Title

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THE PERFECT STORM

FLOW THROUGH A RESTORED COMPOUND CHANNEL
TASSAJARA CREEK, DUBLIN, CA

ASSESSMENT OF THE ROUGHNESS, FLOW, FLOODPLAIN CONVEYANCE, AND COMPOUND CHANNEL CAPACITY OF THE RESTORATION OF TASSAJARA CREEK FROM THE HIGH-WATER MARKS OF A 20-YEAR STORM

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Abstract

In 1999, Alameda County completed the restoration of a 1-mile stretch of Tassajara Creek in Dublin, California. The project created a compound channel with a low-flow channel capacity of Q5 in the upper and middle reaches, Q2 in the lower reach, and a natural floodplain terrace along all reaches to accommodate the design-estimated 100-year flood of 5,200 cfs. Downstream of the restoration reach is a trapezoidal concrete channel. On December 30th and 31st of 2005, a 20-year storm with a cumulative rainfall of 3.56 inches passed over the Tassajara Creek watershed, generating flows that overtopped the low-flow banks of Tassajara Creek, providing an opportunity to assess flow capacity of the compound channel configuration. We conducted long profile and cross-section surveys along the entire restoration reach, and the first 100 feet of the concrete channel. Using the Manning Equation in a HEC-RAS steady flow model, we used the geometry of the concrete channel and the elevation of the high water marks to estimate the peak flow from the storm as 1,500 cfs. We then back-calculated the roughness coefficient (Manning's "n") for each compound channel cross-section by matching model water surface elevations to observed elevations. We also compared the design 2-year and 5-year low flow channel water surface elevations to modeled water surface elevations using our calibrated roughness values. Finally, we compared the calculated flow capacity to the original design estimations, and we determined that the compound channel successfully accommodates the 100-year flood, although at one cross section with only 1 foot of freeboard.

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Introduction

Alameda County completed the restoration of a 1-mile stretch of Tassajara Creek in Dublin, California in 1999. The stream drains a 23.2 square-mile (mi^2) basin east of the San Francisco Bay (Lave 2003). Tassajara Creek has long suffered erosion and incision as a result of increased runoff from cattle grazing, which began in the area in the 19th century (Oden and DeHollan 2004). The United States Navy had previously owned the site and lined the channel with concrete sometime before the mid 1960s, in an attempt to stabilize it, but the erosive forces were so powerful as to break up the concrete and deposit it on the floodplain (Lave 2003). When Alameda County acquired the property, they initiated plans to ensure flood control for the land surrounding Tassajara Creek, which they intended to develop for residential and commercial purposes (Hudzik and Truit 2001). The project designed for a natural compound channel floodplain to accommodate the estimated 100-year flood (Sycamore Associates, 1996), which contrasts markedly with the trapezoidal concrete channel immediately downstream. The restoration project had the additional goals of addressing incision, improving riparian habitat, enhancing the stream corridor's appearance, and providing a key link in Dublin's trail system (Lave 2003), with a long-term goal of linking with the Iron Horse Trail (East Bay Regional Park District 2000).

The project created a compound channel with a low-flow channel capacity of Q_5 in the upper and middle reaches, Q_2 in the lower reach, and a natural floodplain terrace along all reaches to accommodate the 100-year flood. In this paper, we calculate the roughness at six cross-sections using high water marks from a 20-year peak storm to determine whether the compound channel's design dimensions accommodate the estimated 100-year flood, i.e. the flood which has a 1% chance of occurring each year. We also assess whether the low flow channel contains the flows for which it was designed. We conclude with a discussion of the relevance of roughness and offer suggestions for future study.

Several others (Hudzik and Truitt 2001, Lave 2002, Lave 2003, Krofta and Novotney 2003, Oden and DeHollan 2004, Tompkins 2005) have conducted follow-up studies since the project's completion, with somewhat inconclusive or contradictory results regarding the existence or degree of incision along the channel. These studies acknowledge that the geomorphology and vegetative state of the channel is constantly evolving due to complementary natural processes, including high flows, bank erosion, aggradation, channel scouring and incision. However, they do not incorporate these natural factors into an overall assessment of the effectiveness of the restoration project in meeting the original design guidelines for channel capacity and incision mitigation. Krofta and Novotney (2003) note: "it is also possible that Tassajara Creek has not experienced a rainfall event large enough to produce bed shear forces great enough to create significant incision during the past few years." Lave (2003) concurs: "there had not yet been a year with substantial run-off since project construction."

On December 30th and 31st of 2005, a California Irrigation Management Information System (CIMIS) rain gauge station located 2 miles from Tassajara Creek in Pleasanton, CA, recorded 3.56 inches of rainfall in a 22-hour period, the largest 24-hour period of rainfall in the 20-year period of record for rain gauges in the surrounding area. The storm passed over the Tassajara Creek watershed, generating flows that overtopped the low-flow banks of Tassajara Creek and providing an opportunity to evaluate flow capacity of the compound channel configuration. The intensity and duration of the rainfall created peak flows and obvious new high-water marks throughout the restored stretch of Tassajara Creek, as well as along the concrete trapezoidal channel south of the restoration project. These high-water marks provided a unique opportunity to assess the roughness and channel capacity of the restoration project, and to identify whether the actual performance of the restoration projects meets the standards and expectations of the original restoration design. We surveyed a long profile of the entire restored stretch of the creek, as well as a section at the north end of the concrete channel. We returned at a later date to survey cross-sections along the restored reach. In all, we surveyed six of the eight cross-sections established in the original as-built design. We recorded elevations of the new high-water marks as well as vegetation and sedimentation characteristics. We used these field observations, along with rain gauge data, to calculate roughness and channel capacity in an effort to determine whether the restored reach of Tassajara Creek accommodates the flood levels for which it was designed.

Methods

COMPUTATION OF OPEN CHANNEL FLOW

We calculated actual flow levels for the peak storm in December 2005, based upon high water marks recorded during our long profile and cross-sectional surveys of Tassajara Creek. Using our survey data, we also estimated water elevations for the 100-year flood at six different cross-sections. To do so, we relied upon the Manning Equation, as follows:

$$v = \frac{1.49(s^{0.5}R^{0.67})}{n}$$

where v = velocity in feet per second (ft/s), R = hydraulic radius, or cross-sectional area divided by the wetted perimeter, and s = slope of the stream (gradient). "n" is a coefficient for roughness, a parameter which corresponds to friction and slows the velocity of the flow (Dunne and Leopold 1943). We input our survey data into an Army Corps of Engineers' HEC-RAS steady flow model to calculate values for the variables in the Manning Equation. In addition, we used the equation for discharge

$$Q = vA$$

to calculate flow for the high water marks and cross-sectional areas for the 100-year flood. We further explain our survey methods below.

Roughness cannot be measured but rather, it must be estimated based upon characteristics specific to the channel, such as the degree of irregularity; effect of obstructions, such as rocks; and the presence of vegetation (Kondolf 2006a). The Manning Equation was instrumental to our analysis, allowing us to back-calculate for "n" rather than estimate it subjectively.

LONG PROFILE AND CROSS SECTION SURVEY

On February 25, 2006, we performed a long profile survey of the entire mile-long reach of the restoration project and recorded high water marks along the reach and near each cross-section. On April 25, 2006, we surveyed cross-sections E, F and at the beginning of the trapezoidal concrete channel beneath the Interstate 580 (I-580) bridge and recorded the high water marks from the late December twenty year storm event. We utilized survey data gathered by Mark Tompkins in February 2006, for cross-sections B, D, G and H. We plotted the survey data to assess the shape of the channel at each cross-section and identify the water elevations corresponding to the high water marks we recorded.

RAIN GAUGE ANALYSIS

We downloaded rainfall gauge data from CIMIS, the California Irrigation Management Information System of the Department of Water Resources, Office of Water Use Efficiency. The nearest rain gauge to Tassajara Creek is Station ID 191, located approximately 2 miles away in Pleasanton, CA, which has data from April 22, 2004 to the present.

Many factors influence rainfall recorded at one position (such as Pleasanton, CA) relative to another position (such as Tassajara Creek), including differences in location, elevation, surrounding ridgelines, temperature, wind velocity, storm direction, and many other factors. Moreover, because the Pleasanton rain gauge had data only from April 22, 2004, in order to put the intensity of the storm in historical context, we researched other nearby rain gauge data to identify the rain gauges whose measurements correlate most closely with the rain gauge in Pleasanton.

Table 1 shows the cumulative rainfall for the Pleasanton rain gauge and the surrounding area rain gauges for the period of record of the Pleasanton rain gauge, April 22, 2004 to April 12, 2006, as well as the precipitation recorded during the 24-hour period of the storm on December 30-31, 2005.

Table 1. Comparison of Area Rain Gauges

Station ID	Location	Distance from Tassajara Creek	Cumulative Precipitation (inches)	% of Pleasanton, CA	24-hour Precipitation (inches)	% of Pleasanton, CA
191	Pleasanton, CA	4 miles	47.58	100	3.56	100
170	Concord, CA	9 miles	39.65	83	2.93	82
47	Brentwood, CA	7 miles	33.57	71	1.63	46
171	Union City, CA	9 miles	33.57	86	0.49	14
70	Manteca CA	20 miles	33.57	75	0.64	20

AC 5.1.06

The table indicates that the Concord rain gauge provides the best equivalent to the Pleasanton rain gauge; however it has data going back only to April 6, 2001. The Brentwood rain gauge is the next best fit, and it has data going back to January 1, 1986. Therefore, to complete our historical analysis, we downloaded data from the two rain gauges, Station ID 170 in Concord, CA, and Station ID 47 in Brentwood, CA. **Table 2** displays the time periods of rain gauge data in our analysis from each station.

Table 2. Rain Gauges Used in Rainfall Analysis

Station ID	Location	Start Date	End Date	Distance from Tassajara Creek
191	Pleasanton, CA	April 22, 2004	April 16, 2006	2 miles
170	Concord, CA	April 6, 2001	April 21, 2004	9 miles
47	Brentwood, CA	January 1, 1986	April 5, 2001	7 miles

AC 5.1.06

We created historical precipitation graphs for:

- Annual Precipitation for Water Years 1987 to 2006
- Monthly Precipitation from January 1986 to April 2006
- Daily Precipitation for Water Year 2006
- Peak Storm Hourly Precipitation December 30-31, 2005
- Peak 24-Hour Storms from January 1986 to April 2006
- Daily Peak Precipitation from January 1986 to April 2006

MANNING EQUATION FOR ROUGHNESS AND CHANNEL CAPACITY

Using a HEC-RAS steady flow model, we applied the Manning Equation to the concrete, trapezoidal cross section to determine the discharge corresponding to the high water mark we recorded in our surveys. We used a roughness coefficient, or Manning's "n", of 0.011 for a channel with a cement, neat surface (Chow 1959). We used the calculated discharge to back-calculate a roughness coefficient for each cross-section (B through D). Finally, we calculated the Manning's "n" for each cross-section, which we used to predict the water surface elevation at each cross-section for the specified 100-year flood level (Q_{100}). We compared this estimate to the levee top elevation at each cross-section to determine whether the compound channel has the capacity to accommodate the 100-year flood. We compared our estimates against the 100-year flood specified by both Alameda County (5,200 cfs) and FEMA (4,300 cfs). We also compared the capacity of the low-flow channel, using the levels for the 2-year and 5-year floods only as specified by Alameda County. At each cross section, we calculated water surface elevations for either the 2-year or 5-year flood, depending on the design specifications. Cross-sections B, D, E and F, in the upper and middle reaches, were designed to accommodate the 5-year flood of 1,200 cfs, while cross-sections G and H, in the lower reach, are only intended to contain the 2-year flood of 650 cfs.

Results and Discussion

RAIN GAUGE ANALYSIS

The Annual Hydrograph for Water Years 1987 through 2006 shows that the water year 2006 through April 17, 2006, has recorded the 3rd largest amount of precipitation, 22.98 inches, in the 20-year period of record. The water year with the most precipitation was 1998, the “El Niño” winter, with 26.5 inches, followed by water year 2005, which recorded 25.24 inches of precipitation. The Monthly Hydrograph shows that December 2005, the month of the storm under review, recorded the 2nd largest amount of precipitation, 7.77 inches, in the 20-year period of record. The month with the largest amount of precipitation was February 1998, in the “El Niño” winter, which recorded 8.00 inches of precipitation.

The Daily Precipitation Hydrograph for Water Year 2006 shows that December 31, 2005 recorded the largest precipitation for the water year, at 2.64 inches. This is nearly twice as much as the next largest precipitation day, December 18, 2005, which recorded 1.41 inches.

The Peak Storm Hourly Precipitation Hydrograph shows that the storm on December 30-31, 2005 accumulated 3.56 inches of rainfall in a 22-hour period. The Peak 24-Hour Storm Precipitation Hydrograph indicates that this storm is the largest 24-hour accumulation of precipitation in the 20-year period of record. The next largest 24-hour period of rainfall is 2.08 inches on February 2, 1998. The Peak 24-Hour Storm Precipitation Hydrograph indicates a 171% peak over the next largest 24-hour period on record.

The moving 24-hour cumulative total is a significant unit of analysis, because analyzing only the daily calendar totals would not identify this storm on the same order of magnitude. The Daily Peak Precipitation Hydrograph shows that the rainfall total on the calendar date of December 31, 2005, was 2.64 inches, a unit of analysis which still ranks the storm as the largest single day of rainfall for the period of record, but the total is less than the cumulative 24-hour total of 3.56 inches. The next largest calendar date rainfall is 1.94 inches on December 11, 1995. The Daily Peak Precipitation Hydrograph indicates a 136% peak over the next calendar date.

Calculation of Roughness

Using the HEC-RAS steady flow model, we specified the flow boundary condition as normal depth in the concrete, trapezoidal channel with a downstream slope of 0.017. Our application of the Manning Equation provided us with a discharge of approximately 1,500 cfs for our surveyed high water marks. For this level of flow, we then used the model to back-calculate a roughness coefficient (Manning's "n") for each cross-section. **Table 3** displays these results, along with the corresponding high water marks that we calculated in our model and the vegetation that we noted in our surveys.

Table 3. Calculation of Roughness Using HWMs from 1,500 cfs

Cross-Section Station	HWM Elevation (ft)	Calculated Roughness	Vegetation
B	361.20	0.0865	Coyote Brush, Grass, Ruderal, Sedge, Willow
D	357.40	0.0368	Blackberry, Coyote Brush, Grass, Willow
E	354.70	0.0685	*Need field book data*
F	350.70	0.0705	*Need field book data*
G	347.70	0.0520	Cattails, Coyote Brush, Emergent Aquatic, Grass, Pepper Tree, Willow
H	344.50	0.0380	Cattails, Coyote Brush, Grass, Willow
I-580	342.05	0.0110	None

SKH 4.25.2006

By contrast, the project design utilized uniform roughness coefficients for the entire reach of the restored channel – values of 0.040 for in-channel and 0.12 for overbank roughness. For all but two of the cross-sections we surveyed (Cross-Sections D and H), our calculated roughness coefficients exceeded the project design's in-channel roughness value of 0.040. However, for all of our cross-sections, the roughness coefficients we calculated were lower than the design's overbank roughness of 0.12.

We noted a significant amount of vegetation on the low-flow channel's natural terrace, likely the cause of the high roughness coefficients we calculated. However, only ten years have elapsed since project completion, and the vegetation we observed is still quite young. Experience has shown that in the long run, roughness tends to decrease as competition for resources increases, and larger trees, such as willows, effectively shade out smaller varieties (Lovett 1999, Hecht 2006). Therefore, it is likely that the roughness coefficients we calculated will decline in the future, thus decreasing friction and lowering water elevations as velocity increases.

CHANNEL CAPACITY: COMPARISON TO DESIGN PLAN

Alameda County has determined the discharge of the 100-year flood on Tassajara Creek to be 5,200 cfs. Using our previously determined roughness coefficients, we calculated the expected water surface elevation at each cross-section for the 100 year flood of 5,200 cfs. We then compared the predicted water level to the elevation of the lower, left bank (levee top elevation) at each cross-section to determine whether the channel can accommodate a Q_{100} of 5,200 cfs.

Table 4 displays these results, along with the difference between the levee top elevation and the 100-year flood water surface elevation.

Table 4. Comparison of Predicted Q_{100} & Bank-full Water Elevations

Cross-Section	Q_{100} Elevation (ft)	Levee Top Elevation (ft)	Difference (ft)
B	363.61	369.24	5.63
D	361.75	365.59	3.84
E	359.00	359.88	0.88
F	354.47	356.01	1.54
G	351.28	354.10	2.82
H	350.32	352.40	2.08
I-580	346.78	349.14	2.36

SKH 4.25.2006

At each cross-section we surveyed along the compound channel, Tassajara Creek has the capacity to fully contain the 100-year flow of 5,200 cfs. However, cross-section E provides for only 0.88 feet of freeboard, which is less than the one-foot minimum that Alameda County requires. This results from the narrow span of the channel's upper terrace, which is only115 feet across at the top, compared to the widths of other cross section terraces, which range from 170 to 200 feet. The design plans required the accommodation of two large oak trees on the left bank of the floodplain near cross-section E (Cook 2006). Even with the smaller area at cross-section E, however, we still find that the restored portion of Tassajara Creek should contain a 100-year flood discharging 5,200 cfs.

The cross-section (Cross-Section E) that provides less than one estimated foot of freeboard requires further study. Based on observation of the floodplain relative to the compound channel, we hypothesize that the compound channel was designed to be narrower at this cross-section to accommodate the root systems of several large oak trees on the 100-year floodplain. We recommend further study and monitoring of this cross-section, in light of the area's rapid growth, which permits development very near the banks of the creek since it has been removed from the "100-year floodplain" as defined by FEMA, but not necessarily from the floodplain of significantly larger floods such as the 200-year flood, or the 500-year flood, i.e. the floods which have a 0.5% chance and 0.2% chance of occurring, respectively, each year (Kondolf 2006b).

The low flow channel contains the flood-level corresponding to its design plan at only one cross-section we surveyed. In the lower reach, the low flow channel contains the 2-year flood of 650 cfs only at cross-section H; a Q_2 flood overflows onto the natural floodplain terrace at cross-section G. In the upper and middle reaches, all four cross-sections overtop their banks onto the floodplain terrace for a 5-year flood of 1,200 cfs. As previously mentioned, roughness is likely to decrease in the future along the restored reach of Tassajara Creek. This will cause water elevations to drop for flood levels, providing the natural compound channel with greater freeboard to accommodate the 100-year flood. A decline in roughness will also lower the levels of the 2-and 5-year floods, thereby decreasing the amount of flow that overtops the banks of the low flow channel.

Conclusion

The restoration project along Tassajara Creek in Dublin, CA sought to achieve the goal of flood control for new development while at the same time improving riparian habitat. The City of Dublin constructed a natural, compound channel, designed to accommodate the 100-year flood specified by both Alameda County and FEMA. The natural, compound channel created a low flow channel with a natural floodplain terrace to allow for floodplain connectivity in order to improve vegetation and wildlife habitat. Our study evaluated the efficacy of the flood control measures, focusing on the ability of the channel dimensions to accommodate 100-year flows based on our calculation of roughness along six cross-sections.

The storm on December 30-31, 2005, was the largest in the 20-year period of record and provided us an ideal opportunity to evaluate Tassajara Creek's flood carrying capacity. Using high water marks from the storm and hydraulic modeling, we calculated the roughness and estimated water levels for the 100-year flood at each of six cross-sections. Our comparisons to the actual channel dimensions allowed us to conclude that the restored portion of Tassajara Creek successfully contains both the Alameda County and FEMA 100-year flood levels of 5,200 cfs and 4,300 cfs, respectively. Although one cross-section does not currently provide the full one foot of freeboard that Alameda County requires, we suggest further study using a more refined model of roughness that includes calculations of roughness based on discrete horizontal slices at various elevations within the compound channel, estimating the variations in roughness at each successive higher elevation as vegetation at that elevation declines (or increases, as in the case of wide branches of tall trees). Also, a more refined model of roughness would include interface friction factors due to longitudinal channel variation between the low-flow channel and the vegetated floodplain of the compound channel (Pasche 1984, Nuding 1991, Helmiö 2004).

We found that the low flow channel does not fully contain the 2- and 5-year floods for which it was designed. However, this is not necessarily a shortcoming, as it poses no danger to human safety and increases floodplain connectivity between the low flow channel and its natural floodplain terrace. This encourages vegetation growth and provides for wildlife habitat, successfully meeting project goals.

In conclusion, we find that the restoration of Tassajara Creek in Dublin represents a successful pairing of flood control and habitat restoration in one project. The natural, compound channel is an improvement over traditional concrete flood control channels, providing aesthetic and recreational values as well, and it can serve as a model for future flood control projects.

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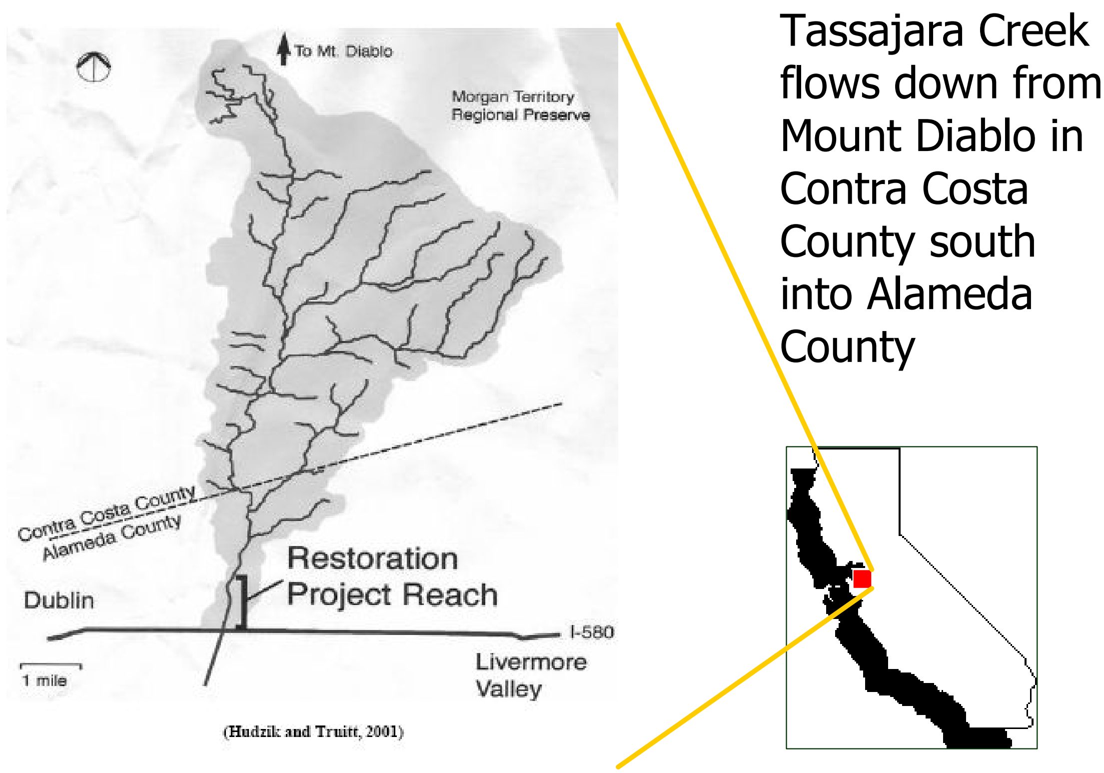
Figures

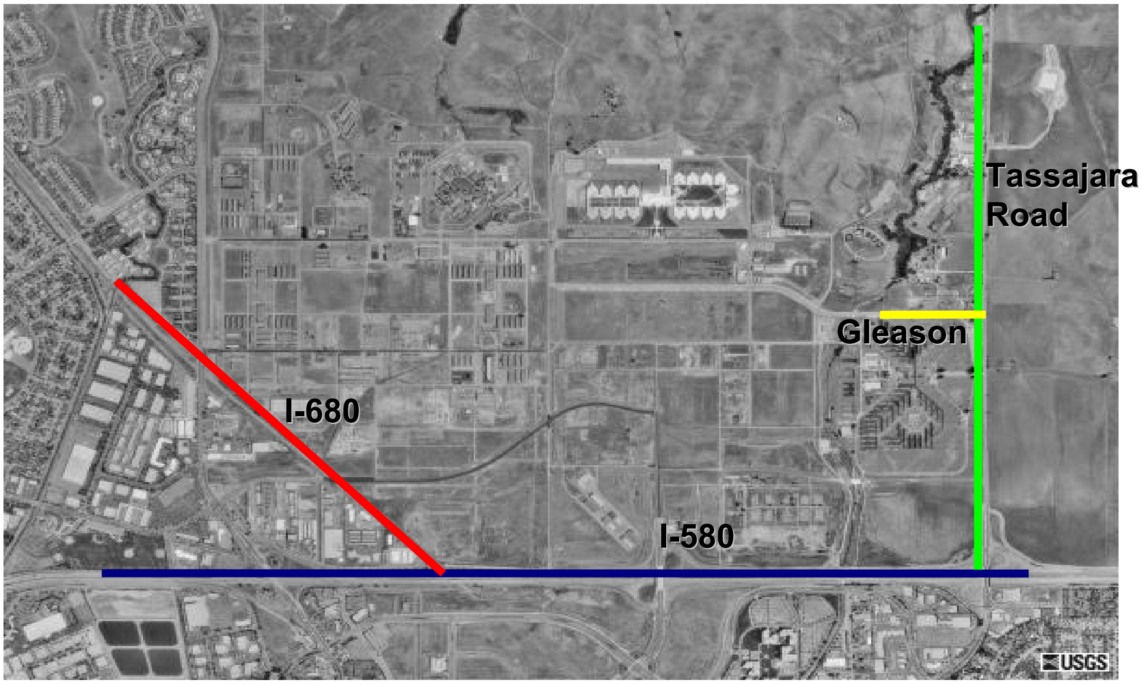
<u>MAPS</u>

Figure 1. Location Map of Tassajara Creek, Dublin CA



Source: Hudzik and Truitt, 2001.

Figure 2. Aerial View of Tassajara Creek, Dublin, CA, 1993

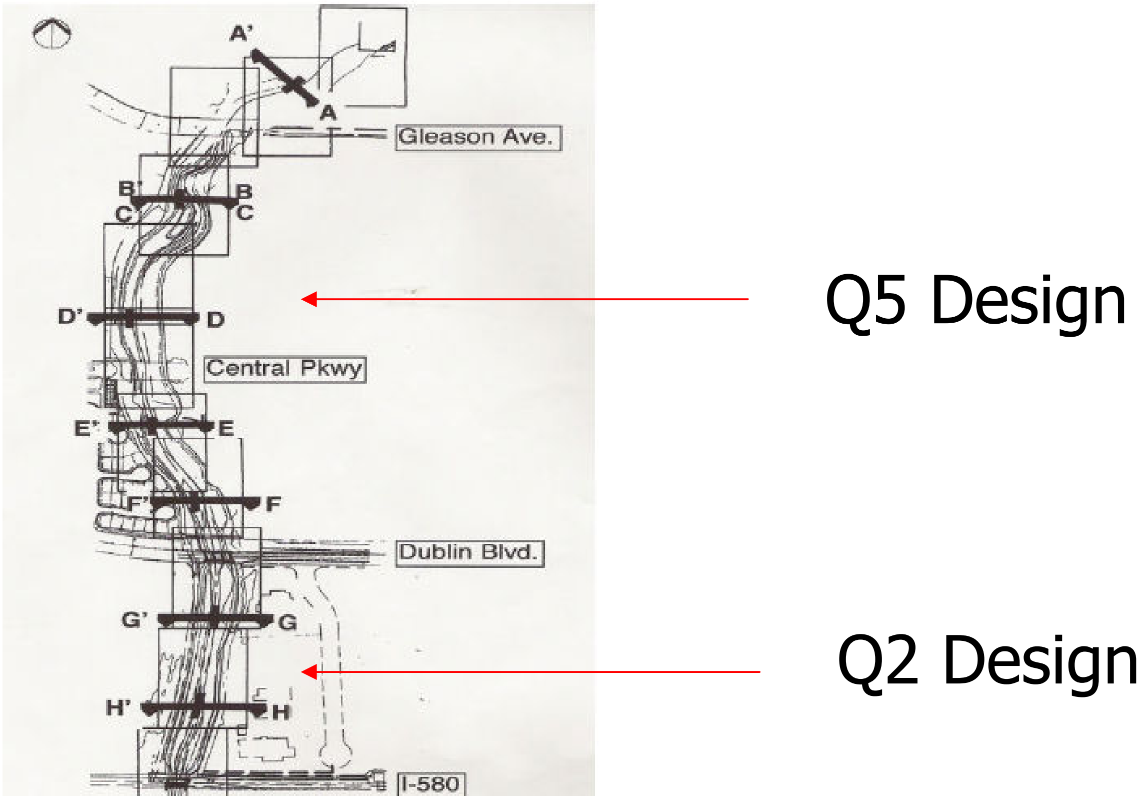


Source: US Geological Survey

Figure 3. Aerial View of Tassajara Creek, Dublin, CA, 2004

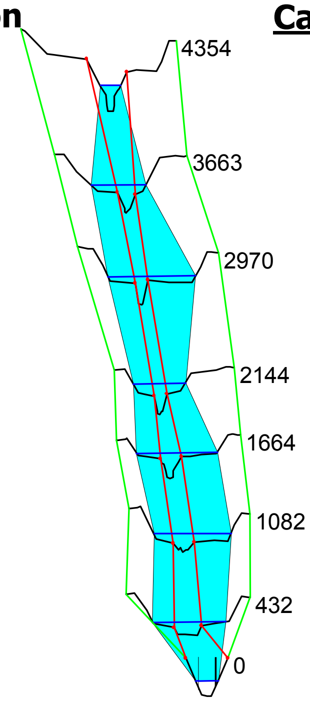
Source: US Geological Survey

Figure 4. Map of Restored Reach, Tassajara Creek, Dublin, CA



Source: Hudzik and Truit, 2001

Figure 5. Roughness Comparison Using December 2005 HWMs



Cross-Section Station	Calculated
B	.0865
D	.0368
E	.0687
F	.0705
G	.0520
H	.0380
I-580	.0110

Design Estimates

In-Channel
0.040

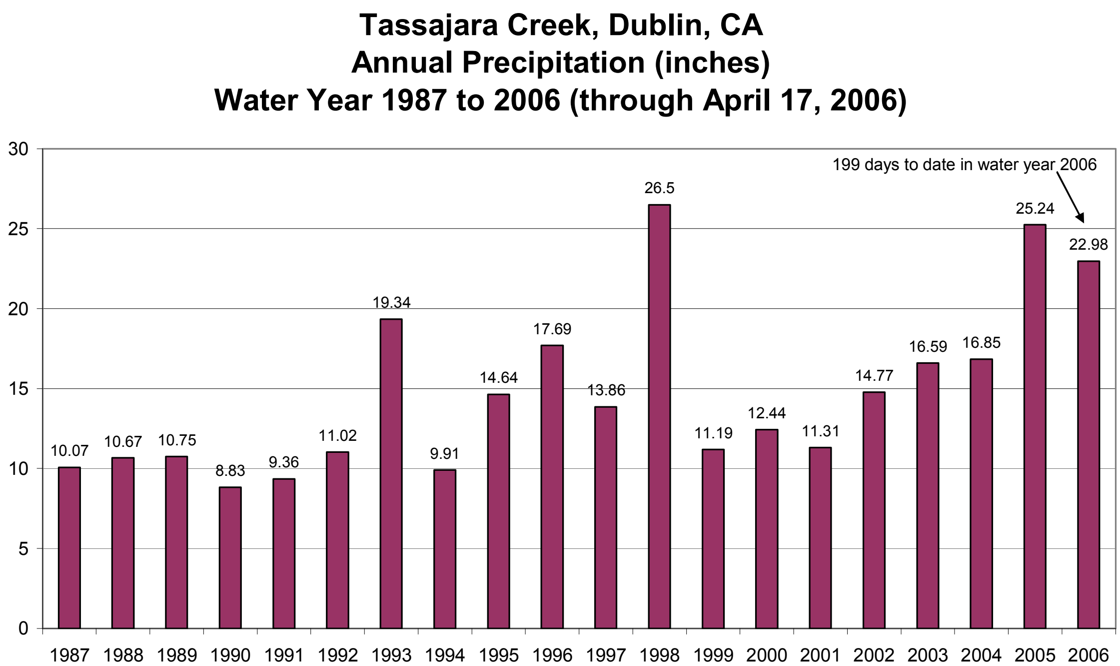
Overbank
0.120

Concrete Channel
0.016

Source: Calculations by Chan and Heard, design estimates from BKF Engineers

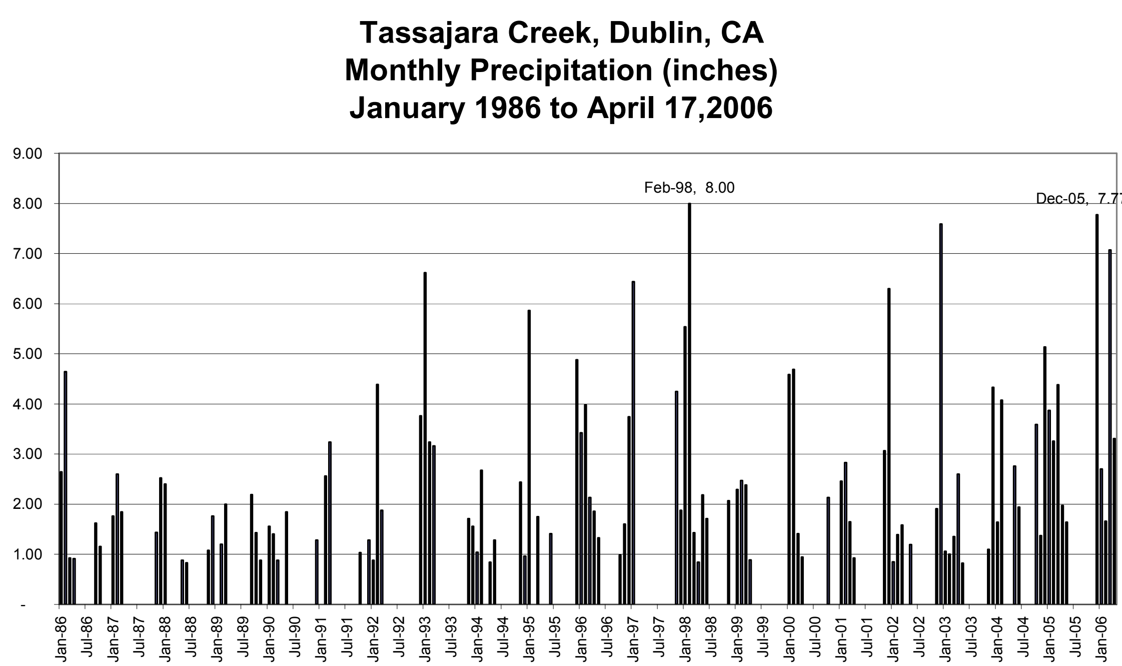
<u>HYDROGRAPHS</u>

Figure 6. Annual Hydrograph Water Year 1987 to April 17, 2006



Source: California Irrigation Management Information System (CIMIS), California Department of Water Resources, Office of Water Use Efficiency, Station ID 47 01/01/86 to 04/05/01, Station ID 170 04/06/01 to 04/21/04, Station ID 191 04/22/04 to Present, Data Downloaded April 18, 2006

Figure 7. Monthly Hydrograph January 1986 to April 17, 2006



Source: California Irrigation Management Information System (CIMIS), California Department of Water Resources, Office of Water Use Efficiency, Station ID 47 01/01/86 to 04/05/01, Station ID 170 04/06/01 to 04/21/04, Station ID 191 04/22/04 to Present, Data Downloaded April 18, 2006

Figure 8. Daily Hydrograph Water Year 2006

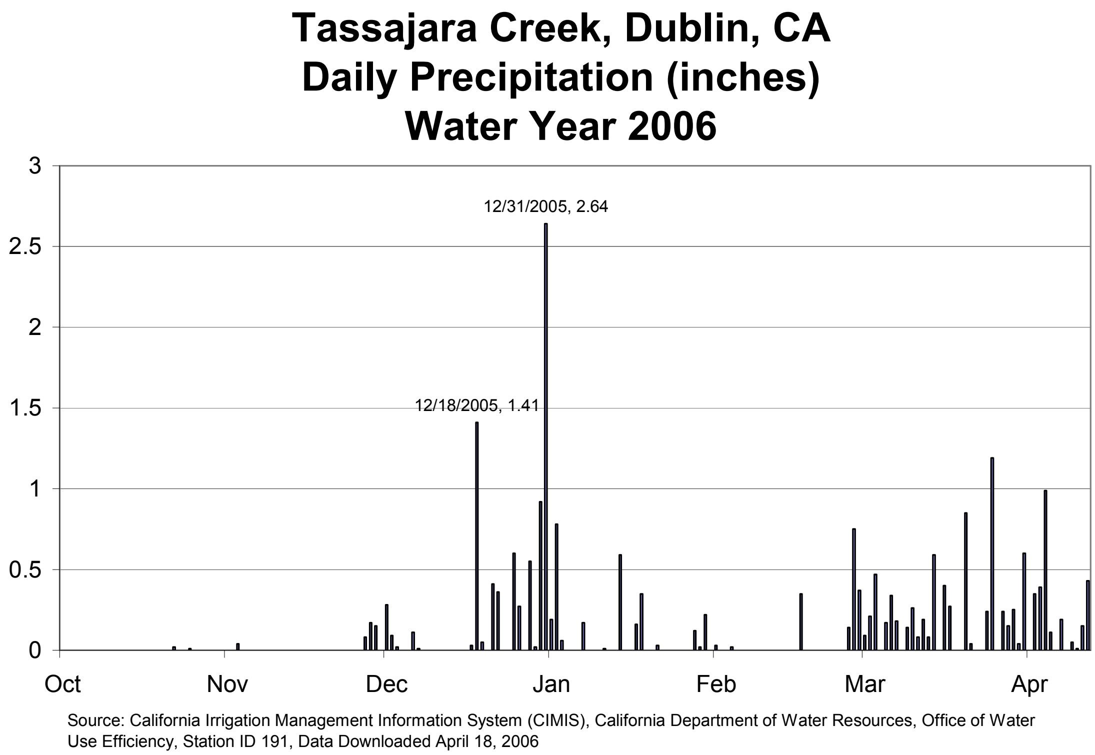


Figure 9. Peak Storm Hourly Hydrograph, December 30-31, 2005

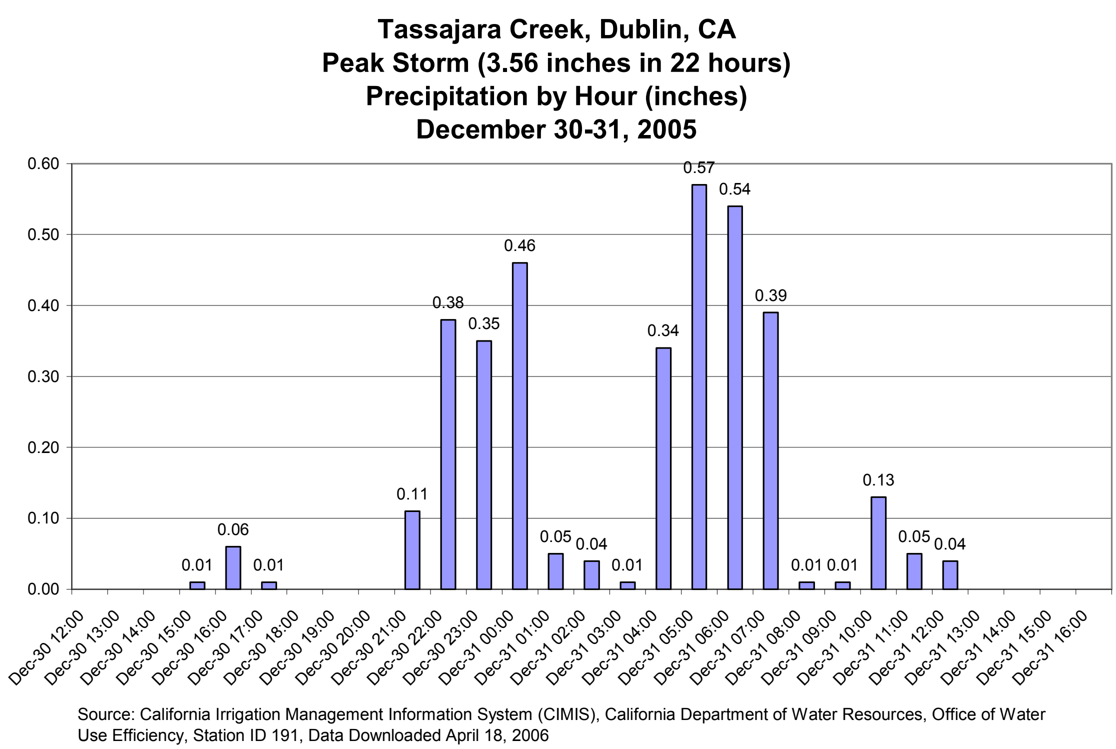
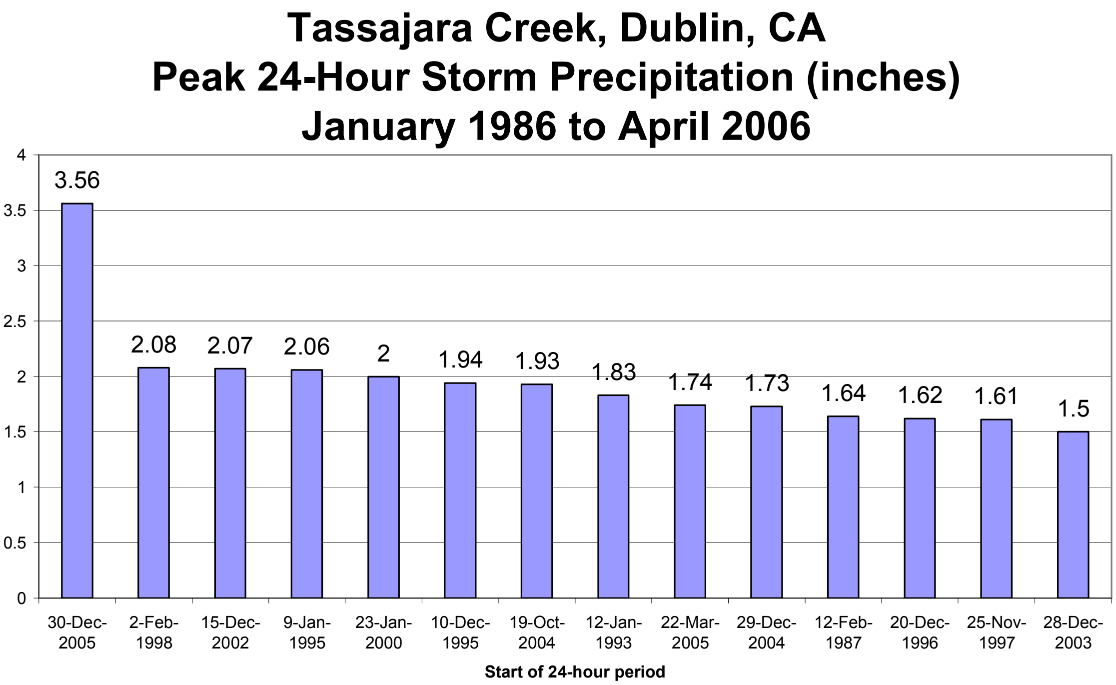
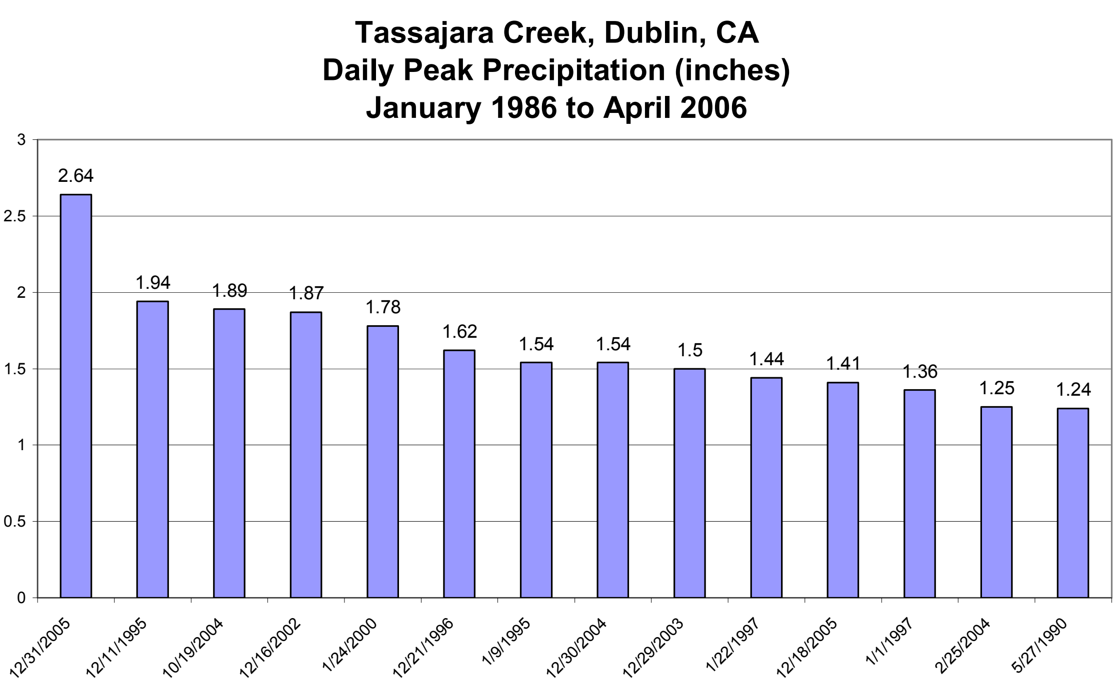


Figure 10. Peak 24-Hour Storm Hydrograph January 1986 to April 2006



Source: California Irrigation Management Information System (CIMIS), California Department of Water Resources, Office of Water Use Efficiency, Station ID 47 01/01/86 to 04/05/01, Station ID 170 04/06/01 to 04/21/04, Station ID 191 04/22/04 to Present, Data Downloaded April 18, 2006

Figure 11. Daily Peak Precipitation Hydrograph January 1986 to April 2006



Source: California Irrigation Management Information System (CIMIS), California Department of Water Resources, Office of Water Use Efficiency, Station ID 47 01/01/86 to 04/05/01, Station ID 170 04/06/01 to 04/21/04, Station ID 191 04/22/04 to Present, Data Downloaded April 18, 2006

Q2 FLOWS AT 650 CFS

Figure 12. Cross-Section G at 650 cfs (Q_2)

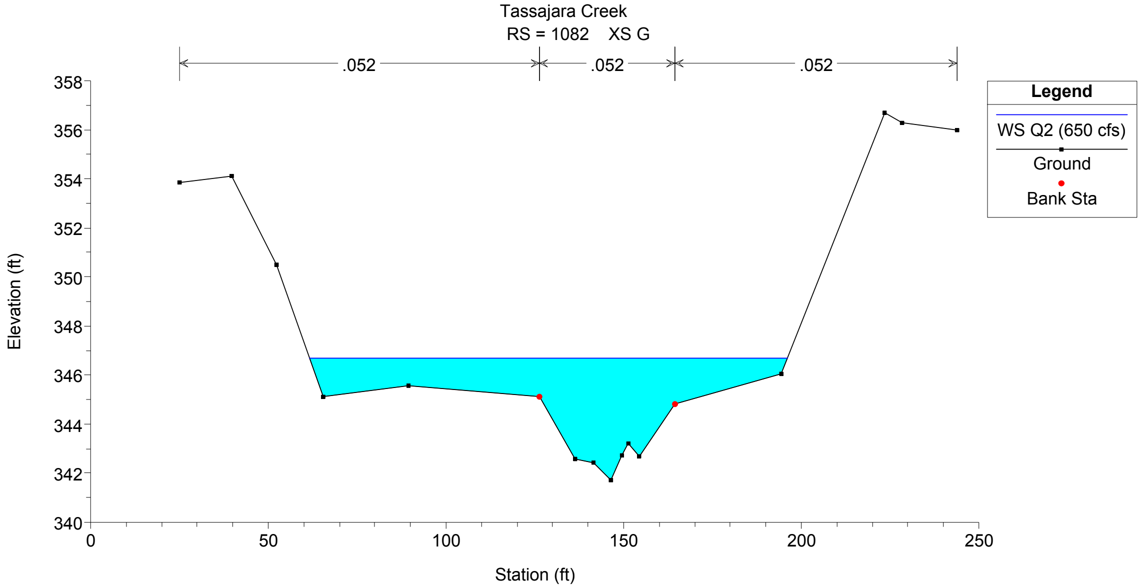


Figure 13. Cross-Section H at 650 cfs (Q_2)

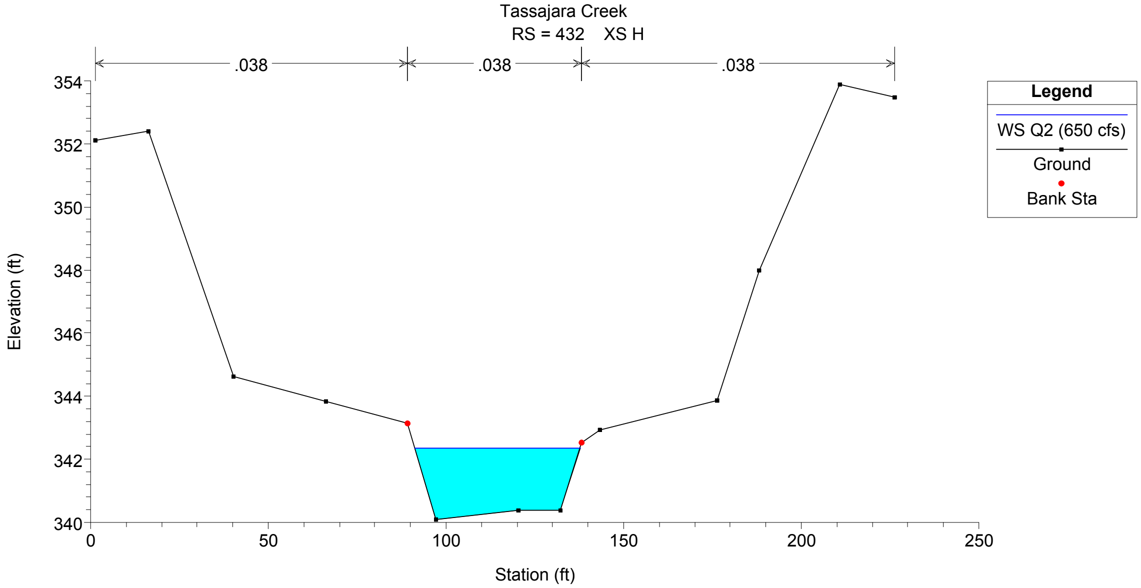
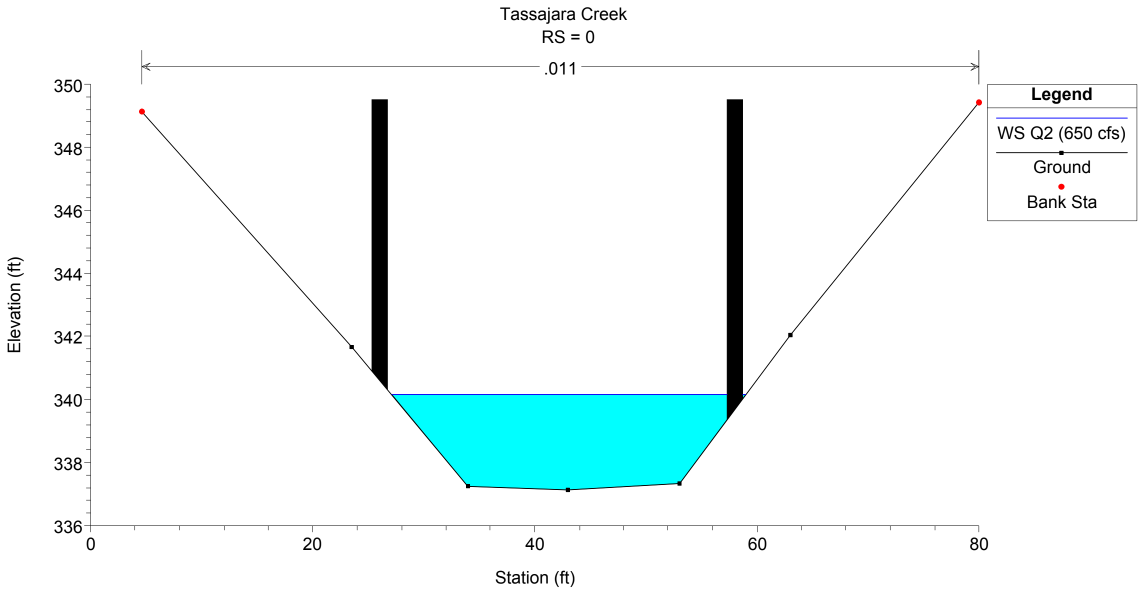


Figure 14. I-580 at 650 cfs (Q_2)



Q5 FLOWS AT 1,200 CFS

Figure 15. Cross-Section B at 1,200 cfs (Q_5)

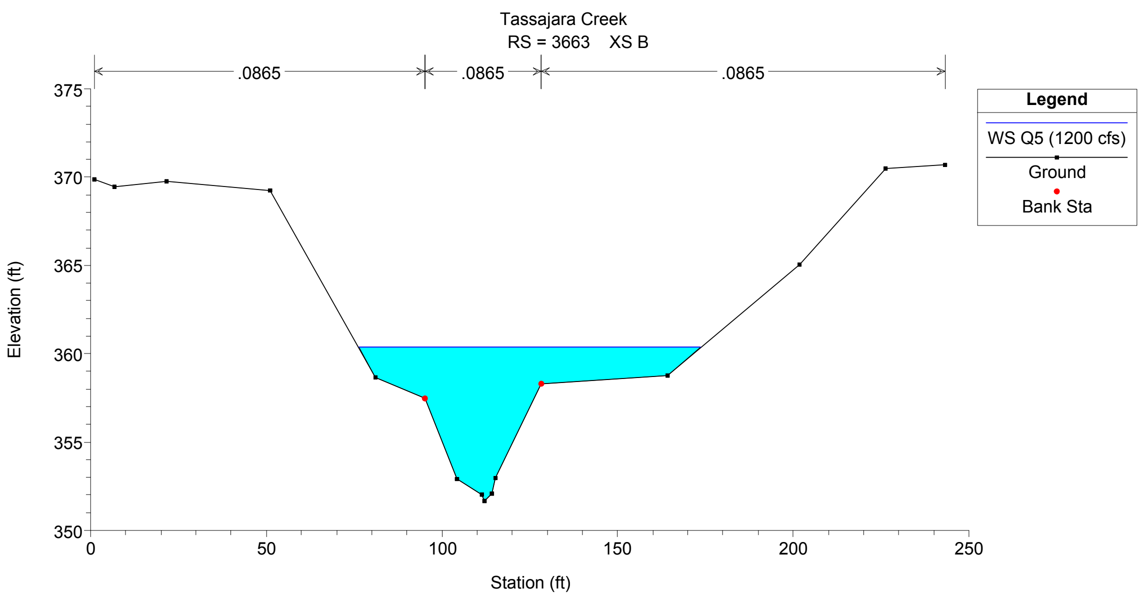


Figure 16. Cross-Section D at 1,200 cfs (Q_5)

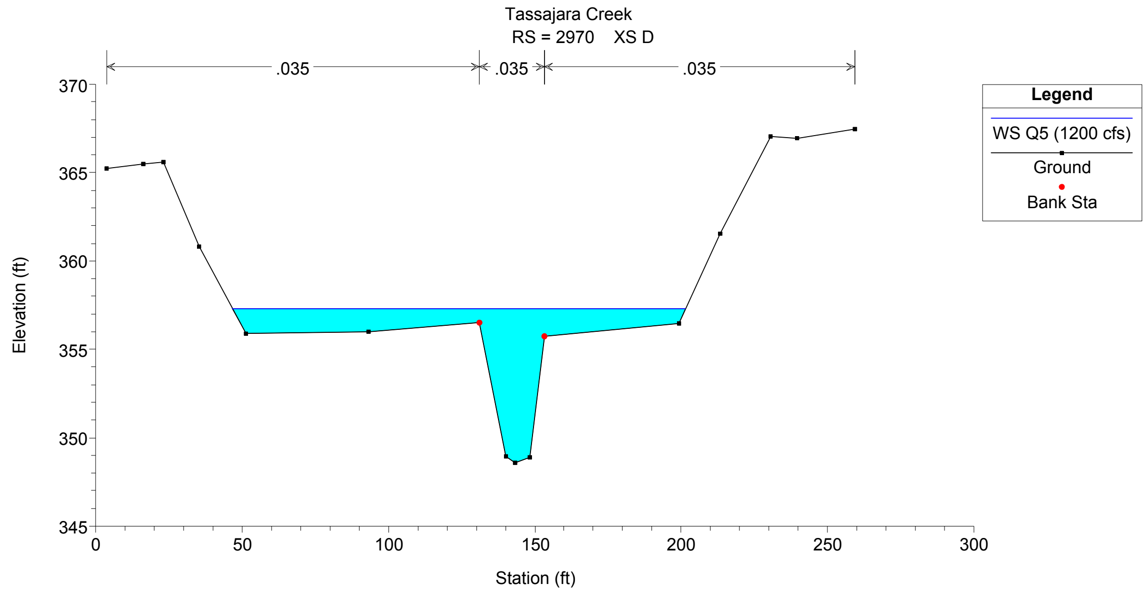


Figure 17. Cross-Section E at 1,200 cfs (Q_5)

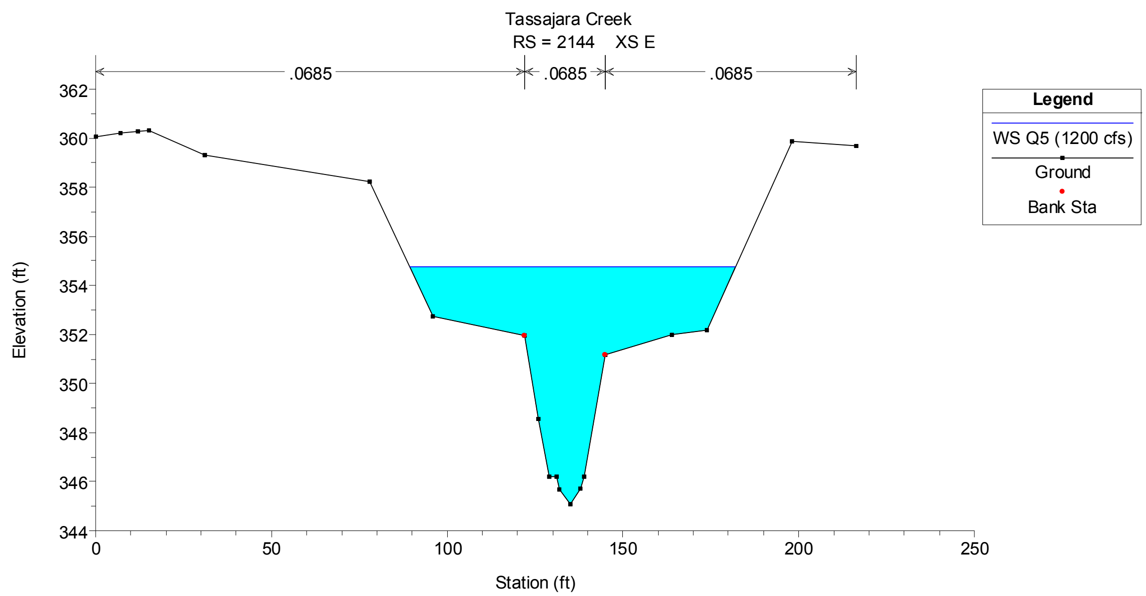


Figure 18. Cross-Section F at 1,200 cfs (Q_5)

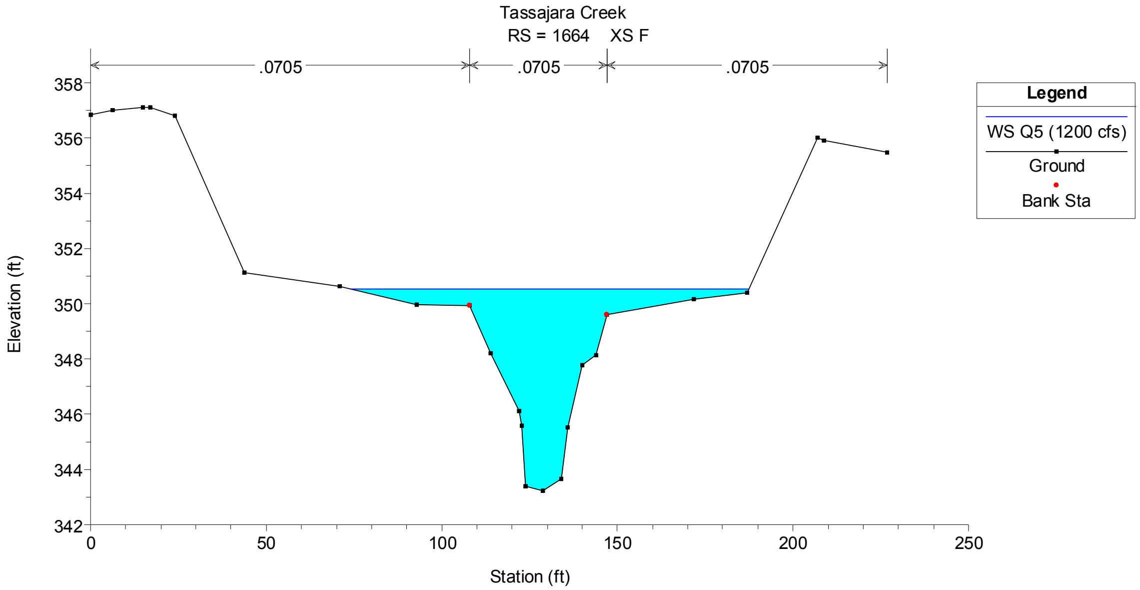
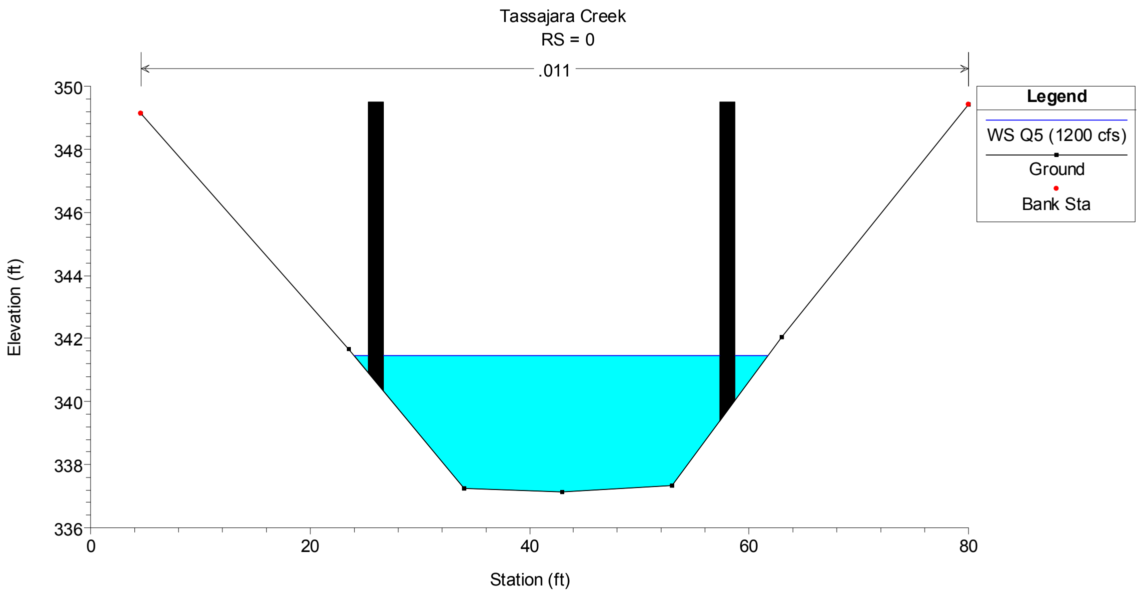


Figure 19. I-580 at 1,200 cfs (Q_5)



DECEMBER 2005 STORM FLOWS AT 1,500 CFS

Figure 20. Cross-Section B at 1,500 cfs

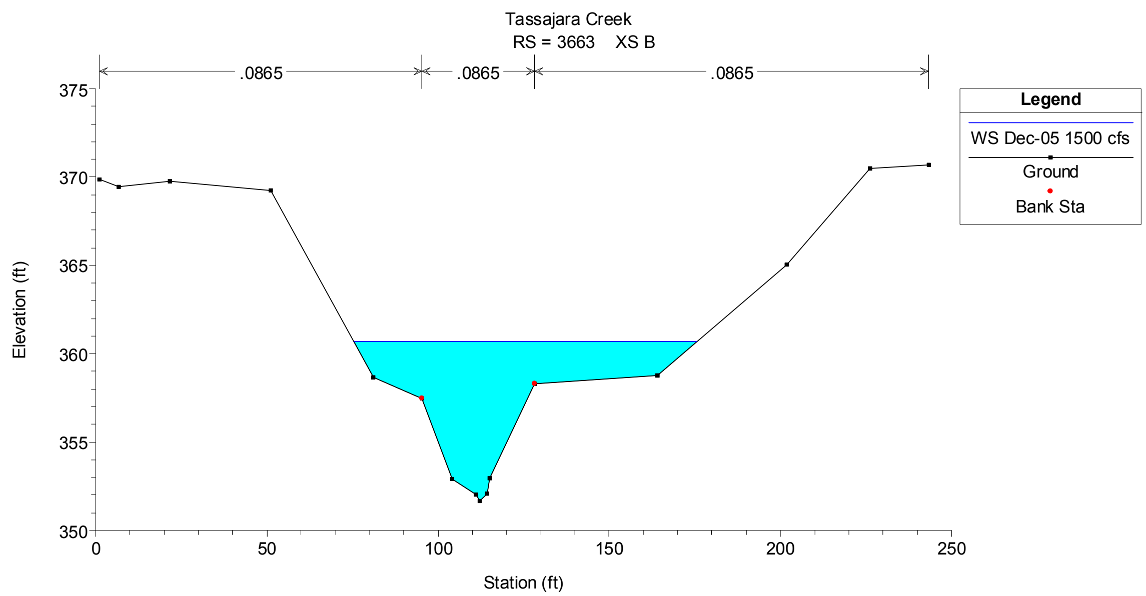


Figure 21. Cross-Section D at 1,500 cfs

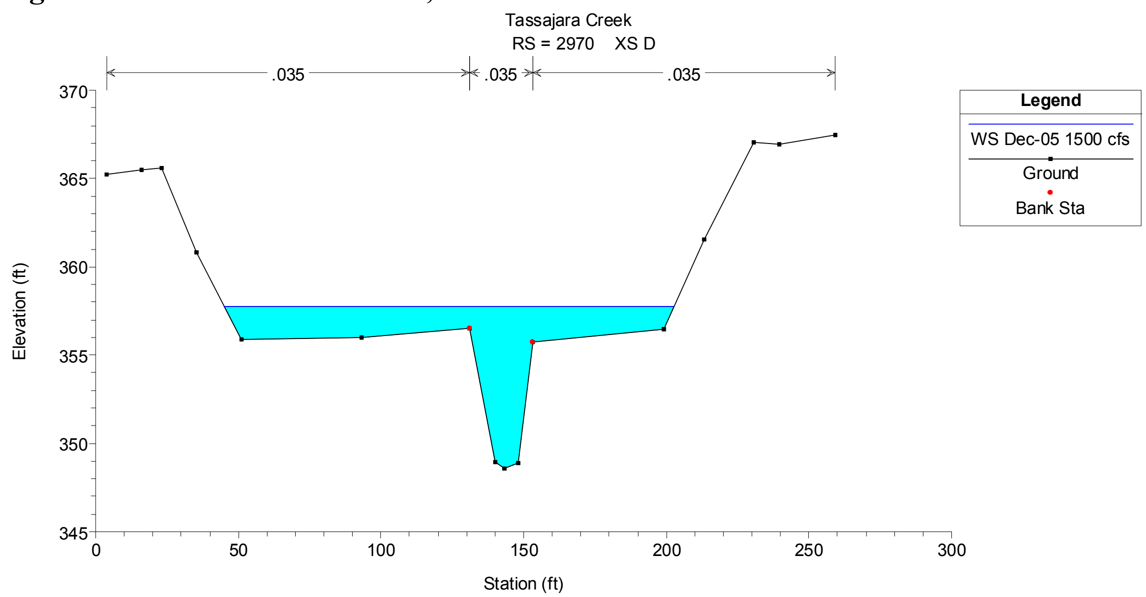


Figure 23. Cross-Section E at 1,500 cfs

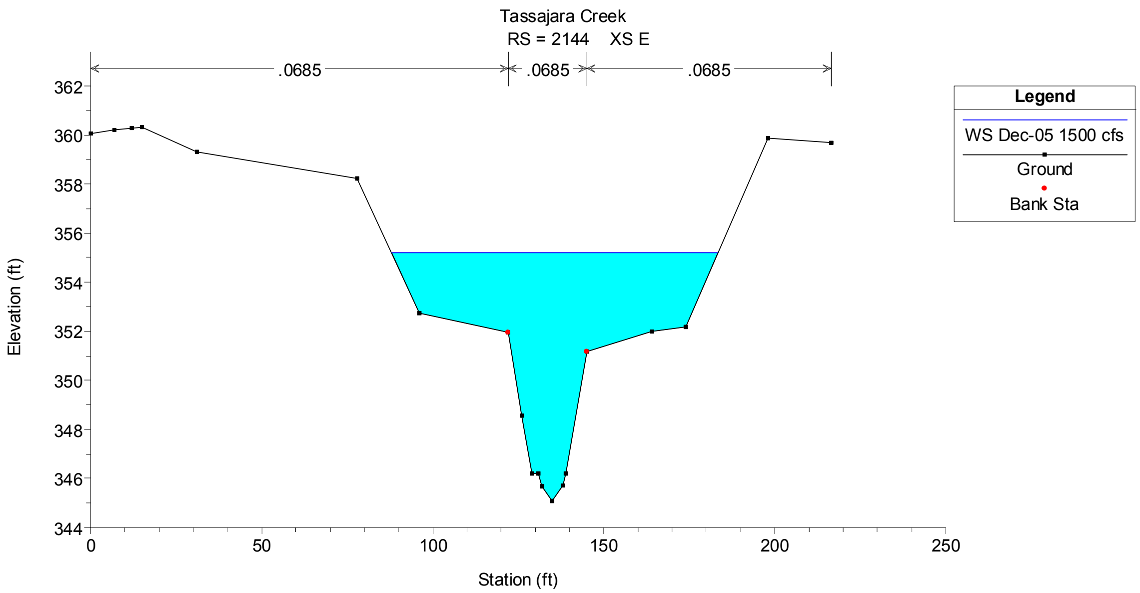


Figure 24. Cross-Section F at 1,500 cfs

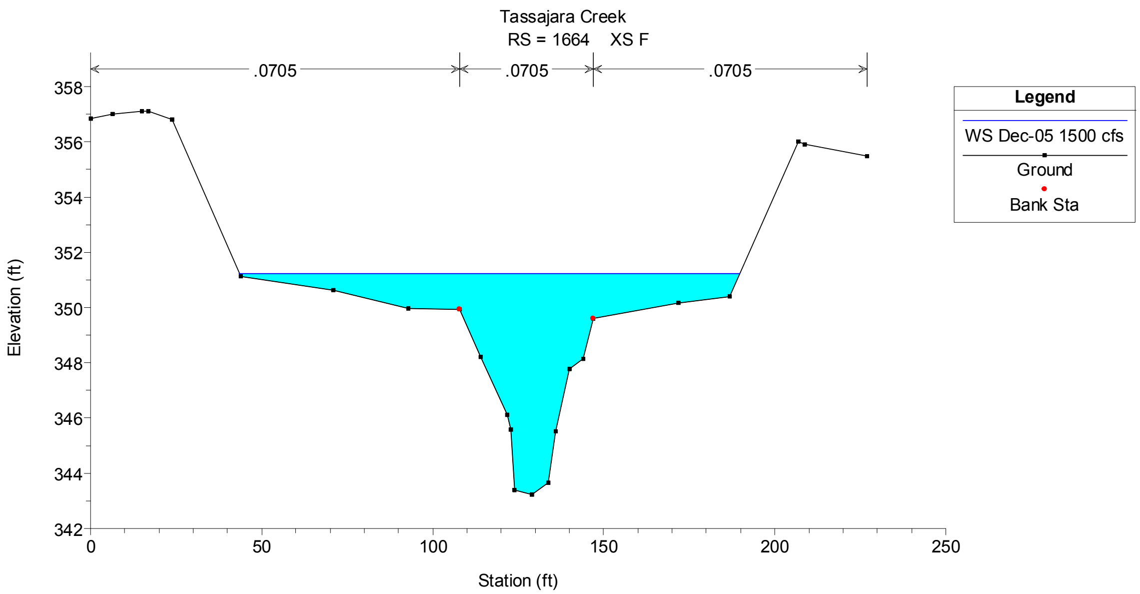


Figure 25. Cross-Section G at 1,500 cfs

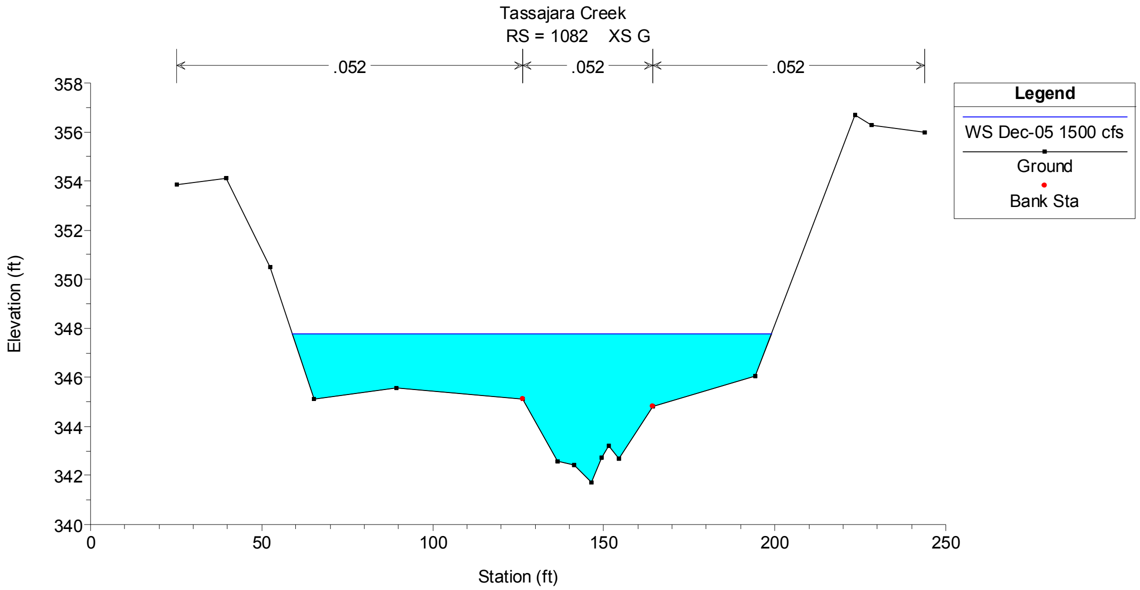


Figure 26. Cross-Section H at 1,500 cfs

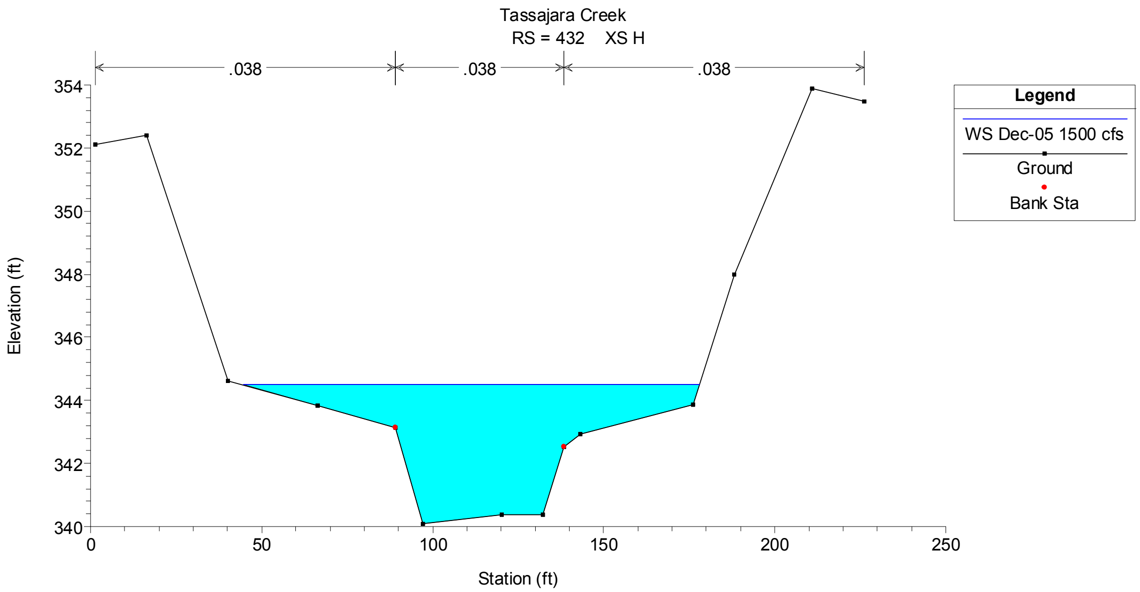
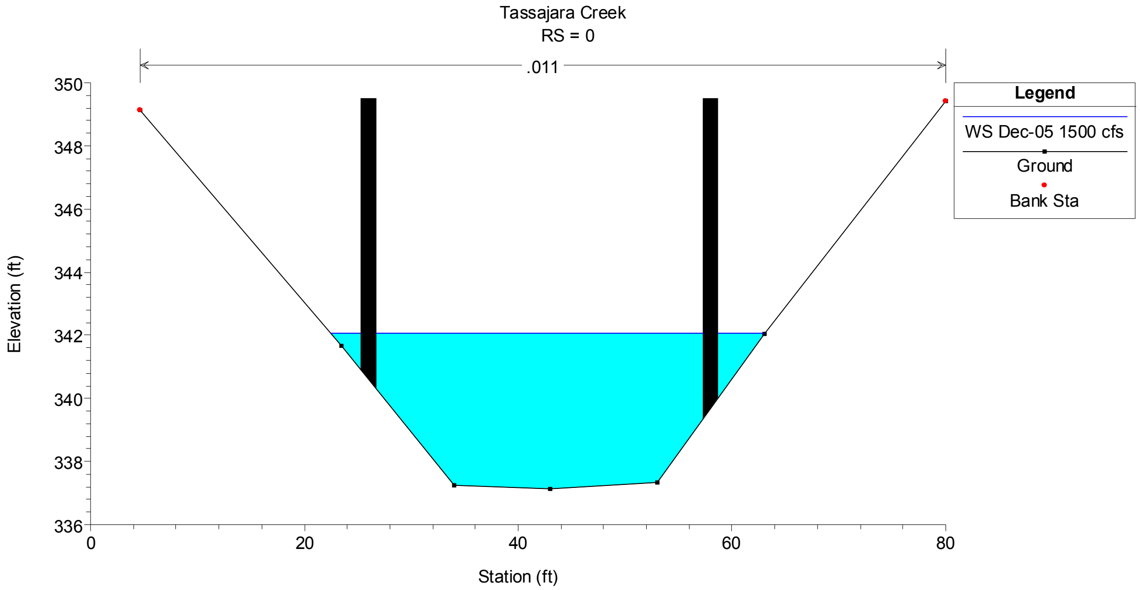


Figure 27. I-580 at 1,500 cfs



Q100 FEMA FLOWS AT 4,300 CFS

Figure 28. Cross-Section B at 4,300 cfs (FEMA Q_{100})

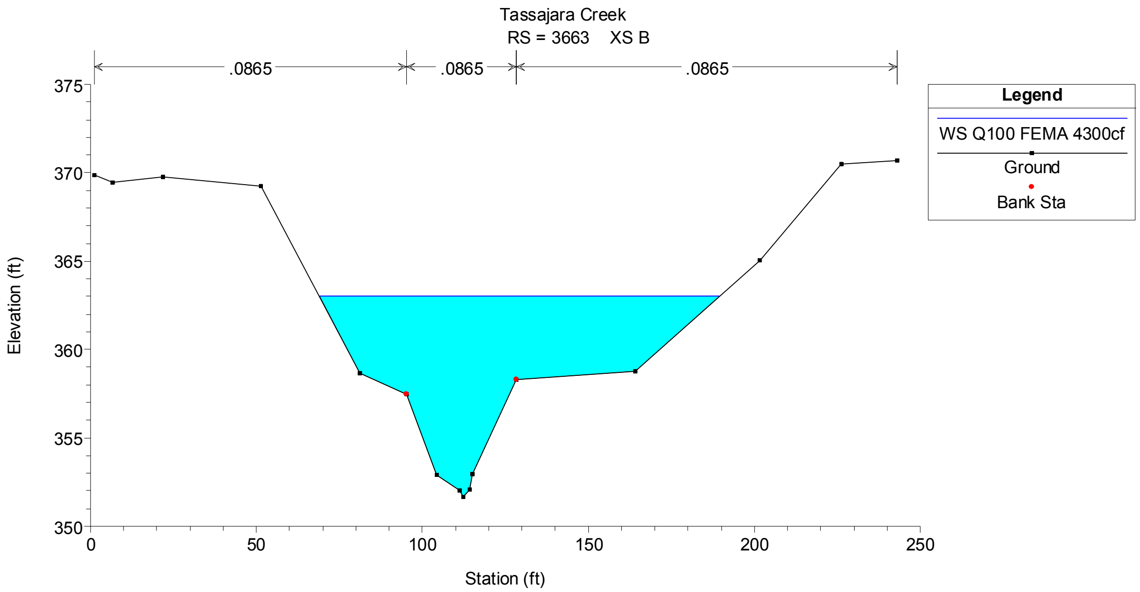


Figure 29. Cross-Section D at 4,300 cfs (FEMA Q_{100})

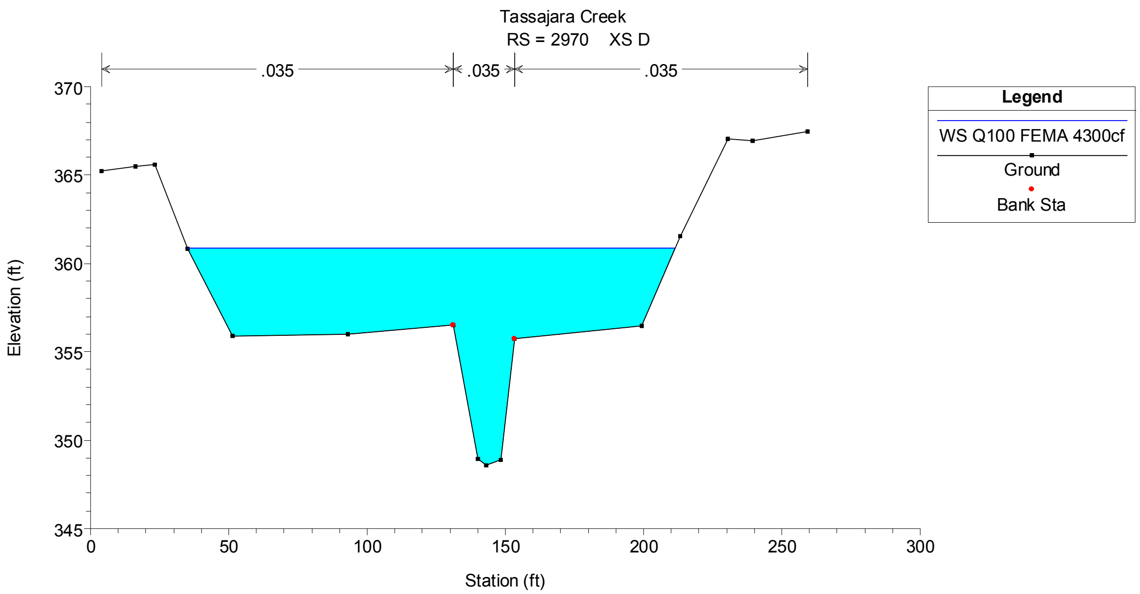


Figure 30. Cross-Section E at 4,300 cfs (FEMA Q_{100})

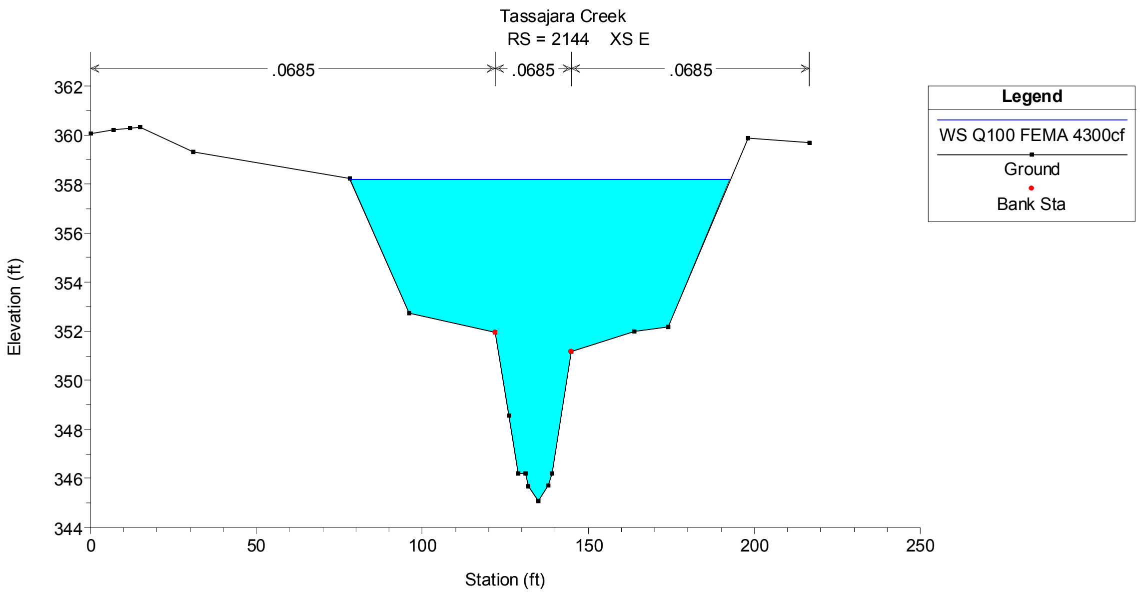


Figure 31. Cross-Section F at 4,300 cfs (FEMA Q_{100})

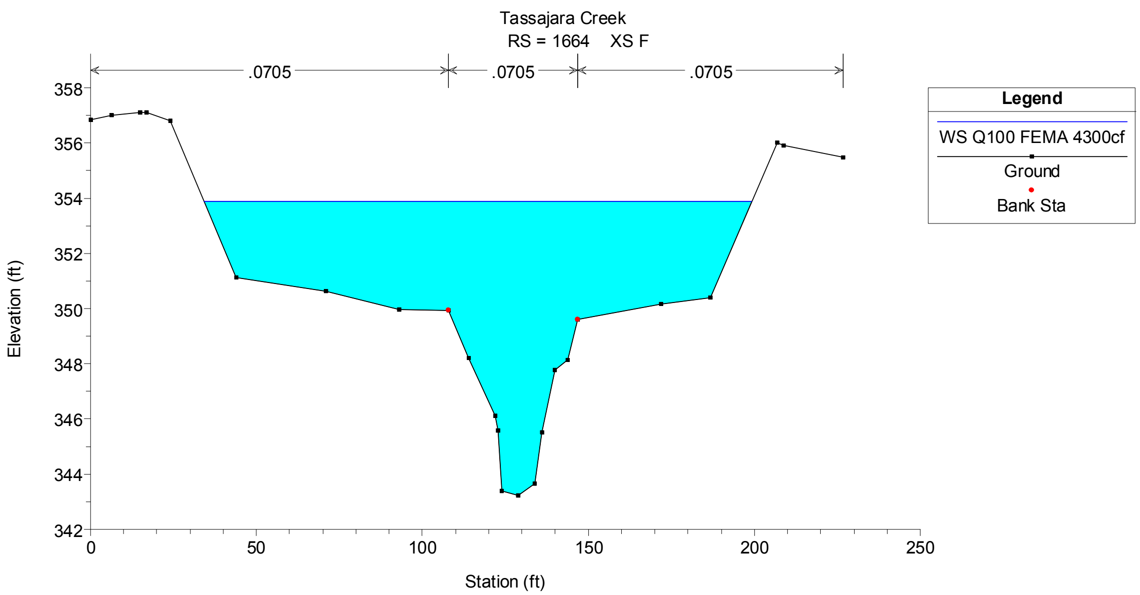


Figure 32. Cross-Section G at 4,300 cfs (FEMA Q_{100})

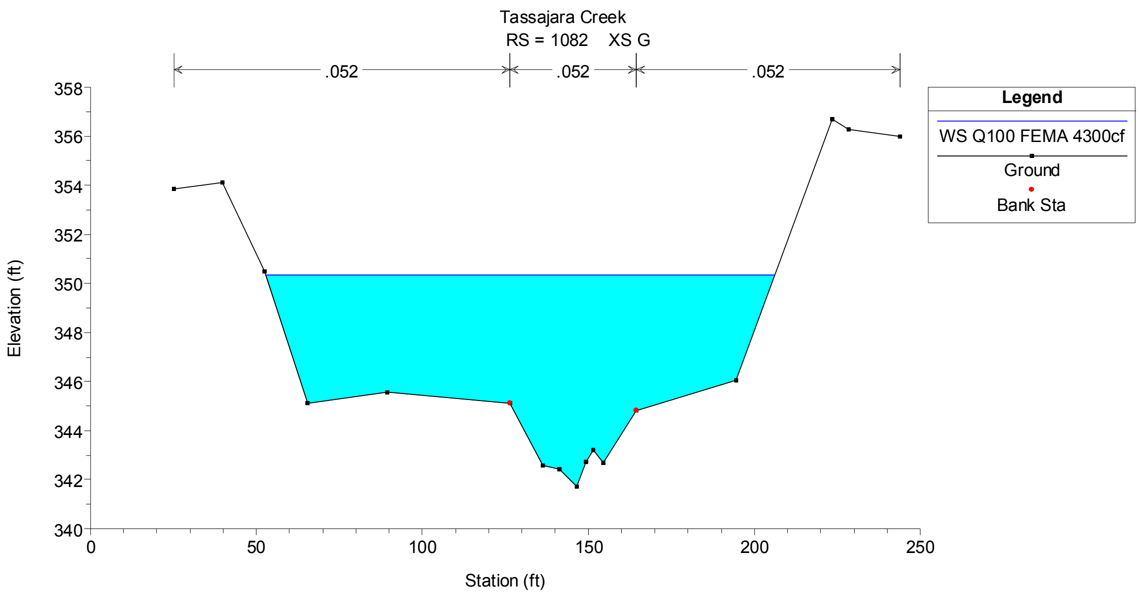


Figure 33. Cross-Section H at 4,300 cfs (FEMA Q_{100})

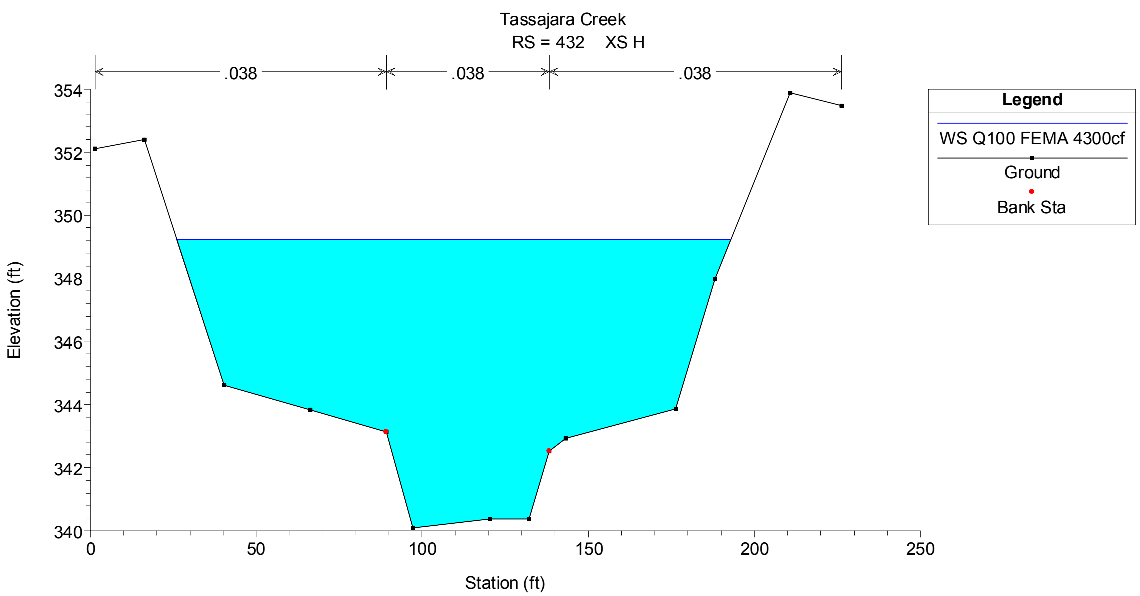
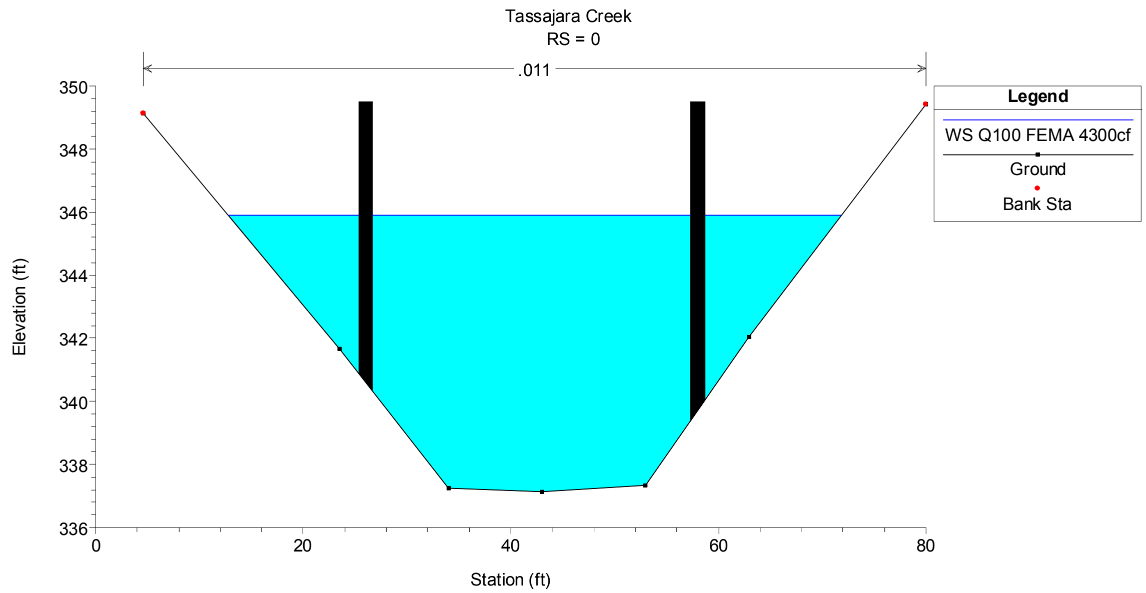


Figure 34. I-580 at 4,300 cfs (FEMA Q_{100})



Q100 ALAMEDA COUNTY FLOWS AT 5,200 CFS

Figure 35. Cross-Section B at 5,200 cfs (AC Q_{100})

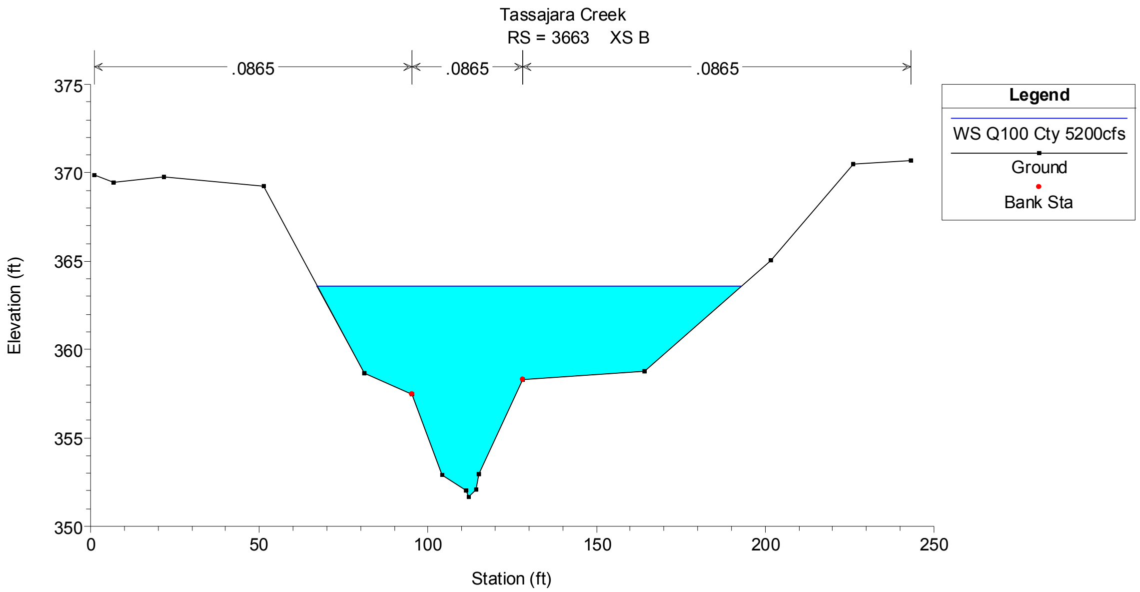


Figure 36. Cross-Section D at 5,200 cfs (AC Q_{100})

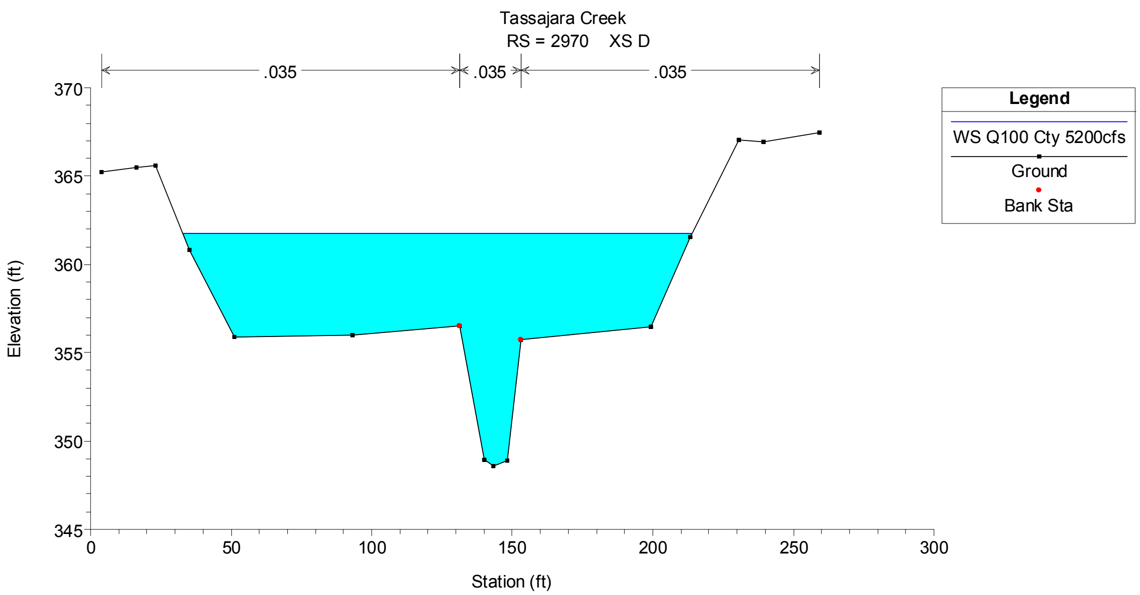


Figure 37. Cross-Section E at 5,200 cfs (AC Q_{100})

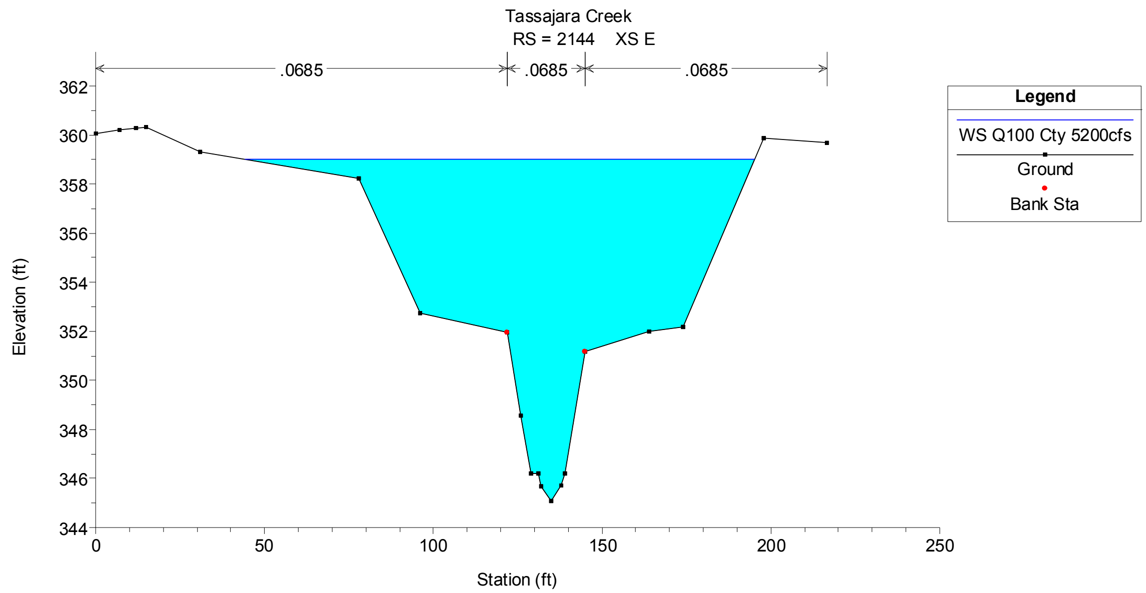


Figure 38. Cross-Section F at 5,200 cfs (AC Q_{100})

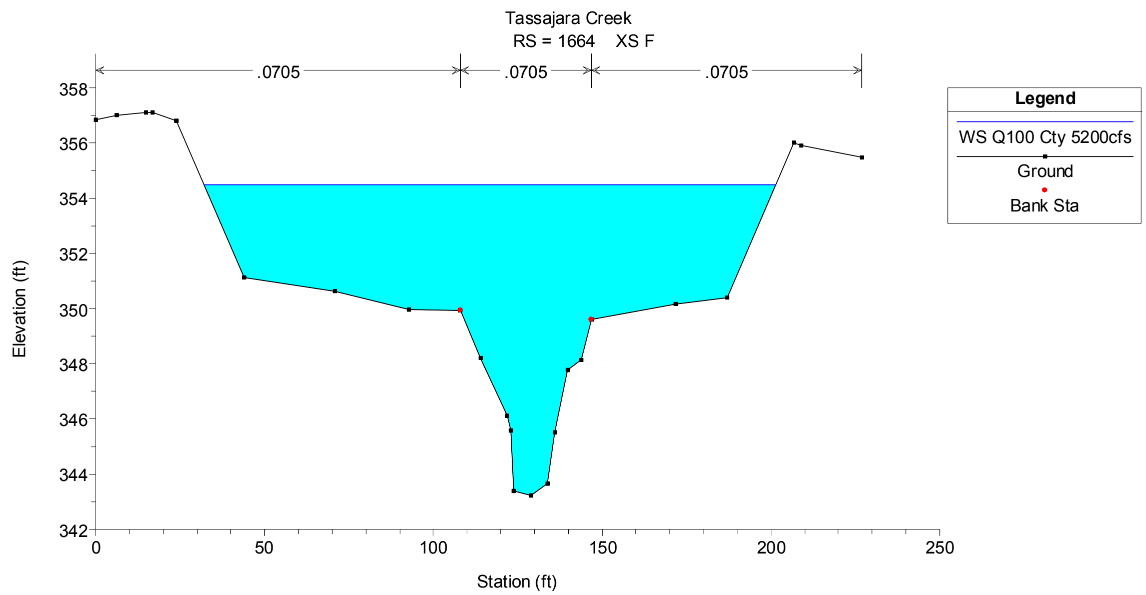


Figure 39. Cross-Section G at 5,200 cfs (AC Q_{100})

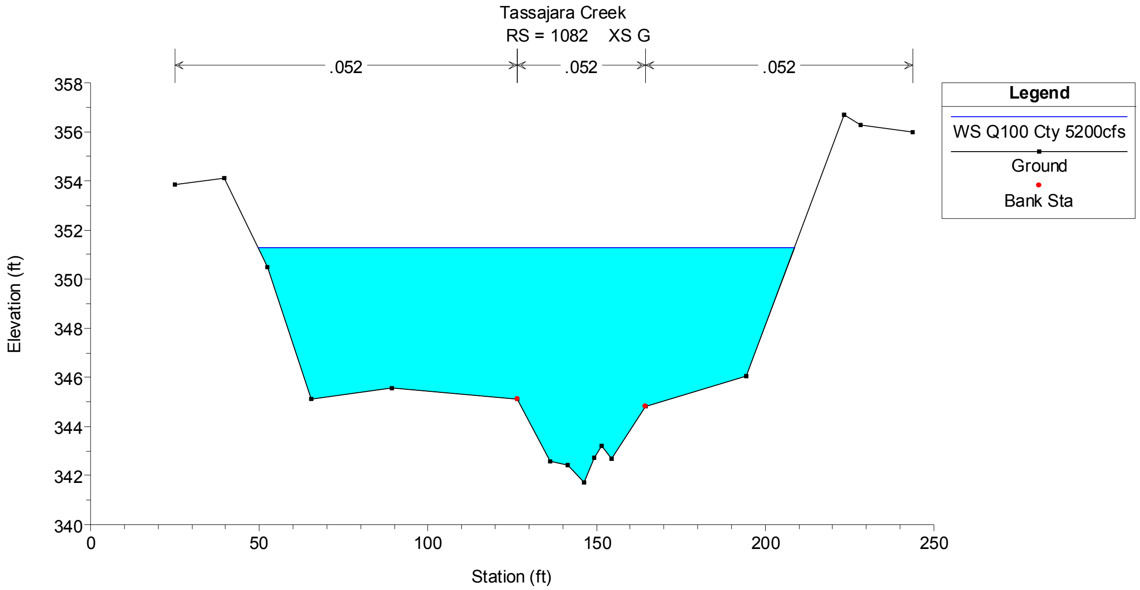


Figure 40. Cross-Section H at 5,200 cfs (AC Q_{100})

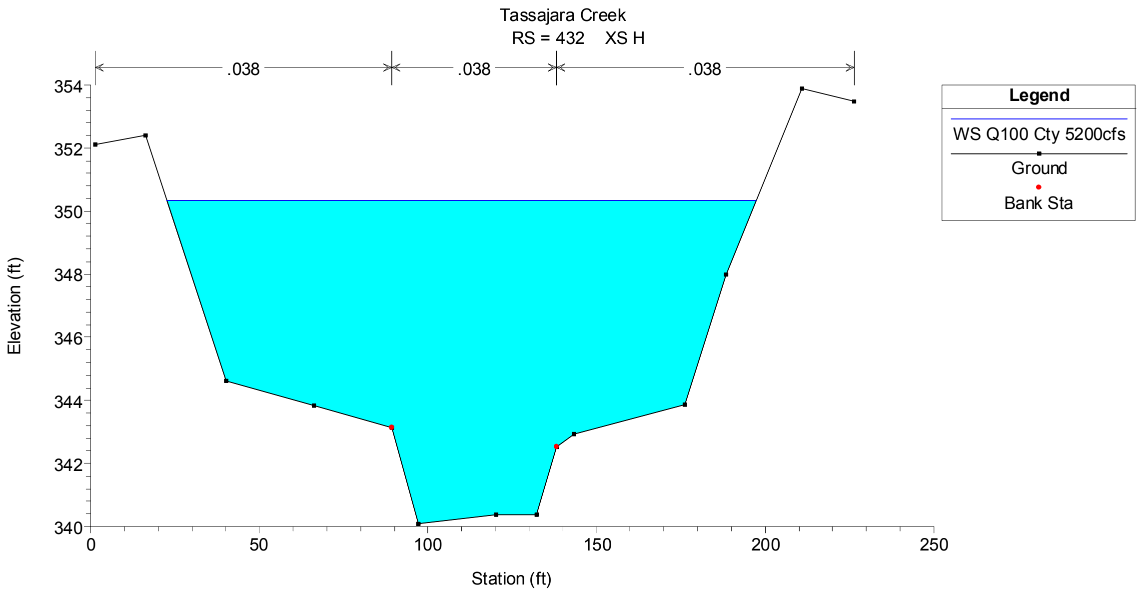
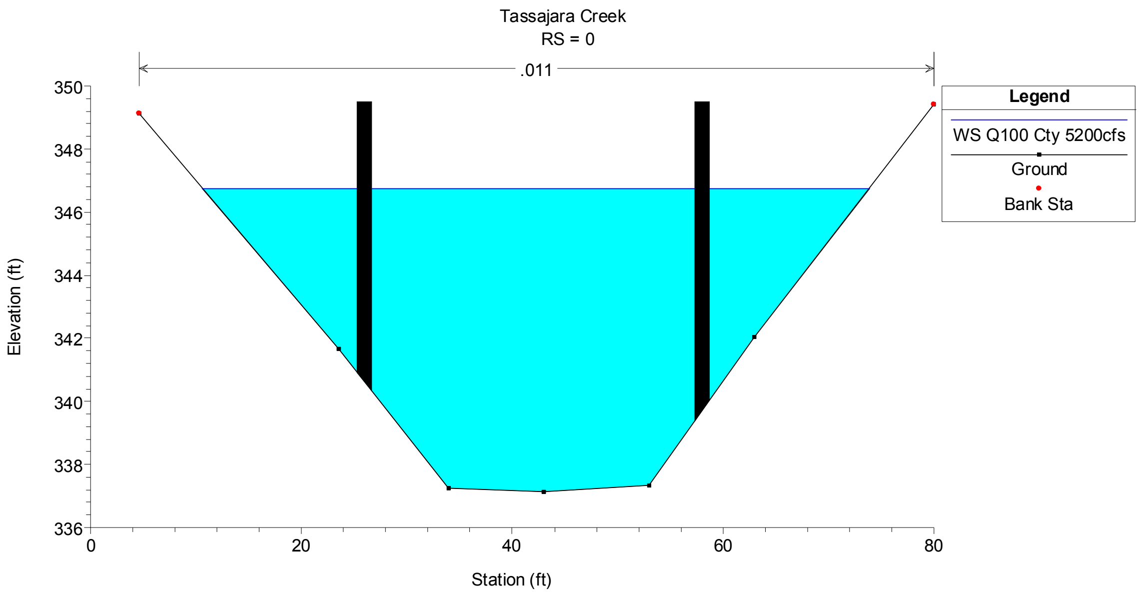


Figure 41. I-580 at 5,200 cfs (AC Q_{100})



PICTURE SERIES A – JULY 21, 2005

Picture A-1

(View of vegetation growth during cross-sectional survey.)

Picture A-2

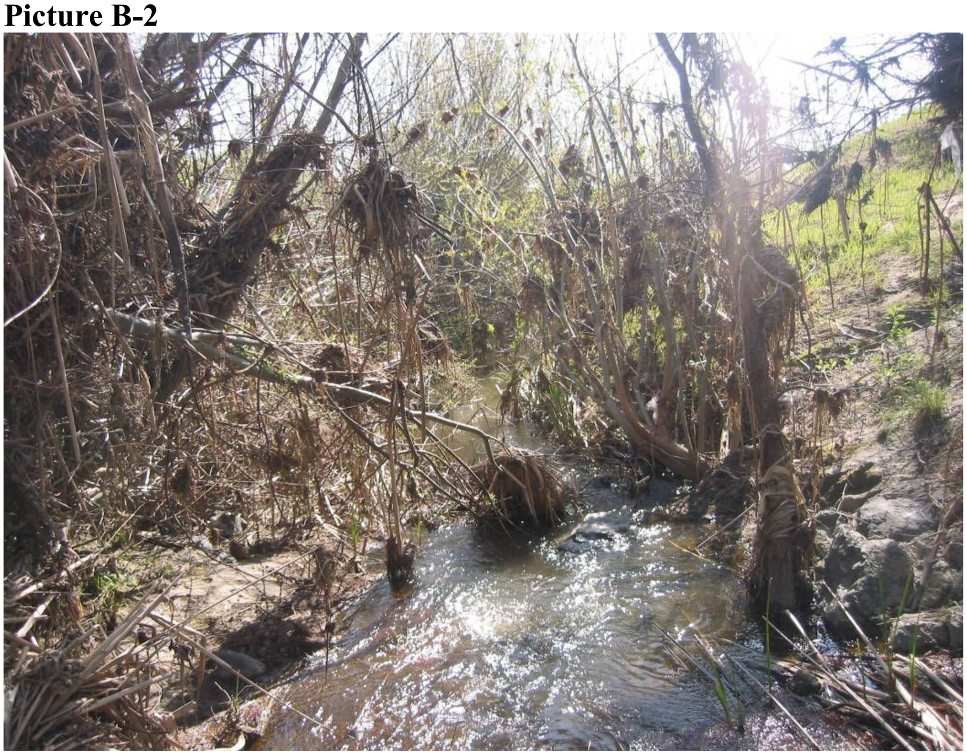
(View of low-flow channel and vegetation.)

PICTURE SERIES B – FEBRUARY 25, 2006

Picture B-1

(Debris from December 2005 storm.)

Picture B-2



(Debris from December 2005 storm.)

Picture B-3

(View of low-flow channel and high water marks.)

Picture B-4

(View of low flow channel and natural floodplain terrace with high water marks.)

Picture B-5

(Debris from December 2005 storm in lower reach.)

Picture B-6

(Sediment deposition on floodplain.)

Picture B-7

(Downstream view from underneath I-580 bridge.)

PICTURE SERIES C – APRIL 23, 2006

Picture C-1

(Downstream view of lower reach from I-580 bridge.)

Picture C-2

(View of sediment deposition on natural floodplain terrace.)

Picture C-3

(Vegetation on natural floodplain terrace, with pencil for scale.)

Picture C-4

(Measurement of sediment deposition depth on natural terrace.)

Picture C-5

(View of the low-flow channel with vegetation and sediment deposition on natural floodplain terrace.)

Picture C-6

(Looking downstream on the natural terrace near Cross-Section E.)