reaction time because of the high velocities of the debris flows. The greatest value of the real-time systems is that they can be automatically linked to lights and barriers at transportation crossings, thereby lessening the risk of pedestrian, rail or automobile casualties.

ES.9. SUMMARY

The results of this investigation of Cornet Creek can be summarized as follows:

1. The existing capacity of the Cornet Creek channel (less than 2-year conveyance capacity) can be increased to convey a 5-year peak flow (20 percent annual probability of flooding) without inducing channel instability;
2. Increasing the conveyance capacity to the 5-year peak flow will require approximately 3,800 cubic yards of excavation;
3. Ideally, the channel improvements along Cornet Creek should be treated as a single system;
4. To maintain the 5-year conveyance capacity, approximately 300 cubic yards of material will have to be removed from the channel annually;

5. The culverts at Dakota and Pacific Avenues will need to be replaced by single-span bridges to achieve the 5-year conveyance capacity;
6. Debris-flow early-warning systems can be installed in the upper Cornet Creek basin, but they will not provide extensive warning of an in-progress event, nor will they reduce damages to structures on the Cornet Creek fan; and
7. Debris retention may be possible at the apex of the fan, but further research into appropriate methods is required.

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1. INTRODUCTION

The Town of Telluride (Town), located in southwestern Colorado in the San Juan Mountains, has developed around two primary watercourses, the San Miguel River and Cornet Creek. The Town lies primarily on the alluvial fan of Cornet Creek, which drains into the westerly flowing San Miguel River (**Figure 1.1**). Although smaller than the river, Cornet Creek drains an approximately 2.4- mi^2 watershed of high mountainous terrain, and has been responsible for the majority of Telluride's historic flooding problems, consisting primarily of mud and debris flows. Historically, numerous debris flows have occurred along Cornet Creek, with the two most destructive events occurring on July 27, 1914, and August 1, 1969. These events caused deposits of mud and rock with widespread depths of about 2 feet ranging to as much as 6 feet in localized areas (Mears et al., 1974). The most recent flooding event occurred on July 23, 2007, blocked culvert and bridge crossings and damaged property on the north side of town. Most of the significant flood events have been caused by heavy rainfall following a period of prolonged wet weather.

In addition to delivering large amounts of debris during flood events, Cornet Creek conveys a significant amount of sediment on an annual basis. In recent years, the bed of the creek has aggraded by up to 3 feet in areas over the period of a single year. As a result, the Town's Public Works Department must routinely remove sediment from the channel under permits obtained from the U.S. Army Corps of Engineers (USACE).

Due to the significant hazards caused by Cornet Creek, the creek and contributing watershed have been the subject of numerous studies:

1973. Preliminary Hazard Map of Telluride, Colorado. P.E. Carrara, March 1973. Report Prepared as Part of NASA-PY Project Grant Number NGL-06-003-200.

1973. Preliminary Report – Mudflow Hazard on Cornet Creek; Telluride, Colorado. August 3, 1973. Mears, Institute of Arctic & Alpine Research, UC.

1974a. Debris Flow Hazard on Cornet Creek at Telluride, Colorado. January 1974. Institute of Arctic & Alpine Research.

1974b. Cornet Creek – Flood Study. October 1974. Wing Engineering, Inc., Montrose, Colorado.

1975. Cornet Creek – Flood Study. January 1975. Wing Engineering, Inc., Montrose, Colorado.

1978. Flood Insurance Study, Town of Telluride, San Miguel County, Colorado, Federal Insurance Administration, March 1978.

1983. Drainage Master Plan, Telluride, Colorado, Dibble & Associates, June 1983.

1983. Coronet Creek – Debris and Flood Control Plan. August 1983. Dibble & Associates Consulting Engineers, Phoenix, Arizona.

1985. Coronet Creek Drainage Study. Town of Telluride, Colorado. May 1985. ARIX, Grand Junction, Colorado.

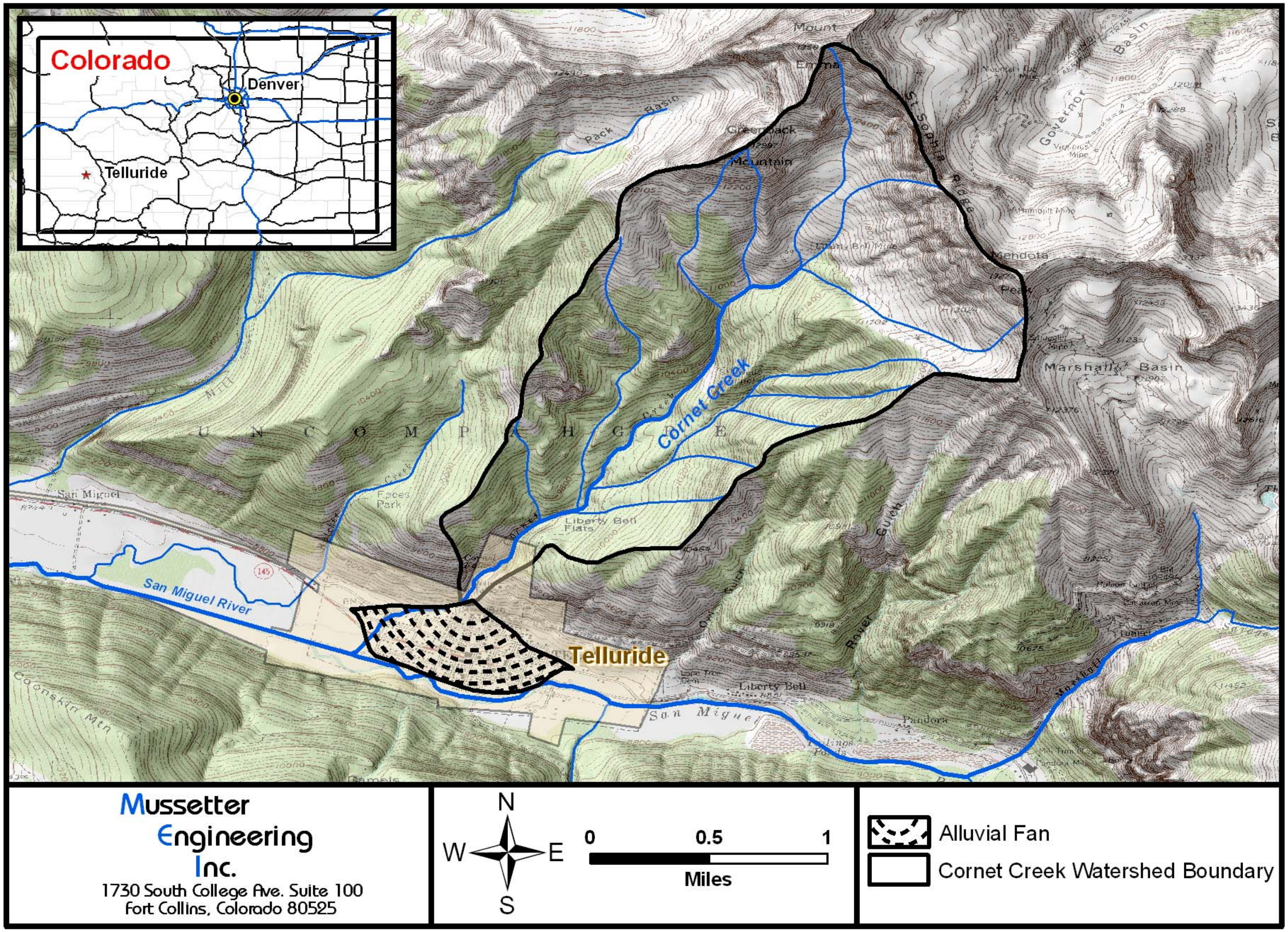


Figure 1.1. Vicinity map of Telluride showing the general location of the study reach.

1992. FEMA Flood Insurance Study. San Miguel County, Colorado and Incorporated Areas. Revised: September 30, 1992.

1996. Town of Telluride Surface Water Hydrology Study. Resource Engineering, Inc., August 28, 1996.

2003. Investigation of Cornet Creek, August 2003 Flooding. Letter report to the Public Works Director. Resource Engineering, Inc., September 3, 2003.

A summary of the recommended actions from the studies listed above was provided by the Town of Telluride, along with a notation of whether the recommendation was implemented (Town of Telluride, 2007):

1. Use a discharge of 14,000 cfs for the design of a flood protection channel through Telluride, which should protect Telluride against recurrences of the 1914 and 1969 floods. (Mears, 1973) [Not implemented.]

2. Keep the channel entirely clear of culverts, bridges, and other constructions unless they are designed to allow passage of large boulders and debris. Boulders moved by past floods were as much as 10 feet long and could easily cause damming and overflowing of the present channel. (Mears, 1973) [Not implemented.]

3. A debris storage area should be [constructed] in the triangle between the highway and West Columbia Avenue. A 72-inch culvert should be located at the west end of the triangle and a new channel cut to the river. (Wing Engineering, 1974) [Not implemented.]

4. To protect the bottom and sideslopes from erosion, it would be necessary to cover the channel with woven wire. There are commercial wire rock baskets (gabions) available. (Wing Engineering, 1974) [Not implemented.]

5. Use a design storm of 7,000 cfs (1914 Flood). The debris storage requirement for a 7,000 cfs flow would be approximately 24,000 cubic yards. The channel would have a bottom width of 15 feet, a top width of 43 feet and a depth of 7 feet. (Wing Engineering, 1974) [Not implemented.]

6. Removal of existing structures and preparation of debris storage area. All bridge crossings and culverts except the crossing at West Columbia and the highway should be removed. Ultimately the highway crossing should be enlarged and the waterway area of the West Columbia Bridge should be improved. All of the remaining structures are totally inadequate to pass large flows and must be removed. (Wing Engineering, 1974; Wing Engineering 1975) [Not implemented.]

7. Concrete (the lower) portion of channel. (Wing Engineering, 1974; Wing Engineering 1975) [Not implemented.]

8. Improve stilling basin and reinforce dike at head of Aspen Street. (Wing Engineering, 1975) [Not implemented.]

9. Investigate, identify and dedicate flooding breakout points upstream from Colorado Avenue for greater dispersal of flood flows. Consider purchase of drainage easement on

undeveloped land to accommodate flooding and cause future development in areas which may be directly exposed to hazards of flooding to construct buildings that minimize flood damage. Potential breakout points are (1) west along Dakota Avenue, (2) south along Townsend Street, (3) west of Galena Avenue, and (4) westerly between Columbia Avenue and Colorado Avenue. (ARIX, 1985) [Partially implemented on an informal basis.]

10. Provide for the design and development of flood breakout channels identified above. (ARIX, 1985) [Not implemented.]

11. Provide for design and development of Coronet Creek Channel improvements within the town limits. (ARIX, 1985) [Not implemented.]

12. Provide for total design and development of an upstream debris basin. (Dibble & Associates, 1983; ARIX, 1985) [Not implemented]

13. Provide for an annual budget item for ongoing capital improvement and maintenance of flood protection facilities. (ARIX, 1985) [Implemented 2003 and 2008; otherwise, maintenance funding used from Streets O&M Budget or other line items.]

14. Provide for design and construction of improvements along Cornet Creek as it relates to the project to replace Colorado Avenue culvert. (ARIX, 1985) [Implemented: Colorado Avenue culvert was replaced sometime in the 1980s; Columbia Avenue Bridge was replaced in the 1990s; Townsend Street culverts were replaced with a span bridge in 2005.]

15. Remove vegetation encroaching into the channel. Willows and trees in the channel have become anchors for debris dams which then cause gravel and sediment deposition. (Resource Engineering, Inc. [REI], 2003) [Implemented on a limited basis when channel clean out occurred after the August 16, 2003, flood/debris event and after the Pacific Street Culverts were cleaned in 2003. Staff would request a recommendation from Council, based on the most recent analyses, as to which trees and willows to remove.]

16. Clean dead wood and debris from the channel. In several locations logs and construction debris were observed. This debris tends to catch on the vegetation and stream banks and create the debris dams. (REI, 2003) [Implemented yearly since 2000 in the spring and sometimes in the summer or fall.]

17. Remove gravel and sediment deposits from the channel. In some areas up to 4 feet of gravel or more has been deposited in the channel. Additional deposition may result in overbank flooding and re-routing the stream. (REI, 2003) [Implemented on a limited basis: Stretch along Galena west of Townsend in 2003 and 2007; stretch between Townsend and Dakota, 2003; stretch between Colorado and Pacific 2003, 2006, 2007.]

Based on recent needs for the Town to routinely clean out portions of the channel to maintain an adequate conveyance capacity, a decision was made to obtain more specific information regarding the appropriate level of excavation in the creek and to provide information about state-of-the-art flood warning systems. Mussetter Engineering, Inc. (MEI) was, therefore, retained by the Town of Telluride to conduct detailed hydrologic, hydraulic, and sediment-transport analyses of Cornet Creek in order to investigate and provide recommendations regarding the following items:

1. Development of an appropriate channel grade to maintain or improve capacity without inducing further channel instability,
2. Impact assessment of the recently replaced North Townsend Street Bridge and of replacing/improving low capacity culverts at Dakota and Pacific Avenues on flooding potential, and
3. Evaluation of the potential for debris flows to occur along Cornet Creek and the investigation of potential debris-flow mitigation techniques and early warning systems.

1.1. Scope of Work

In completing this work, MEI performed the following specific tasks:

1. Current available information and data were obtained and reviewed, including the following specific items:

    a. Historic discharge records and water supply records for Stillwell Tunnel.

    b. Previous debris flow and flood studies of Cornet Creek (listed above).

    c. The Effective Flood Insurance Study (FEMA; 1992a) and Flood Insurance Rate Map (FEMA; 1992b) for the project reach within San Miguel County.

    d. Historic sediment excavation frequencies and quantities.

    e. Black and white aerial photography of the study area taken in 2003, obtained from Foley Associates, Inc (FAI).

    f. 2-foot contour interval digital elevation mapping of the Town of Telluride, developed in 2003, and obtained from FAI.

    g. 10-meter horizontal and 0.1 meter vertical resolution digital elevation model (DEM) of the study area and upstream watershed obtained from GeoCommunity GIS database.

    h. Soil maps of the project area and contributing watershed from the Soil Survey of the Ouray Area, parts of Gunnison, Hinsdale, Ouray, San Juan, and San Miguel counties, Colorado (NRCS, 2007).

    i. Geology maps of the study area (Burbank and Luedke, 1966)

2. A site visit to the project reach was conducted by Dr. Michael Harvey, P.G. and Mr. Chad Morris, P.E. of MEI on June 25 through 27, 2007. During the site visit, the MEI personnel met with the Town's Public Works staff, further evaluated the geomorphic setting, made direct observations of conditions along the creek, collected channel bed samples, and identified locations for subsequent detailed topographic and bathymetric data collection.

    a. Geomorphic mapping. The geomorphic characteristics of the channel and the man-made and natural controls were mapped during the field inspection of Cornet Creek and its watershed. Source areas for previous debris flows were located and inspected.

b. Channel bed-material sampling. Surface bed-material samples were collected at 6 locations along the project reach using the pebble count technique (Wolman, 1954). Subsurface samples were collected at 2 locations for use in the sediment transport analysis.

c. Topographic and bathymetric surveys. Topographic and bathymetric surveys of Cornet Creek from the San Miguel River confluence to the Falls were conducted by FAI to obtain additional information to supplement the 2003 2-foot contour mapping of the area. Details of required data, such as cross-section locations, hydraulic controls, bridges, and top-of-bank locations were specified by MEI's Project Engineer. The existing-conditions topographic survey was conducted in July 2007. An additional topographic survey of the creek was also conducted by FAI in October 2007, to ascertain changes to the creek caused by the July 23, 2007, flood event and subsequent excavation measures.

3. A hydrologic evaluation of the magnitude, duration, and frequency of flows from the Cornet Creek watershed was conducted. This included a review and analysis of all available hydrologic information for the study reach, including previous hydrologic analyses, historic discharge records, and water supply records from the Stillwell Tunnel. Refinement of flow estimates were then determined by developing a rainfall-runoff model created using the USACE HEC-HMS computer software (USACE, 2006).

4. A 1-D hydraulic model of Cornet Creek that extends from the confluence with the San Miguel River to approximately 260 feet upstream from Dakota Avenue was developed using the USACE HEC-RAS computer software (USACE, 2005), the 2003 2-foot contour mapping, and supplemental data collected under Task 2, above. The model was run for a range of flows up to and including the 100-year peak discharge based on the results from the HEC-HMS model. A second HEC-RAS model was also developed to evaluate the effects of the July 23, 2007, flood event and subsequent excavation measures.

5. The clear-water hydraulic conditions along the project reach were analyzed under existing conditions over a range of flows up to and including the 100-year peak discharge. This analysis included an assessment of the channel capacity throughout the study reach, and a flooding impact assessment of the proposed bridge/culvert replacements at Dakota and Pacific Avenues and the recently replaced North Townsend Street Bridge.

6. An incipient motion and sediment-continuity analysis was conducted to assess the range and duration of flows over which the existing bed material is mobilized, and the transport capacity of the bed material during bed-mobilizing flows.

7. Recommendations were made for an appropriate channel profile of Cornet Creek within the project reach that will maintain a reasonable level of stability and represent a vertical boundary during future excavation operations.

8. Preliminary recommendations regarding potential debris-flow mitigation options and early warning systems were developed.

1.2. Authorization and Study Team

This study was performed by Mussetter Engineering Inc. under a contract with the Town of Telluride Public Works Department. The Town's Project Manager is Ms. Karen Guglielmone. Dr. Michael Harvey was MEI's Principal Geomorphologist and Project Manager. Mr. Chad Morris, P.E. was MEI's Project Engineer. Topographic surveys of the project reach were conducted by Foley Associates, Inc. under a subcontract agreement with MEI.

2. DESCRIPTION OF EXISTING WATERSHED AND CHANNEL

2.1. General Watershed Characteristics

Cornet Creek is an approximately 2.7-mile long, south-draining tributary to the San Miguel River (Figure 1.1) that has a drainage area of about 2.4 square miles. The highest elevation in the basin is about 13,300 feet, and the relief from the apex of the Cornet Creek alluvial fan to the upper elevations of the basin is about 4,400 feet, giving an overall basin slope of approximately 3 percent. Glacial erosion of the south and southwest facing slopes between Mount Emma and Mendota Peak has over-steepened the above-timberline slopes (40 to 50 degrees), which along with the continued freeze-thaw erosion of the exposed bedrock, provides an upper basin source of sediment that is conveyed by a system of gullies to a convergence area near the Liberty Bell Mine at an elevation of about 11,500 feet (Mears et al., 1974). Below an elevation of about 11,000 feet the watershed is forested.

The climatic conditions in the watershed, the bedrock geology and geomorphic history are the primary controls on sediment production and yield from the Cornet Creek drainage basin to the alluvial fan on which the Town of Telluride is built. The annual precipitation for the region is approximately 27 inches, 17 inches of which generally falls as snow between October and April, with the remainder falling as summer rainfall and thunderstorms (Dibble and Associates, 1983). Annual snowmelt flows, because of their relatively long duration, are capable of transporting available in-channel sediment supplies from the basin to the alluvial fan, but the magnitudes of both the flows and the sediment transport are relatively low. The highest peak flows are generated by high-intensity rainstorms in the mid-to-late summer. Recently, thunderstorm generated floods have occurred in August 2003 and July 2007. Both storms resulted in significant sediment transport and deposition in the Cornet Creek channel between Dakota Avenue and the San Miguel River. Depending on the antecedent moisture conditions in the basin, the thunderstorm-generated peak flow events can translate into debris flows due to concurrent saturation-induced slope failures within the basin. A more detailed discussion of the basin hydrology is provided in Section 3 of the report.

The bedrock geology of the Cornet Creek basin has the potential to produce large quantities of sediment. The upper and middle sections of the basin are underlain by highly erodible, Tertiary-age volcanic tuffs and breccias (Gilpin Peak Tuff and San Juan Formation) and the lower part of the basin is underlain by erodible Permian–Cretaceous age sedimentary formations (Cutler Fm., Dolores Fm., Morrison Fm., Dakota Sandstone) that are composed of interbedded shales, siltstones, sandstones and limestones (Burbank and Luedke, 1966) (**Figure 2.1**). Pleistocene-age glacial erosion of the erodible rock units and subsequent deposition has resulted in the presence of extensive glacial drift deposits (moraines) in both the upper part of the basin in the vicinity of the Liberty Bell Mine and along one or both valley sideslopes for approximately the lower 1 mile of Cornet Creek above the alluvial fan (Figure 2.1). Saturation of the upper basin glacial deposits prior to the occurrence of intense thunderstorm precipitation predisposed failure of the glacial deposits and generation of the 1969 debris flow that caused extensive damage within the Town of Telluride (Mears et al., 1974). Although mine tailings and mining equipment from the Liberty Bell Mine were incorporated into the debris flow, the tailings were not considered to be the main source of the debris-flow materials (Mears et al., 1974). The bulked peak flow for the 1969 debris flow was estimated to be between 10,000 and 14,000 cfs (Mears et al., 1974).

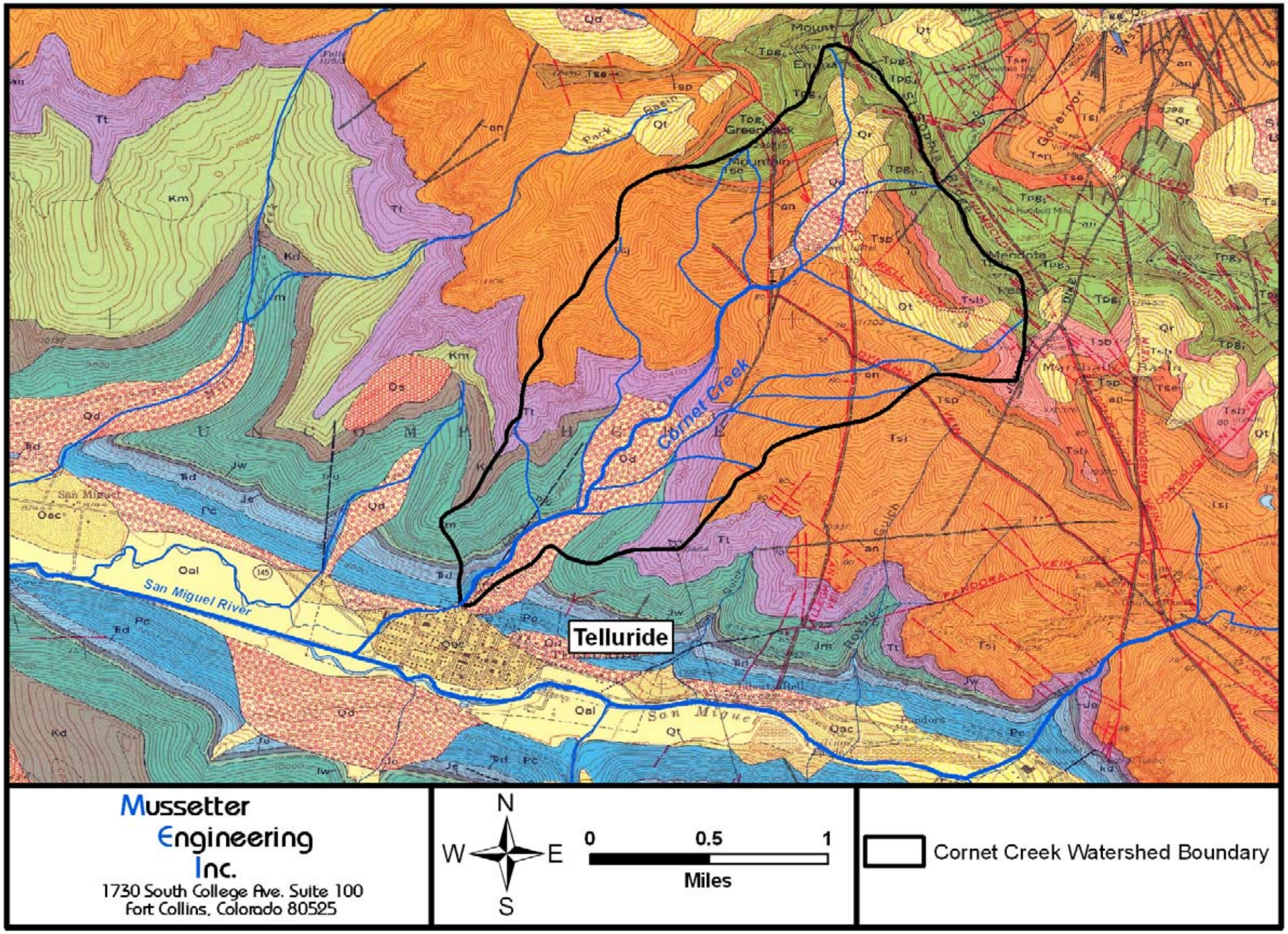


Figure 2.1. Geologic map of the Cornet Creek basin (Burbank and Luedke, 1966). The geologic units of importance to this study are Tpg=Gilpin Peak Tuff; Tsj=San Juan Formation; Pc=Cutler Formation; Trd=Dolores Formation; M=Morrison Formation; Kd=Dakota Sandstone; Qd=Glacial Drift.

Estimates of the bulked peak discharge of the 1914 debris-flow event that was also preceded by saturated soil conditions in the basin, suggest that it was on the same order as the 1969 event (Mears et al., 1974). The primary source area for the 1914 debris flow has not been identified with any certainty, although Liberty Bell Mine tailings have been implicated (Carrara, 1973). It was assumed on the basis of the presence of rock types that are located in the upper basin in the 1914 deposits that the debris flow was generated in the upper basin (Mears et al., 1974), but these same rock types are also present in glacial till deposits in the lower basin.

Field observation of the lower reaches of Cornet Creek above the alluvial fan apex indicated the presence of at least three very large mass failure scarps within glacial till deposits on the east side of the creek both up- and downstream of the Falls in the area described as the Liberty Bell Flats on the USGS topographic map (**Plates 2.1 and 2.2**). Saturation-induced mass failure of the till deposits in this location are very likely to have been associated with historic debris flows on the alluvial fan. Cores extracted from trees growing at the base of the middle scarp which is located in the vicinity of the Falls, indicate that the slope failure occurred at least 76 years ago, and thus may have been associated with the 1914 event if it assumed that it took about 17 years for the site to become stable enough for trees to become established. At the very least, mass failure of the tills would have blocked the narrow canyon, and this would have resulted in a dam-break type of flood. The presence of the mass failure scarps in the lower portion of the basin suggests that debris flows are not just generated in the upper basin. Therefore, it is probably not safe to assume that early warning systems located in the upper portion of the basin will always provide advance notice of a debris-flow event prior to its arrival at the apex of the alluvial fan.

Plate 2.1. View upstream of a failure scarp in glacial till deposits located on the east side of Cornet Creek and immediately downstream of the Falls.

Plate 2.2. View of head of failure scarp on the east side of Cornet Creek upstream of the Falls. The failure took place in glacial till deposits.

2.2. Channel Characteristics

The focus of this study is on the portion of Cornet Creek within the Town of Telluride, which is located on an alluvial fan, and extends from the San Miguel River about half a mile upstream to approximately 260 feet upstream from Dakota Avenue. Based on a 2007 topographic survey conducted by Foley Associates, Inc., average channel gradients range from almost 20 percent in the upper portion of the reach to only about 3 percent just upstream from the San Miguel River, which is typical of an alluvial fan profile. Typical dimensions of the channel are approximately 20 feet wide by 3 feet deep, and the sinuosity (ratio of stream length to valley length) of the creek is very low.

An important characteristic of Cornet Creek relative to the scope of this study is that because the alluvial fan is a steep, coarse-grained, and convex feature, no substantial floodplain feature exists adjacent to the creek. As a result, flood flows that overtop the channel banks are not contained along the stream corridor, and will follow the natural topography which generally slopes away from the creek, impacting numerous public and private properties.

Surface sediment samples collected during the June 2007 site visit using the pebble count technique (Woman, 1954) indicate that the median size (D_{50}) of the bed material ranges from about 37 to 54 mm, and coarsens slightly in the upstream direction (**Figure 2.2 and Appendix A**). Two bulk sediment samples collected in local depositional areas indicate that a considerable amount of sand-sized material is also transported by the creek (Figure 2.2). Typical of steep, low sinuosity, gravel/cobble-bed streams, the upper portion of Cornet Creek on the alluvial fan is characterized by a boulder step-pool morphology, which is not as evident in the downstream portion of the creek where the gradient is much flatter and the bed material is finer. The boulders were probably delivered to the upper portion of the fan by historic debris flows, and within the channel the boulder steps dissipate hydraulic energy (Thomas 1999). The natural morphology of the creek is also somewhat masked due to maintenance activities. Upstream of the fan apex within the lower reach of the canyon, the channel morphology is characterized by boulder-steps. Most of the large boulders within the channel are derived from sedimentary rock formations that border the lower canyon, and thus are probably derived from rockfalls and not from downstream fluvial transport.

The USACE modified the apex of the Cornet Creek alluvial fan after the 1969 flood event by creating a small stilling basin and a berm/dike structure to divert the flow into the realigned channel. The historical alignment of Cornet Creek downstream from approximately Colorado Avenue has also been modified to travel directly south into the San Miguel River rather than west along the valley floor (Appendix A). (The date of realignment is unknown, but it is likely to have also occurred after the 1969 flood while the USACE was making other modifications to the channel.) The current alignment of the entire channel within the study reach is severely encroached upon by homes and infrastructure, and the entire Cornet Creek fan is covered with a combination of residential and commercial developments.

The geomorphic characteristics and existing conditions along the project reach were assessed during the June 2007 site visit to develop an understanding of the factors that control the behavior of Cornet Creek. The primary hydraulic controls along the project reach are man-made and consist of 10 bridges and culverts (**Table 2.1**, Appendix A). The downstream-most bridge is a low profile wooden pedestrian bridge designed to be overtopped at moderately high flows (Photo 1, Appendix A), which under June 2007 conditions only has about 1.5 feet of clearance below the low chord of the structure. Significant aggradation had occurred upstream of Pacific Avenue, which is the next upstream crossing. The two arch culverts under Pacific Avenue were estimated to be at least 5.5 feet in diameter, but sediment deposits have blocked

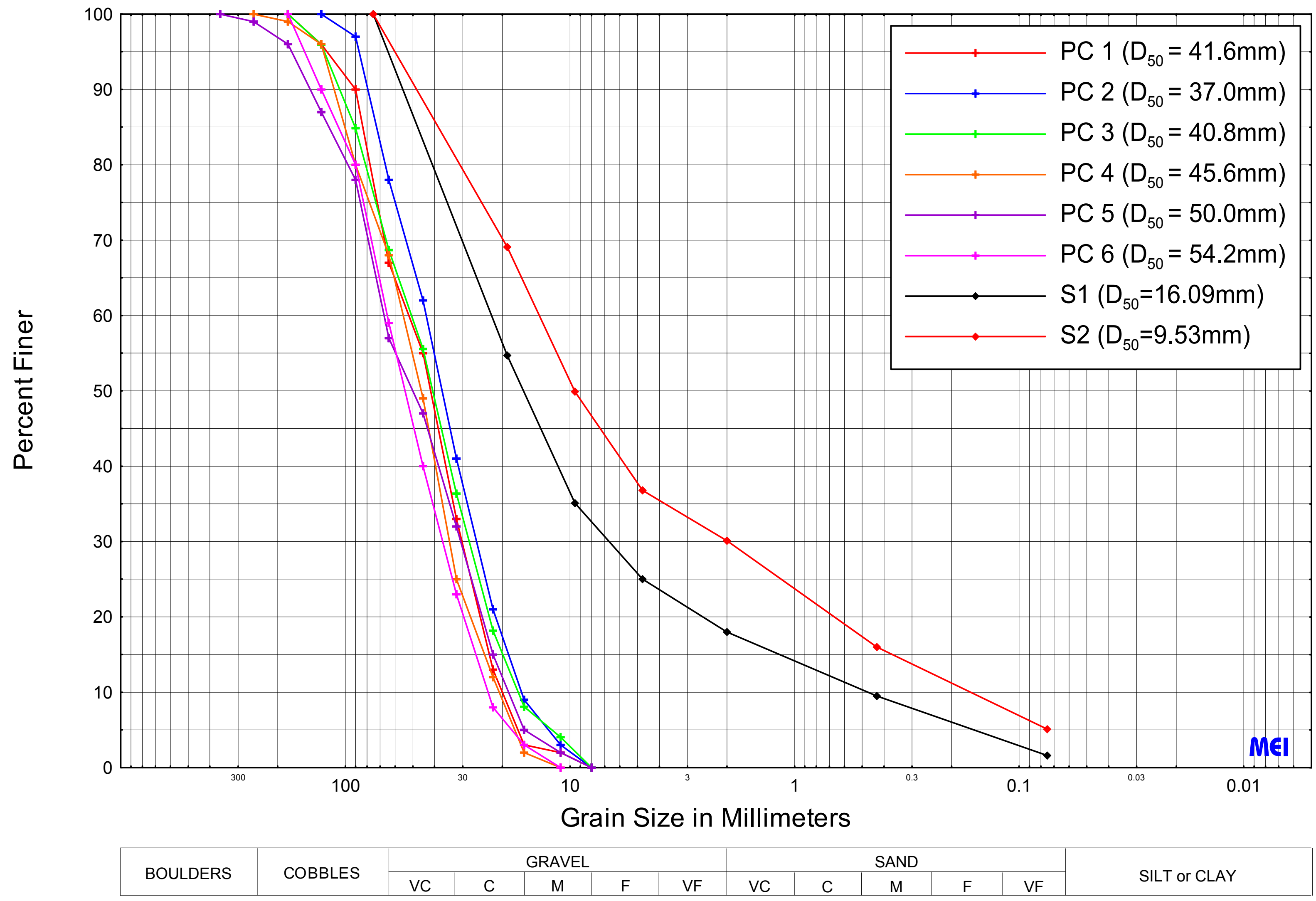


Figure 2.2. Grain-size distribution curves for the surface pebble counts (PC) and bulk sediment samples (S) that were taken along the project reach of Cornet Creek in June 2007.

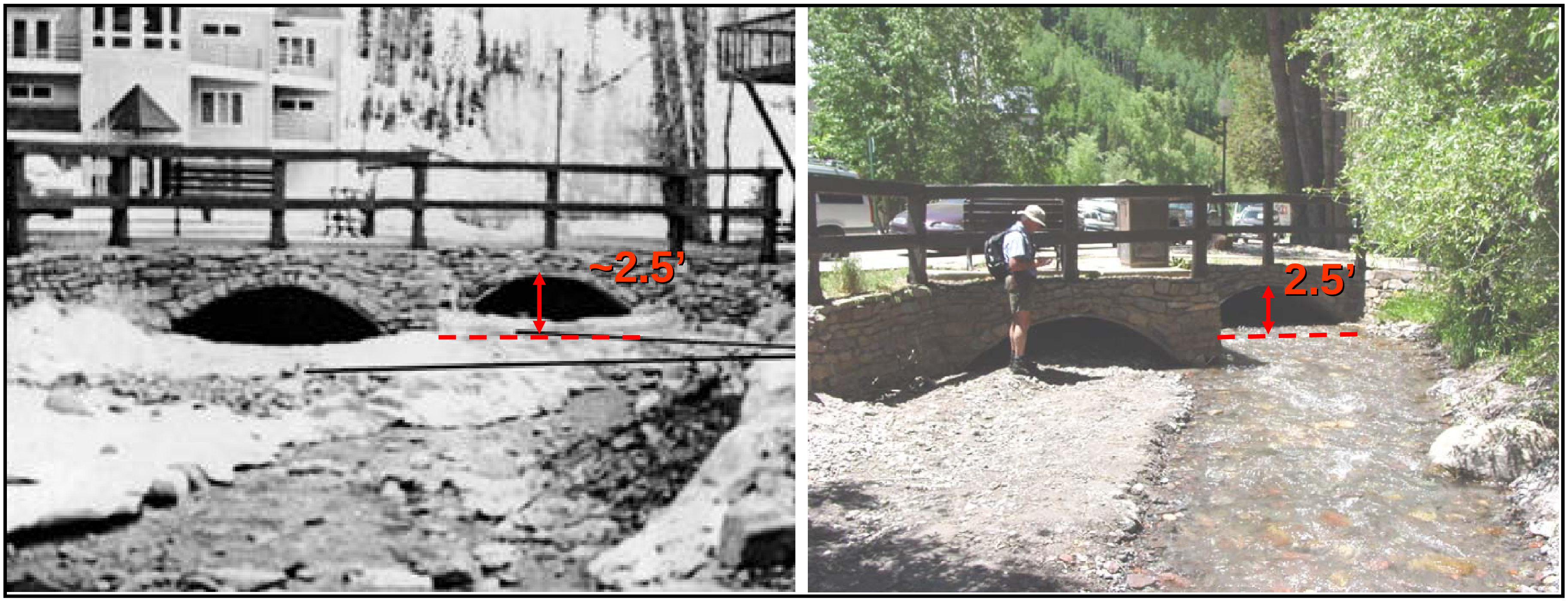
at least half of the available conveyance area (Photos 2 and 3, Appendix A). Based on field observations and results of this analysis (discussed in the following chapters), similar levels of aggradation in this area have occurred for many years. (**Plate 2.3**).

Table 2.1. Summary characteristics of existing bridges and culverts along Cornet Creek between the San Miguel River and approximately 260 feet upstream from Dakota Avenue.

Structure I.D.	Station (ft)	Structure Description
Private Footbridge u/s from Dakota	23+65	45-foot, pre-fab steel truss pedestrian bridge
Dakota Avenue Culvert	22+85	6-foot diameter corrugated metal pipe (cmp)
Private Footbridge d/s from Dakota	21+85	55-foot, pre-fab steel truss pedestrian bridge
Townsend St. Bridge	18+60	25-foot, pre-cast concrete roadway bridge deck with vertical concrete abutments
Galena Footbridge	17+90	18-foot, pre-fab steel truss pedestrian bridge
Columbia Avenue Culvert	11+40	20 feet wide by approx. 5 feet high, concrete span roadway bridge with vertical concrete abutments and concrete floor slab.
Footbridge d/s from Columbia Avenue	10+20	40-foot, pre-fab steel truss pedestrian bridge
Colorado Avenue Culvert	8+00	8 feet wide by 8 feet high concrete box culvert
Pacific Avenue Culverts	3+40	2, approximately 5.75-foot high corrugated metal arch culverts
San Miguel River Trail Footbridge	1+10	Low profile wooden pedestrian bridge

The segment of Cornet Creek between Pacific and Colorado Avenues (Photo 4, Appendix A) is characterized by relatively low bank heights, and shows signs of significant aggradation, which is very noticeable at the Colorado Avenue Culvert. This culvert is an 8 feet wide by 8 feet high concrete box, but field measurements indicate that only about 4 feet of available height remained at the outlet (Photo 5, Appendix A), with even less at the inlet. Upstream from Colorado Avenue near Sta 10+20 is located one of the four pre-fabricated pedestrian truss bridges in the reach. All of these pedestrian-truss bridges are high enough to not significantly impact the flow.

Columbia Avenue (Sta 11+40) is the next upstream road crossing. The capacity of this bridge has been improved in the past, and does not appear to be significantly impacted by aggradation (Photo 7, Appendix A). Upstream from Columbia Avenue, the channel gradient increases noticeably, and small cobble/boulder steps form the bed of the channel. Similar to most of the creek, this segment of channel is significantly encroached upon by buildings and roads (Photos 8 and 9, Appendix A). In an attempt to protect private property from flooding, a small earthen berm has been constructed along the left bank between Sta 15+00 and Sta 18+00 (Appendix A).



(a) **(b)**

Plate 2.3. View looking downstream at inlets to Pacific Avenue Culverts showing similar levels of aggradation in (a) 1985 and (b) 2007.

Upstream from the Galena Avenue footbridge, the channel gradient increases to about 15 percent and the channel contains many large boulders that were placed in the creek (recent discussions with the Town suggest that some of the boulders will likely be removed) (Photo 10, Appendix A). At Sta 18+60, the recently replaced Townsend Street Bridge (November 1, 2005) spans the creek, but since the channel bed is only about 4 feet below the low chord of the bridge, the overall capacity is somewhat limited (Photo 11, Appendix A).

Upstream from Townsend Street, the channel gradient is approximately 10 percent, and larger cobble/boulder steps characterize the bed. This portion of channel also contains larger boulders along the toe of the bank at various locations (Photo 12, Appendix A). However, determination of whether the boulders had been placed naturally or manually could not be ascertained during the field reconnaissance. A small berm has been constructed along the right bank and the adjacent access road (**Plate 2.4**).

Plate 2.4. View looking upstream at small berm upstream from Townsend Street between access road and right channel bank near Sta 20+00.

Approximately 120 feet downstream from Dakota Avenue, the channel widens and the gradient flattens significantly (Photo 13, Appendix A). This section represents a potential sediment deposition reach that is characterized by deposits of finer sediment. The elevation of the right bank is also extremely low, which increases the possibility of damages caused by flood or debris-flow events.

The channel is steeper immediately downstream from Dakota Avenue, and also contains some very large boulders that were probably added for erosion protection (Photo 14, Appendix A). Photos 15 and 16 of Appendix A show the outlet and inlet of the 6-foot diameter culvert at Dakota Avenue, respectively. The capacity of this culvert is very limited, as demonstrated during the July 2007 flood event, and replacement of this structure should definitely be considered. Upstream from Dakota Avenue, the gradient of the stream increases to almost 20 percent, and the creek can be described as more of as a series of boulder drops rather than a well-defined channel (Photos 17 and 18, Appendix A).

Just beyond the upper end of the detailed study reach, the Jud Weibe trail crosses the creek (Sta 26+75). Immediately upstream from the Jud Weibe trail bridge, on the trail to Cornet Falls, is the crown of a berm that was constructed by the USACE after the 1969 flood (Photo 19, Appendix A). The location of the small sediment detention basin (previously referred to as the "stilling basin") also constructed by the USACE is directly upstream from the Jud Weibe trail bridge in a wider section of the creek (Photo 20, Appendix A).

Cornet Creek continues approximately 0.2 miles upstream to Cornet Falls. This canyon-bound section of the creek is relatively narrow, and steeper than the portion of the creek on the downstream alluvial fan (**Plate 2.5**). It was not included in the detailed analysis of this study, but would need to be examined in further detail if two-dimensional debris-flow modeling is considered by the Town in the future.

Plate 2.5. View looking downstream at Cornet Creek from between Cornet Falls and the downstream alluvial fan.

3. HYDROLOGIC ANALYSIS

To provide a basis for assessing the characteristics of Cornet Creek, an understanding of the flood hydrology within the study reach and upstream drainage basin is required. Therefore, a hydrologic evaluation of the magnitude, duration, and frequency of flows from the Cornet Creek watershed was conducted. This included a review and analysis of all available hydrologic information for the study reach, including previous hydrologic analyses, historic discharge records, and water supply records from the Stillwell Tunnel.

The investigation revealed that very little flow data exists for Cornet Creek. It is an ungaged watercourse and the gaging station on the San Miguel River near Telluride was only operated for five years between 1961 and 1965. The gaging station on the San Miguel River near Placerville has about a 72-year record, but in addition to this gage being more than 15 miles downstream from Telluride, there is not enough information to directly compute corresponding flows on Cornet Creek. As a result, previously estimated peak discharge-frequency relationships have been based on empirical methods. The existing FEMA Flood Insurance Study (FIS) reports peak discharges on Cornet Creek based on the SCS Curve Number technique (FEMA, 1992), and a more recent hydrologic analysis determined peak runoff flows based on the Rational Method (REI, 1996). The results of these studies are not consistent (shown later in this section).

Due to the lack of available flow data and the discrepancies between previously estimated peak flows, the flood hydrology of the study reach and upstream drainage basin was developed from a rainfall-runoff model using the USACE HEC-HMS computer software (USACE, 2006).

3.1. Model Development and Assumptions

The hydrologic model was developed using HEC-HMS, which simulates the surface-water runoff response of a stream to precipitation by representing the basin as a system of interconnected hydrologic and hydraulic components. HEC-HMS simulates the precipitation-runoff response of the watershed by performing mathematical computations for four hydrologic and hydraulic processes:

1. Precipitation
2. Infiltration/interception
3. Transformation of precipitation excess to subbasin outflow
4. Hydrograph routing

The 2.4-mi^2 watershed was modeled using six sub-watershed basins, which were delineated on the basis of terrain type and drainage patterns, with areas ranging from 0.28 to 0.7 mi^2 (**Figure 3.1**). The subbasin boundaries, slopes, and elevations were based on the USGS 1:24,000 scale Quadrangle and digital elevation model (DEM). Stream flowpaths in the watershed were delineated from a combination of the USGS quadrangle and the DEM using Geographical Information System (GIS) ARC Hydro software.

The purpose of the model was to estimate the runoff hydrographs at the mouth of the canyon, where Cornet Creek enters the Town of Telluride, for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval (RI) storms. Simulations for each recurrence interval were performed using

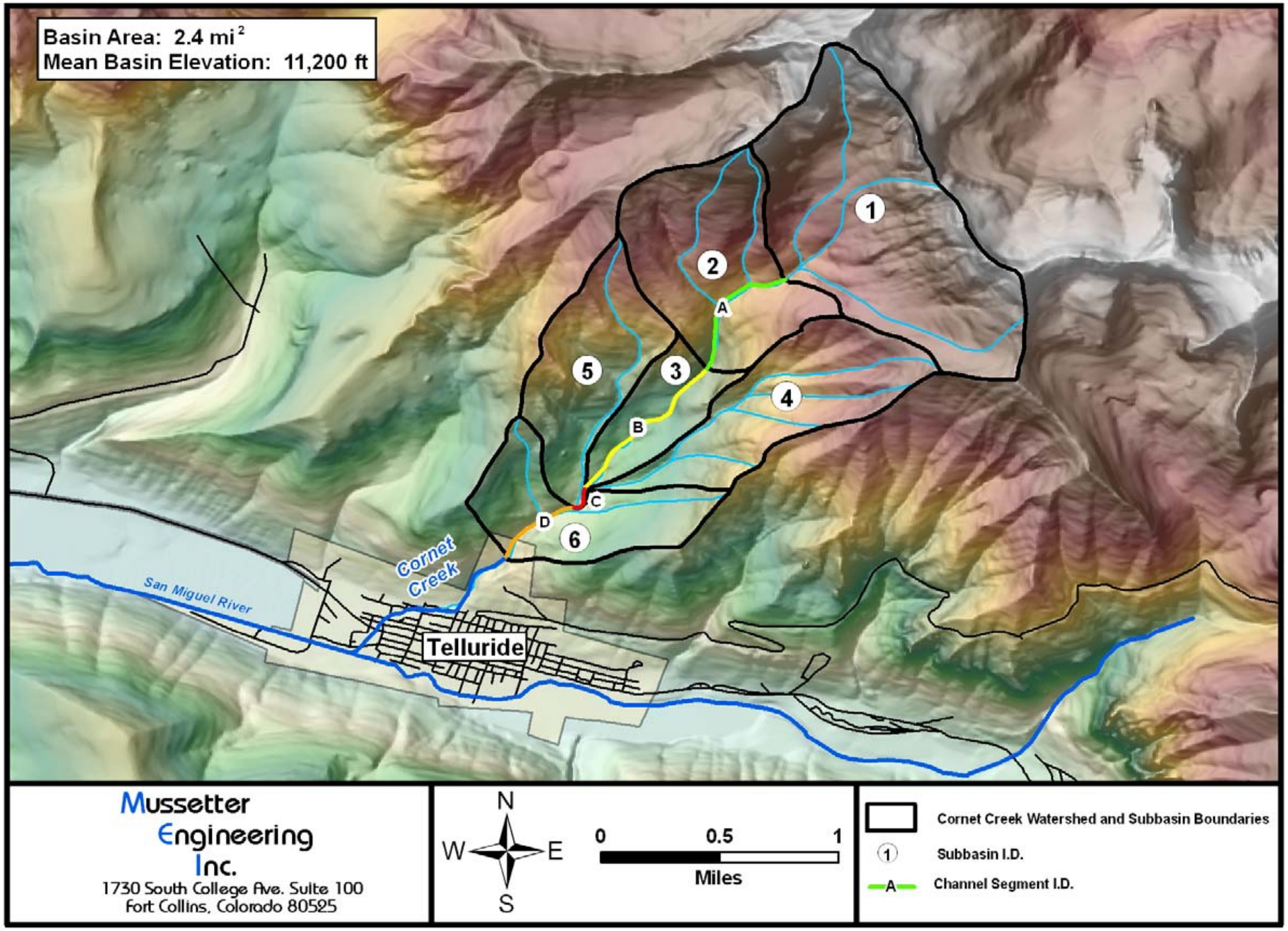


Figure 3.1. Map showing subbasin boundaries and channel alignments used in the HEC-HMS model.

a 24-hour storm duration with 5-minute computation intervals, and a peak intensity position of 25 percent (i.e., 6 hours into the storm). The total precipitation-frequency depths (i.e., input hyetographs) for each storm were developed using the Frequency Storm Method based on rainfall intensity-duration data obtained from isopluvial maps in the National Oceanic and Atmospheric Administration Atlas 2 (NOAA, 1973) (**Table 3.1**). Peak flow events on Cornet Creek are typically rainfall driven. Therefore, the precipitation depths were based on the NOAA partial-duration series precipitation-frequency atlas for the period from May to October, which primarily represents rainfall. Based on this source, the total precipitation depths range from 0.21 inches at 15 minutes into the 2-year storm to 1.45 inches at 24 hours into the 2-year storm, and from 0.51 inches to 2.9 inches at 15 minutes and 24 hours into the 100-year storm, respectively (Table 3.1).

Table 3.1. Total precipitation-frequency estimates for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval (RI) storms for the Cornet Creek drainage basin.

Recurrence Interval (yrs)	Precipitation Frequency (in.)							
	5 min	15 min	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr
2	0.21	0.41	0.72	0.84	0.92	1.06	1.26	1.45
5	0.27	0.53	0.93	1.10	1.21	1.40	1.61	1.80
10	0.32	0.62	1.09	1.27	1.39	1.60	1.86	2.10
25	0.39	0.76	1.33	1.50	1.61	1.80	2.08	2.40
50	0.44	0.86	1.50	1.68	1.80	2.00	2.34	2.70
100	0.51	1.00	1.75	1.89	2.00	2.20	2.56	2.90

The infiltration (movement of water into the soil) and interception (surface storage in topographic depressions and vegetation) of the precipitation were simulated using Soil Conservation Service (now the Natural Resources Conservation Service [NRCS], U.S. Department of Agriculture) curve numbers, an empirical parameter that describes the drainage characteristics of soil based on typical soil cover, land use, and antecedent moisture conditions. Curve numbers were estimated based on the hydrologic soil group and cover type determined from the NRCS soil survey (NRCS, 2007) with guidance from the NRCS National Engineering Handbook (NEH; NRCS, 2004a and 2004b). Percent of impervious cover primarily represented the amount of rock outcrop in the basin and was also determined from the NRCS soil survey. The initial abstraction (amount of precipitation that must fall before there is surface excess) was computed as a function of the curve number as specified in the NRCS NEH. A summary of the watershed subbasin parameters are provided in **Table 3.2**.

The SCS dimensionless unit hydrograph method was used to transform the excess rainfall (precipitation remaining after infiltration and interception) to subbasin flow. The lag-times used for each subwatershed (i.e., time between the center of mass of rainfall excess and the peak of the unit hydrograph) were predicted by procedures specified in the NEH (NRCS, 1985), and are also summarized in Table 3.2. The individual subbasin hydrographs were routed through the connections and main channels using the kinematic wave method, which is the most appropriate method for steep channels such as Cornet Creek where flow momentum dominates, and channel storage and hydrograph attenuation are minor. A summary of the required input parameters for the channel routing is provided in **Table 3.3**. The most recent previous hydrology study conducted by REI (1996) was based on a minimum time of concentration of 15 minutes because shorter values appeared to overestimate the peak flows entering Telluride. The time of concentrations associated with the lag times reported in Table 3.2 range from 17 minutes in Subbasin 1 to about 46 minutes in Subbasin 4.

Table 3.2. SCS curve numbers and other subbasin parameters.

Subbasin	Subbasin Area (mi^2)	Subbasin Slope (ft/ft)	SCS Curve No.	Impervious %	Initial Abstraction (in.)	Length of Longest Watercourse (ft)	Lag time (min)	Time of Concentration (min)
1	0.700	0.4429	83	22.0	0.399	5,714	10.4	17.26
2	0.397	0.4738	76	8.7	0.639	5,734	12.7	21.19
3	0.413	0.1690	64	0.0	1.125	3,610	20.2	33.74
4	0.332	0.3428	64	0.0	1.125	7,497	25.5	45.51
5	0.281	0.3850	70	0.0	0.841	5,714	16.4	27.26
6	0.290	0.3192	65	0.0	1.077	5,890	21.2	35.39

Table 3.3. Input channel parameters for kinematic wave channel routing in the upstream watershed.

Channel Segment	Channel Length (ft)	Channel Slope (ft/ft)	Channel Roughness (Manning's, *n*)	Bottom Width (ft)	Sideslopes (_H:1V)
A	2,720	0.3063	0.1	10	3
B	3,600	0.1633	0.1	15	3
C	490	0.1399	0.1	15	3
D	3,170	0.2145	0.1	15	2

3.2. Hydrologic Model Results

Peak discharges at the outlet of Subbasin 5 (i.e., entering the Town of Telluride) predicted by the model range from about 290 cubic feet per second (cfs) (30 acre-feet (ac-ft)) during the 2-year storm to about 1,490 cfs (121 ac-ft) during the 100-year storm (**Table 3.4 and Figure 3.2**). Comparison with previous studies shows that results from the HEC-HMS model are slightly lower than estimates based on the Rational Method (REI, 1996), but that the peak flows reported in the FEMA FIS underestimate the actual peak discharges that Cornet Creek is likely to experience (**Table 3.5**).

Table 3.4. Summary of computed peak discharges and flow volumes entering the Town of Telluride from the Cornet Creek watershed, based on the HEC-HMS model results.

Recurrence Interval (yrs)	Peak Discharge (cfs)	Storm Event Volume (ac-ft)
2	287	30
5	482	47
10	659	65
25	915	84
50	1,176	106
100	1,491	121

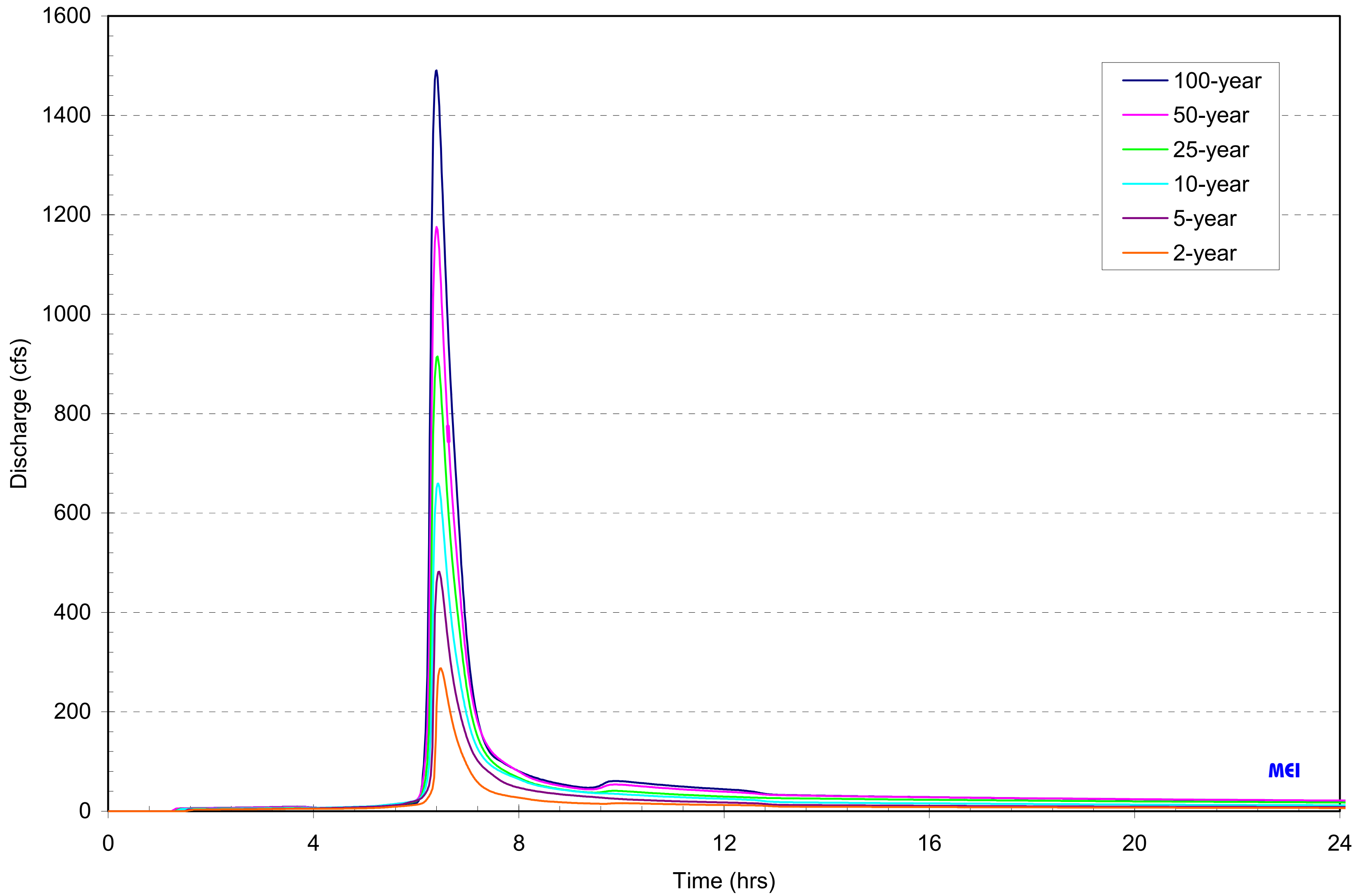


Figure 3.2. Estimated 2-, 5-, 10-, 25-, 50-, and 100-year outflow hydrographs for the Cornet Creek watershed.

Table 3.5. Comparison of HEC-HMS-derived and previous peak flow estimates for Cornet Creek.

Study	Study	Year	Method	Recurrence Interval (yrs)					
				2	5	10	25	50	100
				Peak Discharge (cfs)					
Cornet Creek Drainage Maintenance and Flood Mitigation Study	MEI	2007	HEC-HMS	287	482	659	915	1,176	1,491
Town of Telluride Surface Water Hydrology Study	REI	1996	Rational Method	428	610	917	1,111	1290	1,460
Flood Insurance Study (FEMA FIS)	SLA	1992	SCS Curve Number	--	--	210	386	415	590

4. HYDRAULIC ANALYSIS

A hydraulic analysis of Cornet Creek between the confluence with the San Miguel River and approximately 260 feet upstream from Dakota Avenue was performed for the range of flows up to, and including the 100-year peak discharge using the USACE HEC-RAS v.3.1 computer software (USACE, 2005). The primary purpose of the analysis was to evaluate existing channel capacity and to estimate hydraulic conditions (e.g., velocity, depth, shear stress) in the creek to facilitate incipient motion and bed-material transport capacity calculations throughout the project reach.

4.1. Model Development

The HEC-RAS model consists of 56 cross sections that extend from the San Miguel River approximately 80 feet upstream from the South Tomboy Street Bridge to about 260 feet upstream from Dakota Avenue (**Figure 4.1**). The cross sections were primarily based on topographic and bathymetric data that were collected by FAI specifically for this project. Overbank portions of cross sections that required extension beyond the surveyed top-of-bank elevations were derived from the 2-foot contour mapping developed in 2003.

Except for one footbridge, all bridge and culvert crossings were incorporated into the model based on information from available design plans, survey data collected by FAI, and field measurements taken both by MEI and the Town of Telluride Public Works Department. The single bridge excluded from the hydraulic model is an extremely high footbridge located immediately downstream of Dakota Avenue, which does not contain any abutments that would impact flow, and is located well above the 100-year peak discharge.

The downstream boundary conditions for the model were based on normal depth calculations at the average bed slope. The downstream cross section in the hydraulic model is located within the San Miguel River, and the Cornet Creek channel bed profile is relatively steep as it extends upstream from the river. As a result, any minor errors in the downstream boundary condition will not propagate very far upstream.

Because the creek is relatively steep, the model indicates that flow could approach supercritical conditions in areas throughout the study reach under rigid-boundary conditions. However, given that numerous obstructions located within the channel such as culvert crossings, vegetation, and even large boulders will inhibit the creek's ability to maintain supercritical flow over any distance, the model was run based on a subcritical flow regime while allowing critical depth to be assumed when appropriate. As a result, critical depth conditions are common throughout the project reach.

As described in Section 2, field observations indicate that the bed-material coarsens in the upstream direction, the channel gradient becomes steeper, and small drops associated with the existence of larger boulders in upstream portions of the reach cause additional energy losses in the channel. To account for these changes, Manning's *n* roughness coefficients for unvegetated portions of the main channel ranged from 0.036 near the San Miguel River Trail footbridge to 0.07 in extremely steep sections of the creek upstream from Dakota Avenue. *N*-values used for the overbanks ranged from 0.04 in areas representing clean, hardened surfaces such as in the vicinity of road crossings to 0.10 in areas containing dense vegetation. Using these values, the model predicts water-surface elevations that are in reasonable agreement with those measured during the July 2007 site survey, when the discharges ranged between 5 and 10 cfs (**Figure 4.2**). Specific data were not available to validate the model at

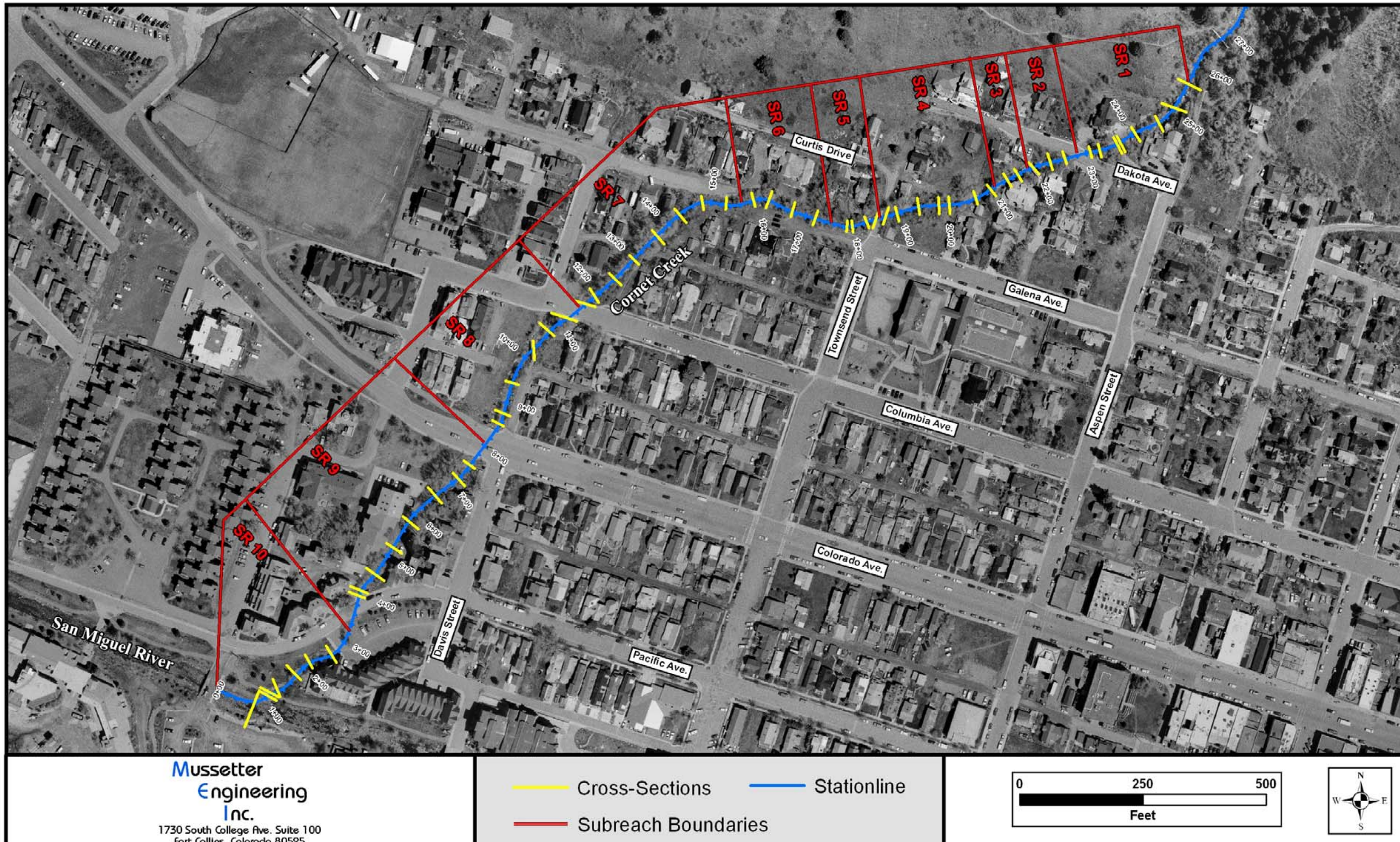


Figure 4.1. Aerial photograph showing extent of detailed study reach, including the locations of cross sections used to develop the hydraulic model and subreach boundaries.

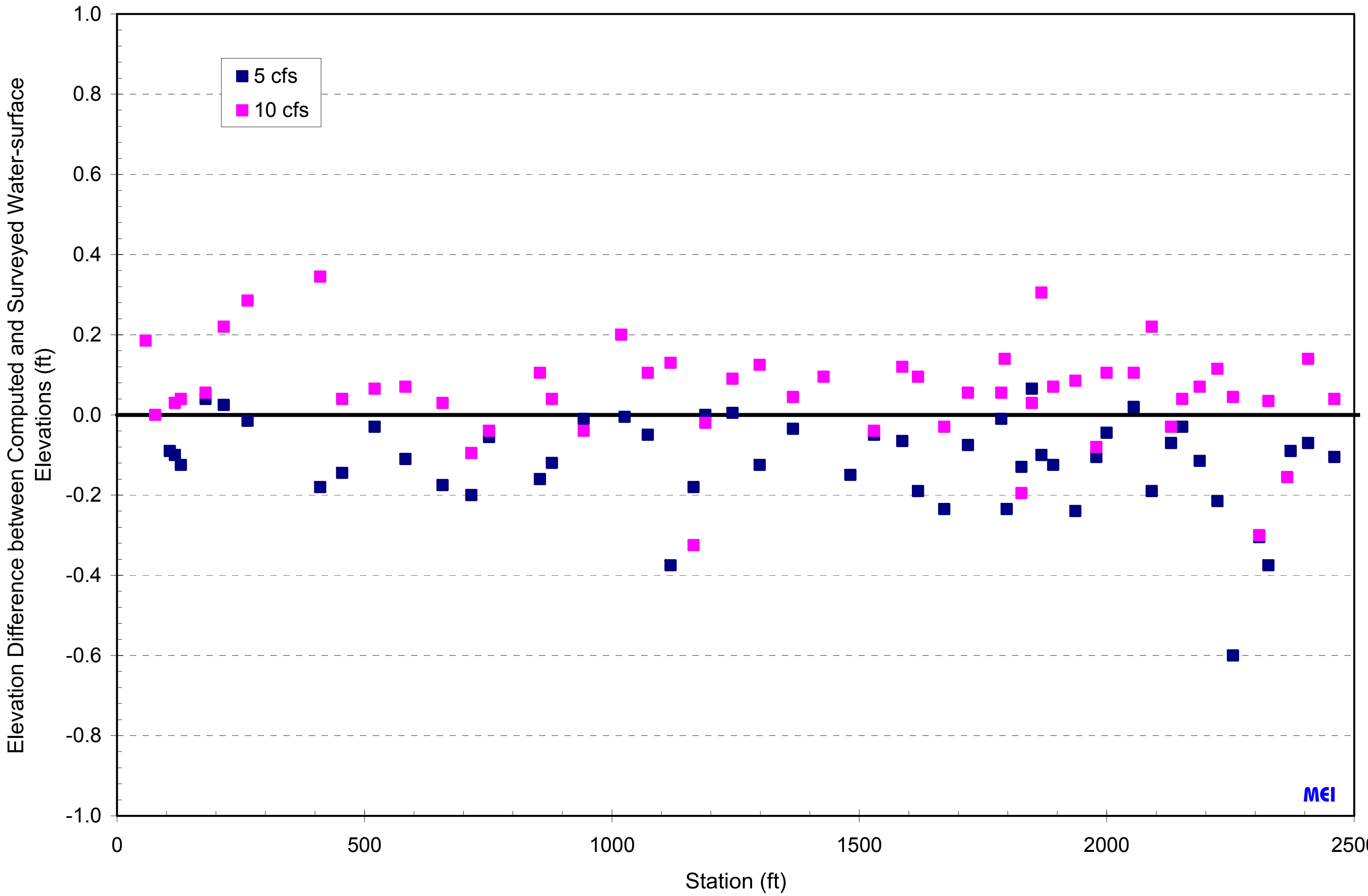


Figure 4.2. Differences between computed water-surface elevations at 5 and 10 cfs and surveyed water-surface elevations measured within the same estimated range of flows during the July 2007 site survey.

higher flows, the range of *n*-values is consistent with published values for similar types of streams (Barnes, 1967; Hicks and Mason, 1991; Arcement and Schneider, 1989), and the model is therefore, believed to be reliable over the full range of flows that are being considered in this study.

As will be discussed in more detail in the following section, flood flows significantly overtop the existing channel banks and spill onto adjacent overbank areas. To estimate the amount of potential flow that would break out of the channel, lateral weir structures that represent the top-of-bank profile were added to the model in critical areas throughout the project reach, thereby, allowing realistic adjustments to be made to the in-channel discharges. The 2003 2-foot contour mapping was used to determine the locations that flow would re-enter the channel, or whether it would continue to drain away from the creek.

4.2. Existing Conditions Hydraulic Model Results

The HEC-HMS hydraulic model was used to predict water-surface elevations and hydraulic conditions for a range of flows up to and including the 100-year peak discharge of 1,490 cfs. To facilitate the sediment transport analysis, the study reach was subdivided into a series of 10 subreaches, based on similarity of geomorphic and hydraulic conditions (Figure 4.1 and **Figure 4.3**).

Based on computed water-surface elevations, the existing (June 2007) capacity of the channel (i.e., elevation at which the flows would begin to break out of the channel and impact adjacent property or infrastructure) is lower than the 2-year peak discharge in many locations throughout the project reach. Reach-averaged hydraulics and channel capacities were determined for each subreach as discussed below:

Subreach 1 (Upper end of study reach to Dakota Avenue)

The upstream subreach (Subreach 1) is bounded by a relatively high bank along the left side of the channel, which except for directly upstream from Dakota Ave., fully contains the flow up to and including the 100-year peak discharge (**Figures 4.4 and 4.5**). The right bank is not as high, but has a capacity of about the 5-year event. Flows greater than the 5-year peak flow in this location will not necessarily flow away from the channel, but they will likely impact a large house that is situated directly along the right overbank. The existing capacity of the culvert crossing at Dakota Avenue is approximately 240 cfs (Figure 4.4). The majority of overtopping flows on the left side of the crossing are likely to re-enter the channel on the downstream side of the road, but any overtopping flow on the right side will probably travel west along Dakota Avenue away from the channel (Figure 4.5).

Based on model results, approximately 40 cfs leaves the channel at the 10-year event, and about 280 cfs spills out of the channel during the 100-year peak flow. **Figure 4.6** shows computed water-surface elevations based on actual in-channel discharges remaining in the creek after overtopping losses. Reach-averaged velocities in Subreach 1 for remaining in-channel discharges range from 8.2 feet per second (fps) at the peak of the 2-year event to about 11.8 fps at the 100-year peak (**Table 4.1**). Hydraulic depths range from about 2.0 to 4.3 feet and effective widths range from about 17.6 to 29.5 feet at the 2- and 100-year events, respectively.

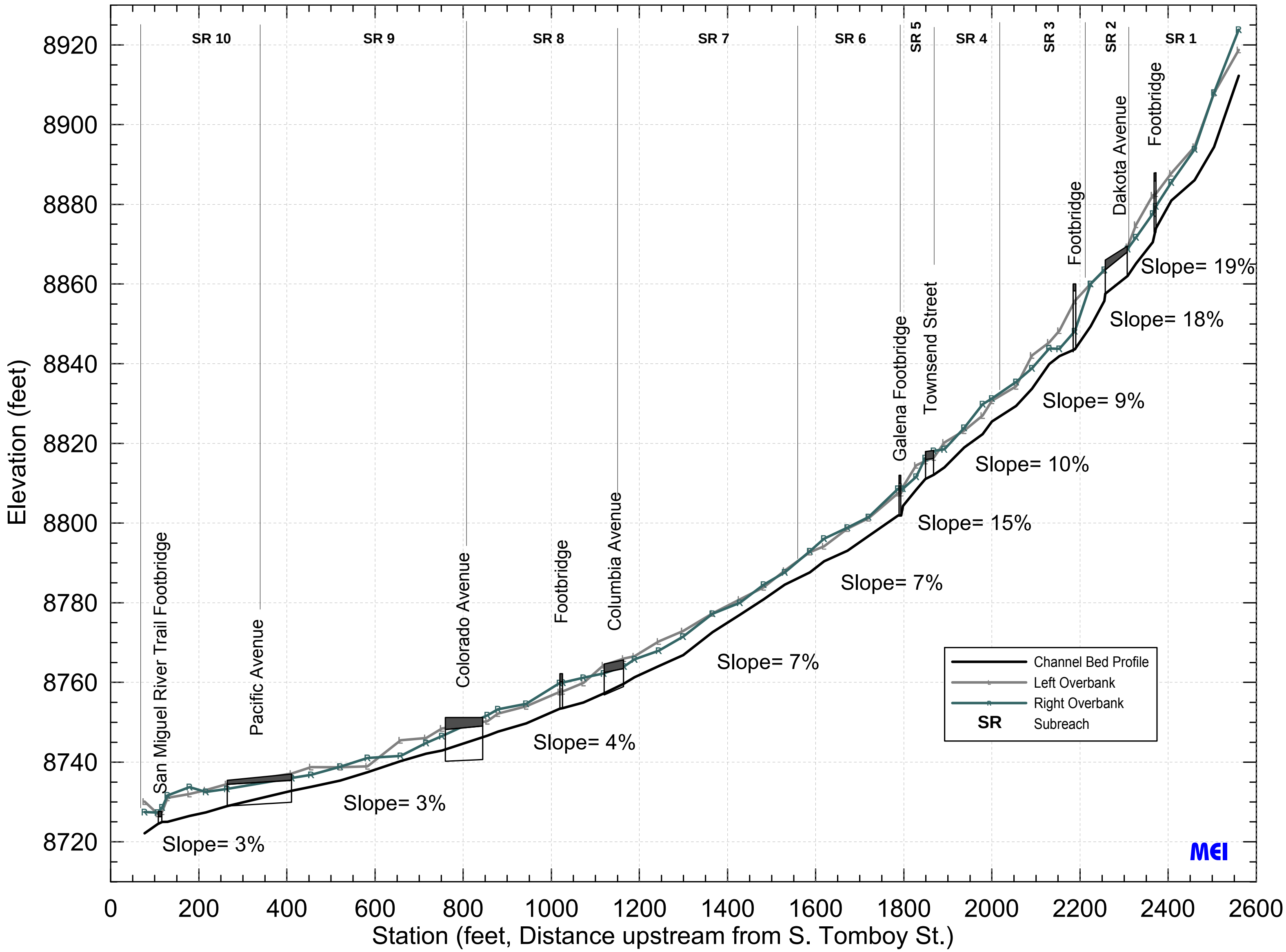


Figure 4.3. Existing channel bed profile (June 2007) of the study reach of Cornet Creek, showing the locations of hydraulic and sediment-transport subreach boundaries.

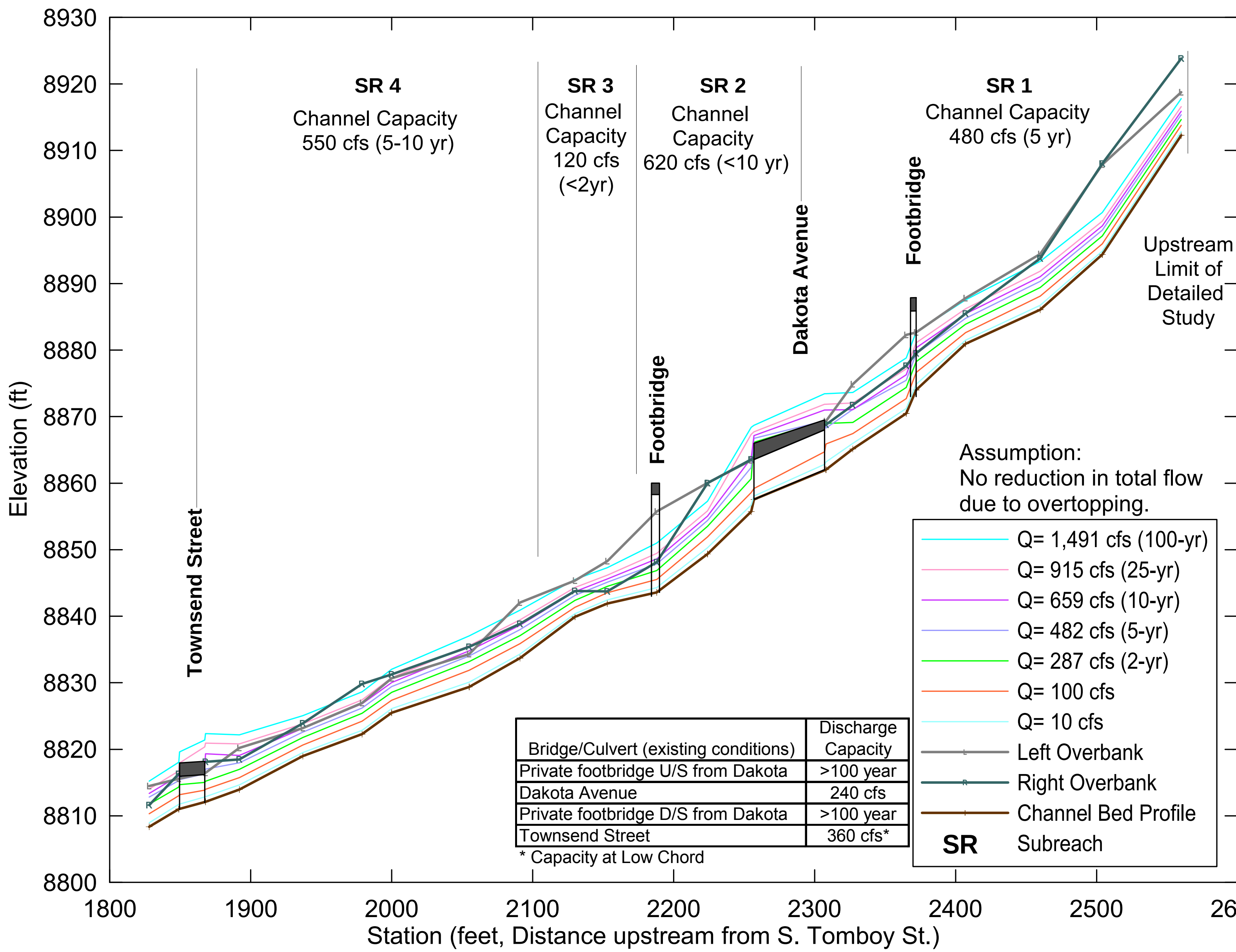


Figure 4.4. Computed water-surface profiles assuming no overbank flow losses for Subreaches 1 through 4 of Cornet Creek for a range of flows up to the 100-year peak discharge.

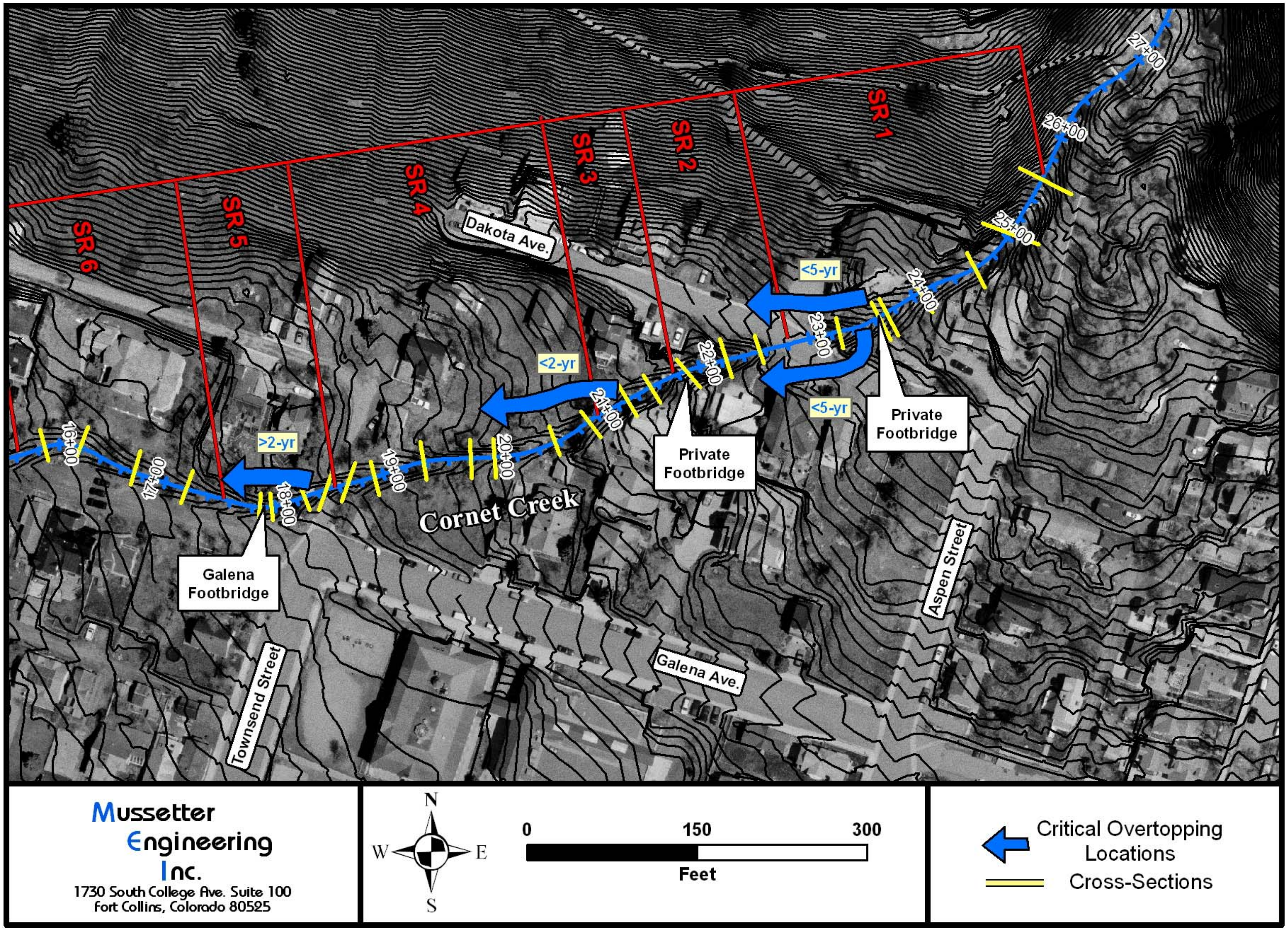
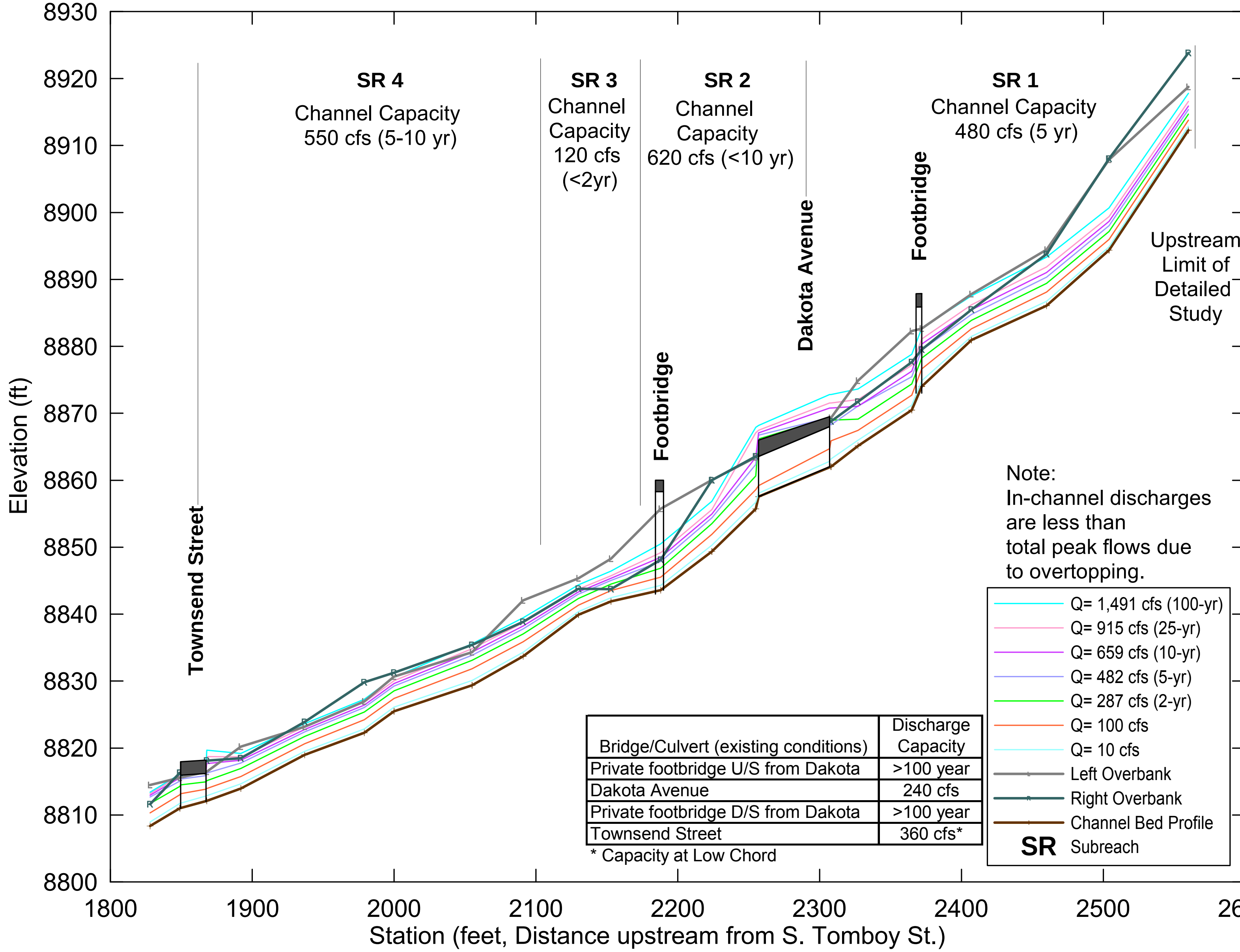


Figure 4.5. Map of Subreaches 1 through 4 of Cornet Creek showing critical channel overtopping locations and the recurrence intervals of the overtopping flows.



Bridge/Culvert (existing conditions)	Discharge Capacity
Private footbridge U/S from Dakota	>100 year
Dakota Avenue	240 cfs
Private footbridge D/S from Dakota	>100 year
Townsend Street	360 cfs*

* Capacity at Low Chord

Figure 4.6. Computed water-surface profiles for Subreaches 1 through 4 of Cornet Creek for a range of flows up to the 100-year peak discharge, based on remaining in-channel discharges after overbank flow losses.

Table 4.1. Summary of existing (June 2007) reach-averaged hydraulic conditions for Cornet Creek.

Subreach	Profile (yr)	Total Discharge (cfs)	Main Channel Discharge (cfs)	Main Channel Velocity (ft/s)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)	Approximate Location
1	2	287	287	8.17	2.00	17.6	0.0555	Upper end of study reach to Dakota Ave.
	5	482	482	8.84	2.59	21.1	0.0504	
	10	659	659	9.81	2.90	23.2	0.0557	
	25	915	914	10.53	3.36	25.9	0.0549	
	50	1176	1171	11.10	3.81	27.8	0.0528	
	100	1491	1477	11.79	4.27	29.5	0.0519	
2	2	287	287	8.29	2.09	16.6	0.0430	Dakota Ave. to 100 feet downstream (steep section below culvert outlet).
	5	482	482	9.37	2.65	19.4	0.0434	
	10	627	627	9.90	3.06	20.7	0.0413	
	25	836	835	10.73	3.54	22.0	0.0419	
	50	1046	1043	11.41	3.97	23.1	0.0412	
	100	1297	1292	12.05	4.44	24.2	0.0408	
3	2	279	265	7.19	1.79	21.3	0.0300	100 feet to 150 feet downstream from Dakota Ave (short depositional area).
	5	445	411	8.21	2.27	23.4	0.0296	
	10	553	504	8.68	2.54	24.5	0.0296	
	25	696	628	9.25	2.84	25.8	0.0295	
	50	831	745	9.73	3.10	26.4	0.0290	
	100	987	878	10.26	3.40	27.0	0.0296	
4	2	277	277	7.81	1.89	18.7	0.0673	150 feet downstream from Dakota Ave. to Townsend Street.
	5	436	436	8.69	2.31	21.7	0.0684	
	10	538	538	9.10	2.53	23.3	0.0683	
	25	665	664	9.50	2.81	25.0	0.0660	
	50	771	769	9.90	3.03	25.7	0.0652	
	100	869	864	10.24	3.24	26.2	0.0646	
5	2	277	277	8.22	2.11	16.0	0.0615	Townsend Street to Galena Footbridge
	5	430	413	8.57	2.68	18.5	0.0513	
	10	513	485	9.00	2.87	19.6	0.0529	
	25	584	546	9.32	3.03	20.3	0.0536	
	50	621	577	9.43	3.12	20.6	0.0528	
	100	646	598	9.53	3.18	20.8	0.0525	
6	2	277	277	7.60	1.99	18.3	0.0582	Galena Footbridge to Galena Ave.
	5	436	436	8.22	2.46	21.6	0.0570	
	10	527	526	8.56	2.71	22.7	0.0563	
	25	615	612	8.93	2.92	23.6	0.0568	
	50	681	676	9.16	3.08	24.2	0.0560	
	100	734	726	9.33	3.19	24.6	0.0556	
7	2	277	277	7.33	1.91	19.8	0.0619	Galena Ave. to Columbia Ave.
	5	387	383	7.98	2.28	21.2	0.0607	
	10	418	412	8.18	2.38	21.4	0.0603	
	25	435	428	8.24	2.45	21.5	0.0590	
	50	449	441	8.13	2.52	21.8	0.0558	
	100	460	451	8.11	2.56	22.1	0.0544	
8	2	277	277	6.65	2.02	20.6	0.0431	Columbia Ave. to Colorado Ave.
	5	397	397	7.04	2.42	23.3	0.0414	
	10	447	447	7.12	2.59	24.3	0.0394	
	25	505	504	7.28	2.76	25.2	0.0389	
	50	558	557	7.47	2.90	25.8	0.0390	
	100	602	600	7.64	3.01	26.1	0.0394	
9	2	239	228	6.59	1.53	23.6	0.0274	Colorado Ave. to Pacific Ave.
	5	285	270	6.96	1.68	24.2	0.0272	
	10	309	291	7.10	1.77	24.5	0.0268	
	25	325	306	7.19	1.82	24.6	0.0265	
	50	343	322	7.28	1.88	24.9	0.0260	
	100	350	328	7.31	1.90	24.9	0.0257	
10	2	191	185	4.54	2.19	19.2	0.0230	Pacific Ave. to San Miguel River
	5	205	199	4.65	2.25	19.6	0.0233	
	10	215	208	4.72	2.29	19.9	0.0238	
	25	220	212	4.74	2.31	20.0	0.0237	
	50	224	217	4.78	2.33	20.1	0.0239	
	100	230	222	4.81	2.35	20.2	0.0238	

Subreach 2 (Dakota Avenue to 100 feet downstream from Road Crossing)

Subreach 2 represents the steep channel section that extends about 100 feet downstream from Dakota Avenue. The left bank is also high in this subreach and prevents any flow from escaping. The elevation of the right bank decreases significantly about 70 feet downstream from Dakota Avenue, containing flows up to almost the 10-year event (Figures 4.4 and 4.5). Any flows greater than about 620 cfs will probably impact the adjacent house located on the right bank, but will not drain away from the channel until a little farther downstream.

Reach-averaged velocities in this subreach range from 8.3 fps at the 2-year peak to about 12.1 fps at the 100-year peak flow (Table 4.1). Hydraulic depths are similar to Subreach 1 ranging from about 2.1 to 4.4 feet at the 2- and 100-year events, respectively. This subreach is slightly more confined at higher flows with effective widths that range from about 16.6 feet (2-year) to only about 24.2 feet (100-year).

Subreach 3 (100 feet to 150 feet downstream from Dakota Avenue)

Subreach 3 is a short, wider section with a much flatter channel gradient. As a result, this area is characterized by less intense hydraulics, which creates a natural deposition zone. Discharges up to the 100-year event are still contained by the relatively high left bank, but the right bank continues to drop in this subreach, and contains flows only up to about 120 cfs, which is much less than the 2-year peak discharge (Figure 4.4). Any overtopping flows in this area will probably drain west along the road towards Curtis Drive, and will not immediately re-enter the channel (Figure 4.5).

Model results indicate that as much as 5 cfs spills out of the channel in this reach at the 2-year event, 30 cfs leaves the channel at the 10-year event, and 120 cfs spills during the 100-year peak flow. Reach-averaged velocities in Subreach 3 based on remaining in-channel discharges are lower than the upstream reaches, and range from 7.2 fps at the peak of the 2-year event to about 10.3 fps at the 100-year peak (Table 4.1). The slightly larger effective widths (1.8 to 3.4 feet at the 2- and 100-year events, respectively) cause a slight decrease in the hydraulic depths, which range from about 1.8 feet during the 2-year to 3.4 feet during the 100-year peak flow.

Subreach 4 (150 feet downstream from Dakota Avenue to Townsend Street)

The left and right top-of-bank elevations are more uniform and similar to one another in Subreach 4, and they contain flows up to about 550 cfs, which is between a 5- and 10-year recurrence interval (RI) event (Figure 4.4). The slightly higher channel capacity in this reach limits the additional flow loss to about 10 cfs during the 10-year event. However, because the bank heights are more uniform (i.e., extremely high bank heights do not occur), an additional 230 cfs is estimated to spill during the 100-year event. Based on the 2003 contours, any spilled flow is unlikely to return to the channel in this subreach (Figure 4.5).

Figure 4.6 shows computed water-surface elevations based on actual in-channel discharges remaining in the creek after overtopping losses. Reach-averaged velocities for the remaining in-channel flows range from 7.8 fps to 10.2 fps at the 2- and 100-year events, respectively (Table 4.1). At the same corresponding flows, hydraulic depths range from 1.9 to 3.2 feet, and effective widths range from 18.7 to 26.2 feet.

Townsend Street Bridge, which was replaced in November 2005, is located at the downstream end of this subreach, and has a lower existing conditions capacity. Flow reaches the low chord of the bridge at a discharge of about 360 cfs and begins flowing over the left bank at approximately 440 cfs (i.e., slightly less than the 5-year peak). The original configuration of this crossing consisted of two 5-foot diameter corrugated metal pipes (cmp) with a bridge deck comprised of a combination of wood cribbing, rocks, and fill. Based on hydraulic results, the original culverts had a capacity of about 150 cfs (i.e., much less than the 2-year peak flow) before flow began to spill over the left bank. The recent bridge improvements at Townsend Street increased the capacity of the crossing by almost 300 cfs, and reduced the frequency of overtopping by about 50 percent.

Subreach 5 (Townsend Street to Galena Footbridge)

Subreach 5 is a short, steep section immediately downstream from the Townsend Street Bridge. The left overbank is slightly higher in this reach, but discharges greater than about 300 cfs (approximately the 2-year peak) can break out along the right bank (**Figure 4.7**). All flows that escape into the right overbank in this reach are likely to re-enter the channel downstream from the Galena Footbridge (**Figure 4.8**). In general, the reach-averaged hydraulics based on remaining in-channel flows are similar to those in Subreach 4 (Table 4.1).

The Galena Footbridge located at the downstream end of this reach has a higher capacity than the channel. Flow begins to spill over the banks at approximately 520 cfs before water-surface elevations reach the low chord of the bridge (Figure 4.7).

Subreach 6 (Galena Footbridge to Galena Avenue)

Subreach 6 is an approximately 230-foot long section of channel with relatively uniform channel and overbank profiles (Figure 4.7). It extends downstream from the Galena Footbridge to the abrupt bend in the channel where Galena Avenue is truncated by the creek. The existing channel capacity is about 460 cfs, which is slightly less than the 5-year recurrence interval peak flow. Based on the 2003 contour mapping, flows overtopping the right bank in the upper portion of the reach will probably remain adjacent to and re-enter the channel where it splits Galena Avenue (Figure 4.8). Flows escaping along the left bank will potentially re-enter the channel near the upstream side of the Columbia Avenue Bridge.

Overtopping flows from the right bank of Subreach 5 are likely to return to the channel in Subreach 6 downstream from the Galena Footbridge. Farther downstream, however, about 23 cfs and 140 cfs will spill out of the channel during the 10- and 100-year peak flows, respectively. **Figure 4.9** shows computed water-surface elevations in Subreaches 5 through 7 based on actual in-channel discharges remaining in the creek after overtopping losses. A more significant trend of decreasing reach-averaged velocities begins in Subreach 6, with values of 7.6 fps at the peak of the 2-year event and 9.3 fps at the 100-year peak (Table 4.1). Hydraulic depths do not change much and range from about 2.0 feet to 3.2 feet, and effective widths increase slightly to between 18.3 feet and 24.6 feet at the 2- and 100-year events, respectively.

Subreach 7 (Galena Avenue to Columbia Avenue)

The channel characteristics of Subreach 7 are very similar to those in Subreach 6. Both have similar average channel gradients of about 7 percent (Figure 4.3), and relatively uniform bed and bank profiles. The left bank of the channel tends to be a little higher in the downstream portion of the reach, but the channel capacity is still only about 270 cfs (i.e., slightly less than the 2-year peak; Figure 4.7). Any flow that overtops the right bank in this reach will probably drain in a south-westerly direction away from the channel, but the local topography indicates that flow spilled on the left side is likely to re-enter the channel near the upstream side of Columbia Avenue (Figure 4.8).

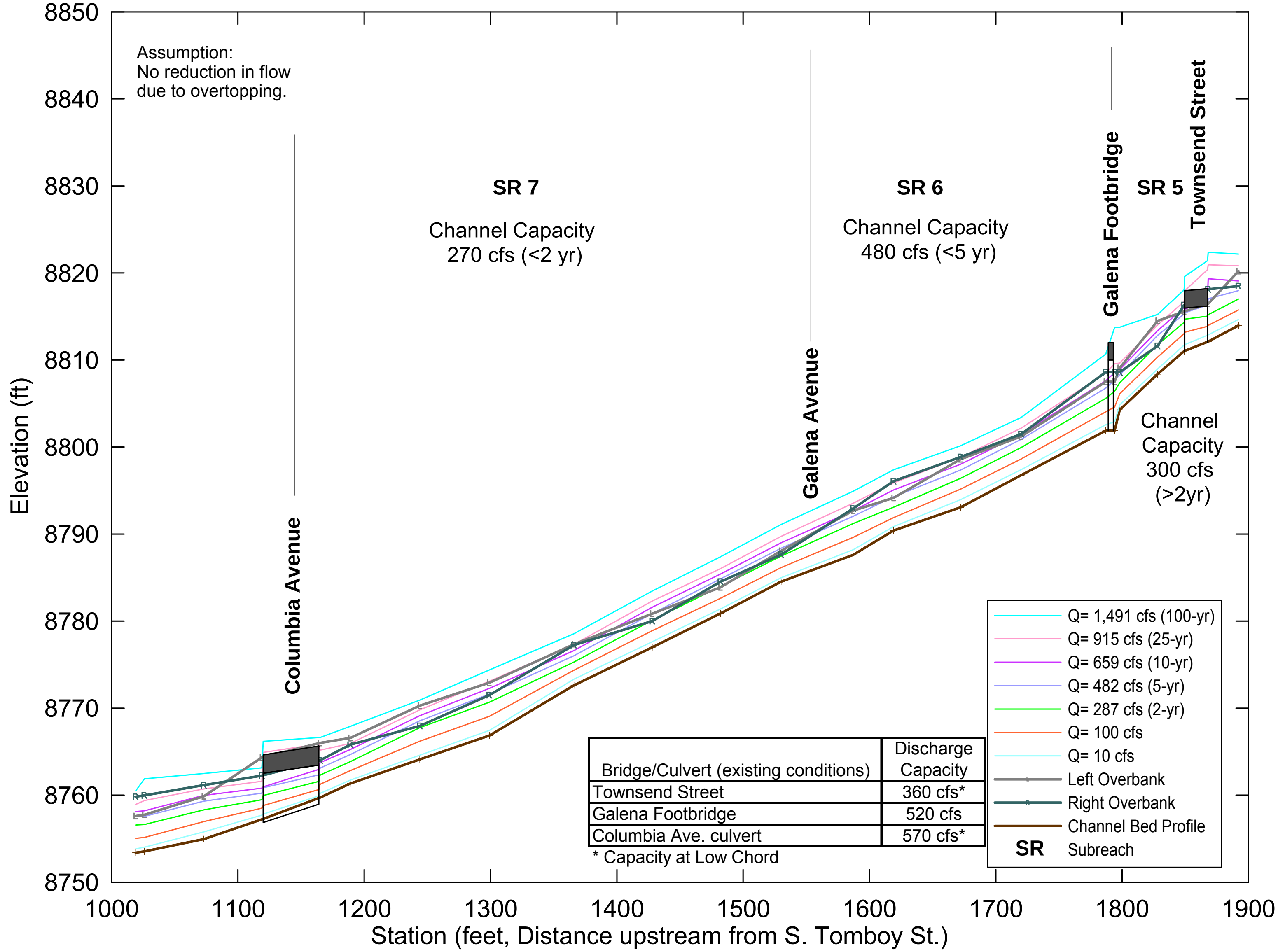


Figure 4.7. Computed water-surface profiles assuming no overbank flow losses for Subreaches 5 through 7 of Cornet Creek for a range of flows up to the 100-year peak discharge.

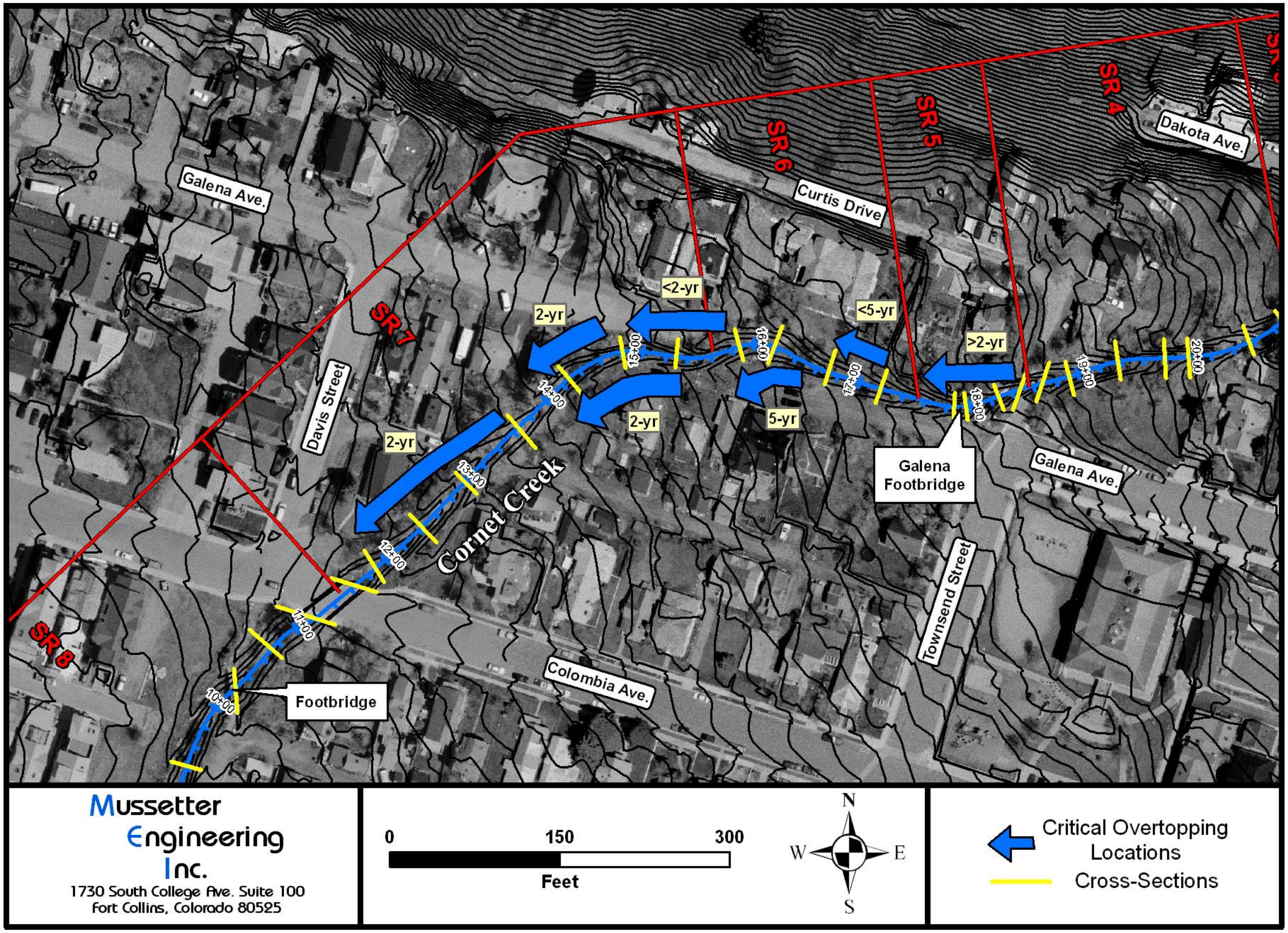
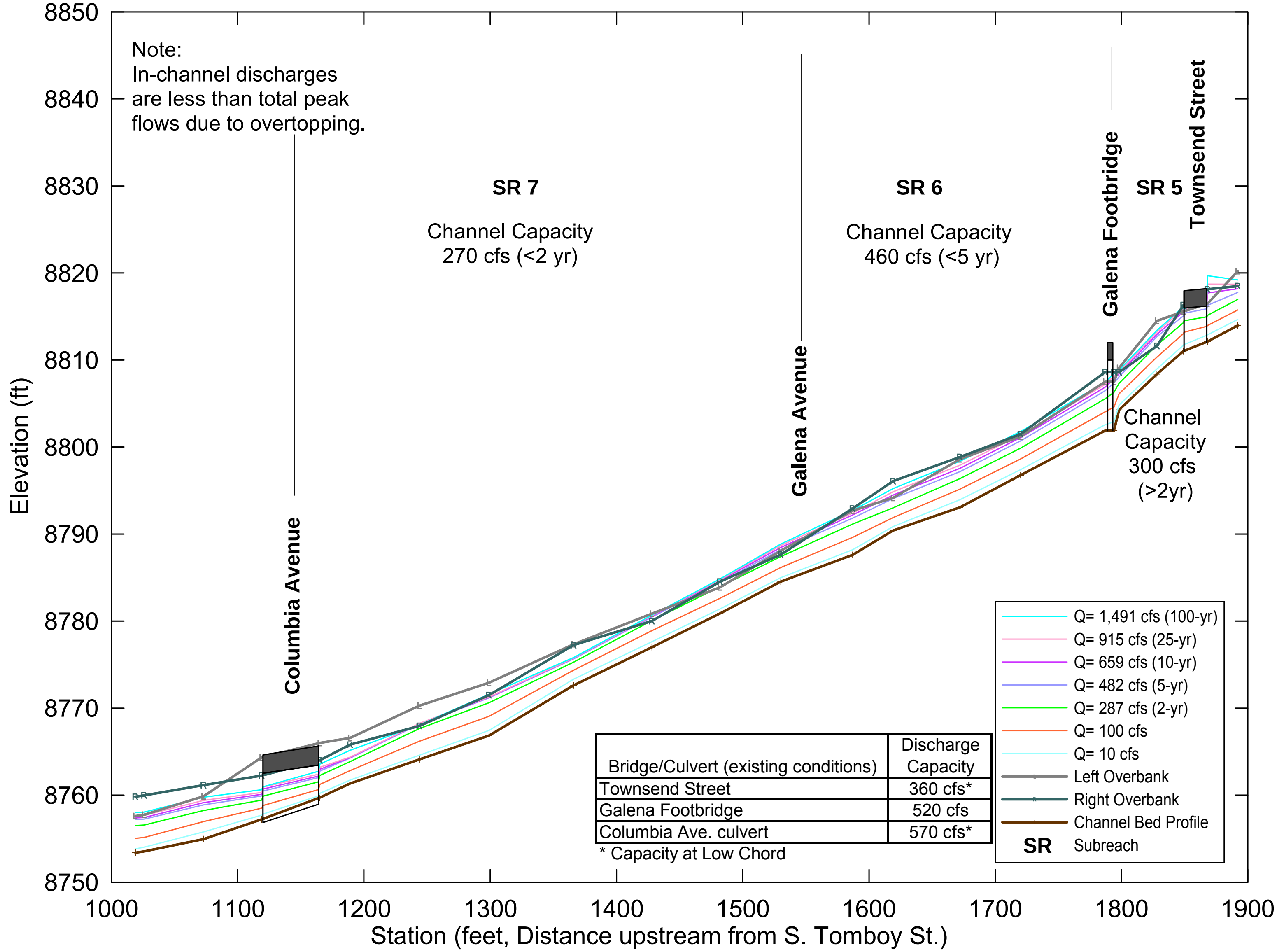


Figure 4.8. Map of Subreaches 5 through 7 of Cornet Creek showing critical channel overtopping locations and the recurrence intervals of the overtopping flows.



Bridge/Culvert (existing conditions)	Discharge Capacity
Townsend Street	360 cfs*
Galena Footbridge	520 cfs
Columbia Ave. culvert	570 cfs*

* Capacity at Low Chord

Figure 4.9. Computed water-surface profiles for Subreaches 5 through 7 of Cornet Creek for a range of flows up to the 100-year peak discharge, based on remaining in-channel discharges after overbank flow losses.

The estimated net discharge remaining at the upstream end of this segment of channel during the 100-year peak is approximately 620 cfs, and an additional 200 cfs is predicted to overtop the banks throughout the reach. An estimated 56 and 112 cfs are predicted to spill from the channel in Subreach 7 at the 5- and 10-year events, respectively. The reach-averaged channel velocities continue to decrease in this subreach to between 7.3 fps at the 2-year event and 8.1 fps at the 100-year event. Hydraulic depths decrease to between about 1.9 feet and 2.6 feet and effective widths also slightly decrease ranging from 19.8 feet and 22.1 feet at the 2- and 100-year peak discharges, respectively (Table 4.1).

Columbia Avenue Bridge is located at the downstream limit of Subreach 7 and has an existing capacity greater than that of the upstream channel. Flows are predicted to reach the low chord of the bridge at approximately 570 cfs and begin to overtop the road at about 760 cfs.

Subreach 8 (Columbia Avenue to Colorado Avenue)

The right bank in Subreach 8 is relatively high for about 100 feet downstream from Colorado Avenue, where a footbridge crosses the channel, and contains flows greater than the 25-year peak discharge (**Figure 4.10**). Downstream from the footbridge, the elevations of the right bank decrease but remain higher than those along the left side of the channel. The capacity in this reach is primarily limited by the left overbank, which will convey flows up to approximately 290 cfs before overtopping in areas immediately upstream from Colorado Avenue (Figure 4.10 and **Figure 4.11**). The majority of flows escaping from the channel in this subreach are likely to travel west along Colorado Avenue, especially those that spill over into the right overbank.

The footbridge located about 100 feet downstream from Columbia Avenue has an existing capacity of about 500 cfs, due to overtopping flows that occur along the left bank prior to any water-surface elevations reaching the low chord of the bridge. The concrete box culvert opening at Colorado Avenue is extremely reduced due to sedimentation and has an existing capacity of only about 210 cfs, which is less than the 2-year event and similar to this portion of the channel.

Results from the hydraulic model indicate that discharges lost due to overtopping range from about 10 cfs at the 10-year event to more than 50 cfs at the 100-year event. **Figure 4.12** shows computed water-surface elevations in Subreaches 8 through 10 based on actual in-channel discharges remaining in the creek after overtopping losses. Computed hydraulics based on the remaining in-channel flows show an approximate 1 fps reduction in reach-averaged velocities from Subreach 7 to between 6.7 fps at the 2-year peak and 7.6 fps at the 100-year peak (Table 4.1). Hydraulic depths are between 0.1 and 0.4 feet greater and effective widths are between about 0.8 and 4.1 feet greater than in Subreach 7 at the 2- and 100-year recurrence interval flows, respectively.

Subreach 9 (Colorado Avenue to Pacific Avenue)

Subreach 9 has a significantly flatter channel gradient (3 percent) than the upstream subreaches (Figure 4.3) and a much reduced channel capacity. The relatively low bank heights in this reach result in a channel capacity of only about 100 cfs (Figure 4.10). Except along the left bank immediately downstream from Colorado Avenue, flows can spill out of the channel throughout the entire subreach, impacting numerous adjacent properties. Left overbank flow will likely re-enter the channel on the downstream side of Pacific Avenue, but most of the flows within the right overbank will continue in a southwest direction away from the channel (Figure 4.11).

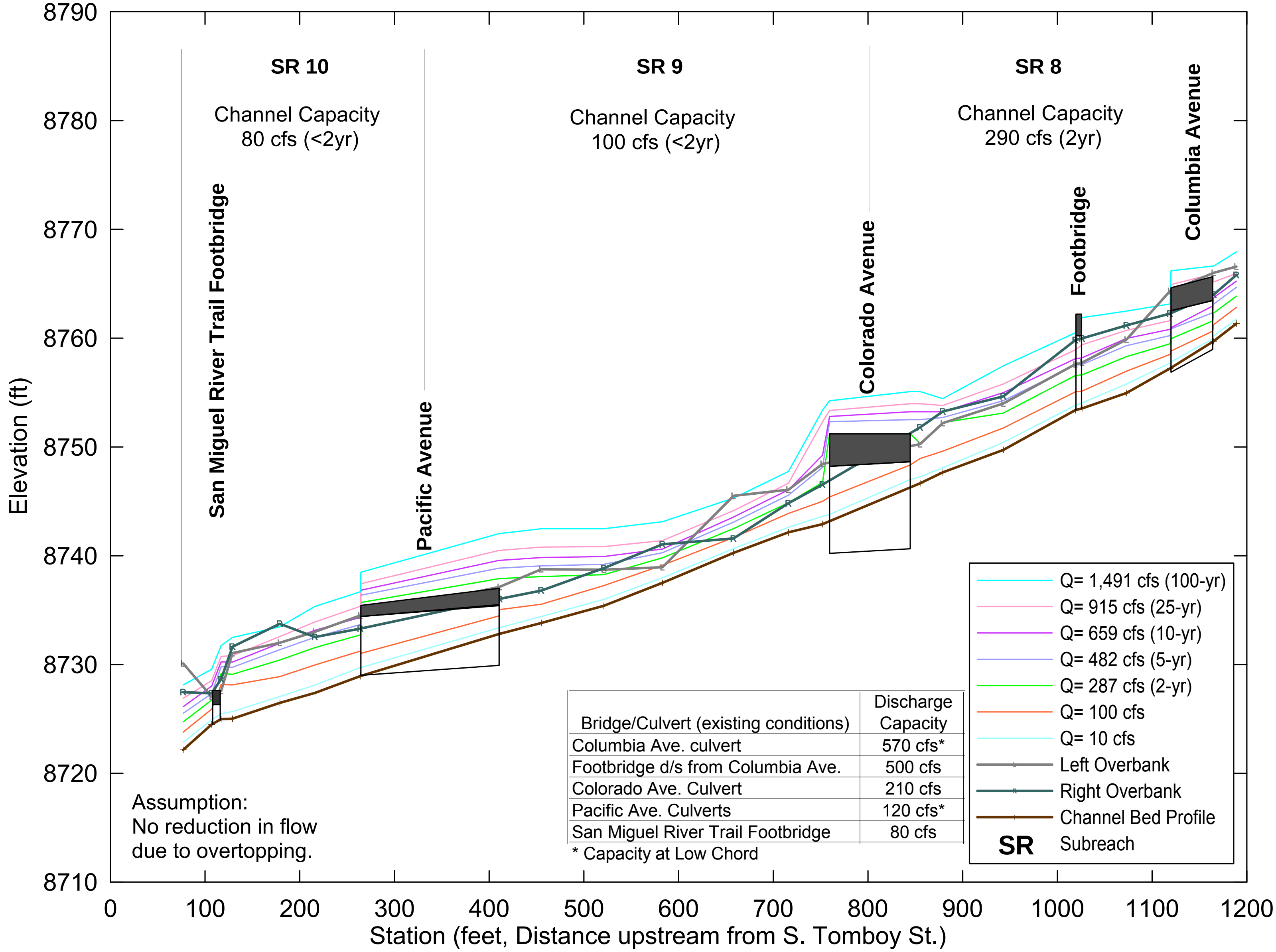


Figure 4.10. Computed water-surface profiles assuming no overbank flow losses for Subreaches 8 through 10 of Cornet Creek for a range of flows up to the 100-year peak discharge.

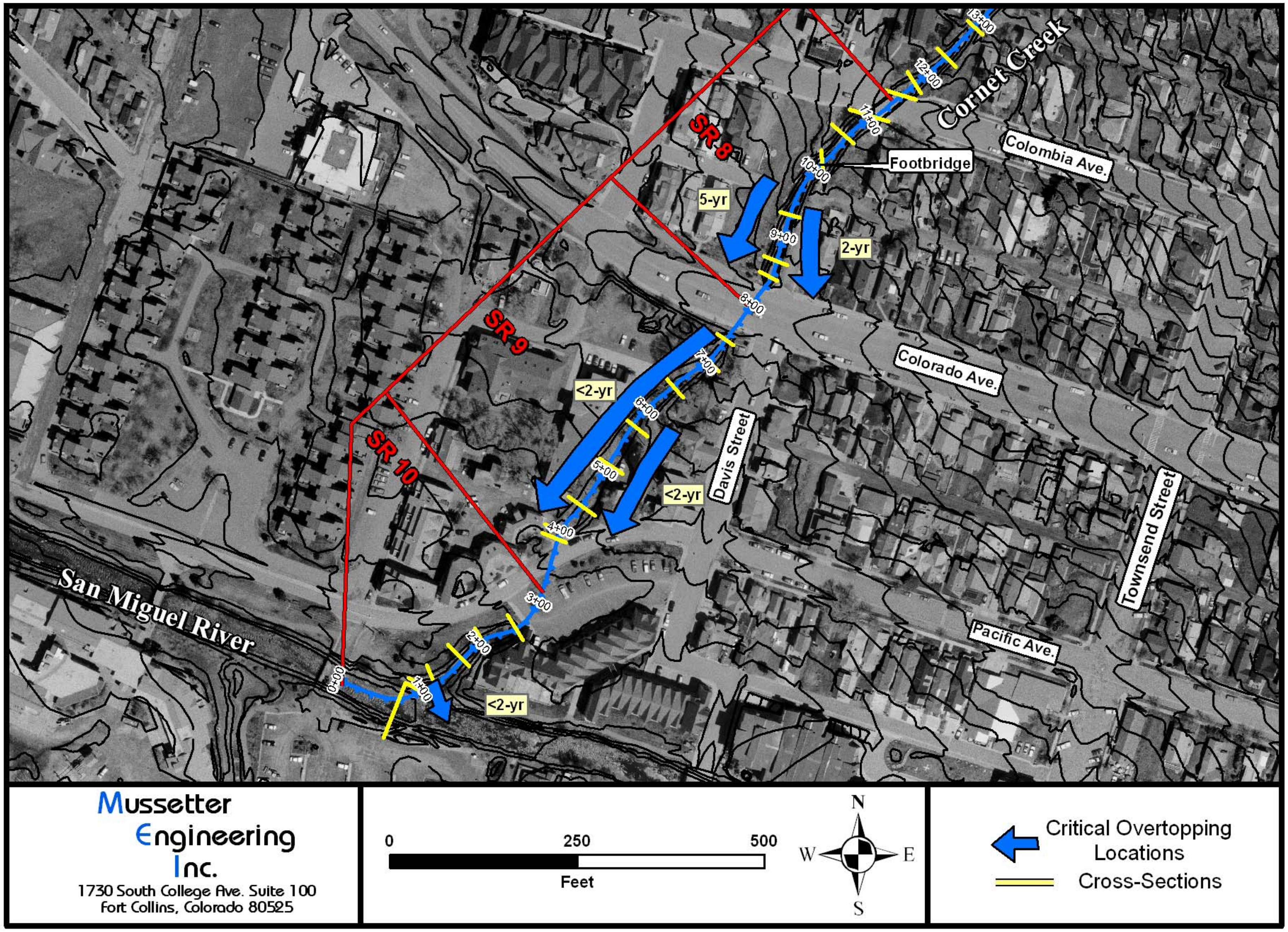


Figure 4.11. Map of Subreaches 8 through 10 of Cornet Creek showing critical channel overtopping locations and the recurrence intervals of the overtopping flows.

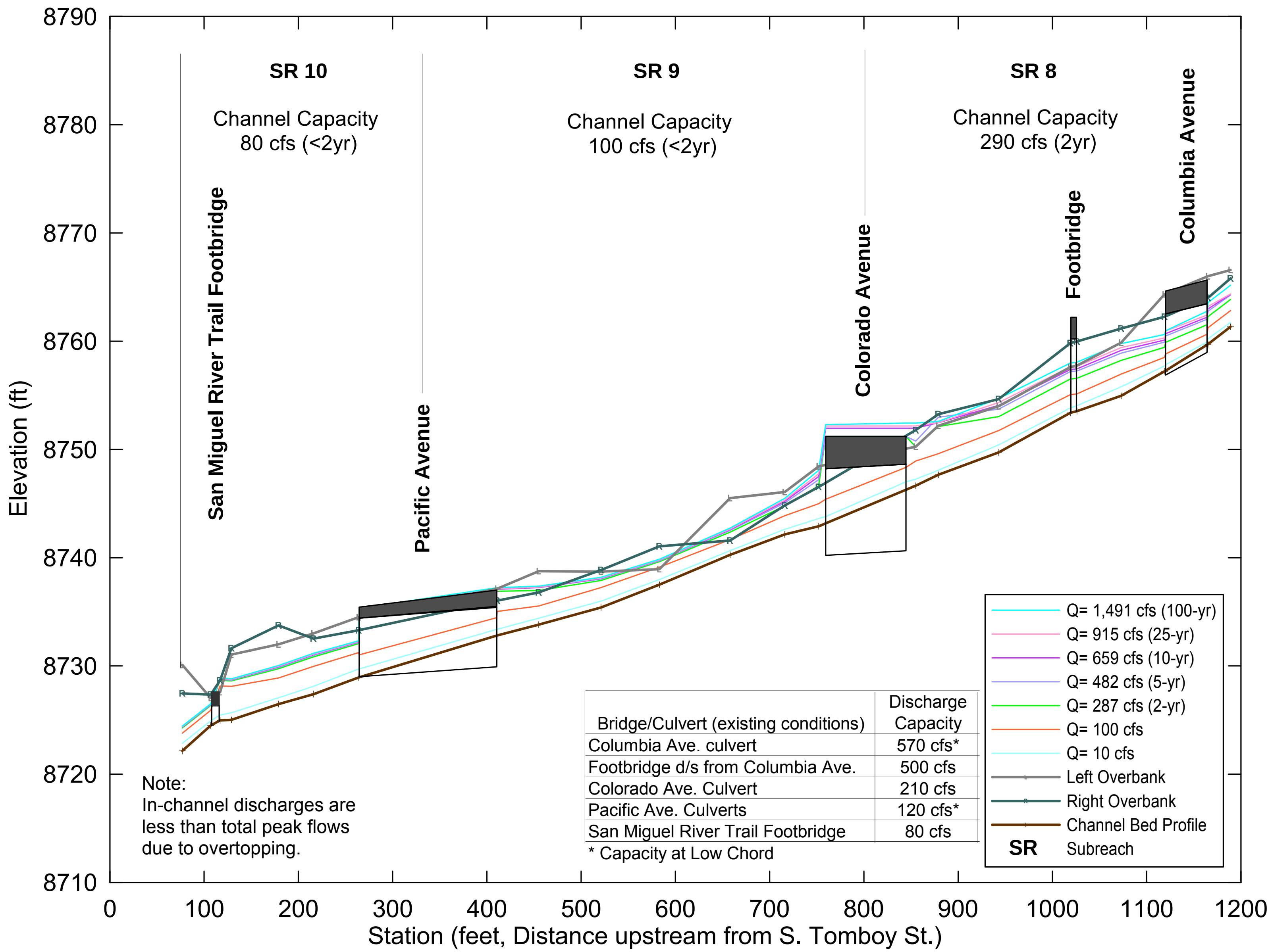


Figure 4.12. Computed water-surface profiles for Subreaches 8 through 10 of Cornet Creek for a range of flows up to the 100-year peak discharge, based on remaining in-channel discharges after overbank flow losses.

The road crossing at Pacific Avenue consists of two corrugated metal arch culverts that have also been severely impacted by sedimentation. As a result, the existing capacity of the structure is much less than the 2-year peak discharge, with flows reaching the top of the pipes at about 120 cfs and beginning to overtop the roadway at approximately 180 cfs. In-channel flows in Subreach 9 have already been significantly diminished due to overtopping of the banks throughout upper portions of the channel, but discharges of about 104, 209, and 286 cfs still escape from the channel during the 2-, 10- and 100-year events. The resulting reach-averaged velocities range from 6.6 fps during the 2-year event to 7.3 fps during the 100-year event. Hydraulic depths remain less than 2 feet and effective widths range between 23.6 and 24.9 feet over the entire range of flows up to and including the 100-year peak flow.

Subreach 10 (Pacific Avenue to the San Miguel River)

The downstream subreach (Subreach 10) is characterized by a channel gradient of about 3 percent (Figure 4.3). Both the left and right bank heights are relatively similar, containing flows up to about 400 cfs upstream from the San Miguel River Trail Footbridge (Figure 4.10). Both banks drop down significantly in the vicinity of the footbridge to accommodate the river trail. These low areas have a capacity of only about 80 cfs, but any flows that escape in this portion of the channel will simply drain directly into the river and do not negatively impact any property or infrastructure (Figure 4.11). The river trail footbridge was designed to be overtopped at moderate to high flows and has a capacity of approximately 80 cfs.

Figure 4.12 shows the estimated in-channel water-surface profile in Subreach 10 based on actual in-channel discharges remaining in the creek after the loss of overtopping flows. Considering the potential for overtopping flow along the entire project reach, the reach-averaged discharges in Subreach 10 are predicted to range from 191 cfs to only 230 cfs at the 2- and 100-year recurrence interval (RI) events (Table 4.1). At these discharges, associated reach-averaged velocities range between 4.5 and 4.8 fps, hydraulic depths range from 2.2 to 2.4 feet, and effective widths range from 19.2 to 20.2 during the 2- and 100-year peak flows, respectively.

Summaries of the existing channel capacity analyses for each subreach and at each channel crossing structure described in this section are provided in **Tables 4.2 and 4.3**.

4.3. Post-flood Hydraulic Analysis

On July 23, 2007, the Town of Telluride experienced a moderate flood event on Cornet Creek that delivered large quantities of mud and rock debris to the upper part of the Town. Most of the debris was derived from a left bank failure within the upstream canyon that eliminated the trail to the Falls. The debris plugged the culvert at Dakota Avenue, causing much of the material to overtop the road and impact adjacent homes. Significant amounts of sediment were transported downstream along the creek, which resulted in problematic levels of aggradation.

After the flooding, the Town excavated critical portions of the channel to restore an adequate level of conveyance capacity to the creek. A supplemental survey of the channel was conducted approximately 1 month after the excavation was completed, and a post-flood conditions (October 2007) HEC-RAS hydraulic model was developed to ascertain and evaluate changes to the creek caused by the flood and subsequent excavation measures.

Based on the supplemental survey, a considerable amount of additional material still resides in the channel for a distance of about 80 feet up- and downstream from Dakota Avenue (**Figure 4.13, Plate 4.1**). As a result, the channel capacity in Subreach 1 has been reduced from about

Table 4.2. Summary of existing channel capacities for each subreach in the detailed study reach of Cornet Creek.			
Subreach	Subreach Extent	Existing Channel Capacity (discharge; approximate recurrence interval)	Primary Overtopping Location
1	Upper end of study reach to Dakota Avenue	480 cfs; 5yr	Along right overbank near private footbridge
2	Dakota Ave. to 100 feet downstream (steep section below culvert outlet)	620 cfs; <10yr	Downstream portion of steep reach below Dakota Avenue
3	100 feet to 150 feet downstream from Dakota Avenue (short depositional area)	120 cfs; <2yr	Along low right overbank adjacent to house
4	150 feet downstream from Dakota Ave. to Townsend Street	550 cfs; 5yr-10yr	Along left overbank near Sta 20+60.
5	Townsend Street to Galena Footbridge	300 cfs; >2yr	Right overbank
6	Galena Footbridge to Galena Avenue	460 cfs; <5yr	Left and right overbanks, primarily in lower portion of reach
7	Galena Avenue to Columbia Avenue	270 cfs; <2yr	Left and right overbanks, primarily in upper portion of reach. Also along right overbank near Sta 12+50.
8	Columbia Avenue to Colorado Avenue	290 cfs; 2yr	Left overbank at lower end of subreach, just above Colorado Avenue
9	Colorado Avenue to Pacific Avenue	100 cfs; <2yr	Left and right overbanks in various locations
10	Pacific Ave. to San Miguel River	80 cfs; <2yr	Lower end of reach near trail crossing.

Table 4.3. Summary of existing bridge and culvert capacities in the detailed study reach of Cornet Creek.		
Structure	Existing Capacity (discharge; approximate recurrence interval)	Remarks
Private Footbridge u/s from Dakota	>100yr	--
Dakota Ave. Culvert	240 cfs; <2yr	Culvert plugged during July 23, 2007, flood
Private Footbridge d/s from Dakota	>100yr	--
Townsend St. Bridge	360 cfs; 2yr - 5yr 440 cfs; <5yr	360 cfs reaches low chord; 440 cfs flows over left overbank
Galena Footbridge	520 cfs; >5yr	Flow spills over banks before reaching low chord.
Columbia Ave. Culvert	570 cfs; 5yr - 10yr 760 cfs; >10yr	570 cfs reaches low chord; 760 cfs flows over road
Footbridge d/s from Columbia Ave.	500 cfs; 5yr	Flow spills over left bank before reach low chord.
Colorado Ave. Culvert	210 cfs; <2yr	--
Pacific Ave. Culverts	120 cfs; <2yr 180 cfs; <2yr	120 cfs reaches low chord; 180 cfs starts to overtop bridge
San Miguel River Trail Footbridge	80 cfs; <2yr	--

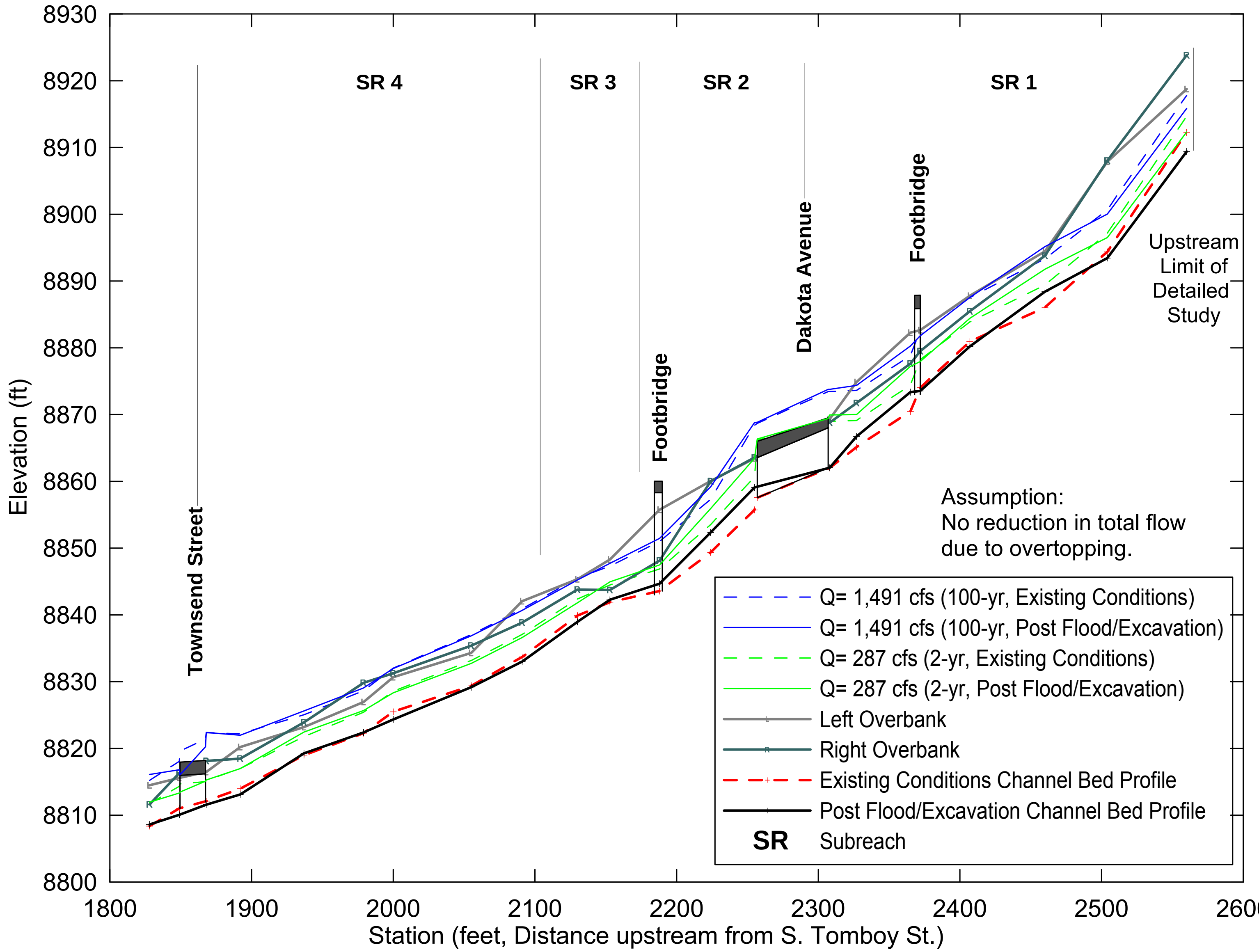


Figure 4.13. Existing conditions and post-flood/excavation conditions channel bed and computed 2- and 100-year water-surface profiles for Subreaches 1 through 4.

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Plate 4.1. View looking upstream at Cornet Creek from Dakota Avenue, showing existing (pre-flood) and post-flood/excavated conditions.

480 to 260 cfs, and the capacity of the Dakota Avenue culvert has decreased from 240 to about 190 cfs (**Tables 4.4 and 4.5**). Deposition also occurred under the Townsend Street Bridge, but the channel was partially excavated (primarily along the left side of the channel looking downstream; **Plate 4.2**), which returned the capacity to approximately that of existing (pre-flood) conditions.

No significant changes occurred in Subreach 5, but the post-flood survey indicated net aggradation in Subreaches 6 and 7 (**Figure 4.14**). As a result, the channel capacity in Subreaches 6 and 7 has been reduced from 460 to 310 cfs and from 270 to 220 cfs, respectively (Table 4.2). A reasonable amount of excavation was carried out after the flood near the lower end of Subreach 7 at the Columbia Avenue Bridge (**Plate 4.3**), slightly increasing the bridge capacity from about 570 to 600 cfs (Table 4.3).

Large amounts of material were excavated from the channel at Colorado Avenue extending to at least 200 feet upstream into Subreach 8 (**Figure 4.15**). During the post-flood excavation, the bed of the channel was lowered to the bottom of the 8-foot by 8-foot concrete box culvert at Colorado Avenue (Karen Guglielmone, Town of Telluride Public Works Department, personal communication, November 2007). Figure 4.15 and **Plate 4.4** indicate that although the channel bed elevations are lower than before the flood, about 4 feet of deposition occurred locally near the inlet to the Colorado Avenue culvert between completion of the post-flood excavation and the channel survey. The resulting channel capacity in Subreach 8 has increased from 290 to about 440 cfs, and the capacity of the Colorado Avenue Culvert has increased from 210 to about 300 cfs.

The post-flood/excavated profile is not significantly different from existing conditions in the two lower subreaches (Subreach 9 and 10; Figure 4.15). However, the excavation efforts did increase the capacity of the Pacific Avenue culverts from 120 cfs to about 170 cfs. Deposition at the San Miguel River Trail Footbridge has reduced the capacity from about 80 cfs to only 30 cfs (**Plate 4.5**).

Table 4.4. Summary of existing (June 2007) and post-flood/excavated (October 2007) channel capacities.

Subreach	Subreach Extent	Existing channel capacity (discharge; approximate recurrence interval)	Post-flood/excavated channel capacity (discharge; approximate recurrence interval)
1	Upper end of study reach to Dakota Avenue	480 cfs; 5yr	260 cfs; <2yr
2	Dakota Ave. to 100 feet downstream (steep section below culvert outlet)	620 cfs; <10yr	510 cfs; 5yr-10yr
3	100 feet to 150 feet downstream from Dakota Avenue (short depositional area)	120 cfs; <2yr	200 cfs; <2yr
4	150 feet downstream from Dakota Ave. to Townsend Street	550 cfs; 5yr-10yr	550 cfs; 5yr-10yr
5	Townsend Street to Galena Footbridge	300 cfs; >2yr	340 cfs; >2yr
6	Galena Footbridge to Galena Avenue	460 cfs; <5yr	310 cfs; >2yr
7	Galena Avenue to Columbia Avenue	270 cfs; <2yr	220 cfs; <2yr
8	Columbia Avenue to Colorado Avenue	290 cfs; 2yr	440 cfs; <5yr
9	Colorado Avenue to Pacific Avenue	100 cfs; <2yr	120 cfs; <2yr
10	Pacific Ave. to San Miguel River	80 cfs; <2yr	30 cfs; <2yr

Table 4.5. Summary of existing (June 2007) and post-flood/excavated (October 2007) bridge and culvert capacities.

Structure	Existing capacity (discharge; approximate recurrence interval)	Post-flood/excavated capacity (discharge; approximate recurrence interval)
Private Footbridge u/s from Dakota	>100yr	>100yr
Dakota Ave. Culvert	240 cfs; <2yr	190 cfs; <2yr
Private Footbridge d/s from Dakota	>100yr	>100yr
Townsend St. Bridge	360 cfs; 2yr - 5yr	380 cfs; 2yr - 5yr
Galena Footbridge	520 cfs; >5yr	480 cfs; 5yr
Columbia Ave. Culvert	570 cfs; 5yr - 10yr	600 cfs; <10yr
Footbridge d/s from Columbia Ave.	500 cfs; 5yr	460 cfs ; <5yr
Colorado Ave. Culvert	210 cfs; <2yr	300 cfs; 2yr
Pacific Ave. Culverts	120 cfs; <2yr	170 cfs; <2yr
San Miguel River Trail Footbridge	80 cfs; <2yr	30 cfs; <2yr

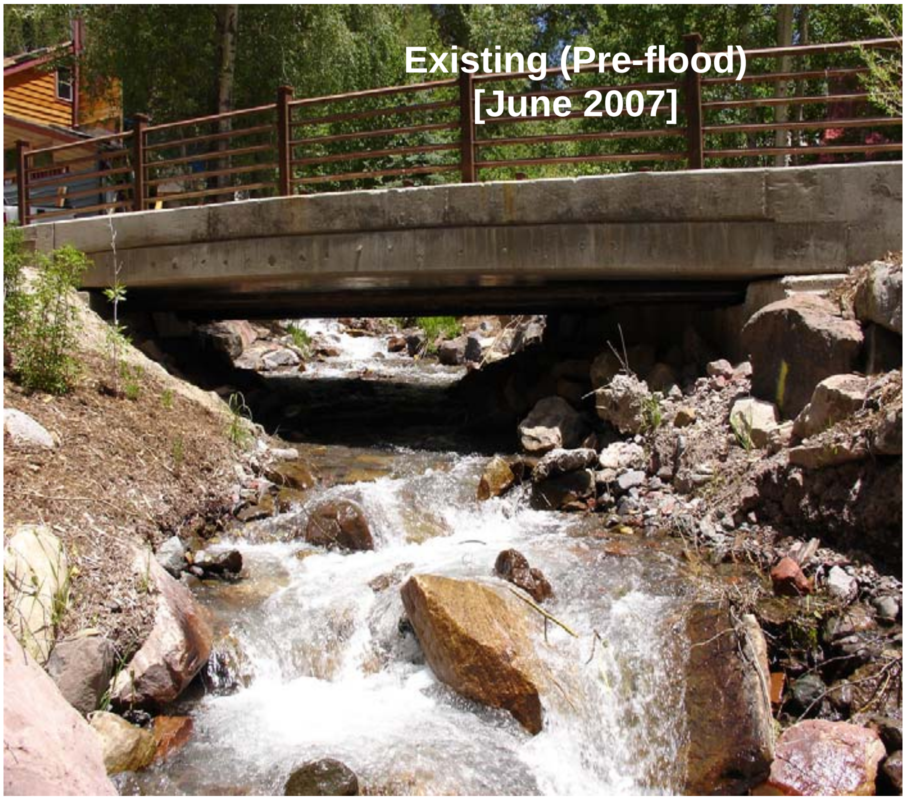

Plate 4.2. View looking upstream at Townsend Street Bridge on Cornet Creek, showing existing (pre-flood) and post-flood/excavated conditions.

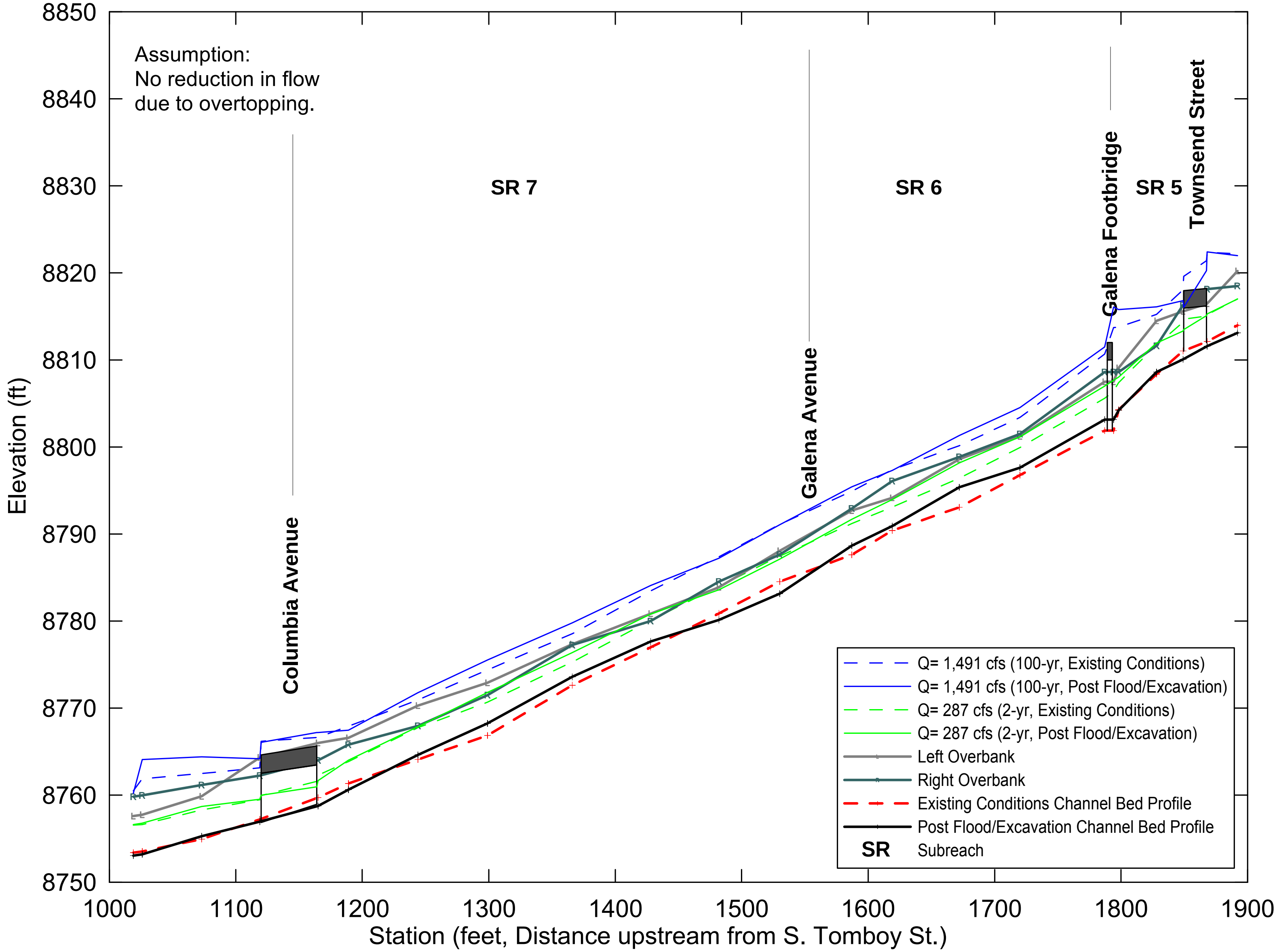

Figure 4.14. Existing conditions and post-flood/excavation conditions channel bed and computed 2-year and 100-year water-surface profiles for Subreaches 5 through 7.

Plate 4.3. View looking upstream at Columbia Avenue Bridge on Cornet Creek, showing existing (pre-flood) and post-flood/excavated conditions.

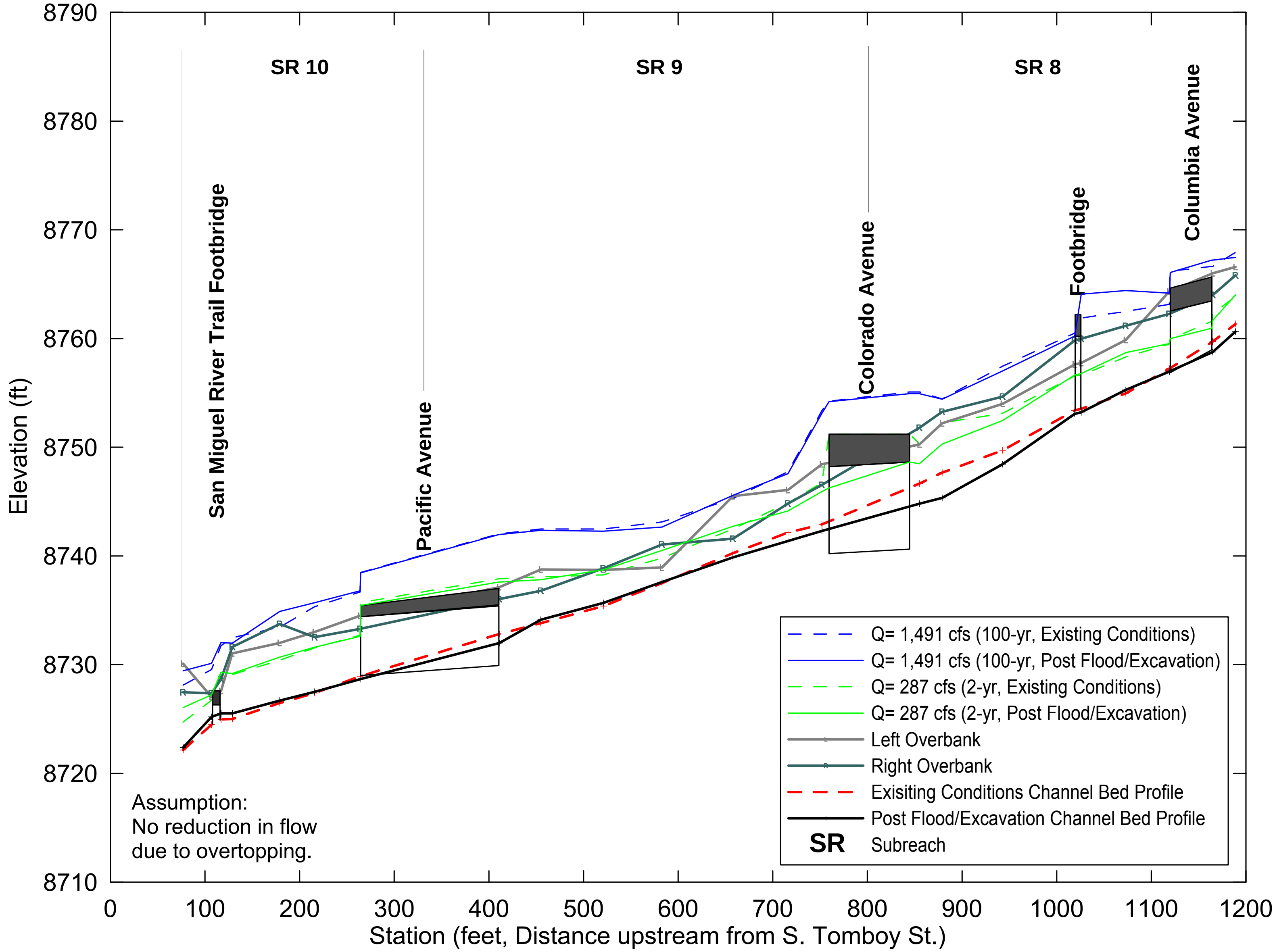

Figure 4.15. Existing conditions and post-flood/excavation conditions channel bed and computed 2-year and 100-year water-surface profiles for Subreaches 8 through 10.

Plate 4.4. View looking downstream along Cornet Creek at Colorado Avenue showing post-flood/excavated conditions in the channel.

Plate 4.5. View looking upstream at the San Miguel River Trail Footbridge on Cornet Creek, showing existing (pre-flood) and post-flood/excavated conditions.

5. SEDIMENT-TRANSPORT ANALYSIS

Based on the results of the hydraulic analysis, a sediment-transport analysis was carried out to evaluate the vertical stability of the project reach under existing and recommended design conditions. The investigation consisted of an incipient motion analysis to assess the range of flows over which the existing bed material is mobilized, and a sediment-continuity analysis to evaluate potential aggradation/degradation trends among the ten subreaches defined in the hydraulic analysis (Section 4.2, Figures 4.1 and 4.3).

5.1. Incipient Motion Analysis

The incipient motion analysis was performed by evaluating the effective shear stress on the channel bed in relation to the amount of shear stress that is required to move the sizes of sediment that are present. The shear stress required for bed mobilization was estimated using the Shields (1936) relation, given by:

$$\tau_C = \tau_{*_C} (\gamma_S - \gamma) D_{50} \tag{5.1}$$

where τ_c = critical shear stress for particle motion,
τ_{*c} = dimensionless critical shear stress (often referred to as the Shields parameter),
γ_s = unit weight of sediment (~165 lb/ft^3),
γ = unit weight of water (62.4 lb/ft^3), and
D_{50} = median particle size of the bed material.

In gravel- and cobble-bed streams, when the critical shear stress for the median particle size is exceeded, the bed is mobilized and all sizes up to about five times the median size can be transported by the flow (Parker et al., 1982; Andrews, 1984).

Reported values for the Shields parameter range from 0.03 (Neill, 1968; Andrews, 1984) to 0.06 (Shields, 1936). A value of 0.047 is commonly used in engineering practice, based on the point at which the Meyer-Peter, Müller bed-load equation would indicate no transport (Meyer-Peter and Müller, 1948). Detailed evaluation of Meyer-Peter and Müller's data and more recent data (Parker et al., 1982; Andrews, 1984) indicate that true incipient motion occurs at a value of about 0.03 in gravel- and cobble-bed streams. Neill (1968) concluded that a dimensionless shear value of 0.03 corresponds to true incipient motion of the bed-material matrix while 0.047 corresponds to a low, but measurable transport rate. A value of 0.03 was used in this analysis.

In performing the incipient-motion analysis, the bed shear stress due to grain resistance (τ') is used rather than the total shear stress, because it is a better descriptor of the near-bed hydraulic conditions that are responsible for sediment movement. The grain shear stress is computed from the following relation:

$$\tau' = \gamma R' S \tag{5.2}$$

where R' = the portion of the total hydraulic stress associated with grain resistance (Einstein, 1950), and
S = the energy slope at the cross section.

The value of R' is computed by iteratively solving the semi-logarithmic velocity profile equation:

$$\frac{V}{U_*^{'}} = 5.75 + 6.25\log(\frac{R'}{k_s}) \qquad (5.3)$$

where V = mean velocity at the cross section,
k_s = characteristic roughness of the bed, and
U_*' = shear velocity due to grain resistance given by:

$$U_*^{'} = \sqrt{gR'S} \qquad (5.4)$$

The characteristic roughness height of the bed (k_s) was assumed to be 3.5 D_{84} (Hey, 1979).

The dimensionless grain shear stress for a specific discharge, which is defined as ratio of the grain shear stress (Equation 5.2) to the critical shear stress (Equation 5.1), provides a measure of the relative ability of that discharge to mobilize the bed material. Values of the dimensionless shear stress less than 1 indicate that the bed material is not mobile and values greater than 1 indicate bed-material mobility. Additionally, when the dimensionless grain shear stress is between 1 and about 1.5, the bed-material transport rate is very low. Under these marginal transport conditions, when the upstream supply is also low, the bed will quickly armor and significant downcutting will not occur. When the value of the dimensionless shear stress exceeds 1.5, there is general mobilization of the bed material.

The dimensionless grain shear stress was computed for each subreach over the range of discharges up to and including the 100-year peak flow based on reach-averaged hydraulics and bed-material gradations. However, throughout most of the reach, flows greater than generally the 2- to 5-year event spill into the overbanks. As a result, additional increases in discharge cause a limited increase in shear stress in the channel.

The incipient motion analysis indicates that extremely small discharges of less than 10 cfs are required to mobilize the bed material in all 10 subreaches (**Figure 5.1**) because of the steepness of the channel. This result is consistent with field observations during the June 25 through 27 site visit, which indicated that the bed material in the channel was at or near incipient-motion conditions at a measured discharge of approximately 5 cfs.

An analysis of the effective channel shear stress shows that, as expected, bed shear stresses are greater in the upper portion of the study reach, where the channel gradients are significantly steeper (Figure 5.1 and **Figure 5.2**). The average shear stress in each subreach is also an indicator of the channel's ability to transport sediment relative to adjacent subreaches. Details of the sediment-transport analysis are discussed in the following sections, but the reach-averaged shear stresses do identify certain locations that are likely to experience considerable aggradation. For example, the bed shear stresses in Subreach 3 are much lower than those in Subreach 2 over the entire range of modeled discharges (Figure 5.2), which depending on the relative size of the bed material, suggests that Subreach 3 may not be able to fully convey all of the sediment entering from upstream, thus indicating that deposition is likely to occur. Similarly, deposition is likely to occur in Subreaches 8, 9 and 10. The results of the shear stress analysis are supported by the patterns of deposition that were observed following the July 2007 flood.

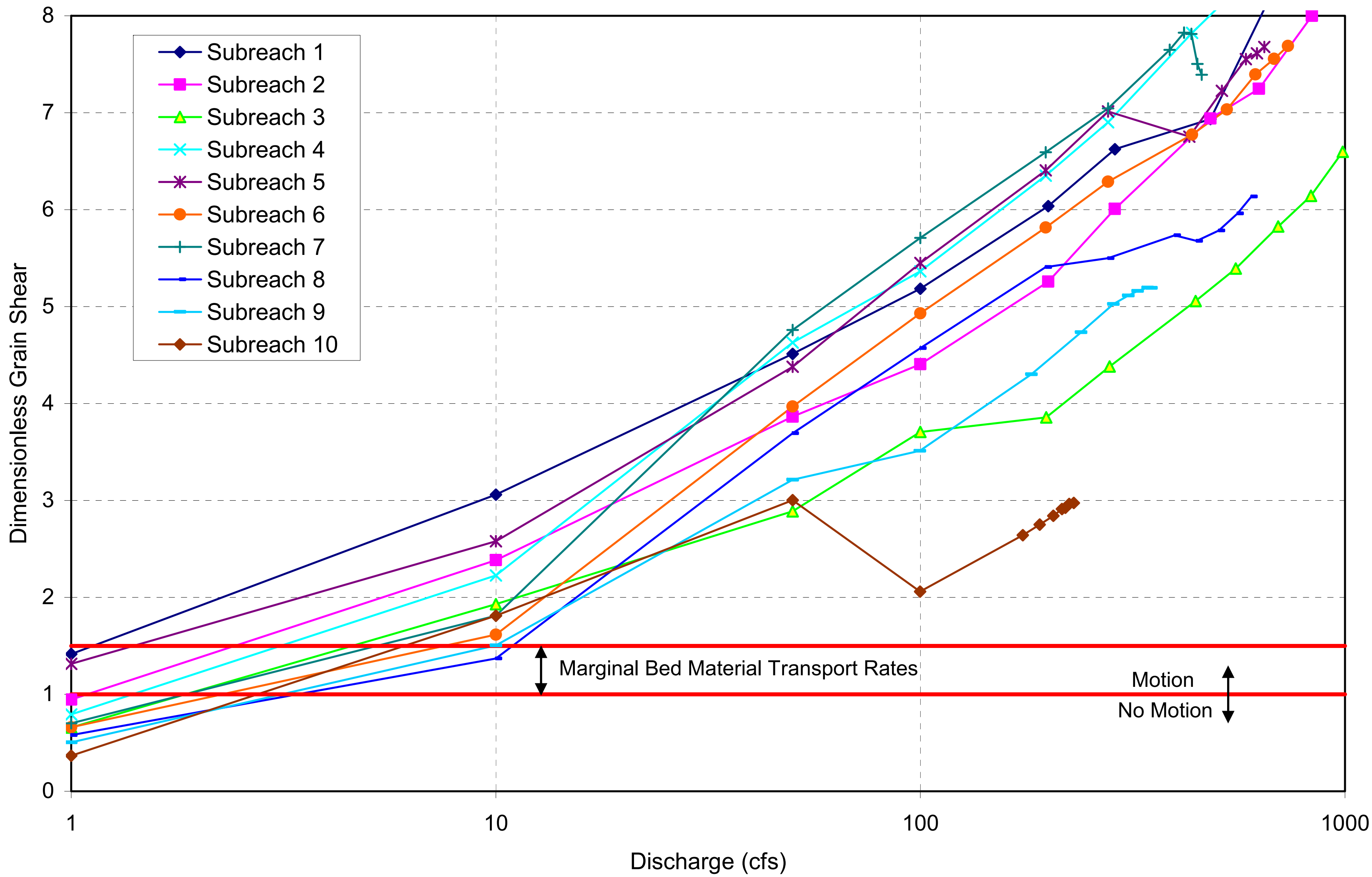

Figure 5.1. Variation in dimensionless grain shear stress with discharge in Subreaches 1 through 10 of the study reach of Cornet Creek.

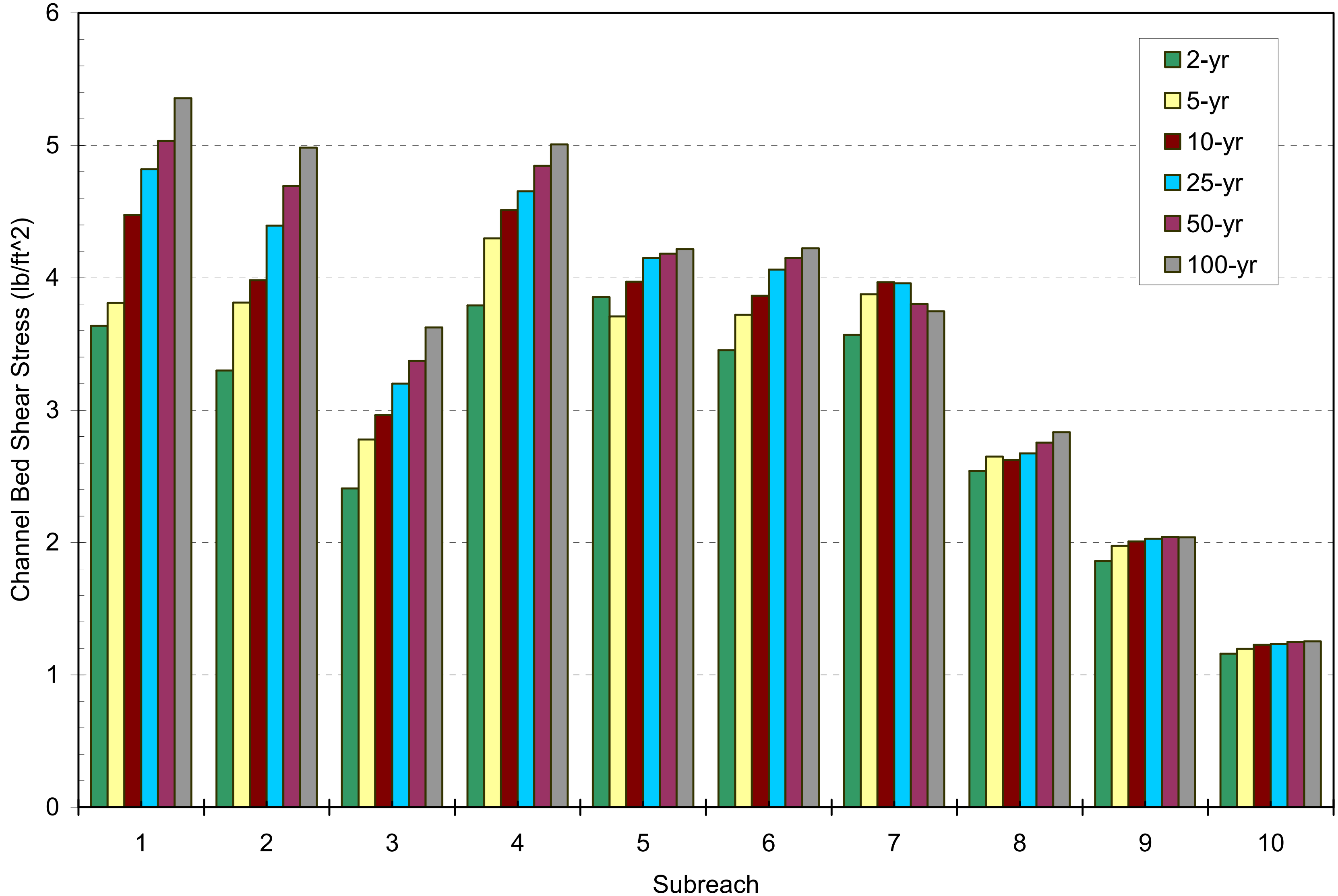

Figure 5.2. Effective channel bed shear stress in Subreaches 1 through 10 for the 2- through 100-year peak flow events.

The reach-averaged bed shear stresses also demonstrate the effect of flow loss in the system. As described in Section 4, the upstream subreaches have a slightly higher channel capacity (e.g., Subreaches 1, 2, and 4), which allows the shear stress to continue to increase with increases in discharge (Figure 5.2). The subreaches where the banks are overtopped at relatively low flows (e.g., Subreaches 7 through 10), however, show only minor increases in shear stress at higher discharges.

5.2. Bed-material Transport Capacity

The bed-material transport capacity for each subreach was determined by developing a bed-material sediment rating curve (i.e., the relationship between bed-material transport capacity and discharge) using the reach-averaged hydraulics predicted by the HEC-RAS model, representative bed-material sediment gradations, and an appropriate sediment-transport function.

5.2.1. Representative Bed-material Gradation

Representative bed-material sediment gradations for use in the sediment-continuity analysis were developed by incorporating a specified portion of the sand fraction from the bulk bed samples into the pebble count gradations (**Figure 5.3**). The appropriate amount of sand added to the gradation was determined by ensuring that the transported bed-material gradations computed by the selected sediment-transport function (see following sections) were similar to those of the actual samples. The median diameters (D_{50}) of the representative gradations range from 39 mm in Subreach 9 to about 55 mm in Subreaches 1 through 6.

5.2.2. Bed-material Transport Function

Several possible bed-material transport capacity relationships were examined for use in this study based on previous experience in similar environments. The Wilcock-Crowe (2003) relationship for surface-based transport was selected because the conditions for which it was developed are similar to those in the study area, and because it predicts sediment loads that are consistent with observed loads in channels with a similar range of hydraulic conditions and bed-material composition (i.e., mobile cobble/gravel bed with considerable amounts of sand).

The resulting bed-material transport capacity rating curves for the project reach under existing conditions are summarized in **Figure 5.4**. As indicated in the figure, the transport capacities for each subreach are relatively similar, but there is a trend of generally decreasing transport rates in the downstream direction for equivalent discharge. In addition, the rating curves demonstrate that Subreach 3 has a much lower transport capacity than the upstream subreaches, which is consistent with the shear stress results discussed in Section 5.1.

5.2.3. Sediment-continuity Analysis

A sediment-continuity analysis was performed for the study reach under existing conditions to assess the potential for aggradation (raising of the channel bed) or degradation (lowering of the channel bed) in response to the individual recurrence interval storms, and on an average annual basis. The analysis was performed by integrating the bed-material rating curves for each subreach over the respective storm hydrographs, and comparing the resulting bed-material transport volumes with the upstream supply to each subreach. Where the transport capacity of a particular subreach exceeds the supply, the channel will respond by either degrading (i.e., channel downcutting) or coarsening its bed material. In systems with significant gravel and coarser sediment, such as Cornet Creek, degradation tendencies can also lead to the

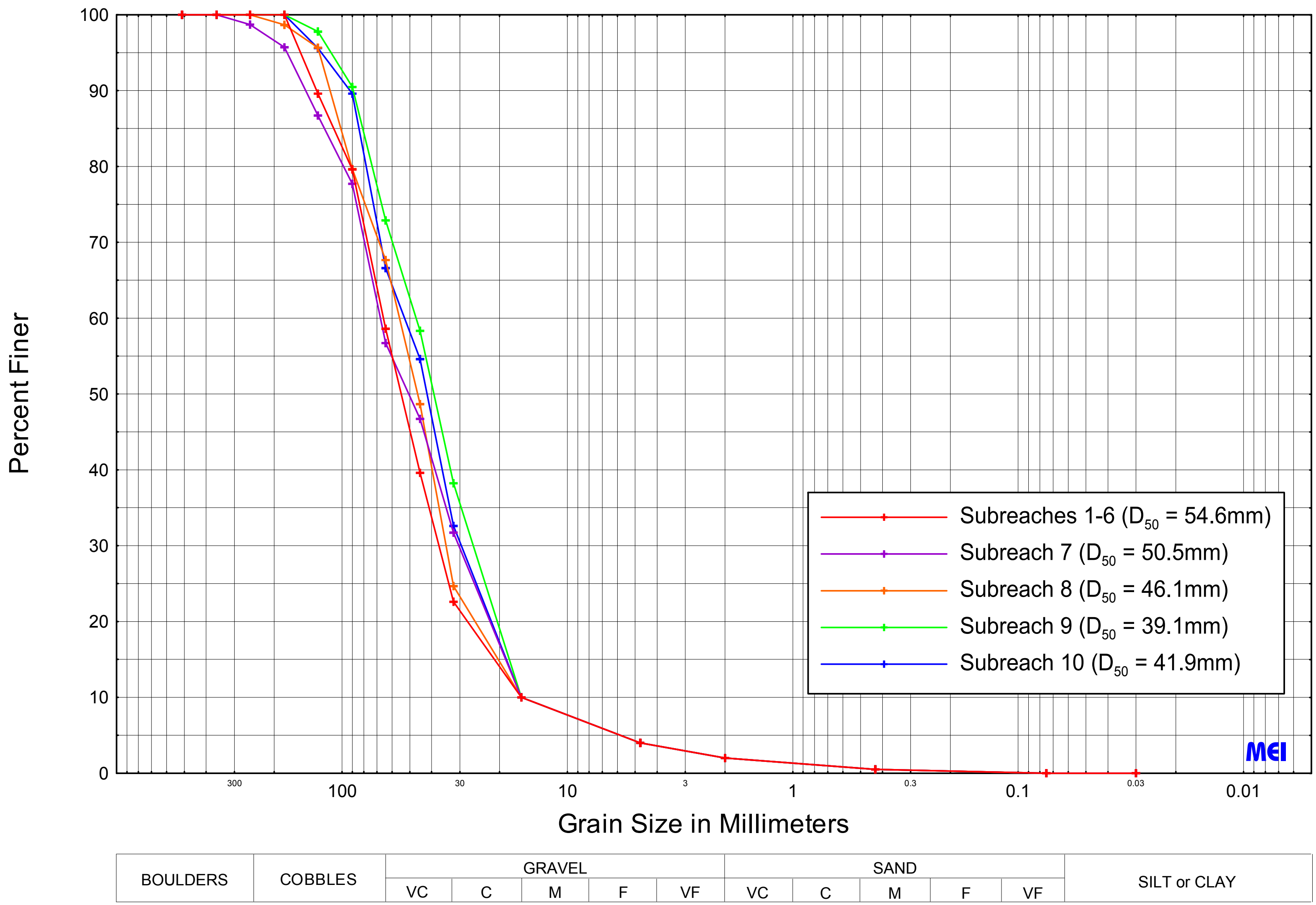

Figure 5.3. Representative bed-material gradations used in the sediment-transport analysis.

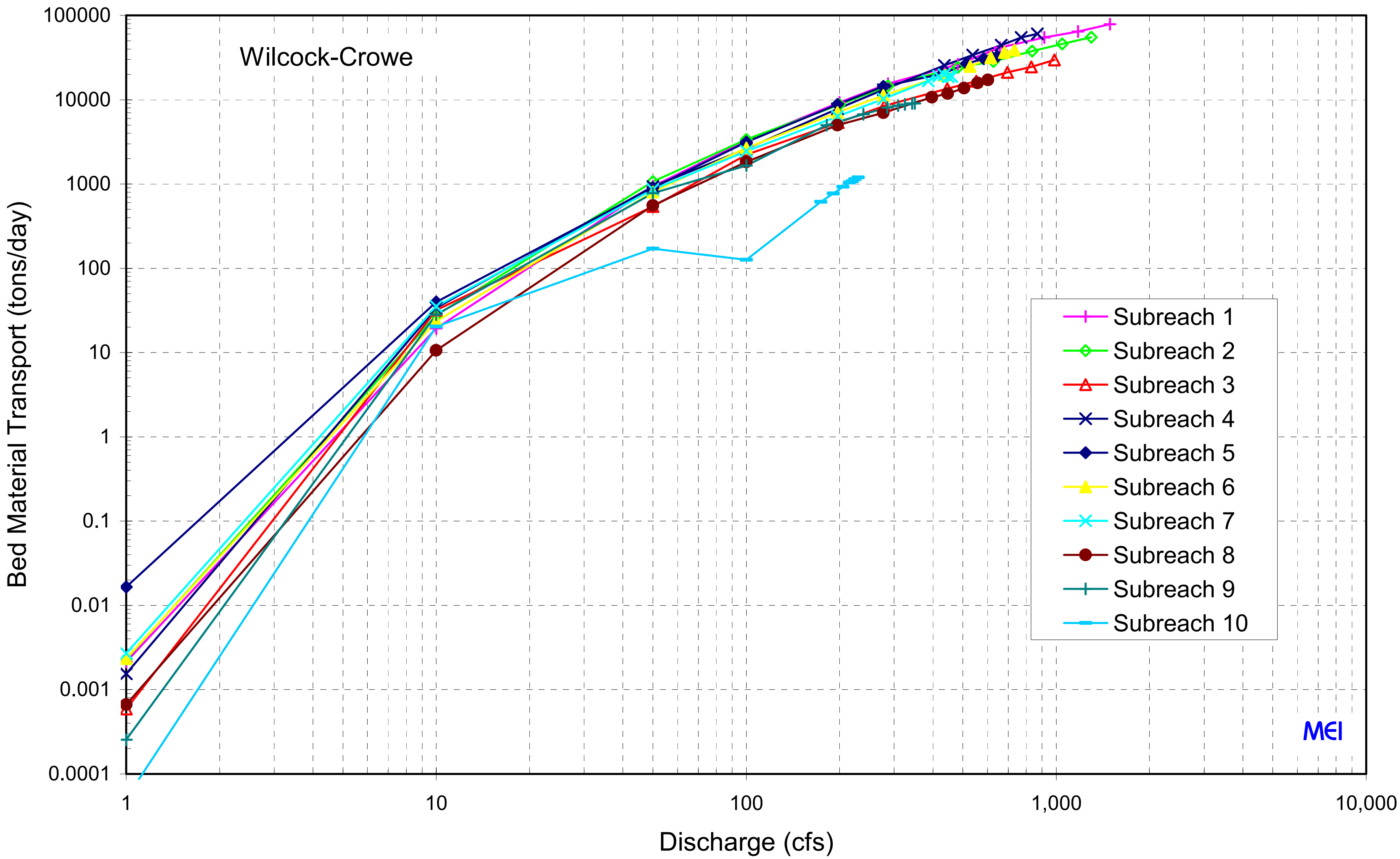

Figure 5.4. Existing conditions bed-material transport rating curves for Subreaches 1 through 10 of the study reach of Cornet Creek.

development of an armor layer that will inhibit downcutting. Where the supply exceeds the capacity, the channel will respond by aggrading and/or fining its bed material (i.e., decreasing the bed-material size as smaller-grained sediment is deposited on the bed).

For purposes of comparison, the difference in sediment volume between the upstream supply and transport capacity was converted to an average change in bed elevation based on the subreach length and average channel width. It should be noted that the aggradation/ degradation depths are averages for the subreach; the actual amount that would occur in any particular location can vary significantly from the average, depending on the local flow and sedimentation patterns.

The sediment volumes obtained by integrating the rating curves in Figure 5.4 over the individual storm hydrographs indicate a general trend of decreasing transport in the downstream direction (**Figure 5.5**). Transport volumes in Subreach 1 range from about 280 tons during the 2-year event to about 2,200 tons during the 100-year event. The transport rates are much higher in Subreach 1 at the higher recurrence interval flows because the majority of the reach, except for immediately upstream from the Dakota Avenue culvert contains the flows up to the 100-year peak. As the channel bed gradient decreases in the downstream direction, transport capacities also decrease to a range of about 25 tons during the 2-year peak flow to about 80 tons during the 100-year peak flow.

Comparison of the subreach transport volumes with the supply from the next upstream subreach indicates that the majority of the channel is aggradational for all of the storms that were analyzed, which is consistent with field observations following the July 2007 flood and long-term maintenance requirements that the Town has encountered. Subreach-averaged aggradation/degradation depths were estimated for the entire reach (**Figure 5.6**). For the purpose of this analysis, the transport capacities computed for Subreach 1 were assumed to adequately represent the potential sediment supply to the study reach. As a result of having transport capacities equal to the supply, Subreach 1 is shown as being essentially in equilibrium. Based on the responses to recent flood events, however, this subreach is likely to be slightly aggradational under certain flow conditions due to the limited culvert size at Dakota Avenue. Subreach 2, on the downstream side of Dakota Avenue, shows potential aggradation depths ranging from 0.1 feet at the 2-year event to about 1.8 feet at the 50-year event. The aggradation depth at the 100-year peak flow of more than 5 feet is due to the large amount of flow that escapes from the channel in the vicinity of Dakota Avenue during this event, which drastically reduces the transport capacity. Subreach 3 is an obvious sediment trap with estimated aggradation depths of between 0.6 feet at the 2-year event to almost 4 feet at the higher peak flows (Figure 5.6).

The estimated aggradation/degradation depths in Subreach 4 are negligible and fluctuate slightly over the modeled range of flows. This indicates that Subreach 4 is essentially in equilibrium under existing conditions, but may experience minor erosional or depositional conditions from time to time. Figure 4.13 shows that the channel bed profile in Subreach 4 did not change significantly during the recent July 2007 flood event, which is consistent with the sediment continuity results. The recent replacement of Townsend Street Bridge in November 2005 increased the flow capacity of the structure, but given the relatively steep channel gradient, it did not significantly impact the hydraulic or sediment-transport conditions in the upstream subreach (Subreach 4). The bridge replacement, however, should help reduce the potential for blockage. A short section of channel between Townsend Street and the Galena Avenue Footbridge represents the extent of Subreach 5. This portion of the channel is slightly steeper than upstream of Townsend Street, but the additional loss of flow and obstructions in the channel reduce the overall transport capacity. As a result, aggradation depths could

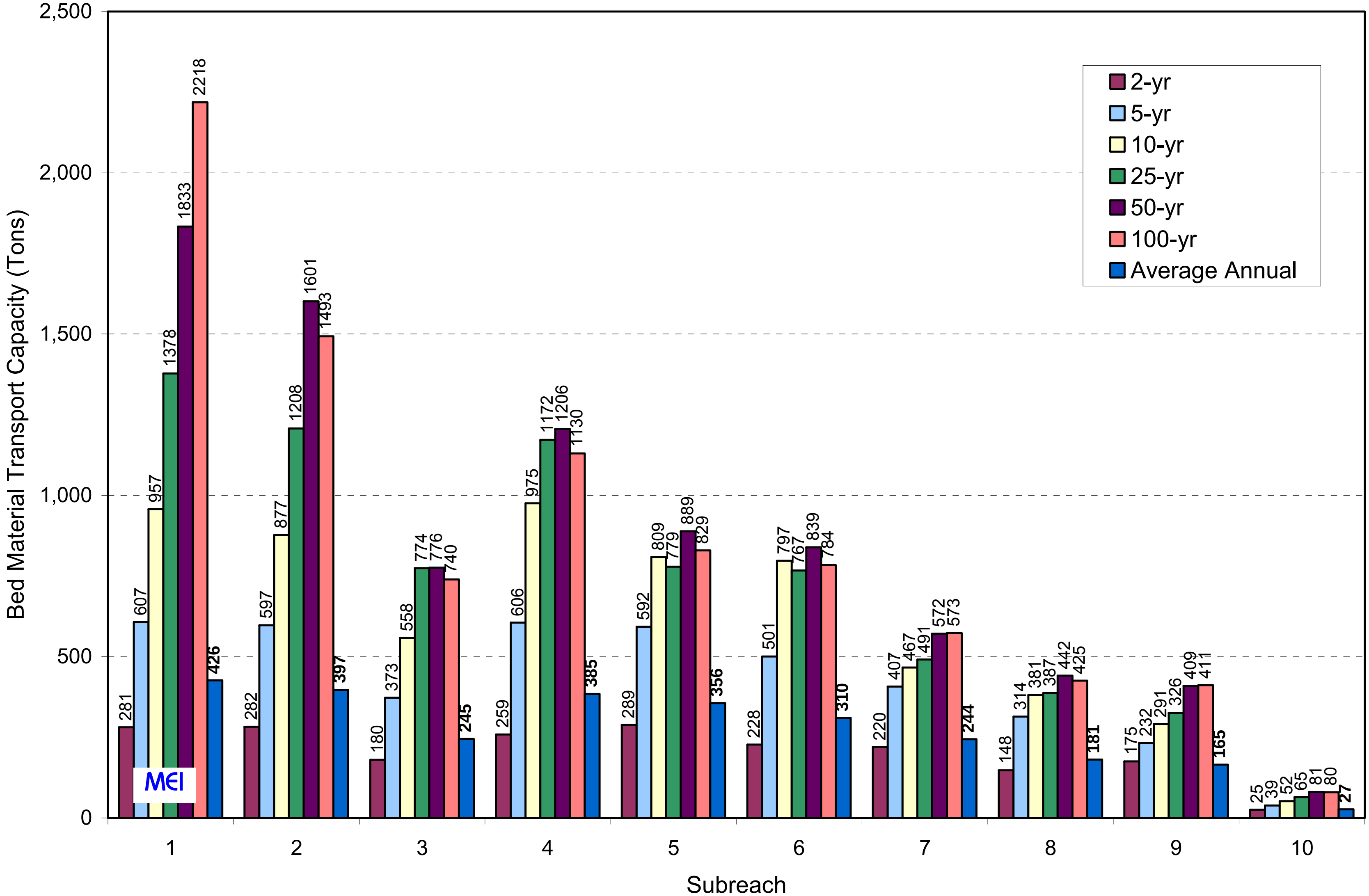

Figure 5.5. Existing conditions reach-averaged bed-material transport capacities for Subreaches 1 through 10 of the project reach of Cornet Creek.

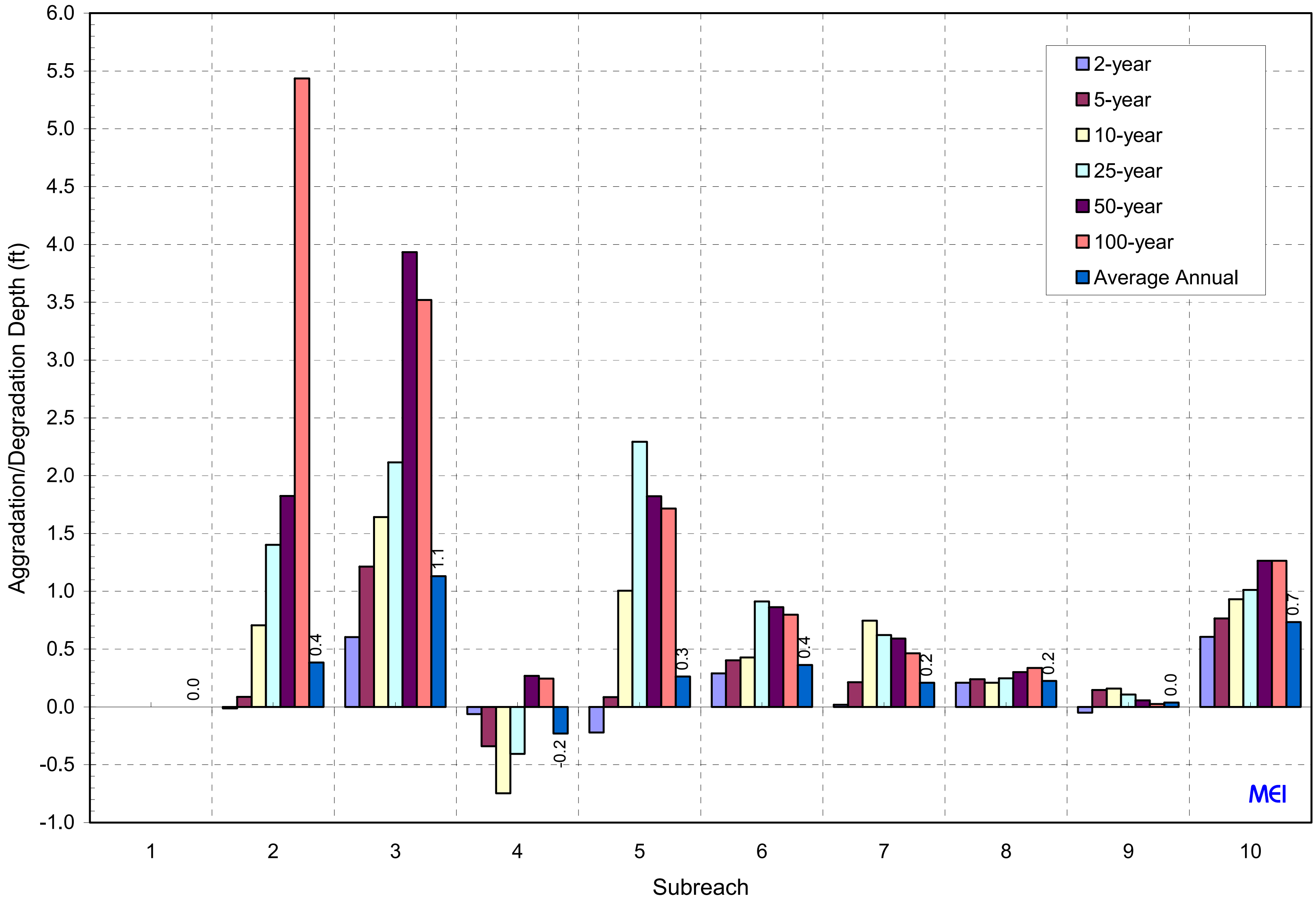

Figure 5.6. Existing conditions reach-averaged aggradation/degradation depths for Subreaches 1 through 10 of the project reach of Cornet Creek.

potentially reach 2.3 feet during the 25-year flood event (Figure 5.6). Because this subreach is relatively short, a considerable amount of sediment is likely to pass downstream into Subreach 6.

Subreaches 6 and 7 encompass the portion of the creek in the vicinity of Galena Avenue (Figure 4.1). These segments of channel have a limited capacity, and indicate similar estimated aggradation depths of less than 1 foot over the entire range of modeled flows (Figure 5.6). As mentioned above, however, greater levels of aggradation could potentially occur in Subreach 6 due to the likelihood of sediment passing through Subreach 5, which is relatively short in length. The aggradation trend continues downstream into Subreaches 8 and 9, but estimated depths are relatively low due to the similar magnitude of the transport capacities (Figure 5.5). Sediment transport rates in the lower end of the study reach (Subreach 10) decrease further resulting in increased aggradation depths that range from 0.6 feet during the 2-year event to about 1.3 feet during the 100-year event (Figure 5.6).

In general, results from the sediment transport analysis indicate that almost the entire portion of Cornet Creek located within the Town limits is primarily aggradational, which does correspond with historical observations. Furthermore, the sediment transport results are consistent with the behavior observed during recent flood events, which show that the vertical response of the channel to varying sediment loads over the range of modeled flows is much more dynamic in the upper portion of the reach, but that significant levels of aggradation are still likely to occur in lower sections of the creek.

6. PRELIMINARY DESIGN AND RECOMMENDATIONS

Previous studies of Cornet Creek (refer to Chapter 1) have identified limitations in the creek's ability to convey water and sediment, and have recommended improvements to all drainage facilities, including the capacity of the creek. Many of the previous recommendations, however, have not been implemented due to financial costs, environmental issues, logistics, or public acceptance.

The Town has attempted to maintain an adequate conveyance capacity of the channel through periodic maintenance, which has primarily consisted of manual excavation (clean out) of the creek and improvements to bridge and culvert crossings. Issues that needed to be addressed to support the maintenance of the conveyance capacity of the creek and to address the occurrence of debris flows included:

3. Development of an appropriate channel grade to maintain or improve capacity without inducing channel instability,

4. Preliminary assessment of replacing/improving low capacity culverts at Dakota and Pacific Avenues to reduce flooding potential, and

5. Evaluation of the potential for debris flows to occur along Cornet Creek and the investigation of potential debris-flow mitigation techniques and state-of-the-art flood-warning systems.

6.1. Channel Improvement Design

The limited conveyance capacity of Cornet Creek as it traverses its alluvial fan is exacerbated by the fact that it does not have an adjacent floodplain to help convey flood flows. As a result, improvements designed to increase flow capacity are primarily restricted to the channel itself. Recommendations for an appropriate channel profile for Cornet Creek within the project reach were developed to increase channel capacity while maintaining a reasonable level of stability and providing a vertical limit during future channel maintenance excavation operations. The recommended channel improvements were based on the following assumptions and constraints:

1. Culvert and bridge crossing configurations at Colorado and Columbia Avenues are not scheduled to be replaced in the near future and should be considered as structures that will not be altered as part of the design,

2. Townsend Street Bridge was recently replaced (November 2005) and the overall structure will not be altered as part of the design. However, minor changes to the elevation of the channel bed in the vicinity of the bridge may be acceptable upon approval by the Town,

3. Existing culvert crossings at Pacific and Dakota Avenues have been identified by the Town as having very limited conveyance capacities, and will likely be replaced in the relatively near future. As a result, preliminary recommendations for the proposed channel profile and dimensions of these crossings were incorporated into the design,

4. Cornet Creek is encroached upon considerably by private property and infrastructure within the Town, which greatly limits the potential for channel widening,

5. All existing water, sewer, electric, phone, and miscellaneous utility lines that cross the channel of Cornet Creek can be relocated in order to prevent future damage due to natural processes in the channel or maintenance activities,

6. The recommended channel design will represent a 'target' condition for channel improvement, but alterations to the design will likely need to be field-engineered in order to accommodate previously unidentified limitations.

The recommended channel improvements were developed by initially modifying the existing longitudinal profile to identify an appropriate grade that will increase channel capacity within the limits imposed by the elevations of existing structures that are not going to be modified. The second objective was to develop appropriate stable channel dimensions to maximize capacity within the narrow stream corridor. To achieve this objective, a cross-sectional channel template consisting of a bottom width of 4 feet with 2H:1V sideslopes was selected. Ideally, a slightly wider channel width would be preferred. However, applying a wider channel width at the target bed elevation in conjunction with stable sideslopes of about 2H:1V results in channel top widths (i.e., widths of the channel between top-of-banks) that are wider than the available space along the stream corridor. Consequently, following discussions with Town staff, a 4-foot channel bottom width was selected as being the narrowest practical width that existing equipment available to the Town could realistically construct while also providing a reasonable increase in channel capacity.

The recommended channel profile is shown in **Figures 6.1 through 6.3**. The culvert profiles at Colorado and Columbia Avenues are based on existing conditions due to the low probability of these structures being replaced in the near future. Lowering the channel bed elevation at Townsend Street Bridge by approximately 2 feet will improve conveyance along the creek. Inspection of the bridge plans and discussions with the original designers of the bridge at Buckhorn Engineering, Inc., indicate that lowering the channel bed by 2 feet should not significantly reduce the stability of the bridge, especially since severe degradation is not likely in this reach because of the coarseness of the bed material. However, if the channel bed is lowered, the stability of the bridge should be evaluated under these altered conditions and either additional protective measures should be incorporated, if necessary, or the Town may need to simply recognize and accept a slightly greater level of risk. The channel bed elevations at Pacific and Dakota Avenues were lowered in an attempt to increase the capacity of these crossings and reduce the potential for future blockages by debris (Figures 6.1 and 6.3).

Numerous obstacles exist along the creek (e.g., buildings, roadways, large trees, power transformers, etc.) that can limit the feasibility of widening the creek as part of the recommended design. As a result, slight variations to the channel dimensions were made to account for known limitations such as buildings and roadways. These variations primarily include the use of retaining walls (or similar structures) along portions of the bank in critically narrow areas to stabilize the channel slopes while reducing the total width of the channel. It may also be possible to utilize existing, yet steeper sideslopes to help minimize the overall channel width. However, this option could result in unstable sideslopes, and will need to be evaluated on-site and probably field-engineered on a case-by-case basis. Cross-sectional schematics showing examples of variations to the channel template are shown in **Figures 6.4 through 6.7**. Previously unknown segments of channel that were lined (at least partially) by large boulders and/or riprap were discovered by representatives of the Town's Public Works

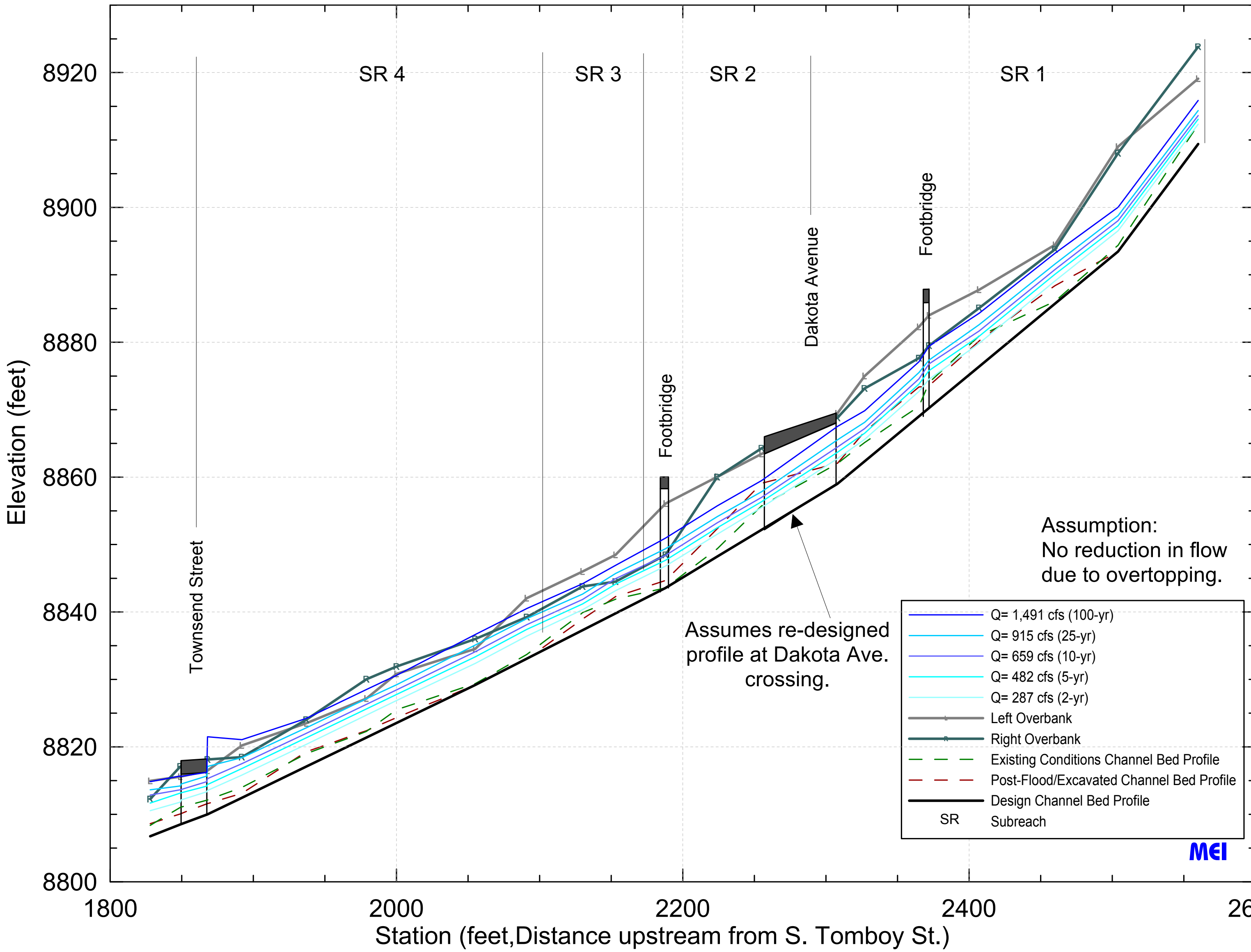

Figure 6.1. Design conditions channel bed and computed water-surface profiles for Subreaches 1 through 4 for a range of flows up to the 100-year peak discharge.

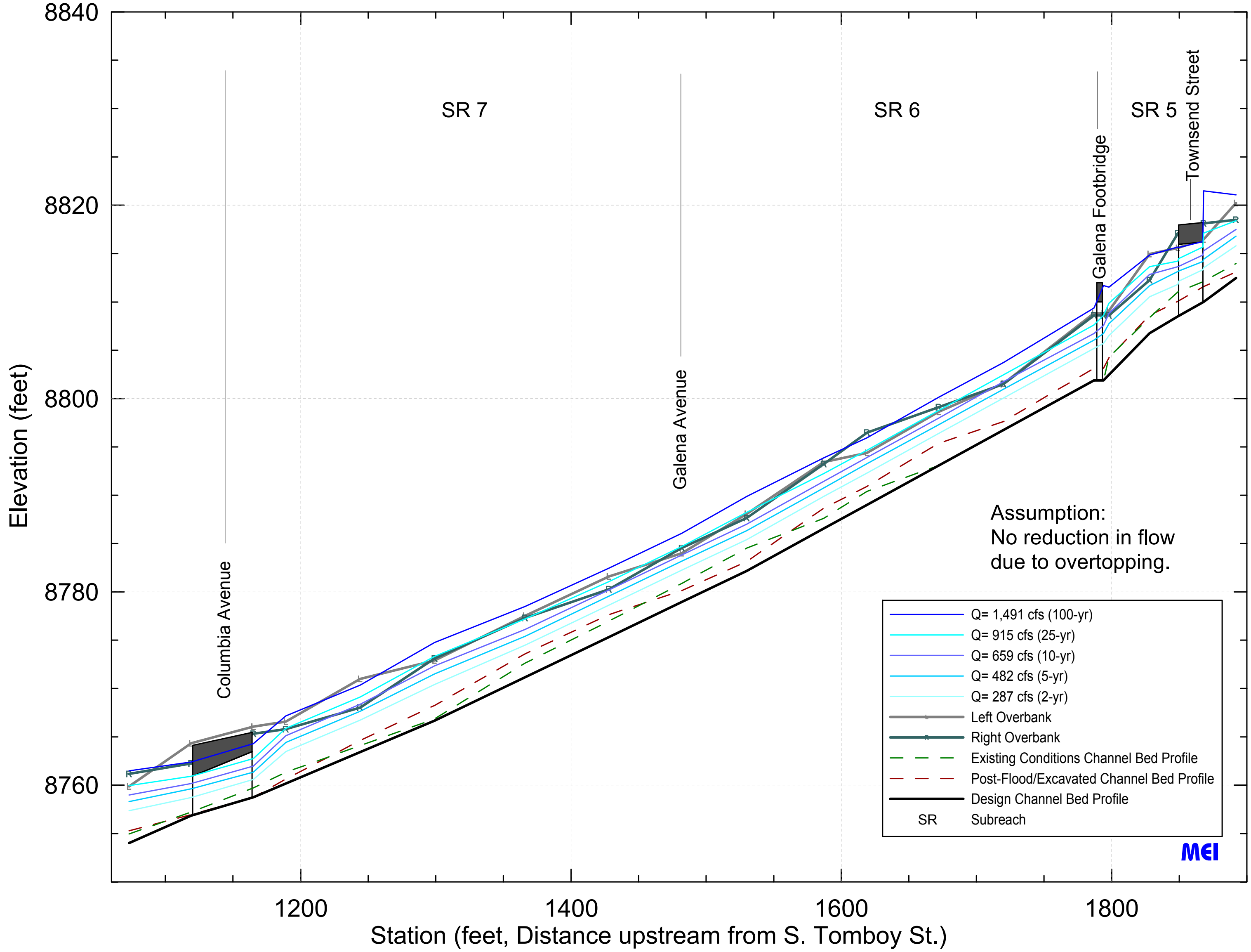

Figure 6.2. Design conditions channel bed and computed water-surface profiles for Subreaches 5 through 7 for a range of flows up to the 100-year peak discharge.

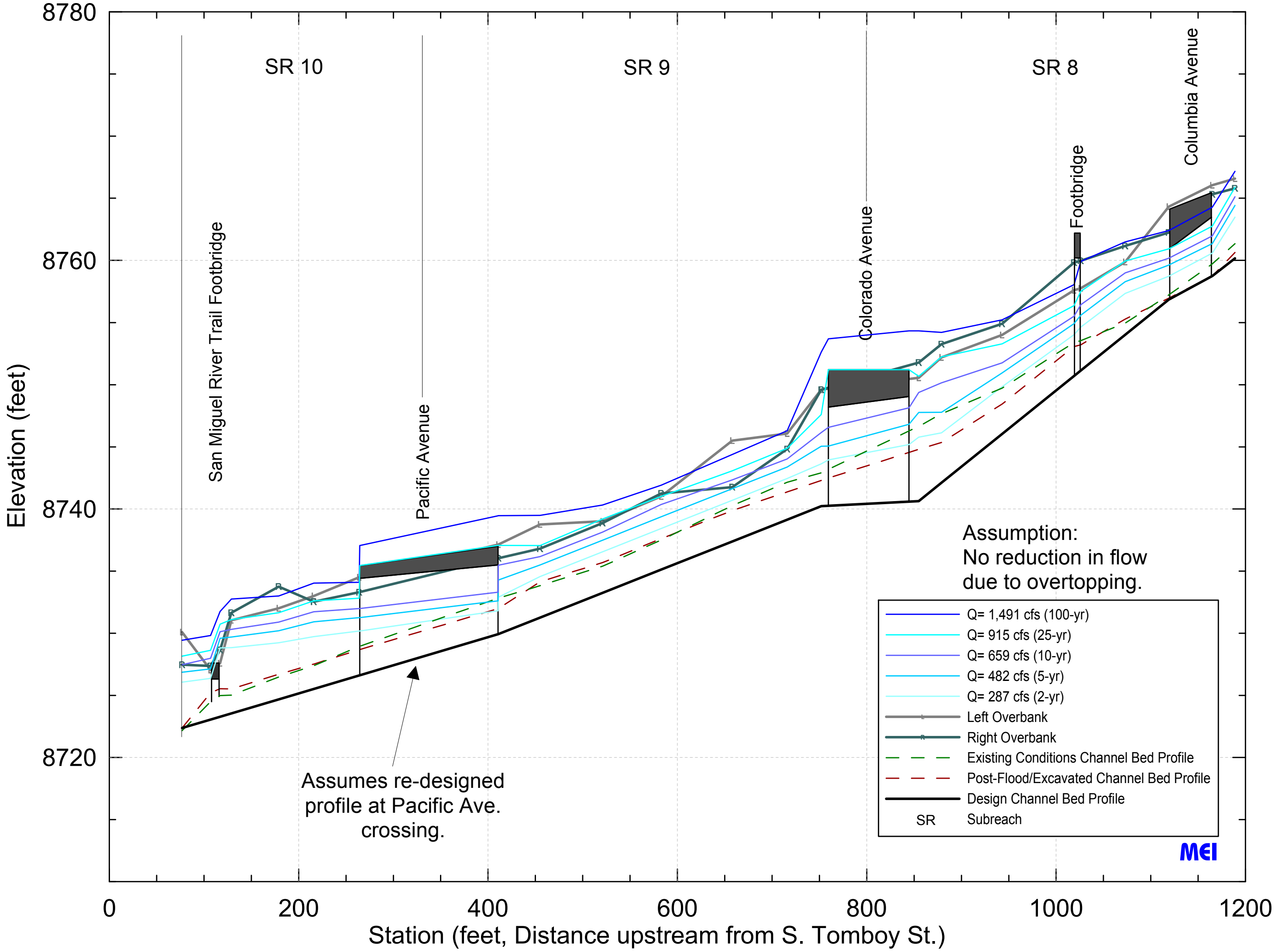

Figure 6.3. Design conditions channel bed and computed water-surface profiles for Subreaches 8 through 10 for a range of flows up to the 100-year peak discharge.

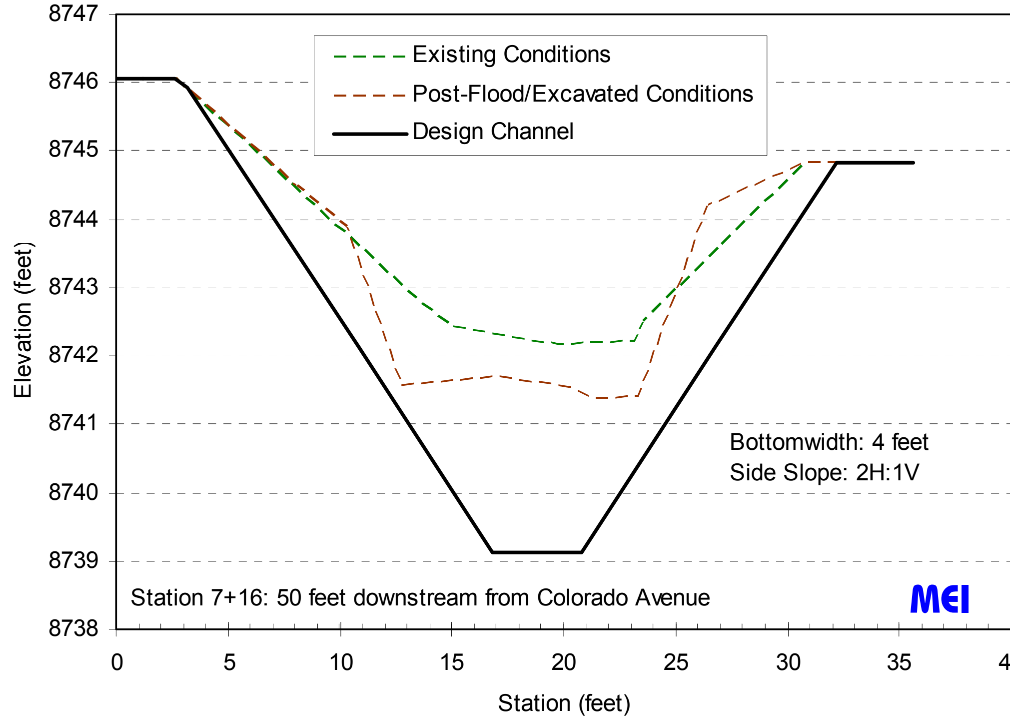

Figure 6.4. Cross-sectional schematic showing existing and post-flood/excavated conditions channel and channel design template at Sta 7+16.

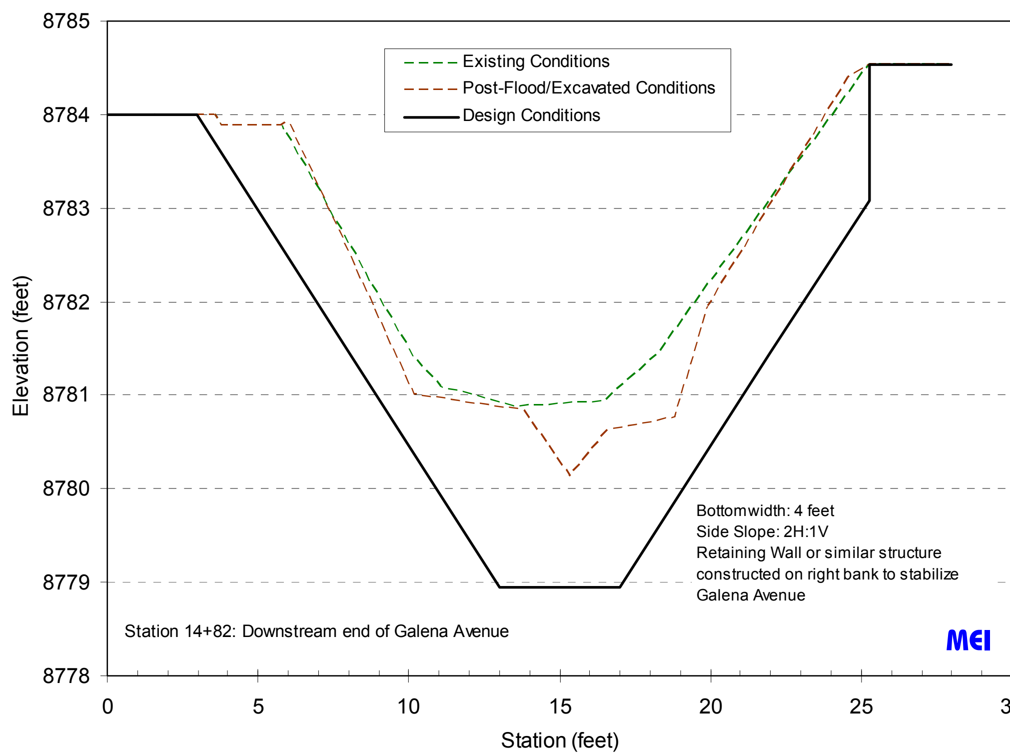

Figure 6.5. Cross-sectional schematic showing existing and post-flood/excavated conditions channel and channel design template at Sta 14+82.

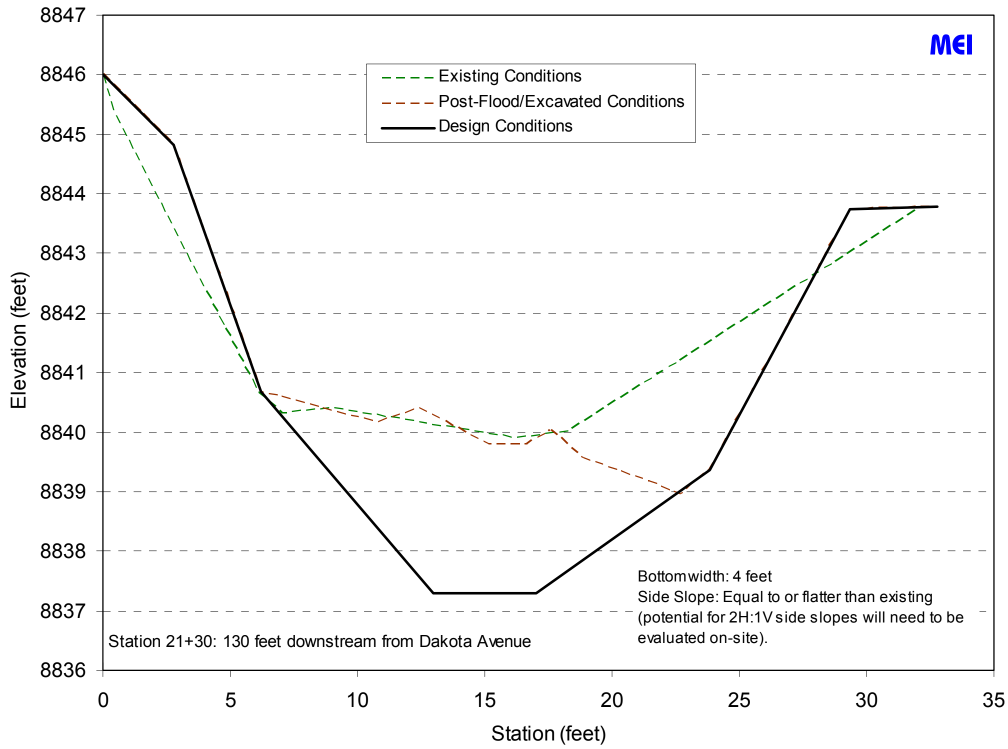

Figure 6.6. Cross-sectional schematic showing existing and post-flood/excavated conditions channel and channel design template at Sta 21+30.

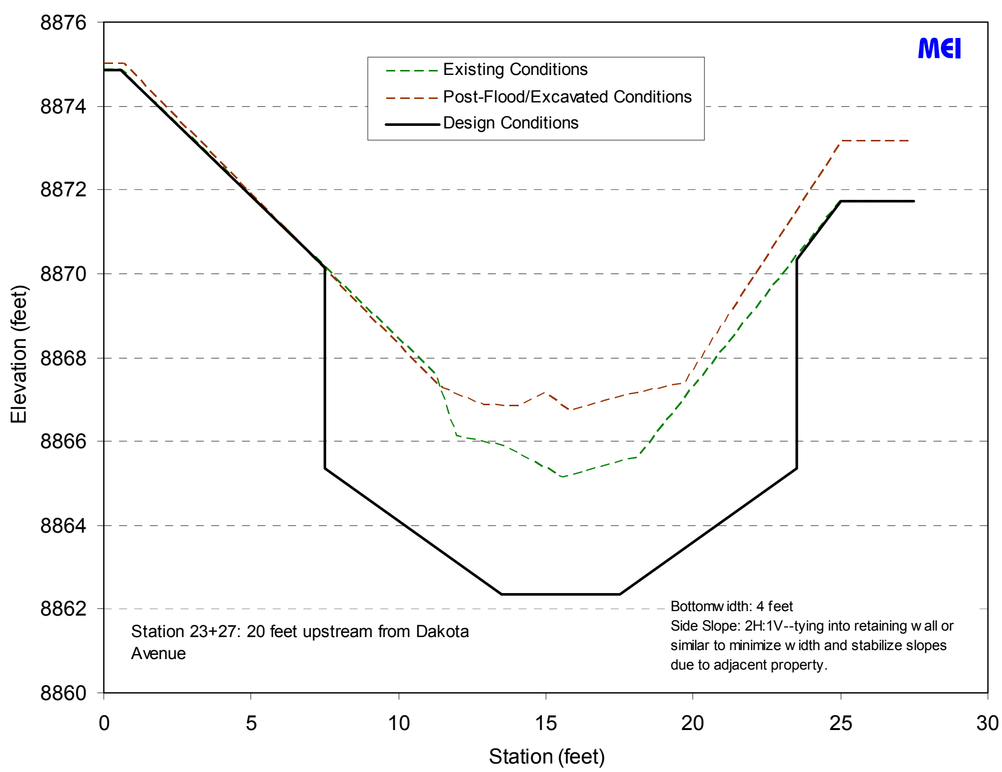

Figure 6.7. Cross-sectional schematic showing existing and post-flood/excavated conditions channel and channel design template at Sta 23+27.

Department during recent excavation activities in 2007. Utilizing the bank stability added by these structures to reduce channel widths may also be a potential variation to the design.

At the time of this report, Dakota Avenue is scheduled to be the next crossing that is replaced along Cornet Creek. To adequately improve the channel capacity in this location, the bed elevation needs to be lowered by about 3 feet (Figure 6.1), and widened to the recommended minimum of 4 feet. Due to the limited available width in this area, it is likely that some form of bank stabilization providing steeper sideslopes will be required (Figure 6.7).

6.1.1. Hydraulic Impact of Channel Improvements

A HEC-RAS hydraulic model of the improved channel was developed to evaluate impacts of the recommendations. Based on the hydraulic model results, the design should increase channel capacity by fully containing flows up to at least the 5-year peak discharge, and in many cases, up to the 10-year peak as well (not accounting for potential aggradation associated with these flow events) (Figures 6.1 through 6.3). In general, the combination of reducing flow losses due to overtopping and narrowing of the channel at low discharges results in slight increases in channel velocities and hydraulic depths. Increases in reach-averaged velocities range from less than 0.1 fps at the 2-year event to about 3 fps at the higher recurrence interval flows (Table 4.1 and **Table 6.1**). Estimated hydraulic depths under the improved design conditions are shown to increase by almost 2 feet during the 100-year event.

6.1.2. Sediment-Transport Impact of Improved Channel Design

An evaluation of the effects of the improved channel design on sediment transport was also carried out. In general, the analysis indicated that sediment-transport capacities are more uniform up to the 5-year event from upstream to downstream, and that the increased hydraulic capacity of the channel allows the associated sediment-transport capacity to continue to rise during larger flood events (at least until flow begins to exceed the channel capacity) (**Figure 6.8**). The sediment-continuity analysis indicated that up to the 5-year event the channel is mildly aggradational in all of the subreaches except Subreaches 3 and 7, where it is mildly degradational (**Figure 6.9**). At the higher magnitude, lower frequency events, the proposed channel improvements do not significantly affect the existing aggradational/degradational patterns along Cornet Creek.

6.1.3. Summary

Based on the hydraulic and sediment-transport results discussed above, the recommended channel improvements will increase channel capacity within the confines of the available channel corridor without negatively impacting the sediment-transport behavior or stability of Cornet Creek. Because of the very limited capacity of the existing channel, any measures taken to improve channel capacity along the creek will require a considerable amount of excavation. Assuming that the channel can be excavated to the recommended elevations, with a bottom width of 4 feet, and stable 2H:1V sideslopes wherever corridor widths allow, the estimated volume of material that would need to be excavated is approximately 3,800 cubic yards. A table summarizing the design channel bed elevations is provided in **Appendix B**. In addition to the initial excavation phase to modify the dimensions and profile of the existing channel, annual maintenance will be required to maintain the desired configuration. Based on the average annual sediment-transport capacities and sediment-continuity results (Figures 6.8 and 6.9), annual maintenance of the channel could potentially require the removal of up to an estimated 300 cubic yards of material per year.

Table 6.1. Summary of design conditions reach-averaged hydraulic conditions for Cornet Creek.

Subreach	Profile (yr)	Total Discharge (cfs)	Main Channel Discharge (cfs)	Main Channel Velocity (ft/s)	Hydraulic Depth (ft)	Effective Width (ft)	Energy Slope (ft/ft)	Approximate Location
1	2	287	287	8.27	2.05	17.0	0.0456	Upper end of study reach to Dakota Ave.
	5	482	482	9.48	2.72	18.7	0.0445	
	10	659	659	10.20	3.14	20.6	0.0465	
	25	915	915	11.09	3.65	22.6	0.0467	
	50	1176	1176	11.74	4.05	24.8	0.0478	
	100	1491	1491	12.52	4.49	26.5	0.0489	
2	2	287	287	8.34	2.16	16.0	0.0359	Dakota Ave. to 100 feet downstream (steep section below culvert outlet).
	5	482	482	9.69	2.84	17.5	0.0372	
	10	659	659	10.49	3.37	18.6	0.0364	
	25	915	911	11.21	3.93	20.8	0.0362	
	50	1176	1168	11.99	4.43	22.1	0.0371	
	100	1491	1478	12.69	4.96	23.6	0.0368	
3	2	287	287	8.15	2.03	17.3	0.0215	100 feet to 150 feet downstream from Dakota Ave (short depositional area).
	5	482	482	9.22	2.59	20.2	0.0217	
	10	652	648	9.65	3.11	21.7	0.0204	
	25	845	828	10.28	3.59	22.8	0.0204	
	50	1031	999	10.86	3.97	23.7	0.0207	
	100	1239	1189	11.46	4.35	24.6	0.0212	
4	2	287	287	8.33	2.11	16.3	0.0340	150 feet downstream from Dakota Ave. to Townsend Street.
	5	482	482	9.37	2.66	19.4	0.0408	
	10	652	652	9.96	3.04	21.5	0.0429	
	25	840	840	10.53	3.41	23.4	0.0444	
	50	1010	1008	11.00	3.77	24.4	0.0440	
	100	1166	1161	11.41	4.07	25.1	0.0440	
5	2	287	287	9.29	2.63	11.8	0.0290	Townsend Street to Galena Footbridge
	5	482	482	10.44	3.31	13.9	0.0290	
	10	645	633	10.35	4.10	14.9	0.0226	
	25	806	764	10.78	4.66	15.3	0.0211	
	50	934	867	11.37	4.96	15.5	0.0216	
	100	1016	932	11.69	5.15	15.7	0.0221	
6	2	287	287	8.14	2.04	17.3	0.0233	Galena Footbridge to Galena Ave.
	5	482	482	9.14	2.53	20.8	0.0327	
	10	652	650	9.62	2.97	22.8	0.0343	
	25	840	833	10.26	3.37	24.3	0.0361	
	50	981	970	10.71	3.68	24.9	0.0364	
	100	1078	1064	10.93	3.89	25.3	0.0360	
7	2	287	287	8.28	2.10	16.5	0.0280	Galena Ave. to Columbia Ave.
	5	482	482	9.35	2.64	19.5	0.0361	
	10	650	650	9.98	3.05	21.4	0.0394	
	25	804	801	10.23	3.41	23.0	0.0388	
	50	854	849	10.33	3.55	23.3	0.0381	
	100	880	874	10.45	3.60	23.3	0.0383	
8	2	287	287	8.18	2.18	16.1	0.0198	Columbia Ave. to Colorado Ave.
	5	482	482	9.30	2.78	18.7	0.0230	
	10	645	645	10.08	3.26	19.6	0.0241	
	25	767	767	10.48	3.56	20.6	0.0246	
	50	849	849	10.54	3.75	21.5	0.0248	
	100	918	918	10.61	3.92	22.1	0.0242	
9	2	287	287	8.17	2.04	17.2	0.0170	Colorado Ave. to Pacific Ave.
	5	482	482	9.09	2.53	21.0	0.0195	
	10	631	626	9.24	2.94	23.2	0.0180	
	25	724	712	9.58	3.14	24.0	0.0184	
	50	781	765	9.70	3.27	24.6	0.0181	
	100	822	803	9.72	3.37	25.1	0.0179	
10	2	287	286	4.66	2.69	22.9	0.0118	Pacific Ave. to San Miguel River
	5	482	475	5.47	3.31	26.6	0.0143	
	10	626	614	6.01	3.68	28.3	0.0159	
	25	707	691	6.25	3.89	29.0	0.0162	
	50	755	736	6.40	4.00	29.3	0.0164	
	100	787	766	6.50	4.08	29.5	0.0167	

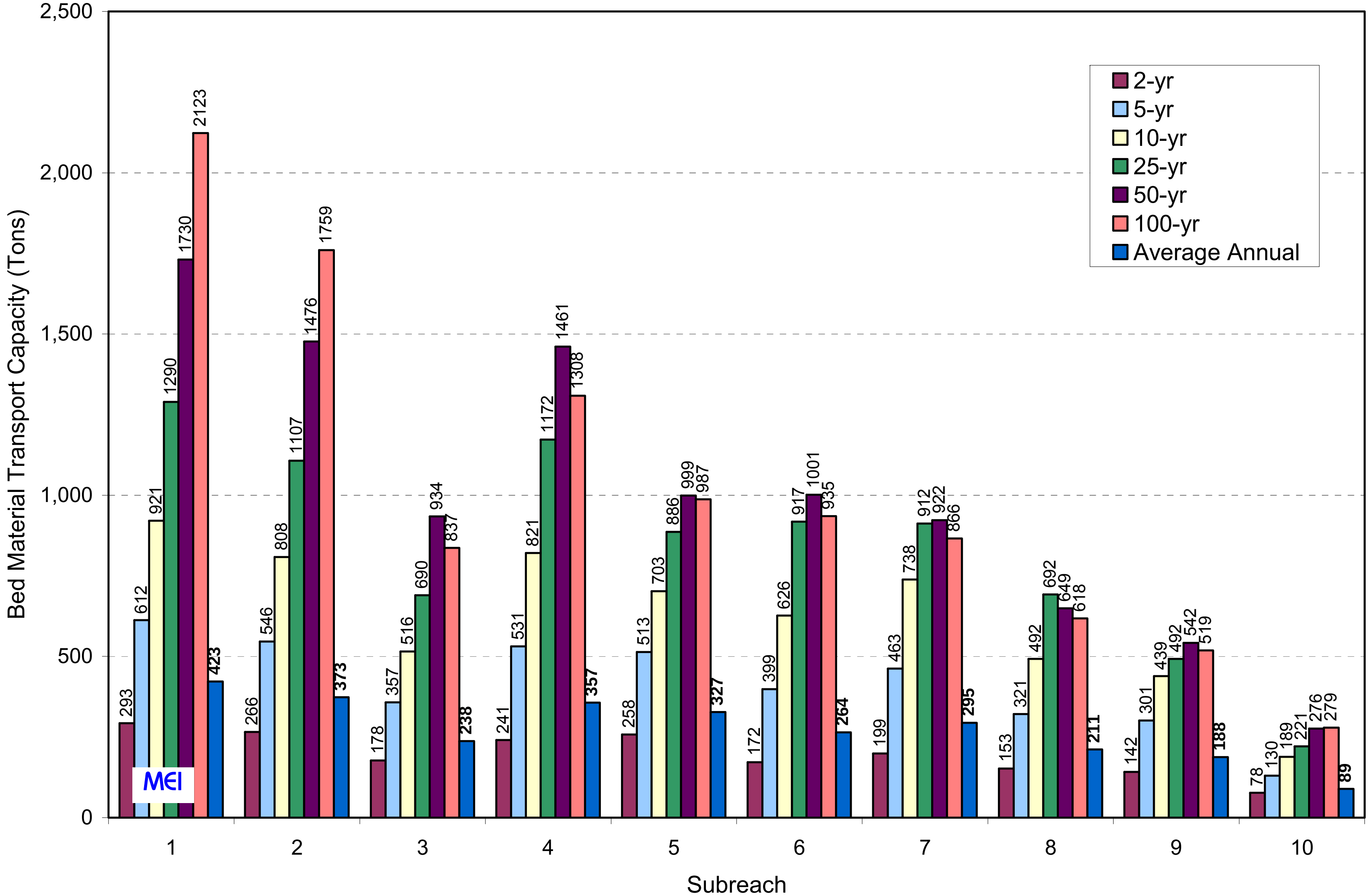

Figure 6.8. Design conditions reach-averaged bed-material transport capacities for Subreaches 1 through 10 of the project reach of Cornet Creek.

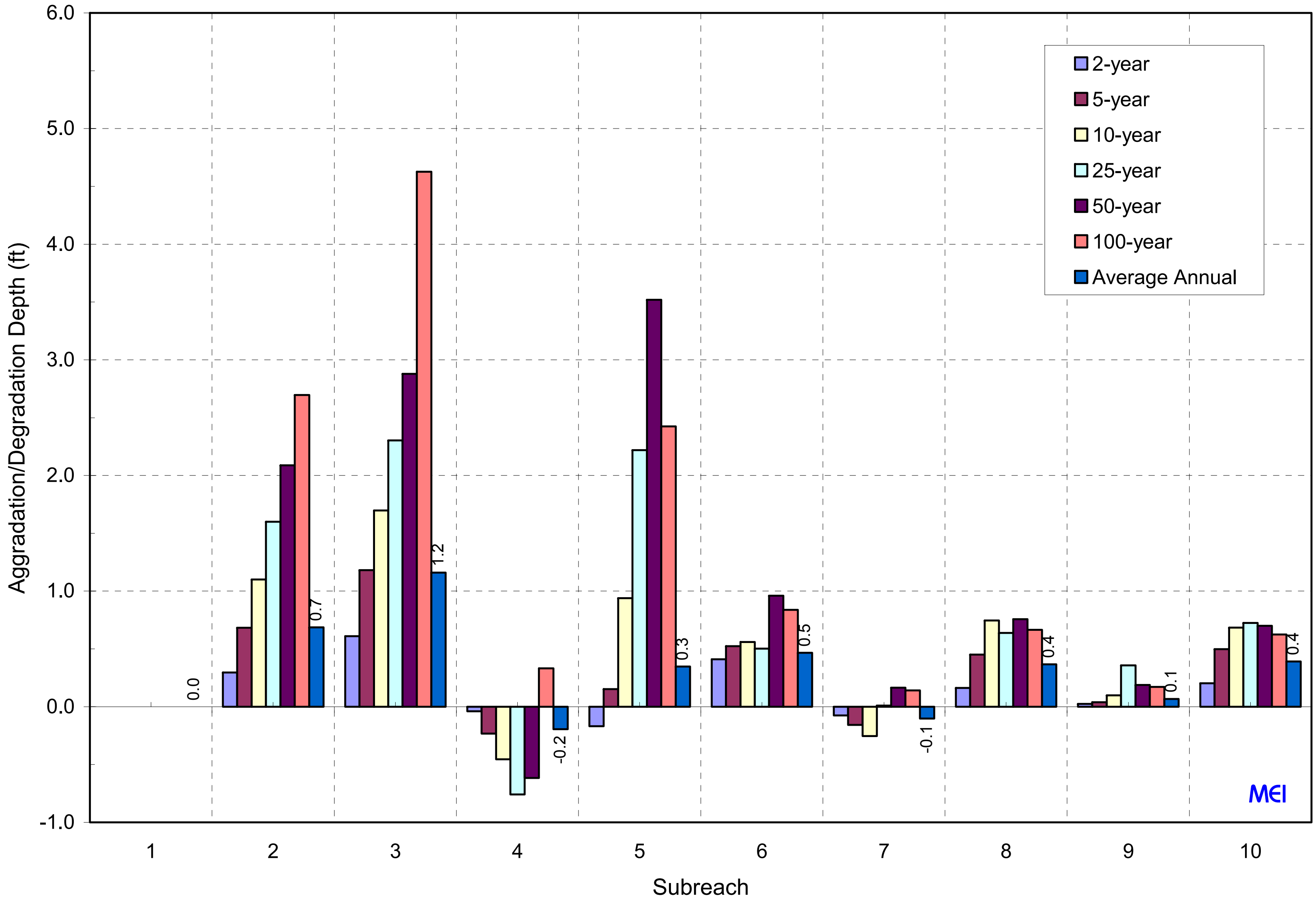

Figure 6.9. Design conditions reach-averaged aggradation/degradation depths for Subreaches 1 through 10 of the project reach of Cornet Creek.

MEI

6.2. Debris Flow Prediction and Mitigation

Prediction of, and mitigation for, debris flows are dependent to a great extent on the ability to identify sediment source areas, triggering causes (hydro-meteorological), in-channel or channel-margin sediment sources that cause bulking of floods to develop mud and debris flows, run-out geometries on the downstream alluvial fans and event frequencies. Managing the risks of these natural hazards can include land use planning, installation of preventative measures, stabilization of slopes, implementation of early warning systems, installation of protective structures and development of measures and procedures to restore normal conditions after the event (Greminger, 2003).

Risk analysis is based on the best scientific evidence available as well as a realistic appraisal of the physical conditions in the watershed (Petrascheck and Kienholz, 2003). For example, if it were assumed that the 1914 and 1969 debris flows were the result of failure of the Liberty Bell Mine tailings, then it could be expected that with time the level of risk would diminish as the tailings were eroded. However, if the source of the debris flows is in fact the glacial till deposits within the basin, then the risk would not diminish over human time scales. Based on the historic evidence, there have been two large debris-flow events on Cornet Creek in the last 93 years (1914 and 1969) which were both triggered by similar hydro-meteorological events (Mears et al., 1974), and therefore, the annual probability of occurrence of a similar type of event, or the risk, is about 2 percent (1 in 50). The frequency of debris flows in Cornet Creek is very similar to that reported for the Colorado Plateau (Webb et al., 1987) and the European Alps (Stefanini and Ribolini, 2003) where there are similar trigger mechanisms.

Individual perception of risk, however, tends to be based on personal experience as well as communicated information, but given the infrequency of occurrence of debris-flow events on Cornet Creek (the last one in 1969 was nearly 40 years ago), there tends to be a low general level of risk perception for debris flows. Consequently, since 1969 there has been a significant increase in development on the Cornet Creek fan and the channel has been further encroached by buildings, houses and road crossings. Risk evaluation is the process used to differentiate between benefits and damages in the context of a risk (Greminger, 2003). Because the magnitude of the bulked flows during historic debris flows (9,000 to 14,000 cfs) have so greatly exceeded the capacity of the Cornet Creek channel, even if the recommended improvements are implemented (about 500 cfs), it is apparent that there is a very high risk of damages on the fan if a similar magnitude event were to occur in the future. Put in another way, with the existing constraints and encroachments it would not be possible to construct a channel that would safely convey a 14,000-cfs debris flow across the Cornet Creek fan (benefit) as recommended by Mears et al. (1974), so therefore, damages are likely to be high on the fan.

Risk management basically refers to the sum of measures that can be instituted to reduce, control and regulate risk (Greminger, 2003). In the context of the Cornet Creek alluvial fan, since it is not possible to convey the debris flow across the fan in a constructed channel (e.g., the debris conveyance flumes in the Town of Ouray), the only other measures available are to reduce the volume of the debris flow by trapping a portion of it upstream of the fan, or to provide an early-warning system that would reduce the risk to persons, but would not reduce damages to structures.

A debris basin and an associated dike across the eastern portion of the fan apex were most probably constructed by the USACE up-fan from the terminus of Aspen Street following the 1969 debris flow (Dibble & Associates, 1983). Dibble & Associates (1983) and ARIX (1985) both recommended renovation of the debris basin and construction of a debris dam to reduce the sediment delivery to the fan. Numerous types of debris-trapping structures have been used in regions of the world where debris flows are a problem (Romang et al., 2003), but concrete check dams have been the most commonly used types of structure (Wu and Chang, 2003). Solid check dams have no ability to selectively retain debris and generally fill up with sediment, thus requiring maintenance removal to retain effectiveness. As a result, open- or slit-type check dams using beam, grid or column structures have been constructed more recently to allow some sediment bypass (**Plate 6.1**) (Fiebiger, 1997; VanDine et al., 1997). In general, any open-type check dam should serve one or both of two purposes: debris-flow breaking and debris-flow retention (Wu and Chang, 2003). A functional debris-flow breaker should separate solid debris from the transporting fluid, whereas the debris-flow retention function should selectively retain harmful debris and allow the finer sediment to return to the river. Physical model and field prototype results in Taiwan have shown that crossing-truss dams that are composed of two rows of overlapping triangular trusses with suitable spacing within the impact row serve as debris-flow breakers and the spacing produced by the overlapping of impact and outlet rows creates solid-fluid separation (**Figure 6.10**) (Wu and Chang, 2003). Access and volume constraints at the head of the Cornet Creek fan are likely to make it very difficult to construct any types of debris-flow check structures.

Early-warning systems for debris-flow generation are generally based on an analysis of antecedent meteorological conditions that include both precipitation and temperature as well as real-time monitoring of precipitation event intensity. Such systems require a relatively dense network of automated weather stations and have not been shown to be very reliable for debris-flow prediction, but they have been used successfully in Japan to develop evacuation plans (Onda et al., 2003). Improvements in the ability to predict debris-flow hazards in mountainous regions where there are few ground-based rainfall measurement stations have been made with remote sensing of rainfall using Doppler radar (NEXRAD) and NOAA satellite-based infrared imagery (Wieczorek et al., 2003). However, in many steep mountainous areas, debris-flow velocities are so high (~30 ft/s) that early warnings of debris flow triggering may not provide sufficient warning time to downstream communities (Liu and Chen, 2003). Real-time warning devices consisting of trip wire sensors, infrared photo beams, acoustic sensors and ground-vibration sensors have been used successfully to identify in-progress debris-flow events and their velocities and composition, but because of the velocity of the debris flows, they tend to provide little advance warning time for evacuation of impacted areas (Arrattano, 2003; Chang, 2003). They can, however, be used to provide warnings at bridges and other structures that could be impacted.

Chang (2003) categorized debris-flow warning systems into: (1) prior-warning systems, and (2) real-time warning systems. Prior-warning systems are generally based on the characteristics of the rainfall which may trigger debris flows. The primary advantage of these systems is that they provide sufficient time for dealing with the threat, and they are most applicable to residential areas where there is a high potential for loss of life. The primary disadvantage of the prior-warning systems is that the forecast accuracy is usually poor, and thus there is a high potential for false alarms. Real-time warning systems, on the other hand, tend to have a much higher accuracy, but they generally provide a much shorter reaction time. The greatest value of the real-time systems is that they can be automatically linked to lights and barriers at transportation crossings, thereby lessening the risk of pedestrian, rail or automobile casualties.

In summary, the historic debris flows (1914, 1969) on Cornet Creek upstream of the fan apex were estimated to have velocities up to 30 ft/s (Mears et al., 1974). If it is assumed that the source of a debris flow was in the vicinity of the Liberty Bell Mine, a channel distance of about 10,000 feet upstream of the fan apex, a real-time early warning system would provide an alert time of about 11 minutes if the average velocity of the debris flow was 15 ft/s. At higher or lower assumed velocities, the alert times would be correspondingly shorter or longer. Even under an assumption of a lower average velocity, it is unlikely that the system would provide sufficient lead time to evacuate people from the fan. The historic debris flows have had bulked peak flows on the order of 10,000 cfs at the fan apex (Mears et al., 1974). If it assumed that the peak flows lasted for 5 minutes, and the peak flow contained 40-percent solids, the volume of material that would have to be trapped at the fan apex would very conservatively be on the order of 45,000 cubic yards (27.5 ac-ft). Given the space constraints in the lower part of the canyon, it is highly unlikely that a debris trapping structure could be built that would detain the debris-flow volume. However, a crossing-truss dam structure (Wu and Chang, 2003) might be able to detain enough of the coarse material to reduce the downstream damages. Further research into the performance and costs of this type of structure is required.

Plate 6.1. View looking upstream at self-cleaning concrete debris flow check dam on the Schwarzach River, near Huben, Austria.

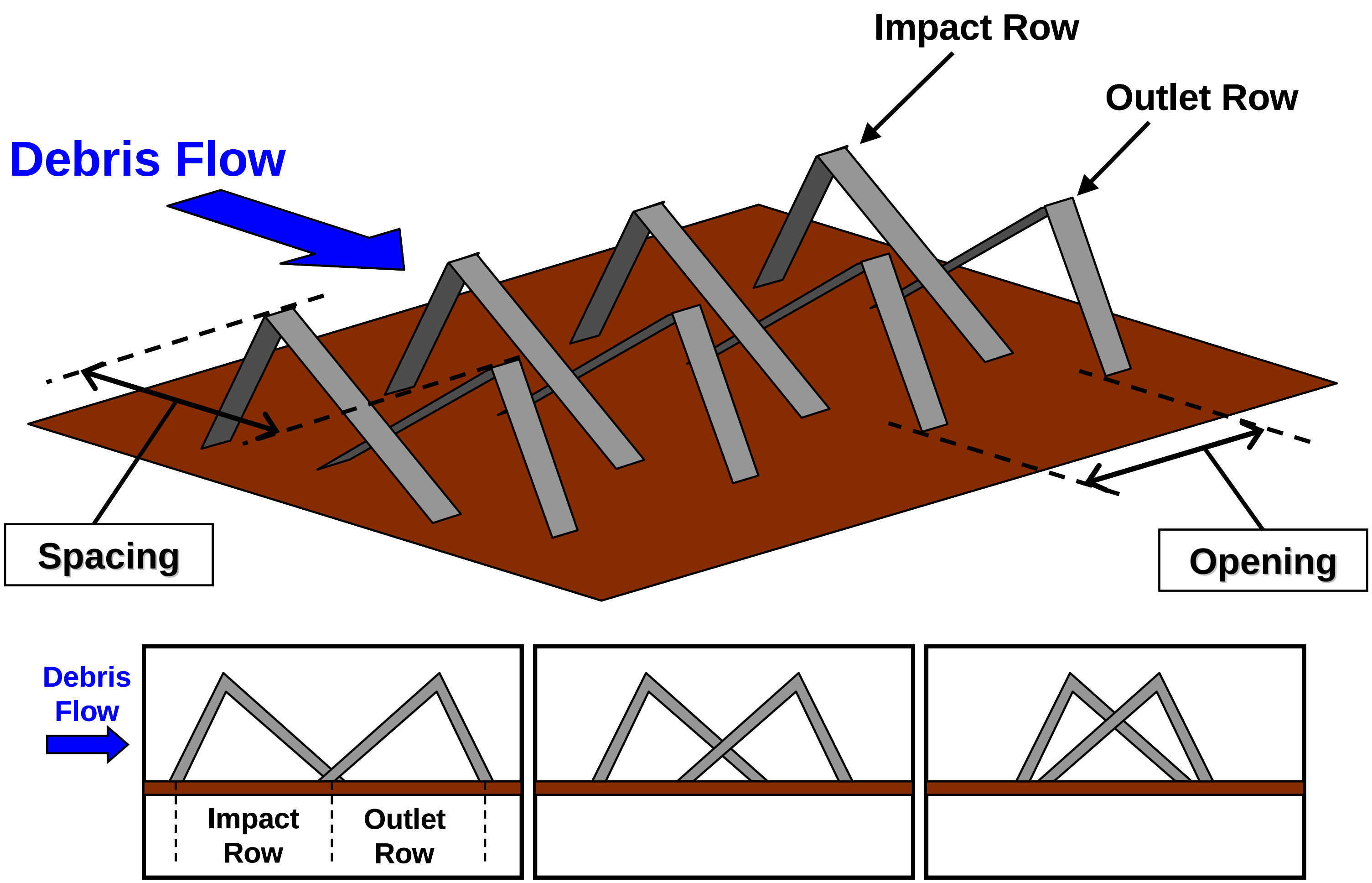

Figure 6.10. Schematic example of crossing-truss dam configuration (modified from Wu and Chang, 2003).

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APPENDIX A

Cornet Creek Study Site Inventory Map

APPENDIX B

Summary of Design Channel Bed Elevations, Cornet Creek

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
49928.0	48137.8	77	8722.37	San Miguel River	X
49928.3	48140.9	80	8722.44		
49930.8	48150.5	90	8722.66		
49935.3	48159.4	100	8722.89		
49938.9	48165.4	107	8723.05	San Miguel River Trail Footbridge	
49940.4	48168.0	110	8723.12	San Miguel River Trail Footbridge	
49943.8	48174.1	117	8723.27	San Miguel River Trail Footbridge	X
49944.9	48177.0	120	8723.34		
49948.4	48185.2	129	8723.55		X
49948.9	48186.1	130	8723.57		
49954.1	48194.7	140	8723.80		
49960.2	48202.6	150	8724.02		
49967.0	48209.9	160	8724.25		
49974.3	48216.6	170	8724.48		
49981.0	48222.7	179	8724.68		X
49981.7	48223.4	180	8724.70		
49989.0	48230.2	190	8724.93		
49996.1	48237.3	200	8725.15		
50003.0	48244.6	210	8725.38		
50006.7	48249.2	216	8725.51		X
50008.6	48252.8	220	8725.60		
50012.1	48262.1	230	8725.83		
50014.8	48271.7	240	8726.06		
50017.1	48281.5	250	8726.28		
50020.6	48290.8	260	8726.51		
50022.4	48294.4	264	8726.60	Pacific Avenue Culverts	X
50025.7	48299.4	270	8726.74	Pacific Avenue Culverts	
50032.3	48306.9	280	8726.96	Pacific Avenue Culverts	
50039.9	48313.4	290	8727.19	Pacific Avenue Culverts	
50048.3	48318.8	300	8727.41	Pacific Avenue Culverts	
50057.3	48323.2	310	8727.64	Pacific Avenue Culverts	
50066.6	48326.9	320	8727.86	Pacific Avenue Culverts	
50076.0	48330.3	330	8728.09	Pacific Avenue Culverts	
50085.4	48333.5	340	8728.32	Pacific Avenue Culverts	
50095.0	48336.5	350	8728.54	Pacific Avenue Culverts	
50104.6	48339.4	360	8728.77	Pacific Avenue Culverts	
50114.2	48342.1	370	8728.99	Pacific Avenue Culverts	
50123.9	48344.8	380	8729.22	Pacific Avenue Culverts	
50133.5	48347.4	390	8729.45	Pacific Avenue Culverts	
50143.1	48350.3	400	8729.67	Pacific Avenue Culverts	
50152.4	48353.9	410	8729.90	Pacific Avenue Culverts	
50153.3	48354.3	411	8729.92	Pacific Avenue Culverts	X
50161.0	48358.9	420	8730.19		
50168.3	48365.7	430	8730.49		
50175.3	48372.8	440	8730.80		
50182.9	48379.4	450	8731.10		

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
50187.0	48382.3	455	8731.25		X
50191.0	48385.2	460	8731.40		
50199.3	48390.8	470	8731.70		
50207.6	48396.4	480	8732.00		
50216.0	48401.8	490	8732.31		
50224.5	48407.2	500	8732.61		
50232.9	48412.5	510	8732.91		
50241.4	48417.7	520	8733.21		
50242.3	48418.3	521	8733.24		X
50249.9	48422.9	530	8733.51		
50258.5	48428.1	540	8733.82		
50267.1	48433.3	550	8734.12		
50275.6	48438.4	560	8734.42		
50284.2	48443.6	570	8734.73		
50292.7	48448.9	580	8735.03		
50295.2	48450.5	583	8735.12		X
50300.8	48454.7	590	8735.33		
50308.4	48461.3	600	8735.63		
50315.4	48468.3	610	8735.93		
50322.2	48475.7	620	8736.23		
50328.9	48483.2	630	8736.54		
50335.4	48490.7	640	8736.84		
50341.8	48498.3	650	8737.14		
50346.9	48504.5	658	8737.38		X
50348.2	48506.1	660	8737.44		
50354.2	48514.0	670	8737.74		
50360.1	48522.2	680	8738.04		
50365.9	48530.4	690	8738.35		
50371.6	48538.5	700	8738.65		
50377.3	48546.7	710	8738.95		
50381.0	48551.5	716	8739.13		X
50383.7	48554.5	720	8739.25		
50391.1	48561.1	730	8739.55		
50399.0	48567.4	740	8739.86		
50406.9	48573.3	750	8740.16		
50408.5	48574.5	752	8740.22	Colorado Avenue Culvert	X
50415.0	48579.3	760	8740.25	Colorado Avenue Culvert	
50422.9	48585.3	770	8740.29	Colorado Avenue Culvert	
50431.0	48591.2	780	8740.33	Colorado Avenue Culvert	
50439.1	48597.2	790	8740.37	Colorado Avenue Culvert	
50447.1	48603.1	800	8740.41	Colorado Avenue Culvert	
50455.2	48609.1	810	8740.45	Colorado Avenue Culvert	
50463.2	48615.0	820	8740.49	Colorado Avenue Culvert	
50471.3	48620.9	830	8740.53	Colorado Avenue Culvert	
50479.3	48626.9	840	8740.57	Colorado Avenue Culvert	
50487.3	48632.8	850	8740.61	Colorado Avenue Culvert	

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
50491.5	48635.5	855	8740.63	Colorado Avenue Culvert	X
50496.2	48637.4	860	8740.94		
50505.8	48640.2	870	8741.55		
50514.4	48642.7	879	8742.10		X
50515.4	48642.9	880	8742.16		
50525.0	48645.5	890	8742.78		
50534.7	48648.1	900	8743.39		
50544.4	48650.8	910	8744.01		
50554.0	48653.5	920	8744.62		
50563.6	48656.3	930	8745.24		
50573.1	48659.3	940	8745.86		
50575.9	48660.2	943	8746.04		X
50582.6	48662.5	950	8746.47		
50591.9	48666.1	960	8747.08		
50601.0	48670.2	970	8747.70		
50609.9	48674.8	980	8748.31		
50618.5	48679.9	990	8748.93		
50626.7	48685.6	1000	8749.54		
50634.2	48692.3	1010	8750.16		
50640.2	48698.9	1019	8750.71	Footbridge	X
50640.9	48699.7	1020	8750.77	Footbridge	
50644.9	48704.2	1026	8751.14	Footbridge	
50647.5	48707.2	1030	8751.39		
50654.6	48714.3	1040	8752.00		
50662.0	48721.0	1050	8752.61		
50669.5	48727.5	1060	8753.22		
50677.1	48734.0	1070	8753.84		
50679.4	48736.0	1073	8754.02		X
50684.6	48740.7	1080	8754.45		
50691.3	48748.1	1090	8755.07		
50696.6	48756.6	1100	8755.68		
50701.1	48765.5	1110	8756.30		
50705.3	48773.4	1119	8756.85	Columbia Avenue Bridge	X
50705.9	48774.2	1120	8756.89	Columbia Avenue Bridge	
50712.2	48782.0	1130	8757.30	Columbia Avenue Bridge	
50718.7	48789.6	1140	8757.72	Columbia Avenue Bridge	
50725.2	48797.2	1150	8758.13	Columbia Avenue Bridge	
50731.6	48804.9	1160	8758.54	Columbia Avenue Bridge	
50734.8	48808.8	1165	8758.75	Columbia Avenue Bridge	X
50737.9	48812.6	1170	8759.05		
50744.3	48820.4	1180	8759.64		
50750.0	48827.3	1189	8760.17		
50750.6	48828.1	1190	8760.23		
50756.9	48835.8	1200	8760.82		
50763.4	48843.4	1210	8761.41		
50770.1	48850.9	1220	8762.01		

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
50776.9	48858.2	1230	8762.60		
50783.8	48865.5	1240	8763.19		
50786.6	48868.4	1244	8763.43		X
50790.9	48872.6	1250	8763.78		
50798.1	48879.4	1260	8764.38		
50805.4	48886.3	1270	8764.97		
50812.7	48893.1	1280	8765.56		
50819.9	48900.0	1290	8766.15		
50826.5	48906.1	1299	8766.68		X
50827.2	48906.8	1300	8766.75		
50834.6	48913.6	1310	8767.42		
50841.9	48920.5	1320	8768.09		
50849.0	48927.5	1330	8768.76		
50856.1	48934.5	1340	8769.43		
50863.2	48941.6	1350	8770.10		
50870.1	48948.8	1360	8770.77		
50874.3	48953.1	1366	8771.17		X
50877.0	48956.1	1370	8771.44		
50883.8	48963.3	1380	8772.11		
50890.7	48970.6	1390	8772.78		
50897.8	48977.7	1400	8773.45		
50904.8	48984.8	1410	8774.12		
50911.8	48992.0	1420	8774.79		
50917.4	48997.7	1428	8775.33		X
50918.7	48999.2	1430	8775.46		
50924.6	49007.2	1440	8776.13		
50929.7	49015.9	1450	8776.80		
50934.4	49024.7	1460	8777.48		
50938.3	49033.9	1470	8778.15		
50941.0	49043.5	1480	8778.82		
50941.3	49045.5	1482	8778.95		X
50941.5	49053.5	1490	8779.49		
50940.6	49063.4	1500	8780.16		
50939.4	49073.4	1510	8780.83		
50938.1	49083.3	1520	8781.50		
50936.6	49093.2	1530	8782.17		
50936.6	49093.2	1530	8782.17		X
50935.3	49103.1	1540	8782.94		
50935.3	49113.0	1550	8783.70		
50937.2	49122.9	1560	8784.47		
50940.1	49132.4	1570	8785.24		
50943.3	49141.9	1580	8786.00		
50945.4	49148.6	1587	8786.54		X
50946.2	49151.5	1590	8786.77		
50947.8	49161.4	1600	8787.54		
50947.4	49171.3	1610	8788.31		

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
50945.1	49180.0	1619	8789.00		X
50944.7	49180.9	1620	8789.08		
50940.7	49190.1	1630	8789.84		
50936.0	49198.9	1640	8790.61		
50931.2	49207.8	1650	8791.38		
50926.9	49216.8	1660	8792.15		
50923.2	49226.0	1670	8792.92		
50922.5	49227.9	1672	8793.07		X
50920.1	49235.5	1680	8793.68		
50917.0	49245.0	1690	8794.45		
50914.0	49254.6	1700	8795.22		
50911.1	49264.1	1710	8795.98		
50908.2	49273.7	1720	8796.75		X
50908.2	49273.7	1720	8796.75		
50905.6	49283.4	1730	8797.52		
50903.0	49293.0	1740	8798.28		
50900.2	49302.7	1750	8799.05		
50897.5	49312.2	1760	8799.82		
50894.9	49321.9	1770	8800.59		
50892.9	49331.7	1780	8801.35		
50892.2	49338.7	1787	8801.89	Footbridge	X
50892.1	49341.6	1790	8801.89	Footbridge	
50892.2	49345.7	1794	8801.89	Footbridge	
50892.6	49349.6	1798	8802.46		
50893.0	49351.6	1800	8802.75		
50895.6	49361.2	1810	8804.18		
50899.1	49370.6	1820	8805.61		
50901.8	49378.2	1828	8806.75		X
50902.4	49380.0	1830	8806.92		
50905.7	49389.5	1840	8807.75		
50908.4	49398.1	1849	8808.50	Townsend Street Bridge	X
50908.6	49399.1	1850	8808.58	Townsend Street Bridge	
50911.1	49408.7	1860	8809.37	Townsend Street Bridge	
50913.0	49416.5	1868	8810.00	Townsend Street Bridge	X
50913.5	49418.5	1870	8810.21		
50915.8	49428.2	1880	8811.23		
50918.4	49437.9	1890	8812.26		
50919.0	49439.8	1892	8812.46		X
50921.5	49447.3	1900	8813.28		
50924.8	49456.8	1910	8814.31		
50927.9	49466.3	1920	8815.33		
50930.8	49475.9	1930	8816.36		
50932.3	49482.7	1937	8817.08		X
50932.8	49485.7	1940	8817.39		
50934.0	49495.6	1950	8818.41		
50934.6	49505.6	1960	8819.44		

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
50934.9	49515.6	1970	8820.47		
50935.2	49524.5	1979	8821.39		X
50935.3	49525.6	1980	8821.49		
50935.5	49535.5	1990	8822.52		
50936.0	49545.5	2000	8823.55		X
50936.0	49545.5	2000	8823.55		
50937.1	49555.5	2010	8824.58		
50938.6	49565.3	2020	8825.60		
50940.7	49575.1	2030	8826.63		
50943.7	49584.7	2040	8827.65		
50947.3	49594.0	2050	8828.68		
50949.3	49598.6	2055	8829.19		X
50951.4	49603.2	2060	8829.73		
50956.0	49612.0	2070	8830.82		
50961.7	49620.2	2080	8831.90		
50967.9	49628.0	2090	8832.98		
50968.5	49628.8	2091	8833.09		X
50973.3	49636.5	2100	8834.06		
50978.1	49645.3	2110	8835.14		
50982.6	49654.2	2120	8836.22		
50987.2	49663.1	2130	8837.30		X
50987.2	49663.1	2130	8837.30		
50991.9	49671.8	2140	8838.38		
50996.8	49680.6	2150	8839.47		
50998.2	49683.3	2153	8839.79		X
51001.5	49689.4	2160	8840.55		
51005.9	49698.4	2170	8841.63		
51009.9	49707.6	2180	8842.71		
51012.7	49715.1	2188	8843.58	Footbridge	X
51013.3	49716.9	2190	8843.84	Footbridge	
51016.4	49726.5	2200	8845.12		
51019.0	49736.2	2210	8846.41		
51021.5	49745.8	2220	8847.70		
51022.5	49749.7	2224	8848.21		X
51023.8	49755.5	2230	8848.98		
51026.3	49765.2	2240	8850.26		
51028.7	49774.9	2250	8851.55		
51030.0	49779.8	2255	8852.19	Dakota Avenue Culvert	X
51031.2	49784.6	2260	8852.83	Dakota Avenue Culvert	
51033.8	49794.3	2270	8854.12	Dakota Avenue Culvert	
51036.3	49804.0	2280	8855.40	Dakota Avenue Culvert	
51038.8	49813.6	2290	8856.69	Dakota Avenue Culvert	
51041.3	49823.3	2300	8857.97	Dakota Avenue Culvert	
51043.2	49831.1	2308	8859.00	Dakota Avenue Culvert	X
51043.7	49833.1	2310	8859.35		
51046.0	49842.8	2320	8861.11		

APPENDIX B. Summary of design channel bed elevations.					
Northing[1] (ft)	Easting[1] (ft)	Station[2] (ft)	Elevation[3] (ft)	Remarks	Displayed on Mapping (See Appendix C)
51047.5	49849.6	2327	8862.34		X
51048.2	49852.5	2330	8862.87		
51050.3	49862.3	2340	8864.63		
51053.5	49871.8	2350	8866.38		
51058.2	49880.6	2360	8868.14		
51060.6	49884.9	2365	8869.02	Footbridge	X
51063.1	49889.2	2370	8869.90	Footbridge	
51064.1	49891.0	2372	8870.25	Footbridge	
51068.2	49897.8	2380	8871.66		
51073.4	49906.4	2390	8873.41		
51078.6	49914.9	2400	8875.17		
51082.1	49921.0	2407	8876.40		X
51083.7	49923.6	2410	8876.93		
51088.3	49932.4	2420	8878.69		
51092.0	49941.7	2430	8880.44		
51095.4	49951.1	2440	8882.20		
51099.0	49960.5	2450	8883.96		
51103.5	49969.3	2460	8885.72		X
51103.5	49969.3	2460	8885.72		
51109.1	49977.6	2470	8887.48		
51115.7	49985.2	2480	8889.23		
51122.8	49992.0	2490	8890.99		
51130.7	49998.2	2500	8892.75		
51134.1	50000.5	2504	8893.45		X
51139.1	50003.8	2510	8895.16		
51147.6	50009.0	2520	8898.00		
51156.3	50014.1	2530	8900.85		
51164.8	50019.2	2540	8903.70		
51173.5	50024.2	2550	8906.54		
51182.3	50028.9	2560	8909.39		
51182.3	50028.9	2560	8909.39		X

[1]Reported coordinates are based on Town of Telluride local grid coordinate system
[2]Station represents feet upstream from South Tomboy Street (i.e., bridge to Carhenge)
[3]Reported elevations are based on Town of Telluride local survey datum

APPENDIX C

Cornet Creek Design Channel Bed Elevations Map