1994 Strawberry Creek Cross-Section Report

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Office of Environment, Health, and Safety
University of California, Berkeley

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Introduction and Purpose

This report is part of an ongoing Strawberry Creek watershed monitoring program at the University of California, Berkeley (UC Berkeley). It describes cross-sectional elevation measurements taken in early 1994 at fifteen locations on the UC Berkeley campus and compares these elevations to 1990 measurements (Huse & Gonzalez, 1990). The purpose of such comparison is to determine the effectiveness of past erosion control measures and to provide the University with additional information needed for prioritizing current erosion control and habitat restoration efforts. This study is a follow-up to erosion studies performed in 1988, 1990, and 1991 as part of the Strawberry Creek Restoration Program (a program initiated in 1987 by the UC Berkeley to restore the habitat quality and beneficial uses of Strawberry Creek that had been degraded from the urban development of the watershed). The primary objective of this study is to provide reliable baseline data for future long-term erosion and habitat monitoring.

Background

Study Site Description: Strawberry Creek is a small 4th order creek draining from the East Bay Hills of the San Francisco Estuary watershed. Strawberry Creek flows from headwaters in Strawberry and Blackberry Canyons (37° 52' 30" N, 122° 15' 30" W, Cities of Berkeley and Oakland, Alameda and Contra Costa Counties, California) through the University of California, Berkeley campus, into the City of Berkeley, and into San Francisco Bay where it enters at the Berkeley Marina (Attachment 1, map 1). The creek is a prominent feature and the major focus of open space on the UC Berkeley main campus, providing drainage and flood conveyance, riparian and wildlife habitats, and opportunities for aesthetic and educational activities. After exiting the campus at Oxford Street, most of the length of Strawberry Creek is culverted until it exits into the San Francisco Bay.

Development of the Strawberry Creek watershed (1163 acres above Oxford St.) and subsequent creek erosion began in the late 1800s when land was cleared for grazing and the establishment of the University of California. By 1987 approximately 40% of the watershed was urbanized, primarily by UC Berkeley institutional land uses (Charbonneau, 1987). This urbanization increased impermeable surface areas in the watershed, resulting in extensive alterations to the natural hydrologic regime, including greater peak flows, higher flood stages downstream, erosion of banks and channels, undercutting of campus buildings and bridges, and degraded aquatic and riparian habitat quality. The conversion of permeable surface area in the watershed to impermeable has slowed, but is still increasing, as can be witnessed from construction projects such as the UC Botanical Gardens parking lot expansion and Witter Field resurfacing. (The 1989 UC Berkeley Long Range Development Plan estimated that development

would only convert approximately 11 acres in the Hill Area, approximately 1% of the watershed, to impermeable surface area by the year 2005/6.) Alterations of the stream bed, such as the creation of the Centennial Drive retention dam, the culverting of the South Fork under the University Stadium (the Big Inch culvert), and channelization, have reduced the threat of flooding and undercutting of campus buildings and infrastructure. However, ongoing inspection and maintenance of these alterations is necessary to maintain these protections.

Erosion is most severe on the South Fork of the creek, due in part to higher flows in this branch. The South Fork also has fewer check-dams, and therefore has a steeper effective gradient (Charbonneau, 1987). Erosion is occurring on many reaches of the North Fork as well, as seen in the right bank of the creek below the Wickson bridge (Attachment 2, photo 1).

There are a number of factors contributing to channel and bank erosion on Strawberry Creek in addition to a flashy hydrologic regime. Lack of substantial riparian vegetation in many areas has lessened bank stability. Because many reaches of the creek are bordered by Redwood trees, it is often more difficult to establish an understory under these shaded and dry conditions (photo 2). In areas where the creek has downcut dramatically, a lowered water table also makes it difficult to plant on the dry upper banks.

In many cases, poorly designed and deteriorating check-dams also contribute to further downstream erosion. This is especially true where the check-dam stabilizes an excessively large vertical drop (Philip Williams and Associates (PWA), 1991). In such cases, the scour pool created beneath the dam may extend further downstream and erode channel banks (photo 3). PWA found that many of the check-dams on campus were currently unstable and that all dams greater than two feet in height were failing. This is due primarily to undercutting of the downstream face, but also from erosion around the sides of dams that are inadequately tied into the banks. Where the force of discharges from pipes and storm-drains is not adequately dissipated with flow-aprons or gravel lining of the channel bed, these outflows may also contribute to scouring of the stream bed or banks (photo 4).

Effects of erosion: Although erosion is a natural process in stream migration, Strawberry Creek is constricted to such an extent by campus structures and landscaping, that any instability in creek banks often directly translates to an instability of buildings or structures on campus. Erosion is also of concern where it degrades habitat for wildlife.

Severe channel incision leads directly to undercutting of retaining walls and bank failure. Bank undercutting has precipitated undercutting of the retention walls behind Electrical Power Distribution Sub-Station 1 and upstream of the Redwood Cribwall (photos 5 and 6). In reaches of natural creek bank, slumping has occurred on both the right and left banks (photo 7). Above the Oxford culvert, two large Redwoods are currently being undermined by erosion on the right bank

(photo 8). When the debris barrier clogs during storm events, flows overbank and create a new channel through the roots of these trees.

An increase in channel gradient and therefore in water velocity has contributed to greater scouring of the creek bed. This disruption to aquatic habitat may flush organisms downstream. If peak flows are too great in magnitude, fish may be unable to find refuges during storm events (Kondolf, personal communication, 1994). Accelerated erosion can also increase sediment load to the creek environment, degrading habitat by clogging creek substrate. Finally, erosion contributes to degraded habitat by disrupting the natural pool/riffle sequence of a stream.

Past Study

An erosion monitoring program was first initiated in 1987 by PWA, in conjunction with the Strawberry Creek Restoration Program. The 1987 PWA study provided recommendations for bank stabilization and prioritized areas most in need of attention. Out of this program came the Redwood Cribwall, located on the South Fork above Stephen's Hall (photo 9), and a number of check dams on the south fork and on the portion of the north fork that flows through the University House Gardens. In 1991, PWA completed a follow-up study of this report, in order both to evaluate the effectiveness of work completed in 1988, and to prioritize remaining areas in need of stabilization. In 1990, an additional study was completed by Susan Huse and Gustavo Gonzalez, two graduate students working with UC Berkeley Department of Landscape Architecture Professor Matt Kondolf. They surveyed at 15 sites on the main campus (map 2). Some of these areas were previously established cross-sections from the PWA study. New sites are denoted with a decimal.

Methodology

Field methods: To determine the extent of changes in cross-section along the creek, the fifteen cross-sections established in 1990 were resurveyed. A Keuffel & Esser #77 0002 builder's level was used to determine elevations. This instrument is currently (May, 1994) maintained by the Department of Landscape Architecture equipment office (Wurster Hall, Rm. 309, 2-3713). A more precise level is also available for use from the Physical Plant - Campus Services (contact: Al Vera, 2-3693).

Before taking any measurements, all right and left banks monuments were located and flagged, using 1990 photographs and descriptions (Attachment III). In cases where the left or right bank pin was missing or could not be relocated, a new pin was established by measuring the distance from the opposite pin and from any other monument of known distance.

Survey protocol: The following protocol was used to determine the profile for each cross-section (see Attachment 4 for further discussion and detail on the principles of surveying):

- Level location- The instrument is placed at a location higher than all points to be measured on the cross-section and then leveled. Where possible, the level is placed at a point from which more than one cross-section is visible.
- Horizontal measurements- To measure horizontal distances along the cross-section, a tape measure is tied between the left and right bank pins, with 0.0 at the left bank pin.
- Profile points- In order to survey a representative profile, points are chosen wherever bank slope changed. (Among the cross-sections, from 11 to 29 points were measured along a transect.)
- True elevations- True elevations along the cross-sections are calculated in two ways: 1) elevation is determined by surveying back to monuments of known elevation [cross-sections 2.1, 4.0, and 31] using the 1990 University of California survey control monuments map [as based on the National Geodetic Vertical Datum of 1929, map 3], or 2) elevations are based on 1990 benchmark elevations [remainder of 1994 cross-sections].

Sources of Error: Possible sources of error correspond primarily to measurement of horizontal distances. For example, variation in the tautness of the measuring tape could lead to discrepancies in 1990 and 1994 profiles. At cross sections where the tape ran a fair distance above the creek bottom, error in reading accurate horizontal distances was also introduced. In sections where a bank monument was re-established, some variation in cross section would be expected. Also, it was not noted whether elevations were taken at the upstream or downstream side of the tape. Although this is probably an insignificant difference, future studies should be consistent in this regard, and should note from which side elevations are taken. For comparative purposes, future monitoring should include elevations taken at the specific horizontal distances used in previous years.

Data analysis: To determine the extent of change between 1990 and 1994 survey results, for each cross-section the elevation is plotted of both years on one graph. Estimations of elevation change were made visually. Points taken on top of the left or right bank pin (OLBP/ORBP) are not included in the elevation profile, as they are not natural features of the cross-section.

Results

The majority (66%) of areas surveyed in the 15 cross-sections did not show elevation changes greater than a few inches (Attachment 5). This difference at these sites is within the margin of experimental error and, therefore, cannot be considered definitive evidence of erosion. The five cross-sections (33%) that do show greater changes in elevation are discussed below.

Cross-sections 16.1, 17.1, and 17.2. In 1988, a low-flow diversion weir was installed in front of the box-culvert on the South Fork downstream of Electrical Power Distribution System Sub-Station 1 (photo 10). The purpose of this structure was to re-direct flow south, back into the former natural creek channel to the south of the culvert. Cross-sections 16.1, 17.1, and 17.2 are all located in the re-established meander bend below this weir (map 2). Due to this alteration in flow pattern, it was expected that these three cross-sections would show the most change from 1990 elevation levels, and in fact that was the case.

Cross-section 17.2, located approximately five meters downstream of the low flow diversion weir, exhibited both channel and bank erosion (Attachment 5, pg. 7). Since 1990, the creek channel has downcut by approximately a foot. This undercutting is contributing to erosion on the left bank along the outside of the meander (photo 11). Three meters further downstream at cross-section 17.1 (photo 12), channel downcutting is also occurring (Attachment 5, pg. 6). However, at this date, no changes in bank elevation were observed. Cross-section 16.1, located at the inside of the meander (photo 13), would be expected through the process of natural stream migrations to experience increased deposition. This is corroborated by the finding that the elevation of the gravel bar and creek bottom have increased by approximately 0.3 feet since 1990 (Attachment 5, pg. 5). In regard to the active channel and bank erosion occurring in cross-sections 17.1 and 17.2, PWA recommended lowering the height of the diversion weir so as to direct peak flows back through the culvert (PWA 1991, pg. 5).

Cross-section 2.1 (photo 2) illustrates the channel downcutting that is occurring in other reaches of the creek as well. The creek bed elevation through this reach has lowered by approximately half a foot since 1990 (Attachment 5, pg. 1). This level of channel incision is an example of how the high energy system of the creek is altering channel morphology.

Cross-sections 31 and 34 emphasize an earlier point whereby the primary objective of this study is to provide reliable baseline data for future monitoring (Attachment 5, pg. 12 & 14). In sections where there is a significant discrepancy between 1990 and 1994 profiles, (a discrepancy which could be attributed to experimental error) cross-sections should be resurveyed to ensure complete data accuracy. At cross-sections 4.0 and 34, where true benchmark elevation was determined by resurveying, 1990 values should be readjusted so that these elevations match for 1990 and 1994.

Suggestions for Future Erosion Control

In order to ensure that stable reaches do not erode and that further erosion on more affected areas is prevented, a number of measures can be taken to reduce creek velocity and downcutting. Reduction in erosion by physical alteration of the creek banks and bed will serve to prevent the

need for costly future repairs of unstable eroded banks near campus structures. This is one of the primary purposes for the University committing resources to Strawberry Creek restoration.

Check Dams: A crucial step in stabilizing channel gradient is the repair of existing check-dams. In their 1991 follow-up report, PWA prioritized the dams most in need of repair and laid out guidelines for the construction of future dams, with attention to maximum height, minimum width, and the extent to which the dam should be tied into the banks (PWA 1991, pg. 6). An updated list of Strawberry Creek erosion control priorities is in progress by PP-CS. Check-dams are currently the most effective grade-control measures for the creek, but they must be maintained in order to function properly.

Energy Dissipation Devices: Other structural methods by which erosion can be controlled are energy dissipation devices, such as flow deflectors and spill aprons. Properly placed, these structures can prevent bank erosion and help recreate a natural riffle/pool sequence beneficial to aquatic organisms. Bank stabilization can also be achieved through a number of different biotechnical techniques such as placement of live fascines (wattles), brush matting, use of gabions, and construction of wooden crib walls (Charbonneau, 1987).

Channel Roughness: Increasing channel roughness is another method by which creek velocity can be decreased. This can be accomplished through placement of boulders in the creek bed, and through riparian revegetation (photo 14). In addition, erosion control through banks stabilization techniques involving riparian habitat restoration serve to increase other campus benefits of the Strawberry Creek restoration effort. Improved wildlife habitat leads to greater aesthetic and educational opportunities. Naturalized stream banks have also been shown to improve water quality by increasing degradation of urban pollutants, such as deposited automobile exhaust. The Strawberry Creek Management Plan provides a listing of recommendations made by the former Landscape Architecture Advisory Subcommittee regarding revegetation. These recommendations emphasize replanting with natives, both for their value as wildlife habitat and for lowered maintenance requirements. Currently, a major obstacle to native revegetation is the prevalence of ivy throughout many reaches of the creek (photo 15). While ivy is fairly effective in bank stabilization, it often precludes the establishment of natives.

Conclusions

One third of the cross-sections measured in this study showed measurable changes due to erosion since previous measurements in 1990. Two thirds of the cross-sections showed only minor alterations. In addition to the measured erosional alterations, undermining of trees, walls, check dams, and other man-mad structures was observed in many stretches of Strawberry Creek.

While reduction in erosion by physical alteration of the creek banks and bed channel measures (check dams, channel roughness, energy dissipation devices, etc.) will improve the aesthetic quality of the creek and provide some level of channel and bank stability, without additional measures they will not be effective in long-term erosion control. In order to achieve more far-reaching goals, land-use planning must be incorporated into a more effective watershed management strategy. In the Long Range Development Plan for the University, the "construction of projects involving additional impervious surface area" is listed as an impact to creek hydrology with possible cumulative long term deleterious effects (Campus Planning Office, 1989, sec. 4.8-1). In order to mitigate the resulting acceleration in creek erosion, it is stated that such additions in impervious areas be "offset by creation of open space areas in the Central Campus Park," and that storm drainage facilities "be designed to limit new flow levels to no greater than current flow rates." In order to control future erosion on the creek, all new construction projects within the watershed should provide a net increase in pervious surface area. Not only will this aid in controlling erosion, but it will also offset some of the other deleterious effects often associated with construction activities, such as increased sedimentation and pollution run-off. In addition, other recommendations of the LRDP Mitigation Measure, such as construction of detention facilities, use of open or porous paving and landscaping to absorb runoff from roofs and walkways, need to be considered in the planning of all construction and landscaping projects.

Recent degradation of South Fork habitat quality (Resh, V. personal communication, 1994) has been associated with increases in water turbidity and streambed siltation. Whether this is due predominately to accelerated erosion or to sedimentation from construction activities deserves further attention and mitigation. Although not evident from this study, there are many reaches in the creek where ongoing erosion has been observed. Improvement in creek quality and beneficial uses requires ongoing monitoring of erosion as well as other physical, biological, and chemical conditions.

Recommendations for Future Research

For future erosion monitoring, establishing new cross-sections in areas of active streamside erosion and in stretches of the creek which are not currently being monitored is recommended. Areas which fit these descriptions are between cross-sections 7.1 and 15.1, and between 17.2 and 22 on the South Fork, and between 34 and 36 on the North Fork.

Project Assistance

Technical and administrative assistance for this project were provided by:

G. Mathais Kondolf,
Associate Professor of Landscape Architecture

Al Vera
Physical Plant- Campus Services

UC Berkeley Strawberry Creek Environmental Quality Committee

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References

Campus Planning Office, 1989, Long Range Development Plan, University of California, Berkeley

Charbonneau, 1987, *Strawberry Creek Management Plan*, Office of Environment, Health, and Safety, University of California, Berkeley

County of Alameda Public Works Agency, 1983, *Revegetation Manual For the Alameda County Flood Control and Water Conservation District Revegetation Program*, Harvey and Stanley Associates, Hayward, CA.

Huse, Susan and Gustavo Gonzalez, 1990, Establishment of permanent cross-sections for monitoring Strawberry Creek, Department of Landscape Architecture, University of California, Berkeley.

Kondolf, Matt, Professor of Landscape Architecture, University of California, Berkeley.

Philip Williams & Associates, principal investigator Jeffrey Haltiner, 1988, *Strawberry Creek bank stabilization study, University of California, Berkeley*, San Francisco, CA.

Philip Williams & Associates, principal investigator Jeffrey Haltiner, 1991, *Strawberry Creek bank stabilization monitoring and follow-up report: University of California, Berkeley,* San Francisco, CA.

Resh, Vince, Professor of Entomology, University of California, Berkeley

Attachment I

Strawberry Creek and Campus Maps

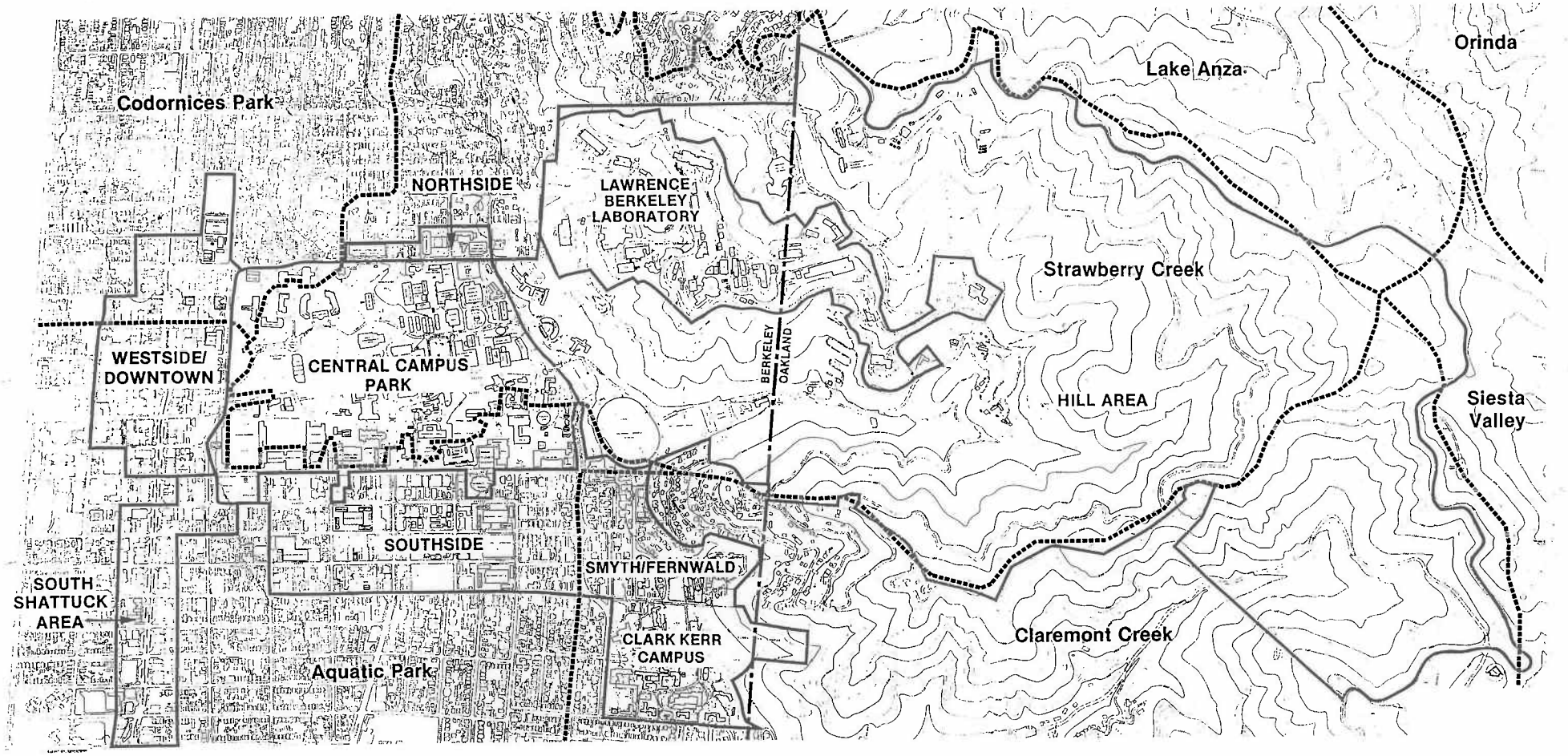
Map I- Strawberry Creek Watershed (2 maps- upper and lower watersheds)

Map II- Study Cross-section Locations

Map III- Survey Control

WATERSHEDS ON CAMPUS

FIGURE 4.8-1



SOURCE: CHARBONNEAU 1987, US GEOLOGICAL SURVEY, 1980

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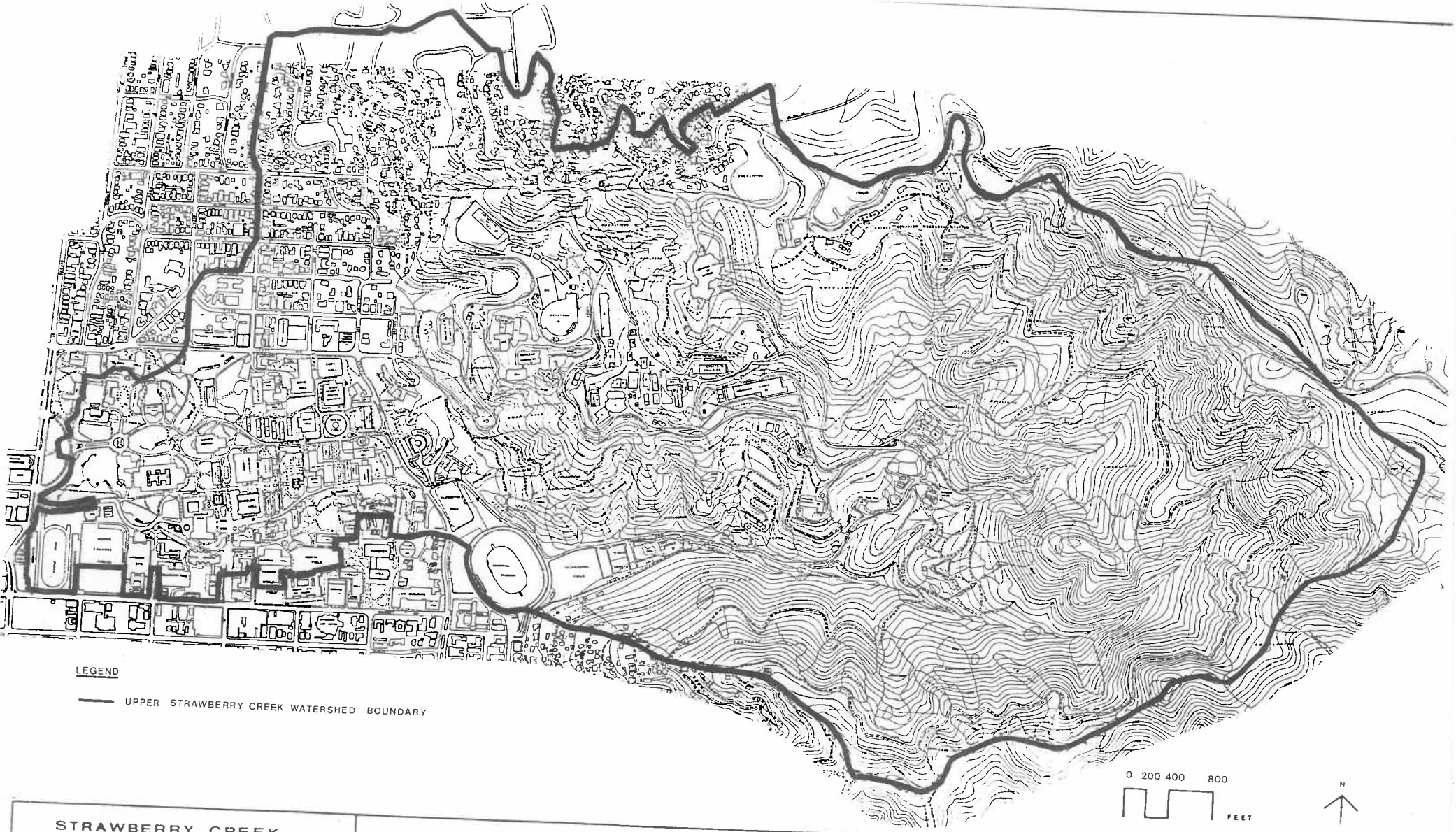
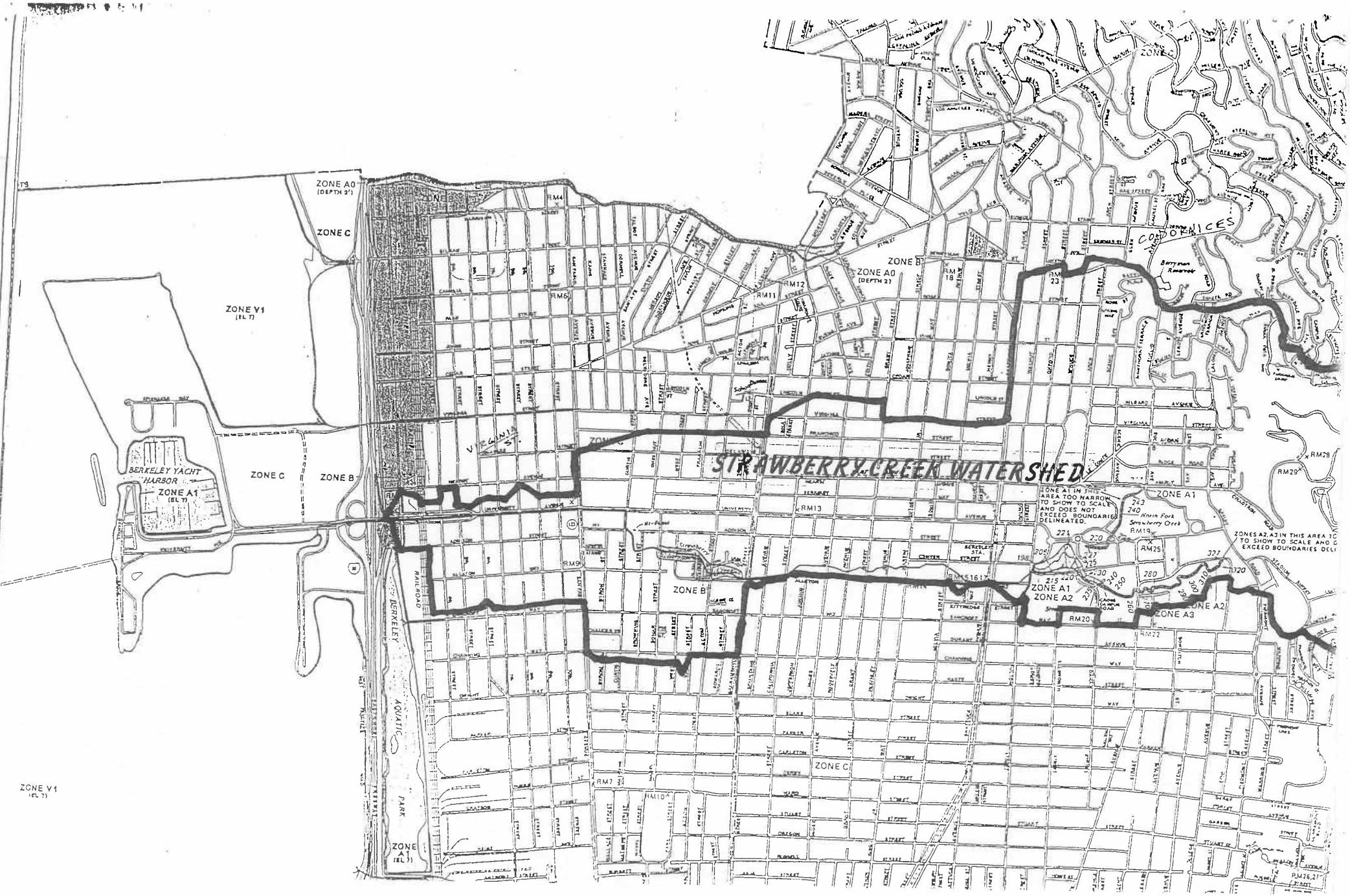


STRAWBERRY CREEK MANAGEMENT PLAN	PREPARED FOR THE OFFICE OF ENVIRONMENTAL HEALTH AND SAFETY UNIVERSITY OF CALIFORNIA, BERKELEY SEPTEMBER 1987	FIGURE 2 UPPER WATERSHED


STRAWBERRY CREEK WATERSHED
ZONE A0 (DEPTH 2')
ZONE C
ZONE V1 (EL 7)
BERKELEY YACHT HARBOR
ZONE A1 (EL 7)
ZONE B
ZONE A0 (DEPTH 2)
ZONE A1
ZONE A2
ZONE A3
ZONE A1 IN THIS AREA TOO NARROW TO SHOW TO SCALE AND DOES NOT EXCEED BOUNDARIES DELINEATED
ZONES A2, A3 IN THIS AREA TO SHOW TO SCALE AND DO EXCEED BOUNDARIES DELI
North Fork Strawberry Creek
Berryman Reservoir
CONDOMINICES
BERKELEY STA.
BERKELEY AQUATIC PARK
RAILROAD
VIRGINIA
RM4
RM6
RM7
RM9
RM10
RM11
RM12
RM13
RM16
RM18
RM19
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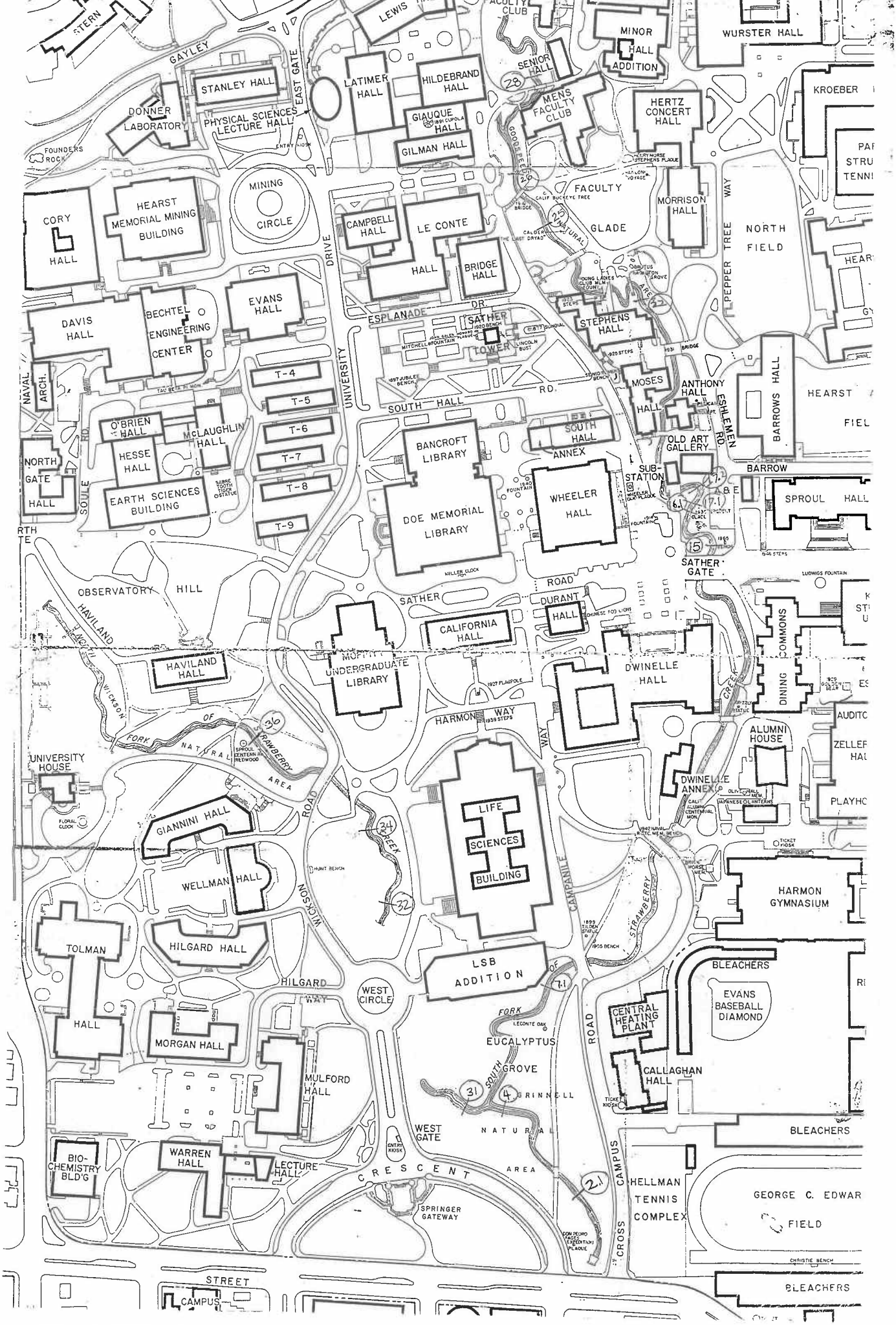
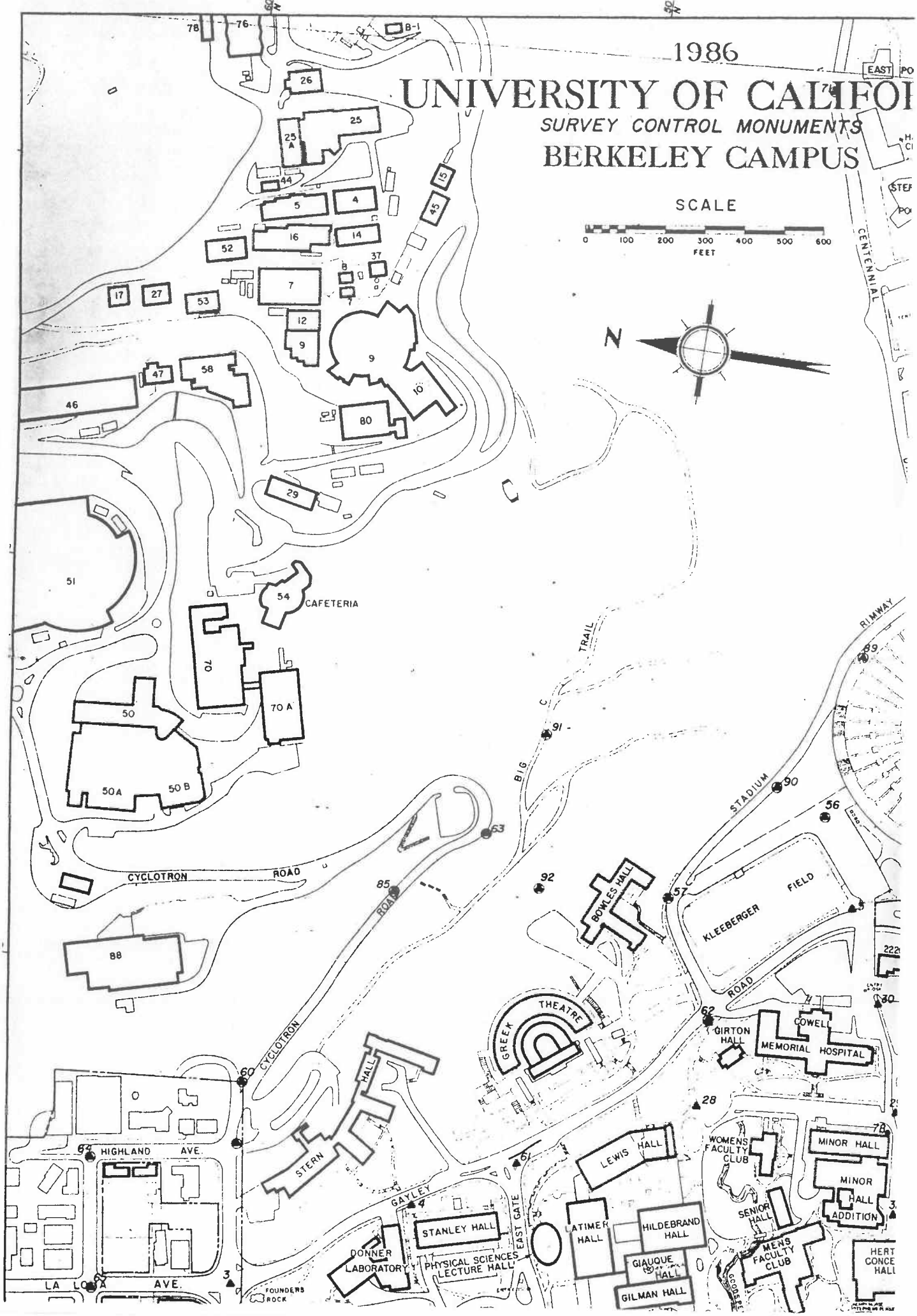
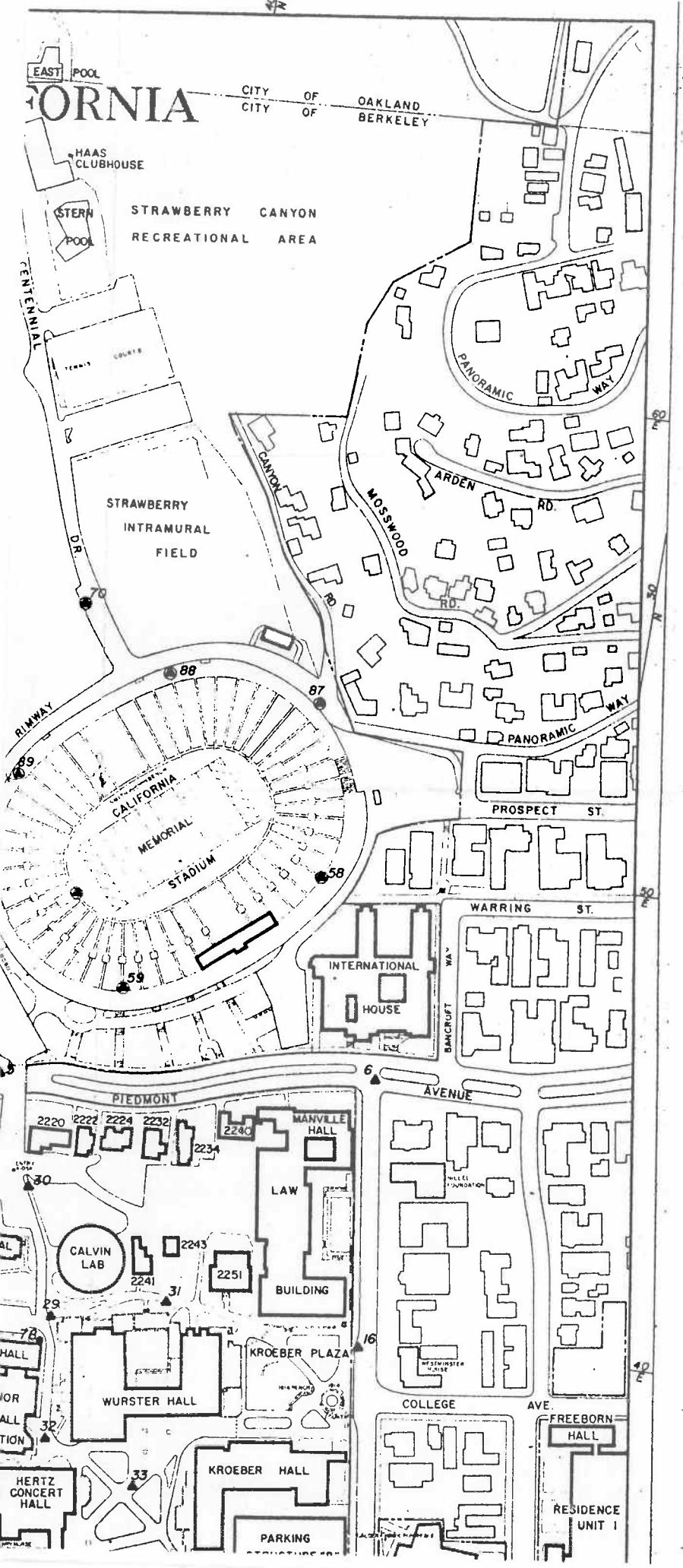
RM23
RM25
RM28
RM29
RM36,37

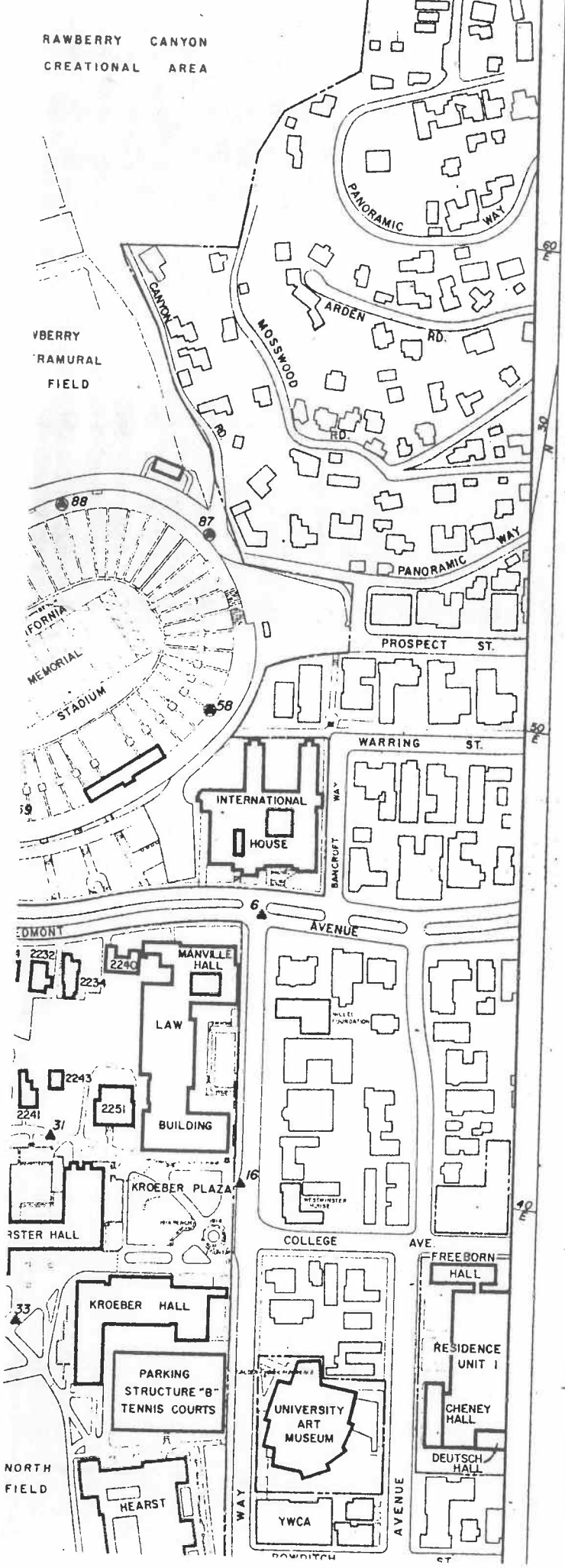

STERN
GAYLEY
STANLEY HALL
PHYSICAL SCIENCES LECTURE HALL
DONNER LABORATORY
FOUNDERS ROCK
EAST GATE
LEWIS
LATIMER HALL
HILDEBRAND HALL
GIAUQUE HALL
GILMAN HALL
FACULTY CLUB
SENIOR HALL
MINOR HALL ADDITION
WURSTER HALL
MENS FACULTY CLUB
HERTZ CONCERT HALL
KROEBER
MINING CIRCLE
CORY HALL
HEARST MEMORIAL MINING BUILDING
CAMPBELL HALL
LE CONTE HALL
BRIDGE HALL
FACULTY GLADE
MORRISON HALL
NORTH FIELD
PEPPER TREE WAY
DAVIS HALL
BECHTEL ENGINEERING CENTER
EVANS HALL
ESPLANADE DR.
SATHER TOWER
STEPHENS HALL
UNIVERSITY DRIVE
NAVAL ARCH.
O'BRIEN HALL
HESSE HALL
McLAUGHLIN HALL
T-4
T-5
T-6
T-7
T-8
T-9
SOUTH HALL RD.
MOSES HALL
ANTHONY HALL
ESHLEMEN RD.
BARROWS HALL
HEARST FIELD
NORTH GATE HALL
SOULE RD.
EARTH SCIENCES BUILDING
BANCROFT LIBRARY
SOUTH HALL ANNEX
OLD ART GALLERY
SUB-STATION
BARROW
SPROUL HALL
DOE MEMORIAL LIBRARY
WHEELER HALL
SATHER GATE
OBSERVATORY HILL
HAVILAND
SATHER ROAD
DURANT HALL
CALIFORNIA HALL
COMMONS
DINING
HAVILAND HALL
MOFFITT UNDERGRADUATE LIBRARY
DWINELLE HALL
HARMON WAY
ALUMNI HOUSE
UNIVERSITY HOUSE
NATURAL AREA
STRAWBERRY CREEK
DWINELLE ANNEX
PLAYHOUSE
GIANNINI HALL
LIFE SCIENCES BUILDING
WELLMAN HALL
WILCKSON ROAD
CAMPANILE WAY
HARMON GYMNASIUM
TOLMAN HALL
HILGARD HALL
LSB ADDITION
BLEACHERS
HILGARD WAY
WEST CIRCLE
EVANS BASEBALL DIAMOND
CENTRAL HEATING PLANT
EUCALYPTUS GROVE
MORGAN HALL
MULFORD HALL
CALLAGHAN HALL
GRINNELL
WEST GATE
NATURAL AREA
BIO-CHEMISTRY BLD'G
WARREN HALL
LECTURE HALL
CRESCENT
SPRINGER GATEWAY
HELLMAN TENNIS COMPLEX
CAMPUS DRIVE
GEORGE C. EDWARDS FIELD
CAMPUS STREET
BLEACHERS


1986
UNIVERSITY OF CALIFOR
SURVEY CONTROL MONUMENTS
BERKELEY CAMPUS
SCALE
0 100 200 300 400 500 600
FEET
N
CENTENNIAL
CAFETERIA
CYCLOTRON ROAD
CYCLOTRON ROAD
BIG TRAIL
STADIUM ROAD
RIMWAY
KLEEBERGER FIELD
BOWLES HALL
GREEK THEATRE
GIRTON HALL
COWELL MEMORIAL HOSPITAL
WOMENS FACULTY CLUB
MINOR HALL
MINOR HALL ADDITION
SENIOR HALL
MENS FACULTY CLUB
LEWIS HALL
STERN HALL
HIGHLAND AVE.
LA LOMA AVE.
GAYLEY
EAST GATE
STANLEY HALL
DONNER LABORATORY
PHYSICAL SCIENCES LECTURE HALL
LATIMER HALL
HILDEBRAND HALL
GIAUQUE HALL
GILMAN HALL
FOUNDERS ROCK
EAST

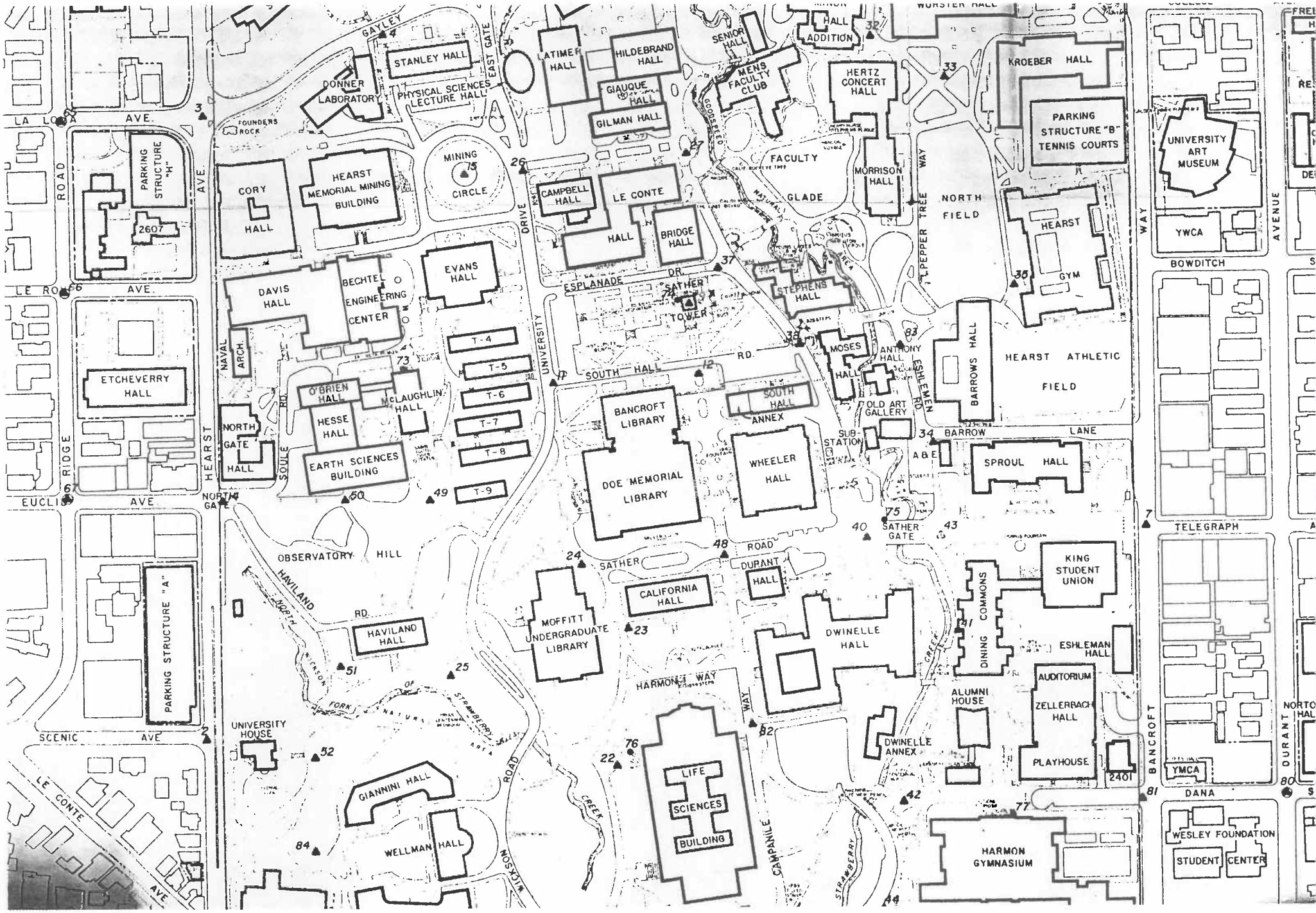


UNIVERSITY OF CALIFORNIA
BERKELEY
SURVEY CONTROL MONUMENTS

#	NORTH	EAST	ELEV.	TYPI
1	5128.58	523.70	218.33	Railroad
2	5398.51	1717.91	314.07	Railroad
3	5607.68	3266.15	397.20	Railroad
4	5185.79	3528.61	388.99	St'd UC
5	4202.44	4417.33	387.24	Brass cc
6	3416.41	4526.18	362.66	Railroad
7	3151.70	2542.97	265.92	Railroad
8	2877.76	526.33	196.31	Brass cc
9	3347.65	553.30	199.21	Brass cc
10	3672.99	622.38	199.10	Brass cc
11	4662.33	2707.05	304.08	St'd UC
12	4306.43	2780.59	304.45	St'd UC
13	5260.94	1187.12	258.63	Railroad
14	5435.71	2302.51	310.90	Brass cc
15	4941.94	3200.80		UC Origl
16	3368.40	3966.66	320.57	Railroad
17	4207.51	548.55	205.96	Brass cc
~~18~~	~~3071.60~~	~~1867.62~~	~~246.75~~	~~Destroye~~
19	4233.88	581.85	207.17	Brass cc
20	4294.34	832.51	224.37	Brass cc
21	4371.99	1305.84	230.43	Brass cc
22	4381.52	1782.40	239.91	St'd UC
23	4401.74	2120.09	265.18	St'd UC
24	4538.10	2264.37	278.35	St'd UC
25	4829.05	1942.88	258.16*	PK nail
26	4805.93	3234.06	342.24	St'd UC
27	4405.85	3323.82	324.88	St'd UC
28	4522.40	3877.63	342.76	St'd UC
29	4015.08	3921.66	342.57	St'd UC
30	4110.96	4192.43	373.01	St'd UC
31	3782.64	3996.38	338.35	St'd UC
32	3992.69	3679.13	326.87	St'd UC
33	3795.63	3608.06	320.40	St'd UC
34	3703.49	2685.02	277.73	Lead &
35	3553.43	3101.87	299.90	Railroad
~~36~~	~~3819.08~~	~~2919.80~~	~~290.07~~	~~Destroye~~
37	4292.03	3045.73	318.45	St'd UC
38	4063.13	2884.44	302.88	St'd UC
39	5646.01	3615.15	446.45*	RR spik
40	3841.53	2424.97	275.95	St'd UC
41	3586.66	2224.51	254.36	St'd UC
42	3667.43	1782.52	240.21	St'd UC
43	3656.66	2456.59		Temp. p
44	3681.96	1510.24	237.42	Brass c
45	3886.13	1300.34	226.04	St'd UC
46	3799.39	1051.42	222.71	St'd UC
~~47~~	~~4060.42~~	~~1925.62~~	~~245.07~~	~~Destroye~~
48	4184.97	2336.84	272.42	St'd UC
49	4922.59	2377.37	294.40	St'd UC
50	5131.60	2350.56	317.06	St'd UC
51	5090.86	1935.98	275.78	St'd UC
52	5127.92	1705.23	286.58	St'd UC
~~53~~	~~5093.79~~	~~1473.60~~	~~279.48~~	~~Destroy~~
54	4908.58	1212.75	258.78	St'd UC
55	4714.40	1242.98	254.15	Railroa
56	4301.55	4649.78	404.34	PK nai
57	4666.90	4380.71	396.78	+ in c
58	3603.60	4937.12	468.73	Stadiur
59	3978.94	4638.27	468.84	Stadiur
60	5649.46	3756.35	469.41*	4" iror
61	4944.81	3665.09	380.80	PK nai
62	4526.77	4073.19	374.21	PK nai
63	5145.14	4475.78	555.91*	1" iror
64	5999.59	3528.98	465.88*	C.O.B.
65	5950.16	3189.74	414.14	C.O.B.
66	5887.47	2759.36	369.11*	C.O.B.
67	5814.06	2255.57	334.93*	C.O.B.
68	2633.74	1284.44	214.75*	C.O.B.
69	2537.03	609.47	191.77*	C.O.B.
70	4192.63	5433.29	477.19	Railroc
71	4568.33	6460.66	520.01	Railroc
72	3866.68	608.38	199.85	PK na
73	BM NGS # M-29		329.678	3" Brc
74	BM NGS # L-29		320.905	3" Brc
75	BM NGS # K-29		270.659	3" Brc
76	BM NGS # H-29		246.627	3" Brc
77	BM NGS # J-29		252.025	3" Brc
78	BM UC # 106		344.75	2" Brc
79	4353.74	2948.67	624.60*	Campc
80	2725.67	1925.77	238.94*	C.O.B.
81	3069.54	1868.47	246.76	PK na
82	4054.24	1905.64	243.55	Railroc
83	3819.07	2919.78	290.05	St'd L
84	5095.62	1474.25	279.56	St'd L
85	5348.36	4293.07	533.38	PK na
~~86~~	~~4105.67~~	~~4827.23~~	~~409.14*~~	~~PK na~~
87	3659.95	5300.51	465.66*	RR sp



#	NORTH	EAST	ELEV.	TYPE
1	5128.58	523.70	218.33	Railroad spike
2	5398.51	1717.91	314.07	Railroad spike
3	5607.68	3266.15	397.20	Railroad spike
4	5185.79	3528.61	388.99	St'd UC mon.
5	4202.44	4417.33	387.24	Brass cap
6	3416.41	4526.18	362.66	Railroad spike
7	3151.70	2542.97	265.92	Railroad spike
8	2877.76	526.33	196.31	Brass cap
9	3347.65	553.30	199.21	Brass cap
10	3672.99	622.38	199.10	Brass cap
11	4662.33	2707.05	304.08	St'd UC mon.
12	4306.43	2780.59	304.45	St'd UC mon.
13	5260.94	1187.12	258.63	Railroad spike
14	5435.71	2302.51	310.90	Brass cap
15	4941.94	3200.80		UC Origin
16	3368.40	3966.66	320.57	Railroad spike
17	4207.51	548.55	205.96	Brass cap
~~18~~	~~3071.60~~	~~1867.62~~	~~246.75~~	~~Destroyed~~
19	4233.88	581.85	207.17	Brass cap
20	4294.34	832.51	224.37	Brass cap
21	4371.99	1305.84	230.43	Brass cap
22	4381.52	1782.40	239.91	St'd UC mon.
23	4401.74	2120.09	265.18	St'd UC mon.
24	4538.10	2264.37	278.35	St'd UC mon.
25	4829.05	1942.88	258.16*	PK nail
26	4805.93	3234.06	342.24	St'd UC mon.
27	4405.85	3323.82	324.88	St'd UC mon.
28	4522.40	3877.63	342.76	St'd UC mon.
29	4015.08	3921.66	342.57	St'd UC mon.
30	4110.96	4192.43	373.01	St'd UC mon.
31	3782.64	3996.38	338.35	St'd UC mon.
32	3992.69	3679.13	326.87	St'd UC mon.
33	3795.63	3608.06	320.40	St'd UC mon.
34	3703.49	2685.02	277.73	Lead & tack
35	3553.43	3101.87	299.90	Railroad spike
~~36~~	~~3819.08~~	~~2919.80~~	~~290.07~~	~~Destroyed~~
37	4292.03	3045.73	318.45	St'd UC mon.
38	4063.13	2884.44	302.88	St'd UC mon.
39	5646.01	3615.15	446.45*	RR spike "Highland"
40	3841.53	2424.97	275.95	St'd UC mon.
41	3586.66	2224.51	254.36	St'd UC mon.
42	3667.43	1782.52	240.21	St'd UC mon.
43	3656.66	2456.59		Temp. point
44	3681.96	1510.24	237.42	Brass cap
45	3886.13	1300.34	226.04	St'd UC mon.
46	3799.39	1051.42	222.71	St'd UC mon.
~~47~~	~~4060.42~~	~~1925.62~~	~~245.07~~	~~Destroyed~~
48	4184.97	2336.84	272.42	St'd UC mon.
49	4922.59	2377.37	294.40	St'd UC mon.
50	5131.60	2350.56	317.06	St'd UC mon.
51	5090.86	1935.98	275.78	St'd UC mon.
52	5127.92	1705.23	286.58	St'd UC mon.
~~53~~	~~5093.79~~	~~1473.60~~	~~279.48~~	~~Destroyed~~
54	4908.58	1212.75	258.78	St'd UC mon.
55	4714.40	1242.98	254.15	Railroad spike
56	4301.55	4649.78	404.34	PK nail
57	4666.90	4380.71	396.78	+ in curb
58	3603.60	4937.12	468.73	Stadium "D"
59	3978.94	4638.27	468.84	Stadium "C"
60	5649.46	3756.35	469.41*	4" iron plate
61	4944.81	3665.09	380.80	PK nail
62	4526.77	4073.19	374.21	PK nail
63	5145.14	4475.78	555.91*	1" iron pipe
64	5999.59	3528.98	465.88*	C.O.B. mon.
65	5950.16	3189.74	414.14	C.O.B. mon.
66	5887.47	2759.36	369.11*	C.O.B. mon.
67	5814.06	2255.57	334.93*	C.O.B. mon.
68	2633.74	1284.44	214.75*	C.O.B. mon.
69	2537.03	609.47	191.77*	C.O.B. mon.
70	4192.63	5433.29	477.19	Railroad spike
71	4568.33	6460.66	520.01	Railroad spike
72	3866.68	608.38	199.85	PK nail
73	BM NGS # M-29		329.678	3" Bronze disc
74	BM NGS # L-29		320.905	3" Bronze disc
75	BM NGS # K-29		270.659	3" Bronze disc
76	BM NGS # H-29		246.627	3" Bronze disc
77	BM NGS # J-29		252.025	3" Bronze disc
78	BM UC # 106		344.75	2" Bronze disc
79	4353.74	2948.67	624.60*	Campanile
80	2725.67	1925.77	238.94*	C.O.B. mon.
81	3069.54	1868.47	246.76	PK nail
82	4054.24	1905.64	243.55	Railroad spike
83	3819.07	2919.78	290.05	St'd UC mon.
84	5095.62	1474.25	279.56	St'd UC mon.
85	5348.36	4293.07	533.38	PK nail
~~86~~	~~4105.67~~	~~4827.23~~	~~409.14*~~	~~PK nail Destroyed~~
87	3659.95	5300.51	465.66*	RR spike "Stadium"
88	3991.25	5317.09	470.80*	Stadium A-1
89	4269.07	5056.67	471.00	Stadium B-1
90	4433.56	4703.02	435.49*	7" nail in walk
91	5035.77	4746.70	565.47*	3/4" rebar
92	4992.27	4353.83	484.50*	3/4" rebar
93	4054.41	894.95		1.5" IP w/brass cap
94	4123.92	1145.59		1.5" IP w/brass cap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LE ROY AVE.
EUCLID AVE
SCENIC AVE
LE CONTE AVE
RIDGE ROAD
HEARST AVE.
GAYLEY
EAST GATE
NORTH GATE
SOULE RD.
UNIVERSITY DRIVE
ESPLANADE DR.
SOUTH HALL RD.
SATHER ROAD
HAVILAND RD.
HARMON WAY
CAMPANILE WAY
STRAWBERRY CREEK
NICKSON ROAD
NORTH FORK OF STRAWBERRY CREEK
NATURAL AREA
PEPPER TREE WAY
ESHLEMAN RD.
BARROW LANE
BOWDITCH
TELEGRAPH
BANCROFT
DANA
DURANT AVENUE
FOUNDERS ROCK
DONNER LABORATORY
STANLEY HALL
PHYSICAL SCIENCES LECTURE HALL
LATIMER HALL
HILDEBRAND HALL
GIAUQUE HALL
GILMAN HALL
SENIOR HALL
MENS FACULTY CLUB
HERTZ CONCERT HALL
KROEBER HALL
PARKING STRUCTURE "B" TENNIS COURTS
UNIVERSITY ART MUSEUM
YWCA
PARKING STRUCTURE "H"
2607
CORY HALL
HEARST MEMORIAL MINING BUILDING
MINING CIRCLE
CAMPBELL HALL
LE CONTE HALL
BRIDGE HALL
FACULTY GLADE
MORRISON HALL
NORTH FIELD
HEARST GYM
DAVIS HALL
BECHTEL ENGINEERING CENTER
EVANS HALL
SATHER TOWER
STEPHENS HALL
MOSES HALL
ANTHONY HALL
BARROWS HALL
HEARST ATHLETIC FIELD
NAVAL ARCH.
ETCHEVERRY HALL
O'BRIEN HALL
HESSE HALL
McLAUGHLIN HALL
NORTH GATE HALL
EARTH SCIENCES BUILDING
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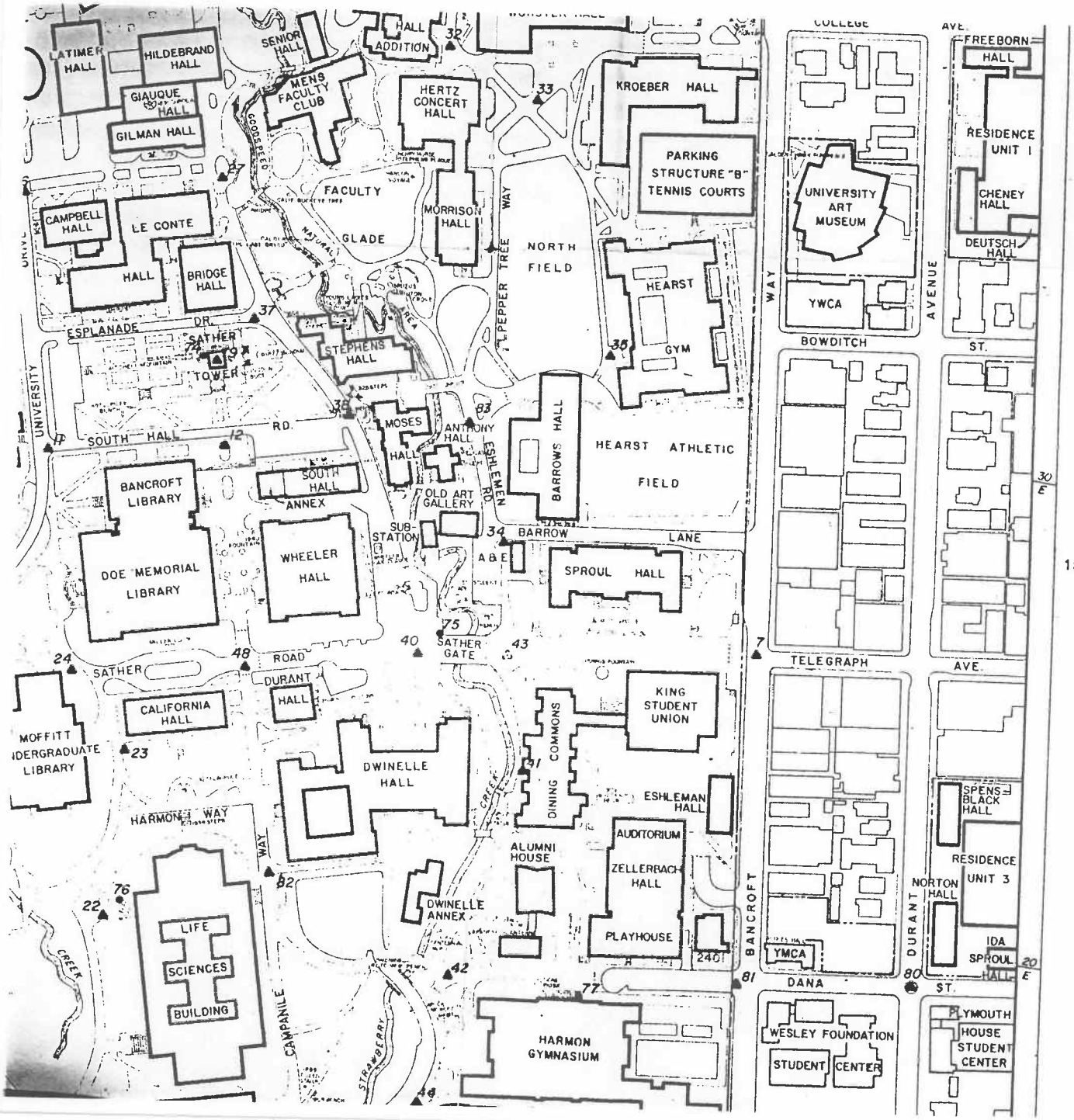
BANCROFT LIBRARY
SOUTH HALL ANNEX
OLD ART GALLERY
SUB-STATION
A&E
SPROUL HALL
DOE MEMORIAL LIBRARY
WHEELER HALL
SATHER GATE
OBSERVATORY HILL
CALIFORNIA HALL
DURANT HALL
KING STUDENT UNION
DINING COMMONS
MOFFITT UNDERGRADUATE LIBRARY
DWINELLE HALL
ESHLEMAN HALL
HAVILAND HALL
AUDITORIUM
ZELLERBACH HALL
ALUMNI HOUSE
PARKING STRUCTURE "A"
UNIVERSITY HOUSE
DWINELLE ANNEX
PLAYHOUSE
2401
GIANNINI HALL
LIFE SCIENCES BUILDING
WELLMAN HALL
HARMON GYMNASIUM
WESLEY FOUNDATION STUDENT CENTER



77	BM NGS # J-29		252.025	3" Bronze disc
78	BM UC # 106		344.75	2" Bronze disc
79	4353.74	2948.67	624.60*	Campanile
80	2725.67	1925.77	238.94*	C.O.B. mon.
81	3069.54	1868.47	246.76	PK nail
82	4054.24	1905.64	243.55	Railroad spike
83	3819.07	2919.78	290.05	St'd UC mon.
84	5095.62	1474.25	279.56	St'd UC mon.
85	5348.36	4293.07	533.38	PK nail
~~86~~	~~4105.67~~	~~4827.23~~	~~409.14*~~	~~PK nail Destroyed~~
87	3659.95	5300.51	465.66*	RR spike "Stadium"
88	3991.25	5317.09	470.80*	Stadium A-1
89	4269.07	5056.67	471.00	Stadium B-1
90	4433.56	4703.02	435.49*	7" nail in walk
91	5035.77	4746.70	565.47*	3/4" rebar
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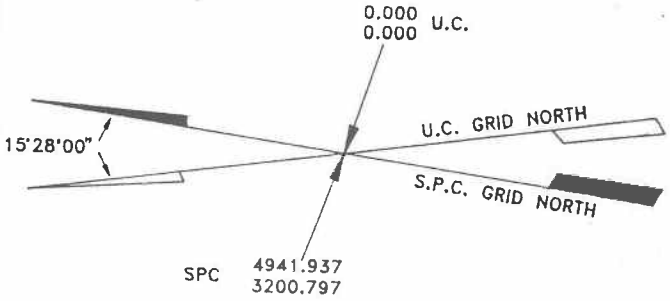
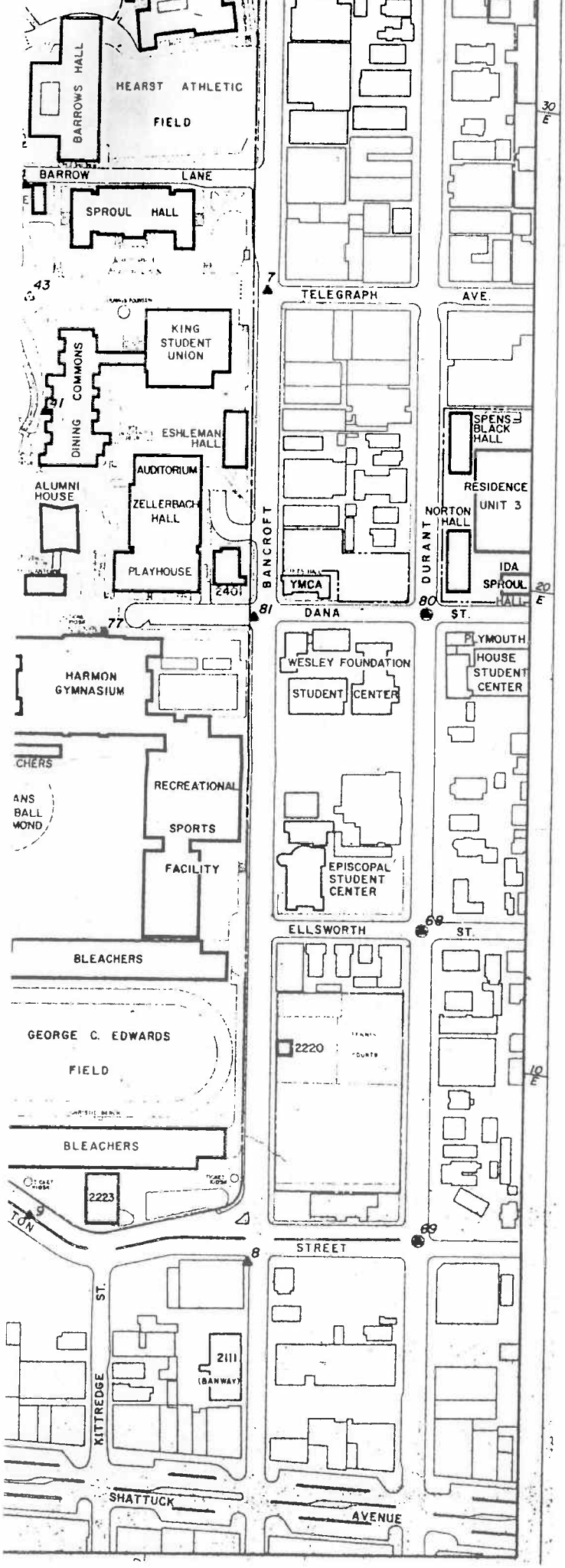
93	4054.41	894.95		1.5" IP w/brass cap
94	4123.92	1145.59		1.5" IP w/brass cap

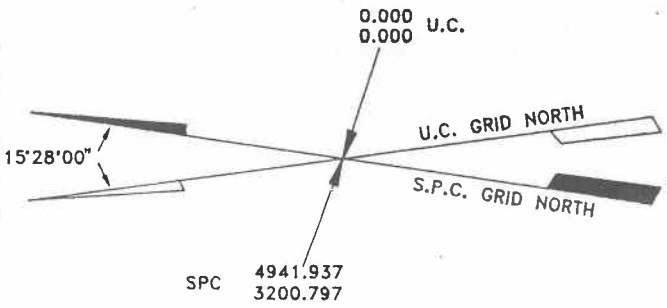


GRID TRANSLATION EQUATION

NOTES

1. Standard monument is a 3/4" aluminum rod from 18" to 60" in length with a 3 1/4" aluminum cap stamped "UNIVERSITY OF CALIFORNIA SURVEY MONUMENT" and monument number composed of year set and monument number: 85-48.
2. Co-ordinates are based upon the California Co-ordinate System, Zone 3. (truncated)
3. State Plane Grid mapping angle at 85-48: -1°04'36.54"
4. To obtain ground distance multiply grid distance by 1.0000817.
5. The mapping angle and scale factor were calculated at monument 85-48.





GRID TRANSLATION EQUATION

NOTES

1. Standard monument is a 3/4" aluminum rod from 18" to 60" in length with a 3 1/4" aluminum cap stamped "UNIVERSITY OF CALIFORNIA SURVEY MONUMENT" and monument number composed of year set and monument number: 85-48.
2. Co-ordinates are based upon the California Co-ordinate System, Zone 3. (truncated)
3. State Plane Grid mapping angle at 85-48: -1°04'36.54"
4. To obtain ground distance multiply grid distance by 1.0000817.
5. The mapping angle and scale factor were calculated at monument 85-48.
6. Elevations are based upon the National Geodetic Vertical Datum of 1929 (NGVD-29).
7. To convert map elevations to elevations based on U.C. datum, subtract 4.12 feet.
8. An elevation followed by an asterisk (*) denotes trigonometric elevation.

SYMBOLS

- ▲ MAIN CAMPUS SURVEY NET
- TRAVERSE STATION
- ● ELEVATION ONLY

LICENSED LAND SURVEYOR DAVID J. RUSSELL STATE OF CALIFORNIA

DAVID J. RUSSELL L.S. 3174 (6-30-92)

SURVEY CONTROL MONUMENTS

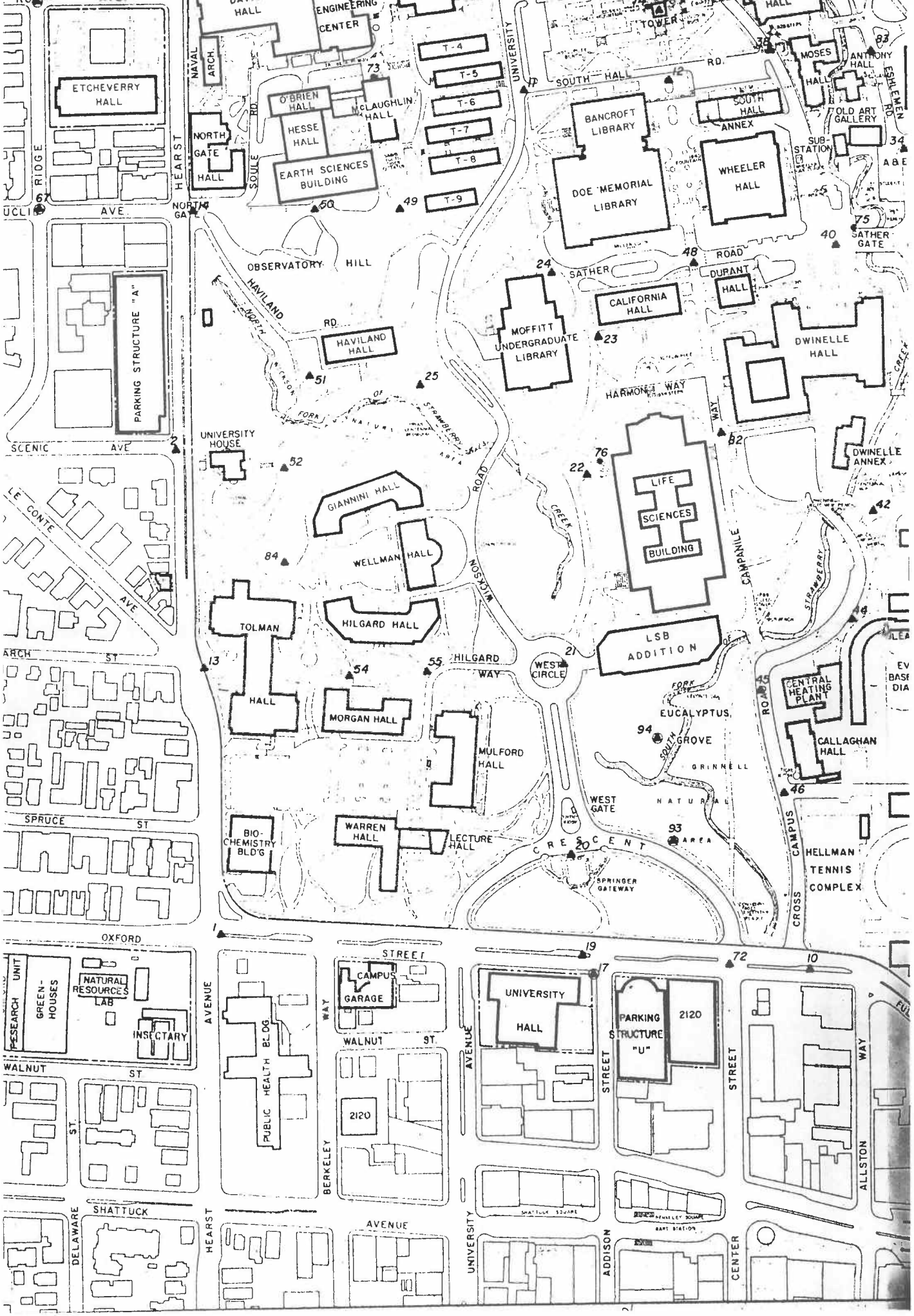
UNIVERSITY OF CALIFORNIA

BERKELEY, CALIFORNIA

NOVEMBER 1990

SCALE: 1" = 200'

DWG. NO. 0639


ENGINEERING CENTER
TOWER
T-4
T-5
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UNIVERSITY
SOUTH HALL
RD.
MOSES HALL
ANTHONY HALL
ESHLEMEN RD.
OLD ART GALLERY
SUB-STATION
ETCHEVERRY HALL
NAVAL ARCH.
O'BRIEN HALL
HESSE HALL
McLAUGHLIN HALL
NORTH GATE HALL
SOULE RD.
EARTH SCIENCES BUILDING
BANCROFT LIBRARY
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UCLA
AVE.
NORTH GATE
OBSERVATORY HILL
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PARKING STRUCTURE "A"
SATHER GATE
SATHER ROAD
DURANT HALL
CALIFORNIA HALL
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DWINELLE HALL
HARMON WAY
DWINELLE ANNEX
UNIVERSITY HOUSE
SCENIC AVE
LE CONTE AVE
GIANNINI HALL
WELLMAN HALL
LIFE SCIENCES BUILDING
WICKSON ROAD
CAMPANILE WAY
TOLMAN HALL
HILGARD HALL
HILGARD WAY
WEST CIRCLE
LSB ADDITION
CENTRAL HEATING PLANT
EUCALYPTUS GROVE
GRINNELL NATURAL AREA
CALLAGHAN HALL
MORGAN HALL
MULFORD HALL
WEST GATE
ARCH ST
SPRUCE ST
BIO-CHEMISTRY BLD'G
WARREN HALL
LECTURE HALL
CRESCENT
SPRINGER GATEWAY
CROSS CAMPUS
HELLMAN TENNIS COMPLEX
OXFORD
RESEARCH UNIT
GREEN-HOUSES
NATURAL RESOURCES LAB
INSECTARY
AVENUE
PUBLIC HEALTH BLDG.
WAY
CAMPUS GARAGE
STREET
WALNUT ST.
2120
BERKELEY
UNIVERSITY HALL
PARKING STRUCTURE "U"
STREET
ALLSTON WAY
SHATTUCK SQUARE
BERKELEY SQUARE
BART STATION
DELAWARE ST.
SHATTUCK AVENUE
HEARST
UNIVERSITY
ADDISON
CENTER

Attachment II

1994 Photographs

Photo 1 (top): Upstream view of North Fork, below Wickson Bridge. Note the right bank erosion.

Photo 2 (bottom): Cross-section 2.1. Main Fork, downstream view, above Oxford Culvert. Note the absence of significant understory.

Photo 3 (top): Upstream view of North Fork check dam, immediately above confluence.

Photo 4 (bottom): Ground water de-watering pipes below Wickson Bridge. North Fork, left bank.

Photo 5 (top): Left bank view of South Fork, behind Electrical Power Distribution Substation 1. Note undercutting of retention wall foundation.

Photo 6 (bottom): Upstream view of South Fork at and above Redwood Cribwall. Note undercutting of left bank wall.

Photo 7 (top): Upstream view of South Fork behind Anthony Hall.

Photo 8 (bottom): Upstream view of Main Fork, as seen from Oxford Culvert. Note right bank erosion.

Photo 9 (top): Redwood Cribwall. Left bank of South Fork below Stephens Hall.

Photo 10 (bottom): Downstream view of diversion weir on South Fork above Sather Gate.

Photo 11 (top): Cross-section 17.2, South Fork, looking upstream.

Photo 12 (bottom): Cross-section 17.1, South Fork, looking upstream.

Photo 13 (top): Cross-section 16.1, South Fork, looking upstream toward Box Culvert.

Photo 14 (bottom): Riparian revegetation along South Fork above Stephens Hall, looking upstream.

Photo 15 (top): Ivy covering left and right banks of South Fork, north of Faculty Glade.

Attachment III

Cross-Sections Monument Photographs

CROSS-SECTION 2.1- 1990 photograph

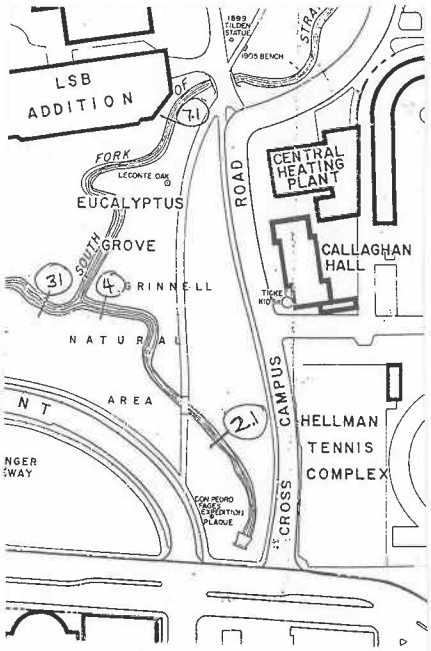
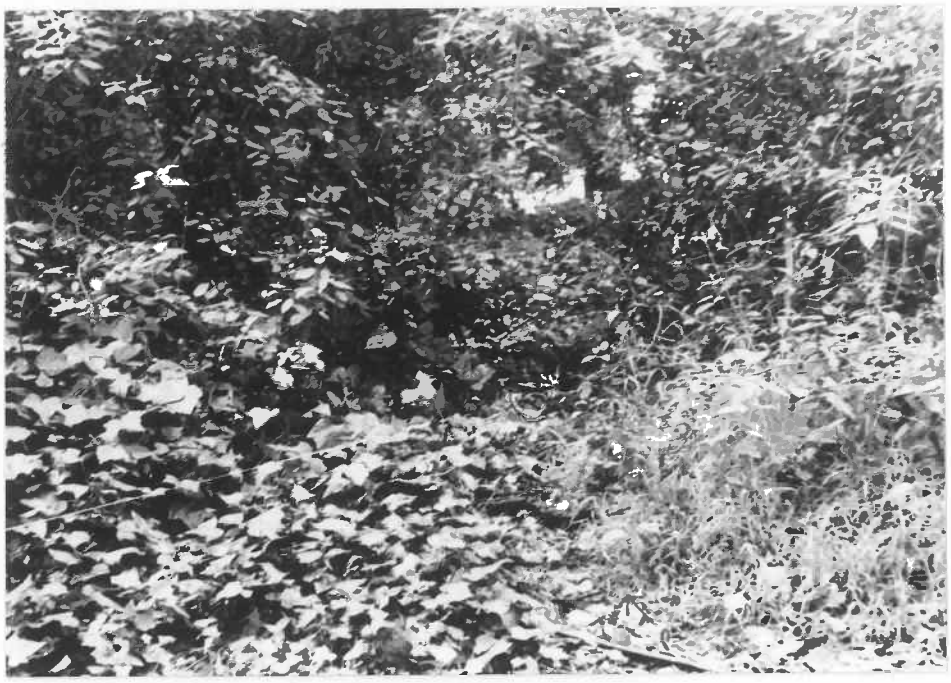


Photo XS 2.1 RBP: Right Bank Pin- in tree in ivy

Photo XS 2.1 LBP: Left Bank Pin- First tree upstream from pumphouse

CROSS-SECTION 4 - 1990 photograph



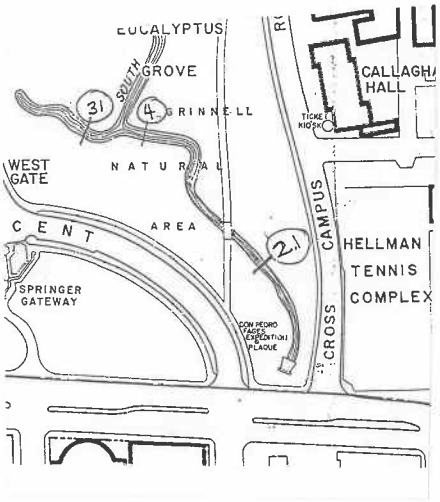


Photo XS 4 RBP : Right Bank Pin (1990 note- in dirt path below ivey & grass)

Photo XS 4 LBP: Left Bank Pin- the LBP is rebar 3" above ground in dirt beneath bay shrub adjacent to the former LBP, which was a stick with a nail in top (1990 note- in dirt beneath bay shrub adjacent to former LBP= stick with nail in top, probably not accurate)

CROSS-SECTION 4 - 1990 photograph

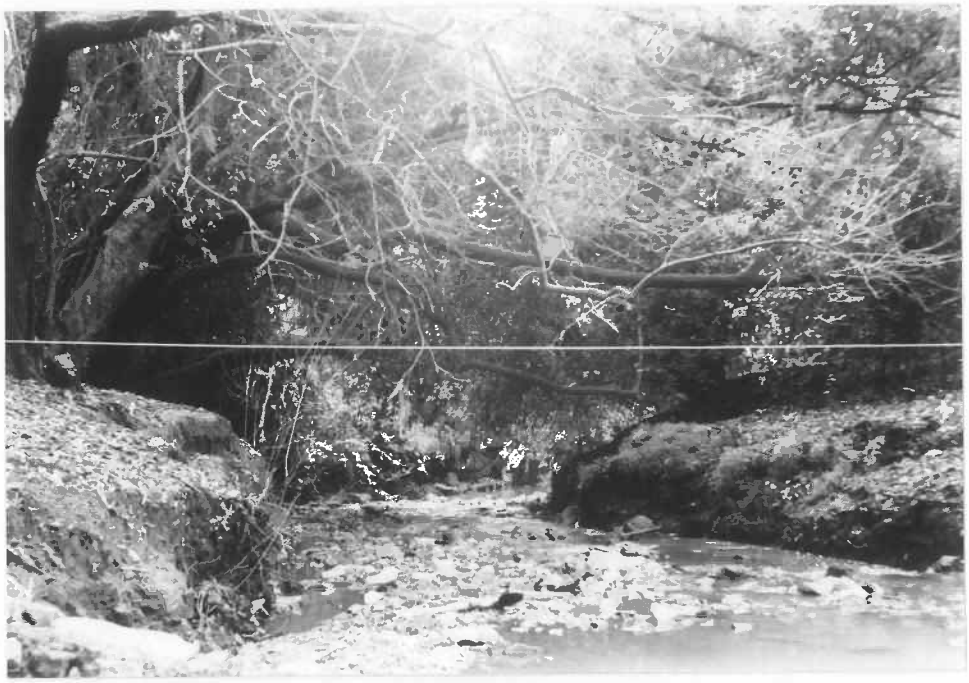


Photo XS 4RBP : Downstream view of section and tape.

Photo XS 4 LBP: ~~Upstream~~ DOWNSTREAM view of section and tape.

CROSS-SECTION 7.1 - 1990 photograph

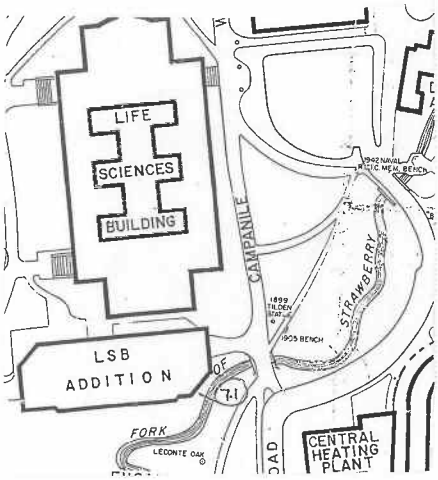


Photo XS 7.1 RBP : Right Bank Pin- foot on nail

Photo XS 7.1 LBP: Left Bank Pin- looking upstream

CROSS-SECTION 15- 1990 photograph

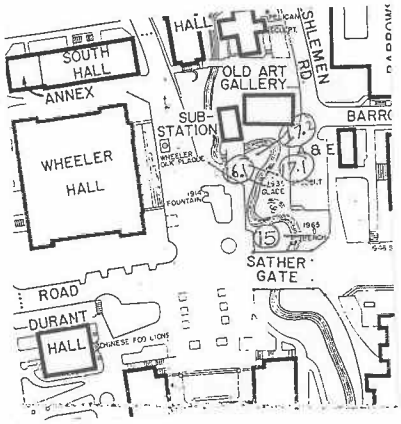


Photo XS 15RBP : Right Bank Pin

Photo XS 15: Section looking upstream

CROSS-SECTION 16.1 - 1990 photograph

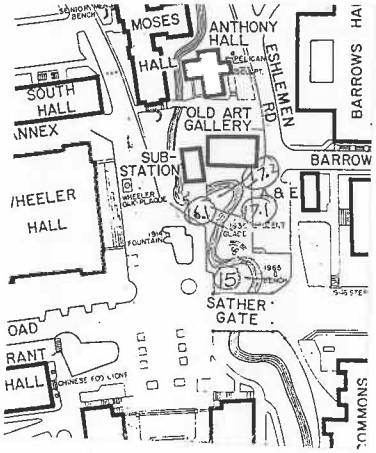


Photo XS 16.1 RBP : Right Bank Pin (1990 Note- RMP= rebar ~1 foot above ground, (but rock/concrete below in several places), about 3 feet from bank wall. Former RBP= stick(not reliable)

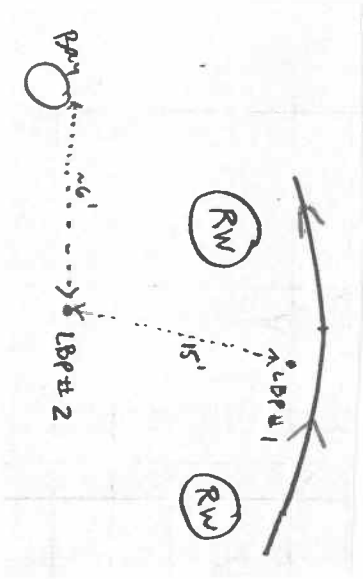


Photo XS 16.1 LBP: Left Bank Pin us shot showing foot on LBP. (1990 Note- 2 new LBP. LBP #1= rebar ~1/2" above ground, flagged, coverd with bottle cap & three rocks, ~1 foot from bank wall, about 2 feet us from where wall curves. LBP#2+ rebar ~1: above ground, coverd with bottle cap & rock, 15 feet from LBP#1, farhter into bank.)

CROSS-SECTION 17.1 - 1990 photograph

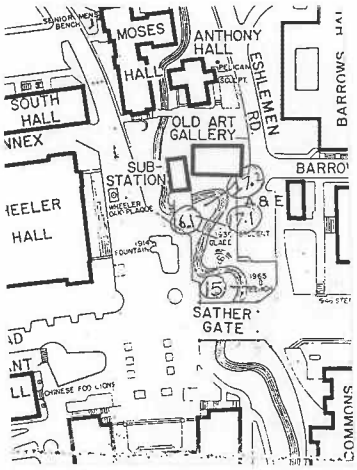


Photo XS 17.1 LBP: Left bank to right bank view. Left bank pin in redewood tree.

Photo XS 17.1: Looking upstream

CROSS-SECTION 17.2 - 1990 photograph

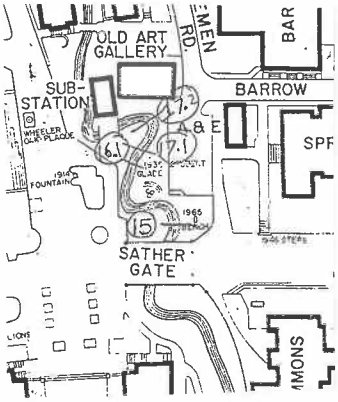


Photo XS 17.2 LBP: Left bank pin. Looking upstream.

Photo XS 17.2 RBP: Right bank pin. View from left bank to right bank

CROSS-SECTION 22 - 1990 photograph

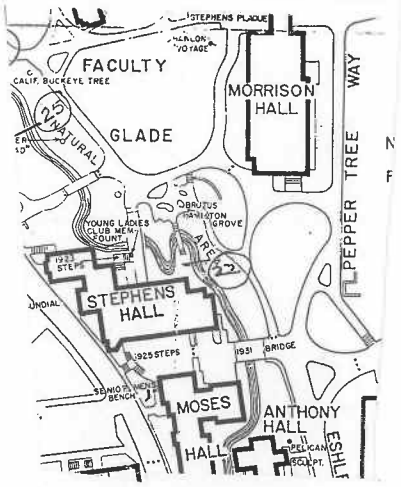


Photo XS 22 RBP : Right Bank Pin. View from left bank.

CROSS-SECTION 25 - 1990 photograph

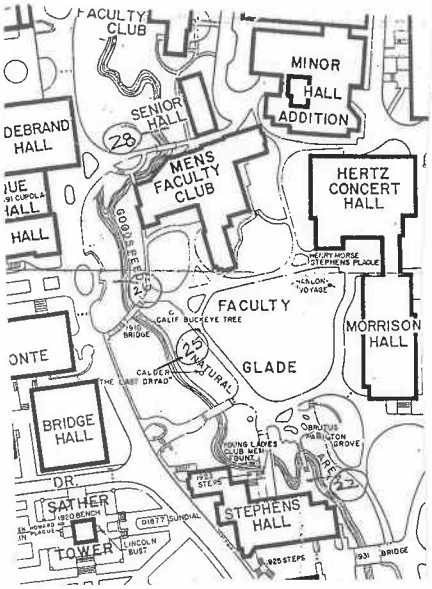


Photo XS 25 RBP : Right Bank Pin, about two feet above ground in the shrub.

Photo XS 25 LBP: Left Bank Pin.

CROSS-SECTION 26 - 1990 photograph

Photo XS 26 RBP : Right Bank Pin

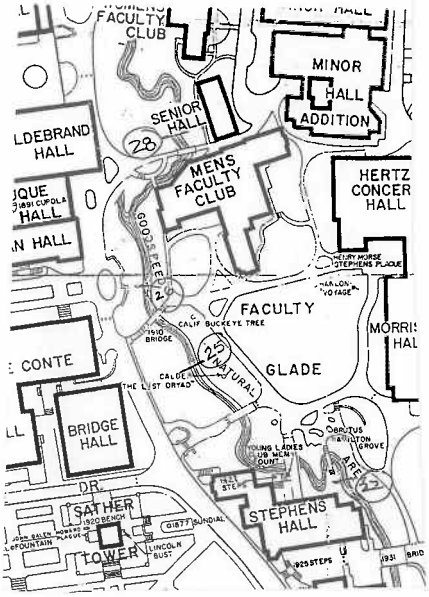


Photo XS 26

CROSS-SECTION 28 - 1990 photograph

Photo XS 28 RB : Right Bank from left bank.
(Note: 1990- RBP= rebar ~1" above ground, ~ 3" from wooden stake)

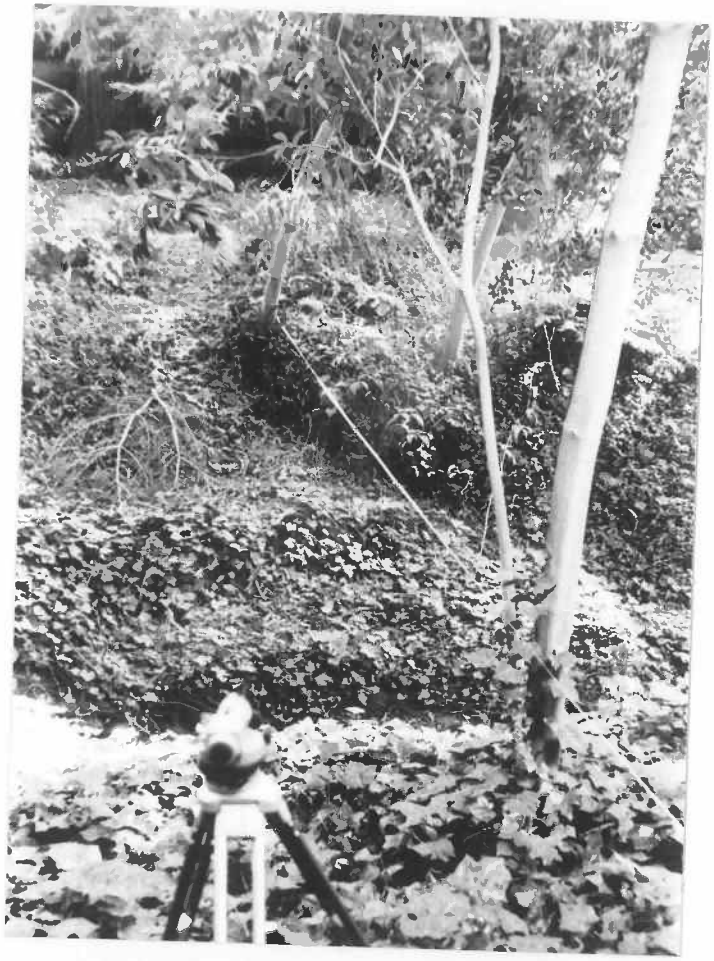


Photo XS 28 LBP: Left Bank Pin, from right bank

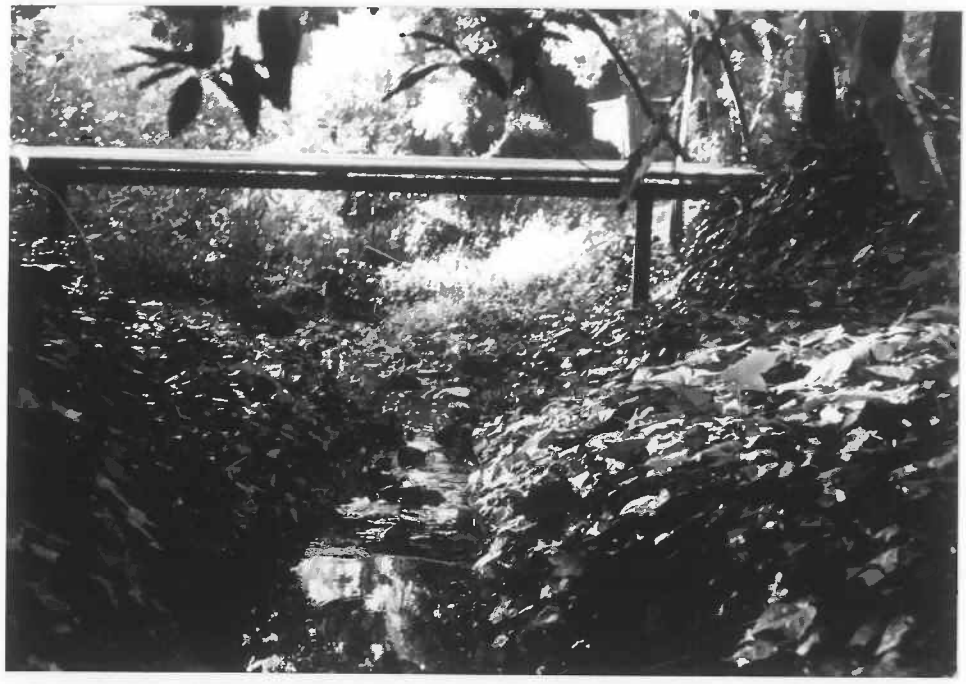
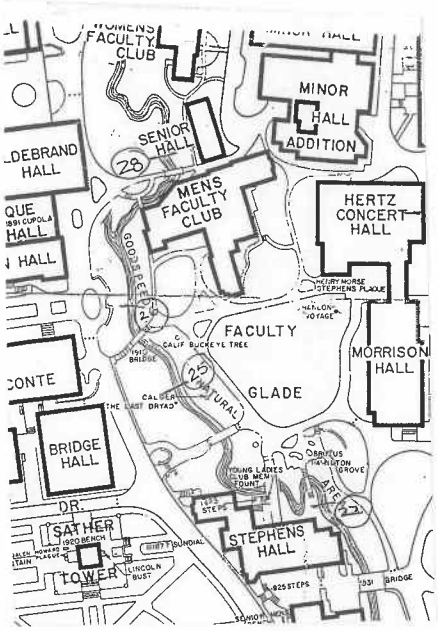


Photo XS 28



CROSS-SECTION 31 - 1990 photograph

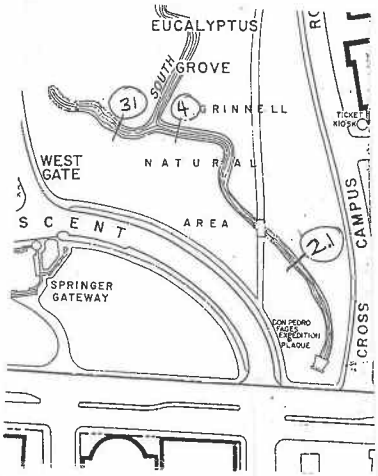


Photo XS 31 RBP: Right Bank Pin

Photo XS 31

Notes: 1994 LBP no longer present. Re-established on eucalyptus, blue spray painted

CROSS-SECTION 32 - 1990 photograph

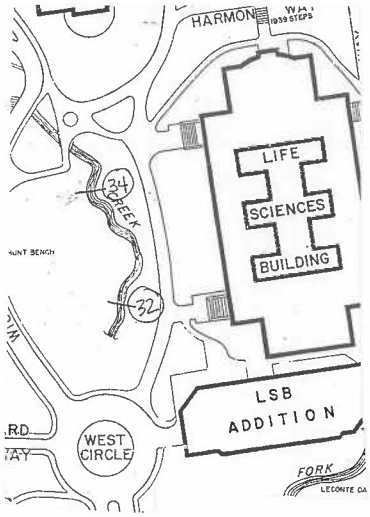
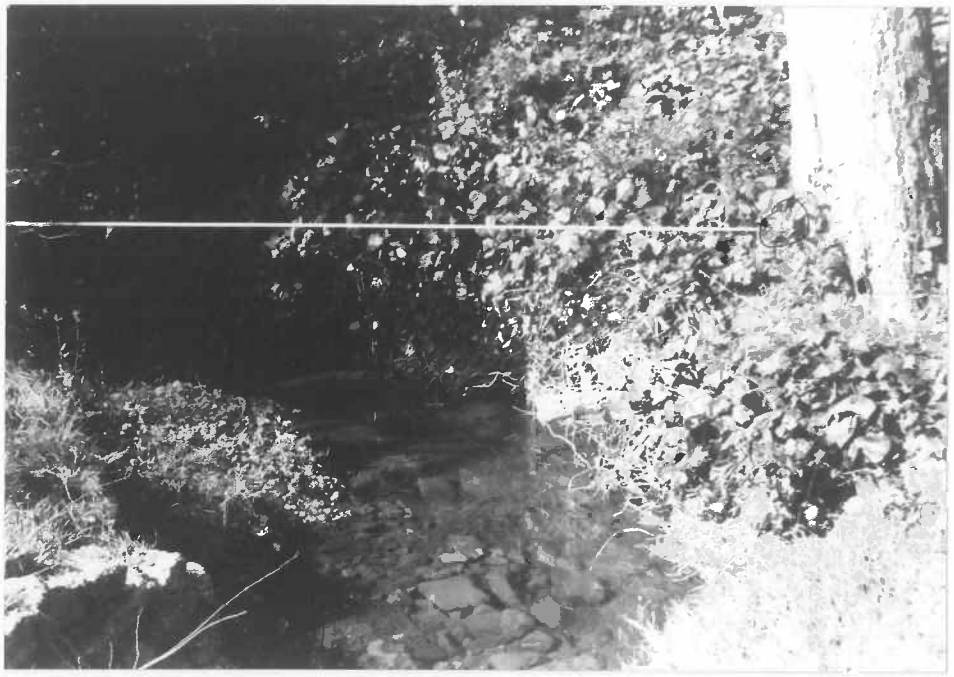


Photo XS 32 RBP : Right Bank Pin

Photo XS 32

CROSS-SECTION 34 - 1990 photograph



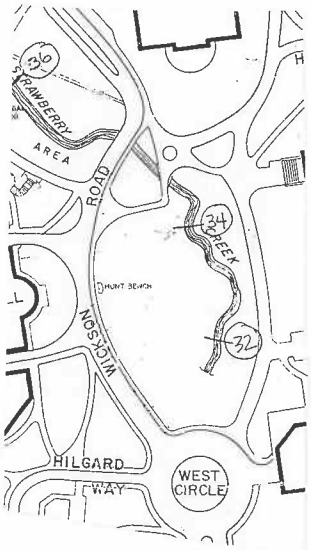


Photo XS 34 RBP : Right Bank Pin

Photo XS 34 LBP: Left Bank Pin (Notes: 1990 LBP= nail in stump just above cut where former tree had been)

CROSS-SECTION 36 - 1990 photograph

Photo XS 36 RBP : Right Bank Pin

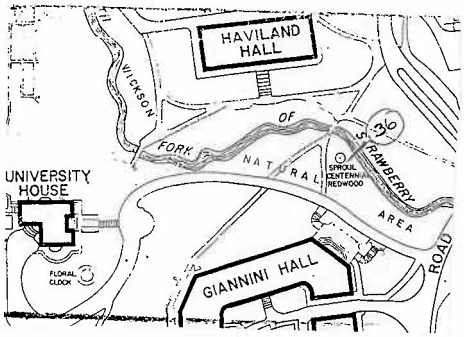
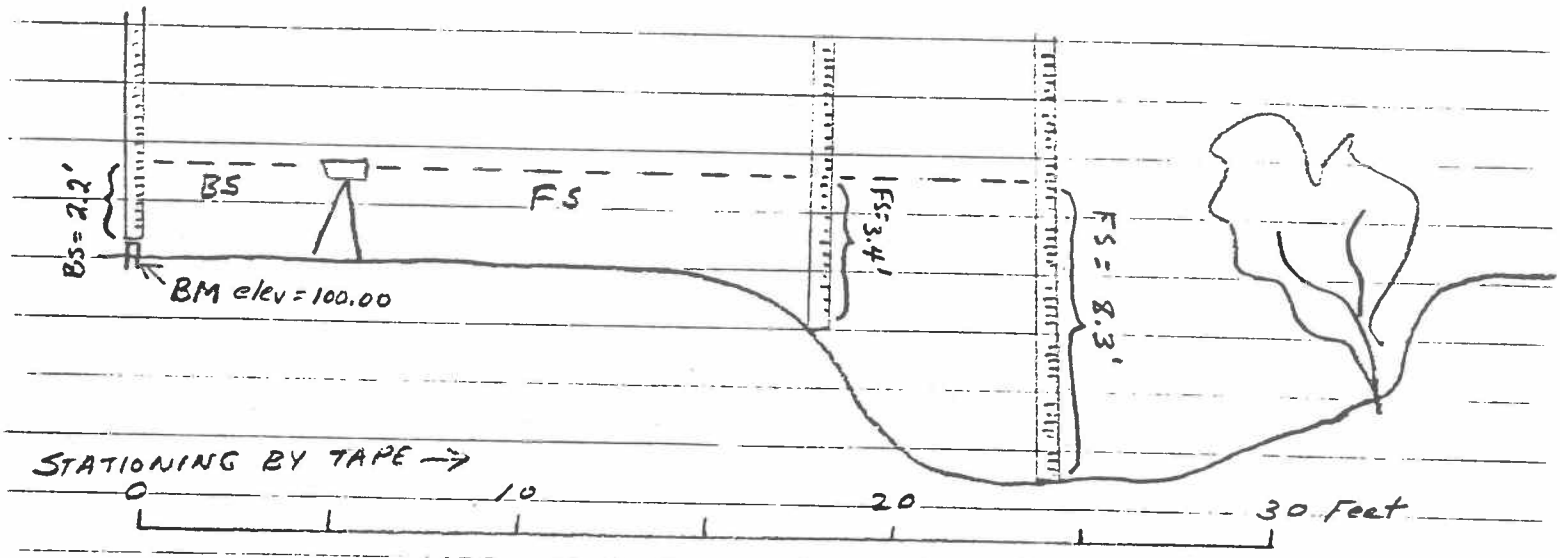


Photo XS 36: downstream view

LBP notes: 1990- LBP= rebar at spraypainted ivy ~1" above duff in ground. 1994- no spray painted ivy seen. No rebar found. LBP re-established 1.15 feet towrd Creek and 0.4 feet ds from a wooden stake stump that may have been a former marker.

Attachment IV

Surveying Protocol



STA'	BS' (+)	FS' (-)	HI'	EL'	NOTES
0	2.2			100.0	BM = rebar pin, elev. arbitrary
			102.2		
18		3.4		98.8	
24		8.3		93.9	

Figure 4.3

Chapter 2

Measurement of Horizontal Distances

2-1. HORIZONTAL DISTANCES One of the basic operations of surveying is the determination of the distance between two points on the surface of the earth. In surveys of limited extent the distance between two points at different elevations is reduced to its equivalent horizontal distance either by the procedure used to make the measurement or by computing the horizontal distance from a measured slope distance. Distances are measured by scaling from a map, by pacing, by using an odometer, tacheometry, electronic distance meters (EDM's), or by taping. This chapter will emphasize the use of the tape and the EDM's.

2-2. PACING Where approximate results are satisfactory, distances can be obtained by pacing. A person can best determine the length of his pace by walking over a line of known length several times, maintaining a natural walking stride. No particular advantage is obtained by developing a pace of, say, 3 ft. The natural stride is reproducible from day to day, whereas an artificial pace is not. The number of paces can be counted with a tally register or by the use of a pedometer, which is carried like a watch in a vertical position in the pocket.

2-3. TAPES Steel tapes for most surveying operations are graduated in feet or metres together with decimal parts of these units. Their lengths vary from 50 to 300 ft and from 15 to 100 m although the 100-ft and the 30-m tape are the most common length. Lightweight tapes may be graduated to hundredths of a foot or centimetre for the entire length. However, the usual foot tape is graduated to feet, with an end foot divided to hundredths of a foot. Metric tapes are usually graduated to decimetres throughout with an end decimetre divided to millimetres.

As most engineering and architectural plans show dimensions in feet and inches, a tape graduated in feet and inches is an advantage on construction work for layout purposes.

Tapes graduated in metres are used in most countries outside of the United States. These tapes are also used on most geodetic work in all countries.

Tapes of cloth, or of cloth containing threads of bronze or brass, are sometimes used where low precision is permissible and where a steel tape might be broken, as in cross sectioning for a railroad or a highway.

For extreme precision an invar tape, made of an alloy of steel and nickel, is used. The advantage of a tape of this material is that its coefficient of thermal expansion is about one-thirtieth that of steel, and hence its length is not so seriously affected by temperature changes. However, since such a tape is

expensive and must be handled very carefully to prevent kinking, invar tapes are not used for ordinary work.

Tapes are calibrated by comparison with a standard which is maintained by the National Bureau of Standards and by certain state, county, and city agencies. A few universities and state sections of the American Congress on Surveying and Mapping also maintain standard lengths by which tapes can be calibrated. The owner of the tape specifies under what conditions the tape should be calibrated, that is, what tension should be applied to the tape, and whether it is to be supported throughout its entire length or whether it is to be supported only at the ends. The calibration report then gives the length of the tape under the specified conditions and at some standard temperature, usually 68°F or 20°C.

2-8. EQUIPMENT USED FOR TAPING For the direct measurement of a line several hundred feet or metres long, the equipment used consists of a 100-ft or 30-m steel tape, two plumb bobs, one or more line rods, a set of taping pins, and, if the ground is hilly, a hand level. These items are shown in Fig. 2-1. A spring balance, which can be seen to the extreme right in Fig. 2-3, is used to apply the desired pull or tension to the tape during the measurement. The tension is expressed in pounds or kilograms.

Line rods, also called "range poles," are from $\frac{3}{8}$ in. to more than 1 in. in diameter and from 6 to 8 ft (2 m) long. They are pointed at one end and are painted with alternate bands of white and red. The rods are used to sight on and thus keep the forward and rear ends of the tape on the line that is being measured.

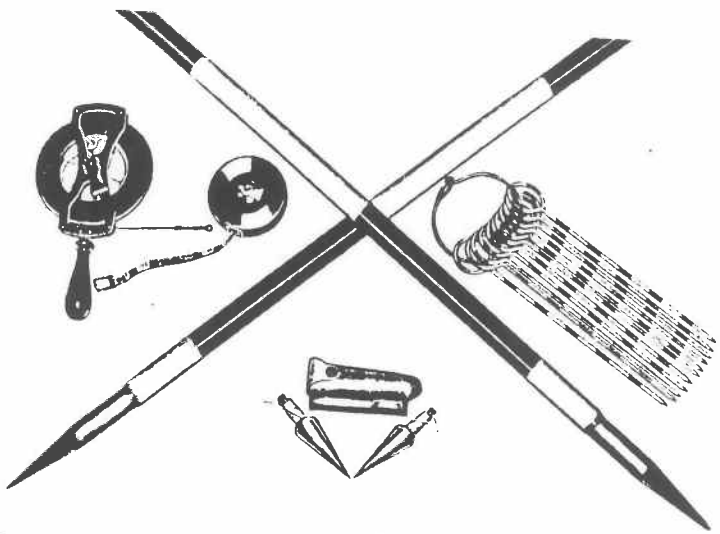


Figure 2-1. Equipment used for taping. Courtesy of W. & L. E. Gurley Co.

Taping pins are used to mark the positions of the ends of the tape on the ground while a measurement is in progress. A taping pin may be a heavy spike, but is usually a piece of No. 10 wire that is 10 to 18 in. long, is sharpened at one end, and is provided with an eye at the other end. Pieces of colored cloth can be tied to the eyes to make the pins more visible in tall grass or weeds.

The hand level is used to keep the two ends of the tape in the same horizontal plane when a measurement is made over rough or sloping ground.

The plumb bobs are used to project a point on the ground up to the tape, or to project a point on the tape down to the ground.

Some tapes are kept on reels when not in use. But a metal tape must be entirely removed from the reel when any length other than a few feet or metres is to be measured. If such a tape is not supplied with thongs on both ends with which to hold the tape, a taping pin can be slipped through the eye at the end of the tape and used as a handle. A tape that is thrown together in the form of a series of loops when not in use must be carefully unwrapped and checked for short kinks before it can be used for measurement. As long as a tape is stretched straight, it will stand any amount of tension that two people can apply. If kinked or looped, however, a very slight pull is sufficient to break it.

2-9. MEASUREMENTS WITH TAPE HORIZONTAL The horizontal distance between two points can be obtained with a tape either by keeping the tape horizontal or by measuring along the sloping ground and computing the horizontal distance. For extreme precision, such as is required in the length of a baseline in a triangulation system, the latter method is used. This method is also advantageous where steep slopes are encountered and it would be difficult to obtain the horizontal distance directly.

For moderate precision where the ground is level and fairly smooth, the tape can be stretched directly on the ground, and the ends of the tape lengths can be marked by taping pins or by scratches on a paved area. Where the ground is level but ground cover prevents laying the tape directly on the ground, both ends of the tape are held at the same distance above the ground by the forward tapeman and the rear tapeman. The tape is preferably held somewhere between knee height and waist height. The graduations on the tape are projected to the ground by means of the plumb bobs. The plumb-bob string is best held on the tape graduation by clamping it with the thumb, so that the length of the string can be altered easily if necessary (this can be seen in Fig. 2-9). When a tape is supported throughout its length on the ground and subjected to a given tension, a different value for the length of a line will be obtained than when the tape is supported only at the two ends and subjected to the same tension (see Section 2-18). Where fairly high accuracy is to be obtained, the method of support must be recorded in the field notes, provided both methods of support are used on one survey. Experienced tapemen

should ob[illegible]s good results by plumbing the ends of the tape over the marks as they wi[illegible]ain by having the tape supported on the ground.

When the [illegible]ound is not level, either of two methods may be used. The first is to hold one end of the tape on the ground at the higher point, to raise the other end of the tape until it is level, either by estimation or with the aid of the hand level, and then to project the tape graduation over the lower point to the ground by means of a plumb bob. The other method is to measure directly on the slope as described in Sections 2-11 and 2-12. These methods are shown in Fig. 2-2.

For high precision, a taping tripod or taping buck must be used instead of a plumb bob. Such a tripod is shown in Fig. 2-3. Taping tripods are usually used in groups of three, the rear tripod then being carried to the forward position. A pencil mark is scribed at the forward tape graduation, and on the subsequent measurement the rear tape graduation is lined up with this mark in order to carry the measurement forward. Since taping is usually done on the slope when tripods are used, the elevations of the tops of the tripods must be determined at the same time the taping proceeds. The elevations, which are determined by leveling (Chapter 3), give the data necessary to reduce the slope distances to horizontal distances as discussed in Section 2-12.

The head tapeman carries the *zero* end of the tape and proceeds toward the far end of the line, stopping at a point approximately a tape length from the point of beginning. The rear tapeman lines in the forward end of the tape by sighting on the line rod at the far end of the line. Hand signals are used to bring the head tapeman on line. The rear tapeman takes a firm stance and holds the tape close to his body with one hand, either wrapping the thong around his hand or holding a taping pin which has been slipped through the

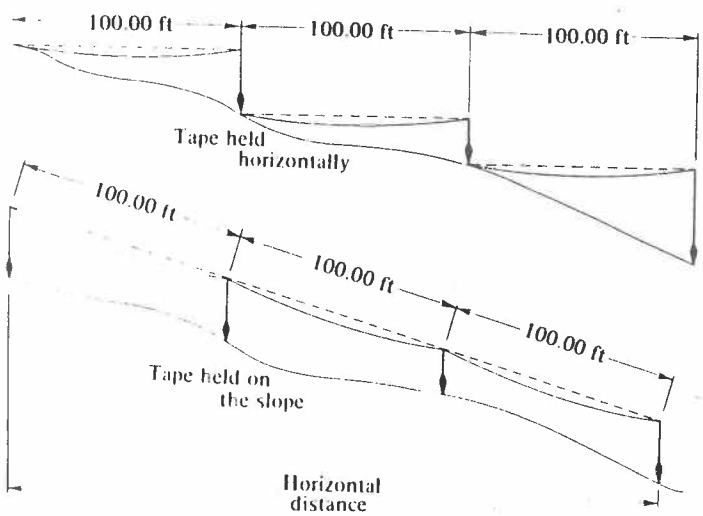


Figure 2-2. Taping over sloping ground using 100-ft tape.

Figure 2-3. Taping tripod.

eye of the tape as shown in Fig. 2-4. Standing to the side of the tape, he plumbs the end graduation over the point on the ground marking the start of the line. The tip of the plumb bob should be less than $\frac{1}{8}$ in. or about 3 mm above the ground point.

The head tapeman applies the tension to be used, either by estimation or by means of a spring balance fastened to the zero or forward end of the tape. At approximately the correct position on the ground, he clears a small area where the taping pin will be set. After again applying the tension, the head tapeman waits for a vocal signal from the rear tapeman indicating that the latter is on the rear point. When the plumb bob has steadied and its tip is less than $\frac{1}{4}$ in. or about 5 mm from the ground, the head tapeman dips the end of the tape slightly so that the plumb bob touches the ground. Then he, or a third member of the taping party, sets a taping pin at the tip of the plumb bob to mark the end of the first full tape length as shown in Fig. 2-5. The pin is set at right angles to the line and inclined at an angle of about 45° with the ground away from the side on which the rear tapeman will stand for the next measurement. The tape is then stretched out again to check the position of the pin. The notekeeper records the distance, 100.00 ft, or 30.000 m, in the field notes. The tape is advanced another tape length, and the entire process is repeated.

Figure 2-4. Plumbing over point.

If the taping advances generally downhill, the head tapeman checks to see that the tape is horizontal by means of the hand level. If the taping advances generally uphill, the rear tapeman checks for level.

When the end of the line is reached, the distance between the last pin and the point at the end of the line will usually be a fractional part of a tape length. The rear tapeman holds the particular full-foot or decimetre graduation that will bring the subgraduations at the zero end of the tape over the point marking the end of the line. The head tapeman rolls the plumb-bob string

Figure 2-5. Setting taping pin to mark forward position of tape.

along the subgraduations with his thumb until the tip of the plumb bob is directly over the ground point marking the end of the line.

Two types of end graduations of a tape that reads in feet are shown in Fig. 2-6. In view (a) the subgraduations are outside the zero mark, and the fractional part of a foot is added to the full number of feet. Hence the distance is 54 + 0.44 = 54.44 ft. This type is referred to as an *add* tape. In view (b) the subgraduations are between the zero and the 1-ft graduation, and the fractional part of a foot must be subtracted from the full number of feet. So the distance is 54 − 0.28 = 53.72 ft. This type is called a *cut* tape. Because of the variation in the type of end graduations, the rear tapeman must call out the

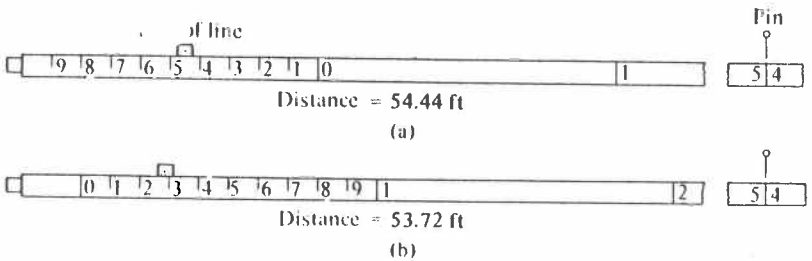


Figure 2-6. Graduations at end of foot-graduated tape.

actual foot mark he holds, and both the head tapeman and the notekeeper must agree that the value recorded in the field notes is the correct value.

Add tapes are more convenient to use than cut tapes simply because it is easier to add than to subtract the decimal part of the whole unit. The design of an add metric tape which is divided in decimetres throughout its length is shown in Fig. 2-7. The outside decimetre is further subdivided to centimetres and then to millimetres. In the illustration, the rear tapeman holds the 14.7-m mark at the pin, and the head tapeman reads 0.072 m at the end of the line. The distance is thus 14.7 + 0.072 = 14.772 m.

Where the slope is too steep to permit bringing the full length of the tape horizontal, the distance must be measured in partial tape lengths, as shown in Fig. 2-8. It is then necessary to enter a series of distances in the field notes. Some or all of them will be less than a full tape length. For a partial tape length, the head tapeman holds the zero end and the rear tapeman holds a convenient whole foot or decimetre mark which will allow the selected length of tape to be horizontal. When the forward pin is set, this partial tape length is recorded in the field notes. The head tapeman than advances with the zero end of the tape, and the rear tapeman again picks up a convenient whole foot or decimetre mark and plumbs it over the pin. Each partial tape length is recorded as it is measured or as the forward pin is set. In Fig. 2-9 is illustrated the use of a device called a *tape clamp* for holding a tape at any place other than at an end.

If a tape clamp is not available, the rear tapeman must then hold the tape in one hand in such a manner that it neither injures his hand nor damages the tape. At the same time he must be able to sustain a tension of between 10 and

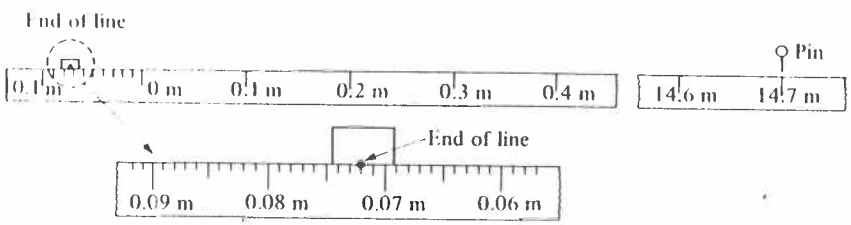


Figure 2-7. Graduations at end of metric tape.

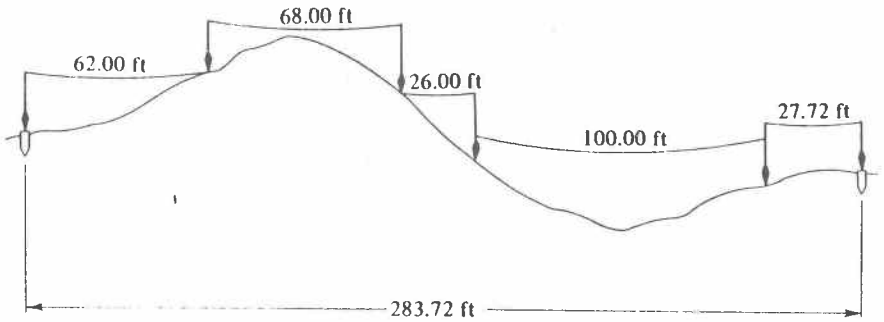


Figure 2-8. Breaking tape.

20 lb, or between 5 and 10 kg. The technique shown in Fig. 2-10 is a satisfactory solution to this problem. The tape is held between the fleshy portion of the fingers and that of the palm. Enough friction is developed to sustain a tension upward of 25 to 30 lb (10 to 15 kg) without injury or discomfort to the tapeman. He must not turn his hand too sharply, however, otherwise the tape may become kinked.

All distances should be taped both forward and backward, to obtain a better value of the length of the line and to detect or avoid mistakes. When the backward measurement is made, the new positions of the pins should be

Figure 2-9. Use of tape clamp.

Chapter 3
Leveling

3-1. GENERAL Leveling is the operation in surveying performed to determine and establish elevations of points, to determine differences in elevation between points, and to control grades in construction surveys. The elevation of a point has been defined as its vertical distance above or below a given reference level surface. The reference level surface most commonly used in the United States is the National Geodetic Vertical Datum of 1929, formerly called the Sea Level Datum of 1929. This surface was established by connecting all of the major level lines (see Section 3-24) in the country to 26 tidal benchmarks along the Atlantic, Gulf of Mexico, and Pacific Coasts. Other vertical datums are used locally by engineers and surveyors, but this practice causes confusion when tying together level lines which originate on different datums.

A benchmark is a permanent or semipermanent physical mark of known elevation. It is set as a survey marker in order to provide a point of beginning for determining elevations of other points in a survey. A good benchmark is a bronze disk set either in the top of a concrete post or in the foundation of a structure. Other locations for benchmarks are the top of a culvert headwall, the top of an anchor bolt, or the top of a spike driven into the base of a tree. The elevations of benchmarks are determined to varying degrees of accuracy by the field operations to be described in this chapter. Benchmarks established throughout the country by the National Geodetic Survey (NGS) to a high order of accuracy define the National Vertical Geodetic Datum of 1929.

The basic instrument used in leveling is a spirit level which establishes a horizontal line of sight by means of a telescope fitted with a set of cross hairs and a level bubble. The level is described in later sections of this chapter. Other instruments used for determining vertical distances are the engineer's transit, the theodolite, the EDM, the aneroid barometer, the hand level and the telescopic alidade. The use of the transit, the theodolite, and the telescopic alidade are explained in Chapters 4 and 14.

Modern inertial survey systems described in Chapter 9 and satellite Doppler receivers described in Chapter 10 can be employed to determine elevations quite rapidly. However the accuracy of the measured elevations is not as high as that obtained by most of the methods to be discussed in this chapter.

3-4. DIRECT DIFFERENTIAL LEVELING The purpose of differential leveling is to determine the difference of elevation between two points on the earth's surface. The most accurate method of determining differences of elevation is with the spirit level and a rod, in the manner illustrated in Fig. 3-4. It is assumed that the elevation of point A is 976 ft and that it is desired to determine the elevation of point B. The level is set up, as described in Section 3-22, at some convenient point so that the instrument is higher than both A and B. A leveling rod is held vertically at the point A, which may be on the top of a stake or on some solid object, and the telescope is directed toward the rod. The vertical distance from A to a horizontal plane can be read on the rod where the horizontal cross hair of the telescope appears to coincide. If the rod reading is 7.0 ft, the plane of the telescope is 7.0 ft above the point A. The elevation of this horizontal plane is $976 + 7 = 983$ ft. The leveling rod is next held vertically at B and the telescope is directed toward the rod. The vertical distance from B to the same horizontal plane is given by the rod reading with which the horizontal cross hair appears to coincide. If

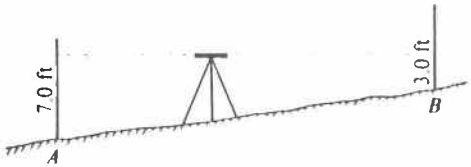


Figure 3-4. Direct differential leveling

the rod reading at B is 3.0 ft, the point B is 3.0 ft below this plane and the elevation of B is $983 - 3 = 980$ ft. The elevation of the ground at the point at which the level is set up need not be considered.

The same result may be obtained by noting that the difference in elevation between A and B is $7 - 3 = 4$ ft, and that B is higher than A. The elevation of B equals the elevation of A plus the difference of elevation between A and B, or $976 + 4 = 980$ ft.

3-13. SELF-LEVELING OR AUTOMATIC LEVEL The level shown in Fig. 3-17 is said to be self-leveling. When the bull's-eye bubble has been centered, a prism carried on a pendulum supported by two pairs of wires reflects the light rays entering the objective lens on back to the eyepiece end of the telescope. The lengths of the supporting wires and the positions of the points of suspension are so designed that the only rays of light reflected back to the intersection of the cross hairs by the swinging prism are the horizontal rays passing through the optical center of the objective lens. Hence, as long as the prism is free to swing, a horizontal line of sight is maintained, even

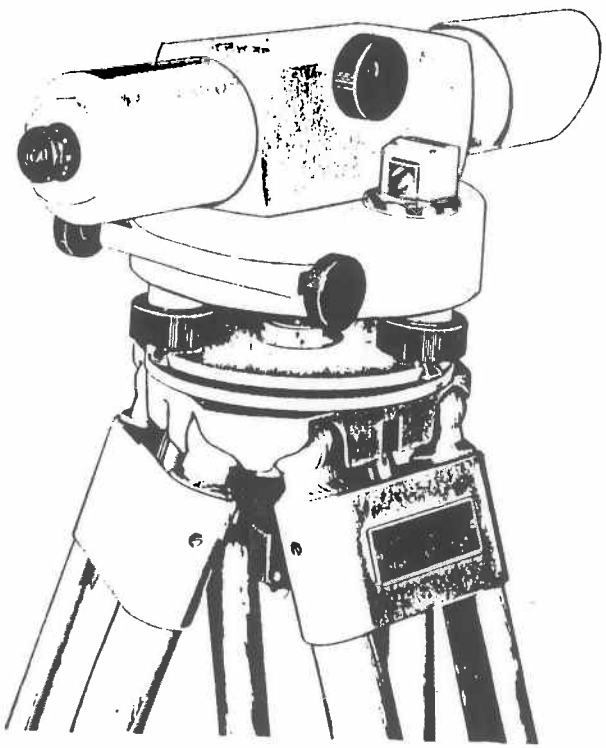


Figure 3-17. Self-leveling or automatic level. Courtesy of Keuffel & Esser Co.

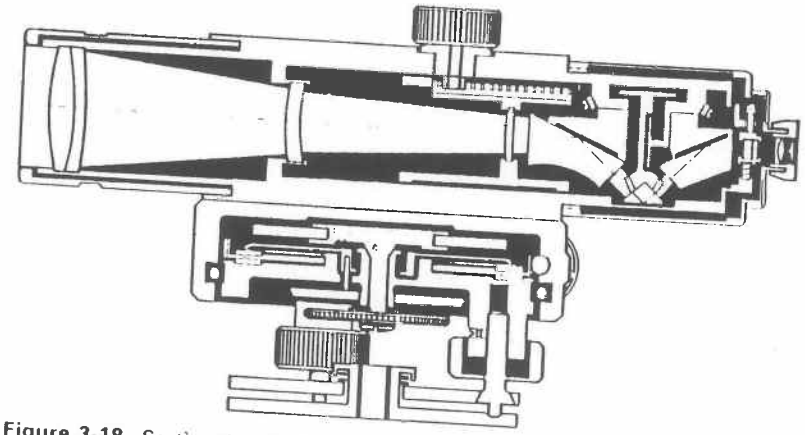


Figure 3-18. Section through self-leveling level.

though the telescope barrel itself is not horizontal. A damping device brings the pendulum to rest quite rapidly, so that the observer does not have to wait until it settles of its own accord. This type of level has the advantage of offering very rapid instrument setups and of eliminating random errors in centering the level bubble.

The self-leveling level shown in Fig. 3-17 is shown in cross section in Fig. 3-18. A small triangular-shaped prism can be seen slightly forward of the eyepiece near the bottom of the telescope. Light on entering the objective lens passes through the focusing lens and then through a fixed prism. It is then reflected off the triangular swinging prism and enters a second fixed prism, from which it is deflected into the eyepiece system.

The automatic level shown in Fig. 3-19 operates by means of a gravity-actuated compensator similar in general principle to that described above. This level contains an optical micrometer, the function of which is to allow very precise readings of graduated level rods as described in Section 3-21.

All automatic levels operate generally on the same principle as that described in this section.

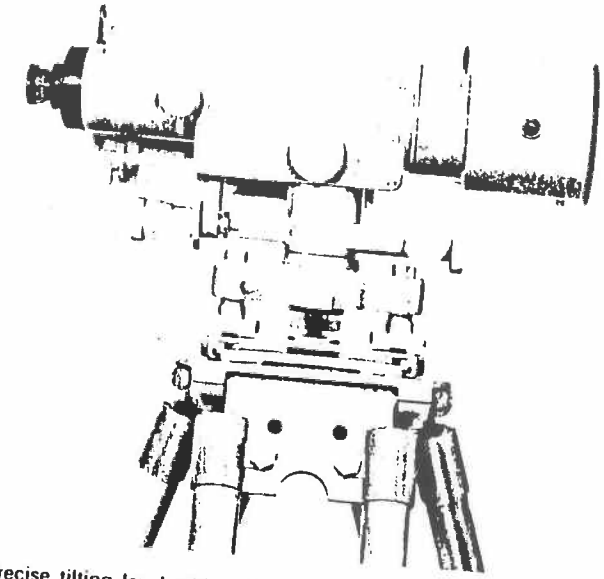


Figure 3-20. Precise tilting level with optical micrometer used for geodetic leveling. Courtesy of Wild Heerbrugg Instruments, Inc.

3-15. HAND LEVEL AND CLINOMETER The hand level, shown in Fig. 3-21, is a brass tube with a small level tube mounted on the top. A 45° mirror on the inside of the main tube enables the user to tell when it is being held horizontally. As the rod viewed through the level is not magnified, the length of sight is limited by the visibility of rod readings with the naked eye.

The hand level is used on reconnaissance surveys where extreme accuracy is unnecessary and in taping to determine when the tape is being held horizontally. It is also used to advantage for estimating how high or how low the engineer's level must be set in order to be able to read the leveling rod.

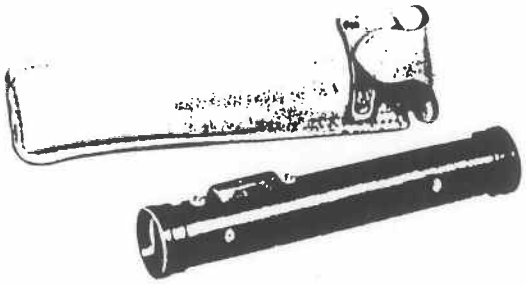


Figure 3-21. Hand level Courtesy of Keuffel & Esser Co

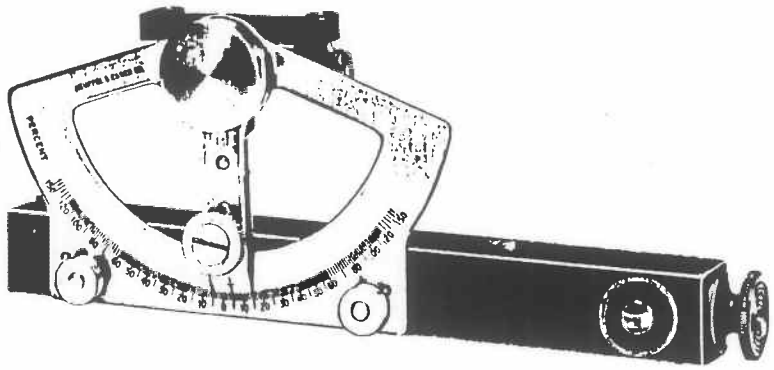


Figure 3-22. Clinometer. Courtesy of Keuffel & Esser Co.

The clinometer, shown in Fig. 3-22, can be used in the same manner as the hand level. In addition it can be employed for measuring vertical angles where approximate results are sufficient.

3-16. LEVELING RODS There are two general classes of leveling rods, namely, self-reading and target rods. A self-reading rod has painted graduations that can be read directly from the level. When a target rod is used, the target is set by the rodman as directed by the levelman, and the reading is then made by the rodman. Some types of rods can be used either as self-reading rods or as target rods.

The graduations on self-reading rods should appear sharp and distinct for the average length of sight. In the United States the rods are ordinarily graduated so as to indicate feet and decimals, the smallest division usually being 0.01 ft. Metric rods are graduated to centimetres, and rod readings are estimated to millimetres.

3-17. PHILADELPHIA ROD The Philadelphia rod, front and rear views of which are shown in Fig. 3-23, is made in two sections that are held together by the brass sleeves *a* and *b*. The rear section slides with respect to the front section, and it can be held in any desired position by means of the clamp screw *c* on the upper sleeve *b*. The rod is said to be a *short rod* when the rear portion is not extended, and a *long* or *high rod* when it is extended. The short rod is used for readings up to 7 ft. For readings between 7 and 13 ft, the long rod must be used. When the rod is fully extended, the graduations on the face are continuous.

3-19. READING THE ROD DIRECTLY If the target is not used, the reading of the rod is made directly from the telescope. The number of feet is given by the red figure just below the horizontal cross hair when the level has an erecting telescope, or just above the horizontal cross hair when it has an inverting telescope. The number of tenths is shown by the black figure just below or above the hair, the position depending on whether the telescope is erecting or inverting. If the reading is required to the nearest hundredth, the number of hundredths is found by counting the divisions between the last tenth and the graduation mark nearest to the hair. If thousandths of a foot are required, the number of hundredths is equal to the number of divisions between the last tenth and the graduation mark on the same side of the hair as that tenth, and the number of thousandths is obtained by estimation.

The readings on the rod for the positions x, y, and z in Fig. 3-25(a) are determined as follows: For x, the number of feet below the cross hair is 4, the number of tenths below is 1, and the cross hair coincides with the first graduation above the tenth mark; consequently, the reading is 4.11 ft to the nearest hundredth, or 4.110 ft to the nearest thousandth. For y, the feet and tenths are again 4 and 1, respectively; also, the hair is just midway between the graduations indicating 4 and 5 hundredths, and therefore the reading to the nearest hundredth can be taken as either 4.14 or 4.15 ft. In determining the hundredths it is convenient to observe that the hair is just below the acute-angle graduation denoting the fifth hundredth, and it is therefore unnecessary to count up from the tenth graduation. If thousandths are required, the number of hundredths is the lower one, or 4; and since the hair is midway between two graduation marks on the rod and the distance between the graduations is 1 hundredth or 10 thousandths of a foot, the number of thousandths in the required reading is $\frac{1}{2} \times 10$, or 5. Hence the reading to the nearest thousandth is 4.145 ft. For z, the reading to the nearest hundredth is 3.96 ft and that to the nearest thousandth is 3.963 ft.

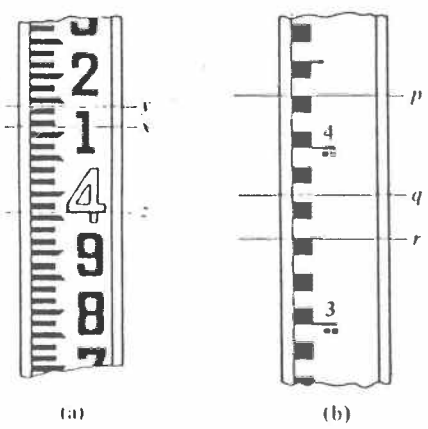


Figure 3-25. Direct reading of rod (a) Rod graduated to 0.01 ft (b) Rod graduated to 1 cm

Direct high-rod readings are made with the rod fully extended, as the graduations on the face of the rod then appear continuous.

The metric rod shown in Fig. 3-25(b) is numbered every decimetre and graduated in centimetres. The double dot shown below the decimetre numbers indicates the readings are in the 2-m range. The readings for positions p, q, and r are, respectively, 2.430 m, 2.373 m, and 2.349 m.

3-22. SETTING UP THE LEVEL The purpose of direct leveling, as explained in Section 3-4, is to determine the difference of elevation between two points by reading a rod held on the points. These rod readings can be made by the levelman without setting the target, or the target can be set as directed by the levelman and the actual reading made by the rodman. At the instant the readings are made, it is necessary that the line of sight determined by the intersection of the cross hairs and the optical center of the objective be horizontal. In a properly adjusted instrument this line will be horizontal only when the bubble is at the center of the bubble tube.

The first step in setting up the level is to spread the tripod legs so that the tripod head will be approximately horizontal. The legs should be far enough apart to prevent the instrument from being blown over by a gust of wind, and they should be pushed into the ground far enough to make the level stable. Repairs to a damaged instrument are always expensive. For this reason, neither a level nor a transit should be set up on a pavement or a sidewalk if such a setup can possibly be avoided. When the instrument must be so set up, additional care should be exercised to protect it from possible mishaps.

If the level contains four leveling screws, the telescope is turned over either pair of opposite leveling screws as shown in Fig. 3-32(a). The bubble is then brought approximately to the center of the tube by turning the screws in *opposite* directions. The level bubble moves in the direction of the left thumb, a point well worth remembering. No great care should be taken to bring the bubble exactly to the center the first time.

The next step is to turn the telescope over the other pair of screws and to bring the bubble exactly to the center of the tube by means of these screws. This is shown in Fig. 3-32(b). The telescope is now turned over the first pair of screws once more, and this time the bubble is centered accurately. The telescope is then turned over the second pair of screws and if the bubble has

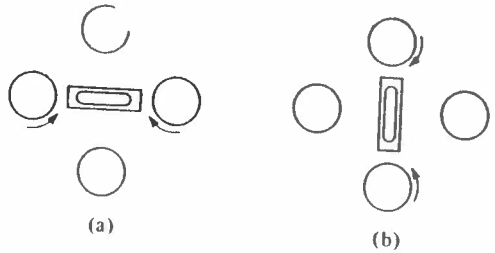


Figure 3-32. Manipulation of four leveling screws.

moved away from the center of the tube, it is brought back to the center. When the instrument is finally leveled up, the bubble should be in the center of the tube when the telescope is turned over either pair of screws. If the instrument is in adjustment, the bubble should remain in the center as the telescope is turned in any direction.

The beginner will need considerable practice in leveling up the instrument. It is by practice alone that he is able to tell how much to turn the screws to bring the bubble to the center. The more sensitive the bubble, the more skill is required to center it exactly. For the final centering when the bubble is to be moved only a part of a division, only one screw need be turned. The screw that has to be tightened should be turned if both are a little loose, and the one that has to be loosened should be turned when they are tight. When the telescope is finally leveled up, all four screws should be bearing firmly but should not be so tight as to put a strain in the leveling head. If the head of the tripod is badly out of horizontal, it may be found that the leveling screws turn very hard. The cause is the binding of the ball-and-socket joint at the bottom of the spindle. The tension may be relieved by loosening both screws of the other pair.

When a three-screw instrument is to be leveled, the level bubble is brought parallel with a line joining any two screws, such as *a* and *b* in Fig. 3-33(a). By rotating these two screws in opposite directions, the instrument is tilted about the axis *l l*, and the bubble can be brought to the center. The level bubble is now brought perpendicular to the line joining these first two screws, as shown in Fig. 3-33(b). Then only the third screw *c* is rotated to bring the bubble to the center. This operation tilts the instrument about the axis *m m*. The procedure is then repeated to bring the bubble exactly to the center in both directions.

When leveling a three-screw tilting level or an automatic level equipped with a bull's-eye bubble, as shown in Fig. 3-33(c), opposite rotations of screws *a* and *b* cause the bubble to move in the direction of the axis *m m*. Rotation of screw *c* only causes the bubble to move in the direction of the axis *l l*.

In walking about the instrument the levelman must be careful not to step near the tripod legs, particularly when the ground is soft. Neither should any part of the level be touched as the readings are being made, because the bubble can be pulled off several divisions by resting the hand on the telescope or on

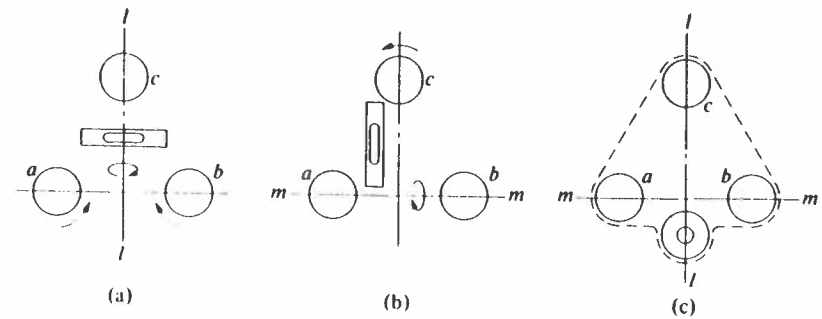


Figure 3-33. Manipulation of three leveling screws.

a tripod leg. The bubble will not remain in the center of the tube for any appreciable length of time. The levelman should form the habit of always checking the centering of the bubble just before and just after making a reading. Only in this way can he be sure that the telescope was actually horizontal when the reading was made.

3-23. SIGNALS In running a line of levels it is necessary for the levelman and the rodman to be in almost constant communication with each other. As a means of communication, certain convenient signals are employed. It is important that the levelman and the rodman understand these in order to avoid mistakes. When the target is used, it is set by the rodman according to signals given by the levelman. Raising the hand above the shoulder, so that the palm is visible, is the signal for raising the target; lowering the hand below the waist is the signal for lowering the target. The levelman, viewing the rod and the rodman through the telescope, should remember that he can see them much more distinctly than he can be seen by the rodman. Hence his signals should be such that there is no possible chance of misunderstanding. A circle described by the hand is the signal for clamping the target, and a wave of both hands indicates that the target is properly set, or all right. The signal for plumbing the rod is to raise one arm above the head and then to lean the body in the direction in which the rod should be moved.

3-24. RUNNING A LINE OF LEVELS In the preliminary example of direct leveling, given in Section 3-4, it was assumed that the difference of elevation between the two points considered could be obtained by a single setting of the level. This will be the case only when the difference in elevation is small and when the points are relatively close together. In Fig. 3-34, rods at the points *A* and *K* cannot be seen from the same position of the level. If it is required to find the elevation of point *K* from that of *A*, it will be necessary to set up the level several times and to establish intermediate points such as

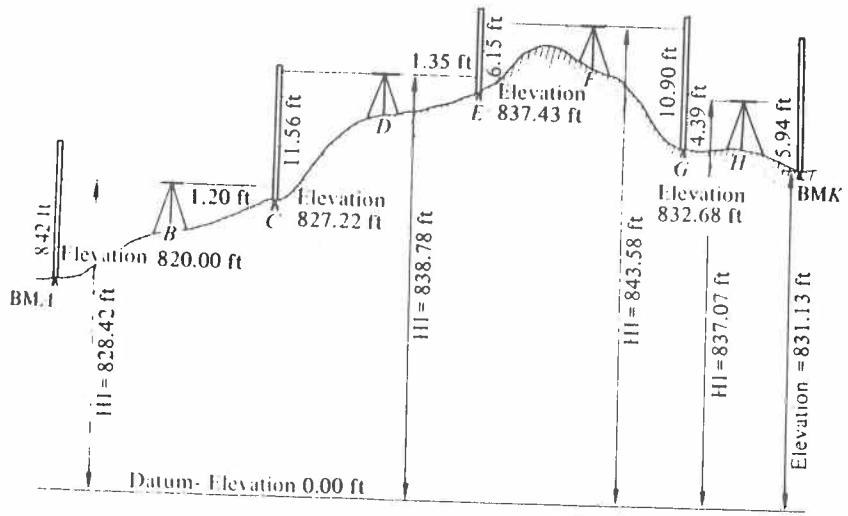


Figure 3-34. Direct leveling.

C, *E*, and *G*. These are the conditions commonly encountered in the field and may serve as an illustration of the general methods of direct leveling.

Let the elevation of the benchmark (abbreviated BM) at *A* be assumed as 820.00 ft. This is recorded as shown in the leveling notes of Fig. 3-35. The level is set up at *B*, near the line between *A* and *K*, so that a rod held on the BM will be visible through the telescope; the reading on the rod is found to be 8.42 ft. This reading is called a backsight reading, or simply a backsight (abbreviated BS), and is recorded as such in the notes. A backsight is the rod reading taken on a point of known elevation to determine the *height of instrument* (abbreviated HI). If the BS of 8.42 ft is added to the elevation of *A*, the HI is obtained. Thus HI = 820.00 + 8.42 = 828.42 ft. This is shown in the notes.

After the HI has been established, a point *C* called a *turning point* is selected that is slightly below the line of sight. This point should be some stable unambiguous object, so that the rod can be removed and put back in the same place as many times as may be necessary. For this purpose, a sharp-pointed solid rock or a well-defined projection on some permanent object is preferable. If no such object is available, a stake or a railroad spike can be driven firmly in the ground and the rod held on top of it. After the turning point at *C*, designated TP-1, is set or selected, a reading is taken on the rod held on TP-1. If this reading is 1.20 ft, TP-1 is 1.20 ft below the line of sight, and the elevation of TP-1 is 828.42 − 1.20 = 827.22 ft as shown in the notes. This rod reading is called a foresight (abbreviated FS). An FS is taken on a point of unknown elevation in order to determine its elevation from the height of instrument.

LEVELING, BMA to BMK

SHATTUCK AVE SEWER PROJECT BERKELEY

MAY 18, 1981

STA	BS	HI	FS	ELEV		
					LEVEL # 4096	LEVEL J. BROWN
					ROD # 18	ROD F SMITH
BMA	8.42	828.42		820.00		
TP_1	11.56	838.78	1.20	827.22	BMA is top of iron pipe, S E Cor	
TP_2	6.15	843.58	1.35	837.43	Shattuck + Maple Sts.	
TP_3	4.39	837.07	10.90	832.68		
BMK			5.94	831.13	BMK is Bronze disk in sidewalk	
					N W Cor. Shattuck + Vincent Sts	
Σ BS	+30.52	Σ FS	-19.39			
	-19.39					
	+11.13				831.13	
					-820.00	
					+ 11.13 checks	

Figure 3-35. Level notes.

Occasionally successive foresights and backsights are taken on an overhead point such as on a point in the roof of a tunnel. The foresight taken on such a point is added to the HI to obtain the elevation of the point. The backsight taken on the point is subtracted from the elevation of the point to determine the HI. Such readings must be carefully noted in the field notes.

While the rodman remains at *C*, the level is moved to *D* and set up as high as possible but not so high that the line of sight will be above the top of the rod when it is again held at *C*. This can be quickly checked by means of a hand level. The reading 11.56 ft is taken as a backsight. Hence, the HI at *D* is 827.22 + 11.56 = 838.78 ft. When this reading is taken, it is important that the rod be held on exactly the same point that was used for a foresight when the level was at *B*.

After the backsight on *C* has been taken, another turning point *E* is chosen and a foresight of 1.35 ft is obtained. The elevation of *E* is 838.78 − 1.35 = 837.43 ft. The level is then moved to *F* and the backsight of 6.15 ft taken on *E*. The new HI is 837.43 + 6.15 = 843.58 ft. From this position of the level a foresight of 10.90 ft is taken on *G*, the elevation of which is 843.58 − 10.90 = 832.68 ft. The level is then set up at *H*, from which position a backsight reading of 4.39 ft is taken on *G* and a foresight reading of 5.94 ft is taken on the new BM at *K*. The final HI is 832.68 + 4.39 − 837.07 ft, and the elevation of *K*

is 837.0 5.94 = 831.13 ft. As the starting elevation was 820.00 ft, the point K is 11.1 higher than A.

3-25. CHECKING LEVEL NOTES To eliminate arithmetical mistakes in the calculation of HI's and elevations, the arithmetic should be checked on each page of notes. Adding backsights gives $\sum$ BS; adding foresights gives $\sum$ FS. Then $\sum$ BS − $\sum$ FS should equal the difference in elevation (DE) between the starting point to the last point on the page. This is shown in the notes of Fig. 3-35. $\sum$ BS − $\sum$ FS = +11.13 ft, and the calculated DE is also +11.13 which checks the arithmetic. The last point on the page should then be carried to the following page before the BS on that point is recorded in the notes.

3-26. CHECK LEVELS Although the arithmetic in the reduction of the field notes may have been verified, there is no guarantee that the difference of elevation is correct. The difference of elevation is dependent on the accuracy of each rod reading and on the manner in which the field work has been done. If there has been any mistake in reading the rod or in recording a reading, the difference of elevation is incorrect.

The only way in which the difference of elevation can be checked is by carrying the line of levels from the last point back to the original benchmark or to another benchmark whose elevation is known. This is called "closing a level circuit." If the circuit closes on the original benchmark, the last point in the circuit, BMK in Fig. 3-34, must be used as a turning point; that is, after the foresight has been read on the rod at K from the instrument set up at H, the level must be moved and reset before the backsight is taken on K in order to continue the circuit to closure. Otherwise, if a mistake was made in reading the rod on the foresight to K from the setup at H, this mistake will not be discovered when checking the notes. A plan view of the level circuit between A and K in which the circuit is closed back on A is shown in Fig. 3-36(a). Note that the level has been reset between the FS taken on K and the BS taken on K. In Fig. 3-36(b), the level circuit has been continued to a known benchmark P to close the circuit. BMK is used as a turning point in this instance.

If the line of levels is carried back to BMA in the above example, on return the measured elevation of A should be 820.00 ft. The difference represents the error of closure of the circuit and should be very small. If a large discrepancy exists, the mistake may have been made in adding and subtracting backsight and foresight readings. This will be discovered on checking the notes. Otherwise there was a wrong reading of the rod or a wrong value was entered in the field notes. The adjustment of a level circuit based on the error of closure is discussed in Section 5-11.

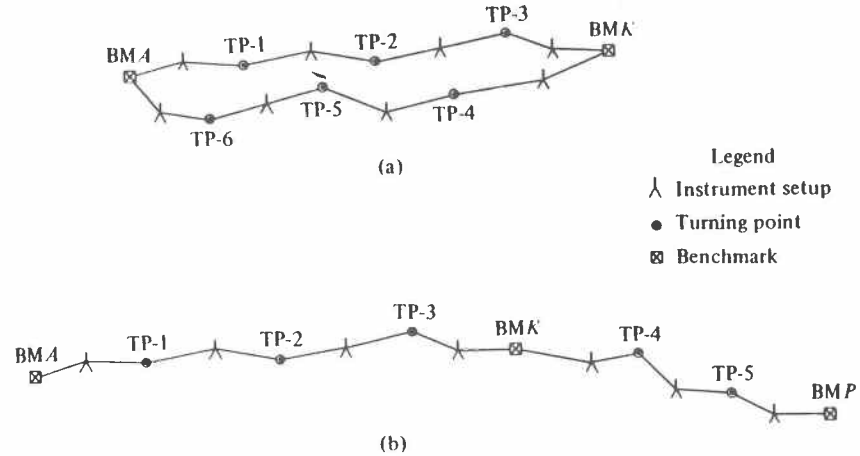


Figure 3-36. Closing on known benchmark.

3-27. SOURCES OF ERROR IN LEVELING The principal sources of error in leveling are instrumental defects, faulty manipulation of the level or rod, settling of the level or the rod, errors in sighting, mistakes in reading the rod or in recording or computing, errors due to natural sources, and personal errors.

3-28. INSTRUMENTAL ERRORS The most common instrumental error is caused by the level being out of adjustment. As has been previously stated, the line of sight of the telescope is horizontal when the bubble is in the center of the tube, provided the instrument is in perfect adjustment. When it is not in adjustment, the line of sight will either slope upward or downward when the bubble is brought to the center of the tube. The various tests and adjustments of the level are given in Appendix B.

Instrumental errors can be eliminated or kept at a minimum by testing the level frequently and adjusting it when necessary. Such errors can also be eliminated by keeping the lengths of the sights for the backsight and foresight readings nearly equal at each setting of the level. Since it is never known just when an instrument goes out of adjustment, this latter method is the more certain and should always be used for careful leveling.

In Fig. 3-37 the line of sight with the level at B should be in the horizontal line $EBGK$. If the line of sight slopes upward as shown and a sight is taken on a rod at A, the reading is AF, instead of AE. This reading is in error by the amount of $EF = e_1$. When the telescope is directed toward a rod held at C or D, the line of sight will still slope upward through the same vertical angle if it is assumed that the bubble remains in, or is brought to, the center of the tube.

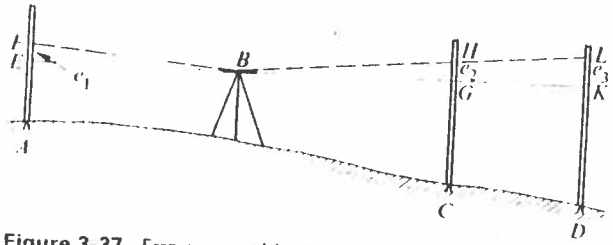


Figure 3-37. Errors caused by imperfect adjustment of level.

The rod reading taken on C will be CH, which is in error by an amount $GH = e_2$. If the horizontal distances BE and BG are equal, the errors e_1 and e_2 will be alike, and the difference between the two rod readings AF and CH will be the true difference of elevation between A and C. A rod reading taken at D will be DL, with an error $KL = e_3$. Since BK is longer than BE, the difference in elevation between A and D will be in error by an amount equal to the difference between e_3 and e_1. Similar reasoning applies if the line of sight slopes downward instead of upward. This error also exists in an automatic level.

It is not always feasible or even possible to balance a foresight distance with a backsight distance. This situation might occur on account of the terrain or when a series of foresights must be taken from a single instrument setup as in checking grade over a large area. It is advisable in such cases to check the level and make the necessary adjustments (see Section 3-38 and also Appendix B). An alternative is to employ a level like the one shown in Fig. 3-16 in which the rod can be read with the telescope right-side up and upside down, giving a mean reading free of error from a sloping line of sight. Another alternative is the technique of reciprocal observations discussed in Section 3-36.

Extremely long sights should also be avoided. The further the rod is from the level, the greater will be the space covered on the rod by the cross hair and the more difficult it will be to determine the reading accurately. For accurate results, sights with the engineer's level should be limited to about 300 ft.

3-29. ERRORS DUE TO MANIPULATION As has been previously stated, the careful levelman will form the habit of checking the position of the bubble just before and just after making each rod reading. This is the only way in which he can be certain that he is getting the proper reading.

The amount of the error due to the bubble being off center will depend on the sensitiveness of the bubble. A very convenient way to determine this error for any given bubble and for any given distance is to remember that an error of 1′ in angle causes an error of about 1 in. or 0.08 ft at a distance of 300 ft (3 cm in 100 m). Thus if a 30″ bubble is off one division at the instant the reading is made, the resulting error will be about 0.04 ft when the rod is 300 ft away (15 mm in 100 m). This type of error does not exist in an automatic level.

A common mistake in handling the rod is in not being careful t that the target is properly set before a high-rod reading is made with t... .arget. Many rods have been damaged by allowing the upper portion to slide down rapidly enough to affect the blocks at the bottom of the lower section and the top of the upper section. If this has been done, it is probable that the reading on the back of a Philadelphia rod will not be exactly 7 ft when set as a low rod. In this case the target should be set at that reading, rather than at exactly 7 ft, for a high-rod reading with the target. When a high rod is being read directly from the level, the rodman should make sure that the rod is properly extended and has not slipped down.

When the target is being used, the levelman should check its position after it has been clamped in order to make sure that it has not slipped. The beginner will be astonished at the care that must be exercised in making the target coincide exactly with the horizontal cross hair. Readings taken on the same point about 300 ft from the level may vary by several hundredths of a foot if the bubble is not exactly centered and if the target is carelessly set.

3-30. ERRORS DUE TO SETTLEMENT If any settlement of the level takes place in the interval between the reading of the backsight and the reading of the foresight, the resulting foresight will be too small, and all elevations beyond that point will be too high by the amount of the settlement. Also, if the turning point should settle while the level is being moved forward after the foresight has been taken, the backsight on the turning point from the new position of the level will be too great and all elevations will be too high by the amount of the settlement.

Errors due to the settlement of the level can be avoided by keeping the level on firm ground. If this is impossible, stakes can be driven in the ground and the tripod legs can be set on these instead of directly on the ground. In precise leveling, two rods and two rodmen are used in order that the backsight and foresight readings from a setup can be made more quickly. The error is still further diminished by taking the backsight first at one setup of the level and the foresight first at the next setup.

A proper choice of turning points should eliminate error from their settlement. If soft ground must be crossed, long stakes should be used as turning points.

3-31. ERRORS IN SIGHTING If parallax exists between the image formed by the focusing lens and the plane of the cross hairs, an error will be introduced in the rod reading. Parallax is eliminated by the procedure discussed in Section 3-11.

The rod should be plumb when the reading is made. The levelman can tell whether or not the rod is plumb in one direction by noting if it is parallel to

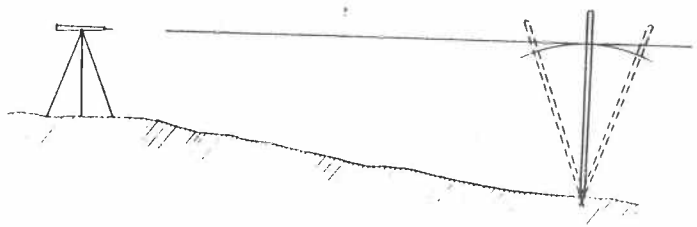


Figure 3-38. Waving the rod.

the vertical cross hair. He cannot tell, however, if it is leaning toward or away from him. The leveling rods used on precise work are equipped with circular levels, so that the rodman can tell when he is holding the rod vertically. For less accurate work, the rodman can balance the rod between his fingers if the wind is not blowing, or he can wave it slowly toward and then away from the level as indicated in Fig. 3-38. The least reading obtainable is the proper one. If the target is being used, the line dividing the colors should just coincide with the cross hair and then drop away from it. Errors from failure to hold the rod plumb will be much greater on readings near the top of the rod than for those near the bottom. For this reason more care should be exercised when making high-rod readings.

For careful work the lengths of the backsight and the foresight from the same setup should be kept nearly equal (see Section 3-28). If in ascending a steep hill the level is always kept on the straight line between the turning points, the distance to the backsight will be about twice as great as the distance to the foresight, and considerable error may result if the instrument is not in good adjustment. If there are not obstructions, these two distances can be kept nearly equal by setting the level some distance away from the straight line between the turning points. By thus zigzagging with the level, this source of error can be eliminated.

3-32. MISTAKES IN READING ROD, RECORDING, AND COMPUTING A common mistake in reading the rod is to misread the number of feet or tenths or metres and decimetres. The careful levelman observes the foot and tenth marks both above and below the cross hair. On close sights, no foot mark may appear within the field of the telescope. In this case the reading can be checked by directing the rodman to place his finger on the rod at the cross hair or if the reading is a high one, by having him slowly raise the rod until a foot mark appears in the telescope. In case of doubt the target can always be used.

Some instruments for precise leveling are equipped with three horizontal cross hairs. All three hairs are read at each sighting. If the hairs are evenly spaced, the difference between the readings of the upper and the middle hairs should equal the difference between the readings of the middle and lower hairs. This comparison is always made before the rodman leaves a turning point.

Where readings to thousandths of a foot are being made with the et, a common mistake in recording is to omit one or more ciphers fr.... such readings as 5.004, and to record instead 5.04 or 5.4. Such mistakes can be avoiding by making sure that there are three decimal places for each reading. Thus the second reading, if correct, should be recorded as 5.040, and the third as 5.400. If the values are not so recorded, the inference would be that the levelman was reading only to hundredths of a foot on the second reading and only to tenths on the third.

Other common mistakes of recording are the transposition of figures and the interchanging of backsight and foresight readings. If the levelman will keep the rodman at the point long enough to view the rod again after recording the reading, mistakes of the first type can often be detected. To prevent the interchange of readings, the beginner should remember that ordinarily the first reading taken from each position of the level is the backsight reading and that only one backsight is taken from any position of the level. Any other sights taken are foresights.

Mistakes in computations, as far as they affect the elevations of turning points and benchmarks, can be detected by checking the notes, as described in Section 3-25. This should be done as soon as the bottom of a page is reached, so that incorrect elevations will not be carried forward to a new page.

3-33. ERRORS DUE TO NATURAL SOURCES One error due to natural sources is that caused by curvature and refraction as described in Section 3-2. The error from this source amounts to but 0.0002 ft in a 100-ft sight (0.01 mm/30 m) and to about 0.002 ft in a 300-ft sight (0.7 mm/100 m). So for ordinary leveling it is a negligible quantity. It can be practically eliminated by keeping the backsight and foresight distances from the same setup equal. In precise leveling if the backsight and foresight distances are not substantially equal, a correction is applied to the computed difference of elevation.

The familiar heat waves seen on a hot day are evidence of refraction and when they are seen, refraction may be a fruitful source of error in leveling. When they are particularly intense, it may be impossible to read the rod unless the sights are much shorter than those usually taken. Refraction of this type is much worse close to the ground. For careful work it may be necessary to discontinue the leveling for 2 or 3 hr during the middle of the day. It may be possible to keep the error from this source at a low figure by taking shorter sights and by so choosing the turning points that the line of sight will be at least 3 or 4 ft or 1 m above the ground.

Better results will usually be obtained when it is possible to keep the level shaded. If the sun is shining on the instrument, it may cause an unequal expansion of the various parts of the instrument; or if it heats one end of the bubble tube more than the other, the bubble will be drawn to the warmer end of the tube. For precise work the level must be protected from the direct rays of the sun.

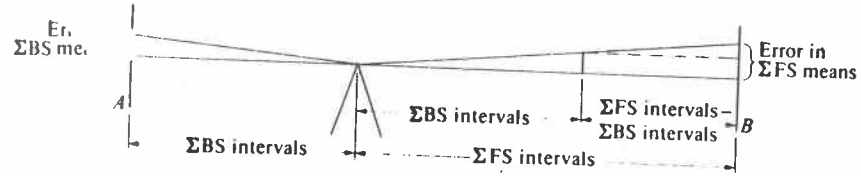


Figure 3-45. Imbalance of sum of backsight and sum of foresight intervals.

is shown as 2.845. The sum of the two means in column 3 gives $(n_1 + n_2)$ in Eq. (3-9); the sum of the two means in column 7 gives $(N_1 + N_2)$. The remaining computations needed to determine C are shown directly in the notes.

Assume that the results obtained when a line of levels was run between two points A and B in Fig. 3-45 are as follows: sum of backsight means, 31.422 ft; sum of foresight means, 12.556 ft; sum of backsight intervals, 22.464 ft; and sum of foresight intervals, 27.845 ft. The difference between $\sum$ FS intervals and $\sum$ BS intervals is $27.845 - 22.464 = 5.381$ ft. Therefore the observed value of $\sum$ FS means must be corrected by an amount equal to the C factor times the difference between the intervals, that is, by $-0.0112 \times 5.381 = -0.060$ ft. The corrected value of $\sum$ FS means is thus $12.556 - 0.060 = 12.496$ ft. The corrected difference in elevation is then $\sum$ BS means $- \sum$ FS means $= 31.422 - 12.496 = +18.926$ ft.

3-39. PROFILE LEVELS The purpose of profile leveling is to determine the elevations of the ground surface along some definite line. Before a railroad, highway, transmission line, sidewalk, canal, or sewer can be designed, a profile of the existing ground surface is necessary. The route along which the profile is run may be a single straight line, as in the case of a short sidewalk; a broken line, as in the case of a transmission line or sewer; or a series of straight lines connected by curves, as in the case of a railroad, highway, or canal. The data obtained in the field are usually employed in plotting the profile. This plotted profile is a graphical representation of the intersection of a vertical surface or a series of vertical surfaces with the surface of the earth, but it is generally drawn so that the vertical scale is much larger than the horizontal scale in order to accentuate the differences of elevation. This is called vertical exaggeration.

3-40. STATIONS The line along which the profile is desired must be marked on the ground in some manner before the levels can be taken. The common practice is to set stakes at some regular interval which may be 100, 50, or 25 ft, or 30, 20, or 10 m depending on the regularity of the ground surface and the accuracy required and to determine the elevation of the ground surface at each of these points. The beginning point of the survey is

designated as station 0. Points at multiples of 100 ft or 100 m from point are termed *full stations*. Horizontal distances along the line are most conveniently reckoned by the station method. Thus points at distances of 100, 200, 300, and 1000 ft from the starting point of the survey are stations 1, 2, 3, and 10, respectively. Intermediate points are designated as *pluses*. A point that is 842.65 ft from the beginning point of the survey is station 8 + 42.65. If the plus sign is omitted, the resulting figure is the distance, in feet, from station 0.

In the remainder of this chapter, reference will be made to stations of 100 ft. However, 100-m stations are handled in exactly the same way if the leveling is performed in metric units.

When the stationing is carried continuously along a survey, the station of any point on the survey, at a known distance from any station or plus, can be calculated. Thus, a point that is 227.94 ft beyond station 8 + 42.65 is 842.65 + 227.94 = 1070.59 ft from station 0 or at station 10 + 70.59. The distance between station 38 + 66.77 and station 54 + 43.89 is 5443.89 − 3866.77 = 1577.12 ft.

In the case of a route survey, the stationing is carried continuously along the line to be constructed. Thus if the survey is for a highway or a railroad, the stationing will be carried around the curves and will not be continuous along the straight lines which are eventually connected by curves. For the method of stationing that is used in surveys of this sort, see Chapter 13.

3-41. FIELD ROUTINE OF PROFILE LEVELING The principal difference between differential and profile leveling is in the number of foresights, or −S readings, taken from each setting of the level. In differential leveling only one such reading is taken, whereas in profile leveling any number can be taken. The theory is exactly the same for both types of leveling. A backsight, or +S reading, is taken on a benchmark or point of known elevation to determine the height of instrument. The rod is then held successively on as many points, whose elevations are desired, as can be seen from that position of the level, and rod readings, called *intermediate foresights* (IFS), are taken. The elevations of these points are calculated by subtracting the corresponding rod readings from the height of the instrument (HI). When no more stations can be seen, a foresight is taken on a turning point, the level is moved forward, and the process is repeated.

The method of profile leveling is illustrated in Fig. 3-46. The level having been set up, a sight is taken on a benchmark, not shown in the sketch. Intermediate foresights are then taken on stations 0, 1, 2, 2 + 65, 3, and 4. The sight is taken at station 2 + 65 because there is a decided change in the ground slope at that point. The distance to this point from station 2 is obtained either by pacing or by taping, the better method depending on the precision required. To determine the elevation of the bottom of the brook between stations 4 and 5, the level is moved forward after a foresight reading has been taken on the

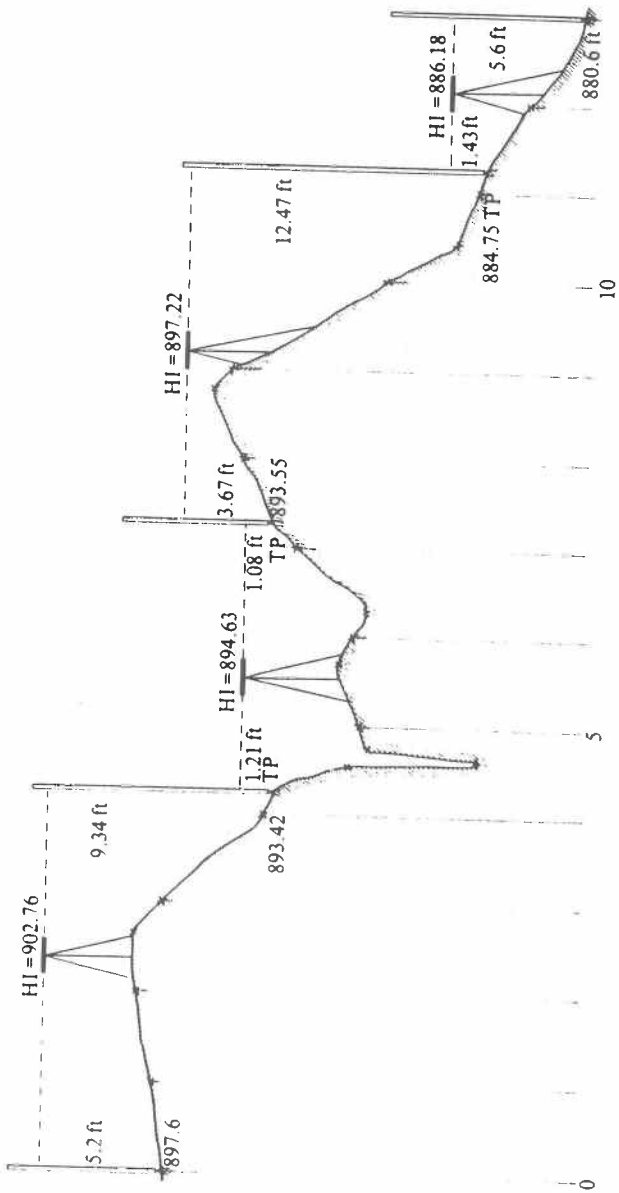


Figure 3-46. Profile leveling.

turning point just beyond station 4. With the level in the new position, a backsight is taken on the turning point and intermediate foresights are taken on stations 4 + 55, 4 + 63, 4 + 75, 5, 5 + 70, 6, 6 + 25, and 7, and lastly a foresight is taken on a turning point near station 7. From the third setup, a backsight is taken on the turning point and intermediate foresights are taken on stations 8, 8 + 75, 9, 10, 10 + 40, and 11, and a foresight is taken on a turning point near station 11. From the final setup shown in the figure, a backsight is taken on this turning point and intermediate foresights are taken on stations 12 and 13. Finally, a foresight is taken on a benchmark not shown in the sketch.

Readings have thus been taken at the regular 100-ft stations and at intermediate points wherever there is a decided change in the slope. The level has not necessarily been set on the line between the stations. In fact, it is usually an advantage to have the level from 30 to 50 ft away from the line, particularly when readings must be taken on intermediate points. More of the rod will then be visible through the telescope and the reading can be made more easily and quickly.

Benchmarks are usually established in the project area by differential leveling prior to running the profile leveling. When running the profile leveling, backsights and foresights on benchmarks and turning points must be taken with the same accuracy as that used to establish the elevations of the project benchmarks, usually to the hundredth of a foot (0.001 m). This is necessary in order to maintain the overall accuracy of the profile leveling. If intermediate foresights along profiles are taken along bare ground, they need only be read to the nearest tenth of a foot (1 cm). However, if the entire profile is a paved surface, it may be required to read the intermediate foresights to the hundredth of a foot, depending on the purpose of the profile. The profile leveling is then adjusted between previously established project benchmarks.

If benchmarks have not been established in advance, they should be established as the work progresses. Benchmarks may be from 10 to 20 stations apart when the differences of elevation are moderate, but the vertical intervals between benchmarks should be about 20 ft where the differences of elevation are considerable. These benchmarks should be so located that they will not be disturbed during any construction that may follow. Their elevations should be verified by running check levels.

The notes for recording the rod readings in profile leveling are the same as those for differential leveling except for the addition of a column for intermediate foresights. The notes and calculations for a portion of the profile leveling shown in Fig. 3-46 are given in Fig. 3-47.

3-42. PLOTTING THE PROFILE To facilitate the construction of profiles, paper prepared especially for the purpose is commonly used. This has horizontal and vertical lines in pale green, blue, or orange, so spaced as to represent certain distances to the horizontal and vertical scales. Such paper is

	P.	LE LEVELING					MAY 21, 1981
STA	BS	HI	FS	IFS	ELEV	LEVEL # 4096	LEVEL J. BROWN
BM	4 18	902 76			898 58	ROD # 18	ROD F SMITH
0				52	897 6		
1				46	898 2		
2				39	898 9		
2+65				38	899 0	Σ BS = 9.06	884 75
3				49	897 9	Σ FS = -22 89	-898 58
4				92	893 6	-13 83	-13 83
TP-1	1.21	89463	934		893 42	Checks	
4+55				44	890 2		
+63				99	884 7		
+75				53	889 3		
5				50	889 6		
5+70				39	890 7		
6				45	890 1		
6+25				54	889 2		
7				22	892 4		
TP-2	3 67	89722	1 08		893.55		
8				24	894 8		
8+75				11	896.1		
9				19	895 3		
10				84	888 8		
10+40				113	885 9		
11				122	885 0		
TP-3			12 47		884 75		
Σ BS =	+9 06	Σ FS =	22 89				

Figure 3-47. Profile-level notes.

called *profile paper*. If a single copy of the profile is sufficient, a heavy grade of paper is used. When reproductions are necessary, either a thin paper or tracing cloth is available. The common form of profile paper is divided into $\frac{1}{4}$-in. squares by fairly heavy lines. The space between each two such horizontal lines is divided into five equal parts by lighter horizontal lines, the distance between these light lines being $\frac{1}{20}$ in. In order to accentuate the differences of elevation, the space between two horizontal lines can be considered as equivalent to 0.1, 0.2, or 1.0 ft, and the space between two vertical lines as 25, 50, or 100 ft, according to the total difference of elevation, the amount of vertical exaggeration desired, the length of the line, and the requirements of the work.

To aid in estimating distances and elevations, each tenth vertical line and each fiftieth horizontal line are made extra heavy. A piece of profile paper

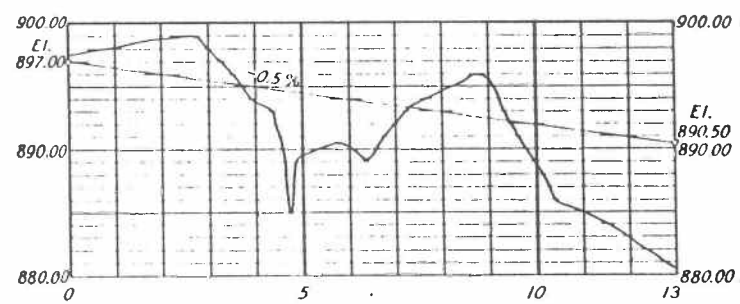
showing the profile for the level notes given in Fig. 3-47 is i ated in Fig. 3-48. The elevation of some convenient extra-heavy horizontal line is assumed to be 900 ft and a heavy vertical line is taken as station 0. Each division between horizontal lines represents 1 ft and each division between vertical lines represents 100 ft, or 1 station. As the elevation or station of each printed line is known, the points on the ground surface can be plotted easily. When these points are connected with a smooth line, an accurate representation of that ground surface should result.

It is often a convenience to have several related profiles plotted on the same sheet. Thus in designing a pavement for a city street three profiles may appear, namely, those of the center line and of the two curb lines. This is practically a necessity when there is any considerable difference in elevation between the two sides of the street. When only a single copy is being made, different colored inks can be used to distinguish one profile from another. When reproductions are to be made, different kinds of lines, such as different combinations of dots and dashes, are used.

As indicated in Section 17-13, it is possible to prepare a profile from a topographic map.

When it is desirable to have both the plan and the profile appear on the same sheet, paper which is half plain and half profile-ruled is used. Plans for highways and sewers often are prepared in this manner, the location plan appearing at the top of the sheet and the profile below it.

Any information that may make the profile more valuable should be added. Thus the names of the streets or streams and the stations at which they are crossed should appear. The locations and elevations of benchmarks may appear as notes on the map. There should be a title giving the following information: what the profile represents, its location, its scales, the date of the survey, and the names of the surveyor and the draftsman.



Preliminary Location
Saline-Manchester Highway
Washtenaw County, Mich.
Scales: Horiz. 1in. = 400 ft.
Vert. 1in. = 15 "
Surveyed Sept. 14, 1959 by J. Brown
Platted Sept. 28, 1959 by A. Wishnefski

Figure 3-48. Profile.

Chapter 15
Topographic Surveys

15-1. GENERAL PROCEDURES Topographic surveying is the process of determining the positions, on the earth's surface, of the natural and artificial features of a given locality and of determining the configuration of the terrain. The location of the features is referred to as *planimetry*, and the configuration of the ground is referred to as *topography* or *hypsography*. The purpose of the survey is to gather data necessary for the construction of a graphical portrayal of planimetric and topographic features. This graphical portrayal is a topographic map. Such a map shows both the horizontal distances between the features and their elevations above a given datum. On some maps the character of the vegetation is shown by means of conventional signs.

Topographic surveying or mapping is accomplished by ground methods requiring the use of the transit, plane table and alidade, level, hand level, tape, and leveling rod in various combinations. Total station EDM's are used to advantage in topographic surveying. The vast majority of topographic mapping is accomplished by aerial photogrammetric methods, as described in Chapter 16. In the photogrammetric methods, however, a certain amount of field completion and field editing must be done by ground methods described in this chapter.

The preparation of a topographic map, including the necessary control surveys, is usually the first step in the planning and designing of an engineering project. Such a map is essential in the layout of an industrial plant, the location of a railway or highway, the design of an irrigation or drainage system, the development of hydroelectric power, city planning and engineering, and landscape architecture. In time of war, topographic maps are essential to persons directing military operations.

15-2. SCALES AND ACCURACY Since a topographic map is a representation, on a comparatively small plane area, of a portion of the surface of the earth, the distance between any two points shown on the map must have a known definite ratio to the distance between the corresponding two points on the ground. This ratio is known as the scale of the map. As stated in Section 8-33, this scale can be expressed in terms of the distance on the map, in inches, corresponding to a certain distance on the ground, in feet. For example, a scale may be expressed as 1 in. = 200 ft. The scale can be expressed also as a ratio, such as 1 : 6000, or as a fraction, as 1/6000. In either of these last two cases, 1 unit on the map corresponds to 6000 units on the ground. A fraction indicating a scale is referred to as the *representative fraction*. It gives the ratio of a unit of measurement on the map to the corresponding number of the same units on the ground.

The scale to which a map is plotted depends primarily on the purpose of the map, that is, the necessary accuracy with which distances must be measured or scaled on the map. The scale of the map must be known before the field work is begun, since the field methods to be employed are determined largely by the scale to which the map is to be drawn. When the scale is to be 1 in. = 50 ft, distances can be plotted to the nearest $\frac{1}{2}$ or 1 ft, whereas if the scale is 1 in. = 1000 ft, the plotting will be to the nearest 10 or 20 ft and the field measurements can be correspondingly less precise.

15-3. METHODS OF REPRESENTING TOPOGRAPHY Topography may be represented on a map by hachures or hill shading, by contour lines, by form lines, or by tinting. *Hachures* are a series of short lines drawn in the direction of the slope. For a steep slope the lines are heavy and closely spaced. For a gentle slope they are fine and widely spaced. Hachures are used to give a general impression of the configuration of the ground, but they do not give the actual elevations of the ground surface.

A *contour line*, or *contour*, is a line that passes through points having the same elevation. It is the line formed by the intersection of a level surface with the surface of the ground. A contour is represented in nature by the shoreline of a body of still water. The *contour interval* for a series of contour lines is the constant vertical distance between adjacent contour lines. Since the contour lines on a map are drawn in their

discu in this chapter is eliminated, except for earthwork of limited extent. The measurements for the determination of volumes can be made directly from stereoscopic models or on topographic maps prepared by photogrammetric methods (see Sections 16-10, 17-13, and 17-14).

17-2. CROSS SECTIONS A cross section is a section taken normal to the direction of the proposed center line of an engineering project, such as a highway, railroad, trench, earth dam, or canal. A simple cross section for a railroad embankment is shown in Fig. 17-1. The cross section for a highway or an earth dam would have similar characteristics. It is bounded by a base b, side slopes, and the natural terrain. The inclination of a side slope is defined by the horizontal distance s on the slope corresponding to a unit vertical distance. The slope may be a rise (in excavation) or a fall (in embankment). A side slope of $3\frac{1}{2}:1$, for example, means that for each $3\frac{1}{2}$ ft of horizontal distance the side slope rises or falls 1 ft. This can be designated as $3\frac{1}{2}:1$ or 1 on $3\frac{1}{2}$.

17-3. PRELIMINARY CROSS SECTION In making a preliminary estimate and in determining the location of a facility, such as a highway or railroad, a preliminary line is located in the field as close to the final location of the facility as can be determined from a study of the terrain supplemented by maps or aerial photographs of the area. The preliminary line is stationed, and profile levels are taken. The configuration of the ground normal to the line is obtained by determining the elevations of points along sections at right angles to the line. This is identical to the process of obtaining elevations for topographic mapping described in Section 15-6.

The values of the elevations and the corresponding distances out to the right or left of the preliminary line can be plotted on specially printed cross-section paper, at a relatively large scale of from 1 in. = 5 ft to 1 in. = 20 ft (however, see Sections 17-13 and 17-14). When the location and grade of a trial line representing a tentative location of the center line of the facility have been established, the offset distance from the preliminary line to the

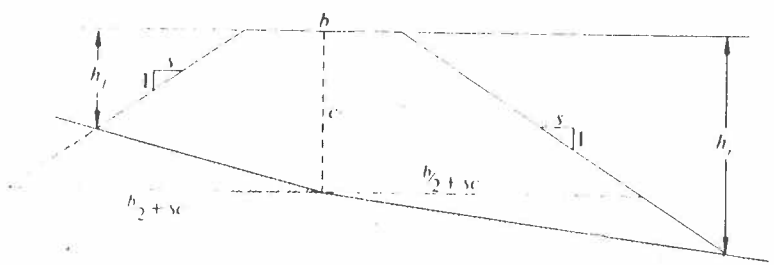


Figure 17-1. Cross-section for railroad embankment.

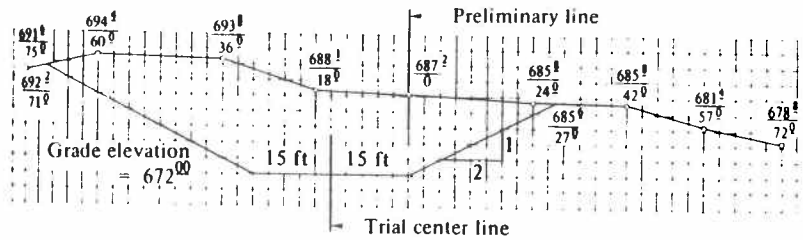


Figure 17-2. Preliminary cross section

trial line is plotted, and the grade elevation of this trial line is plotted in relation to the terrain cross section. In Fig. 17-2 the elevations of, and distances to, the points plotted on the ground line were determined with reference to the preliminary center line. These are shown as fractions, with elevations as the numerators and distances from the preliminary line as the denominators. The offset distance of 15 ft from the preliminary line to the trial line is plotted, and the base of the roadbed is plotted at grade elevation 672.00 ft. At the edges of the roadbed, the side slopes of 2 : 1 are laid off and drawn to intersect the terrain line at the scaled distances and elevations shown as fractions lying under the terrain line. These points of intersection are called *catch points*.

The cross-sectional area bounded by the base, the side slopes, and the ground line of each trial cross section along the trial line is determined from the plotted cross section by using a planimeter or by computation based on the formulas for areas given in Section 8-20. This procedure is discussed in Section 17-8. The volumes of excavation and embankment for this trial line are computed from the successive areas and the distances between the areas by the methods described in Sections 17-9 to 17-11. The volumes for various trial lines are compared. The necessary changes in line and grade are then made to locate the final line and establish the final grade. This location will require a minimum of earthwork costs and, in the case of a highway project, for example, it will at the same time meet the criteria of curvature, maximum grade, and safe sight distances.

17-4. FINAL CROSS SECTIONS The line representing the adopted center line of a facility is staked out in the field and stationed. This line is located by computing and running tie lines from the preliminary line as discussed in Section 8-23. Deflection angles are measured between successive tangents, and horizontal curves are computed and staked out. Reference stakes are sometimes set opposite each station on both sides of the center line at distances of 25, 50, or 100 ft from the center line. These stakes are used to relocate the center line after grading operations are begun. Stakes at a distance on either side equal to half the base width are sometimes driven to facilitate taking final cross sections and setting construction or slope stakes. The center line and the reference lines are then profiled.

ground surface, a topographic map containing contour lines shows not only the elevations of points on the ground, but also the shapes of the various topographic features, such as hills, valleys, escarpments, and ridges.

The classical illustration used to show the relationship between the configuration of the ground and the corresponding contour lines is shown in Fig. 15-1. This illustration used to be printed on the backs of the U.S. Geological Survey quadrangle maps along with an explanation of the topographic map and how it is interpreted and used. Unfortunately the U.S. Geological Survey discontinued this feature of its quadrangle series in the early 1950s. The upper part of the illustration shows a stream lying in a valley between a cliff on the left and a rounded hill on the right. The stream is seen to empty into the ocean in a small bay protected by a sand hook. Other features such as terraces, gulleys, and a gentle slope behind the cliff can be identified. An abrupt cliff to the right of the sand hook plunges almost vertically to the ocean. The lower part of the figure is the contour line or topographic map representation of this terrain. The contour interval of this map is 20 units and could represent either 20 ft or 20 m.

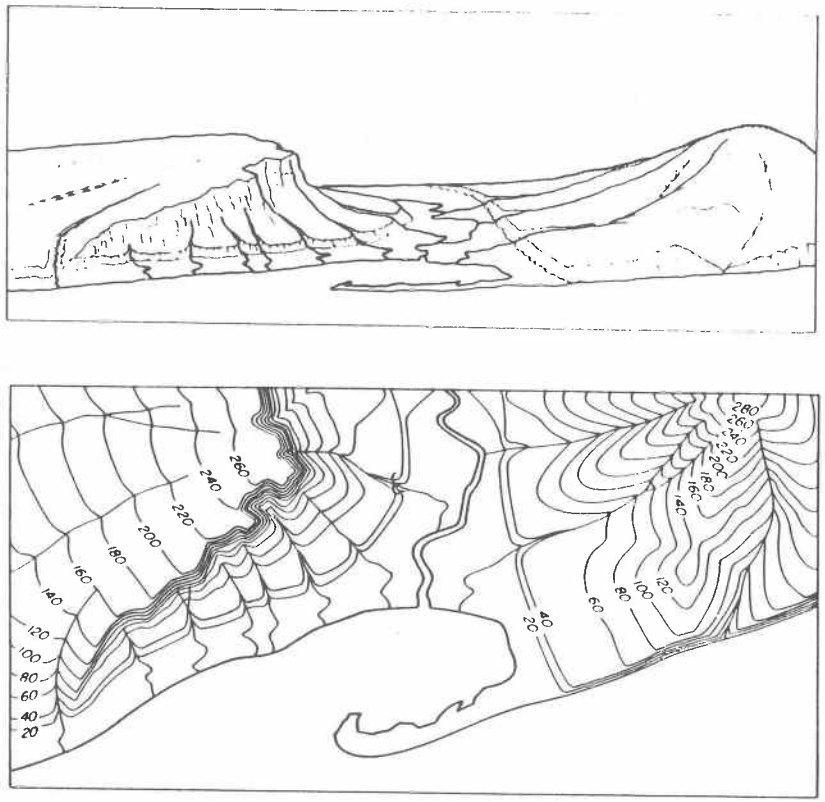


Figure 15-1. Contour line representation of terrain. By permission of U.S. Geological

On maps intended for purposes of navigation, peaks and hilltops along the coast are sometimes shown by means of *form lines*. Such lines resemble contours, but are not drawn with the same degree of accuracy. All points on a form line are supposed to have the same elevation, but not enough points are actually located to conform to the standard of accuracy required for contour lines.

On aeronautical charts and on maps intended for special purposes, such as those that may accompany reports on some engineering projects, elevations may be indicated by tinting. The area lying between two selected contours is colored one tint, the area between two other contours another tint, and so on. The areas to be flooded by the construction of dams of different heights, for example, might be shown in different tints.

15-4. CONTOUR LINES The configuration of the ground and the elevations of points are most commonly represented by means of contour lines, because contours give a maximum amount of information without obscuring other essential detail portrayed on the map. Some of the principles of contours are represented in Fig. 15-2. Four different contour intervals are shown in views (a), (b), (c), and (d). The steepness of the slopes can be determined from the contour interval and the horizontal spacing of the contours. If all four of these sketches are drawn to the same scale, the ground slopes are the steepest in (d), where the contour interval is 20 ft, and are the flattest in (c), where the interval is 1 ft.

The elevation of any point not falling on a contour line can be determined by interpolating between the two contour lines that bracket the point. Quite

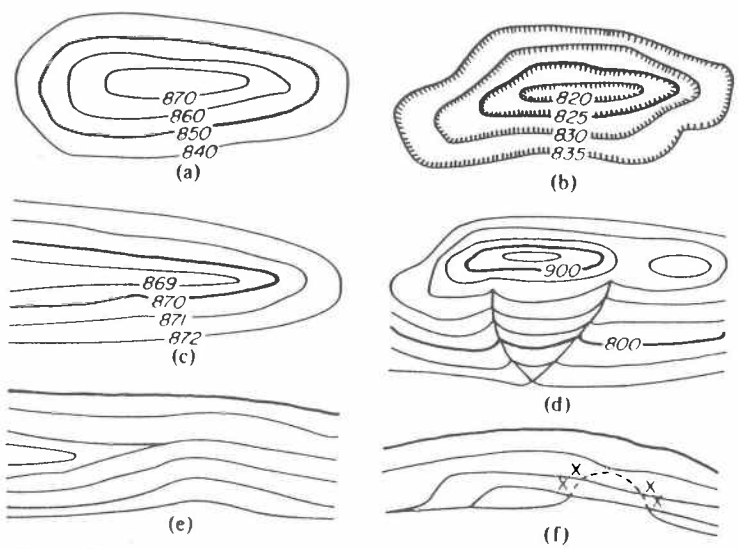


Figure 15-2. Contour lines

often when the scale of the map is large and the terrain is flat, the successive contours are spaced so far apart horizontally that interpolation between adjacent contours does not have much significance. Therefore, in such an instance, the accuracy and utility of the map is greatly increased by showing the elevations of points at regular intervals in some form of a grid pattern. Elevations between these points are then determined by interpolation. Spot elevations are shown on Fig. 15-3.

A contour cannot have an end within the map. It must either close on itself, or commence and end at the edges of the map. A series of closed contours represents either a hill or a depression. From the elevations of the contour lines shown, a hill is represented in Fig. 15-2(a) and a depression in view (b). As indicated, a depression contour is identified by short hachures on the downhill side of the contour. A ravine is indicated by the contours in Fig. 15-2(c). If the elevations were reversed, the same contours would represent a ridge. View (e) is incorrect, as two contours are shown meeting and continuing as a single line; this would represent a knife-edged ridge or ravine, something not found in nature. View (f), if not incorrect, is at least unusual. Several contours are shown merging and continuing as a single line. This would be correct only in the case of a vertical slope or a retaining wall.

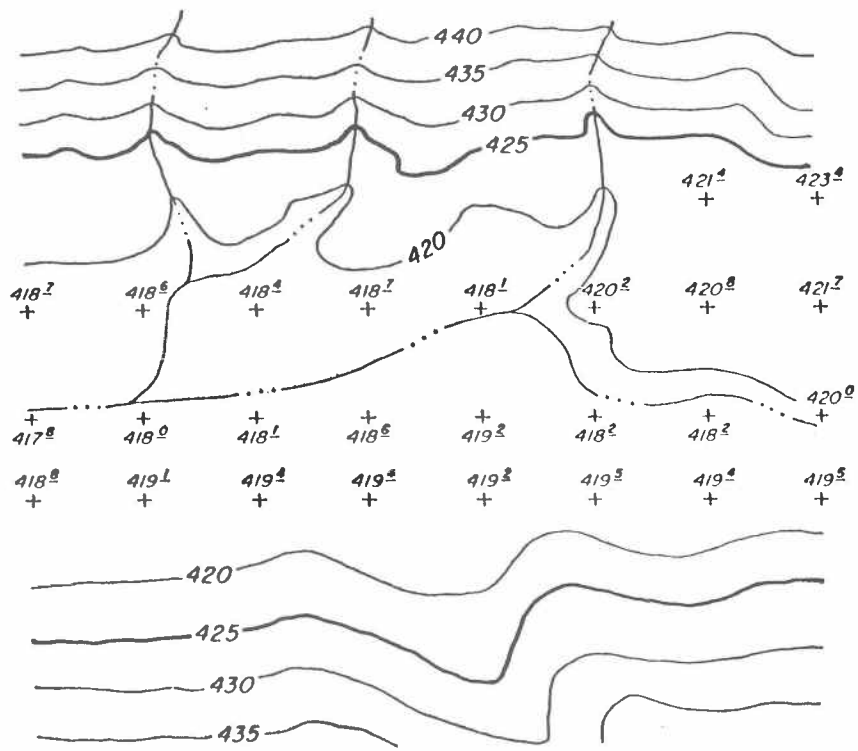


Figure 15-3. Spot elevations to supplement contour lines

Also, one contour is shown to cross two others. Thus each point marked x has two elevations, a condition found only at a cave or an overhanging cliff.

A series of equally spaced contour lines represents a constant slope along a line normal to the contours. A series of straight, parallel, equally spaced contours represents man-made excavations or embankments. The steepest direction from any point on a topographic map is that which runs normal to successive contour lines near the point.

The drainage of the terrain is the primary agent in shaping the topography. Its influence on the shape of the contour lines can be seen in Figs. 15-1, 15-2 (d), and 15-3. Note that as contour lines cross gulleys or streams or other drainage features, the contour lines form modified V's pointing upstream. The form of the V's is determined by the type of underlying soil or rock. In general, if the underlying material is fine grained like a clay soil, the V will be smooth and rounded, and if the material is coarse and granular, the V will be quite sharp.

As a convenience in scaling elevations from a topographic map, each fifth contour is drawn as a heavier line. This is called an *index contour*. When the interval is 1 ft, contours whose elevations are multiples of 5 ft are shown heavy. When the interval is 10 ft, the heavy contours have elevations that are multiples of 50 ft. Enough contours should be numbered to prevent any uncertainty regarding the elevation of a particular contour. Where the contours are fairly regular and closely spaced, only the heavy contours need be numbered.

15-5. FIELD METHODS Among the factors that influence the field method to be employed in the compilation of a topographic map are the scale of the map, the contour interval, the type of terrain, the nature of the project, the equipment available, the required accuracy, the type of existing control, and the extent of the area to be mapped. The area to be mapped for highway or railroad location and design takes the form of a strip with a width varying from 100 ft to perhaps more than 1000 ft (30 to 300 m). The control lines are the sides of a traverse which have been established by a preliminary survey and which have been stationed and profiled as outlined in Chapter 3. The method of locating topography most commonly employed for this purpose is the *cross-section* method.

To make an engineering study involving drainage, irrigation, or water impounding or to prepare an accurate map of an area having little relief, each contour line must be carefully located in its correct horizontal position on the map by following it along the ground. This is the *trace contour* method.

When an area of limited extent is moderately rolling and has many constant slopes, points forming a grid are located on the ground and the elevations of the grid points are determined. This is the *grid* method of obtaining topography.

EXERCISE 4. CHANNEL CROSS-SECTION SURVEY AND FLOW MEASUREMENT

The purpose of this exercise is to gain experience in surveying, especially as applicable to channel geomorphology and hydrology. In addition, you will measure your pace, a useful tool for estimating distances. In geomorphology and hydrology, level surveys are frequently conducted to document changes in landforms (notably river channels) or to measure the slope of the land or water surface for calculations of runoff or channel flow. The precision required of the survey depends on the purpose to which the data will be put.

We will conduct our exercise on Strawberry Creek near the confluence of the North and South forks, just west of the Life Sciences Building on campus. The North Fork drains an urbanized basin (about 0.6 mi^2) north of campus. The South Fork drains a largely undeveloped basin east of the stadium. See the booklet, "Strawberry Creek: a walking tour of campus natural history" for more information on the creek. Also see Charbonneau and Resh (1992) for more background on the stream and the recent efforts to restore its ecological functions.

Sketch Map and Location Map

In river channel surveying, it is a good idea to draw a *sketch map* of the study reach showing cross section locations, benchmarks, and their relation to landmarks. You can measure distances (and compass bearings) from landmarks to your benchmarks (and cross section endpoints) and record this information on your sketch map, as illustrated in an example sketch map shown in Figure 1.

It is also important to locate your study site as nearly as possible on a *location map* at a smaller scale (i.e., covering a larger area), such as the USGS 7.5-minute topographic quadrangle series (Figure 2). These maps can be enlarged to provide a better base map. For this exercise, we will also locate our study site on a larger-scale campus map (Figure 3). Another source of base maps is aerial photography. Enlargements of aerial photographs make particularly good location maps, except for cases of closed canopy tree cover over the channel.

You should also draw a *sketch cross section* at your surveyed cross sections, noting vegetation, substrate, and other features. This is essentially a graphic note-taking that can provide useful information for interpretation of your surveyed cross sections.

Surveying Procedure

Taking notes in a survey is basically an accounting procedure in which you keep track of differences in elevation and

distances. The standard form for these notes is shown in Figure 4, and the procedure for the survey is briefly summarized below. For more detailed discussion, see Dunne and Leopold (1978:652-656) and Gordon et al. (1992:288-345). Begin your notes with a title (e.g., Cross Section Survey, North Fork Strawberry Creek), date and time, weather, and surveying party identified with symbols for the person on the instrument, on the rod, and taking notes as shown in the sample notes page (Figure 5).

1. Set up the instrument and shoot to the benchmark (BM). This is called a *backshot* (BS) because you're shooting from the unknown *instrument height* (HI) back to the known BM. "Back" refers not to forwards or backwards on the ground, but whether you are going forward from the known to the unknown or back from the unknown (here, the HI) to the known (here, the BM). Backshots are also called "+ shots" because you add them to the BM elevation to compute the HI.

2. Now you know the HI, *instrument height* (actually, it's the instrument *elevation*, NOT the height above the ground). You can shoot from the known HI to the unknown ground elevations on your cross section. This is called a *foreshot* (FS) because you are shooting forward from the known to the unknown. Subtract the FS from the HI to compute the *elevation* (EL) of each point.

3. It is possible to carry levels long distances from a benchmark using *turning points* (TPs). The procedure is this: Let's say you need the elevation of a point that you cannot see from your present instrument station. Have the rod-person place the rod on a stable site such as the top of a boulder or a distinctive spot in the pavement. (It is sometimes useful, but not necessary, to mark turning points with paint or other markings so that you can reshoot them later.) You first make a foreshot to the TP, establishing its elevation. After checking the shot, you move the instrument to a new station, from which you can see the TP but which gets you within sight of (or at least partway to) the point you need to survey. You set the instrument up and shoot back to the TP. (This is a *backshot*, establishing your new HI; see notation for turning points in Figure 4.) All this time the rod person has not moved, except to rotate the rod so you can read it from the new instrument station.

4. You can move all over the landscape, carrying levels along via Tps, but it is a good idea to *close your survey* by surveying back to the original BM (or another BM) to see how close your computed elevation (from carried levels) comes to the actual, starting elevation. This provides a measure of your surveying accuracy.

Figure 5 shows

An illustrative example of notes from a survey using TPs (no. 1 and 2) to carry levels from a benchmark (BM) to establish the elevation of the left-bank pin (LBP) and right-bank pin (RBP) on a cross section (designated 14+57) located 200 ft upstream of the Hwy 32 bridge over Stony Creek. The symbols indicate that GMK was on the level and GM was carrying the rod for the survey. Note that each TP includes a FS to establish its elevation, then a BS in which the level (instrument) elevation is established. For the sake of the example, the elevation of the benchmark here (a brass cap) is set as 100.00 ft. The level survey indicated that the left bank pin is at 100.80 ft elevation, the right bank pin at 99.88. After obtaining these data, the survey crew returned to the bridge, two new TPs (no. 3 and 4) en route. The elevation carried back to the BM was 100.01, indicating a net closure error of 0.01 ft over the survey.

Using the Level

The level is a sensitive instrument and must be handled with great care. When transporting the level by automobile, it should be carried on a seat or in a box cushioned with sleeping bags, pads, and the like. **The level should never be transported on the bed of a pickup truck** unless within a box and thoroughly cushioned by several feet of foam and placed at the front end of the bed.

When setting up the tripod, it is best to fully extend all three legs. The legs can be spread out to lower the instrument on the desired eye height. Place the legs on the ground so that the dome head looks level, and walk around the tripod, stepping hard on each leg, driving the points into the ground. This is essential to prevent shifting of the tripod later.

Mount the level on the tripod. Leaving the anchor bolt slightly loose, move the level around on the dome head until the bubble level is reading approximately level. Small adjustments can be made later with the three adjustment knobs.

The department's automatic level is available for use in term projects for this course. Make arrangements in advance with staff in Rm 309 Wurster Hall. You are considered to be trained and authorized to borrow the level after this lab exercise.

Horizontal Distances (Stationing)

Horizontal distances can be measured by tape, stadia intercept, or when precision is not required, by pacing or scaling from a large scale map. *Tapes* are available in English or metric units. If your tape is in English units (feet), note whether it reads in inches or tenths of feet (the latter preferred for ease of computation). An alternative to flat, thin

survey tapes is *survey rope*, which is thicker and round, and much less likely to fail under tension or in the wind.

The *stadia intercept method* relies on the principle of parallax: things appear smaller with increasing distance away from the level. When you look through the eye piece on the level, you see three horizonal lines: a long center line and shorter upper and lower stadia lines. The level is designed so that whatever distance you see encompassed between the upper and lower stadia is equivalent to 1/100th of the distance to the object. Thus, if you look through the eye piece and read 4.57 ft and 4.23 ft for upper and lower stadia, respectively, the rod is 34 ft away from the level (4.57 - 4.23 = 0.34 x 100 = 34 ft).

ASSIGNMENT

We will break up into four teams, two on the North Fork, two on the South Fork. On each fork, one team surveys a long profile, while the second team makes flow measurements. Then, the teams switch equipment, and the second team surveys two cross sections while the first team makes flow measurements. **Please note your group:**

	NF	SF
Long Prof	A	B
Cross Sec	C	D

Each individual is responsible for drawing a sketch map of the entire study reach of both forks. Before we begin, measure your pace if you don't know it.

1. Pace

Working with a partner, estimate 100 feet by eye. Then lay out a tape to measure out 100 feet. Walk the distance, counting your paces, and calculate your pace. (A pace is two steps, as in right step to right step). Then use your pace to measure the distance along the stream channel from confluence to the staff gauges and upstream on both forks to the first culvert.

2. Long Profile and Cross Section Survey

Field Survey of Long Profile. The first team on each fork surveys a longitudinal profile for at least 100 ft along the

channel, proceeding upstream from the confluence of the forks. Survey channel bed at centerline and thalweg (if different), the water surface, *high water marks* (HWMs), and occasional bank tops. You can survey the *water surface elevation* (WSE) directly or, in steep channels like this, you can also put the rod on the bed and read the *water depth* (WD) on the rod to compute WSE. (WSE = bed elevation + WD) Your final plot of the long profile should show bed elevations, WSEs, HWMs, and bank tops. Survey *slope breaks* (points at changes in slope), such as the top and base of riffles or falls. As you proceed, note material making up the bed with distance along the channel. Leave the tape up along the channel so the second team can read the stationing of their cross section locations.

Field Survey of Cross Sections. The second team now surveys two cross sections along the 100-ft study reach. The choice of cross section location depends upon the purpose of the survey, which we will discuss in the field. Because you may use this cross section for a computation later, select at least one of the cross sections in a relatively straight, uniform reach. In your survey, be sure to shoot slope breaks, such as top-of-bank and toe-of-bank. The cross sections should be extend past the bank tops onto the floodplain or terrace.

Identify and survey high water marks (HWMs) recording the peak discharge of this (or another) year, on long profile and cross sections. Each HWM surveyed should be identified as to type of HWM (trash line, wash line) and quality of the HWM (clarity of line, your confidence in its accuracy) as excellent, good, fair, or poor.

Draw sketch cross sections in the field, looking downstream, for each of your surveyed cross sections. Note cross-channel stationing as you move across the cross section so you can tie your sketch section in with the surveyed cross section. The sketch cross sections are basically a form of organized note-taking: annotate your sketch cross section with details that you can incorporate into your plotted cross section later.

Plot the long profile by plotting symbols for the actual data points and connecting them with straight lines, as shown in Figure 6. Plot on graph paper at horizontal scale of 1 inch = 10 ft, vertical scale 1 inch = 2 feet. Express horizontal distances along the stream channel in stationing (ft) upstream of the confluence. Also note the survey datum, e.g. ft above mean sea level (MSL), ft of gauge height, etc. On your long profile, indicate the average slope over the entire surveyed reach, and for shorter sections that display distinct slopes. Also locate cross sections on the long profile, and annotate features of potential interest. What controls the steps in the profile? How deeply is the channel incised below the surrounding upland? Did

the stream overflow its banks this year?

Plot the cross sections (Figure 6) on graph paper using a scale of 1 inch = 5 ft for both horizontal and vertical. The cross sections should be oriented looking **downstream**. On your plotted surveyed cross sections, annotate with information from your sketch cross sections, such as bed material, vegetation, and other features.

Tabulate the survey data manually or on a spreadsheet and present as an appendix.

3. Sketch Map

Draw a sketch map of the study reach (from confluence upstream about 150 ft, include an approximate scale. On your sketch map, label features of interest, such as major check dams, eroding banks, deep pools, inflowing pipes, etc. Indicate flow direction with arrows labelled "flow". Include north arrow and approximate scale.

4. Flow Measurement

Measure flow using three methods: (1) installation of a portable flume, (2) measurement with mini-current meter using standard US Geological survey procedures (Rantz et al. 1982), and (3) estimation by timing a floating object.

Flume. To measure with the flume first requires an evaluation whether the flume can measure the flow (up to perhaps 0.5 cfs). Then it is necessary to seat the flume in the bed and adjust the wings so that all flow is passing through the flume (with minimal leakage around the sides or underneath), and the flume is level. In addition, the water must undergo a free fall off the downstream end, forcing flow to pass through *critical depth*, a concept we will discuss later in lecture. Record *h*, the height of water on the scale (in ft). The flow (in cfs) for this flume is given by: $Q = 4.22\ h^2$.

Current Meter Measurement. As discussed in lecture, a current meter measurement involves dividing the channel into imaginary vertical slices (termed *verticals*), measuring the area of each vertical, measuring the average velocity in each vertical, and summing the discharges in the individual verticals. An example of notes for a standard USGS current meter measurement is attached as Figure 7. Note that for a mini-current meter, velocity is computed as revolutions/sec = ft/sec of velocity, for a full-size (Price AA) current meter, the relation is revolutions/sec = 2.18 ft/sec.

Orange Peel Velocity Estimate. This method involves measuring the time a floating object requires to travel a given distance (measured or paced out along the bank). It is often called the *orange peel method* because orange peels are preferred floating objects by virtue of their high visibility and nearly neutral buoyancy, which causes them to float just below the surface. Select a relatively uniform, simple stretch of channel, measure out a distance (such as ten or twenty feet for a small channel), station someone at the upstream end to drop objects in the water, another at the "finish line", and a third to be the timer. Adjust the measured velocity by a factor of 0.8 to reflect the expected difference between surface velocity and average column velocity, then multiply this value by an estimate of the cross sectional area of the this reach of channel (expressed in ft^2).

Some of the floating objects may get caught on the banks; their times would be nearly infinite so you can discard those runs. Objects that float through the reach will tend to be in the main current at the center of the channel; this is ok but this implies that the measured velocity reflects only the higher velocities near the center of the channel rather than the slower velocities near the margin. Thus, it is easy to over-estimate the discharge by applying the measured velocity to the entire cross sectional area.

You can also drop a large number of small floating objects and observe the variety of paths taken by them. Incorporation of visible elements into the current is a form of *flow visualization* that permits us to better see the complex patterns taken by individual parcels of water.

Tabulate results of each method in a table (Table 1). For current meter measurement, include intermediate information such as width, average depth, and average velocity.

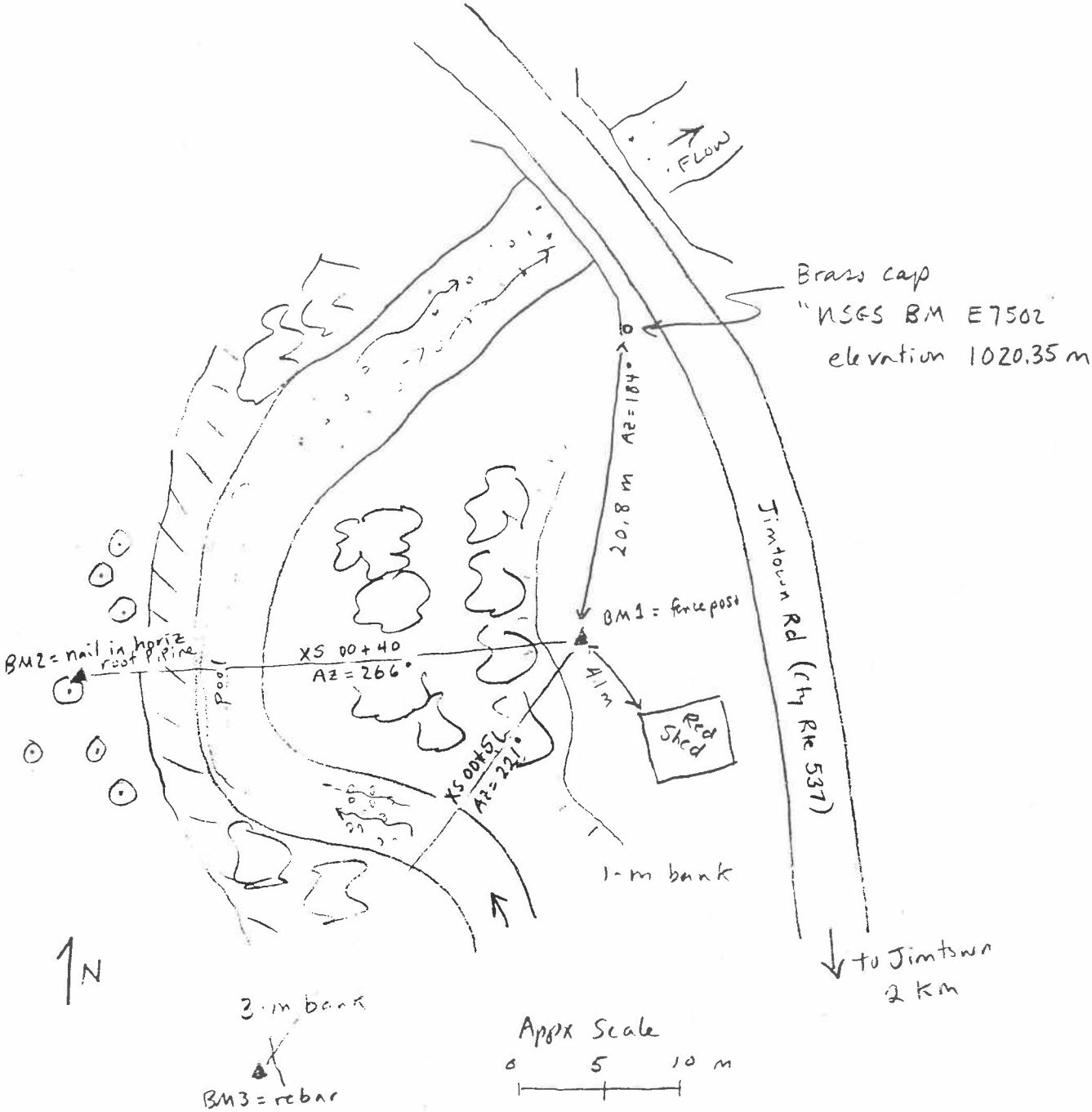
References Cited

Charbonneau, R., and V.H. Resh. 1992. Strawberry Creek on the University of California, Berkeley Campus: A case history of urban stream restoration. *Aquatic Conservation*. 2:293-307.

Rantz. S.E. and others. 1982. Measurement and computation of streamflow: Volume 1. Measurement of stage and discharge. *US Geological Survey Water Supply Paper* 2175.

Summary of Items to Turn In for the Exercise:

One-page write-up	Plotted long profile (annotated)
Location maps	Sketch map
Sketch cross sections	Plotted cross sections (annotated)
Tabulated long profile + cross section survey data (as appendix)	
Current meter measurement notes (photocopy from field book)	
Table summarizing flow measurements	



LEGEND

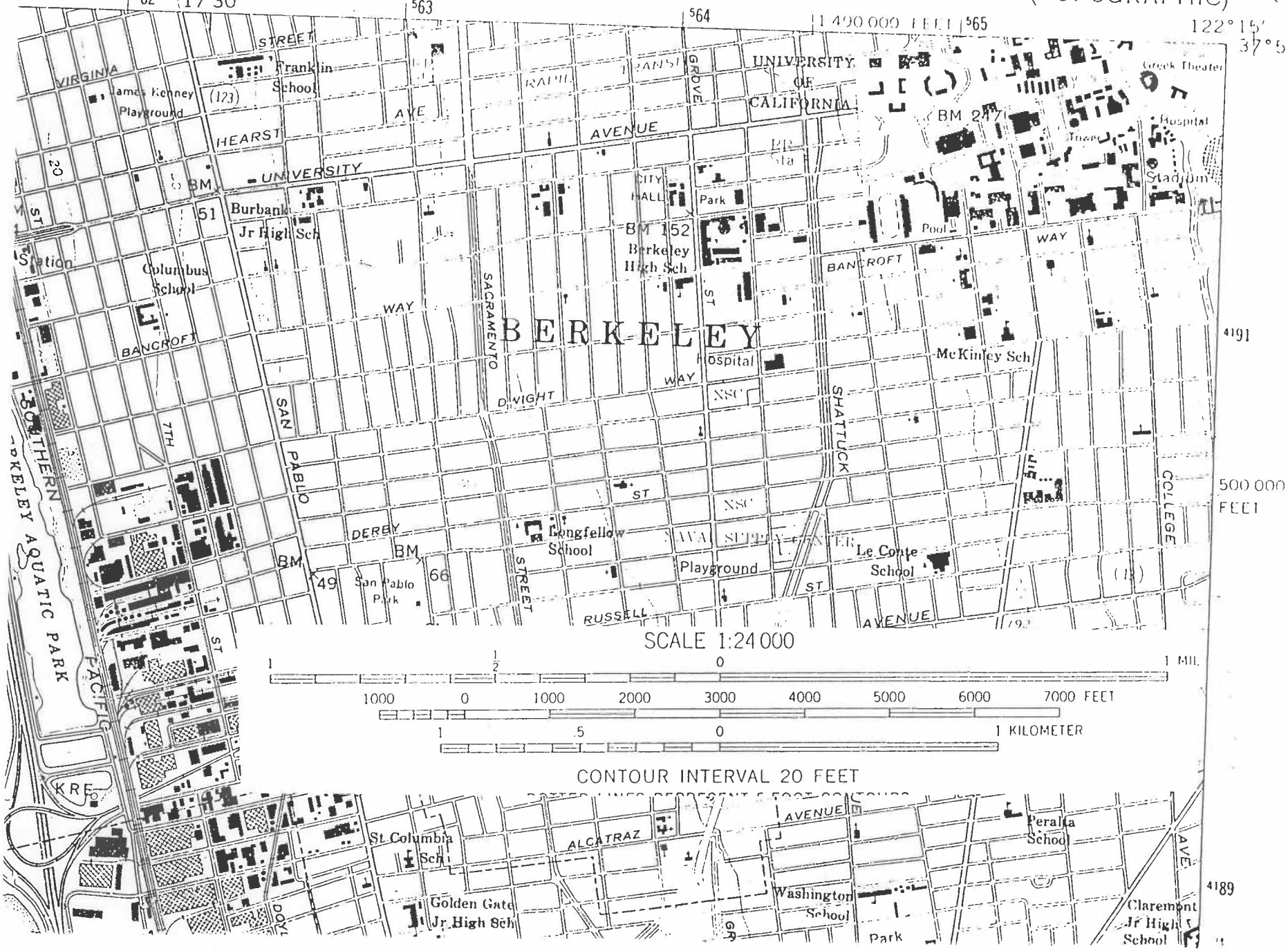
- Ponderosa pine
- willow
- escarpment / vertical bank
- Benchmark

Figure 1. Sample Sketch Map

Fig. 2. Excerpt from USGS quad sheet

OAKLAND WEST QUADRANGLE
CALIFORNIA
7.5 MINUTE SERIES (TOPOGRAPHIC)

1559 I NW (BRIONES VALL.)



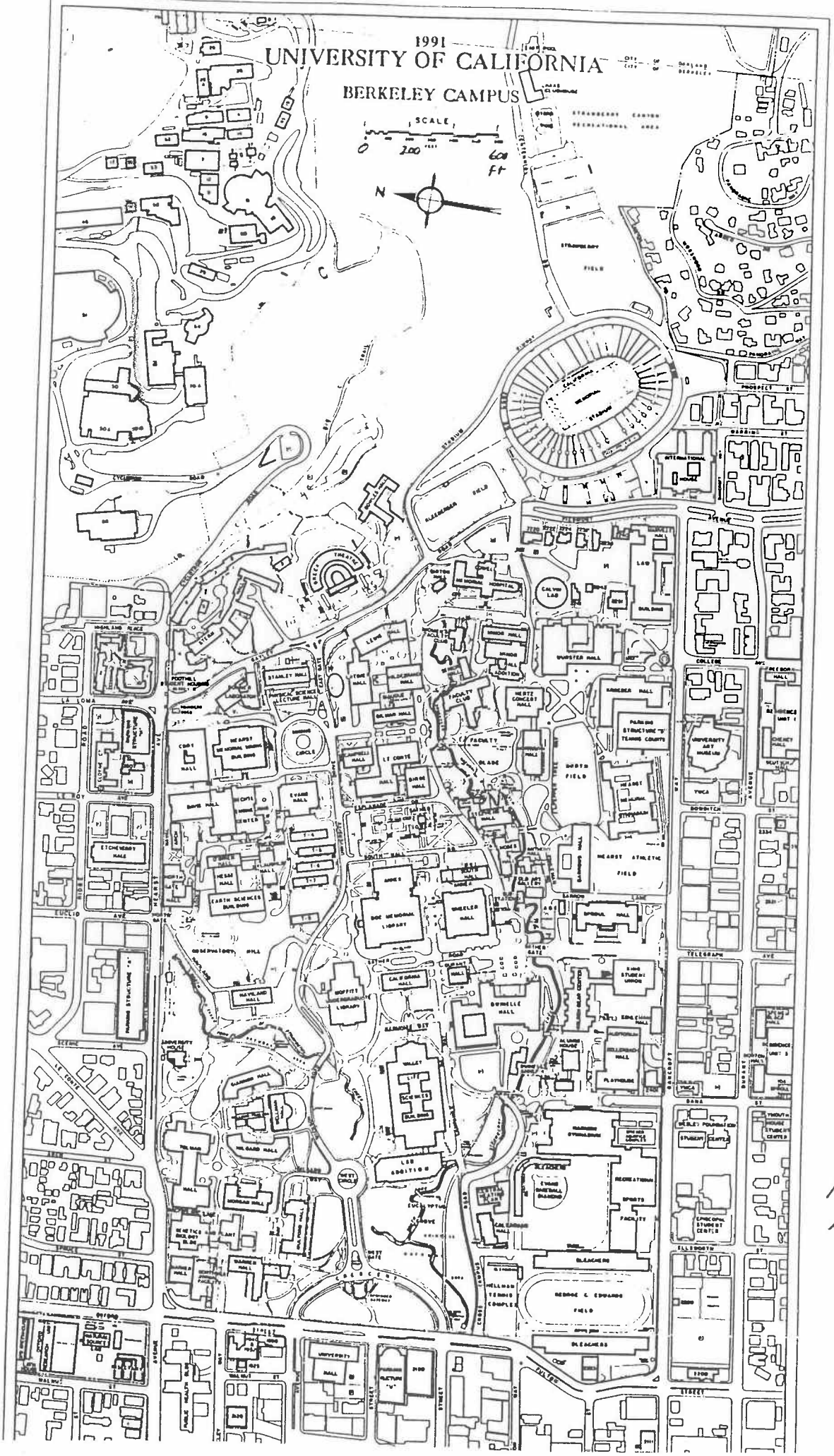
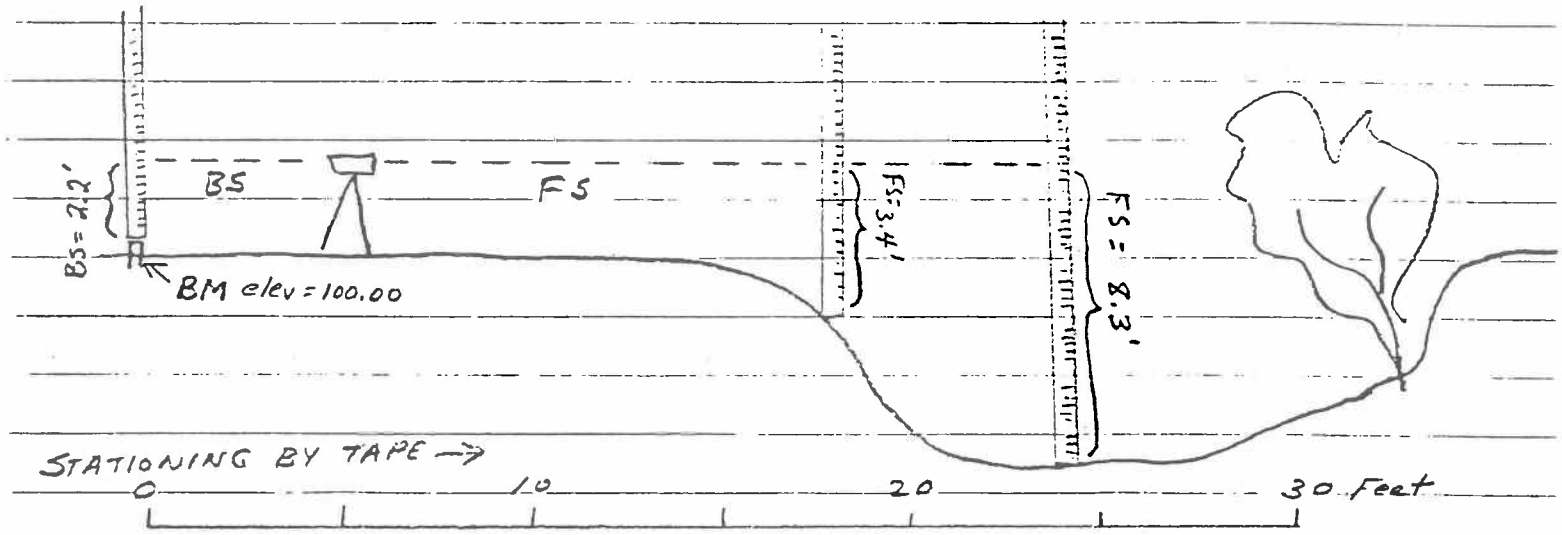


Figure 3.
Location
map

Figure 4. [redacted]



STA'	BS' (+)	FS' (−)	HI'	EL'	NOTES
0	2.2			100.0	BM = rebar pin elev. arbitrary
			102.2		
18		3.4		98.8	
24		8.3		93.9	
TP1		7.2		95.0	
TP1	5.4			95.0	
			100.4		
TP2		9.4		91.0	
TP2	4.0			91.0	
			95.0		

— SAMPLE —

Stony Ck at Hwy 32 6/06/90

Level Survey to Xsec pins @ XS

Sunny, warm (~75° @ 1000)

14+57 (200' us bridge)

℞ GMK ϕ GM

STA	FS	BS	HI	EL	
BM1		5.42		100	
			105.42		
TP1	4.42			101.00	
TP1		2.67		"	
			103.67		
TP2	2.27			101.40	
TP2		3.50		"	
			104.90		
LBP	4.10			100.80	LBP on
RBP	5.02			99.88	RBP is
Close Survey					
TP3	4.86			100.04	
TP3		2.09		"	
			102.13		
TP4	3.65			98.48	
TP4		7.03		"	
			105.51		
BM1	5.50			101.01	
			Closure error	0.01	

BM1 = brass cap, Glenn County DPW No. GC-088-74-12

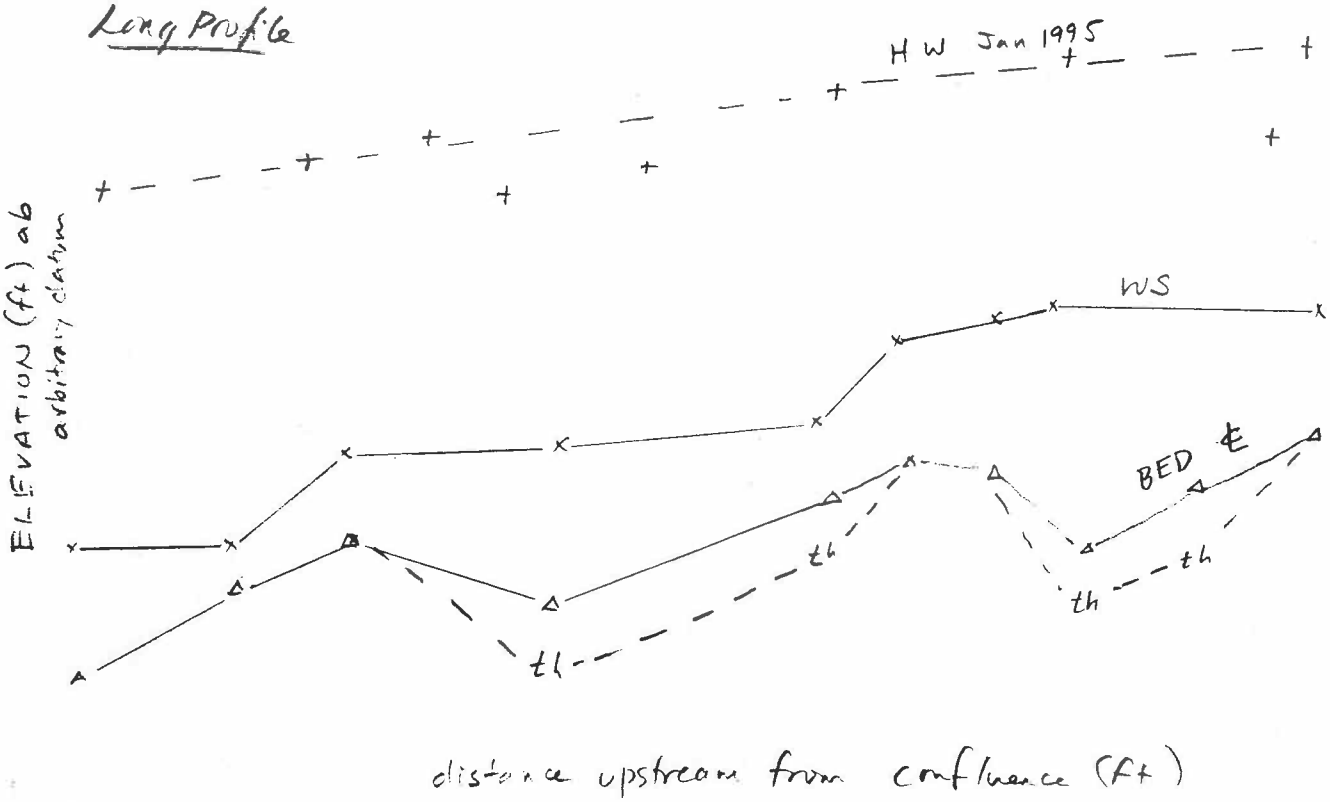
XS 14+57 = rebar at base of fence post nail in horizontal root, base 3' diam sycamore

Figure 3.

Figure 6. 2/01/95

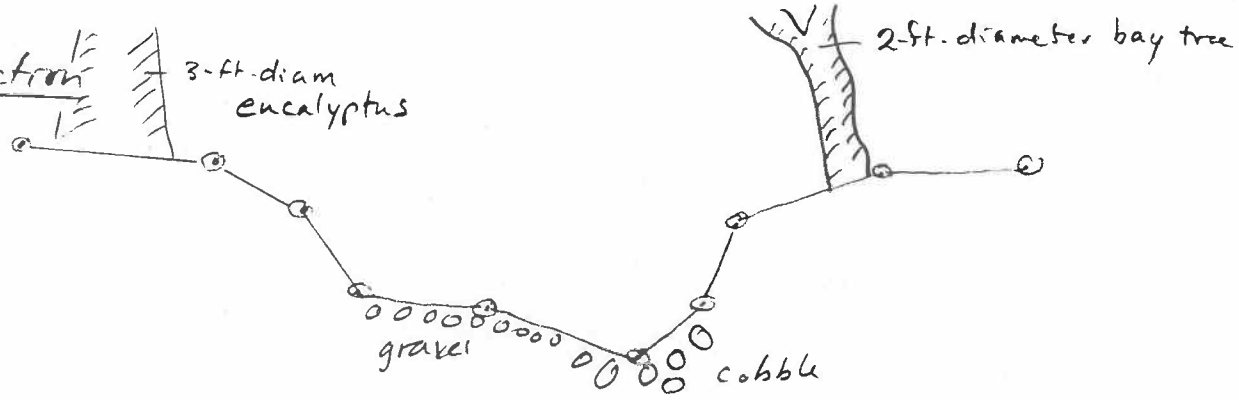
GUIDELINES FOR PLOTTING SURVEY DATA

Long Profile



Generally you use the highest HWMs to reconstruct the peak Q because lower HWMs probably represent recession limb stages. WSEs should decline monotonically downstream. Bed elevations will rise + fall, depending on local channel features.

Cross Section



Plot looking downstream. Annotate with information from your sketch cross section

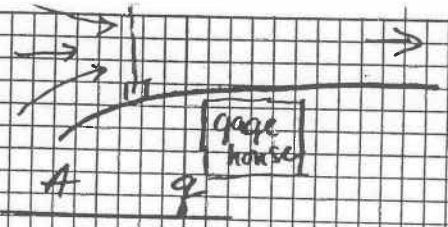
Q Msmt at LADWP Flume

GMK sta — MLS

spin - 90+

Sta	W	d	rev	sec	vel
4.4	LEW @ 1430		WT = 12°C		gh = ?
4.9	0.75	0.9	70	28	5.45
5.4	0.50	0.9	60	27	4.85
5.9	0.5	0.9	50	24	4.55
6.4	0.5	0.9	50	23	4.74
6.9	0.5	0.9	40	21	4.16
7.4	0.5	0.9	40	22	3.96
7.9	0.5	0.9	40	27	3.23
8.4	0.5	0.9	30	23	2.84
8.9	0.75	0.9	30	30	2.19
9.9	1.0	0.85	20	25	1.76
10.9	1.0	0.85	10	20	1.11
11.9	1.0	0.8	10	26	0.86
12.9	0.6	0.8	10	37	0.59
13.0	REW @ 1448				

- Rating = good



0.68	3.68
0.45	2.18
0.45	2.05
0.45	2.13
0.45	1.87
0.45	1.78
0.45	1.45
0.45	1.28
0.66	1.98
0.85	1.50
0.85	0.95
0.80	0.69
0.48	0.28

Q = 21.33 cfs

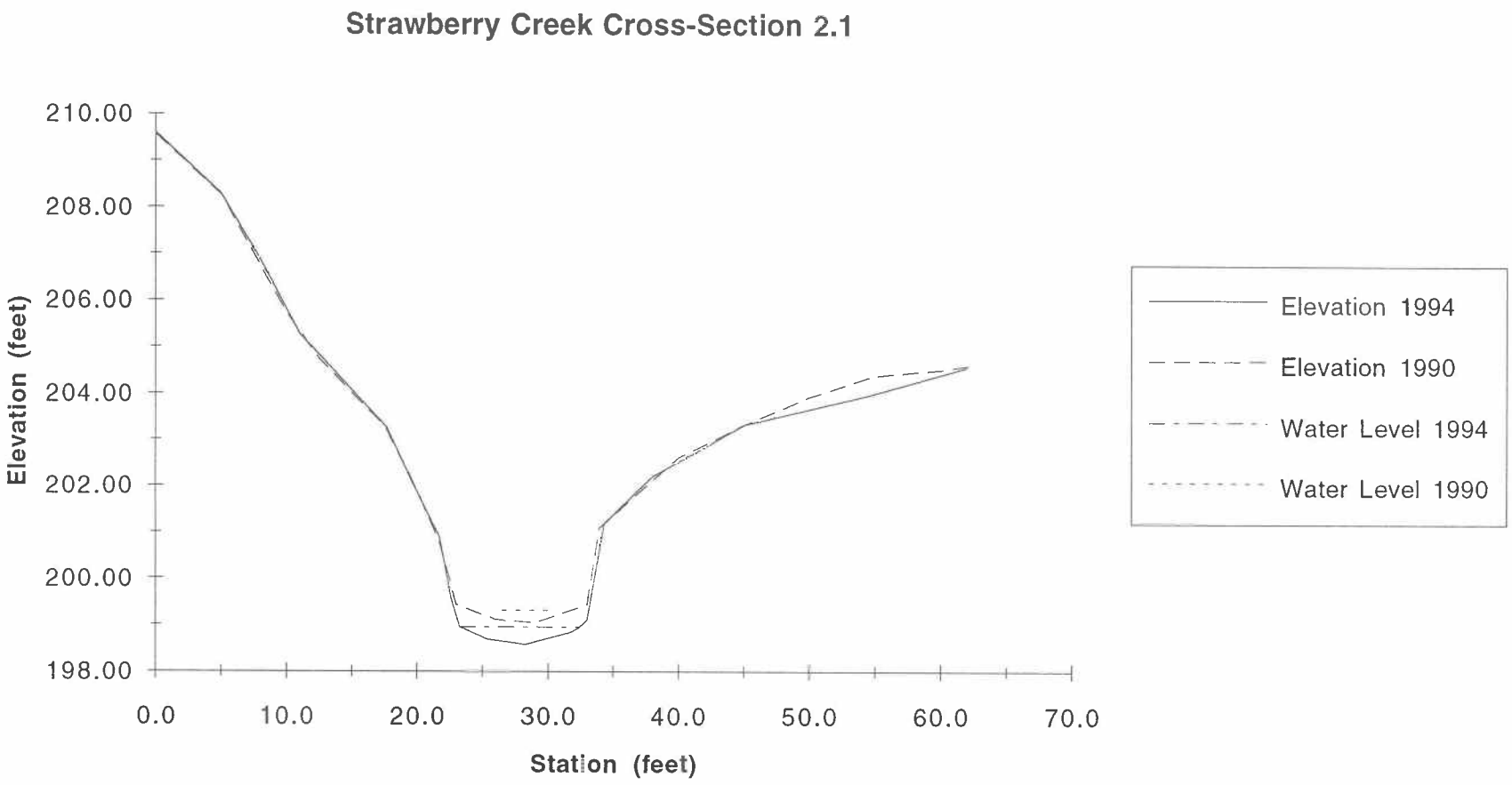
Figure 7.

Attachment V

Cross-Section Data and Graphs

Strawberry Creek Cross Section 2.1

2/25/94 STATION	BS	FS	HI	ELEV	NOTES	1/16/90 STATION	ELEV
BM	9.44		210.56	201.12	tunnel to oxford culvert		
0.0		0.97	210.56	209.59	LBP	0.0	209.55
5.0		2.29	210.56	208.27		5.0	208.26
8.0		3.69	210.56	206.87		10.0	205.72
11.0		5.29	210.56	205.27		12.5	204.73
15.0		6.49	210.56	204.07		17.5	203.30
17.7		7.31	210.56	203.25		20.0	201.89
19.7		8.54	210.56	202.02		21.7	200.75
21.7		9.68	210.56	200.88	Left Bank	23.0	199.44
22.6		10.96	210.56	199.60		26.0	199.12
23.3		11.60	210.56	198.96	Water level	26.5	199.32
25.4		11.87	210.56	198.69		29.0	199.04
28.3		11.99	210.56	198.57	Thalweg	33.0	199.44
31.7		11.73	210.56	198.83		33.9	201.07
32.4		11.61	210.56	198.95		40.0	202.62
33.0		11.45	210.56	199.11	Base of Rt Bank	45.0	203.31
34.3		9.39	210.56	201.17	Rt Bank	50.0	203.93
38.0		8.37	210.56	202.19		55.0	204.40
45.0		7.25	210.56	203.31		60.0	204.52
54.3		6.60	210.56	203.96		62.2	204.61
62.0		5.99	210.56	204.57	RBP		
TP-1		2.27	210.56	208.29			
TP-1	7.29		215.58	208.29			
BM 4.0		5.54	215.58	210.04	SE corner of spillway		


Strawberry Creek Cross-Section 2.1
210.00
208.00
206.00
204.00
202.00
200.00
198.00
Elevation (feet)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

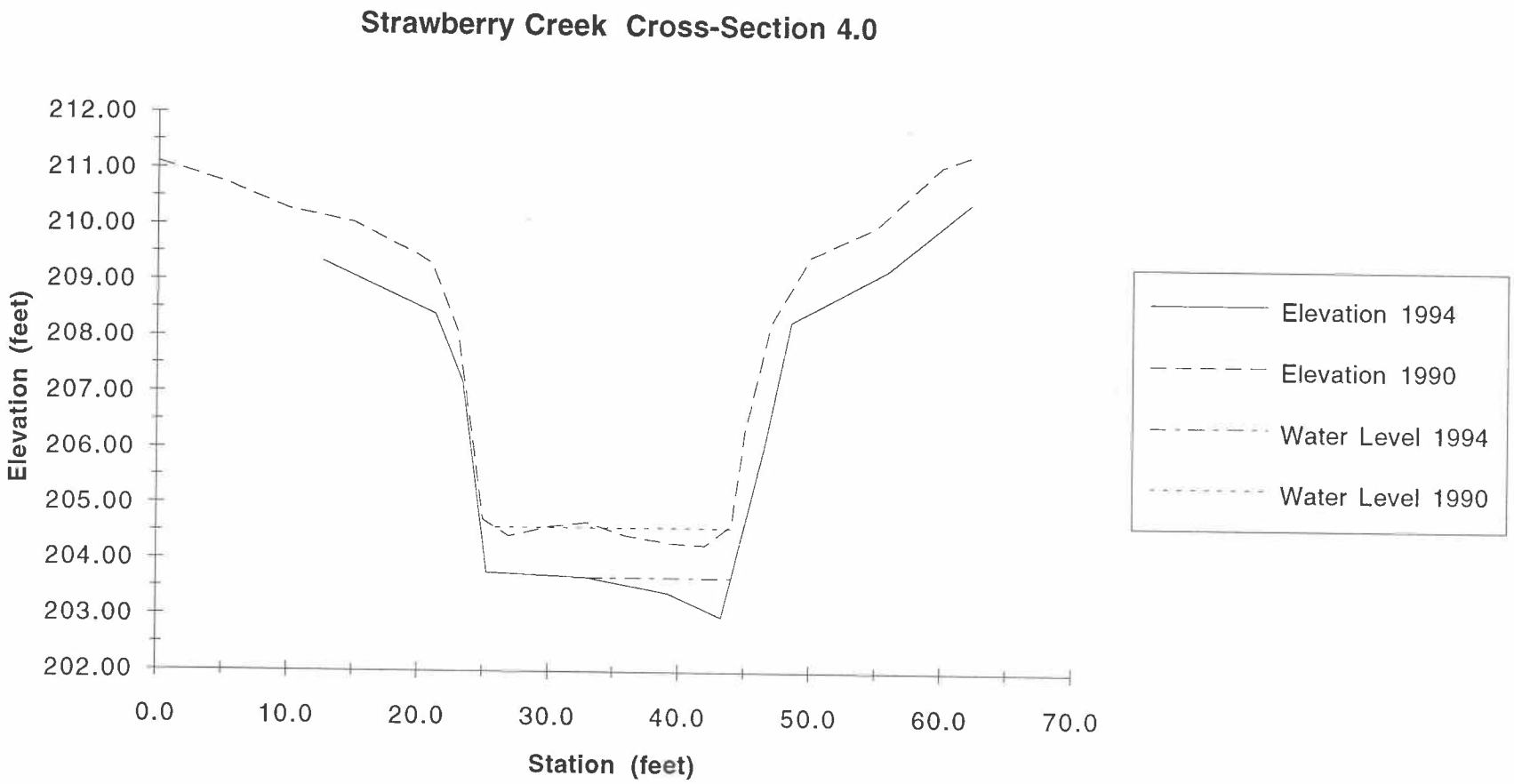
Strawberry Creek Cross-Section 4.0

2/25/94

station	BS	FS	HI	ELEV	notes
BM	5.57		215.61	210.04	SE corner of spillway
12.6		6.26	215.61	209.35	
21.3		7.19	215.61	208.42	Top of L bank
23.4		8.41	215.61	207.20	
25.3		11.84	215.61	203.77	Toe of L bank
33.0		11.92	215.61	203.69	water level
39.2		12.20	215.61	203.41	
43.3		12.63	215.61	202.98	Thalweg, Toe of R ban
46.5		9.57	215.61	206.04	
48.5		7.30	215.61	208.31	Top of R bank
55.9		6.37	215.61	209.24	
62.2		5.15	215.61	210.46	1.4' from RBP
0.0					

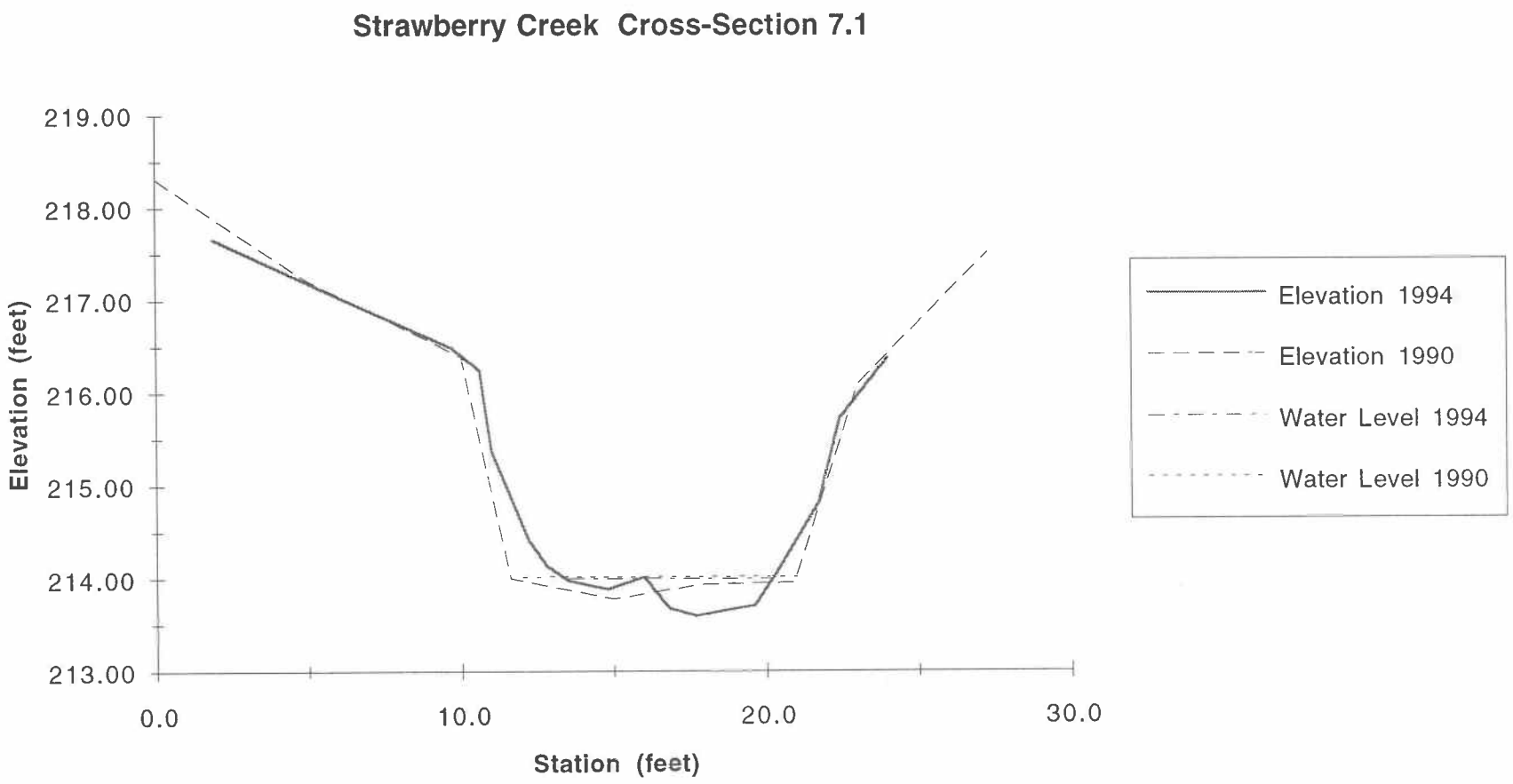
1/16/90

STATION	ELEV
0.0	211.11
5.0	210.76
10.0	210.28
15.0	210.05
20.0	209.46
21.0	209.31
23.0	208.11
25.0	204.74
27.0	204.43
30.0	204.60
33.0	206.68
36.0	204.45
39.0	204.33
42.0	204.27
44.0	204.64
45.0	206.33
47.0	208.30
50.0	209.49
55.0	210.03
60.0	211.13
62.6	211.35


Strawberry Creek Cross-Section 4.0
212.00
211.00
210.00
209.00
208.00
207.00
206.00
205.00
204.00
203.00
202.00
Elevation (feet)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

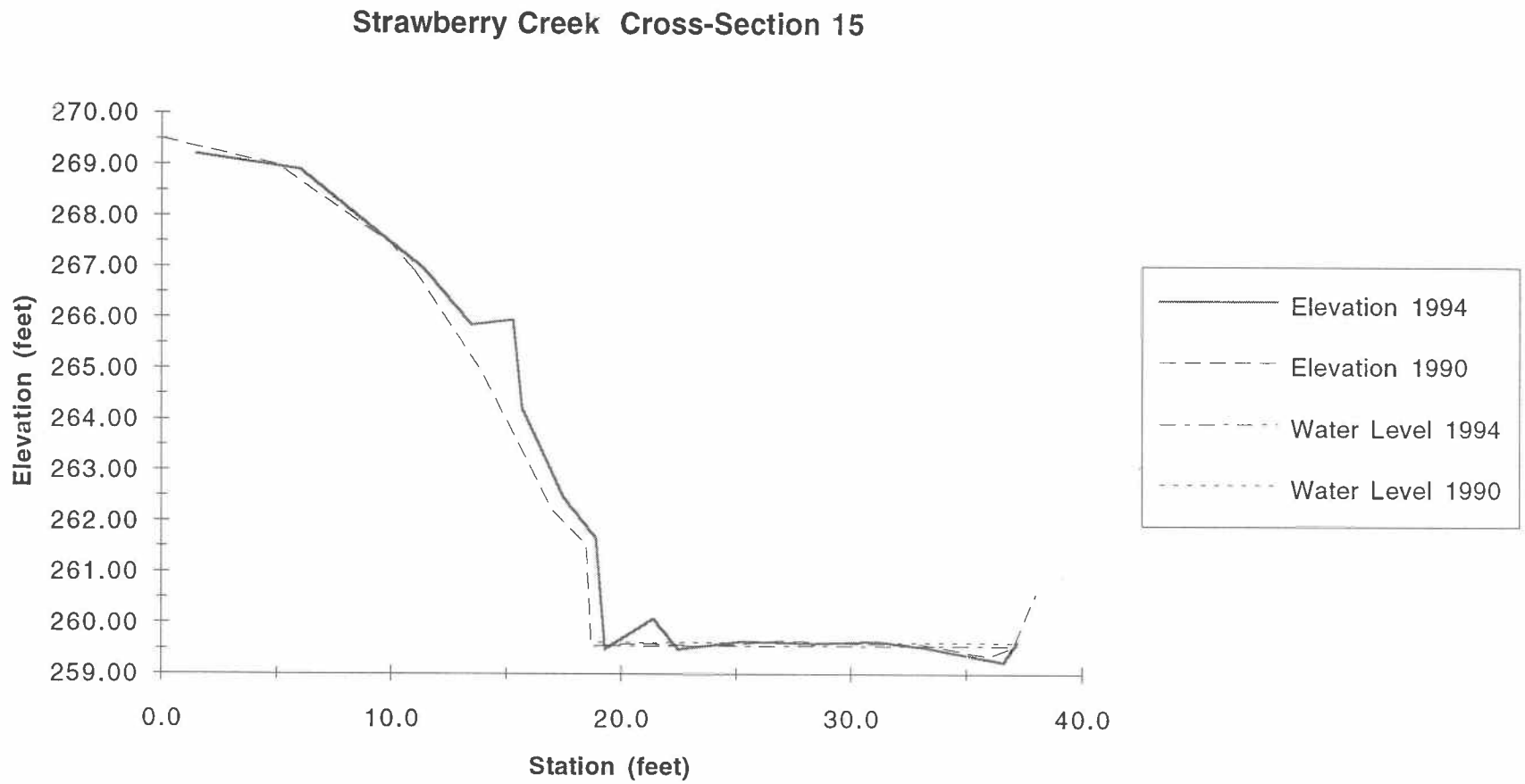
Strawberry Creek Cross-Section 7.1

3/18/94 station	BS	FS	HI	ELEV	notes	1/16/90 STATION	ELEV
BM	2.79		221.77	218.98	OLBP		
0.0		2.79	221.77	218.98		0.0	218.98
1.9		4.11	221.77	217.66		0.0	218.31
6.0		4.74	221.77	217.03		5.0	217.21
9.7		5.29	221.77	216.48		10.0	216.39
10.6		5.53	221.77	216.24	top of LB	11.5	214.16
11.0		6.40	221.77	215.37		11.6	214.00
12.2		7.36	221.77	214.41		15.0	213.78
12.8		7.64	221.77	214.13		18.0	213.94
13.5		7.80	221.77	213.97	water level LB	20.9	213.96
14.8		7.89	221.77	213.88		23.0	216.09
16.0		7.76	221.77	214.01	on top of cobble	26.0	217.09
16.8		8.10	221.77	213.67		27.3	218.00
17.7		8.18	221.77	213.59	thalweg	27.5	217.52
18.7		8.12	221.77	213.65			
19.6		8.07	221.77	213.70			
20.3		7.73	221.77	214.04	water level RB		
21.7		6.96	221.77	214.81			
22.4		6.05	221.77	215.72			
24.0		5.40	221.77	216.37			
27.1		4.31	221.77	217.46	ORBP		


Strawberry Creek Cross-Section 7.1
219.00
218.00
217.00
216.00
215.00
214.00
213.00
Elevation (feet)
0.0
10.0
20.0
30.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 15

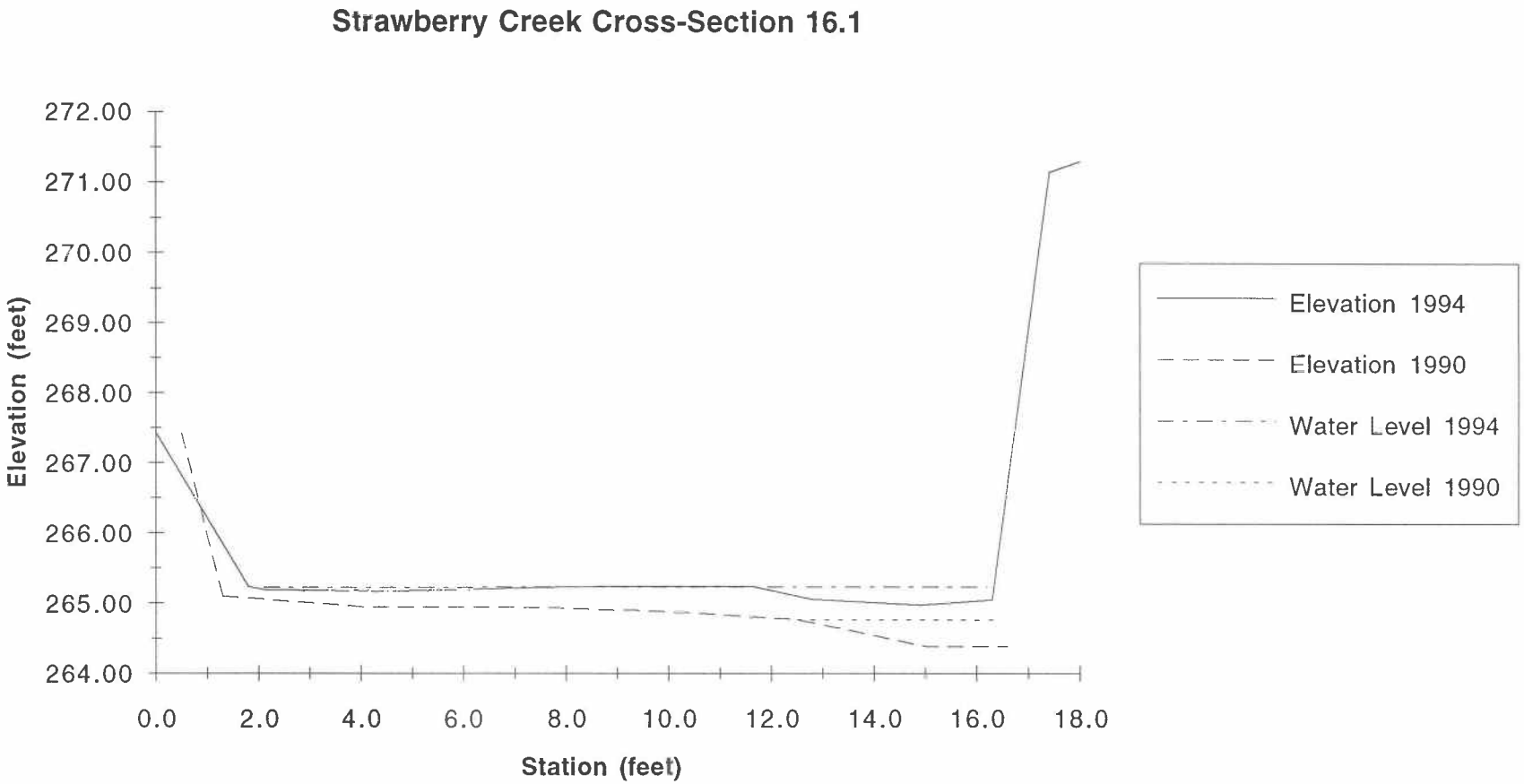
3/9/94 station	BS	FS	HI	ELEV	notes	1/16/90 STATION	ELEV
BM	8.28		271.16	262.88	ORBP		
1.5		1.96	271.16	269.20		0.0	269.98
6.0		2.26	271.16	268.90		0.0	269.50
11.4		4.20	271.16	266.96		5.0	268.99
13.5		5.32	271.16	265.84		10.0	267.45
15.3		5.21	271.16	265.95	NE corner of top step	11.0	266.93
15.7		6.95	271.16	264.21		14.0	264.89
17.5		8.71	271.16	262.45		17.0	262.22
18.9		9.51	271.16	261.65	top of LB wall	18.5	261.54
19.3		11.69	271.16	259.47	water level LB	18.7	259.54
21.4		11.09	271.16	260.07	on top of cobble	21.0	259.60
22.5		11.69	271.16	259.47		24.0	259.54
25.3		11.53	271.16	259.63	gravel bar	27.0	259.65
28.4		11.58	271.16	259.58		30.0	259.6
31.1		11.53	271.16	259.63		33.0	259.58
33.3		11.65	271.16	259.51		36.0	259.35
36.6		11.93	271.16	259.23	thalweg	37.0	259.53
37.2		11.54	271.16	259.62	water level RB	38.0	260.58
38.4		8.28	271.16	262.88	ORBP	38.2	262.88


Strawberry Creek Cross-Section 15
270.00
269.00
268.00
267.00
266.00
265.00
264.00
263.00
262.00
261.00
260.00
259.00
Elevation (feet)
0.0
10.0
20.0
30.0
40.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 16.1

Date: 3/9/94

station	BS	FS	HI	ELEV	notes	1/16/90 STATION	ELEV
BM	3.32		274.63	271.31	grd @ RBP		
0.0		7.10	274.63	267.53	grd@LBP	0.0	267.54
1.8		7.20	274.63	267.43	R edge of top of LB wall	0.5	267.43
2.1		9.40	274.63	265.23	water level, base of LB wall	1.3	265.10
4.3		9.44	274.63	265.19		4.0	264.95
8.2		9.47	274.63	265.16		7.0	264.95
11.6		9.40	274.63	265.23		10.0	264.88
12.8		9.39	274.63	265.24		12.5	264.76
14.9		9.58	274.63	265.05		15.0	264.38
16.3		9.66	274.63	264.97	thalweg	16.6	264.39
17.4		9.59	274.63	265.04	base of RB wall	17.7	
18.0		3.48	274.63	271.15	top of RB wall		
18.7		3.32	274.63	271.31	grd@RBP		


Strawberry Creek Cross-Section 16.1
272.00
271.00
270.00
269.00
268.00
267.00
266.00
265.00
264.00
Elevation (feet)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

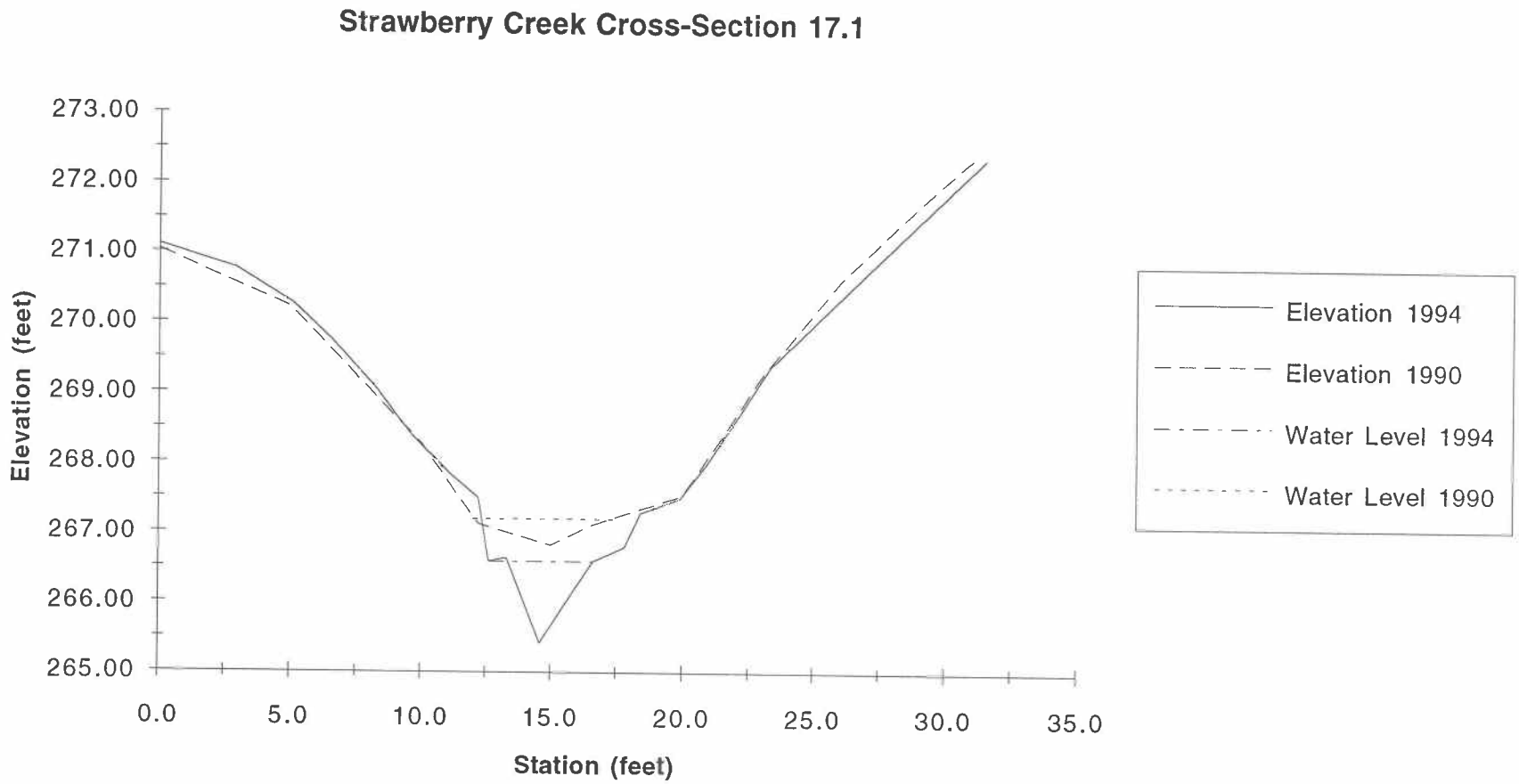
Strawberry Creek Cross-Section 17.1

Date: 3/9/94

station	BS	FS	HI	ELEV	notes
BM	3.00		275.38	272.38	grd@RBP
0.0		4.28	275.38	271.10	grd@LBP
2.9		4.61	275.38	270.77	
5.1		5.11	275.38	270.27	
6.6		5.65	275.38	269.73	
8.3		6.34	275.38	269.04	
9.6		6.98	275.38	268.40	
11.1		7.54	275.38	267.84	
12.2		7.89	275.38	267.49	LB
12.6		8.80	275.38	266.58	water level at LB
13.3		8.75	275.38	266.63	on top of cobble
14.6		9.96	275.38	265.42	thalweg
16.6		8.80	275.38	266.58	water level at RB
17.8		8.59	275.38	266.79	
18.4		8.10	275.38	267.28	RB "wall"
19.9		7.89	275.38	267.49	
20.9		7.39	275.38	267.99	
23.3		6.00	275.38	269.38	
31.5		3.00	275.38	272.38	grd@RBP

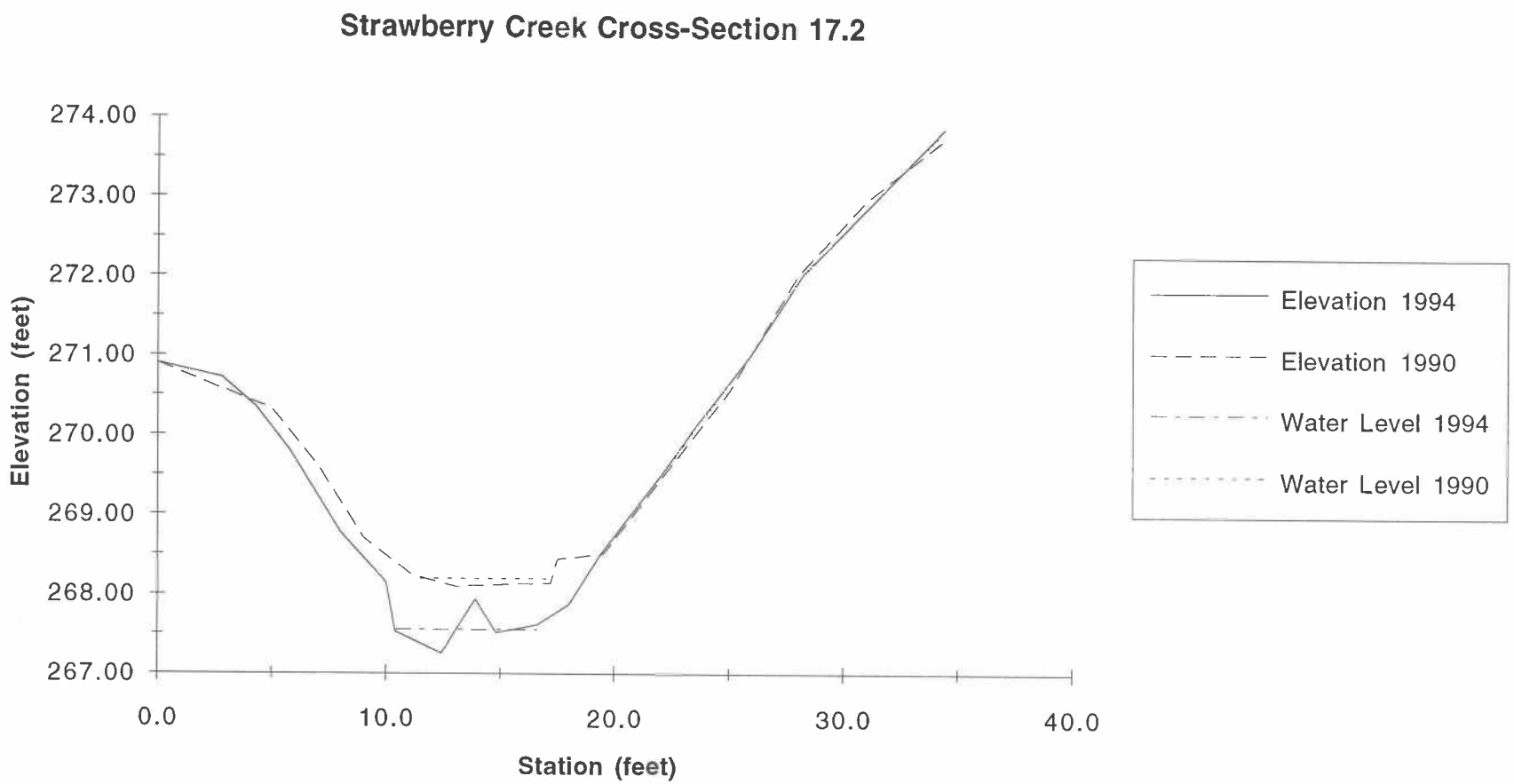
1/16/90

STATION	ELEV
0.0	271.40
0.0	271.03
5.0	270.22
10.0	268.27
12.2	267.12
15.0	266.82
16.5	267.10
20.0	267.54
23.0	269.26
26.0	270.63
29.0	271.72
31.0	272.38
31.15	273.29


Strawberry Creek Cross-Section 17.1
273.00
272.00
271.00
270.00
269.00
268.00
267.00
266.00
265.00
Elevation (feet)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 17.2

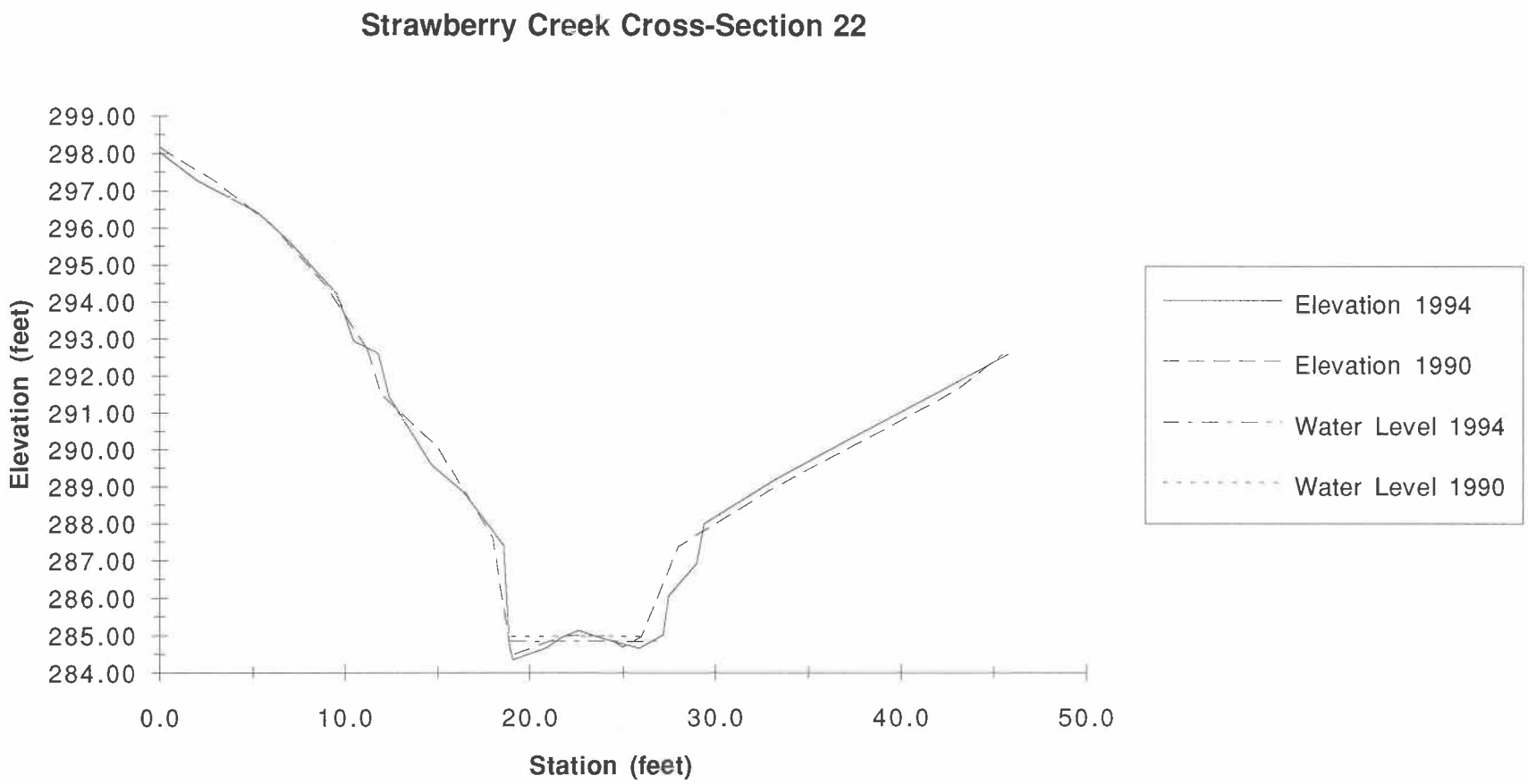
3/9/94 STATION	BS	FS	HI	ELEV	notes	1/16/90 STATION	ELEV
BM	3.94		274.84	270.90	grd@LBP		
0.0		3.94	274.84	270.90	grd@LBP	0.0	271.59
2.8		4.12	274.84	270.72		0.0	270.90
4.3		4.49	274.84	270.35		5.0	270.32
5.8		5.05	274.84	269.79		7.0	269.61
8.0		6.07	274.84	268.77		9.0	268.71
10.0		6.70	274.84	268.14	top of LB	11.1	268.24
10.4		7.32	274.84	267.52	water level at LB	13.0	268.10
12.4		7.59	274.84	267.25	thalweg	14.0	268.11
13.9		6.91	274.84	267.93	on top of cobble	15.0	268.12
14.8		7.33	274.84	267.51		16.0	268.13
16.6		7.23	274.84	267.61	water level at RB	17.2	268.14
18.0		6.97	274.84	267.87		17.5	268.44
19.4		6.33	274.84	268.51		19.5	268.51
22.1		5.32	274.84	269.52		22.0	269.44
23.7		4.67	274.84	270.17		25.0	270.56
25.9		3.84	274.84	271.00		28.0	272.03
28.2		2.81	274.84	272.03		31.0	272.98
34.4		0.97	274.84	273.87	grd@RBP	34.5	273.76
						34.5	274.13


Strawberry Creek Cross-Section 17.2
274.00
273.00
272.00
271.00
270.00
269.00
268.00
267.00
Elevation (feet)
0.0
10.0
20.0
30.0
40.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 22

Date: 3/19/94

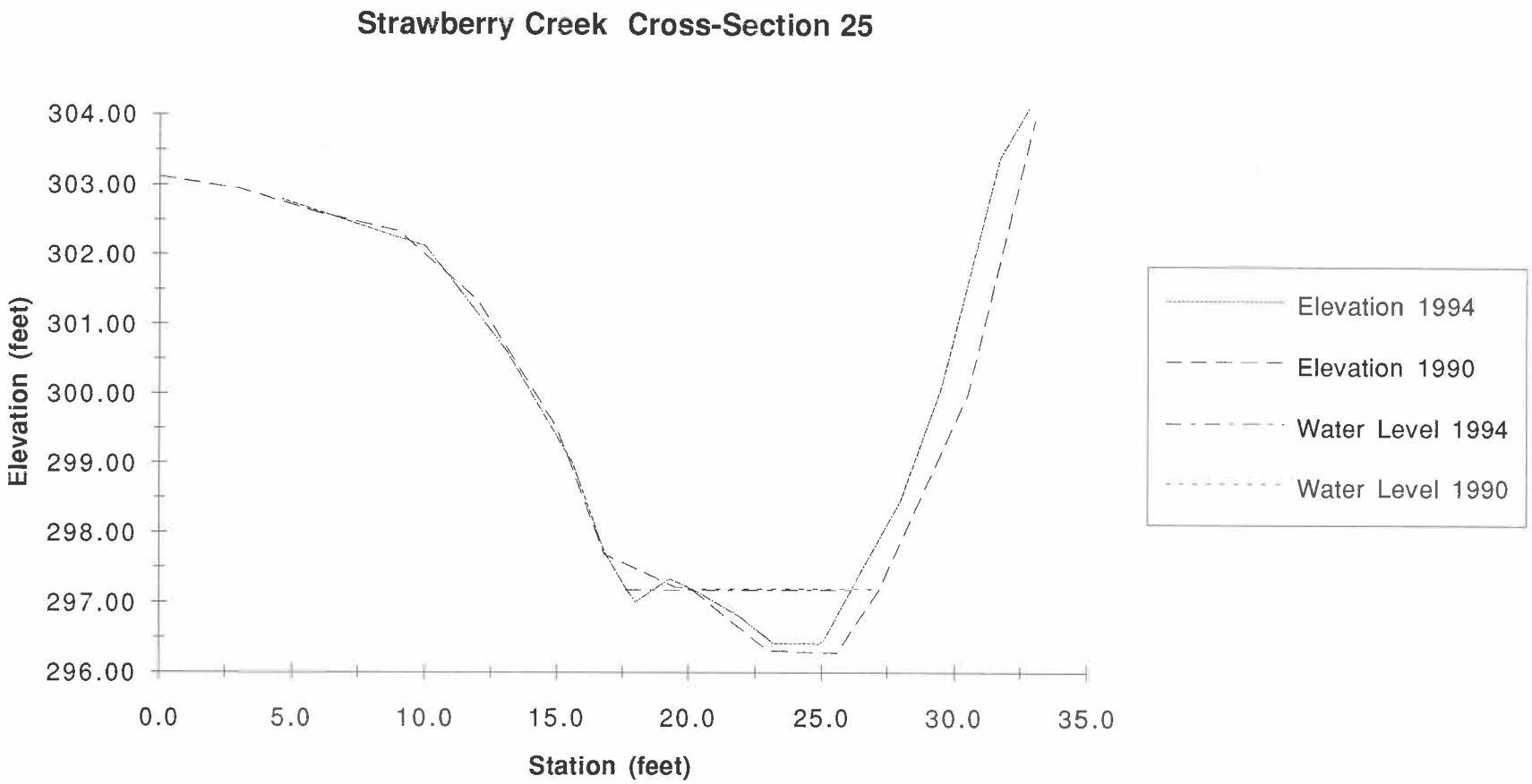
STATION	BS	FS	HI	ELEV	notes	1/19/90 STATION	ELEV
BM	4.81		302.39	297.58	NE corner of elect. box		
0.0		4.34	302.39	298.05		0.0	298.17
2.0		5.11	302.39	297.28		3.0	297.24
5.2		5.96	302.39	296.43		6.0	296.07
7.0		6.78	302.39	295.61		9.0	294.43
9.6		8.20	302.39	294.19	ledge	11.2	292.72
10.5		9.46	302.39	292.93		12.0	291.49
11.8		9.79	302.39	292.60		15.0	290.08
12.4		10.97	302.39	291.42		18.0	287.62
14.7		12.83	302.39	289.56		19.0	284.47
16.4		13.55	302.39	288.84		22.0	285.00
18.6		14.99	302.39	287.40	top of LB wall	24.0	284.99
18.9		17.70	302.39	284.69	water level at LB	25.0	284.67
19.1		18.05	302.39	284.34	left channel thalweg	26.0	284.97
20.9		17.73	302.39	284.66	water level of LB of gravel bar	28.0	287.37
21.7		17.46	302.39	284.93		33.0	288.92
22.6		17.27	302.39	285.12	on top of gravel bar	38.0	290.24
24.0		17.50	302.39	284.89	RB of gravel bar	43.0	291.63
25.9		17.74	302.39	284.65	right channel thalweg	45.5	292.60
27.2		17.39	302.39	285.00	water level at RB		
27.5		16.35	302.39	286.04			
29.0		15.49	302.39	286.90			
29.4		14.41	302.39	287.98			
33.1		13.23	302.39	289.16			
45.8		9.79	302.39	292.60	grd@RBP		


Strawberry Creek Cross-Section 22
Elevation (feet)
299.00
298.00
297.00
296.00
295.00
294.00
293.00
292.00
291.00
290.00
289.00
288.00
287.00
286.00
285.00
284.00
0.0
10.0
20.0
30.0
40.0
50.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 25

Date: 3/9/94

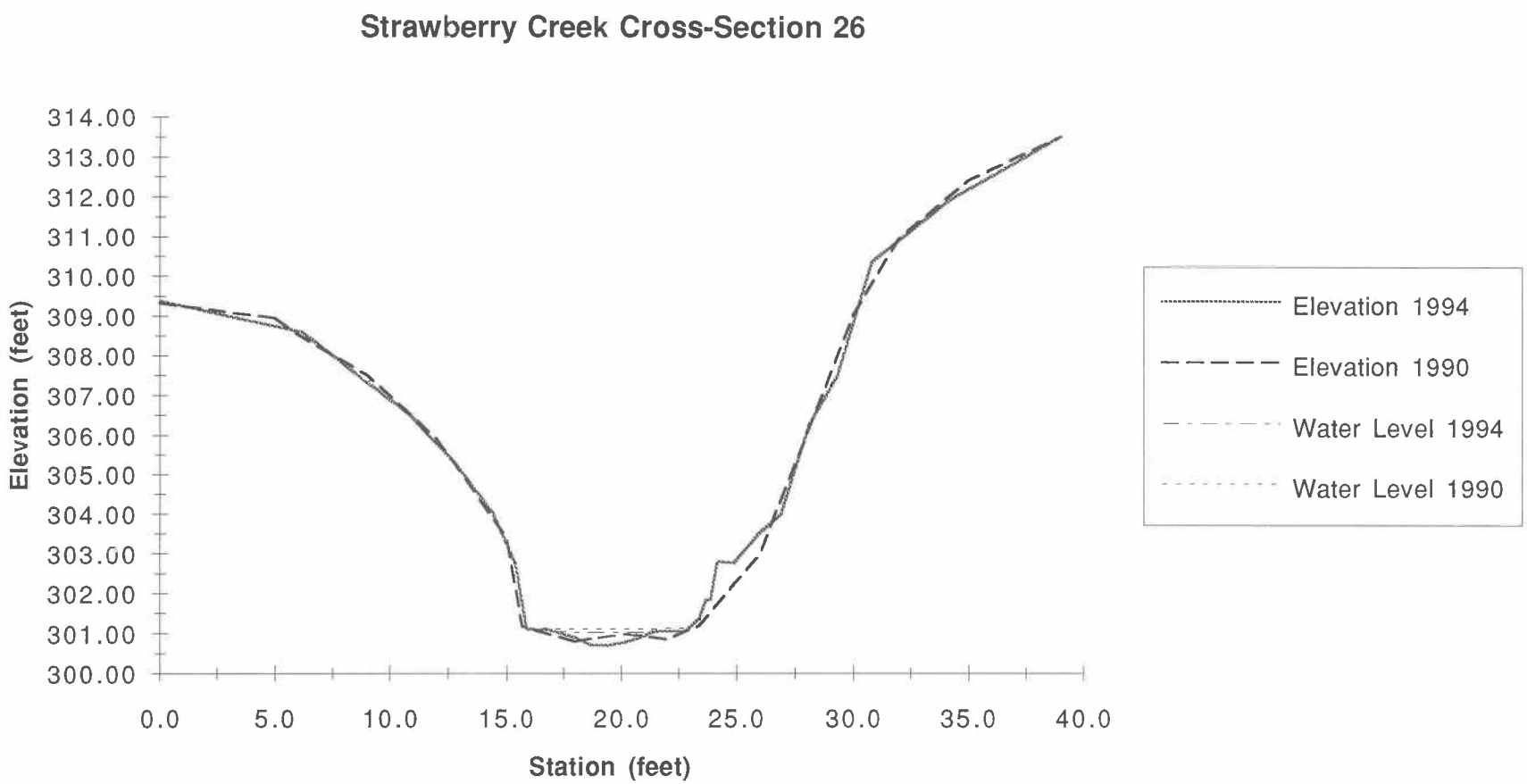
STATION	BS	FS	HI	ELEV	notes	1/19/90 STATION	ELEV	
BM	4.59		308.00	303.41	OLBP			
0.0		4.59	308.00	303.41	OLBP	0.0	303.11	gnd@LBP
4.6		5.20	308.00	302.80		3.0	302.95	
10.0		5.87	308.00	302.13		6.0	302.60	
13.1		7.39	308.00	300.61		9.0	302.34	
15.6		8.99	308.00	299.01		12.0	301.33	
16.8		10.27	308.00	297.73		15.0	299.51	
17.6		10.79	308.00	297.21	water level at LB	16.8	297.69	
18.0		10.99	308.00	297.01	left side of sand bar	19.5	297.22	
19.3		10.66	308.00	297.34	top of sand bar	20.1	297.20	WSE
20.3		10.84	308.00	297.16	water level RB of sand bar	23.0	296.31	
21.9		11.19	308.00	296.81		25.6	296.28	
23.2		11.58	308.00	296.42		27.2	297.18	
24.9		11.59	308.00	296.41	thalweg	30.5	300.01	
25.1		11.53	308.00	296.47		33.0	303.94	
26.1		10.85	308.00	297.15	water level at RB			
28.0		9.53	308.00	298.47				
29.5		7.88	308.00	300.12				
31.7		4.60	308.00	303.40				
34.3		2.88	308.00	305.12				
35.6		1.85	308.00	306.15				
37.2					gnd@RBP			


Strawberry Creek Cross-Section 25
304.00
303.00
302.00
301.00
300.00
299.00
298.00
297.00
296.00
Elevation (feet)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 26

Date: 3/25/94

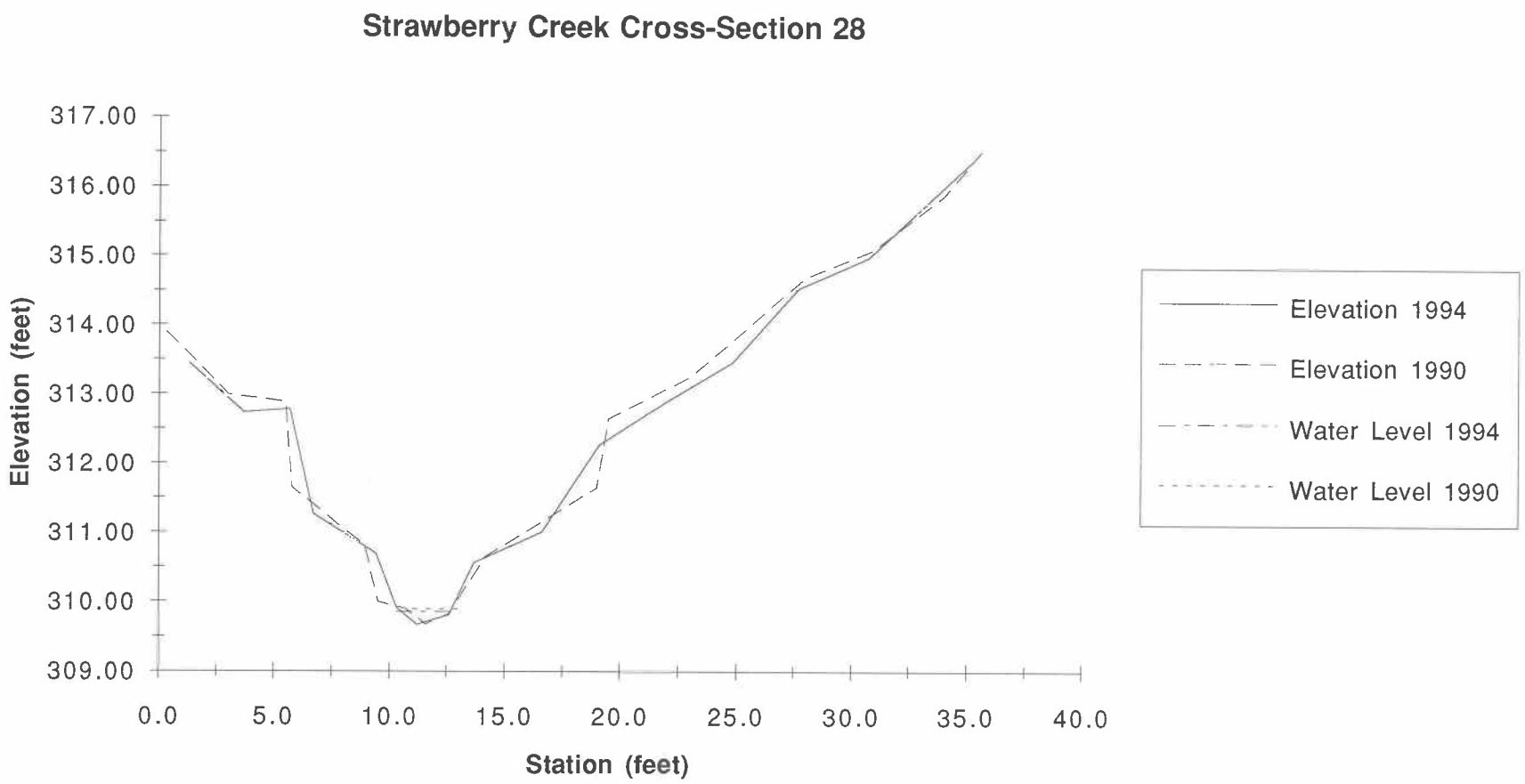
STATION	BS	FS	HI	ELEV	notes	1/19/90 STATION	ELEV	
BM	4.80		314.39	309.59	OLBP			
0.0		5.02	314.39	309.37	grd @LBP	0.0	309.31	grd@LBP
6.2		5.81	314.39	308.58	CU edge of vault (?)	5.0	308.95	
10.9		7.91	314.39	306.48		6.0	308.53	
12.8		9.12	314.39	305.27		9.0	307.52	
14.4		10.37	314.39	304.02		12.0	305.92	
15.4		11.67	314.39	302.72		15.0	303.40	
15.9		13.30	314.39	301.09		15.7	301.16	
16.9		13.32	314.39	301.07	water level at LB	18.0	300.79	
18.0		13.50	314.39	300.89		20.0	300.99	
18.7		13.69	314.39	300.70		22.0	300.85	
19.4		13.71	314.39	300.68	thalweg	23.4	301.19	
20.1		13.63	314.39	300.76		26.0	302.97	
20.7		13.53	314.39	300.86		28.0	306.00	
21.4		13.36	314.39	301.03	water level at RB	30.0	309.05	
22.8		13.34	314.39	301.05		32.0	310.97	
23.4		13.03	314.39	301.36		35.0	312.45	
23.7		12.55	314.39	301.84	pillar footing	38.9	313.51	grd@RBP
23.9		12.56	314.39	301.83	pillar footing			
24.2		11.59	314.39	302.80	2nd step pillar footing			
24.9		11.64	314.39	302.75	2nd step upper edge	16.0	301.13	WSE
26.0		10.88	314.39	303.51				
26.9		10.40	314.39	303.99				
28.1		8.14	314.39	306.25				
29.3		6.90	314.39	307.49				
30.8		4.02	314.39	310.37				
34.2		2.45	314.39	311.94				
39.0		0.85	314.39	313.54	grd@RBP			
39.0		0.52	314.39	313.87	ORBP			


Strawberry Creek Cross-Section 26
314.00
313.00
312.00
311.00
310.00
309.00
308.00
307.00
306.00
305.00
304.00
303.00
302.00
301.00
300.00
Elevation (feet)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 28

Date: 3/9/94

STATION	BS	FS	HI	ELEV	notes	1/19/90 STATION	ELEV	NOTES
BM	3.43		320.21	316.78	ORBP			
1.3		6.77	320.21	313.44		0.3	313.90	grd@LBP
3.7		7.48	320.21	312.73		3.0	312.99	
5.7		7.43	320.21	312.78	rock ledge	5.5	312.89	
6.7		8.95	320.21	311.26		5.8	311.65	
9.4		9.52	320.21	310.69		8.9	310.81	
10.3		10.30	320.21	309.91	water level at LB	9.5	310.00	
11.2		10.55	320.21	309.66	thalweg	10.7	309.89	WSE
12.6		10.41	320.21	309.80	water level at RB	11.6	309.67	
13.7		9.65	320.21	310.56		12.6	309.82	
16.6		9.21	320.21	311.00		14.2	310.65	
19.1		7.95	320.21	312.26		19.0	311.65	
22.0		7.31	320.21	312.90	base of forked tree	19.5	312.65	
24.8		6.75	320.21	313.46		23.0	313.26	
27.7		5.67	320.21	314.54		28.0	314.70	
30.7		5.22	320.21	314.99		31.0	315.12	
35.2		3.83	320.21	316.38	grd. 4" below RBP	34.0	315.89	
						35.6	316.53	grd@RBP


Strawberry Creek Cross-Section 28
317.00
316.00
315.00
314.00
313.00
312.00
311.00
310.00
309.00
Elevation (feet)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

Strawberry Creek Cross-Section 31

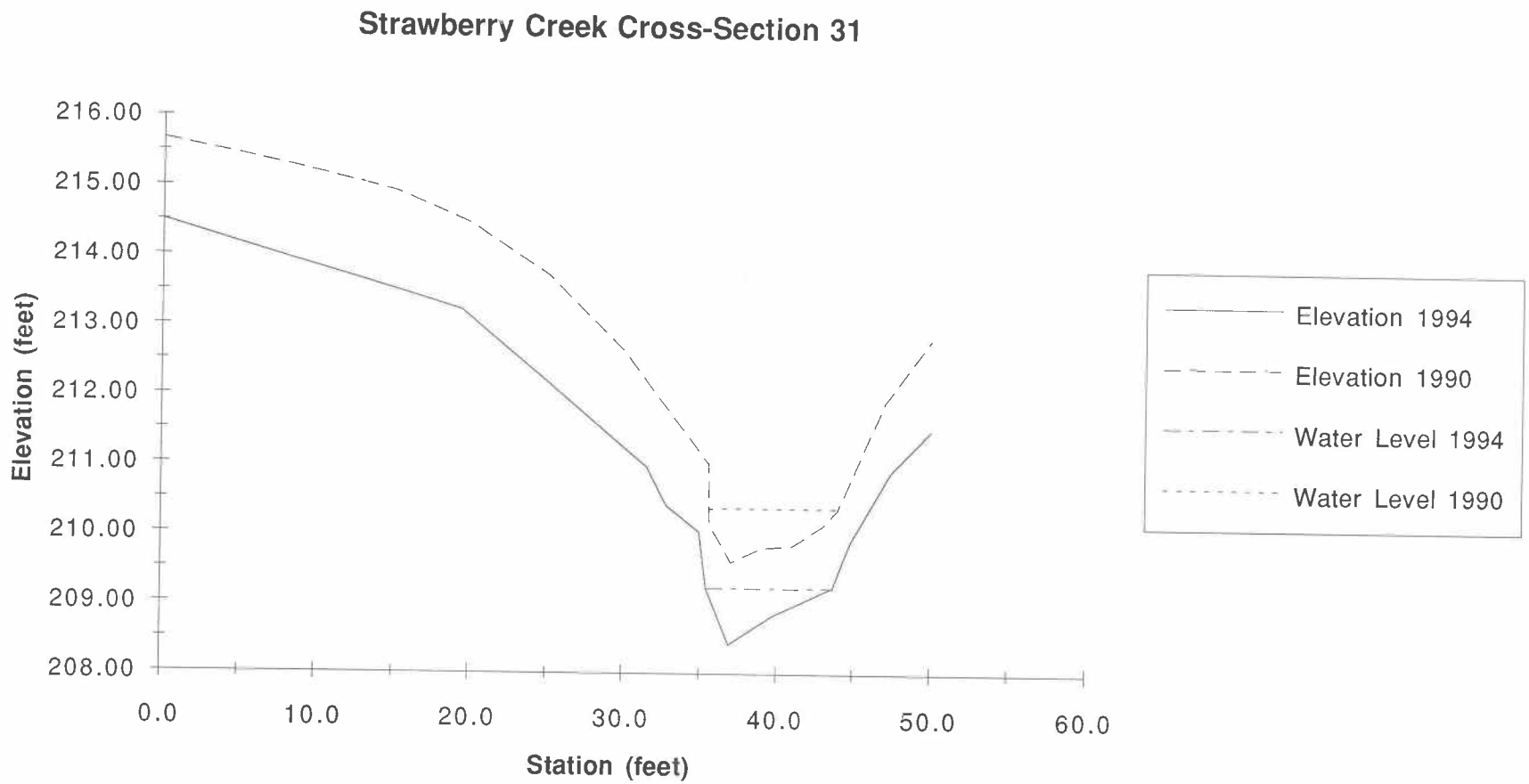
Date: 2/25/94

STATION	BS	FS	HI	ELEV	NOTES
BM	5.57		215.28	209.71	*
0.0		0.78	215.28	214.50	
19.4		2.05	215.28	213.23	
31.5		4.31	215.28	210.97	
32.8		4.87	215.28	210.41	
34.9		5.25	215.28	210.03	LB
35.4		6.06	215.28	209.22	Water Level at LB
36.9		6.86	215.28	208.42	
39.7		6.45	215.28	208.83	thalweg
43.7		6.04	215.28	209.24	water level at RB
44.8		5.39	215.28	209.89	
47.4		4.38	215.28	210.90	
50.0		3.77	215.28	211.51	

* SE corner of tallest E. sq. block of spillway

1/19/90

STATION	ELEV	NOTES
0.0	215.66	grd@LBP
5.0	215.45	
10.0	215.21	
15.0	214.94	
20.0	214.48	
25.0	213.74	
30.0	212.66	
35.5	211.03	
35.6	210.14	
37.0	209.60	
39.0	209.80	
41.0	209.84	
43.0	210.12	
44.0	210.36	
47.0	211.92	
50.0	212.83	grd@RBP
39.0	210.38	WSE


Strawberry Creek Cross-Section 31
216.00
215.00
214.00
213.00
212.00
211.00
210.00
209.00
208.00
Elevation (feet)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

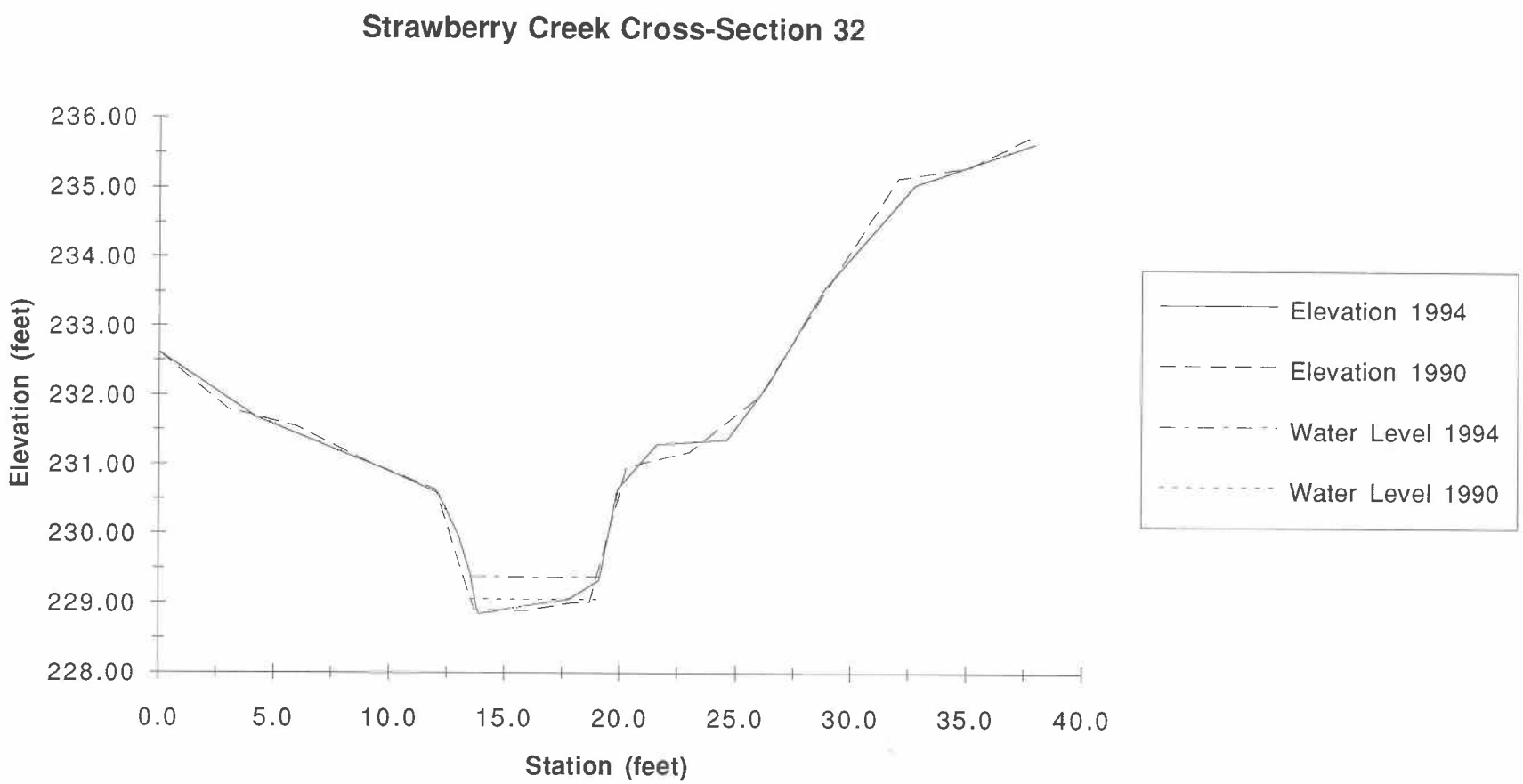
Strawberry Creek Cross-Section 32

Date: 3/18/93

STATION	BS	FS	HI	ELEV	NOTES
BM	5.52		238.14	232.62	grd@LBP
0.0		5.52	238.14	232.62	
4.2		6.46	238.14	231.68	
9.6		7.19	238.14	230.95	
12.1		7.56	238.14	230.58	
13.0		8.18	238.14	229.96	top of LB
13.5		8.69	238.14	229.45	water level at LB
13.8		9.20	238.14	228.94	
13.9		9.30	238.14	228.84	thalweg
14.5		9.27	238.14	228.87	
15.2		9.22	238.14	228.92	
17.8		9.09	238.14	229.05	
19.1		8.82	238.14	229.32	water level at RB
19.9		7.50	238.14	230.64	
21.6		6.84	238.14	231.30	
24.6		6.78	238.14	231.36	
26.4		5.96	238.14	232.18	
28.8		4.59	238.14	233.55	
32.7		3.08	238.14	235.06	
37.9		2.48	238.14	235.66	gnd @RBP

1/20/94

STATION	ELEV	NOTES
0.0	232.62	grd@LBP
3.0	231.80	
6.0	231.55	
9.0	231.04	
12.0	230.63	
13.7	228.90	
16.0	228.90	
18.0	229.01	
18.7	229.02	
20.3	230.97	
23.0	231.19	
26.0	232.00	
29.0	233.60	
32.0	235.16	
35.0	235.32	
37.9	235.79	grd@RBP
14.0	229.06	WSE


Strawberry Creek Cross-Section 32
236.00
235.00
234.00
233.00
232.00
231.00
230.00
229.00
228.00
Elevation (feet)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

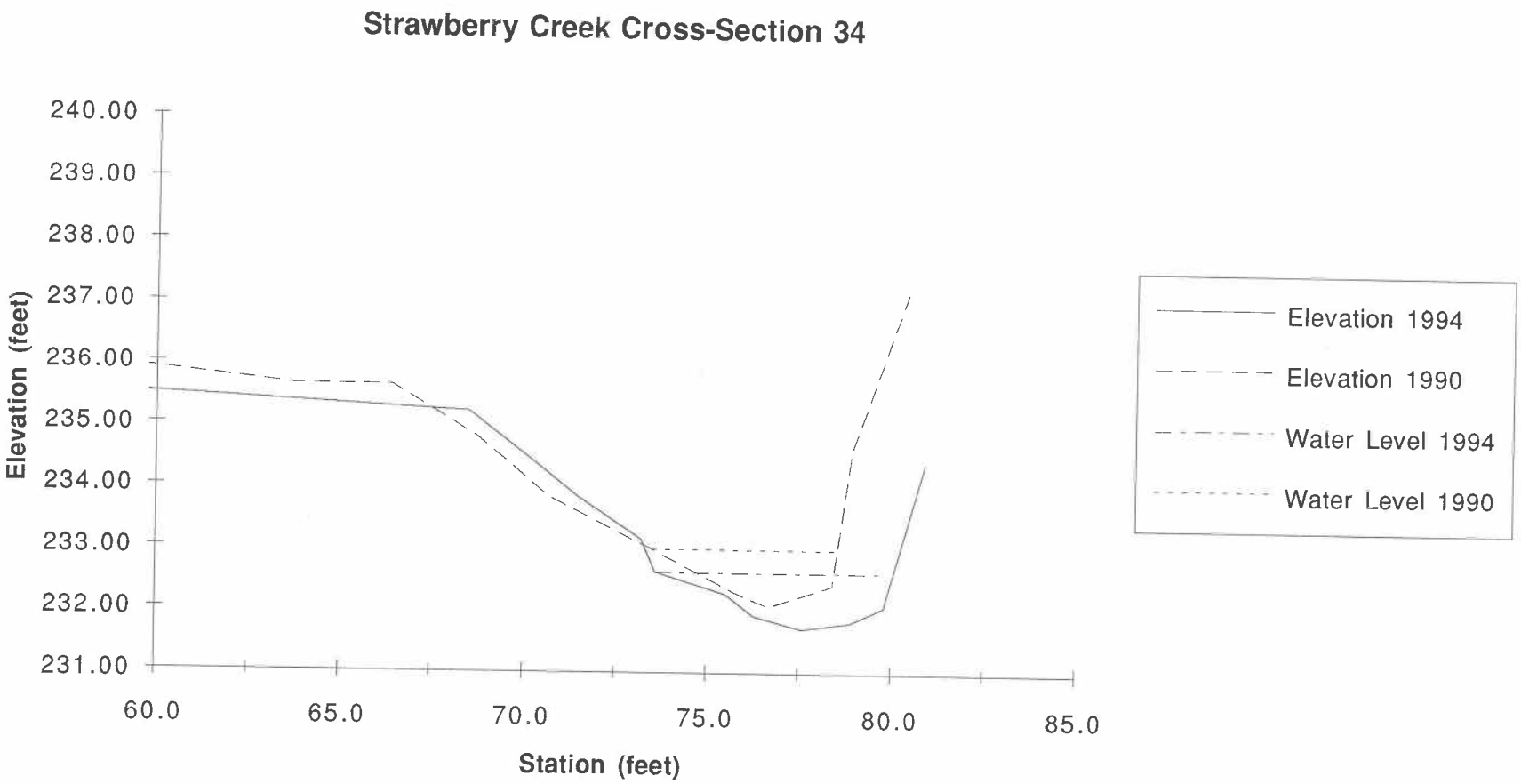
Strawberry Creek Cross-Section 34

Date: 3/25/93

STATION	BS	FS	HI	ELEV	NOTES
BM	4.05		242.49	238.44	OLBP
0.0		4.05	242.49	238.44	OLBP
0.0		4.55	242.49	237.94	grd@LBP
16.6		3.97	242.49	238.52	
35.2		3.58	242.49	238.91	
46.9		7.28	242.49	235.21	
51.5		6.74	242.49	235.75	
68.5		7.26	242.49	235.23	
71.5		8.66	242.49	233.83	
73.2		9.33	242.49	233.16	Top of LB
73.6		9.87	242.49	232.62	water level at LB
75.5		10.22	242.49	232.27	
76.3		10.57	242.49	231.92	
77.6		10.78	242.49	231.71	thalweg
78.9		10.67	242.49	231.82	
79.8		10.42	242.49	232.07	
80.9		8.09	242.49	234.40	top of RB
81.9		5.70	242.49	236.79	ORBP
79.8		9.87	242.49	232.62	WSE

1/20/94

STATION	ELEV	NOTES
0.0	238.44	OLBP
0.0	238.05	grd@LBP
10.7	238.65	
58.7	235.98	
63.7	235.65	
66.4	235.65	
68.7	234.83	
70.7	233.83	
73.5	233.00	WSE
75.7	232.32	
76.7	232.08	
78.4	232.42	
78.9	234.62	
80.4	237.19	grd@RBP
80.7	235.15	ORBP


Strawberry Creek Cross-Section 34
240.00
239.00
238.00
237.00
236.00
235.00
234.00
233.00
232.00
231.00
Elevation (feet)
60.0
65.0
70.0
75.0
80.0
85.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990

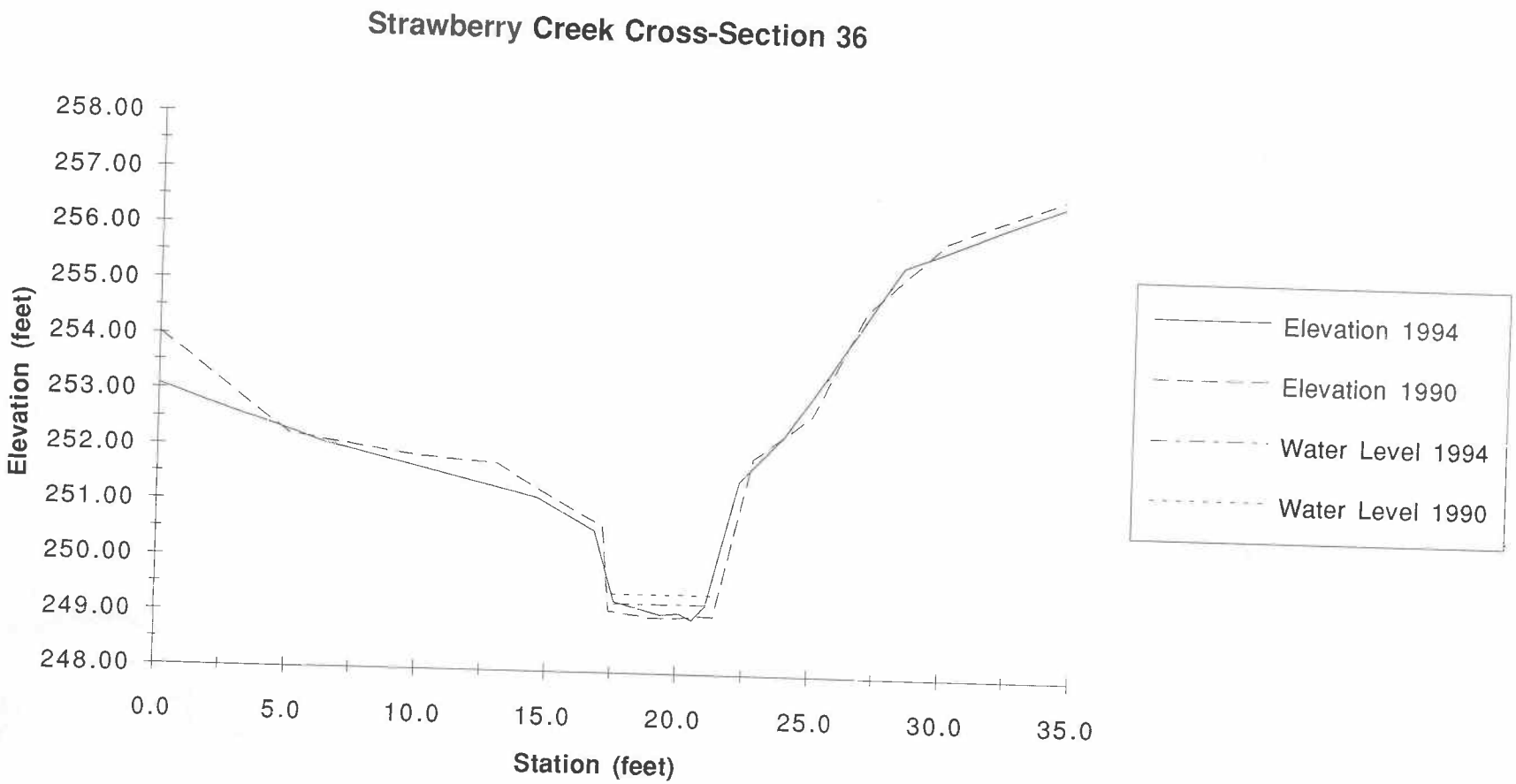
Strawberry Creek Cross-Section 36

Date: 3/25/93

STATION	BS	FS	HI	ELEV	NOTES
BM	3.79		260.75	256.96	ORBP
0.0		7.67	260.75	253.08	grd@LBP
6.3		8.66	260.75	252.09	
14.6		9.60	260.75	251.15	
16.8		10.19	260.75	250.56	top of LB
17.6		11.46	260.75	249.29	water level at LB
18.6		11.58	260.75	249.17	
19.4		11.68	260.75	249.07	
20.1		11.66	260.75	249.09	
20.6		11.77	260.75	248.98	thalweg
21.1		11.51	260.75	249.24	water level at RB
22.3		9.23	260.75	251.52	top of RB
23.9		8.42	260.75	252.33	
25.6		7.27	260.75	253.48	
28.4		5.29	260.75	255.46	
34.5		4.16	260.75	256.59	grd@RBP
34.5		3.79	260.75	256.96	ORBP

1/20/94

STATION	ELEV	NOTES
0.0	255.67	OLBP
0.0	254.02	grd@LBP
5.0	252.23	
10.0	251.86	
13.0	251.76	
14.7	251.28	
17.1	250.64	
17.4	249.12	
19.0	249.01	
20.0	249.01	
21.0	249.05	
21.4	249.04	
22.8	251.93	
25.0	252.74	
27.0	254.62	
30.0	255.93	
34.5	256.73	grd@LBP
34.5	256.96	OLBP
17.4	249.44	WSE


Strawberry Creek Cross-Section 36
258.00
257.00
256.00
255.00
254.00
253.00
252.00
251.00
250.00
249.00
248.00
Elevation (feet)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
Station (feet)
Elevation 1994
Elevation 1990
Water Level 1994
Water Level 1990