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The Tucson, Arizona, Flood of October 1983

Committee on Natural Disasters
Commission on Engineering and Technical Systems
National Research Council

THE METEOROLOGICAL EVENT AND THE WARNING PROCESS

Episodes of heavy rainfall over extensive parts of northwestern Mexico and the southwestern United States occur nearly every year in late summer and early fall. The location of areas receiving maximum precipitation from the storms varies from one storm period to the next and depends on the meteorological conditions prevailing at the time of the event. Storms that produce the largest amounts of precipitation occur when ocean temperatures in the southeastern part of the Pacific Ocean are high, when moist air associated with the remnants of a tropical cyclone in that region is drawn across Mexico into the American Southwest, and when this air then interacts with a significant frontal system associated with an upper-level cold trough or low over the region.

This "tropical connection" has been examined by a number of scientists, notably Douglas (1972), Pyke (1975), and Court (1980). They have identified the significant events in the precipitation history of the Southwest attributable to tropical storms. Their work has been analyzed and amplified by Hansen and Schwarz (1981), who have established the conditions under which a hypothetical storm would produce maximum concentrations of precipitation in Arizona. They suggest that the ideal conditions are:

1. Antecedent synoptic-scale weather features that permit the accumulation and transport of significant moisture into the Southwest well ahead of a tropical cyclone circulation. This moisture is necessary for the rainfall to be of long duration. The August 1951 storm that gave the greatest long-duration rainfall of record had such an antecedent weather feature. This feature allows for substantial rainfall prior to that associated with the hypothesized tropical cyclone circulation.

2. The southward development of a midlatitude cold trough aloft to help accelerate the storm as it turns northward or north-northeastward and crosses Baja California's coast and mountainous backbone at its lowest elevation (near 29°N latitude). The accelerated speed decreases the time the storm spends over land and therefore minimizes loss of intensity. In the optimum case the tropical cyclone should regain some of its intensity as it moves over the small area of warmer waters in the northern portion of the Gulf of California.

3. Maximum or near maximum sea surface temperature (SST) off the west coast of Baja California. This permits an offshore tropical cyclone to remain fully developed farther north than under normal SST conditions. This apparently was the case with the September 24-26, 1939, storm. Tropical storm Joanne in October 1972 was also fed by above-normal SST.

4. A well-formed tropical cyclone gaining intensity well south of Baja California and moving slowly northwest or northward so as to permit the optimum realization of the antecedent tropical cyclone rainfall.

5. A tropical cyclone track that, after reaching the latitude of Baja California, parallels the coast at just the right distance offshore so that, in addition to having a good supply of energy from Pacific Ocean waters, the outer fringes of the massive storm circulation draw from the very warm waters of the Gulf of California.

6. Entrance into southwestern Arizona with a circulation of great strength, after which the remnant storm interacts with a significant midlatitude frontal system associated with an extremely cold trough or low pressure aloft, as occurred in the disastrous storm of September 1970.

The list of conditions established by Hansen and Schwarz was derived by examining a large number of episodes of heavy rainfall that had tropical origins and were associated with the remnants of tropical storms that hit the American Southwest. Table 1 lists some of the most significant tropical cyclones that have produced moisture in the area.

THE STORM OF SEPTEMBER 28-OCTOBER 3, 1983

The August before the storm of September 28-October 3, 1983, had been a very wet month; so had September. In fact, it had rained almost every other day at many climatological stations in Arizona. This, in the normally dry fall season, was unusual. On September 28 the surface weather map exhibited few unusual features. A thermal low lay over the head of the Gulf of California, and the tail end of a weak cold front appeared across the Great Basin to the north. (Weather maps may be found in Appendix A.)

At the 500-mb level, however, an immense trough elongated in a southwesterly to northeasterly direction had developed, bringing tropical moisture into the area. At the same time, a tropical storm, Octave, was gaining strength off the tip of Baja California. Winds at virtually all levels above the surface were from the south to southwest. Isobars at the 500-mb level trended in the same direction. The National Weather Service (NWS) Forecast Office in Phoenix noted this and forecast the renewal of summer monsoon-type showery weather.

Precipitation in Tucson began innocently enough at 5:00 p.m. on Wednesday, September 28. Off and on until midnight, 0.07 in. of rain fell. The rain then ceased until noon on September 29, when a shower occurred. Rain then persisted until noon on September 30. The first flash flood warning of the period was issued by the Tucson NWS Office on September 29 for the period between 5:40 p.m. and 8:00 p.m. At 10:20

TABLE 1 Selected Storms that Have Affected the American Southwest

Date	Tropical Storm	Affected Area
Sept. 24-26, 1939	--	Arizona/California/Nevada
Aug. 26-29, 1951	Charlie	Northwest Mexico/California/Arizona
Aug. 17-19, 1960	Diana	Baja California/Sonora
Sept. 15-19, 1963	Katherine	Southern California
Sept. 23-26, 1965	Hazel	Northwest Mexico
Aug. 29-Sept. 2, 1967	Katrina	Southeast California/Southwest Arizona
Sept. 7-14, 1969	Glenda	Central Arizona
Sept. 4-6, 1970	Norma	Central Arizona
Aug. 27-Sept. 6, 1972	Hyacinth	Southern California/Arizona
Sept. 30-Oct. 6, 1972	Joanne	Southern, central, and eastern Arizona
Sept. 6-10, 1976	Kathleen	Southern California/Arizona
Aug. 11-15, 1977	Doreen	NW Mexico/California/Arizona
Oct. 6-11, 1977	Heather	Mexico/Arizona
Sept. 28-Oct. 3, 1983	Octave	Mexico/Arizona

p.m. a weather statement was issued indicating decreased rainfall and thunderstorm activity. Furthermore, it added:

> The Santa Cruz River is quite high--and persons living near the river should be cautious--as there may still be localized flooding of the lower banked areas. . . . Light to moderate rain is still falling over eastern Pima County--and dips and washes may still have some running water for the next few hours. Motorists in the affected areas should continue to use caution in eastern Pima County.

By Friday afternoon there were clues appearing that more heavy precipitation was due. Moisture, in the form of clouds, could be seen streaming northward from tropical storm Octave (Figure 1), and meteorologists at the Phoenix NWS Office marked the location of several embedded, precipitation-enhancing short waves rotating around the major upper-level trough (Figure 2). As a consequence, the hydrologists with the Joint Federal-State Flood Warning Office issued a statement:

> Heavy rainfall during the past few days has caused significant rises along many rivers and streams throughout the eastern two thirds of Arizona.
>
> Although no mainstream flooding has been reported, . . . the San Francisco River near Clifton remains near bankfull . . . . and lowland overflow has been reported along the Santa Cruz River near Marana today. Significant flows have also been reported

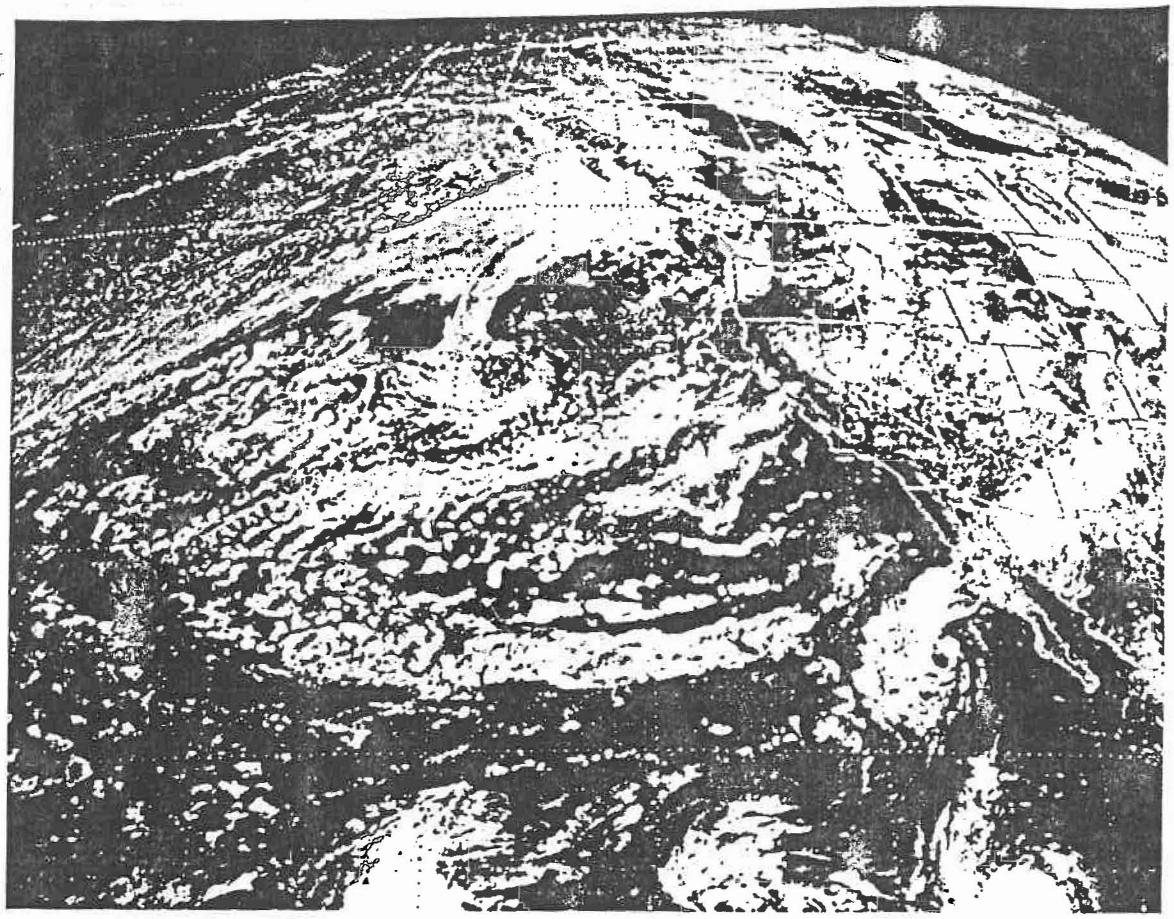


FIGURE 1 Moisture from tropical storm Octave, located off the west coast of Baja California, may be observed in the band of clouds extending across northwest Mexico, Arizona, and New Mexico. This photograph was taken at 6:15 p.m. GMT on October 2, 1983.

> along portions of the Verde River and its tributaries and along Tonto Creek.
>
> The rains have saturated the ground and filled most streams and rivers. . . . Any additional rainfall will run off rather rapidly and could cause increased or renewed rises.

Thus, when it began raining shortly after midnight on Saturday, October 1, the Phoenix NWS Office issued a flash flood watch for all parts of central and southern Arizona and, in particular, south- and west-facing mountain slopes. It said:

> Many areas of central and southern Arizona have received from 2 to 4 inches or more of rainfall since Wednesday, September 28 . . . with lesser amounts elsewhere. The ground has now become

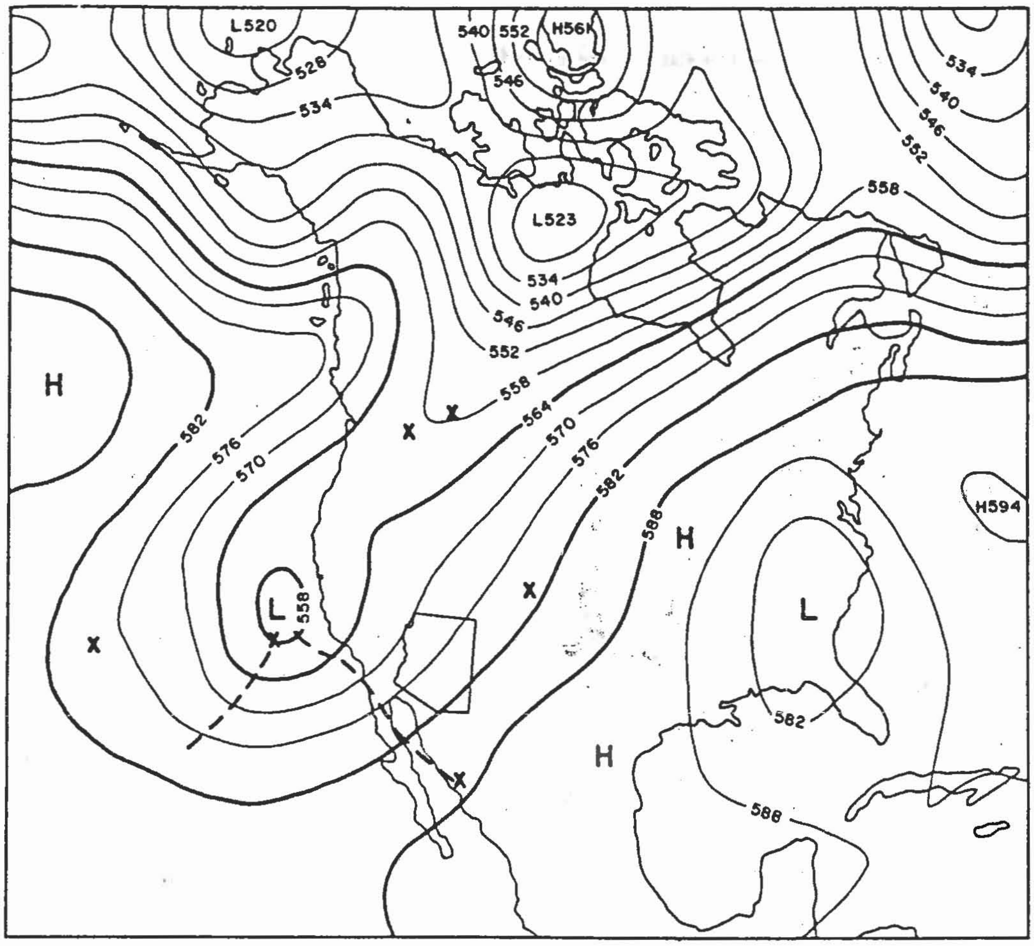


FIGURE 2 National Weather Service 500-mb analysis for Friday evening, September 30, 1983. Embedded short waves are shown by dashed lines.

saturated in a large portion of the state. As a result . . . additional rainfall in these areas now has a very high risk of running off almost immediately.

Satellite photos during the predawn hours show heavy amounts of moisture approaching Arizona from both the west and south. Very strong southwesterly winds in the upper atmosphere will cause this moist air to impinge upon the central mountains . . . and force the air to release its moisture. As a result . . . the potential for heavy rains exist for these south- and west-facing slopes of the central mountains . . . as well as those remaining portions of Arizona south and west of the central mountains.

Persons in the watch area, . . . particularly motorists and those in known flood-prone areas or in areas which have flooded recently, . . . should use extreme caution today. Check preparedness requirements and be ready to move to higher ground immediately if threatening clouds approach or if water levels in your area begin to rise.

Motorists should not attempt to cross flooded roadways . . . as water depths and currents are frequently misjudged.

The heaviest precipitation associated with the storm occurred shortly thereafter and was reported by personnel at the Tucson NWS Office. Their comments appear below:

> At 9:10 a.m. MST Tucson local radar is showing a heavy shower activity throughout eastern Pima, . . . northern Santa Cruz, and northwestern Cochise counties. Most of Pinal County is also affected. An inch and a quarter of rain has fallen at the airport here in Tucson in the last two hours and there have been numerous reports of one and a half inches or more throughout the city and outlying areas.
>
> All rivers, . . . washes, . . . dips, and other low-lying areas are reported running full. Motorists and persons in affected areas are advised to take extreme caution and not enter any flooded areas.

Heavy rains continued over much of Arizona during the morning. Rates of over 1/2 in./h were reported from a number of sites in the southeastern part of the state. By noon the Tucson NWS Office could report a lessening of precipitation, but flooding was widely reported in the metropolitan area.

The 2:50 p.m. statement from the Phoenix NWS Office on Saturday afternoon did not give much hope for relief. It read:

> [There is a] flash flood watch for south-central and southeast Arizona until 5 a.m. MST. . . . The watch area includes Pima, . . . Pinal, . . . Santa Cruz, . . . Cochise, . . . Maricopa, . . . Southern Yavapai, . . . Gila, . . . Graham, and Greenlee counties.
>
> A steady stream of subtropical moisture is moving into Arizona from the south. This will continue to bring numerous and locally heavy showers over all but the far west portions of the state. For tonight the areas most vulnerable to heavy rain are the central and east-central mountains southward.
>
> Heavy rains have continued to fall in the Tucson area through early afternoon. Satellite pictures indicate that much of Pima, . . . Pinal, and Graham counties are continuing to get heavy showers.
>
> A flash flood watch means flash flooding is possible. Motorists should stay out of flooded stream crossings and highway dips and avoid narrow . . . steep-walled canyons.
>
> Heavy rains of the past few days and again today have already brought considerable flooding problems. All persons with property subject to flash flooding should take immediate action for protection if possible.

During Saturday evening, continued runoff and the possibility of additional precipitation brought this flood statement from the Joint Federal-State Flood Warning Office:

> Continuing rains during the past several days and especially today have caused significant rises throughout the Santa Cruz Basin and its tributaries. Water levels throughout the river are extremely high with lowland overflows occurring in the Continental and Marana areas.
>
> Riverbottom road crossings throughout the Tucson area are closed because of the high water. Possible additional rains could cause further increases in river level or renewed rises with some additional overflow.
>
> Persons located near the river should remain alert to current conditions and the possibility of the sudden increases in river levels through Sunday.

The air flowing across Arizona continued to be warm and unstable, and satellite pictures and radar echoes continued to show large thunderstorms over northwestern Mexico. At 9:50 p.m. local radar showed mostly light showers in the Tucson metropolitan area moving toward the northeast. However, at 1:25 a.m. on October 2 the radar picked up a line of showers that extended from Redrock, north of Tucson, to a point just west of Sasabe. These cloud cells were moving toward Tucson at about 20 mph and were forecast to reach the area around 3:00 a.m. Satellite photographs had shown them to be increasing in intensity.

> Local inflow from very heavy and persistent showers and thunderstorms in the Tucson area has also dramatically increased the flow in the Santa Cruz River. The flow in the river has increased between Continental and Tucson. This flow is still far short of that which is needed to cause the river to leave its channel at Tucson. However, . . . local inflow into the Santa Cruz in the Tucson area from these heavy showers and thunder-storms has caused a sharp rise in the river. While the river is still well within its channel, . . . heavy lateral erosion of the river banks has . . . and will continue to take place through at least 9 a.m. this Sunday morning. Those persons affected by this erosion should move to a place of safety immediately.

At the same time, numerous law enforcement agencies were indicating that all dips, washes, rivers, and low-lying areas in the Tucson metropolitan area were full of water. The Rillito at North Country Club Road was overflowing its banks, and there was water in the surrounding streets. Considerable lowland flooding had occurred near Marana and northward along the Santa Cruz. The overflow of the Santa Cruz at Continental continued.

However, the worst of the heavy precipitation was over. Sporadic showers continued in the Tucson area through noon on October 3.

However, continued runoff on major watersheds contributed to severe flooding at Clifton on the San Francisco River, at Safford and Duncan on the Gila River, and on the San Pedro and Santa Cruz rivers. Most of these streams crested before noon on the third, but additional precipitation could have caused additional problems.

By 6:30 p.m. on October 3, showers and thunderstorms in the affected area had decreased markedly. The Phoenix NWS Office issued a flash flood statement that canceled the flash flood watch still in effect. The storm was over--the task of recovery had begun.

WAS THIS "THE STORM OF THE CENTURY"?

Suggestions that Tucson has seen its 100-year storm and need not worry about another one for another century are simplistic and false. Even if it were a 100-year storm, there would still be a one percent chance of another next year or any other year. A review of the probable maximum precipitation (PMP) conditions given above reveals that one ingredient in the maximum storm event was missing. Tropical hurricane Octave, like Norma in 1970, expired quietly at sea and did not penetrate Arizona. However, the copious and continuous precipitation associated with this particular event may be attributed to the presence of the other factors listed in the first part of this section. Ocean temperatures off the west coast of Mexico were above normal (Figure 3), and an elongated upper-level trough that penetrated to the tropics brought warm, moist, unstable air into the region. Also, cold air had been advected into the region on the back side of the trough.

The significant and unusual features of this storm period were the amounts of precipitation that occurred prior to September 28 and the duration of the precipitation in the five days that followed. August and September had been extremely wet over much of the central and eastern part of the state. September was the wettest September of record at a number of locations in Arizona. Then, once it began to rain on September 28, there were only brief respites from precipitation in the days that followed. At the Tucson NWS Office there were 26 different hourly observations in which precipitation was reported on September 28, 29, and 30. After the rain began again between 1:00 and 2:00 a.m. on October 1, it lasted for almost 42 hours with brief pauses between showers (Figure 4). Most of the severe flooding was associated with the persistent rains in the morning hours of October 1 and October 2. But, in comparison to precipitation from a large August thunderstorm, the amounts were small (Table 2), and the return periods for time durations of less than three hours were three years or less (Figure 5).

However, the rain that began falling shortly after 6:00 a.m. in western Tucson on October 1 culminated in a heavy shower of 0.78 in. between 3:00 a.m. and 4:00 a.m. on October 2, bringing a 24-hour total of 3.58 in. Values derived by Paul Kangieser, former State Climatologist for the National Weather Service, from the <u>Precipitation-Frequency Atlas of the Western United States</u> (National Weather Service, 1973),

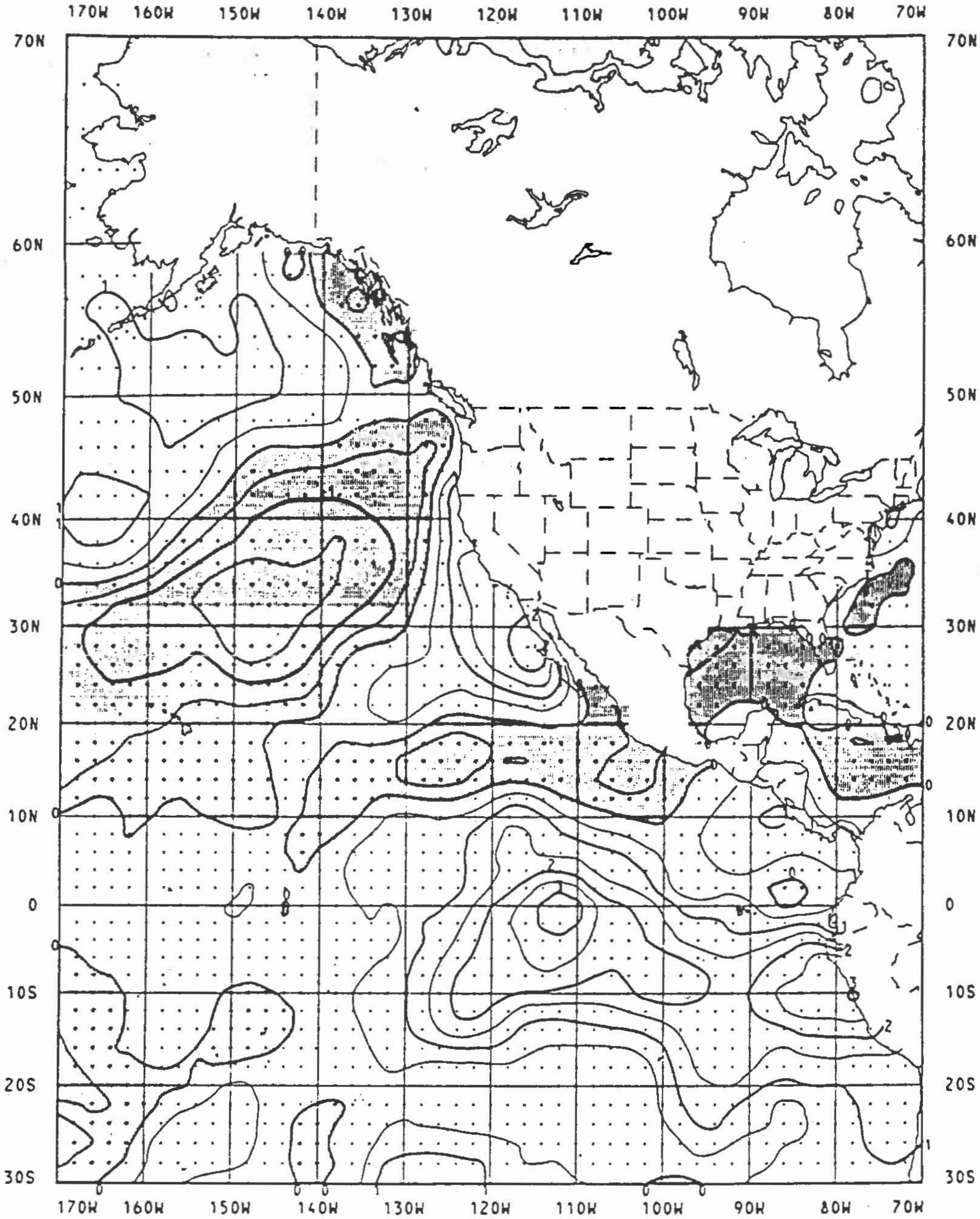


FIGURE 3 Mean sea surface temperature anomalies for September 1983. Note the pool of warm ocean water off the coast of Baja California where tropical storm Octave formed. The monthly anomaly is the difference between the monthly mean sea surface temperature and the climatological monthly mean value. Shading shows where the monthly mean is colder than climatology. The contour line interval is 0.5°. Source: Oceanographic Monthly Summary, 1983.

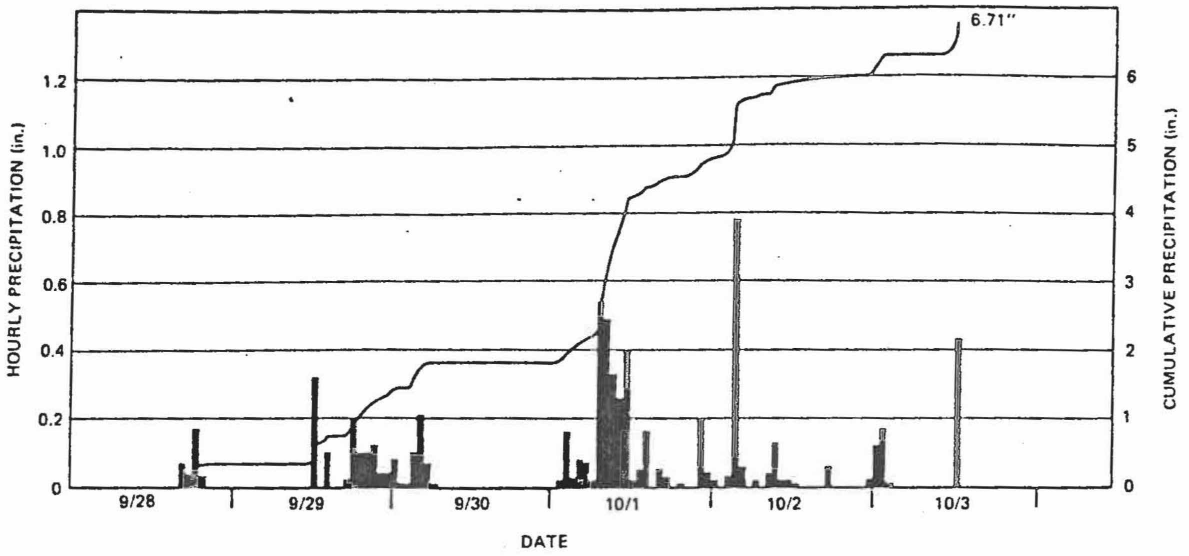


FIGURE 4 Precipitation at the Tucson NWS Office between September 28 and October 3, 1983. Bars show hourly totals; the solid line shows cumulative totals.

TABLE 2 Maximum Precipitation at Tucson from the Storm of September 28-October 3, 1983

Period	Precipitation (in.)	Date at End	Time at End
5 min	0.27	Oct. 2	3:20 a.m.
10 min	0.40	Oct. 2	3:22 a.m.
15 min	0.42	Oct. 2	3:22 a.m.
20 min	0.44	Oct. 2	3:22 a.m.
30 min	0.65	Oct. 2	3:22 a.m.
45 min	0.69	Oct. 2	3:22 a.m.
60 min	0.77	Oct. 2	4:01 a.m.
80 min	0.83	Oct. 2	4:16 a.m.
100 min	0.99	Oct. 1	8:55 a.m.
120 min	1.23	Oct. 1	9:15 a.m.
150 min	1.32	Oct. 1	9:45 a.m.
180 min	1.37	Oct. 1	10:15 a.m.
6 h	2.04	Oct. 1	12:00 p.m.
12 h	2.45	Oct. 1	3:00 p.m.
29 h	3.58	Oct. 2	4:00 a.m.

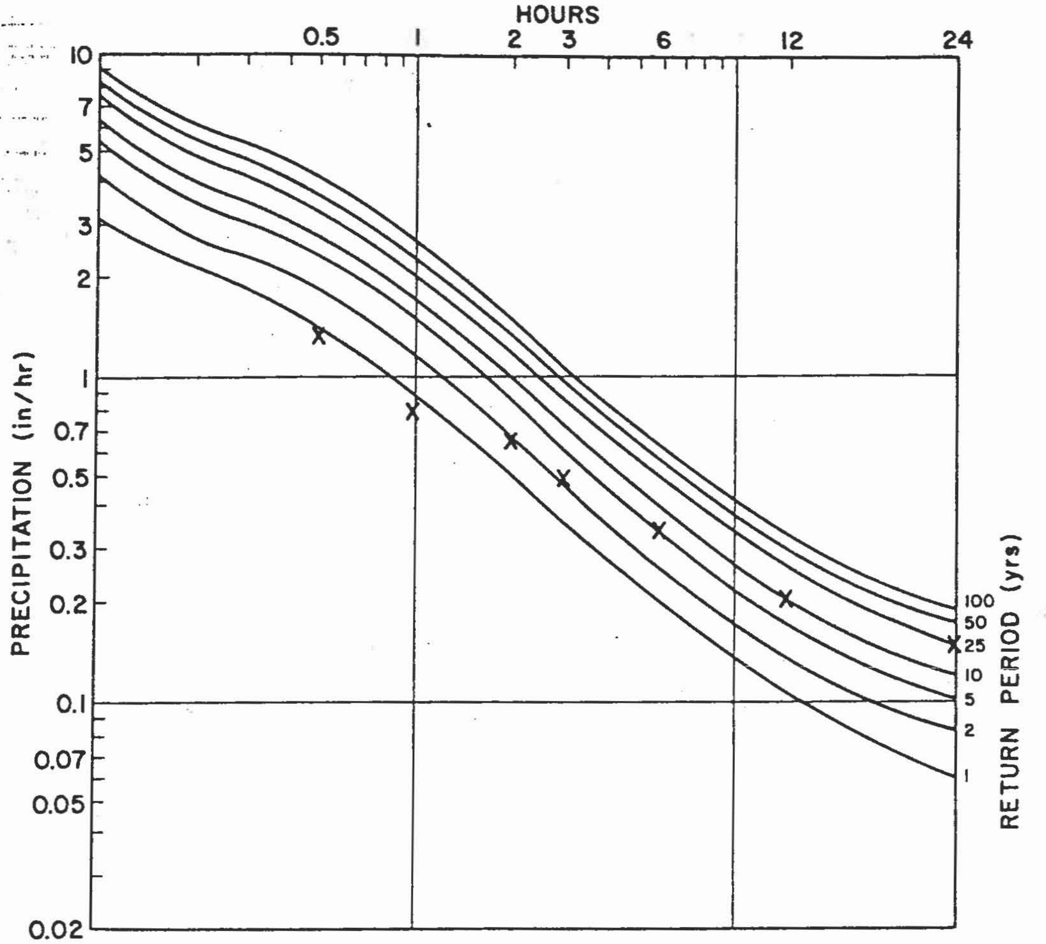


FIGURE 5 Rainfall intensity-frequency distributions at the Tucson NWS Office.

appear as Table 3. It can be seen from the table that precipitation occurring during the period of greatest intensity did not exceed values for a 25-year return period at the weather station located at Tucson International Airport.

There have been a number of differences of opinion as to the intensity of this storm. If one adopts techniques other than those used by Kangieser, including the Pearson III distribution, a return period of nearly 50 years can be derived. However, Kangieser argues that the Pearson III distribution overestimates return periods for rainfall durations longer than 6 hours. A more accurate estimate, he contends, can be made using smoothed regional values of the kind found in the Precipitation-Frequency Atlas of the Western United States (P. Kangieser, personal communication, 1983). Figure 6 compares return periods calculated on the basis of these two techniques.

TABLE 3 Estimated Return Periods for Precipitation in Tucson

Duration	Return Period (yr) 1	2	5	10	25	50	100
5 min	0.27	0.34	0.44	0.50	0.61	0.68	0.77
10 min	0.41	0.52	0.68	0.78	0.94	1.06	1.19
15 min	0.53	0.66	0.86	0.99	1.19	1.34	1.50
30 min	0.73	0.92	1.19	1.37	1.65	1.86	2.09
1 h	0.92	1.16	1.50	1.73	2.09	2.35	2.64
2 h	0.95	1.28	1.67	1.94	2.35	2.64	2.96
3 h	1.06	1.35	1.78	2.08	2.52	2.84	3.18
6 h	1.17	1.50	2.00	2.34	2.84	3.21	3.59
12 h	1.26	1.65	2.24	2.62	3.19	3.62	4.06
24 h	1.37	1.81	2.48	2.92	3.55	4.05	4.54

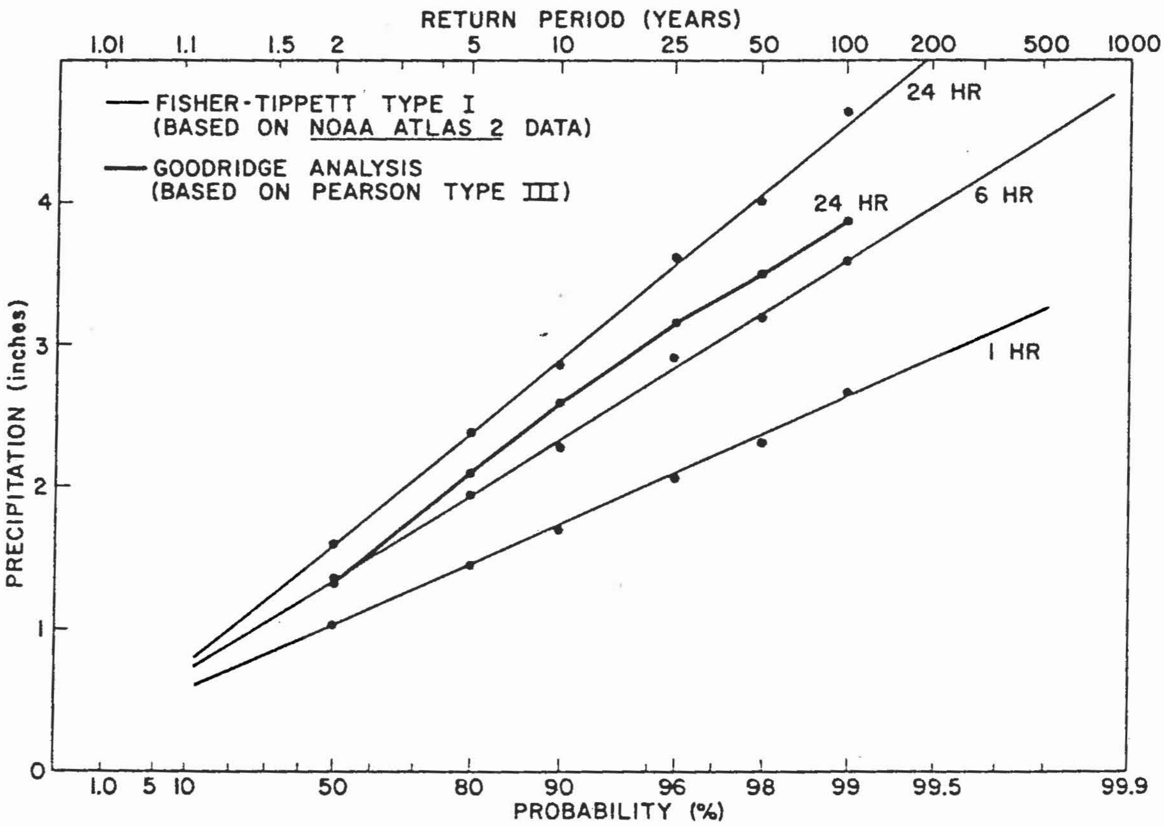


FIGURE 6 Return periods calculated on the basis of a Pearson III distribution and data from the Precipitation-Frequency Atlas of the Western United States (NOAA Atlas 2).

But how unusual was the meteorological event itself? On September 23, just prior to this episode in Sonora and eastern Arizona, the area around Prescott in central Arizona was drenched with rainfall from a tropical air mass. Shortly after October 3 another tropical storm entered the coast of western Mexico, and moisture from the storm contributed to flooding in Mexico and Texas. Some of the other periods when surges of tropical air from the eastern Pacific Ocean affected northwestern Mexico and the southwestern United States were listed in Table 1.

One of the largest and most severe storms with a tropical connection was the Labor Day Storm of 1970, which is generally used as a model of what can happen in Arizona. The isohyetal map for this storm appears in Figure 7. As can be seen, the areal extent of this storm and the amounts of precipitation dropped on the state were comparable to the present event. Intensities during the 1970 storm were higher, and the meteorological conditions were somewhat different, but the results were similar, although the area affected was farther north and west.

However, not too many of these persistent tropical surges have passed over the Tucson area and into the White Mountain region to the northeast. In this case, the flow of air at all levels directed a stream of moisture from off the coast of Baja California toward the northeast. It was the persistence of this flow that was unusual and not the intensity of the precipitation associated with any particular part of it. Much of the precipitation was orographic in nature. Maximum amounts occurred on the south and west slopes of the mountains, and the highest values were recorded at high elevations. Mount Lemmon reported over 10 in.; Blue, farther northeast, in the White Mountains, nearly 11 in. The location of the stream of moist air can be seen by looking at the map of total storm precipitation (Figure 8) and by examining the 500-mb charts in Appendix A.

Thus, although Tucson did not experience its "storm of the century" during September and October of 1983, it is possible that a future surge of tropical moisture across this area will cause such an event for the Tucson metropolitan region. Given the consequences of the 1983 storm, the result will be devastating.

THE WARNING PROCESS

The process of providing the public with timely warnings of impending weather-related disasters involves a number of groups that need to work in unity at the time of the event. To do so effectively requires working and planning together prior to the event. Lack of an adequate observational network, lack of adequate communication facilities, limited cooperation among agencies charged with serving the public, and a lack of understanding among governmental officials about the nature of disasters and about the available warning systems often result in a poor set of responses when an emergency situation exists. The situation in Pima County in early October 1983 was no exception.

During any major flood event, hydrologists and meteorologists need timely reports of precipitation and runoff. During the Tucson flood the

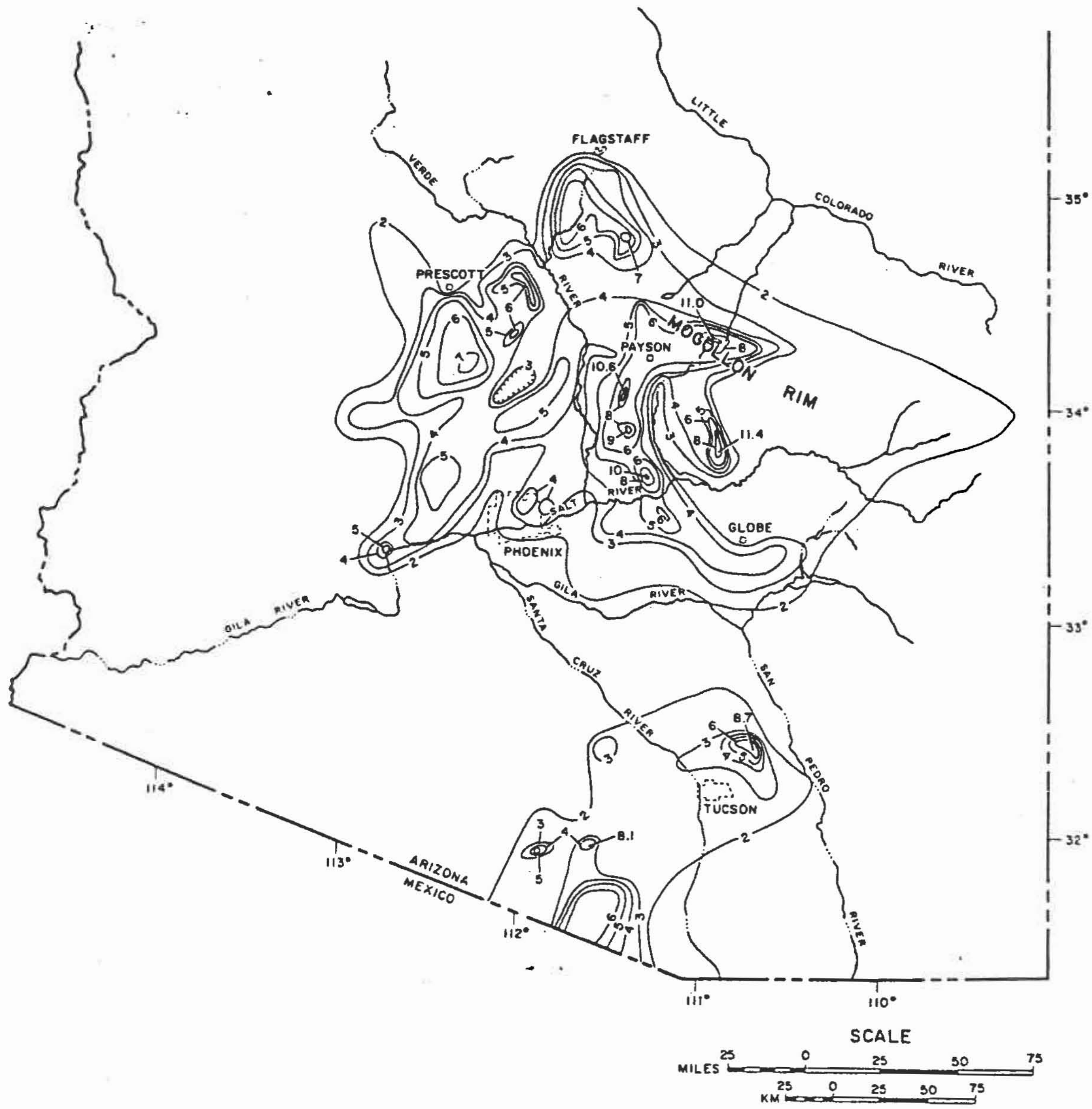


FIGURE 7 Rainfall in southern and central Arizona from the Labor Day Storm of September 4-6, 1970, in inches.

DARDC (Device for Automatic Remote Data Collection) Network of rainfall reporting stations worked reasonably well when queried locally by the Phoenix NWS Office's NOVA 4 computer. However, reports from the same network, when queried by the national Central Area DARDC Automated System (CADAS) failed to respond. Thus the only other precipitation data available to weather forecasters and hydrologists during the storm period were the routine morning and late afternoon reports from regularly reporting climatological substations and other agencies. Collection of reports from these stations and others was hampered in some

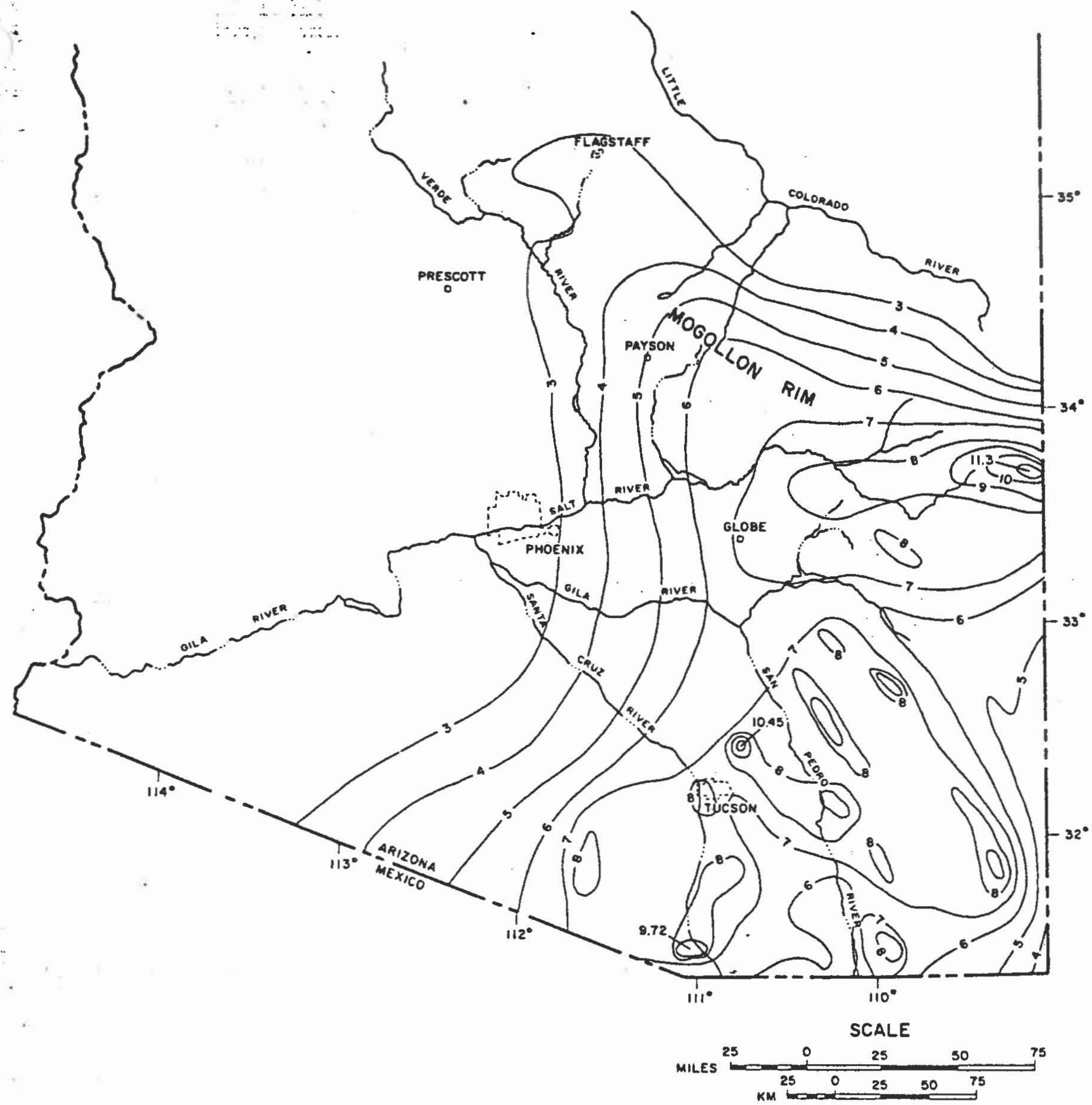


FIGURE 8 Rainfall in Arizona from the storm of September 28-October 3, 1983, in inches.

instances by telephone outages. However, in Tucson the local network of cooperating observers provided valuable information to the Tucson NWS Office.

The system to acquire real-time hydrologic data did not function effectively during the storm period. All stations in the network report at 00Z and every three hours thereafter via satellite to the downlinks at NOAA's Wallops Island facility in Virginia and at the Salt River Project office in Phoenix. Additionally, the stations report every 15 minutes to the Salt River Project after certain specific criteria are met. During the critical period of the storm on October 1, the Salt River Project downlink was out of service from 7:00 a.m. to about 8:30 p.m. due to computer problems. Valuable data were lost and not available to hydrologists concerned with problems of river flooding.

In addition to problems of acquiring data and information on precipitation and flood flows, there existed problems of communication and understanding. In evaluating their performance during the emergency, officials of Pima County admitted that their efforts could not be given superior grades but blamed part of the problem on inconsistent statements from the National Weather Service. As can be seen from the excerpts in the preceding text, this "passing of blame" was unjustified. From 6:00 p.m. on September 29 until midnight on October 5, the Tucson NWS Office issued 20 warnings and statements, of which 13 were radar-generated updates. In the same period the Phoenix NWS Office issued 28 warnings, watches, and statements, of which nine were flood warnings generated by the Joint Federal-State Flood Warning Office.

Weather statements and radar updates reflect observed conditions and vary with those conditions. Watches and warnings are subjective evaluations of meteorologic and hydrologic conditions occurring during the event. The consistency of such issuances depends on the available data and on the timing of their release. Use of weather watches and warnings by the public and by public officials requires that they understand them and that they understand reasons for what seem to be inconsistent statements and warnings.

All statements and forecasts of the National Weather Service are transmitted to users by the NOAA Weather Wire and by NAWAS (the National Warning System). The former is a statewide hardcopy teletype system to users. At the time of the Tucson storm, it included only three of five TV stations and none of the radio stations in the Tucson area. However, the Weather Wire was relayed to the two Tucson newspapers (the Tucson Citizen and the Arizona Daily Star) by the press-wire facilities of the Phoenix offices of the Associated Press and United Press International (incidentally, these organizations relay such issuances to subscribers of their radio services). The NAWAS voice transmissions went to all sheriffs' offices and other public safety agencies in the state. Thus the amount of information transmitted appears to have been sufficient for the public and the officials appointed and elected to care for their safety to have been well aware of the scope and extent of the disaster.

There is a national program to provide warning of impending disasters--the Emergency Broadcast System (EBS). However, activating this system was not possible because no primary EBS station had been designated in the Tucson metropolitan area and the only radio station that had volunteered to fill this gap was not on the Weather Wire and so did not receive EBS activation requests. Although several flood warnings from the Phoenix NWS Office were headlined EBS REQUESTED, they were not implemented. The Pima County Emergency Services Director was quoted in Tucson's Arizona Daily Star of October 12, 1983, as saying, "We just didn't see the need for activating the EBS system."

NOAA Weather Radio provided the best source of information about the progress of the storm to the agencies in the Tucson area that used it to monitor the weather conditions affecting their operations. According to Dave Williams in Tucson's Arizona Daily Star of October 7, 1983, "On Saturday, the best electronic sources of information about the flooding were the NOAA weather broadcasts."

Although the forecasts issued by the National Weather Service in Tucson during this storm episode were accurate and timely, problems did occur within the agency. Once the storm was in progress, many of the systems and networks used to measure precipitation and runoff malfunctioned, making it impossible to have a clear picture of the series of events that occurred during the storm. In addition, it is difficult for a National Weather Service Office such as that at Tucson, which had only one or two individuals answering the phones, making observations, and handling other station duties, to respond to all of the demands placed on it during an emergency period. Reasons for this are not entirely clear, but some of the problems clearly relate to a lack of manpower.

In addition, there evidently were no policies on the retention and analysis of data during and after the storm. In the past it has been customary for both the Corps of Engineers and the National Weather Service to investigate weather events of this magnitude. Only the Bureau of Reclamation Flood Office out of the Denver Federal Center was critically interested and involved in meteorological analysis of this storm episode.

As the events associated with this storm and flood show, the responsibility for particular kinds of analysis of events during and after weather-related disasters needs to be clearly defined. Lack of understanding of interagency responsibilities is a continuing problem. For example, at 4:30 p.m. on Monday, October 3, the Phoenix NWS Office stated, "The [San Carlos] reservoir is expected to begin spilling later Tuesday afternoon." In fact, spilling began about 1:30 p.m. that Tuesday. However, officials of the Arizona State Division of Emergency Services said in a press release issued Monday evening that they expected San Carlos to spill about midnight that night based on estimates of the State Department of Water Resources. When questioned about this difference of opinion, State Emergency Services personnel said that because people would have to be evacuated, they preferred to take the sooner rather than the later time--a difference of about 18 hours!

It would seem that these two forecasts (the former official and the latter not official) did not lend credence to public releases.

RECOMMENDATIONS

Additional efforts are needed to coordinate the work of the various agencies involved with disasters before, during, and after the events. Both state and federal agencies charged with coordinating the acquisition and analysis of meteorological and hydrological data must exert stronger leadership to ensure cooperation of all existing agencies within the affected areas. Some other topics that need attention are listed below.

1. Safeguarding Flood Data and Information

If the Federal Emergency Management Agency is to continue to be the lead agency in disaster mitigation efforts, there needs to be a clearer statement of the duties and responsibilities of NWS personnel during and after severe weather events so that the National Weather Service's contribution to the analysis of events can be clearly defined. During an event, efforts should be made to acquire and retain all data and information essential to understanding and analyzing it. These materials should be retained until needed by hazard investigators. The originals could be kept by the cognizant NWS office for a specified period of time, such as five years, after which they should be appropriately archived.

2. Investigation of Meteorological Conditions

Each major severe meteorological event should be investigated thoroughly by the cognizant local forecast office of the National Weather Service. These investigators should be assisted by such additional experts as are needed from the regional forecast offices. This analysis should be the basis for reports used by other local and federal agencies. It is an inefficient use of tax dollars to have each event handled on an ad hoc basis, with many federal agencies conducting an analysis of the meteorological conditions contributing to a natural disaster.

3. Issuance of Disaster Warnings

If the disaster or potential disaster is related to weather, then the National Weather Service should have sole responsibility for issuance of watches, warnings, and statements about existing and expected conditions to the public. This is true even in gray areas such as the collapse of dams weakened by rains or overtopping of dams because of excessive river flow.

4. Acquisition of Meteorological Data During Events

Local offices of the National Weather Service should encourage cooperative observers to report more faithfully on weather conditions during severe storm events. Also, ways need to be found to ensure redundancy in automatic systems so that data can be obtained under inclement weather conditions.

5. Communication of Warnings to the Public

Although NOAA Weather Radio appeared to operate effectively during the Tucson flood episode, too few people are aware of this service. During severe weather episodes it is essential that all electronic media make full use of all available information. Local radio stations could, for example, retransmit NOAA Weather Radio broadcasts. Additionally, local television stations, staffed with professional meteorologists, could rebroadcast NWS radar pictures and explain them to the public at the same time that they are presenting verbatim the latest NWS advisories on the air.

6. The Tropical Connection

Episodes of extensive, heavy precipitation have their origins south of the U.S.-Mexico border. To give American citizens adequate warning of these events, better cooperative programs should be developed with the states of northwestern Mexico to monitor and record weather events. Two specific suggestions are (1) the extension of the Automatic Hydrologic Observation System (AHOS) managed by the U.S. Geological Survey and (2) the establishment of a cooperative surface observation program with the state of Sonora or the government of Mexico so that severe storms may be monitored on their way across Sonora.

7. Cooperation Among Local, State, and Federal Agencies

During a major weather-related disaster, officials charged with public safety and emergency services should have coordination representatives of their agencies at the local offices of the National Weather Service. These representatives should be trained and knowledgeable in the operations of the National Weather Service and understand the significance and degree of reliability of the various forecasts issued by the local forecast office. They should be able to translate the various issuances of the National Weather Service into statements about the potential impact of severe weather events on their operations.

3

GEOMORPHOLOGY AND HYDROLOGY DRAINAGES IN THE TUCSON BASIN

Tucson is located in a topographic basin bounded by the Santa Catalina and Tortolita Mountains to the north, the Rincon Mountains to the east, the Santa Rita Mountains to the south, and the Tucson Mountains to the west (Figure 9). The principal watercourses in this basin collect drainage from and flow between these ranges. Like many desert mountain masses, those near Tucson have broad piedmont surfaces extending at fairly uniform slopes of 10 to 30 m/km away from much steeper mountain fronts. These piedmont surfaces may be erosional bedrock surfaces, called pediments, or they may be mantled by fan gravels and dissected by deep washes. The ephemeral streams of the piedmont areas convey water and sediment from the mountain fronts to the valley floors in the basin during occasional rainstorms. Coarser gravel and boulders are deposited mainly on the piedmont, while the finer fraction of the load, including sand, silt, and clay, are conveyed to the valley floors.

The Santa Cruz River begins in the San Raphael Valley along the border with Mexico. The river flows south into Sonora and turns abruptly west and north to reenter the United States east of Nogales. The channel extends northward through Tubac and Green Valley to reach the downtown portion of Tucson. Through much of Tucson the Santa Cruz is deeply entrenched into the sediments of the valley floor. The Santa Cruz drains about 5,800 km^2 to the south of Tucson, including large piedmont surfaces extending from mountains on the east and west sides of its valley. Sediments transported to Tucson are mostly fine sand and silt.

Northwest of downtown Tucson the Santa Cruz River is joined by the Rillito system, which drains about 2,400 km^2 to the north and east of Tucson. The Rillito flows about 20 km along the northern boundary of the city at the base of an extensive piedmont extending from the Catalina Mountains. In northeast Tucson the Rillito is split into two major tributaries, Tanque Verde Creek and Pantano Wash (Figure 10). Pantano Wash collects drainage from extensive areas to the southeast and traverses a long section of basin floor. Because of its length, sediment sizes become relatively fine where Pantano Wash reaches Tucson. In contrast, Tanque Verde Creek transports a relatively coarse load from the nearby Santa Catalina and Rincon Mountains. The coarse sand of Tanque Verde Creek mixes with the finer Pantano sediments to give the

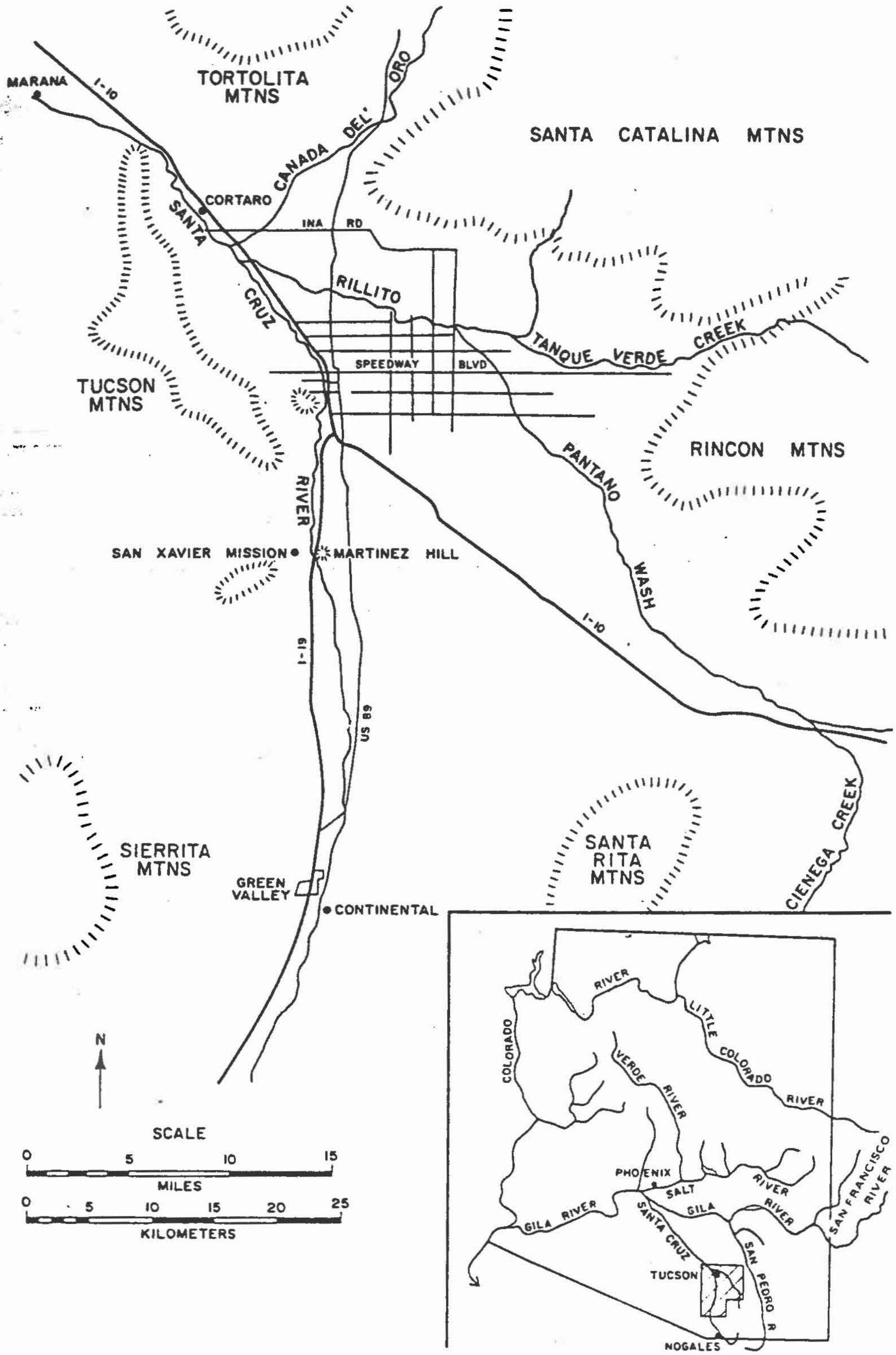


FIGURE 9 Index map of the Tucson Basin showing the principal watercourses, mountain ranges, and transportation routes.

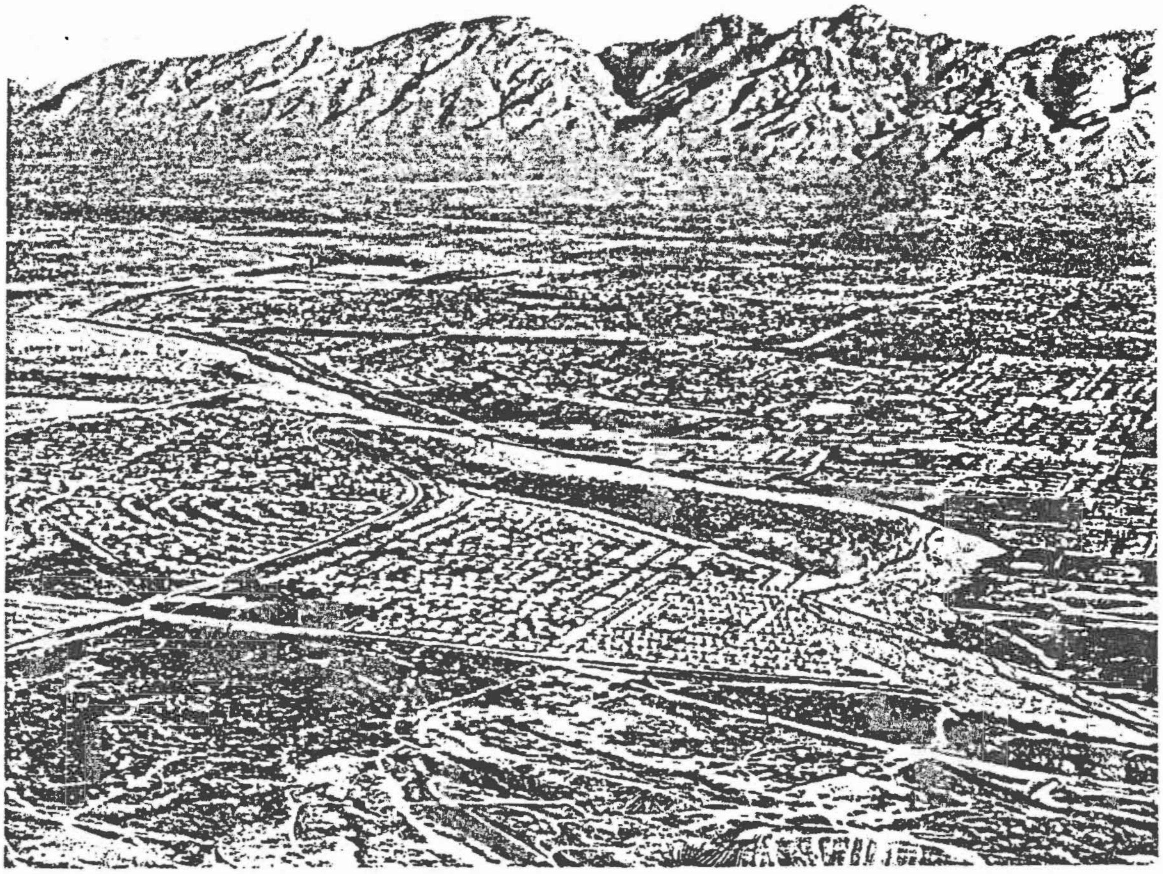


FIGURE 10 Aerial view of the eastern Tucson Basin on October 9, 1983, showing Pantano Wash in the foreground and the Santa Catalina Mountains on the skyline. Flood damage to the Rincon Country mobile home development (bottom center) is shown in Figures 26 and 38. Photograph by Peter Kresan.

Rillito an intermediately coarse sand load, which is then conveyed to the Santa Cruz River, which transports fine sediments. These sediment load characteristics play an important role in the adjustment of the different streams to changing flow conditions.

About 2 km north of the confluence of the Rillito, the Santa Cruz is joined by the Canada del Oro. This stream drains about 660 km^2 from the western slopes of the Santa Catalina Mountains and eastern slope of the Tortolita Mountains.

GEOMORPHIC HISTORY OF TUCSON DRAINAGE COURSES

Floods are usually defined by the damage they do. The flow of water in a stream channel, as studied by hydrologists, is not considered a flood unless it rises sufficiently to cause damage and disruption. In general, flood hazard management emphasizes the economic consequences of rising water levels.

Another view of floods emphasizes their association with the river, the valley, and the valley lands adjacent to the river. This geologic perspective on floods (Moss et al., 1978) is not often incorporated into flood hazard management. In the humid temperate portions of the United States, river channels generally occur within a bottomland surface that is created by the river itself. The river occasionally overtops its banks, carrying and depositing sediment on that surface. This surface is the geologic floodplain of the river. The hazard zones of such rivers show close correlation to this geologic floodplain, since it is inundated relatively frequently. Shallow water might cover it with a 50 percent chance each year, and deep flows with a depth roughly equal to twice the channel bank heights might occur with a 2 percent chance per year (Moss et al., 1978).

The above experience has little applicability to ephemeral sand-bed streams in valley bottomlands of the arid and semiarid West. Streams in that environment may flow directly on broad, low valley floors with little or no channel. In that case every flow event could cause damage, especially as the threads of flow shift on a depositional surface. The contrasting extreme is the incised channel, where the stream has cut through the former bottomlands so deeply that even very rare high flows will not spill out of the incised banks. Both the above conditions may occur on the same stream, either at different localities at a given time or at different times at the same locality. The management of flows in such an environment cannot ignore the geologic complexities of the system. Experience gained in the Tucson flood of 1983 will illustrate this conclusion.

The general channel configurations and cross-sectional shapes on valley floors in the Tucson area today derive from a history of arroyo cutting in the late 1800s and early 1900s. Similar histories are prevalent throughout the southwestern United States, and considerable controversy surrounds attempts to provide a general explanation of the phenomenon (Cooke and Reeves, 1976). At Tucson an extensive historical record allows a precise reconstruction of events associated with the transition of the Santa Cruz from an alluviated valley floor to a narrow, steep-sided channel (Betancourt and Turner, in press).

Prior to the major flood of August 1890, the Santa Cruz River exhibited perennial flow at several reaches near Tucson (Figure 11). The generally high water table intersected the valley floor at these locations and maintained the flow through perennial springs. Two springs upstream of San Xavier Mission, Agua de la Mision and Punta de Agua, served to irrigate fields at the mission. Near such springs small marshes, or cienegas, developed, with lush vegetation, fish, and beaver locally. Reaches between the spring outflows, amounting to 75 percent of the river course in the Tucson Basin, were ephemeral because of infiltration into the dry, sandy riverbed. These reaches were generally marked by shallow swales. However, local sections did have short discontinuous gullies with vertical banks up to 3 m high and 20 m apart.

During the 1880s this system began to reflect a profound human impact. Various impoundment and diversion structures were introduced to

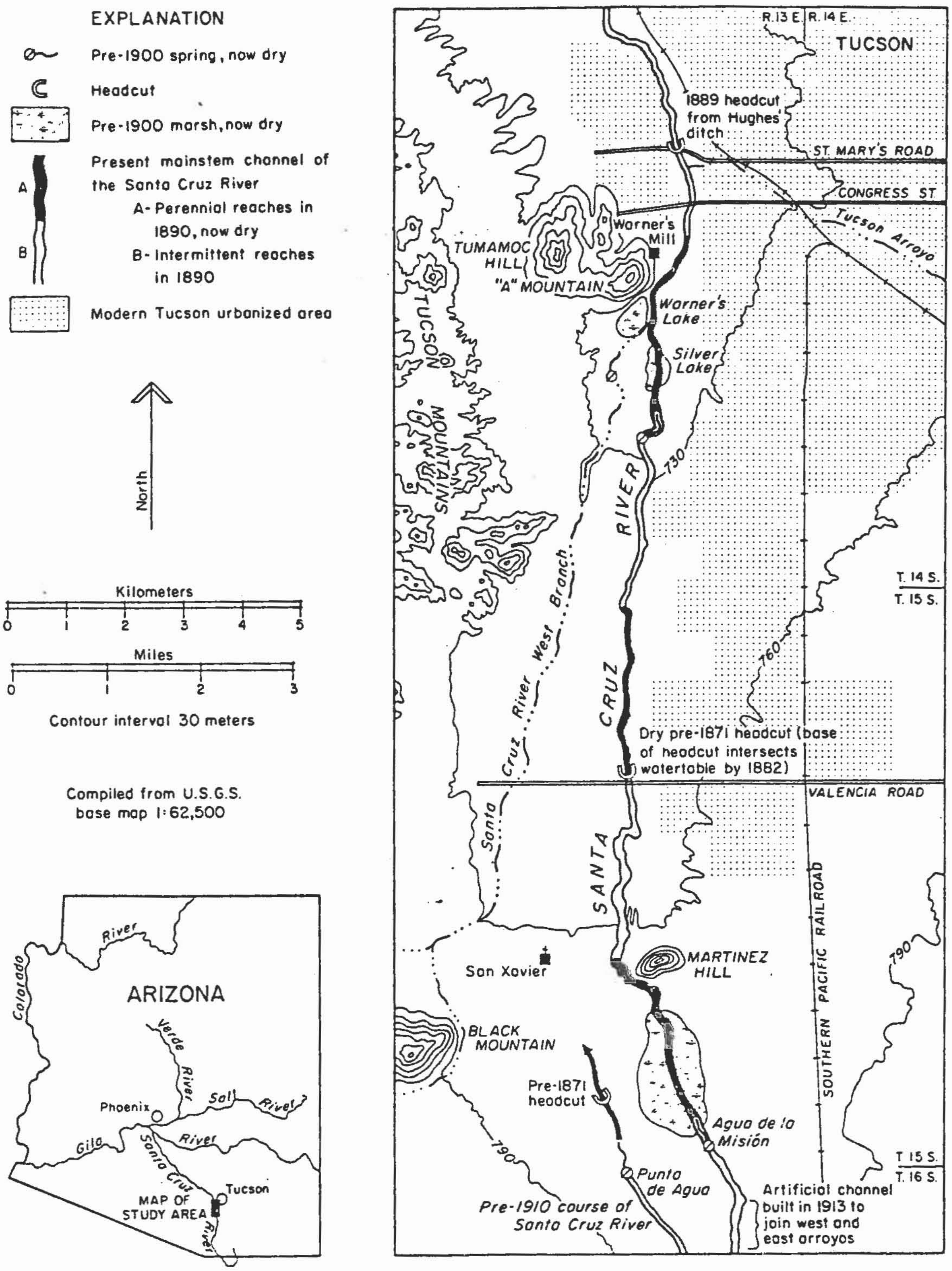


FIGURE 11 Map of a portion of the Santa Cruz valley near Tucson showing the present mainstem channel, perennial and intermittent reaches in 1890, and the location of headcuts and marshes (cienegas) in the late nineteenth century. Source: Betancourt and Turner, in press.

facilitate irrigation on the alluvial valley floor. In 1888 a major ditch was constructed by Sam Hughes near the present St. Mary's Road to intercept underflow in the Santa Cruz alluvium. In October 1889 a relatively small flood cascaded into Hughes' ditch, forming a headcut, or nickpoint. A major flood in July-August 1890 caused this headcut to work its way upstream approximately 4 km to a small impoundment at a former cienega called Silver Lake (Figure 11). The valley floor of the Santa Cruz was now incised with an arroyo about 30 m wide. The headcut continued to recede with each flow event until by 1910 it had reached Martinez Hill.

In the winter of 1914-15 the Santa Cruz experienced several prolonged floods that significantly widened the channel through bank erosion. The Arizona Daily Star on February 1, 1915, recorded the erosion of the east abutment to the Congress Street bridge by bank erosion through meander migration (Figure 12):

> Sudden destructive tendencies developed in the flood that swept down the Santa Cruz River yesterday morning and from 10 o'clock until noon the river rapidly washed away a large section of valuable land enclosed within a wide curve on the east side of the stream just south of the Congress Street road and containing five or more acres, finally about noon destroying more than a hundred and fifty feet [45 m] of embankment that connected Congress Street with the east approach to the bridge. . . . The work of destruction was continued steadily, but more slowly throughout the afternoon and by midnight the rushing water was creeping at the outside of the curve close to the row of cottages just east of the big concrete irrigation ditch and threatening to include the houses in the ruin.
>
> While the river did not rise any higher, it developed a terrific boring power that rapidly crumbled the soft dirt into the swirling current of the muddy Santa Cruz. The current worked with telling effect on the sandy subsoil of the rich arable land of the bottom and the total damage is estimated to be not less than $50,000 at midnight. The east approach to the bridge was swept away leaving 200 feet [60 m] of water between the road and the bridge. The piers of the bridge itself also sank. . . . The sudden driving force that the stream acquired in the morning rendered useless the protective measures taken and there was little that could be done through the day to protect the embankment. Foot after foot went out and by midnight it was estimated that the gap between the bridge and the east end of the road was more than 200 feet [60 m].

With minor changes this description could apply to damage at bridges in the flood of October 1983.

After the floods of 1914-15 the Santa Cruz continued to incise upstream of Martinez Hill (Figure 11). By the 1930s a headcut had migrated from Agua de la Mision up an artificial channel built in 1913 to join the west and east branches of the stream. During the 1920s and the 1930s the incised river near Congress Street began to aggrade. The

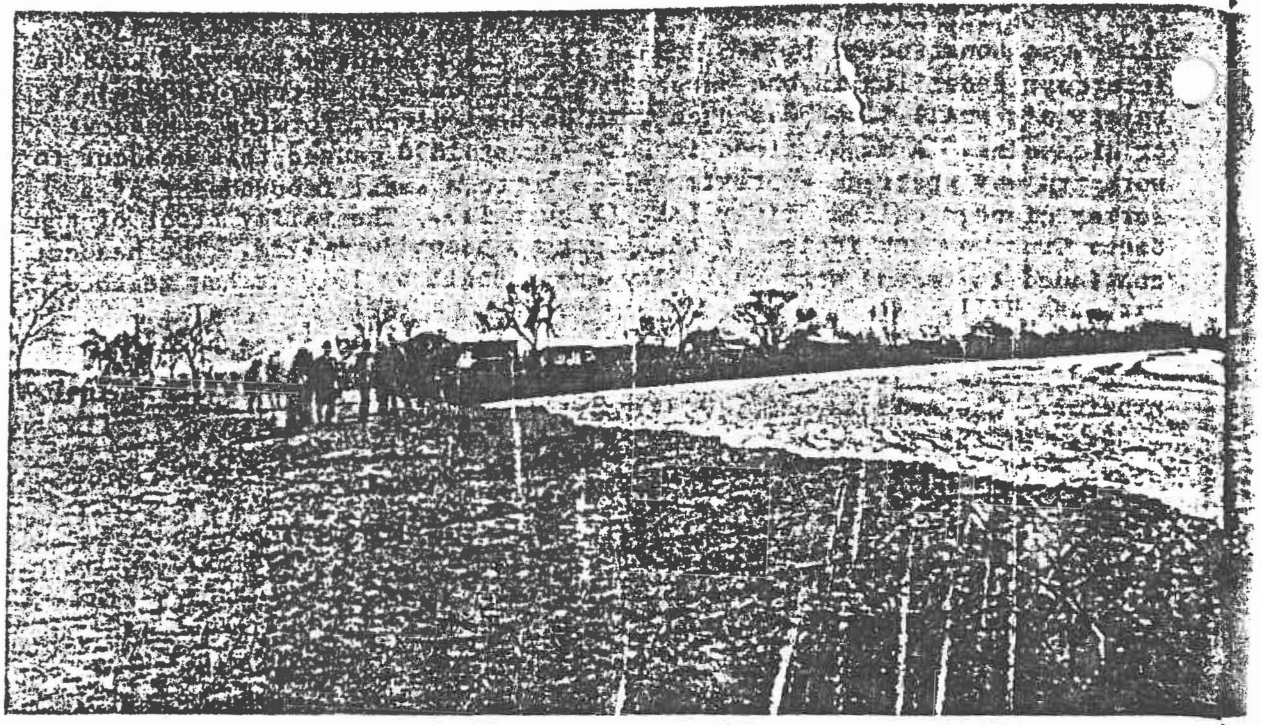
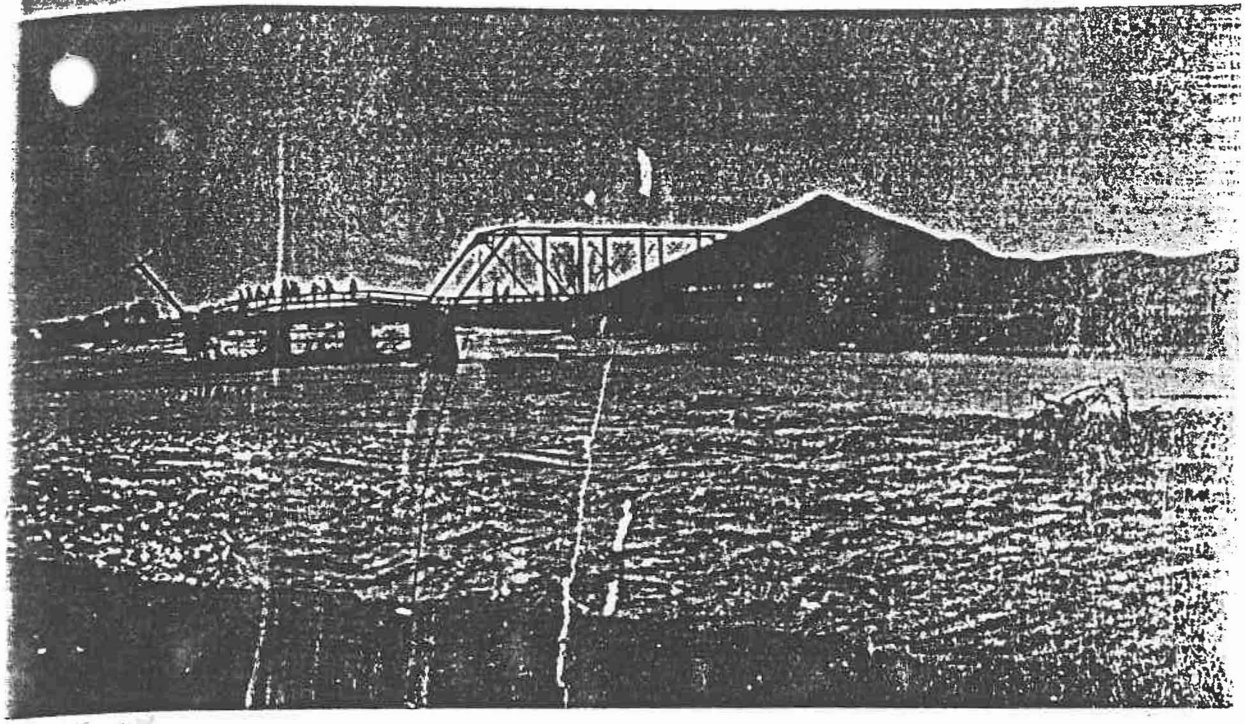


FIGURE 12 Erosion damage to the east abutment of the Congress Street bridge on February 1, 1915. The view is upstream (south), and Sentinel Peak ("A" Mountain) is visible at right. The gap of 60 m from the bridge to the east bank (left in picture) resulted from the downstream migration of a meander bend on that bank. This meander bend was visible in a 1902 photograph (Betancourt and Turner, in press). Source: Arizona Historical Society, Tucson.

aggradation probably resulted from the decreased velocities achieved by the channel widening in 1914-15, from the stabilizing effect of cottonwoods and other riparian vegetation, and from the lack of major winter floods in this period. Because of their relatively low sediment loads (in contrast to summer floods), winter floods are particularly erosive for the sand-bed streams of southern Arizona.

Starting in 1950 the Santa Cruz through Tucson was modified by artificial narrowing through landfill operations and by highway construction. The reach from Martinez Hill to Congress Street had been straightened in 1935 by the efforts of the Works Progress Administration. Flows were deflected from severe bends by means of revetments. The result of these changes was renewed downcutting by higher-velocity flows through the straightened and constricted channel. The zone of aggradation moved downstream, past all the urbanized reaches, to the far northern end of the Tucson Basin.

The Rillito system had an early history similar to that of the Santa Cruz. From a condition in 1858 that included beaver dams along a broad



valley floor, the stream had incised to a wide channel with vertical banks by 1890. The change was attributed by Smith (1910) to cutting of vegetation, overgrazing by cattle, and flood erosion. Channel incision continued into the twentieth century as excessive withdrawal of ground water eliminated riparian vegetation along stream banks and bars.

Studies of aerial photographs since 1941 show that the Rillito-Pantano-Tanque Verde system has been characterized by prolonged periods of channel narrowing locally interrupted by abrupt periods of widening with attendant bank erosion (Figure 13; Pearthree, 1982). Narrowing from 1941 until 1965 occurred during an interval dominated by short-duration floods with peaks less than 300 m^3/s. Most of these flows occurred in the summer and early fall, and they probably transported very high sediment loads. In 1965 a large winter storm produced a prolonged flood that peaked at about 350 m^3/s for Tanque Verde Creek and the Rillito. This flood carried a relatively low sediment load and generated extensive bank erosion for both stream channels. In contrast, Pantano Wash did not experience this flood and continued to narrow.

After the 1965 event both Tanque Verde Creek and the Rillito displayed either natural recovery or local artificial stabilization. In December 1978 another major winter storm affected these streams. As in 1965, this event, which peaked at 464 m^3/s in the Rillito, was characterized by prolonged duration and a relatively low sediment load. In addition, a flow in March 1978 had removed much of the sediment that had accumulated in the stream channels during the preceding recovery period. Extensive bank erosion occurred during a three-day period of flow. Channel realignment, bank protection, and bridge repair after the 1978 flood led to the general condition of the Rillito-Tanque Verde-Pantano system immediately prior to the 1983 flood.

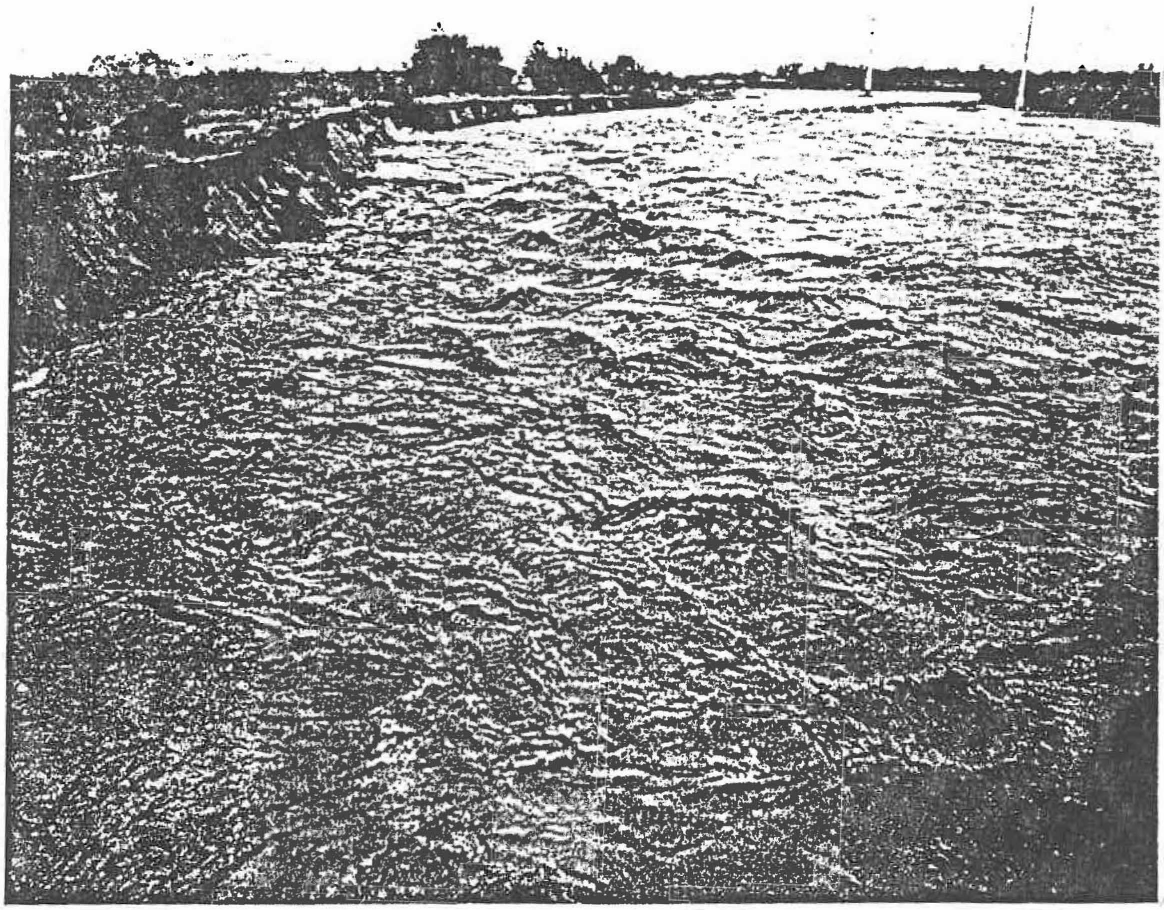


FIGURE 13 View upstream (east) from the Campbell Avenue bridge on October 1, 1983, showing erosion of the north (left) bank by meander flow. Note the dark thread of high-velocity water following the meander thalweg from the next bend upstream. Also note the utility lines in the stream channel. Photograph by Peter Kresan.

It is clear that the streams of the Tucson Basin have been irreversibly altered from conditions that prevailed prior to large-scale human intervention. A return to the channel characteristics of the 1800s is impossible for the following reasons: (1) groundwater overdraft has so lowered the water table that the stabilizing influence of riparian vegetation has been lost, (2) the urbanization process has reduced the influx of sediments from tributaries into the main channels while increasing the influx of water from individual storms, and (3) channels have been constricted by bridges, filling on banks, and revetment works.

HYDROLOGIC ASPECTS OF THE 1983 FLOOD

The flood peak on the Santa Cruz River at Cortaro occurred between 8:00 p.m. on Saturday, October 1, and 9:00 a.m. on Sunday, October 2 (H. W. Hjalmarson, U.S. Geological Survey, personal communication, 1983). Peak flood discharges have been provisionally estimated for several localities in the Tucson Basin (Table 4). Extensive bank erosion resulted in the losses of gages on major watercourses, but postflood hydraulic calculations are being used to determine flood peaks. Final estimates, hydrographs from undamaged gages, and detailed interpretation are still being generated by the U.S. Geological Survey. However, several conclusions concerning flood frequency seem warranted at this preliminary point.

The Santa Cruz River peaked at 1,490 m^3/s (52,700 cfs) at Congress Street. This event exceeds by more than a factor of two any other flood recorded at that station. It exceeds by a factor of 1.75 the magnitude of the 100-year flood designated in the Federal Emergency Management Agency (FEMA) Flood Insurance Study (Federal Emergency Management Agency, 1982). Even more important is the fact that the FEMA flood magnitudes were adjusted upward from the standard procedures. The problem of estimating a true recurrence interval for this event will be treated below. The Rillito also had an extreme flood (Figure 13), estimated at 840 m^3/s (29,700 cfs). The FEMA-determined return period for this event would be between 50 and 100 years.

In contrast to the Santa Cruz and Rillito, other streams in the Tucson area had floods that were much less extreme in relation to past experience. The Canada del Oro at Overton Road had a peak flow of 185 m^3/s (6,600 cfs), as estimated by the U.S. Geological Survey. This compares with the 10-year flood estimate of 280 m^3/s (10,000 cfs), which the federal Flood Insurance Study for the Pima County Department of Transportation and Flood Control District used as the regulatory flood. Pantano Wash peaked at 310 m^3/s (11,000 cfs), which is between the 10-year discharge of 250 m^3/s (9,000 cfs) and the 25-year discharge of 400 m^3/s (14,000 cfs). The record flow of Pantano Wash at Broadway was 570 m^3/s (20,000 cfs) in 1958.

The annual flood peaks of the Santa Cruz River have been recorded at the Congress Street bridge since 1915. In the period 1915 to 1981 (Figure 14), 73 percent of all annual peaks occurred during July and August, 18 percent occurred during September and October, and 9 percent occurred during November through February. No annual peak flows have been recorded in March, April, May, or June. A standard log-Pearson III analysis of this record is shown in Figure 15. This plot was derived according to standard procedures followed throughout the United States (U.S. Water Resources Council, 1981). This plot estimates the 100-year flood as about 650 m^3/s (23,000 cfs), only half the magnitude of the 1983 flood. By this analysis the return period of the 1983 flood would be more than 1,000 years. This is beyond the frequency of events that are considered in usual hazard management.

TABLE 4 Estimated Flood Peak Discharges and Recurrence Intervals for the 1983 Tucson Flood

Locality	Provisionally Estimated Discharge[a] cfs	m^3/s	100-year Flood Discharge[b] cfs	m^3/s
Santa Cruz at Congress Street	52,700	1,490	30,000	850
Santa Cruz at Cortaro	65,000	1,840	40,000	1,130
Rillito at Flowing Wells	29,700	840	32,000	900
Canada del Oro at Overton Road	6,600	185	28,000	800
Pantano Wash at Broadway	11,000	310	25,000	700

[a]H. W. Hjalmarson, U.S. Geological Survey, personal communication, 1984.

[b]Federal Emergency Management Agency Flood Insurance Study.

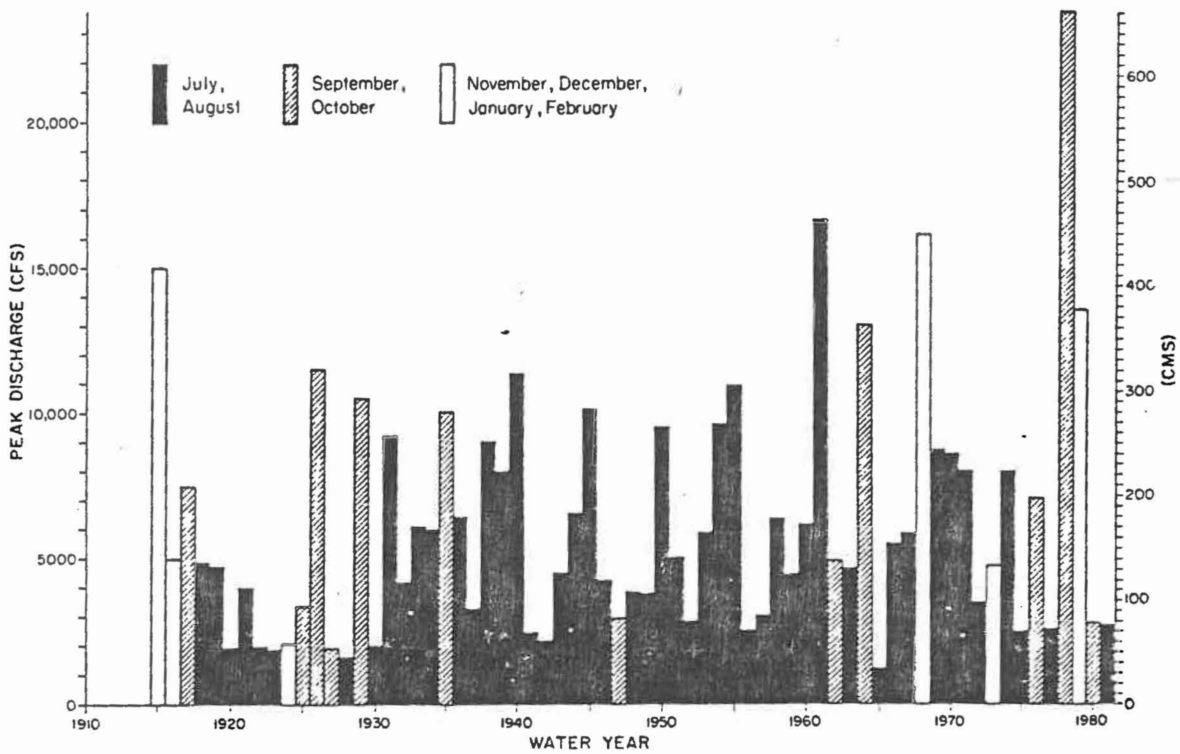


FIGURE 14 Annual peak discharge for the Santa Cruz River at Congress Street between the water years 1915 and 1981. The gage was removed late in 1981. Note the months of peak discharge. Newspaper accounts for the period between 1902 and 1914 also include several major storms in winter. Source: Betancourt and Turner, in press.

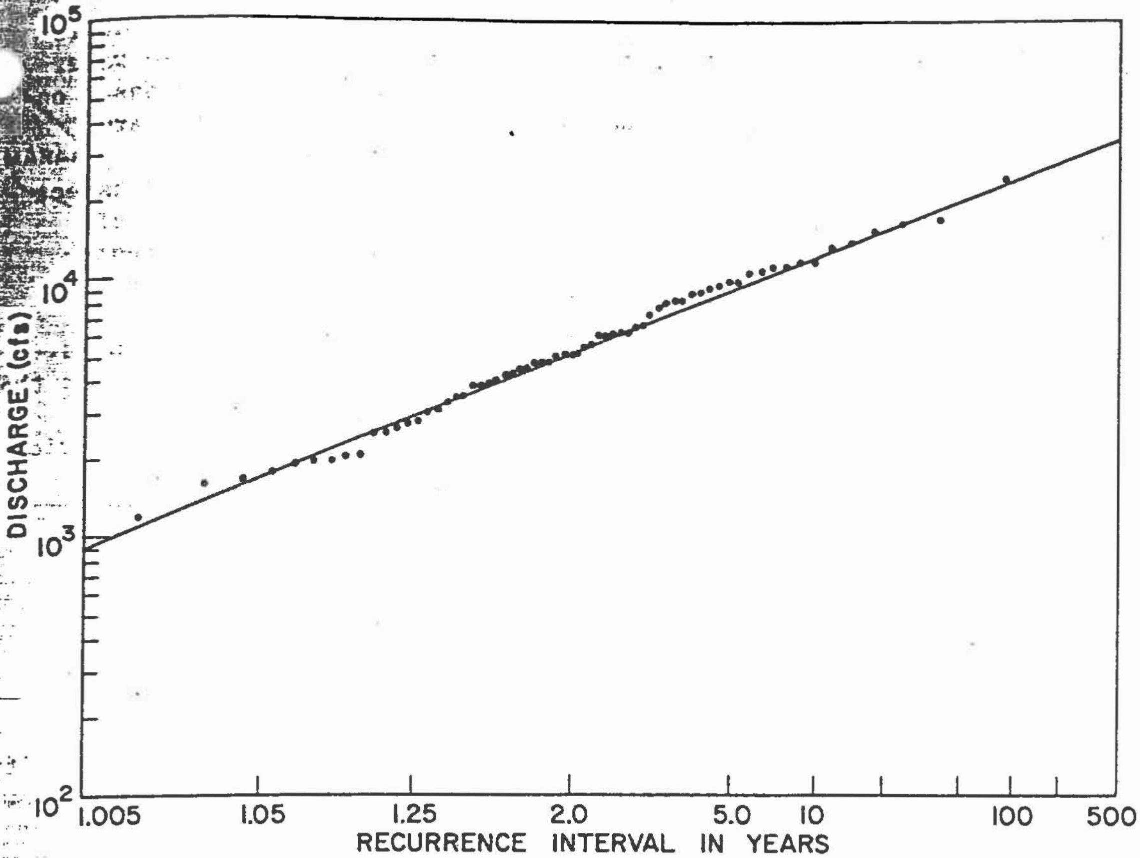


FIGURE 15 Flood-frequency analysis of the 1915-81 annual flood peaks of the Santa Cruz River at Congress Street. The plot was derived according to the nationally mandated procedure (U.S. Water Resources Council, 1981).

Most standard techniques for estimating the recurrence probabilities of flood events assume a stationary mean (U.S. Water Resources Council, 1981). This means that the methods presume a flood history has been extracted from a river whose hydrologic characteristics have not changed over the period of measurement. Clearly this assumption does not apply to the Santa Cruz flood record (Figure 14). From 1915 to 1960 no flood occurred that exceeded 340 m^3/s (12,000 cfs), but since 1960 there have been six such events (including the 1983 flood). Moreover, this change in flood behavior has occurred on a river whose channel has experienced profound morphological change since about 1890. Even more important are probable climatic factors. Note that the big floods since 1960 are predominantly fall and winter events (September through February), as they were in the period 1915 to 1929. However, the smaller annual peaks of 1916 to 1960 are nearly all summer events (July and August). The nature of the storm systems responsible for the flood peaks are clearly different during the latter period of record.

Accelerated channel change and climatic shifts are probable factors in many watersheds throughout the arid and semiarid western United

States. This invalidates the assumption of a stationary mean that is almost universally applied to flood-frequency analyses in these watersheds. It follows that this also invalidates land-use zoning based on nationally mandated flood-frequency analytical procedures that are indiscriminately applied to the arid and semiarid West.

As shown in the next section, the most important geomorphic aspect of the streams in the Tucson Basin is their tendency to alter their cross-sectional shape in response to changes in water and sediment influxes. The streams experience such rapid and large responses to changing conditions that erroneous hazard zonation can arise from conventional engineering hydraulic-hydrologic calculation procedures. For example, these procedures predict the water surface elevation of the 100-year flood in a particular stream channel by assuming that the geometry or position of the channel does not change significantly prior to, during, or after the flood. This assumption is almost always incorrect for the ephemeral stream channels in alluvial basins of semiarid regions. Indeed, the bank erosion of such streams generally presents a hazard to property that exceeds the hazard of damage from floodwater alone.

AN OVERVIEW OF FLOOD EROSION AND DEPOSITION

To assess the regional effects of the Tucson flood, a team of University of Arizona geosciences students surveyed the postflood changes in watercourses throughout the Tucson Basin (Figure 16). The detailed maps of their results are presented in Appendix B. This section briefly summarizes those observations. Because incomplete bank protection was found to be a major factor in localizing bank erosion, a separate section is devoted to that issue.

At Martinez Hill the incised meander train of the Santa Cruz River encounters a bedrock obstruction (Figure 17). Flow is deflected by the obstruction, inducing pronounced bank erosion. Despite extensive riprap revetments, two bridges were lost at this site (Figure 17). The westward meander migration along San Xavier Road was a direct result of the natural deflection of flow by Martinez Hill.

Approximately 2 km downstream of Martinez Hill, at the junction with Santa Clara (Hughes') Wash, the Santa Cruz was modified by a meander cutoff in 1935. This improvement attempted to prevent bank recession by meander migration. The 1983 flood produced extensive erosion in the vicinity of the island created by this cutoff (see Map 1, Appendix B). About 30 m of bank recession occurred on both sides of the artificial cutoff channel. Erosion upstream of the cutoff threatened a city sewer line crossing. After the flood a levee was constructed in the channel to divert flows into the original Santa Cruz meander to lessen the rate of future bank recession near the sewer line.

Downstream of Santa Clara Wash in the vicinity of Valencia Road, the character of the Santa Cruz changes from a broad shallow arroyo, about 200 to 400 m wide, to a deep narrow arroyo about 100 m wide. Bank erosion was less here because mature mesquite vegetation added stability with its deep root systems. The response of the stream through this

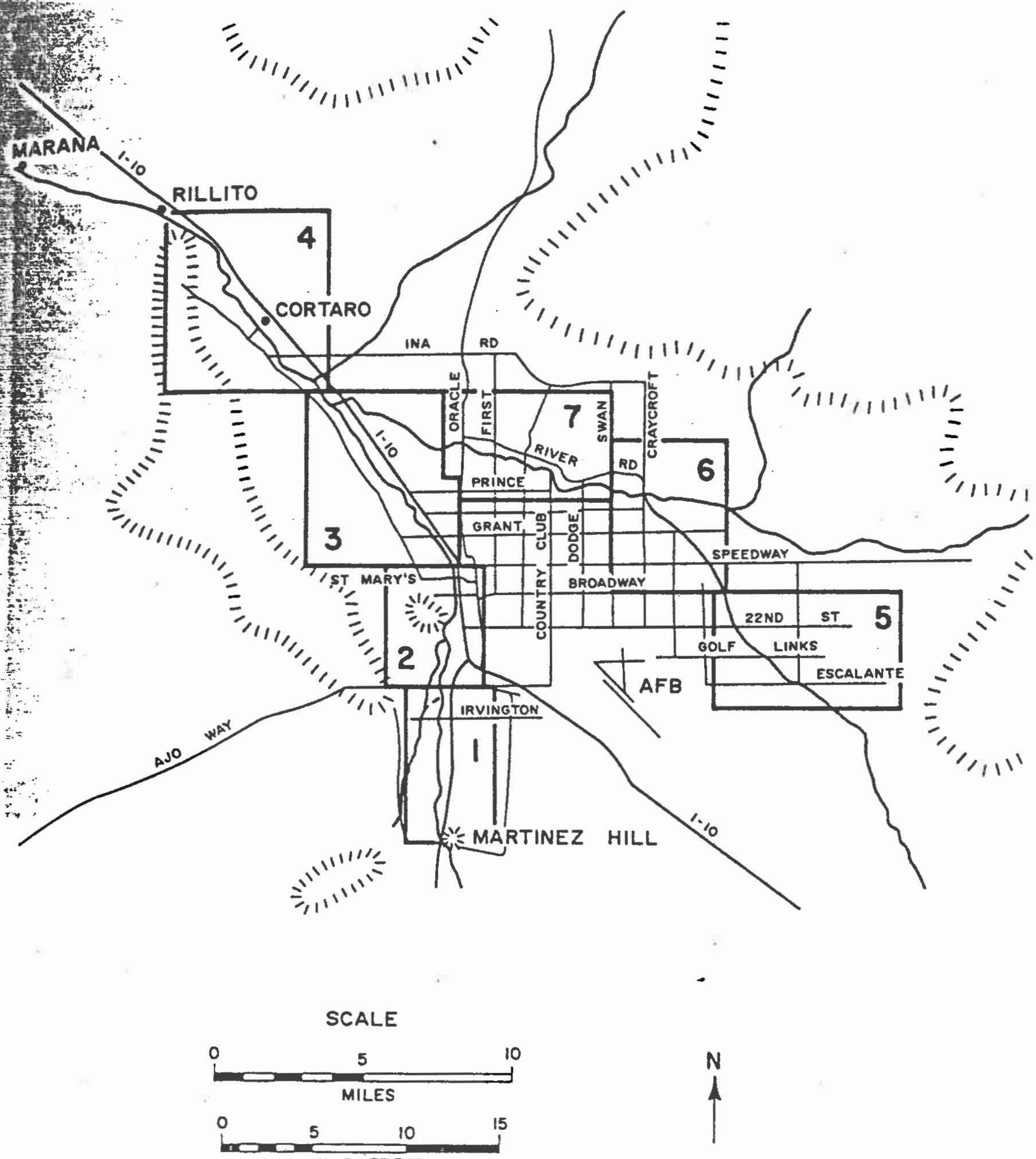


FIGURE 16 Location map of watercourses near Tucson with an index to detailed maps shown in Appendix B of erosion, bank protection, and related flood effects for the October 1983 flood.

reach was mainly by channel scour. Scour of 2 to 3 m can be documented by referring to the preflood footings of high-voltage transmission towers in the channel.

Two prominent meander bends between Valencia Road and Drexel Road illustrate important erosional effects. The cutbank of the upstream

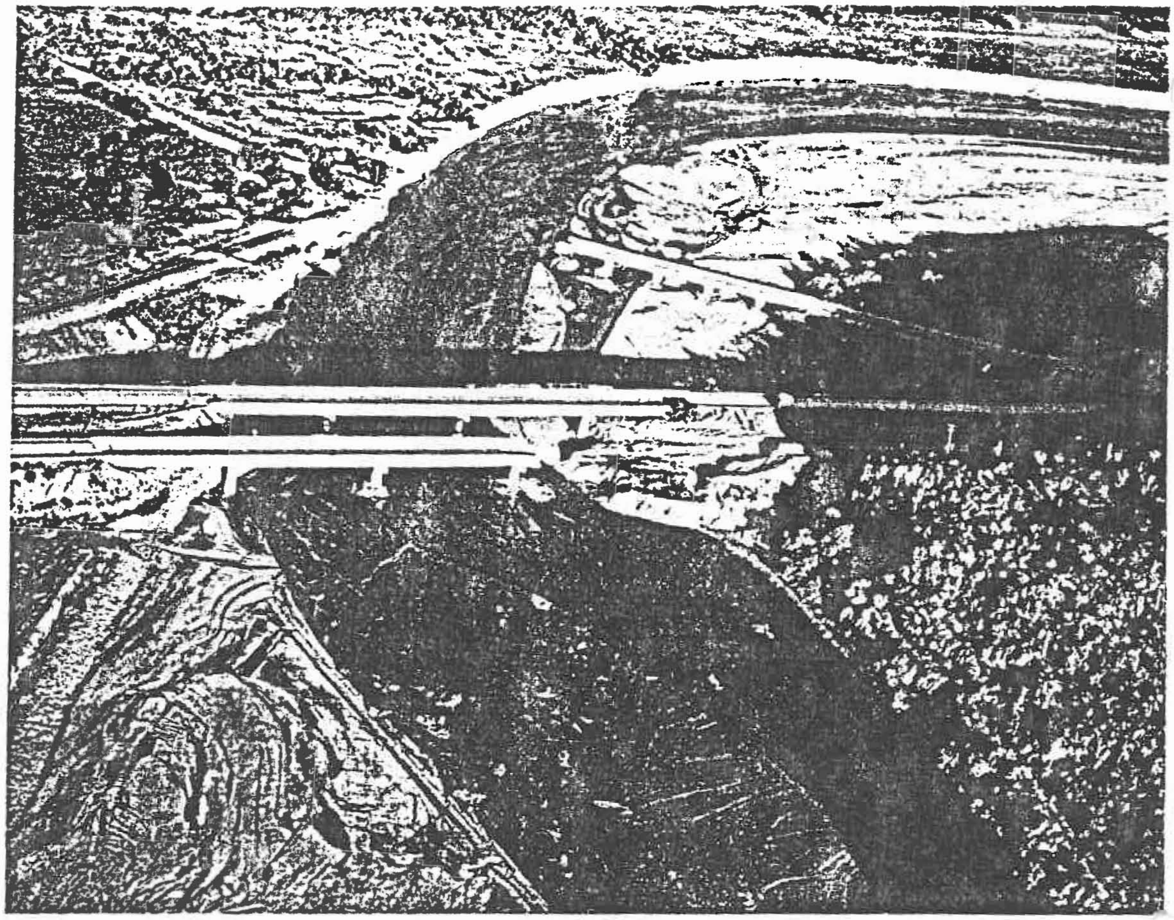


FIGURE 17 The Santa Cruz River at Martinez Hill (lower right) on October 9, 1983. Damage to the north abutment of the northbound lane bridge of Interstate 19 is visible at center. As the flow meandered to the west bank, it eroded the west abutment of the San Xavier Road bridge (top center). Bank erosion extended through the former intersection of San Xavier Road and the access road to Interstate 19 at left center. Note the blocks of collapsed bank material on this bank. Riprap placed along this bank failed to prevent meander migration in the 1983 flood. Photograph by Peter Kresan.

meander bend was protected by 100 m of riprap revetment. Nearly all of this was destroyed during the flood because bed scour undermined the revetment. Approximately 3 m of bank erosion occurred at the bend. Immediately downstream of the protected meander bend, an unprotected bend experienced approximately 40 m of bank recession on its cutbank. The increased erosion at this site is directly related to the protected banks upstream.

Approximately 400 m downstream of Irvington Road, the diversion channel of the West Branch of the Santa Cruz River joins the Santa Cruz main channel. The diversion channel was constructed to lower flood stages at the Midvale Farms subdivision, which lies immediately west of the Santa Cruz River between Drexel and Irvington roads. A concrete drop structure on the West Branch is used to control the gradient change at the confluence, and soil-cement revetments were used to protect channel banks at the confluence. The flood peak in the West Branch is estimated at 85 m^3/s (Brian Reich, City of Tucson, personal communication, 1983). Bank erosion opposite and downstream from these revetments was especially pronounced, averaging about 30 m of recession (Figure 18).

North of Ajo Way an alternating pattern of meander bend erosion continued through an unprotected reach to 29th Street (see Map 2, Appendix B). Between 29th Street and 22nd Street, former meander bends of the Santa Cruz are protected by unsorted debris and/or riprap. Bank erosion occurred at all these bends during the 1983 flood. It was most pronounced where unsorted debris was undermined by scour.

North of 22nd Street to St. Mary's Road the Santa Cruz River is nearly completely confined within artificial banks. The landfill operations near downtown Tucson (Betancourt and Turner, in press) have reduced the stream to a relatively straight channel averaging 50 m in width. This reach is channelized with soil-cement revetment on both banks. Flood discharges through this reach were confined within these artificial banks. Where protected by soil cement, banks were generally unaffected by the flood flows (Figures 19A and 19B). Confinement of flooding to a narrow cross section and reduction of sediment load in the flow because of the protected banks resulted in general bed degradation throughout this reach.

Downstream of Speedway Boulevard the Santa Cruz River channel widens and becomes more sinuous. The 1983 flood caused considerable erosion on the cutbanks of meander bends. Near Grant Road an old sanitary landfill was intercepted by bank recession, spilling refuse into the active streambed. The piecemeal bank protection in this reach showed considerable failure, in contrast with the reach of continuous bank protection lining the channel upstream of St. Mary's Road.

At Fort Lowell Road the sinuosity of the Santa Cruz decreases, and bank erosion by the 1983 flood was less severe than immediately upstream (see Map 3, Appendix B). Some of the stability of this reach can be attributed to riparian vegetation in the channel. Growth of the vegetation was facilitated by an influx of sewage effluent from a treatment plant. Shallow overbank flooding and deposition occurred in this reach.

Downstream of Ruthrauff Road the Santa Cruz showed a major change in character. The channel banks were so stabilized by vegetation that they were unable to enlarge to convey the floodflows. Floodwater spilled on to the adjacent floodplain surface. A secondary channel with a headcut developed on this surface. The width of inundation near Sunset Road was approximately 400 m (Figure 20).

Even greater spreading of the floodflows occurred downstream of the Rillito confluence (see Map 4, Appendix B). Large sand and gravel pits and the Pima County sewage treatment facility are immediately adjacent to the channel. Most of these facilities are along the east banks, with various types of revetments serving to protect that bank. Erosion from the 1983 flood was concentrated on the unprotected west banks.

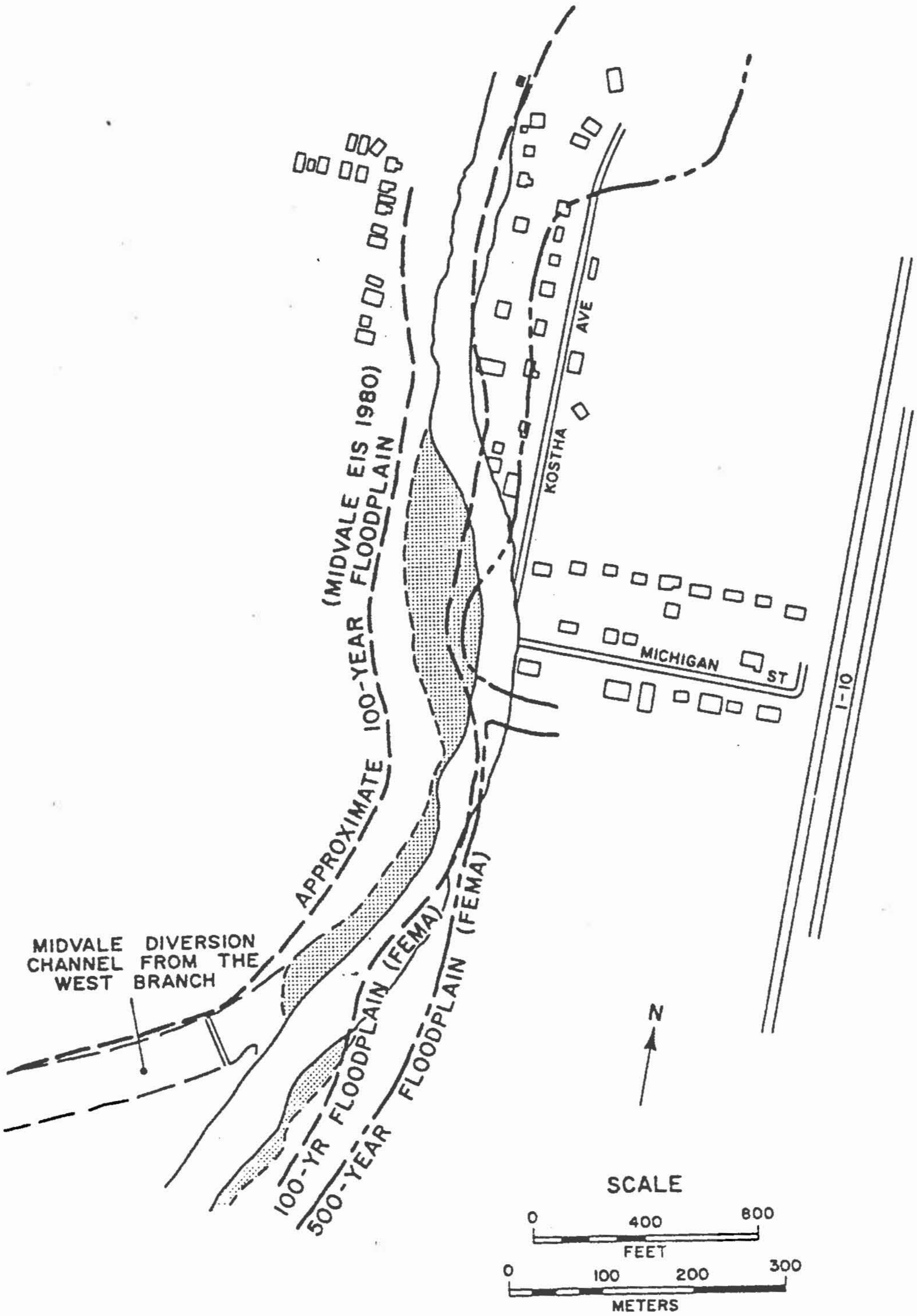


FIGURE 18 Map showing channel migration of the Santa Cruz River from within the 100-year floodplain designated by FEMA in 1982 eastward into areas not designated as hazardous from flooding. Several houses were destroyed near the junction of Kostha Avenue and Michigan Street. The stippled areas are point bars that developed in the October 1983 flood.

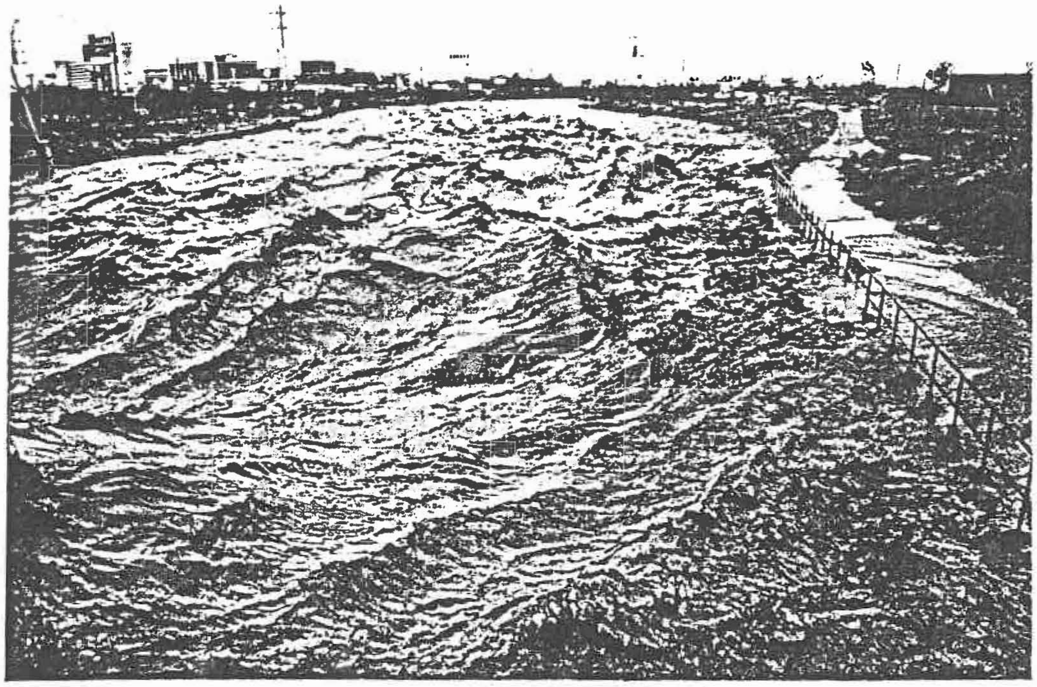


FIGURE 19A View of Santa Cruz River upstream (south) from St. Mary's Road bridge on October 2, 1983. Note confinement of flow within artificial banks composed of soil cement. Photograph by Peter Kresan.

FIGURE 19B View from approximately the same position in late October 1983. Note incision at base of soil-cement slope and meandering thalweg of low-flow channel that developed during waning flow. Photograph by Peter Kresan.

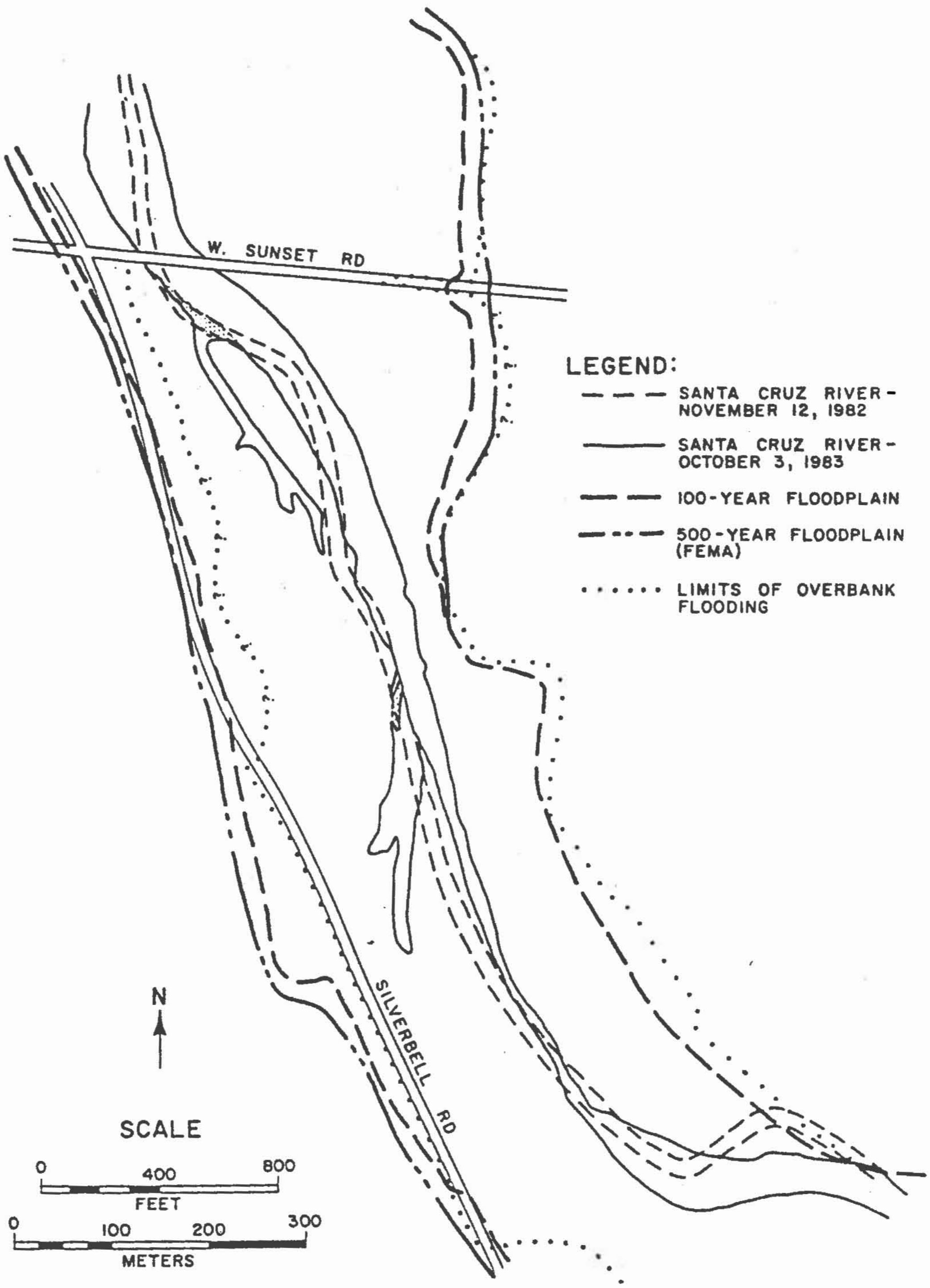


FIGURE 20 Map showing channel changes and FEMA floodplain designations along the Santa Cruz River near West Sunset Road.

At Ina Road spectacular erosion on the west abutment resulted in bank recession that exceeded the original channel width spanned by the Ina Road bridge (Figure 21). Extensive overbank flooding and deposition occurred downstream of this point. Much of Cortaro Road between the Santa Cruz and Interstate 10 was buried by overbank deposition.

A relatively sinuous reach downstream of Cortaro Road was lined with riparian vegetation maintained by sewage effluent. The 1983 flood cut a straight channel through this section and deposited thick overbank sediments to either side of the channel (Figure 22). Note that the Santa Cruz gradually changed to this behavior from an incised, eroding channel near Speedway Boulevard in which the 1983 floodflows were confined.

At Marana (Figure 23) the flooding of the Santa Cruz became a broad shallow inundation over extensive valley floors. Damages to farmland in this region are not described in this report. Significantly, the character of flooding downstream of this point was depositional (Figure 24). Sediment transported from the extensive areas of erosion upstream contributed to aggradation in this area. The aggradation, in turn, led to more extensive inundation. The buildup of sediment from past floods may have so increased slopes in this reach that a new headcut was initiated in the Santa Cruz valley near Picacho Peak (Figure 25).

The postflood erosion survey also documented the behavior of the Rillito-Pantano-Tanque Verde system during the 1983 flood. Pantano Wash was quite interesting because a large drop structure at Broadway separates the system into an incised reach downstream, to the north of Broadway, and a less incised reach upstream, to the south of Broadway (see Maps 5 and 6, Appendix B). Bank erosion from the 1983 flood was minimal upstream of the drop structure. Exceptions were at a prominent meander bend (Figure 26) and at a sand pit north of Golf Links Road.

Downstream of the Broadway drop structure, Pantano Wash is deeply incised. It responded to the 1983 flood by the alternating pattern of bank erosion described above for the Santa Cruz River system. Local areas of bank protection from riprap and wire fence revetment were effective in shifting the concentration of this erosion to unprotected banks. At the downstream end of revetment works around the Tanque Verde Road bridge, a major zone of bank recession caused the loss of residential property.

The upper reaches of Tanque Verde Creek, as with upper Pantano Wash, showed much less bank erosion in the 1983 flood than did entrenched reaches of the Rillito and the Santa Cruz River. Banks were sufficiently low that overbank flooding occurred at the Forty-Niners Country Club Estates. Major bank erosion appeared at the confluence of Sabino Canyon Creek and Tanque Verde Creek. This occurred because of (1) the angle of the stream juncture, which directed flows at an unprotected bank, and (2) changes in sediment loading that occurred as the two flows mixed. The second factor probably also explains the increased erosion that occurred immediately downstream of the confluence of Tanque Verde Creek and Pantano Wash.

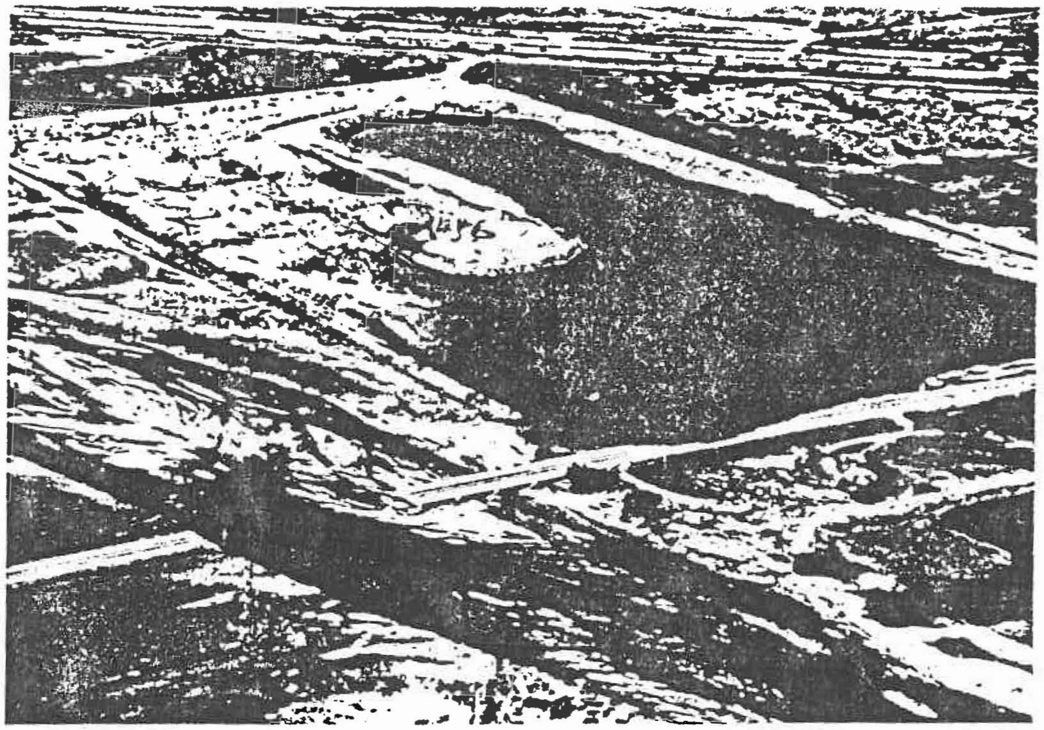


FIGURE 21 The Ina Road bridge over the Santa Cruz River on October 8, 1983, showing extensive erosion of its west abutment. Channel shifting to the west resulted in deposition at the former bridge cross section. Water spilling into the flooded gravel pit (right center) eroded large sections of the eastern approach road. Photograph by V. R. Baker.

The Rillito from Craycroft Road to the Santa Cruz River is an entrenched system. It responded to the 1983 flood by pronounced bank erosion following a pattern of meander bends, as allowed by piecemeal bank protection (see Map 7, Appendix B). From Swan Road to Dodge Road this erosion occurred in a reach that had little bank protection (Figure 27). A major bend at Prince Road resulted in migration of a meander cutbank during the 1983 flood that undermined several houses and townhomes. The bridges at Campbell Avenue and First Avenue (Figure 28) suffered erosion of their northern abutments by meander migration. Many of these effects repeated the experience of floods in 1965 and 1978 (Pearthree, 1983).

AN ASSESSMENT OF BANK PROTECTION

It is clear that unprotected banks in the Tucson area suffered phenomenal erosion during the flood of October 1983 (Figure 29). This hazard has been recognized for many years, and several types of bank protective works were in place at the time of the flood (Table 5). Thus the flood

FIGURE 22 View downstream (north) along the Santa Cruz River north of Pima Farms Road (foreground) on October 8, 1983. The sinuous bends lined with riparian vegetation developed from relatively continuous low discharges from a sewage treatment plant located approximately 5 km upstream of this point. The flood discharge carved a straighter course along the center of the low-flow meander trend. Also note the extensive sedimentation in splay patterns to either side of the central flood channel. Interstate 10 runs diagonally across the top half of the photo, and the Tortolita Mountains are visible on the skyline. Photograph by V. R. Baker.

provided an excellent test of the performance of this protection. The maps of Appendix B document the spatial distribution of this protection in relation to bank erosion.

The experience of the 1983 flood has shown that bank erosion was the most severe hazard encountered on the incised sections of stream channels through the Tucson Basin. To assess the bank protection as a factor in reducing this hazard, two questions can be asked: (1) What was the engineering performance of different bank protection designs--i.e., which designs effectively prevented bank erosion? (2) What was the overall effect of bank protection on the fluvial system? Both questions must be addressed, because piecemeal bank protection can meet the needs of some individuals in protecting their property from bank erosion while the stream system as a whole requires consideration of comprehensive bank protection.

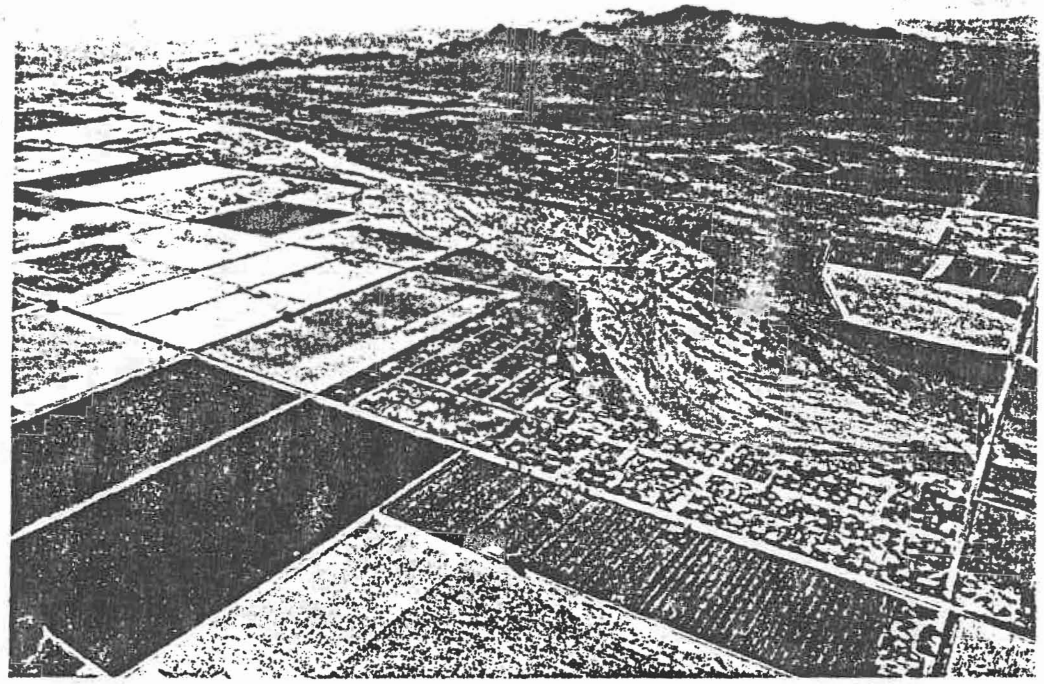


FIGURE 23 View of Marana (center) looking south toward the Tucson Mountains on October 9, 1983. Floodflows in this area spread over extensive agricultural land and caused considerable damage. The prominent meander bend at Marana experienced severe erosion during the prolonged winter flood of 1978. In contrast, the much larger 1983 flood peak resulted in relatively little erosion on this bend. Photograph by Peter Kresan.

Since 1974, the design of bank protective works in Pima County has required the approval of the Floodplain Management Section of the Pima County Department of Transportation and Flood Control District. Non-standard protective works, including cable-tied automobile bodies and rail, wire, and rock revetment, probably predate 1974 in most cases.

The local effectiveness of the three types of bank protection involving engineering design can be seen by observing the banks of the Santa Cruz River along the 12.8-km reach from Speedway Boulevard to the junction of the Rillito. Riprap revetment was used to protect 2.6 km of bank; wire-fence revetment (with rock fill) was used for 0.5 km. Damage to riprap was more extensive during the 1983 flood. Scour into the streambed is facilitated at the junction between the riprap and the unprotected bed. The rock falls into the scour hole and is either transported downstream or buried in scour holes that develop downstream of individual riprap clasts. Over 80 percent of the study reach protected by riprap showed at least some undercutting and bank recession. In contrast, the wire-fence revetment generally showed damage only on the upstream and downstream ends of protected banks. Scour at those sites caused banks to recede behind the revetments. The soil-cement

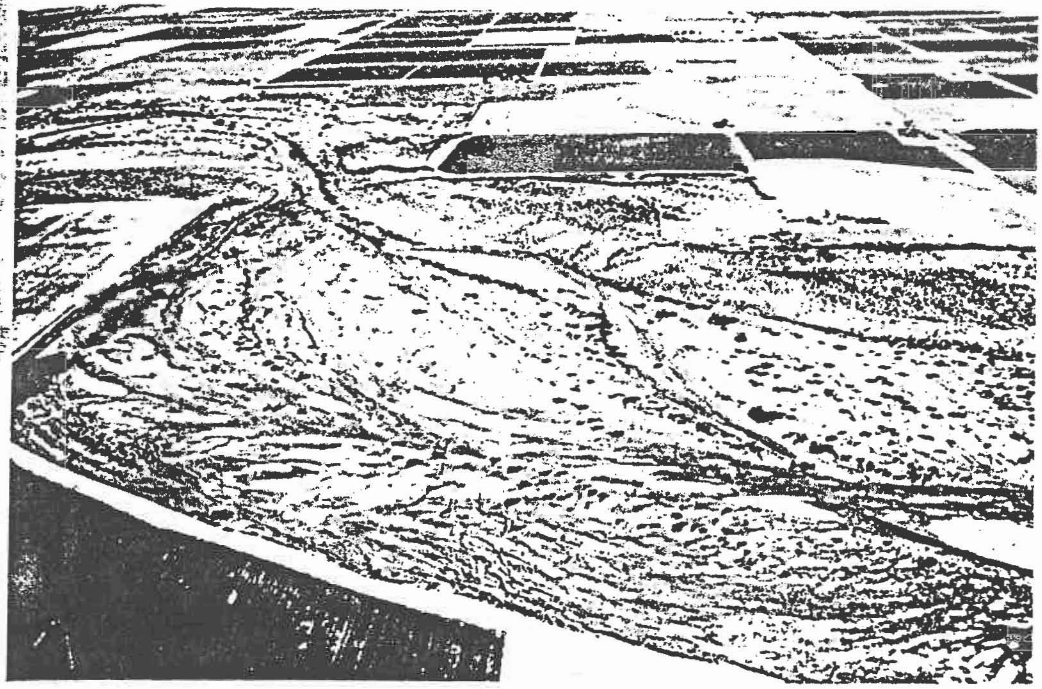


FIGURE 24 Depositional patterns on October 8, 1983, produced by the flooding of the Santa Cruz River immediately downstream of Marana. Photograph by V. R. Baker.

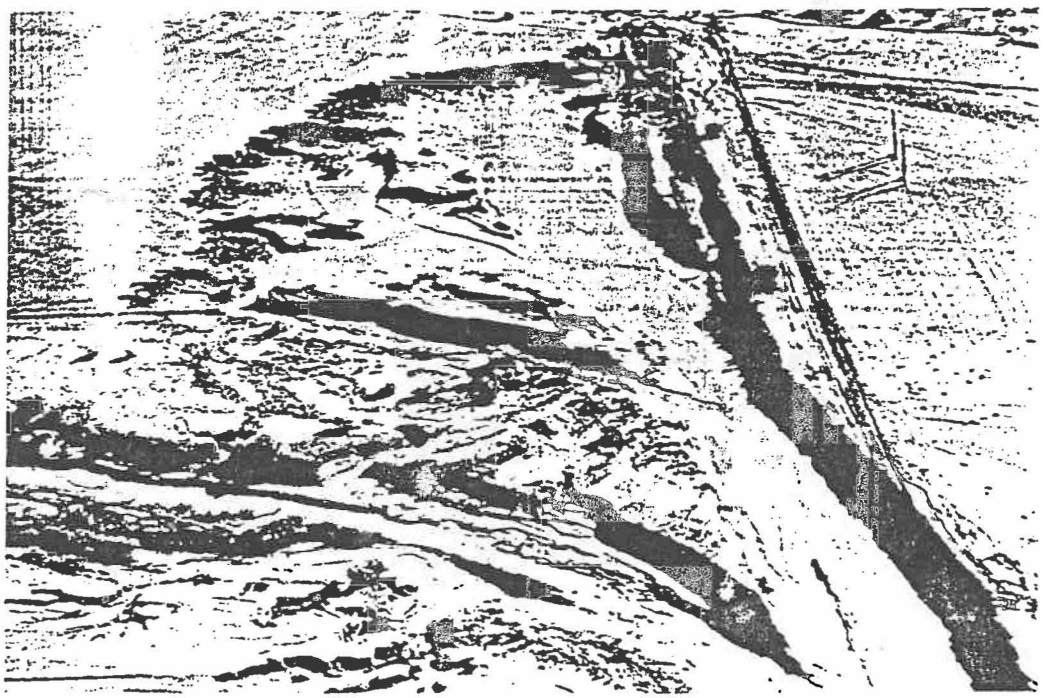


FIGURE 25 Headcut on the Santa Cruz valley floor immediately south of Picacho Peak, 15 km northwest of Marana, on October 8, 1983. Photograph by V. R. Baker.

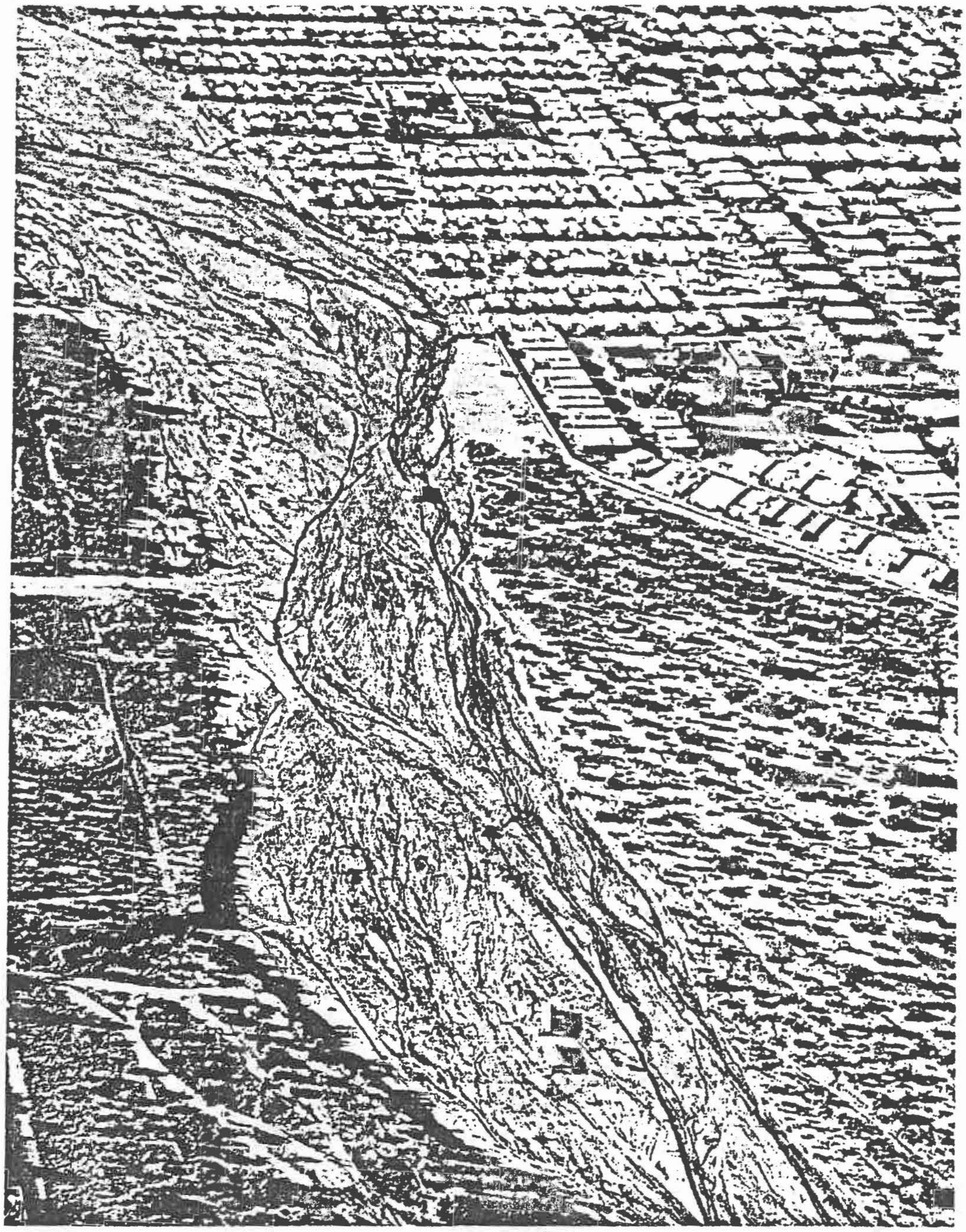


FIGURE 26 Pantano Wash at the Rincon Country mobile home development (upper right) on October 9, 1983. Escalante Road runs along the top (south) end of the photograph. The prominent bend of Pantano Wash at the top center undermined several mobile homes, which can be seen in the channel at the center and bottom of the photograph. Several automobiles were also incorporated into the northward flow of Pantano Wash. A ground view of the channel at this location is shown in Figure 38. Photograph by Peter Kresan.

FIGURE 27 The Dodge Road bridge over the Rillito on October 9, 1983. Damage to the north (left) abutment was caused by meander migration. Note the prominent cutbank on the north side and corresponding point bar on the south (right) side of the stream. The meander bend immediately downstream (bottom right) is threatening an electric utility station. The anomalously wide section of channel at the top center was the site of a preflood channel sand-mining operation. Photograph by Peter Kresan.

FIGURE 28 The Rillito at the First Avenue bridge on October 9, 1983, showing meander migration on the south (left) bank, which undermined the office building complex in the upper left of the photograph. The return flow to the opposite bank at the upper right damaged the north (right) abutment of the bridge. Temporary repair had been effected at the time of this photograph. Figures 35 and 36 show damage to the office buildings. Photograph by Peter Kresan.

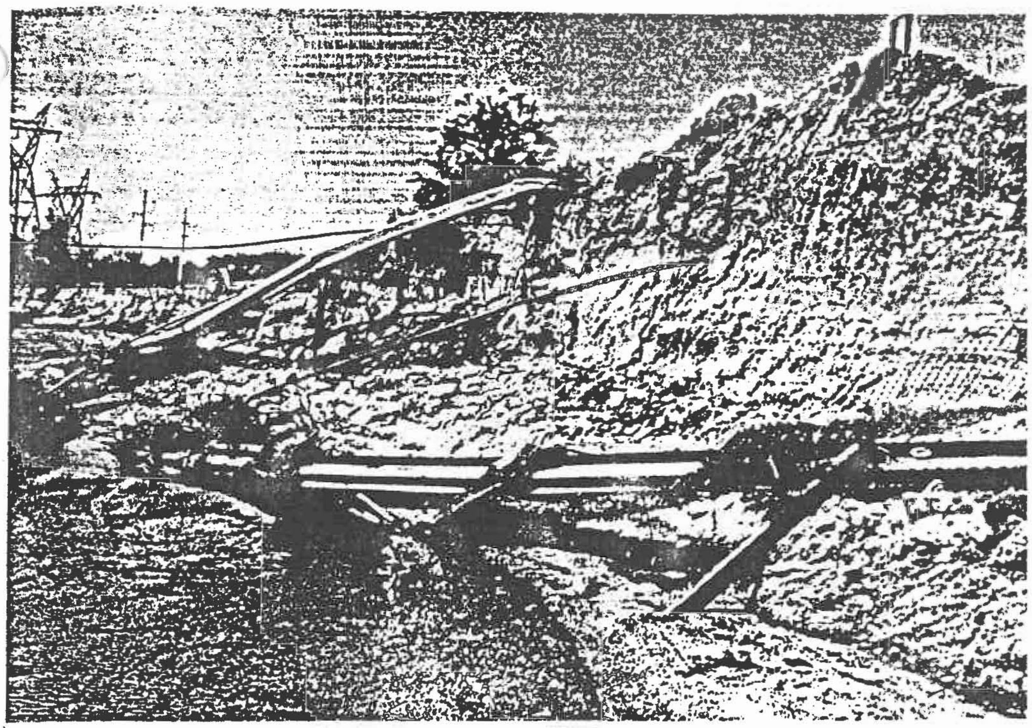


FIGURE 29 Bank erosion along the Rillito at the Dodge Road bridge on October 10, 1983. Photograph by Thomas F. Saarinen.

revetments were undamaged, although prominent erosion of banks occurred immediately downstream of the protection.

The mode of destruction for wire-fence revetments was generally not by scour, as for unconfined riprap. Rather, progressive erosion at the upstream or downstream end of the revetment would scour behind the protected bank. In extreme cases the former bank facing would be isolated in midchannel as the bank receded away from it (Figure 30).

Failure of soil-cement revetment was observed at several localities. Inadequate keying of the soil cement to the upstream or downstream terminus of the protection was the major cause of failure. At Prince Road and the Rillito, a prominent meander bend scoured behind the upstream end of a soil-cement revetment, resulting in major damage to a complex of townhouses (Figure 31).

An example of a properly keyed revetment is the protection for the southeast abutment of the Sabino Canyon Road bridge over Tanque Verde Creek (Figure 32). The pronounced channel widening upstream of the revetment terminates abruptly at this key.

Major bank erosion occurred immediately downstream of reaches that had been extensively protected with soil cement on both banks. An example is the reach of the Santa Cruz immediately downstream (north) of St. Mary's Road. Soil-cement revetments continuously line both banks of the Santa Cruz for nearly 2 km upstream of this point. Erosion appeared

TABLE 5 Types of Stream Bank Protection Observed in Watercourses of the Tucson Basin

Type of Protection	Description
Soil-cement revetment	Embankment facing composed of 8 to 15 percent portland cement mixed with natural bank material. Soil is removed, mixed with cement, and laid on the prepared bank surface in thin layers. The revetment extends to below the level of channel scour and is keyed into the banks at the upstream and downstream termini.
Wire-fence revetment	Wire enclosures held by vertical steel members (often rails) and filled with boulders. A variety of this revetment consists of wire baskets of rock called gabions.
Riprap	A blanket of boulders that exceed the competence of the largest floodflows. The material is used as facing for the bank.
Unsorted debris	Any material dumped directly on stream banks to prevent erosion. Commonly used materials are automobile bodies, concrete blocks, demolitions waste, crushed rock, poured concrete, and rubbish.

immediately at the terminus of this protection (Figures 33A and 33B). It was sufficiently intense to scour the terminus of the soil-cement bank itself (Figure 34). The postflood erosion survey (Appendix B) found that severe erosion occurred at the downstream terminus of every protected bank along deeply incised reaches of the Santa Cruz and the Rillito. Clearly, piecemeal bank protective works concentrate areas of eroded as well as protected banks, and the erosive consequences of such protection should be considered in the overall management of the river system.

The necessity of protecting bridges from loss during floods unavoidably generates a need for localized bank protection. As observed in the 1983 flood, the loss of a bridge most often occurs by bank recession at one abutment. The recession is generally associated with meander migration during the flood. To protect the bridge abutments, a reach both upstream and downstream must be lined with bank protection. The natural tendency of the stream in flood to widen its channel and incorporate added sediment load is thus impeded in the bridge reach. The stream will therefore attempt to scour the streambed at the bridge section,

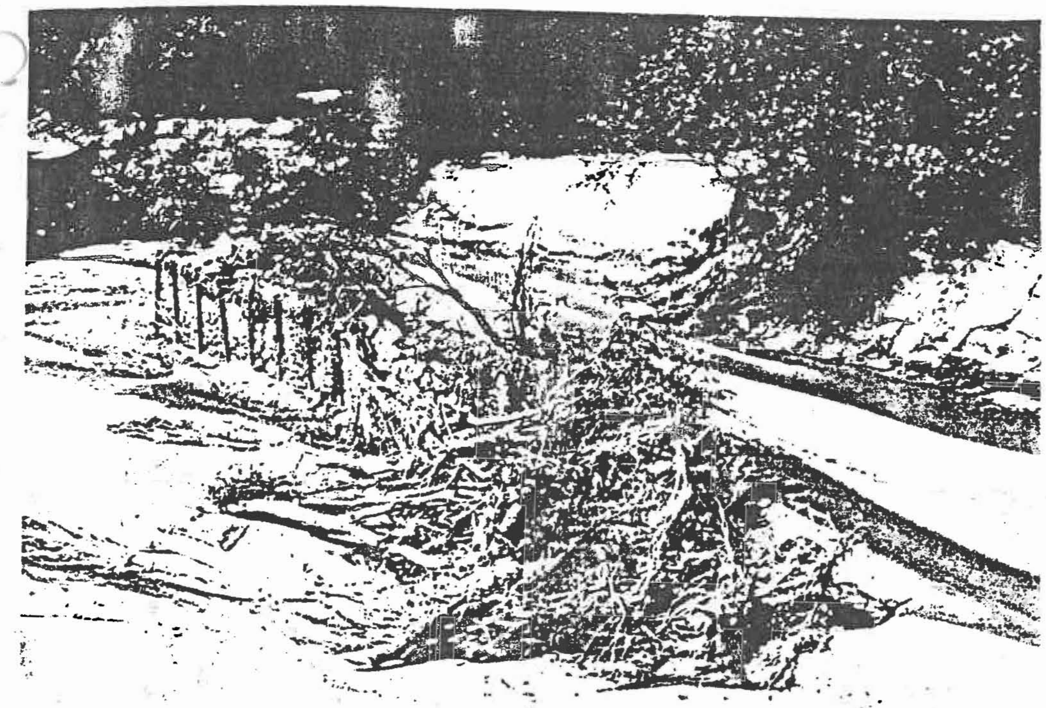


FIGURE 30 Erosion of bank protected by wire-fence revetment along the Rillito at Craycroft on October 10, 1983. Photograph by Thomas F. Saarinen.

leading to failure by an undermining of the bridge piers. If degradation control structures are placed on the bed to prevent scour, the bridge reach is transformed into a rigid-walled flume. High-velocity, sediment-impoverished floodwater passing through the protected bridge reach will be erosive in the reach immediately downstream. As in other examples, given several floodflow events, partial bank protection will beget the need for more bank protection.

SAND AND GRAVEL OPERATIONS

In southern Arizona it is a common practice to mine sand and gravel from active channels by excavating shallow pits in their beds (Bull and Scott, 1974). These operations move from place to place as local areas are depleted and flow events modify older workings. The effects of such gravel mining are extensive along the Rillito-Pantano-Tanque Verde system.

The responses of a stream to the lowering of the bed at sand and gravel pits include the following: (1) upstream bed degradation, (2) bank sloughing near the pits, (3) bank erosion caused by temporary water diversion structures used to protect mining operations, and (4) downstream bank erosion caused by the "sediment trap" action of some pits.

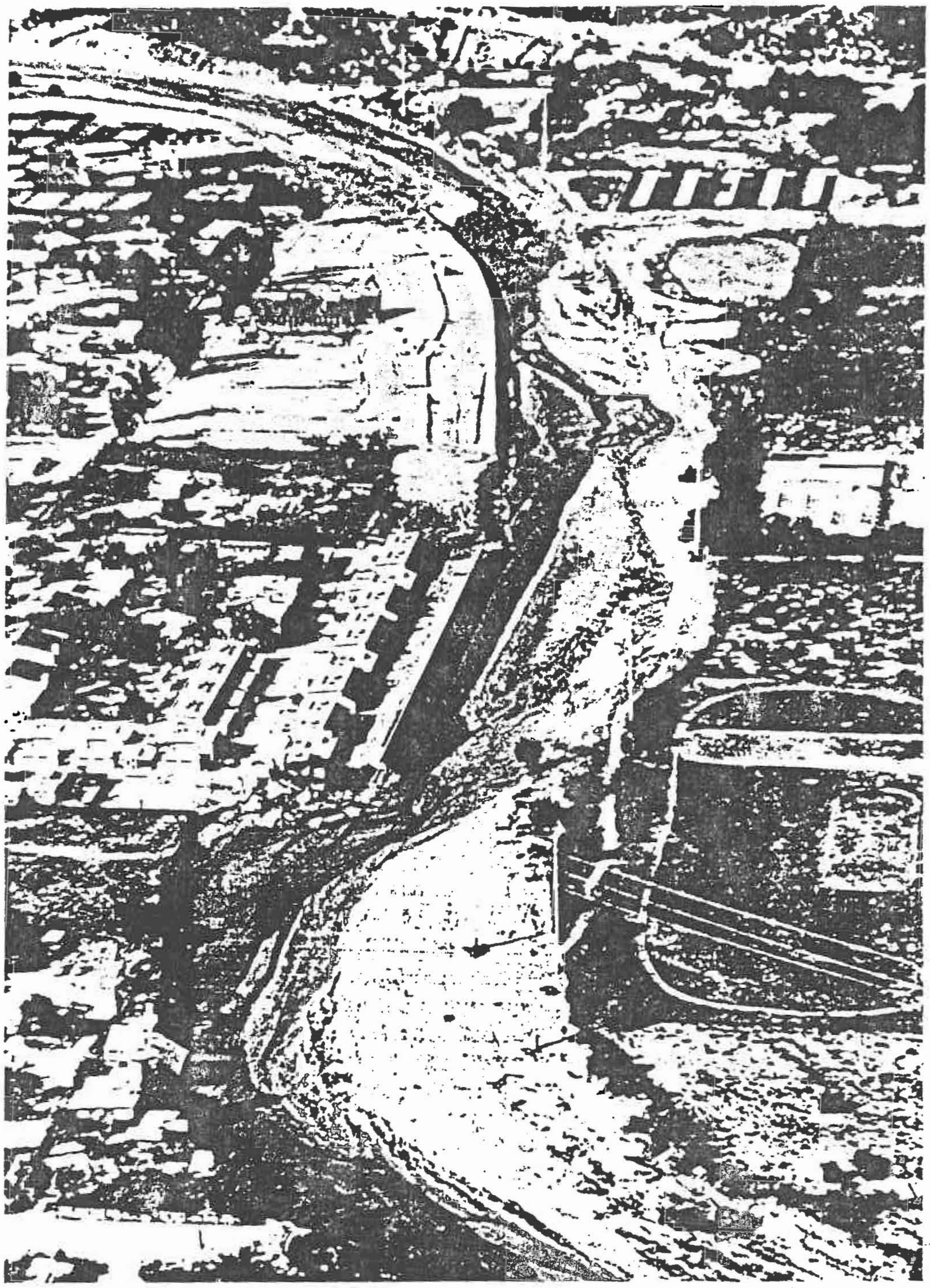


FIGURE 31 The Rillito near Prince Road (lower left) on October 8, 1983. Damage to the Pima Park Townhomes (center) occurred when the prominent meander bend at the bottom center migrated west (left) into the vacant property immediately upstream of the townhouses. This allowed erosion to occur behind the soil-cement bank protection lining the stream at the townhome property. The prominent point bar at the bottom center developed as the meander migrated westward. Photograph by V. R. Baker.

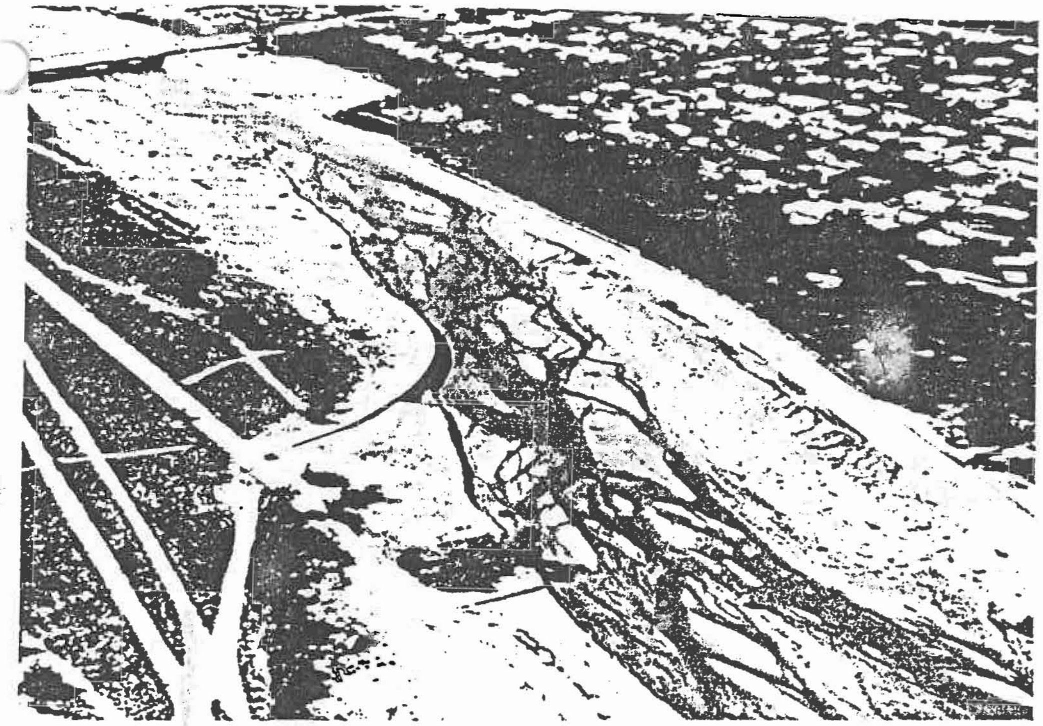


FIGURE 32 Soil-cement revetment works lining both banks of the Tanque Verde Creek immediately east of the Sabino Canyon Road bridge (top left) on October 8, 1983. Photograph by V. R. Baker.

The latter problem arises when a pit traps enough sediment entering its upstream end that the outflow from the pit is sediment impoverished and therefore erosive immediately downstream of the pit.

Along the Santa Cruz River, extensive sand and gravel pits have been developed on the surface of the valley floor immediately adjacent to the entrenched channel of the river. At Ina Road a major pit contributed to the undermining of the approach road to the Santa Cruz bridge (Figure 21). A large abandoned sand pit on the Rillito occurs midway between Swan Road and Dodge Road (see Map 7 in Appendix B). The pronounced meander bend erosion downstream of this point, including the erosion of the north abutment of the Dodge Road bridge, may be at least partly related to this sand pit (Figure 27).

The specific changes in the river system described above for the Tucson flood of October 1983 are in keeping with the general long-term characteristics of southwestern streams, as outlined in the next section.

STREAM CHANNEL STABILITY IN THE TUCSON BASIN

Reports of the U.S. Geological Survey in the 1890s note that while runoff in Arizona is very low, at times floods occur whose violence and duration are phenomenal. This is still true. It is also true that no stream flowing north to the Gila River in Arizona or New Mexico has a permanent discharge at its confluence with the main channel.

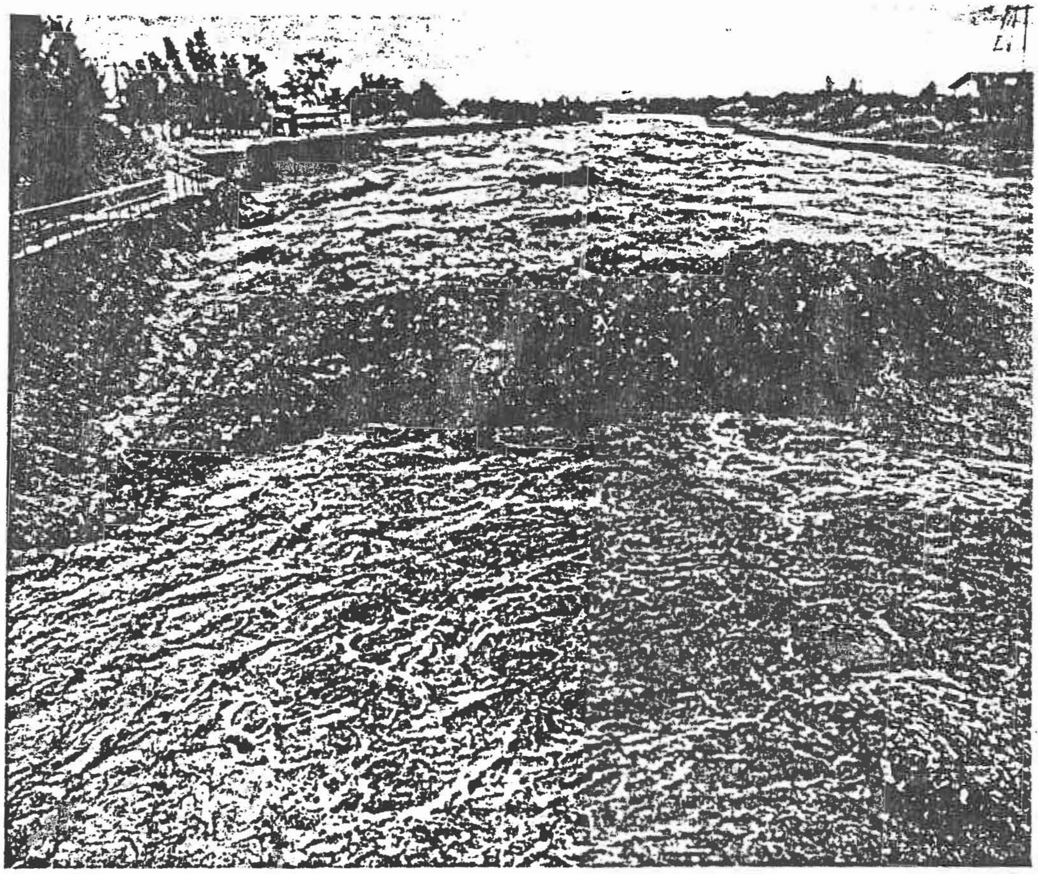


FIGURE 33A View downstream (north) along the Santa Cruz River from the St. Mary's Road bridge on October 2, 1983. Note that the soil-cement banks confine the flow for approximately 100 m beyond the bridge. Pronounced bank erosion occurs at the distal end of this protection, especially on the west (left) bank, toward which the flow is directed. The high-velocity thread of the flow is marked by antidunes and standing waves. Photograph by Peter Kresan.

West of the San Pedro River, even floodflows rarely reach the Gila River. East of the San Pedro River, streams frequently flow to the main stream. Because the transported sediment has to be deposited as discharge declines, western streams are aggrading, but irregularly so. Water spreads widely among areas of deposition and flows over relatively small intermittent channels. Head cutting may be present at high discharges. Under pristine conditions of a high water table, some local permanent water may occur.

Upstream from areas of deposition, river channels are well defined, but again the characteristics of the channel are determined by the discharges of water and sediment. Under natural conditions upstream from the junction of the Santa Cruz and Rillito, the channels were

FIGURE 33B View from approximately the same position on October 16, 1983. Note that the west bank has receded further from its position on October 2. Damage to the distal end of the soil-cement bank protection on the east (right) bank is shown in more detail in Figure 34. Photograph by Peter Kresan.

alternate reaches of incision and deposition, depending on the relative inflow of water and sediment from tributary streams. On a larger scale this is equivalent to the discontinuous gully system.

The stability of this system, such as it is, depends on a supply of sediment that increases with increasing discharge, on vegetative protection of stream banks, or on valley fill. Because the various tributaries to a stream system do not have equal ratios of transported sediment to water discharge, the stability of a system will vary in different reaches of the stream. Also, because sediment transport increases with increasing intensity of rainfall, the ratio between water and sediment varies seasonally. Because sediment moves as slugs, the ratio of water to sediment decreases with succeeding years of high discharge. Eventually the duration of above-normal discharges probably exerts more influence on stream behavior than do peak flows.

The City of Tucson has been built on this unstable system. The

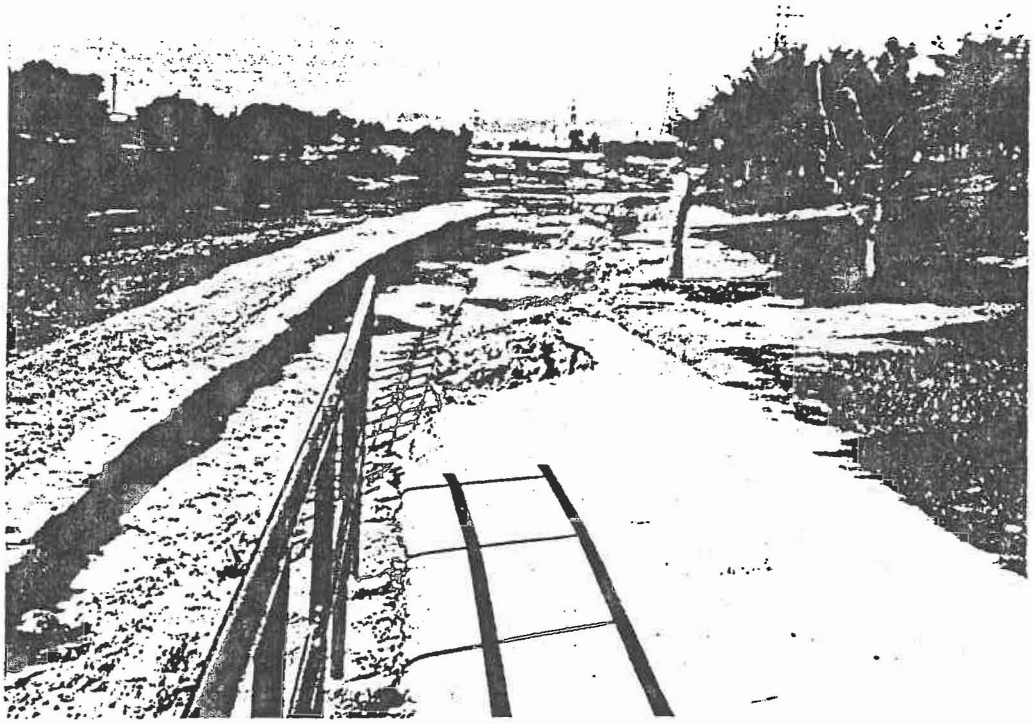


FIGURE 34 View downstream (north) along the Santa Cruz River between the St. Mary's Road bridge and the Speedway Boulevard bridge (visible at top center) on October 10, 1983. The distal terminus of the soil-cement bank protection has been eroded by the flows shown in Figure 33A. Photograph by Thomas F. Saarinen.

increasing demand for water by this growing city can be met only by tapping groundwater in larger and larger amounts. The result has been a lowering of water tables and the disappearance of streamflow at low rates. Downstream of the city, sewage effluent has increased and currently flows as far as Red Rock. The result has been the almost complete disappearance of riparian vegetation within the city. Except where controlled by structures, the streams have become incised, perhaps by as much as 6 ft, over the last 50 years. Cook and Reeves (1976) and Betancourt and Turner (in press) have documented the effects of man's activities on the Santa Cruz River through Tucson.

The city and its built-up environs now cover over 100 square miles. As a result, storm runoff with a low concentration of sediment has increased. Over the last six years there have been more than average winter floods, each finding smaller amounts of sediment to move. Normally, degradation of the main stream increases the sediment discharge from tributary streams, but the construction of roads and other developments paralleling the river has prevented this.

The result has been increasing bank erosion at different points on the streams. If, over a period of the next few years, winter runoff declines while summer thunderstorms increase, the amount of sediment available for transport in winter storms will increase, improving stream stability. If not, bank erosion associated with channel meandering is to be expected. Over time, however, the incised streams in the Tucson Basin will probably become wider and deeper.

The widespread flooding below the junction of the Santa Cruz and Rillito was caused by the magnitude and duration of the stream discharge during the October 1983 flood. While such flooding could be controlled by levees, it is questionable if this is desirable. Control of flooding would push the locus of spreading downstream and could possibly cause incision of the channels. Levees might also interfere with the use of sewage effluent for irrigation. Most important, levees would reduce the area of groundwater recharge in the area of heaviest pumping.

Whatever is done, it should be realized that an area of deposition will constantly grow and that it is basically unstable from either small or large floods.

Because of the expanding developed area in the Tucson Basin, there is no reason to presume that the present problems of bank stability can be reversed naturally. As long as sediment transport is small in terms of discharge and stream gradient, there will be problems of bank erosion. Many reaches of the stream have been protected, at least on one side, by different types of revetments. Most have worked reasonably well. But, in all locations noted, excessive bank erosion has resulted downstream of the revetted area. Bridges pin the location of the stream and limit the possible variation in bank position and river slope. If the stream is fully revetted, the stream can be expected to degrade its channel in some areas and thus undermine the revetments.

No program for the prompt construction of channel revetments through the built-up area can be expected. Such construction will likely be on a piecemeal basis and simply shift the areas of intense erosion. Without bank protection, erosion will continue until a new equilibrium is reached. The state of the art cannot predict what such an equilibrium will be.

One major grade control structure has been built on Pantano Wash. Theoretically, a reduction in slope should result in a narrower channel. The behavior of this stream should be closely observed, because its sediment transport is reduced when the stream flows into pits excavated for sand and gravel. This should result in a wider channel downstream and might cause degradation below the structure.

Little information is available about the hydraulics of the rivers in the Tucson Basin. There are no sediment sampling programs and few if any actual discharge measurements at high flows, and the county has abandoned its regular measurement of stream discharge. Because any available method of calculating discharge at high flows has a probable error of ±40 percent, the effectiveness of any design is uncertain.

This situation is not unique to the Tucson Basin. It exists everywhere. John F. Kennedy's 1983 paper, "Reflections on Rivers, Research, and Rouse," should be required reading for anyone working with fluvial hydraulics.

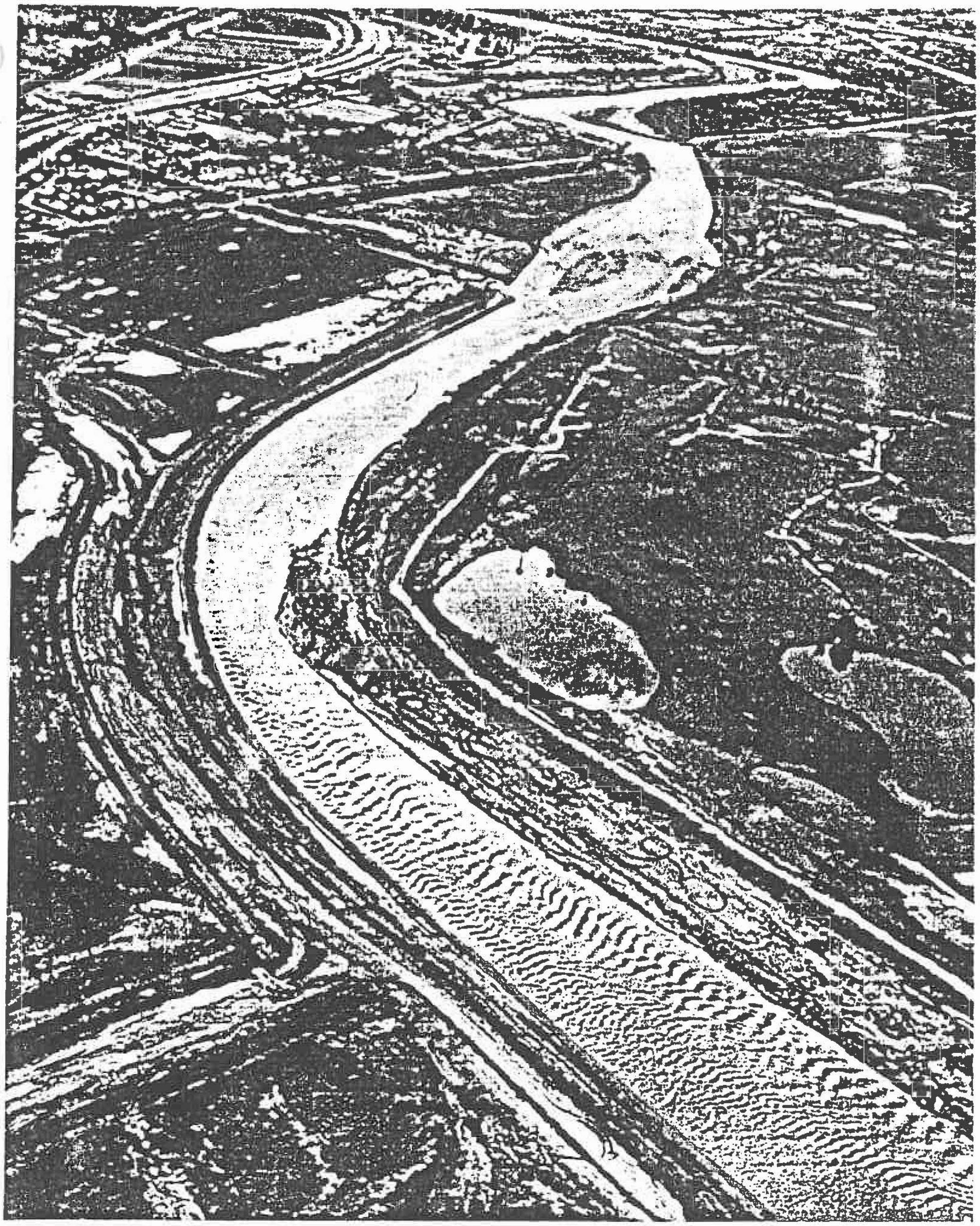
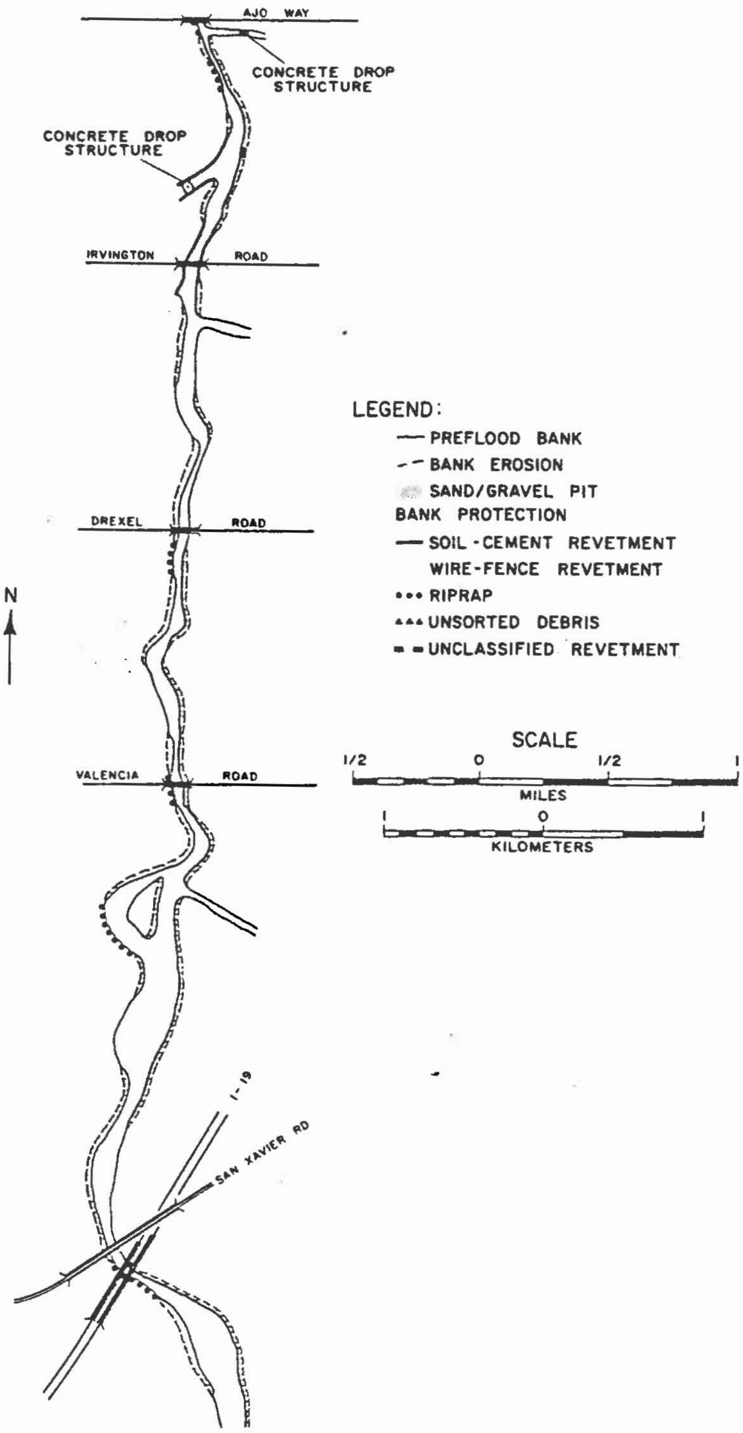


FIGURE 43 Aerial view of Santa Cruz River showing the sudden widening of the stream banks at a point with no bank protection. Photograph by Peter Kresan.

runoff and become, in effect, tributaries of major streams when enough rain falls. Each year after a major storm, pictures appear in the paper joking about how people adapt to these conditions by pulling out their water skis, inner tubes, or boats (Figure 45).



MAP 1 The Santa Cruz River from Martinez Hill to Ajo Way.

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SECTION 920

SOIL-CEMENT FOR BANK PROTECTION, LININGS AND GRADE CONTROL STRUCTURES

920-1 DESCRIPTION

The work under this section shall consist of furnishing all materials and constructing soil-cement bank protection, linings and grade control structures at the locations and in accordance with the details shown on the plans and the requirements of the specifications. The work shall include toe trench excavation, backfill, and dewatering.

920-2 MATERIALS

920-2.01 Portland Cement. Portland cement shall conform to the requirements of ASTM C 150 for Type II.

920-2.02 Fly Ash. Fly ash shall conform to the requirements of Subsection 1006-2.04 (D).

920-2.03 Water. Water shall conform to the requirements of Subsection 1006-2.02.

920-2.04 Aggregate. The soil used in the soil-cement mix shall not contain any material retained on a one and one-half (1-1/2) inch sieve, nor any deleterious material. Soil for soil-cement lining shall be obtained from the required excavations or from borrow areas specified on the plans or approved by the Engineer and stockpiled on the job site as specified herein. The Contractor shall be responsible for providing all the soil required for soil-cement production. Soil, meeting the gradation requirements specified herein, shall be obtained from the project excavations and/or from borrow areas specified on the plans or obtained by the Contractor. Soil suitable for soil-cement shall be obtained from the required excavations, as shown on the project plans, before any borrow material is allowed to be used for this purpose. The actual soil to be incorporated into the soil-cement shall be analyzed by laboratory tests in order to determine the mix proportions to be used on the project. The distribution and gradation of materials in the soil-cement lining shall not result in lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material. Soil shall conform to the following gradation:

Sieve Size	Percent Passing (Dry Weight)
1-1/2"	98% - 100%
#4	60% - 90%
#200	5% - 15%

The Plasticity Index shall be a maximum of 3.

Clay and silt lumps larger than 1/2-inch shall be unacceptable, and screening, in addition to that previously specified, will be required whenever this type of material is encountered.
920-2.05 Design Procedures. The design requirements for the soil-cement shall be such that it has the compressive strength of 750 psi unless otherwise specified in the special provisions at the end of seven days. A compressive strength test will be run after 24 hours to monitor the mix proportions on a daily basis. Twenty-four hour compressive strength results for samples moist cured typically range between 50 and 60 percent of the compressive strength of seven day samples moist cured for six days and soaked in water for 24 hours. Once the mix proportions have been established, compressive strength tests will be run after 24 hours during production. The results of the 24 hour test will be used to monitor the daily output of the central plant. Final acceptance, however, shall be based on the compressive strength results of the seven (7) day samples. The quantity of cementitious material established through the mix design procedures shall be monitored throughout the life of the project with modifications, as required, to account for varying field conditions.

The cementitious portion of the soil-cement mix shall consist of one of the following alternates:

(1) One hundred percent portland cement.

(2) Eighty-five (85) percent portland cement and fifteen (15) percent fly ash.

When the option to incorporate fly ash into the cementitious portion of the mix is used, fly ash shall replace portland cement based on a ratio of one pound of fly ash added for each pound of portland cement removed.

The Contractor shall use the soil aggregate, cementitious material content, and moisture content determined by the Engineer, in accordance with laboratory tests and based on the cementitious material composition selected by the Contractor. The Contractor shall allow a minimum of eight (8) days for the determination of the content of cementitious material. During the course of the work, the Engineer may adjust the mix proportions, whenever necessary, in order to assure that the required minimum compressive strength is being attained, but not excessively exceeded. Changes in the amount of cementitious material required in the soil cement mixture shall be made promptly by the Contractor when so directed by the Engineer.

For bidding purposes only, the estimated content of cementitious material in the soil-cement mix shall be as specified in the special provisions.

The percent of cementitious material to be used in the mix shall be calculated by dividing the total weight of cementitious material by the total weight of the dry compacted soil-cement. The actual mix proportions used on the project shall be determined from laboratory tests on the in-situ aggregate stockpiled for the project. Testing shall be conducted in accordance with the Agency's test procedure, "Determination of Cement Content Required for Soil-Cement Mixtures" a modification of Arizona Test Method 220.

920-3 CONSTRUCTION DETAILS

920-3.01 Equipment. Soil-cement bank protection, linings or grade control structures may be constructed with any combination of machines and/or equipment, except as noted herein, that will produce a completed soil-cement lining or structure meeting the requirements for soil pulverization, cementitious material content and moisture content, mixing, transporting, placing, compacting, finishing, and curing as provided in these specifications.

920-3.02 Required Contractor Submittals. Prior to the start of construction, the Contractor shall submit, in writing for approval, the following items:

(1) The approximate length of soil cement bank protection or area of soil cement lining to be placed prior to starting compaction operations.

(2) The type of compaction equipment to be used.

(3) The number and type of watering equipment to be used.

(4) The method used to keep surfaces continuously moist until subsequent layers of soil cement are placed.

(5) The method used to cure permanently exposed surfaces.

(6) The proposed source of soil, if other than required excavations.

(7) The proposed source(s) of portland cement and fly ash, if used.

Such approval shall not relieve the Contractor of the responsibility for achieving the desired result of constructing sound soil cement, free from defects, according to the specifications and plans, or as directed by the Engineer. Changes in the source(s) of cement or fly ash will not be permitted without the prior approval of the Engineer.

920-3.03 Preparation. Before soil-cement processing begins, the area to be lined shall be graded and shaped to the lines and grades shown on the plans or as directed by the Engineer. The subgrade shall be compacted to a minimum of ninety percent (90%) of the maximum density as determined in accordance with the requirements of ASTM D 698 (A, C, or D) and D 1556 or D 2167.

Immediately prior to placement of the soil-cement mixture, the underlying material shall be moistened if necessary. Soft or yielding native material shall be corrected and made stable in accordance with the requirements of Subsection 203-5.03(A).

920-3.04 Stockpiling of Aggregate. The aggregate stockpile(s) shall be constructed on level, firm ground free of brush, trees, stumps, roots, rubbish, debris and other objectionable or deleterious material and shall be located so as to provide a distance of not less than fifteen (15) feet from the outside bottom edge of any other existing stockpile. The stockpile shall be constructed in layers; each layer not exceeding two (2) feet in thickness. Ramps formed for stockpile construction shall be of the same material as that being stockpiled, and will be considered a part of the stockpile. Before steepening a ramp, any contaminated surface material shall be removed.

Stockpiled material should be thoroughly mixed throughout its depth, width, and length before utilization. The material shall be homogeneous and uniform in color, gradation and moisture throughout.

The Contractor shall be solely responsible for the construction of the stock pile(s), including monitoring for quality and uniformity of the material placed therein. To assure conformance with the requirements of this Subsection and Subsection 920-2.04, the Engineer, at random intervals during construction of the stockpile, will sample and test the material being placed therein. Results from these tests shall be compared against the test results obtained by the Contractor to assure compliance with the requirements of Subsection 920-2.04.

Unless otherwise approved by the Engineer, final sampling for acceptance of the soil aggregate will be conducted after the required quantity of soil aggregate, necessary to complete soil-cement placement on the project, has been stockpiled. After a stockpile has been sampled and accepted, no material will be added to it without the approval of the Engineer.

Stockpiled material shall also conform to the applicable requirements found in Section 106 and Subsection 1006-2.06.

The stockpile(s) shall be completed and approved at least eight (8) days prior to the start of soil-cement production, so as to provide sufficient time for verification of the mix proportions for the project.

The Contractor may have to blend dissimilar soils in order to maintain the optimum soil properties and gradation specified in Subsection 920-2.04, and thus minimize overruns of cementitious material. Blending shall require constructing separate stockpiles for materials to be blended and shall be performed by the utilization of separate storage feed bins at the plant, to the satisfaction of the Engineer.

920-3.05 Blending of Cement and Fly Ash. The blending procedure shall be sufficient to provide a uniform, thorough, and consistent blend of cement and fly ash. The blending method and operation shall be approved by the Engineer prior to the commencement of soil-cement production. In the blending of the cementitious materials, the percent of fly ash content shall not vary by more than $\pm$ 0.50 percent of the contents specified by the Engineer.

Weighing or volumetric measuring devices are required at both the cement and fly ash feeds. At the direction of the Engineer, an additional measuring device may also be required when the cement and fly ash are pre-blended at the site. In the production of soil cement, the percent of cementitious material shall not vary by more than $\pm$ 0.3 percent of the contents specified by the Engineer.

Silos and feeders shall be equipped and operated so as to provide uniform rates of feed and prevent caking. Provisions shall be made to allow for ready sampling of the cementitious material(s).

920-3.06 Mixing Plant. Soil-cement shall be mixed in an approved central-type plant having a stationary twin shaft pugmill mixer of the continuous-mixing type or an approved batch-type pugmill. The mixing plant shall be designed, coordinated and operated to produce a soil-cement mixture of the proportions specified within the required tolerances. The plant shall be equipped with positive means for controlling and maintaining a constant time of mixing. Twin shaft pugmills shall also be equipped with a positive means for maintaining a constant speed of rotation of the shafts. The plant shall be equipped with screening, feeding, and weighing, metering or volumetric measuring devices that will add the soil, cementitious material(s) and water into the mixer in the specified quantities. The blades of twin shaft continuous pugmill mixers shall be adjustable for angular position on the shaft and reversible to retard the flow of the mix.

When the quantity of water is controlled by metering, provisions shall be made by the Contractor whereby the quantity of water delivered through the meter can be readily converted to weight. A water storage tank may be required to prevent the adverse effects created by surge drawdown.

The soil aggregate feed rate shall be controlled by a variable speed belt or a remotely operated gate, calibrated to accurately deliver any specified quantity of material. The feed rate shall be readily adjustable from the control panel to compensate for changes in the moisture content of the soil or to change soil aggregate proportions when blending is required and separate bins are utilized. The combined aggregate belt feeding the mixer shall be equipped with an approved belt scale. The belt scale shall operate automatic controls which will govern the proportions of cementitious material and water as ratios of the total soil aggregate, with provisions for ready changing of the proportions.

When a continuous mixing plant with a fixed soil aggregate feed rate system is used, the belt shall travel at a constant speed. The feed system shall continuously deliver aggregate to the mixer at a constant feed rate, calculated on a dry weight basis, at any locked gate setting. The feed system shall be mechanically interlocked with all other feed devices. The soil aggregate feed monitoring system shall provide the rate of and total quantity of soil aggregate fed into the mixture.

The plant shall be equipped with a hydraulically or mechanically operated discharge holding bin having a minimum capacity of twenty (20) tons.

Mixing shall be sufficient to secure a homogeneous, intimate, uniform mixture of the soil, cement, fly ash, and water within the specified tolerances. Soil and cementitious material shall be mixed sufficiently to prevent cementitious balls from forming when water is added.

Mixing shall not proceed when the soil aggregate or the area on which the soil-cement is to be placed is frozen. Soil-cement shall not be mixed or placed when the air temperature is below 45°F (7°C), unless the air tempera- ture is at least 40°F (5°C) and rising.

Free and safe access to the plant must be provided to the Engineer at all times for inspection of the plant's operation, and for sampling the soil- cement mixture and its components.

920-3.07 **Measuring Devices.** Weighing, metering or volumetric measuring devices specified in Subsection 920-3.05 and 920-3.06 shall record the quantity of the material, and shall have a digital readout such that the total discharged quantity per hour and the cumulative total quantity are displayed.

Measuring devices shall be calibrated, at the Contractor's expense and approved by the Engineer.

Each measuring device shall be calibrated to an accuracy of plus/minus two (2) percent and shall be inspected and calibrated as often as the Engineer deems necessary to assure their accuracy.

920-3.08 **Required Moisture.** The moisture content of the mix shall be adjusted as needed to achieve the compressive strength and compaction requirements specified herein.

920-3.09 **Handling.** The soil-cement mixture shall be transported from the mixing area to the embankment in clean equipment. Hauling equipment shall be outfitted with suitable covers to protect the mixture in unfavorable weather. The total elapsed time between the addition of water to the mixture and the start of compaction shall be the minimum possible. In no case should the total elapsed time exceed thirty (30) minutes. This time may be reduced by the Engineer when the air temperature exceeds 90°F, or when there is a breeze or wind which promotes rapid drying of the soil-cement mixture.

920-3.10 **Placing.** The mixture shall be placed on the moistened subgrade embankment, or previously completed soil-cement with spreading equipment that will produce layers of such widths and thicknesses as are necessary for compaction to the required dimensions of the completed soil-cement layers. The compacted layers of soil-cement shall not exceed eight (8) inches, nor be less than four (4) inches in thickness.

Each successive layer shall be placed as soon as practicable after the preceding layer is completed and certified.

All soil-cement surfaces that will be in contact with succeeding layers of soil-cement shall be kept continuously moist by fog spraying until placement of the subsequent layer, provided that the Contractor will not be required to keep such surfaces continuously moist for a period longer than seven (7) days.

The Contractor shall take all necessary precautions to avoid damage to the completed soil-cement by equipment, and to avoid the deposition of raw earth or foreign materials between layers of soil-cement. Where ramps are constructed over soil-cement that is not to grade, all foreign materials and the uppermost one (1) inch of the previously placed soil-cement mixture must be removed prior to continuation of the soil-cement construction.

920-3.11 Compaction. Soil-cement shall be uniformly compacted to a minimum of 98 percent of maximum density as determined by field density tests taken in accordance with the procedures of Arizona Test Method 230 or 231. Optimum moisture and maximum density shall be determined in accordance with Arizona Test Method 221, 222b and 223 procedures. Wheel rolling with only hauling equipment shall not be an acceptable method of compaction.

At the start of compaction, the mixture shall be in a uniform, loose condition throughout its full depth. Its moisture content shall be such that the required compaction can be achieved with a minimum of effort and manipulation of the mix. No section shall be left undisturbed for longer than (30) minutes during compaction operations. Compaction of each layer shall be conducted in such a manner as to produce a dense surface, free of compaction planes in not longer than one (1) hour from the time water is added to the mixture. Whenever the Contractor's operation is interrupted for more than two (2) hours, the top surface of the completed layer, if smooth, shall be scarified to a depth of at least one (1) inch with a spike tooth instrument, or by other means acceptable to the Engineer, prior to placement of the next layer. The spacing of striations, produced by scarifying, shall not exceed 18 inches measured perpendicular to the length of the soil-cement being placed. The surface, after said scarifying, shall be swept using a power broom, or other method approved by the Engineer, to completely free the surface of all loose material prior to placement of the next layer of soil-cement.

920-3.12 Finishing. After compaction, the soil-cement shall be further shaped to the required lines, grades, and cross sections and rolled to a reasonably smooth surface. Shaping of the face of the soil-cement bank protection shall be conducted daily, at the completion of each day's production.

920-3.13 Curing. Temporarily exposed surfaces shall be kept moist as directed by the Engineer.

Care must be exercised to ensure that no curing material, other than water, is applied to the surfaces that will be in contact with succeeding layers.

Permanently exposed surfaces shall be kept in a moist condition for seven (7) days, or they may be covered with a suitable curing material, subject to the Engineer's approval. Any damage to the protective covering within seven (7) days shall be repaired to the satisfaction of the Engineer.

When a curing material, other than water is used, the permanently exposed surfaces shall be kept moist until the curing material is applied. The curing material is to be applied as soon as practicable, with a maximum time limit of twenty-four (24) hours between the finishing of the surface and the application of the protective cover or membrane.

When necessary, the soil-cement shall be protected from freezing for seven (7) days after its construction by a covering of loose earth, straw, or other suitable material approved by the Engineer.

920-3.14 Construction Joints. At the end of each day's work, or whenever construction operations are interrupted for more than two (2) hours, a 15° minimum skew, transverse construction joint shall be formed by cutting back into the completed work to form a vertical face for the full depth of the soil cement layer, as directed by the Engineer.

920-3.15 Maintenance. The Contractor shall be required, within the limits of his Contract, to maintain the soil-cement in good condition until all work is completed and accepted. Maintenance shall include immediate repairs of any defects that may occur. This work shall be done by the Contractor at his own expense and repeated as often as necessary. Faulty work shall be replaced for the full depth of the layer.

920-3.16 Inspection and Testing. The Engineer, with the assistance and cooperation of the Contractor, shall make such inspections and tests as he deems necessary to insure the work conforms to the Contract documents. These inspections and tests may include, but shall not be limited to: (1) the taking of test samples of the soil-cement and its individual components at all stages of processing and after completion, and (2) the close observation of the operation of all equipment and methods used on the work. Only those materials and methods achieving the requirements specified herein shall be acceptable.

All testing of soil-cement, its individual components, or the mix design unless otherwise provided specifically in the plans or special provisions, shall be in accordance with the latest applicable Agency specifications in effect as of the date of advertisement for bids on the project.

Testing for proper compaction shall be done on at least every other layer of compacted soil-cement, at any location chosen by the testing personnel. If the layer being tested does not achieve the minimum density requirements, as specified in Subsection 920-3.11, it must be reworked until it meets the requirement or be removed at the Contractor's expense. The Contractor shall not be permitted to continue placing subsequent layers of soil-cement on any layer which has failed to achieve the required compaction until such time as the deficient layer has been reworked, retested, and found to meet the density requirements specified.

The initial acceptance of the soil-cement shall in no way preclude fur- ther examination and/or testing at any time during the course of construction or the warranty period that the Engineer suspects that the material is not properly represented by the sample(s) obtained to date. The acceptance, at any time, of material incorporated into the work shall not bar its future rejection if it is subsequently found to be defective in quality or uniformity.

920-4 METHOD OF MEASUREMENT

The work shall be measured: (1) in cubic yards of soil-cement bank protection, lining or grade control structure, as determined by the specified lines, grades, and cross sections shown on the plans or as calculated from the soil-cement plant scale readings, whichever is the lesser; and (2) in tons of cementitious material (i.e., portland cement or portland cement and fly ash) incorporated into the soil-cement used for bank protection, linings, or grade control structure(s) as indicated in the special provisions or in accordance with the instructions of the Engineer.

Cementitious material shall be computed in accordance with the example herein provided. Any waste of cementitious material and/or soil-cement material by the Contractor during the handling, mixing, placing, etc., operations shall not be paid for.

920-4.01 Example Calculation For The Measurement of Cementitious Material Incorporated Into The Soil Cement.

Cementitious Material (Tons) = A x B x C

where:

A = Volume of soil cement as determined (1) from the lesser of the specified lines, grades and cross-sections indicated on the plans or (2) from the total plant scale readings for aggregate and cementitious material multiplied by "B", the average in-place dry density.

B = Average in-place dry density as determined from the compaction test results obtained through the course of construction.

C = Average content of cementitious material as determined from the plant scale readings and calculated by dividing the total weight of cementitious material by the total weight of dry aggregate and cementitious material.

Thus, if: A = 10,000 cubic yards; B = 135 pounds per cubic foot or 1.8225 tons per cubic yard; and C = 8.0%; the quantity of cementitious material, for the purpose of payment, would be 1,458 tons.

920-5 BASIS OF PAYMENT

The accepted quantities of soil-cement, measured as provided above, will be paid for at the contract unit price per cubic yard of soil-cement bank protection, lining, or grade control structure and at the contract unit price per ton of cementitious material (i.e., portland cement or portland cement and fly ash). Such payment shall constitute full reimbursement for all work necessary to complete the soil-cement bank protection, linings or grade control structures, including: dewatering; toe trench excavation; providing and stockpiling soil aggregate; watering; mixing; placing; compacting; shaping and finishing; curing; inspection and testing assistance; and all other incidental operations.

Payment for additional excavation, where determined necessary by the Engineer to remove unsuitable material, in accordance with the requirements of Subsection 203-5.03(A), will be made in accordance with the provisions of Subsection 109-3.

Blending of dissimilar soils, when specified in the plans, indicated in the special provisions, or required by the Engineer to make unsuitable material(s) suitable, shall be considered incidental to and included in the cost of producing and placing soil cement. However, should the Engineer direct the Contractor to blend dissimilar soils when such blending was not indicated in the plans or the special provisions, or required to achieve the gradation requirements of Subsection 920-2.04, payment for the blending, inclusive of the extra cost for material handling, will be made in accordance with the provisions of Subsection 109-3.

SECTION 920

SOIL-CEMENT FOR BANK PROTECTION, LININGS AND GRADE CONTROL STRUCTURES

920-1 DESCRIPTION

The work under this section shall consist of furnishing all materials and constructing soil-cement bank protection, linings and grade control structures at the locations and in accordance with the details shown on the plans and the requirements of the specifications. The work shall include toe trench excavation, backfill, and dewatering.

920-2 MATERIALS

920-2.01 Portland Cement. Portland cement shall conform to the requirements of ASTM C 150 for Type II.

920-2.02 Fly Ash. Fly ash shall conform to the requirements of Subsection 1006-2.04 (D).

920-2.03 Water. Water shall conform to the requirements of Subsection 1006-2.02.

920-2.04 Aggregate. The soil used in the soil-cement mix shall not contain any material retained on a one and one-half (1-1/2) inch sieve, nor any deleterious material. Soil for soil-cement lining shall be obtained from the required excavations or from borrow areas specified on the plans or approved by the Engineer and stockpiled on the job site as specified herein. The Contractor shall be responsible for providing all the soil required for soil-cement production. Soil, meeting the gradation requirements specified herein, shall be obtained from the project excavations and/or from borrow areas specified on the plans or obtained by the Contractor. Soil suitable for soil-cement shall be obtained from the required excavations, as shown on the project plans, before any borrow material is allowed to be used for this purpose. The actual soil to be incorporated into the soil-cement shall be analyzed by laboratory tests in order to determine the mix proportions to be used on the project. The distribution and gradation of materials in the soil-cement lining shall not result in lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material. Soil shall conform to the following gradation:

Sieve Size	Percent Passing (Dry Weight)
1-1/2"	98% - 100%
#4	60% - 90%
#200	5% - 15%

The Plasticity Index shall be a maximum of 3.

SOIL CEMENT APPLICATIONS AND USE IN PIMA COUNTY FOR FLOOD CONTROL PROJECTS

PIMA COUNTY DEPARTMENT OF TRANSPORTATION
AND FLOOD CONTROL DISTRICT
1313 SOUTH MISSION ROAD
TUCSON, ARIZONA 85713

JUNE, 1986

PIMA COUNTY DEPARTMENT OF TRANSPORTATION
AND FLOOD CONTROL DISTRICT
201 North Stone Ave
Tucson, Arizona 85701
Phone: 740-6460

LETTER OF TRANSMITTAL

DATE Sept 21, 1992 JOB NO.

ATTENTION Russ Miracle

RE: Soil Cement

TO SFC Engineering
7776 Point Parkway West
Suite 290
Phoenix, Az 85044

GENTLEMEN:

WE ARE SENDING YOU ☐ Attached ☐ Under separate cover via ______ the following items:

☐ Shop drawings ☐ Prints ☐ Plans ☐ Samples ☐ Specifications

☐ Copy of letter ☐ Change order ☐ ______

COPIES	DATE	NO.	DESCRIPTION
1	1988	12	PC/COT Standard Specifications for Public Improvements
			Section 920: Soil Cement
1	6/1986		Soil Cement Applications & Use in Pima County for
			Flood Control Projects.

THESE ARE TRANSMITTED as checked below:

☐ For approval ☐ Approved as submitted ☐ Resubmit ____ copies for approval

☒ For your use ☐ Approved as noted ☐ Submit ____ copies for distribution

☒ As requested ☐ Returned for corrections ☐ Return ____ corrected prints

☐ For review and comment ☐ ______

☐ FOR BIDS DUE ______ 19 ____ ☐ PRINTS RETURNED AFTER LOAN TO US

REMARKS ______

COPY TO ______

SIGNED: L. Robin

SOIL-CEMENT APPLICATIONS

AND USE IN PIMA COUNTY

FOR FLOOD CONTROL PROJECTS

PIMA COUNTY DEPARTMENT OF TRANSPORTATION
AND FLOOD CONTROL DISTRICT
1313 SOUTH MISSION ROAD
TUCSON, ARIZONA 85713

REVISED JUNE 1986

PRELIMINARY

Prepared for

BOARD OF SUPERVISORS/BOARD OF DIRECTORS

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SECTION 1 - INTRODUCTION

1.1 Problem Statement

Several different types of flood related problems occur in the ephemeral stream systems of the alluvial basins in the southwestern United States. The ephemeral streams of the Santa Cruz River Basin within Pima County, Arizona, demonstrate a variety of hazards during flood events which include overbank inundation, channel bank erosion, channel migration, and channel bed degradation and aggradation. The physical changes to the stream channel (i.e. bank erosion, channel migration, channel bed degradation and aggradation), that occur during flooding events, are a more severe hazard, physically and economically to public facilities and private property, than the hazard of overbank inundation alone. Current methods of economic evaluation of the damage from flood inundation of lands and the associated disruption of activities do not consider flood related damages resulting from bank erosion and channel migration; therefore, these methods seriously underestimate the cost-benefit of planned flood and erosion control measures. The prediction of channel erosion and migration together with associated damage are serious engineering considerations for floodplain management purposes as is the design of public and private facilities which cross or are adjacent to flood hazard zones. Effective flood control improvements that are efficient, economical, and as environmentally sensitive as possible must be developed to minimize and mitigate Southwest flood damage and hazards.

1.2 Objectives of the Report

The objectives of this report are to explain the research, application, design, construction, and design life of soil cement as a cost effective, multi-use flood control measure.

The primary use of soil cement in Pima County has been to reduce the erosion hazard of unstable natural channel banks in order to protect public facilities including roads, bridges, landfills, and private development projects. Soil-cement design specifications, engineering analyses, construction techniques, and current research are explained together with examples of the performance of soil-cement bank stabilization during the October 1983 flood.

1.3 Applications of Soil Cement

While soil cement has a long record of satisfactory service as a paving material for highways, streets, and airports, it has also been successfully used for many years in energy and water resource projects. Applications include slope protection, seepage control, and foundation stabilization. Advantages of soil cement include low cost, ease of construction, and convenient utilization of local or in-place soil, making soil-cement applications economical, practical, and environmentally attractive.

Soil cement has been used, within Pima County, for channel bank stabilization, grade control structures, channel bed protection, detention basins, landfill protection, bridge abutment protection, and as a base material for highways. To date, soil cement has proven to be the only effective method of bank stabilization on the major river systems in Pima County.

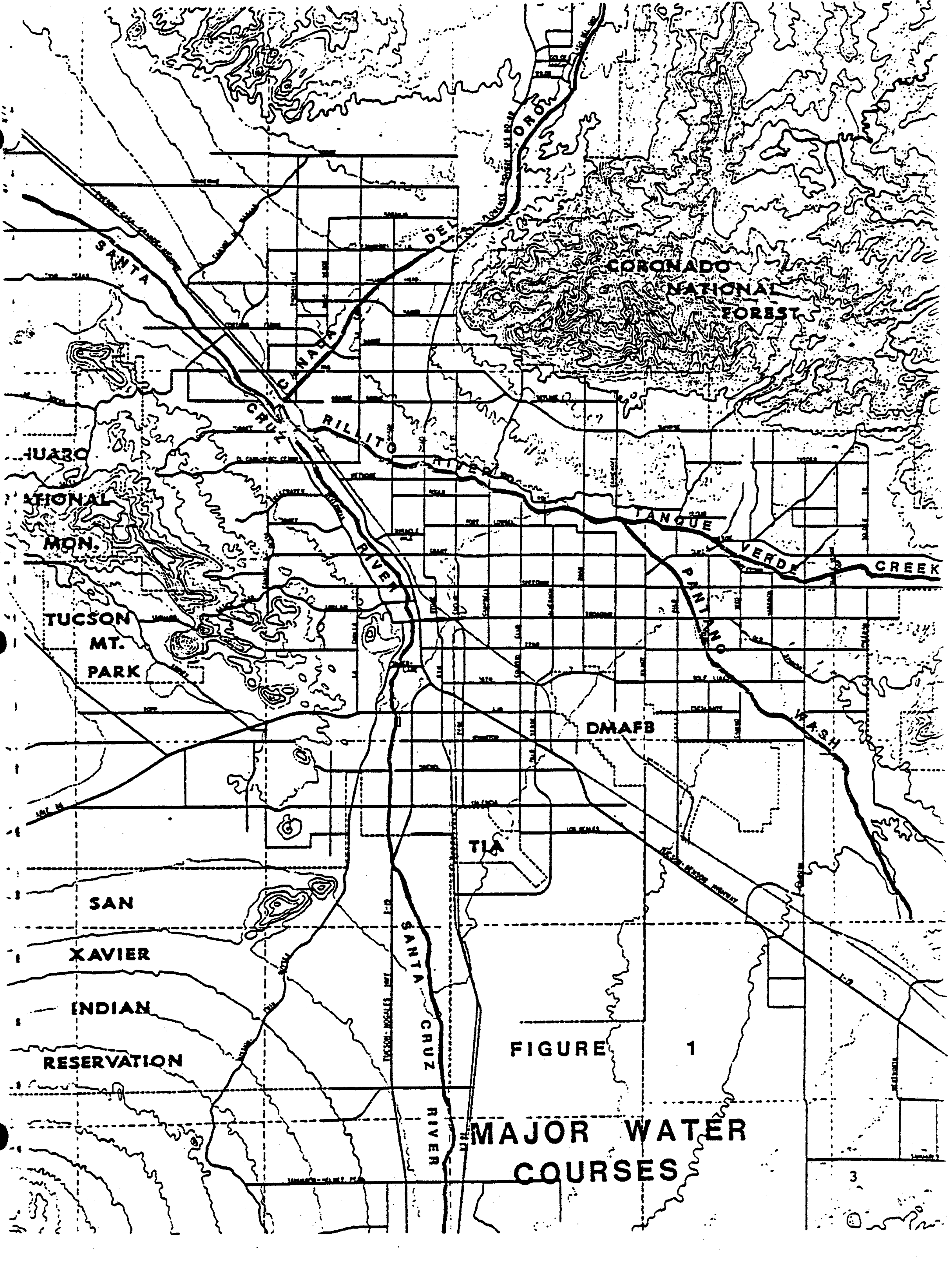


FIGURE 1

MAJOR WATER COURSES

SECTION 2 - SOUTHWEST FLOOD HAZARDS

2.1 Basin/River Characteristics in Pima County

Pima County is located in the geologic Basin and Range Province of Arizona. The lower Santa Cruz Basin, in which Tucson, Arizona is situated, is a topographic basin bounded by the Santa Catalina and Tortolita Mountains to the north, the Rincon Mountains to the east, the Santa Rita Mountains to the south, and the Tucson Mountains to the west. The principal watercourses in this basin collect and transport storm runoff, snow melt, and sediment from these mountain ranges to the alluvial valley. As in many desert mountain masses, those near Tucson have broad piedmont surfaces which extend at fairly uniform slopes away from much steeper mountain fronts. The ephemeral streams of the piedmont areas convey water and sediment from the mountain fronts to the valley floors in the basin mainly on the piedmont, while the finer fraction of sediment load, including sand, silt, and clay are conveyed to the valley floors. Sediments transported to the Tucson Valley are mostly fine sand and silt.

In eastern Pima County, the average annual precipitation varies from 8 inches, in the low lying deserts, to 15 inches in the mountainous areas. Most of the rainfall occurs during the summer season, July to September, as high intensity, localized thunderstorms, or in the winter months, December to February, as regional low-intensity, long-duration storms. Localized summer storms cause major flooding primarily along minor channels and on small- to moderate-sized watersheds. Winter storms cause widespread flooding which affects the larger basins. Additionally, winter storms may result in snowmelt flooding from mountainous basins.

While the summer and winter storms produce significantly different flooding circumstances, the U. S. Geological Survey has found that there is no difference between winter and summer flood peaks for floods with a magnitude greater than the 10-year flood. A third storm variety, caused by the remnants of tropical storms along Baja California and the Pacific Ocean, may occur during the fall months. The flood events of 1977 and October 1983 resulted from this type of late tropical storm.

The most common flood hazards in Pima County result from flash flooding, sheet flooding, and high flow velocities which cause bank erosion, channelization, and channel bed change. Streams and rivers in Pima County are generally ephemeral in nature and flow only in response to precipitation or snowmelt. During the summer, intense thunderstorms may result in flash floods where the streams' flow capacities are limited, especially at dip crossings which pose hazardous threats to the motoring public, and along minor stream channels where residential structures may be jeopardized. Sheet flow affects large areas of Pima County, and occurs along mountain fronts where either alluvial fans or mountain pediments exist. Flows from the mountain areas discharge from confined canyon channels onto these planar sloping surfaces where stream channels, if they exist, are poorly defined. As a result, storm runoff spreads out across the surface flooding large areas. In addition to damages caused by flood waters, severe erosion and property losses are caused by the high flow velocities found in Pima County. Even along major streams which have the capacity to contain the 100-year storm events, floods have caused severe damage to bridges and utilities as well as a significant loss of land.

Major Rivers

The Santa Cruz River Basin, in eastern Pima County, is the major river system with most of the major and minor streams tributaries to the Santa Cruz River. The major river systems are shown in Figure 1, and include the Canada del Oro Wash and the Rillito Creek.

Canada del Oro Wash

The 250 square mile Canada del Oro Basin includes the Canada del Oro Wash, Sutherland Wash, and Big Wash. Sutherland Wash and the Canada del Oro Wash, upstream of the Lago del Oro Area, are very steep mountainous streams with average basin slopes of 10 percent. Because these streams originate at high elevations, they flow in response to snow melt and localized summer thunderstorms caused by orographic effects. Big Wash and the downstream reach of the Canada del Oro Wash flow through desert areas with much flatter (1 to 2 percent) slopes and stream channels of coarse, loose sand. In these lower downstream areas, bed and bank erosion may become major problems as the system tends to be unstable. Big Wash and the Canada del Oro Wash, from U.S. Highway 89 to Magee Road, are braided stream systems. The Canada del Oro Wash, downstream of Magee Road, has become channelized and, as a result, a vertical drop in the stream bed, or head cut, has formed marking the upstream erosion limits. If this were not controlled, the head cut would progress upstream causing degradation of the stream bed. A major soil-cement grade control structure was constructed at Magee Road to stabilize the channel and upstream river system.

Rillito River

The Rillito River drains 940 square miles of watershed. The two major tributaries to Rillito Creek are Pantano Wash and the Tanque Verde Creek. The Pantano Wash system includes Rincon Creek, Davidson Creek, Cienega Creek, and Mescal Arroyo. Except for Rincon Creek, these streams and basins are primarily on the high desert and are highly responsive to summer thunderstorms. Rincon Creek originates in the Rincon Mountains and may also flow from snowmelt. Tanque Verde Creek and its tributaries, Agua Caliente Wash, Sabino Creek, and Ventana Canyon, originate at higher elevations and are, therefore, prone to flooding from winter storms and snowmelt as well as summer storms.

Rillito Creek and Tanque Verde Creek are at an equilibrium state where aggradation and degradation balance over a period of time; however, lateral bank erosion is prevalent during prolonged low flows and major flows, and will require soil-cement bank stabilization at several locations in the future. Pantano Wash is actively degrading and highly unstable, and will also require grade control structures to stabilize the gradient to prevent damage to bridges and private property in the near future

Santa Cruz River

The Santa Cruz River at Cortaro Road Bridge has a drainage area of 3,505 square miles. Major tributaries to the Santa Cruz River, within the urbanized area of eastern Pima County, include the West Branch of the Santa Cruz River, Tucson Arroyo, Julian Wash, Rodeo Wash, and Airport Wash. During large flows, all of these washes will break out of their channels causing damage to public and private property. The Santa Cruz River is degrading both through Tucson and in the upstream channel reaches.

Severe bank erosion associated with overall stream degradation is threatening public utilities, highways, and property at several locations. The Rillito River and Canada del Oro Wash join the Santa Cruz River north of Cortaro Road. In the channel reach downstream of the confluence with the Rillito River, the Santa Cruz channel bed is essentially stable; however, further downstream, near the Town of Marana, the Santa Cruz River is rapidly aggrading which, in turn, has caused overbank flooding from smaller and smaller flows. The Santa Cruz River System is not in an equilibrium state; but, instead, is still adjusting to the environmental changes by aggrading in the downstream reaches and degrading in the upstream reaches.

2.2 Channel Bank Erosion

This section will primarily focus on geomorphic history, natural channel bank erosion, impact on public and private structures, and floodplain management regulations.

Natural streams adjust their characteristics in response to any changes in the environment. These environmental changes may occur naturally, or may result from such human activities as urbanization, sand and gravel mining, bridge and highway construction, and flood control improvements. Such changes distort the equilibrium of streams, and may result in either aggradation or degradation. Channel width variations usually occur in an aggrading or degrading stream. A degrading stream generally tends to assume a narrower width, while an aggrading stream tends to widen itself by flooding adjoining areas.

The cross-sectional shapes and plan-view patterns of an alluvial stream channel are produced by erosional and depositional processes that vary in space, time, and magnitude along the length of the channel. These processes occur in response to the frequency, magnitude and duration of stream flow, and to the amount and type of sediment in transport.

The response of a stream channel to large flow events often consists of an increase in channel cross-sectional area unless the slope, ratio of width to depth, or roughness of the channel are greatly reduced. Increases in channel cross-sectional areas result from either channel entrenchment (degradation), widening of the channel, or a combination of both. A low sediment concentration in stream flow will cause a channel to enlarge by excavating its banks and/or bed. A high sediment concentration will cause aggradation of the channel bed and/or deposition of sediment on the floodplain in the event of overbank flooding.

At a more detailed level, channel width is primarily a function of shear on the banks during flows and resistance of bank materials. The shear is dependent upon discharge, sediment load in transport, and resistance of the banks to flow. In turn, bank resistance is a function of the size, shape, and cementation of the materials that compose the banks as well as the density of riparian vegetation present. Bank erosion occurs when the stress applied exceeds the resistance of the banks, and may result from combinations of flow velocities, formation of eddies downstream of irregularities present in the banks or stream bed, and from deflection of stream lines of flow against the banks by boulders, debris, or deposits of sediment. Bank erosion in the Santa Cruz River system has often included bank caving which is the slumping or sliding of masses of bank materials into a channel when turbulent currents undercut the banks.

Coupled with channel bank erosion is the development of a channel meandering pattern caused by traverse cross waves occurring at channel bends. Due to heterogeneties in nature, such as non-uniform valley topography and sediments, vegetation, bedrock outcrops, the size and shape of meanders vary along individual channels. Additionally, man-made changes such as encroachment, sand and gravel excavation, and land fills also affect channel meandering pattern.

Over the years, long duration low flows and short duration high flows have caused significant channel bank erosion and channel meandering of natural stream banks. Non-structural measures to protect private development have included the adoption of building setbacks along unprotected channel banks within the Pima County Floodplain Management Ordinance. Building setback limits considering discharge, historical erosion patterns, soil stability, and quantitative analysis have been established along all major watercourses. Structural bank stabilization measures, along the major watercourses only, consist of soil cement due to its cost effectiveness and highly successful performance during flooding events.

2.2.1 Geomorphic History of the Santa Cruz River System

The geologic perspective on floods is not often incorporated into flood hazard management. In the humid temperate regions of the United States, river channels generally occur within a bottom land surface created by the river itself. The river occasionally overtops its banks, carrying and depositing sediment on the geologic floodplain of the river. The hazard zones of such rivers show close correlation to this geologic floodplain since it is inundated relatively frequently. There is a 50 percent chance each year that shallow water may inundate the overbanks, and a two percent chance each year that deep flows with a depth roughly equal to twice the channel bank heights may occur. The above experience has little applicability to ephemeral sand bed streams in the valley bottom lands of the arid and semi-arid west. Streams in that environment may flow directly on broad, low valley floors as sheet flow with little or no channel. In that case, every flow event could cause damage, especially as the threads of flow shift on a depositional surface. In extreme contrast is the incised channel where the stream has cut through the former bottom lands so deeply that even very rare high flows will not spill out of the incised banks. Both of the above conditions may occur on the same stream either at different localities at a given time, or at different times at the same locality. The management of flows in such an environment cannot ignore the geologic complexities of the system. Experience gained during the Tucson flood of 1983 illustrated this conclusion.

The general channel configurations and cross-sectional shapes on valley floors in the Tucson area today are derived from a history of arroyo cutting in the late 1800's and early 1900's. Similar histories are prevalent throughout the Southwestern United States, and considerable controversy surrounds attempts to provide a general explanation of the phenomenon. Tucson's extensive historical record allows precise reconstruction of events associated with the transition of the Santa Cruz River Basin from an alluviated valley floor to a narrow, steep-sided channel. Prior to the major flood of August 1890, the Santa Cruz River exhibited perennial flow at several reaches near Tucson. During the 1880's, this system began to reflect a profound human impact. Various impoundment and diversion structures were introduced to facilitate irrigation on the alluvial valley floor. Floods in 1880 and 1890 began the formation of a headcut on the Santa Cruz River beginning at St. Mary's Road in Tucson. The valley floor of the Santa Cruz River Basin was then incised with an arroyo approximately 100 feet wide. The head cut continued to recede with each flow event until, by 1910, it had reached Martinez Hill south of Tucson.

In the winter of 1914-15, the Santa Cruz continued to incise upstream of Martinez Hill. By the 1930's, a head cut had migrated from Agua de la Mission up an artificial channel, built in 1913, to join the west and east branches of the stream. During the 1920's and the 1930's, the incised river near Congress Street began to aggrade. This aggradation was probably a result of decreased flow velocities when the channel was widened in 1914-15, from the stabilizing effects of cottonwoods and other riparian vegetation, and from the lack of major winter floods in this period. Because of relatively low sediment loads, winter floods in contrast to summer floods are particularly erosive for the sand-bed streams of southern Arizona.

In 1950, the Santa Cruz River, through Tucson, was modified by artificial narrowing through landfill operations and highway construction. The reach from Martinez Hill to Congress Street had been straightened in 1935 by the efforts of the Works Progress Administration. Flows were deflected from severe bends by means of revetments. The result of these changes was renewed downcutting by higher velocity flows through the straightened and constricted channel. The zone of aggradation moved downstream, past all the urbanized reaches, to the far northern end of the Tucson Basin.

The Rillito system has an early history similar to that of the Santa Cruz. From a condition in 1858 represented by a broad valley floor, the stream developed to a wide channel with vertical banks by 1890. Smith (1910) attributed this to the cutting of vegetation, overgrazing by cattle, and flood erosion. Channel incision continued into the present as excessive withdrawal of ground water eliminated riparian vegetation along stream banks and bars.

Studies of aerial photographs since 1941 show that the Rillito-Pantano-Tanque Verde system has been characterized by prolonged periods of channel narrowing locally interrupted by abrupt periods of widening with attendant bank erosion. Narrowing, from 1941 until 1965, occurred during an interval dominated by short duration floods with peaks of less than 9,000 cfs. Most of these flows occurred in the summer and early fall, and probably transported very high sediment loads. In 1965, a large winter storm produced a prolonged flood that peaked at approximately 10,000 cfs for the Tanque Verde Creek and Rillito River. This flood carried a relatively low sediment load and generated extensive bank erosion for both stream channels. In contrast, Pantano Wash did not experience this flood and continued to narrow.

After the 1965 event, both Tanque Verde Creek and the Rillito River displayed either natural recovery or local, artificial stabilization. In December 1978, another major winter storm affected these streams. As in 1965, this event, which peaked 16,500 cfs in the Rillito River, was characterized by prolonged duration and a relatively low sediment load. Additionally, a flow in March 1978 removed much of the sediment that had accumulated in the stream channels during the preceding recovery period. Extensive bank erosion occurred during a three-day period of flow. Channel realignment, bank protection, and bridge repair after the 1978 flood led to the general condition of the Rillito-Pantano-Tanque Verde system immediately prior to the 1983 flood.

It is clear that the streams of the Tucson Basin have been irreversibly altered from conditions which prevailed prior to large-scale human intervention. A return to the channel characteristics of the 1800's is impossible for the following reasons: 1) Groundwater overdraft has so lowered the water table that the stabilizing influence of riparian vegetation has been lost. 2) The urbanization process has reduced the influx of sediments from tributaries into the main channels while increasing the influx of water from individual storms, and 3) Channels have been constructed by bridges, encroachment banks, and revetment works.

2.2.2 Overview of 1983 Flood Erosion

As described in The Tucson, Arizona, Flood of October 1983, prepared by the committee on National Disasters, Commission on Engineering and Technical Systems National Research Council, various degrees of channel bank erosion along the Santa Cruz River occurred in response to physical variation of bed material and/or man-made structures.

Upstream of the City of Tucson, flow was deflected due to the bedrock obstruction of Martinez Hill at Interstate 19 which induced pronounced bank erosion from deflective wave action. Despite extensive rock riprap revetments, two bridges were lost. Meander bends also illustrated important erosional effects. A cut bank of an upstream meander bend was protected by 333 feet of riprap revetment. Nearly all of this was destroyed during the flood because bed scour undermined the revetment. Unprotected channel banks experienced various degrees of erosion with bank meandering distances of greater than 100 feet being common.

Through the City of Tucson, where the Santa Cruz River has continuous soil-cement bank stabilization, protected banks were generally unaffected by the flood flows.

The Santa Cruz showed a major change in character downstream from the City of Tucson. The natural channel banks did not enlarge to convey the flood flows, and flood waters spilled onto the adjacent floodplain surface. A secondary channel with a headcut developed on this surface from flow spilling back into the main channel. Even greater spreading of the flood flows occurred downstream of the Rillito River confluence. Large sand and gravel pits and the Pima County Wastewater Treatment facility are located immediately adjacent to the channel. Most of these facilities are located along the east bank and are protected by various types of revetments; erosion from the 1983 flood occurred primarily on the unprotected west banks.

Downstream of the confluence with the Canada del Oro, spectacular erosion on the west bank resulted in bank recession that exceeded the original channel width spanned by the Ina Road Bridge. Extensive overbank flooding and deposition occurred downstream of this point.

A relatively sinuous reach downstream of Cortaro Road was lined with riparian vegetation maintained by sewage effluent. The 1983 flood cut a straight channel through this section and deposited thick overbank sediment on each side of the channel.

At the town of Marana, the flooding of the Santa Cruz became a broad area of shallow inundation over extensive valley floors.

Significantly, the sediment transport character of flooding downstream of this point was depositional. Sediment transported from the extensive areas of erosion upstream contributed to aggrade in this area. The aggradation, in turn, led to more extensive inundation. The buildup of sediment from past floods may have so increased slopes in this reach that a new headcut was initiated in the Santa Cruz Valley near Picacho Peak.

The post flood erosion survey also documented the behavior of the Rillito-Pantano-Tanque Verde system during the 1983 flood. Pantano Wash is interesting because a large drop structure at Broadway separates the system into an incised reach downstream north of Broadway and into a less incised reach upstream south of Broadway. Bank erosion from the 1983 flood was minimal upstream of the drop structure. Exceptions were at a prominent meander bend and at a sand and gravel pit north of Golf Links Road. Downstream of the Broadway Boulevard drop structure, the Pantano Wash is deeply incised. It responded to the 1983 flood by the alternative pattern of bank erosion described above for the Santa Cruz River system. Local areas of bank protection, from riprap and wire fence revetment, effectively shifted the concentration of this erosion to unprotected banks. At the downstream end of revetment works, around the Tanque Verde Road Bridge, a major zone of bank recession caused the loss of residential property.

The upper reaches of Tanque Verde Creek, as with the upper Pantano Wash, showed much less bank erosion in the 1983 flood than did entrenched reaches of the Rillito and Santa Cruz Rivers.

Banks were sufficiently low enough that overbank flooding occurred. Major bank erosion appeared at the confluence of Sabino Canyon and Tanque Verde Creeks. This occurred due to: 1) the angle of the stream juncture which directed flows at an unprotected bank, and 2) changes in sediment loading that occurred as the two flows mixed. The second factor probably explains the increased erosion that occurred immediately downstream of the confluence of Tanque Verde Creek and Pantano Wash.

2.3 Sedimentation

The relationship between sediment transport and runoff is unstable in the arid Southwest resulting in an irregular pattern of aggradation and deposition. The watershed characteristics in Pima County, such as low vegetation cover, unconsolidated sediment, and steep gradients, create high sediment yields; however, due to low annual runoff, lack of perennial flow, high intensity flood peaks, and long duration of low flows, the stream beds fluctuate between periods of aggradation and deposition.

In the downstream reaches, the rivers generally experience aggradation as the declining stream discharge results in sediment deposition, except at high flood discharge volumes where head cutting may be present. This type of aggradation is apparent in the Marana area along the Santa Cruz River. The upstream river reaches have well defined river channels but alternate between a pattern of incision and deposition depending on the relative volume of stream flow and sediment from the tributary streams. The fluctuation between aggradation and degradation is typical of the Rillito River.

The various minor and major tributaries to the Santa Cruz River do not have equal ratios of sediment transport to water discharge; as a result, stream system stability varies among river reaches. Additionally, seasonal variations occur because sediment transport varies with the increasing intensity of the rainfall event. Spatial and temporal variations of storm events and flood peaks result in sediment moving through the stream system as slugs. Therefore, high discharge rates over sucessive years decrease sediment supplies whereas stream bed aggradation occurs during periods of relatively low discharge.

Examples of sedimentation hazards experienced in Pima County from the processes described above are:

1. Severe aggradation along the lower reaches of the Santa Cruz River that reduces natural channel capacity and results in increased overbank flood inundation. This deposition causes channel meandering and headcutting in the overbanks as flow returns to the main channel.

2. The Rillito River stream bed profile fluctuates two to four feet from seasonal variation in storm events. Periods of high summer runoff cause deposition of sediment generated by the lower tributaries whereas periods of winter storm flows generated from snowmelt produce long flow durations from the upper tributaries, remove sediment deposits, and cause erosion and channel degradation. These fluctuations in stream bed profiles significantly change the channel capacities creating additional concern for freeboard in flood control designs.

3. Slug flow of sediment with alternate reaches of incision and deposition resulted in the Canada del Oro Wash during the October 1983 flood event. Upstream head cutting created severe channel incisement with the resultant sediment being deposited in the lower channel reaches. Upstream channel lowering exceeded six feet in the area of the head cut with the downstream areas experiencing deposition of three to five feet.

SECTION 3 - APPLICATIONS OF SOIL CEMENT IN FLOOD CONTROL WORKS

GENERAL

Soil cement has been used extensively in Pima County for various types of flood control improvement projects. Soil cement is a cost effective method for stabilizing channel banks, constructing drop structures, providing detention basin protection, or other types of flood control improvements due to the relative ease in construction and because alternate materials are not available locally necessitating excessive transportation costs. By varying the construction methods, soil cement can be used for stabilization projects along major rivers or for minor tributary and local drainage improvements. Various types of flood control improvements utilizing soil cement in Pima County are described below.

3.1 Bank Stabilization

Bank stabilization using soil cement along the major river systems in Pima County has proven to be the most cost effective means of stabilization and protection of the river channels from excessive erosion. Normal bank protection of dumped rock riprap and/or wire-tied rail and rock gabions are not adequate in the Tucson/Pima County area due to a lack of an adequate rock source, the labor intensive nature of application and construction, the exacting quality control measures necessary to provide a stable system, and because the stream bed profile trend for degradation causes undermining and loss of these types of bank stabilization projects. Soil cement has proved to be a much more adequate material because it is relatively easy to construct, and the natural stream bed materials of sand and silt provide adequate construction material for the construction of soil cement. In most instances, the bank shaping and excavation for channel toe-down provides an adequate quantity of material for the soil cement. Generally, bank stabilization along major rivers is constructed by creating a continuous structure for a reach where normal 6-inch lifts of soil cement, 8 feet in width, are placed on top of one another and compacted to form an 8-foot thick embankment at a 1:1 face slope. Since this method does not create seams or joints, and has a construction thickness of 8 feet, soil cement has demonstrated extreme stability to withstand any removal or scour of the embankment material behind the soil cement even where flood waters have overtopped the soil cement. In this regard, soil cement is far superior to rock riprap or gabions where this type of floodwater inundation would result in failure of the rock riprap.

Areas where major soil-cement bank stabilization projects have taken place are the Rillito River in the Oracle Road area where one of the first applications of soil cement was the construction of a joint Pima County/Arizona Department of Water Resources bank stabilization and flood control project to protect a major electric substation and residential area; bridge abutment protection and channel improvements, such as at the La Cholla, Rillito River Bridge; the Rio Nuevo project on the Santa Cruz River where a soil-cement lined channel for protection to the 28,000 cfs, 100-year discharge was constructed with an overbank shelf area vegetated and landscaped to convey flows up to 50,000 cfs, and along the upper reach of the Canada del Oro Wash near Oro Valley to provide continuous bank protection between major bridge projects and to protect existing residential structures. Soil cement for construction of major bank stabilization projects is ideally suited in the areas that 1) lack alternate material such as rock riprap, 2) experience severe erosion and channel meandering which may attack either the upstream or downstream end points of the stabilization, or where flood waters may overtop the bank protection and <u>cause lack of the supporting embankment (?)</u>, and in areas where existing urbanization or residential structures limit the area for construction. Pima County has successfully utilized soil cement along the major rivers because this type of bank protection has not failed either when upstream channel banks have eroded away behind the upstream turnover or where the floodwaters have overtopped the soil cement. In both cases, the soil cement has not failed as opposed to major failures experienced with rock riprap and rock gabions. Additionally, soil cement has been placed and successfully constructed immediately adjacent to existing residential structures where the room for construction equipment was limited to 30 to 50 feet.

By varying the construction techniques, bank stabilization and soil cement can be used for minor tributaries utilizing a method called slope bank protection. In this method of bank protection, a channel is excavated with 4:1 or 6:1 side slopes, and slope pavement of soil cement in 2 to 4 foot lifts is applied. This permits construction of minor drainage improvements adjacent to roadways or subdivision developments.

3.2 Drop Structures

Incorporated into Pima County's bank stabilization projects is the use of grade control or drop structures as a means of controlling the degradation that has and is occurring along our major rivers. The soil-cement grade control structures are constructed below the existing channel invert essentially as massive retaining structures with wall varying slopes. The soil-cement drop structures are then tied into the soil-cement bank protection along the channel banks. These types of grade control and drop structures have been used along the Canada del Oro River to control a severe 6-foot head cut located at Magee Road, along the Rio Nuevo project in conjunction with bridge pier underpinning improvements to both control the river degradation as well as protect bridge structures and their subsurface structural members, and in the case along the Pantano Wash, to control the degradation adjacent to existing rock gabion bank protection to prevent undercutting and failure of the gabion structures.

3.3 Basin Protection and Detention/Retention

Soil cement has been used as slope stabilization at locations of major detention storm water facilities. The purpose of the soil cement is to either stabilize the steep slopes of the detention basin preventing erosion from storm water flowing over the sides and into the basins, and/or for protection at a major channel in light structures. Examples are the existing Massingale detention basin which uses soil-cement slope protection along the upstream side slopes of the basin as well as for major protection work at the inlet from a major channel carrying 3,000 cfs, and for the Julian Wash detention basin which is a floodwater detention basin located in a braided sheet flow area. Soil-cement bank protection was used all along the upstream face of the detention basin to allow sheet flow to enter the basin in a controlled manner at protected spillways.

3.4 Comprehensive Flood Control

Soil cement has been used in various types of flood control projects; primarily, these types of projects are bank stabilization along major rivers where the 100-year discharge varies from 20,000 to 80,000 cfs. Additionally, by varying the techniques of construction, soil cement can be used for either minor tributary flood control channel stabilization or levee construction. The Pima County Department of Transportation and Flood Control District undertook a major and comprehensive flood control improvement project in the Town of Oro Valley, Arizona, for the construction of approximately $10,000,000 worth of flood control improvements. This flood control project included the construction of two bridges, one at La Canada Boulevard and one at First Avenue; the construction of a flood control levee, and collection and control of side flow tributaries. Starting at the downstream end point, a brief description of the project is as follows:

1. At the La Canada bridge, soil cement was used for the bridge abutment protection as well as construction of upstream spur dikes to collect flow from the 2,500 foot wide-braided floodplain to convey it through a 900 foot long bridge.

2. Upstream of this area, the residential area of the Oro Valley Country Club had approximately 450 residential structures within the floodway and floodplain of the Canada del Oro Wash. As part of the protection project adjacent to these residential structures, a soil-cement levee was constructed along the stream flow side, and compacted earth was constructed adjacent to the residential flood free side to collect and prevent floodwater inundation.

3. Upstream of the Oro Valley Country Club area, major side tributaries from the Catalina foothills created flows of magnitudes varying from 3,000 to 6,000 cfs during the 100-year flow of high-velocity flood waters

generated on the steep mountain slopes. **(NOTE: The following underlined word grouping is not a sentence)** **A considerable side drainage tributary system was constructed to collect and convey floodwaters into a levy (levee) area further upstream at the flood control levee system was continued to provide a 700-foot wide natural channel area and to provide protection for the First Avenue Bridge abutment.** The following figures illustrate the erosion hazard along major watercourses in Pima County along with examples of soil cement for bridge protection, levees, grade control structures, and bank stabilization projects.

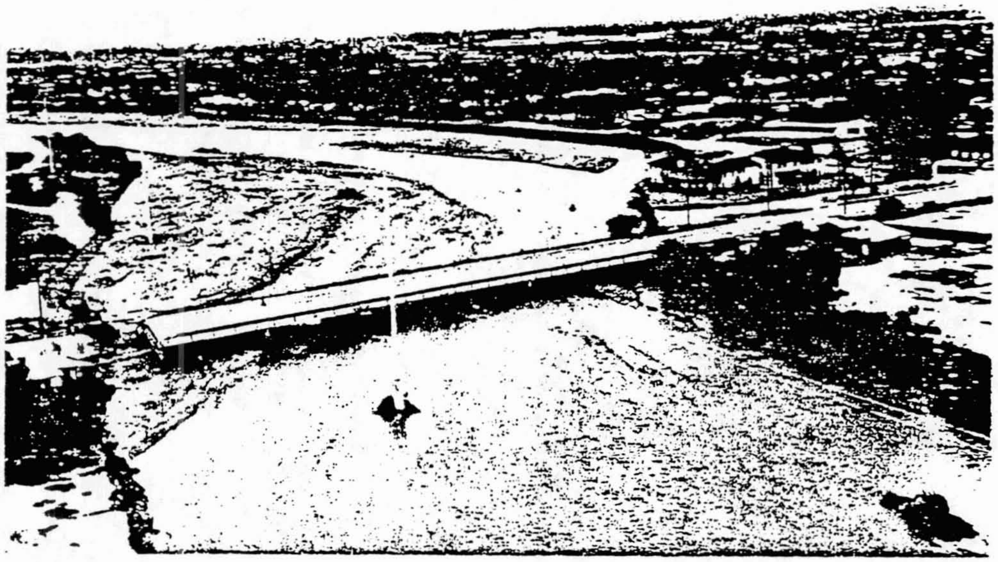
BANK STABILIZATION

WITH SOIL CEMENT

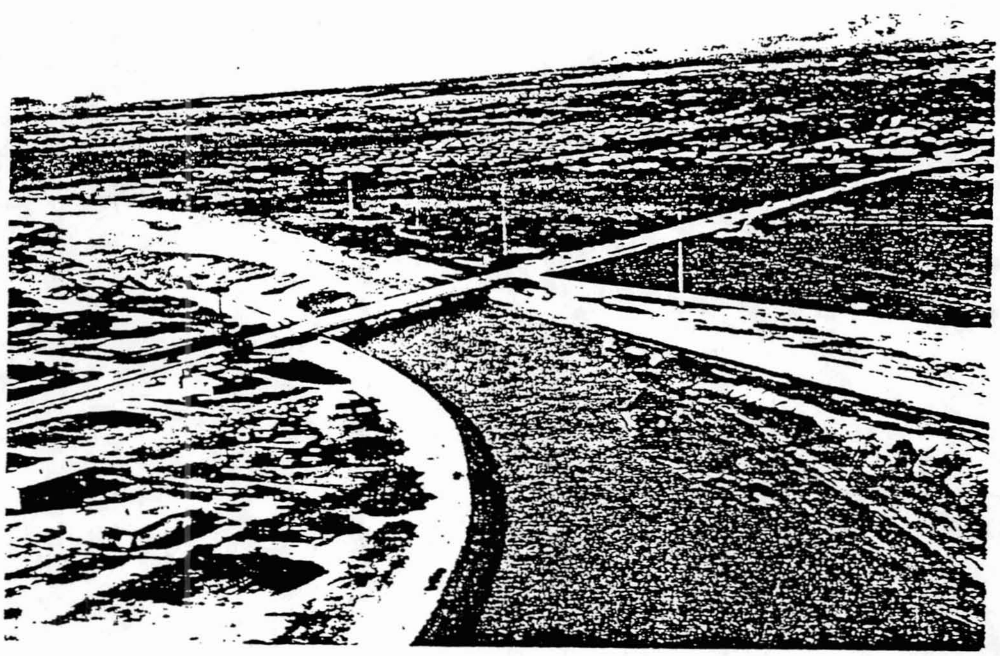
ON MAJOR RIVERS

Channel bank erosion, Pantano Wash
Illustrating natural erosion pattern along unprotected banks

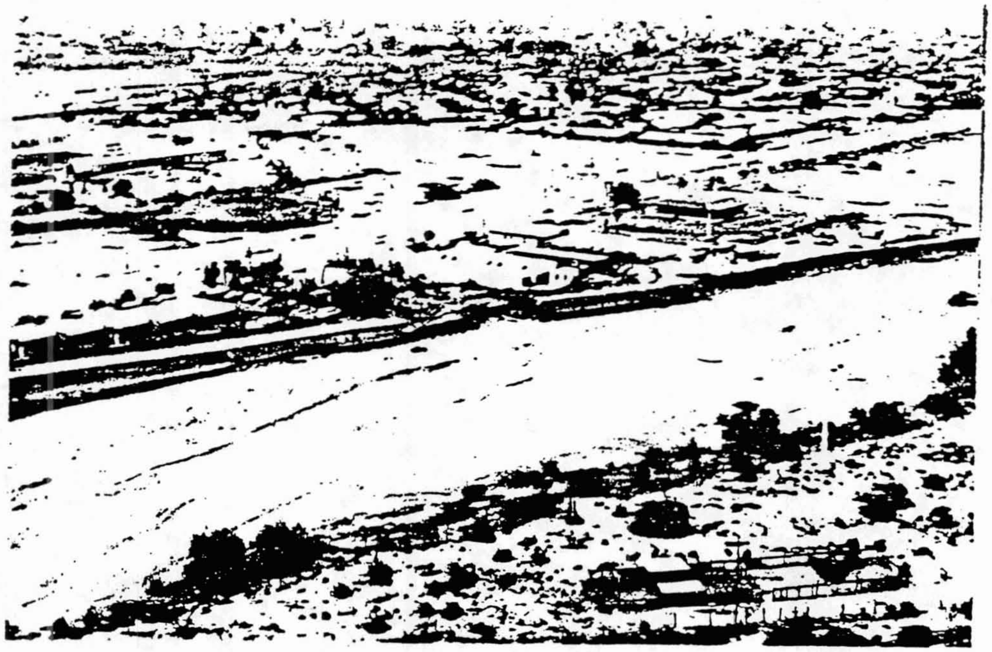
Left bank soil cement protection, Canada Del Oro Wash



Channel bank erosion, Rillito River at First Avenue



Soil Cement bank protection, Rillito River at La Cholla Boulevard
without soil cement protection



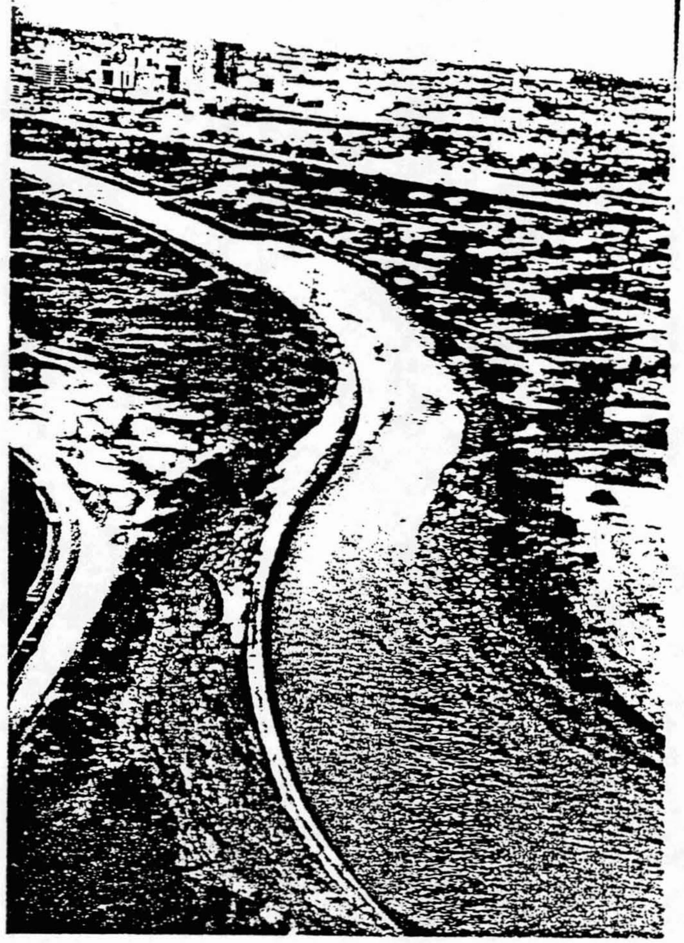
Non-contiguous soil cement bank protection, Pantano Wash protection did not fail even though it was not continous

Continuous soil cement bank protection, Rio Nuevo, Santa Cruz River

Soil cement bank protection, Rillito River at La Cholla Bridge

Soil cement bank protection, Rillito River at La Cholla Bridge

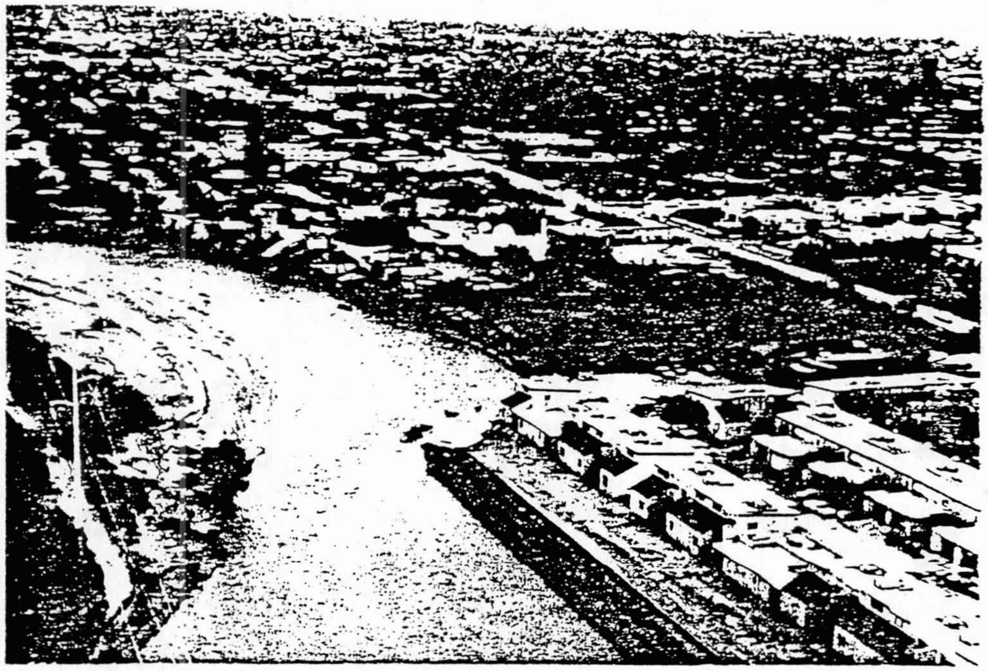
Overtopping of soil cement bank protection
Santa Cruz River at 22nd Street Landfill



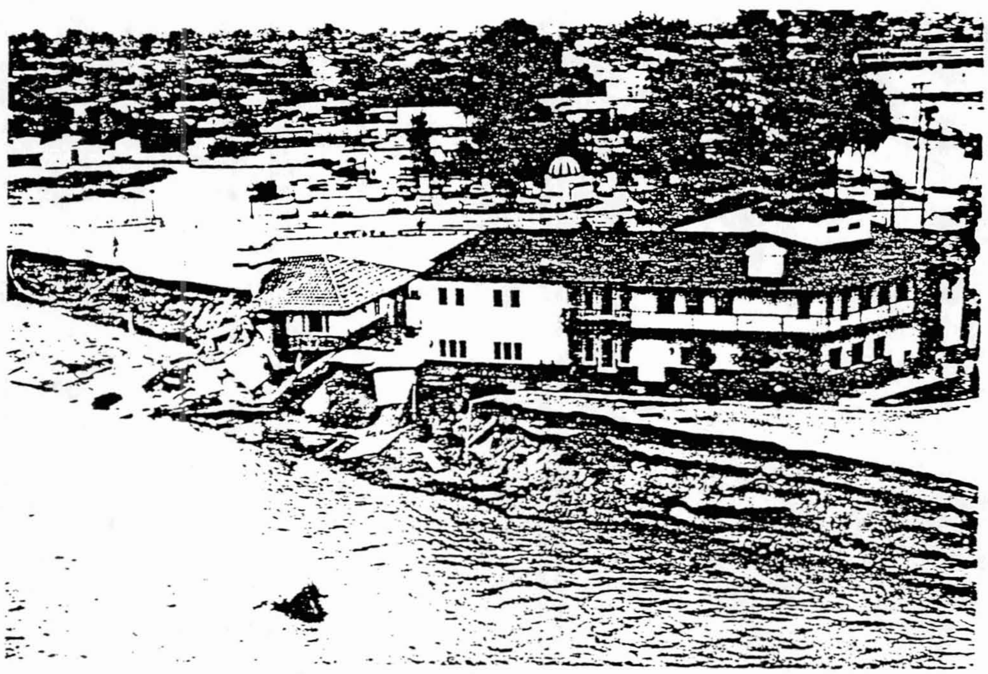
Continuous soil cement bank protection,
Rio Nuevo, Santa Cruz River

Soil cement spur dikes, Pantano Wash at Houghton Road

Soil cement abutment protection, Canada Del Oro at La Canada Bridge



Channel bank erosion upstream of soil cement, Rillito River at Prince Road



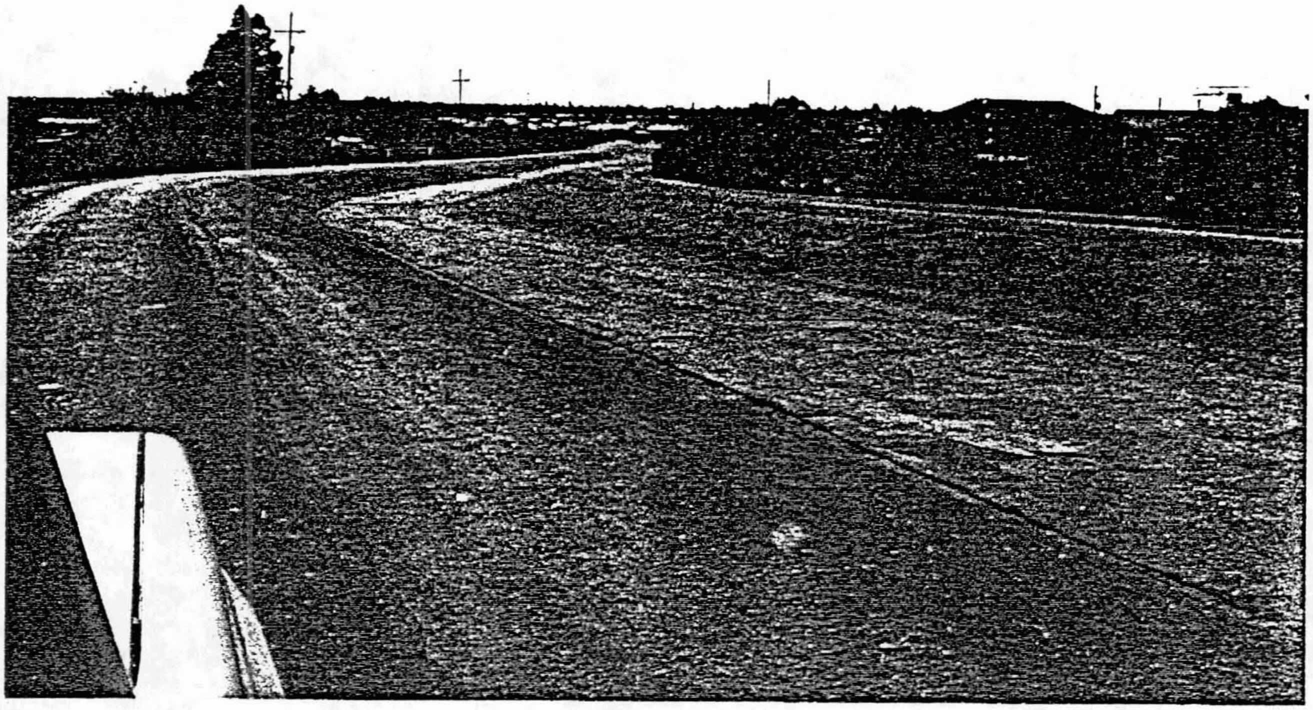
Channel bank erosion, Rillito River upstream of First Avenue

TYPICAL BRIDGE ABUTMENT
PROTECTION AND CHANNEL IMPROVEMENT
EXAMPLE PROJECT INA ROAD
BRIDGE AT CANADA DEL ORO WASH

TYPICAL LOCAL DRAINAGE

CHANNELS CONSTRUCTED

WITH SOIL CEMENT

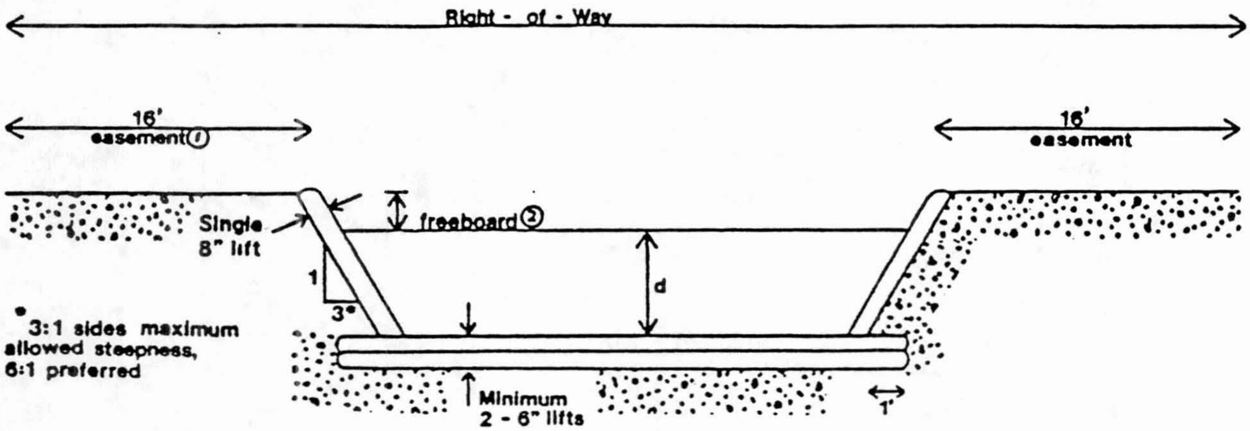


Soil cement drainageway at Sellarole Road

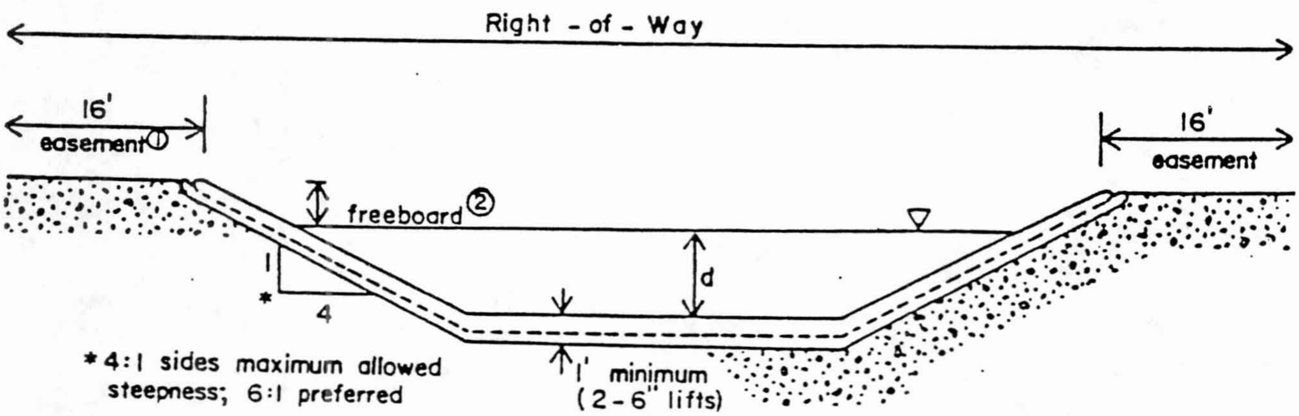
Soil cement drainageway at Sellarole Road

LOCAL DRAINAGE
EXAMPLES OF SOIL CEMENT

CHANNEL CROSS-SECTIONS



Soil Cement Lined Channel
For $v_{100} \leq 15$ fps,
Frequency Of Flow Less Than
5 to 6 Times A Year



Soil Cement Lined Channel
(Excluding Control Structures)

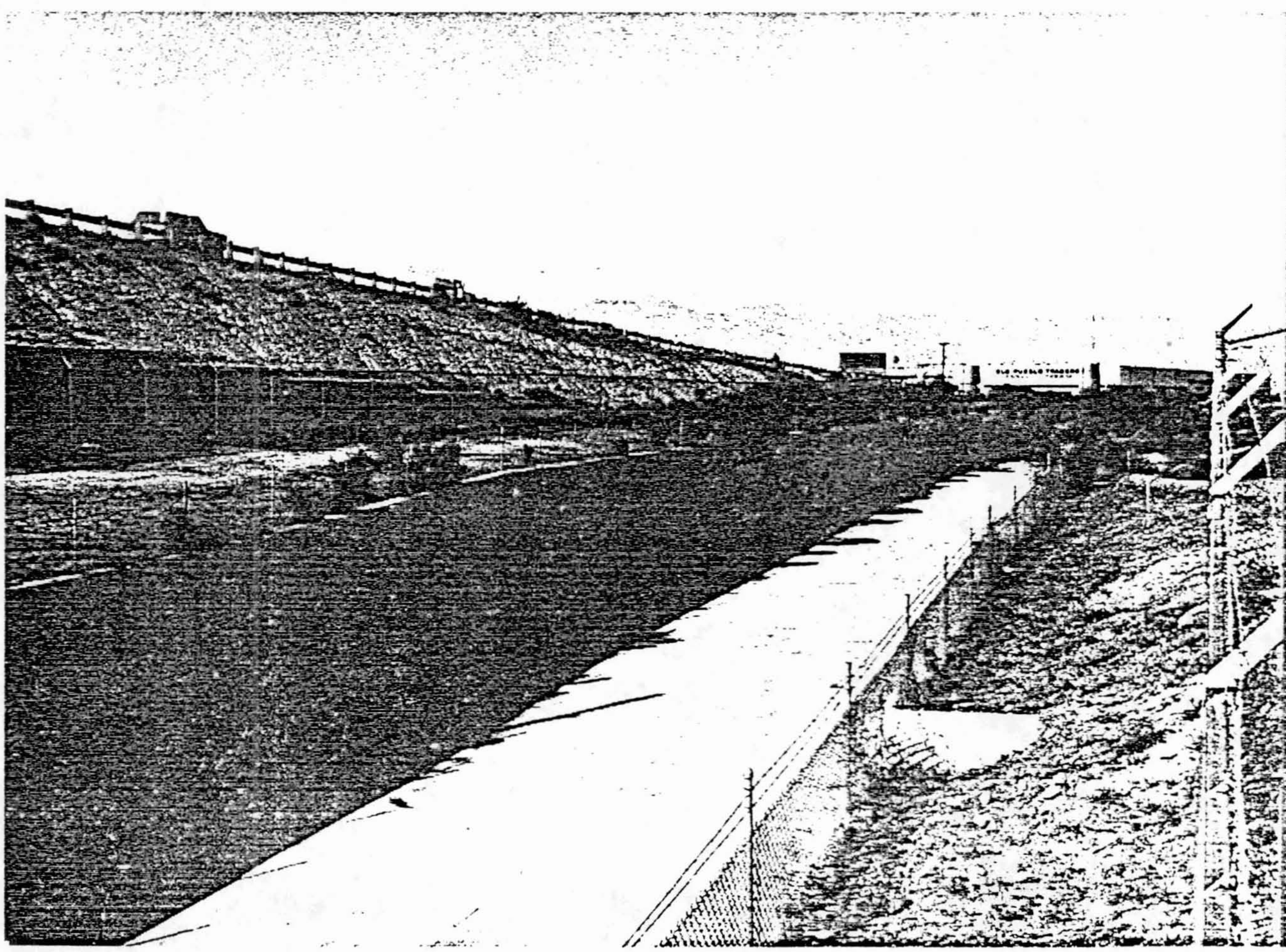
① Where bottom width > 20' only one 16' easement is required.

② $F < 0.86$ Freeboard = 1' minimum*

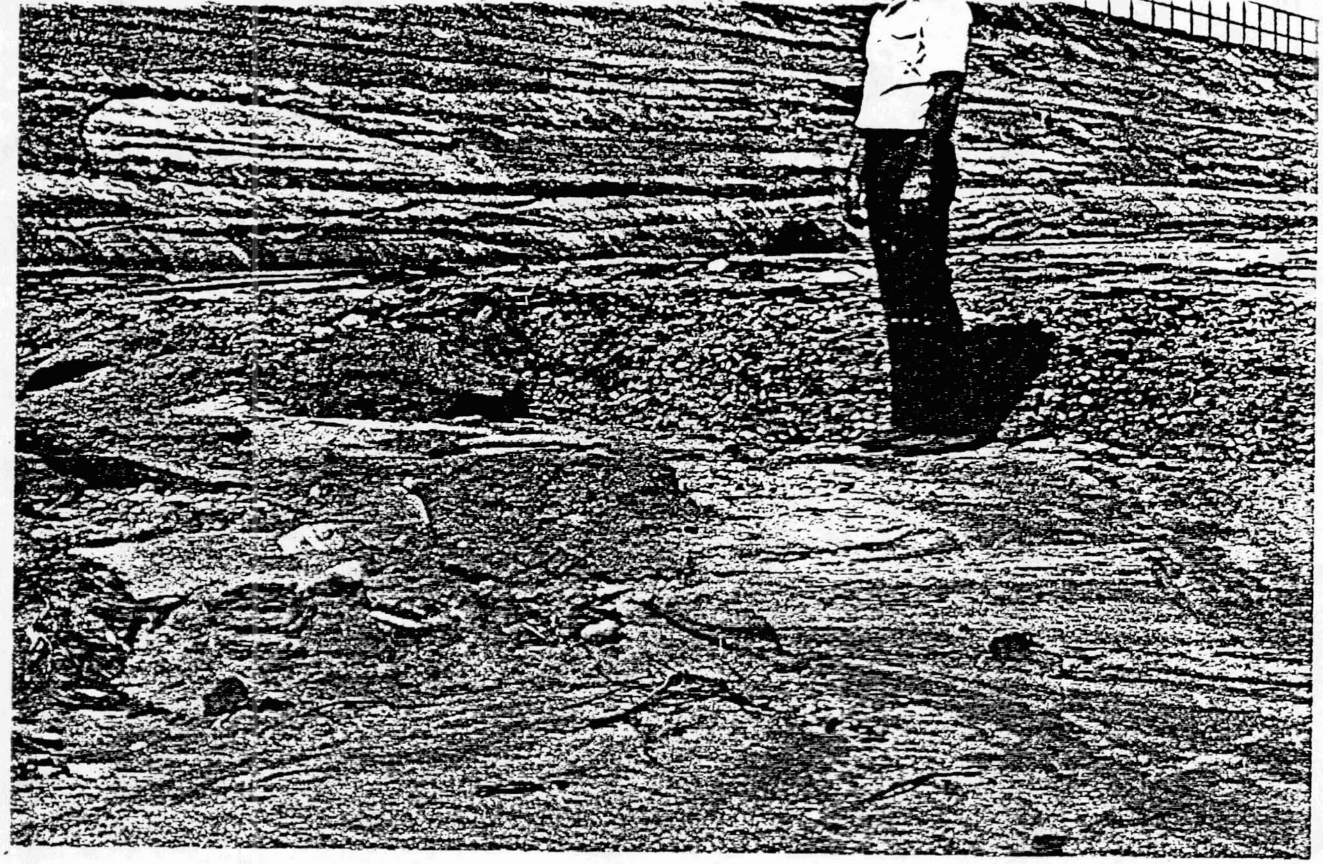
$F \geq 0.86$ Freeboard = $\frac{1}{6}\left[d + \frac{v^2}{2g}\right]$; 1' minimum

* For tranquil flow, Freeboard = 0.25d, when $d \leq 4$ feet.

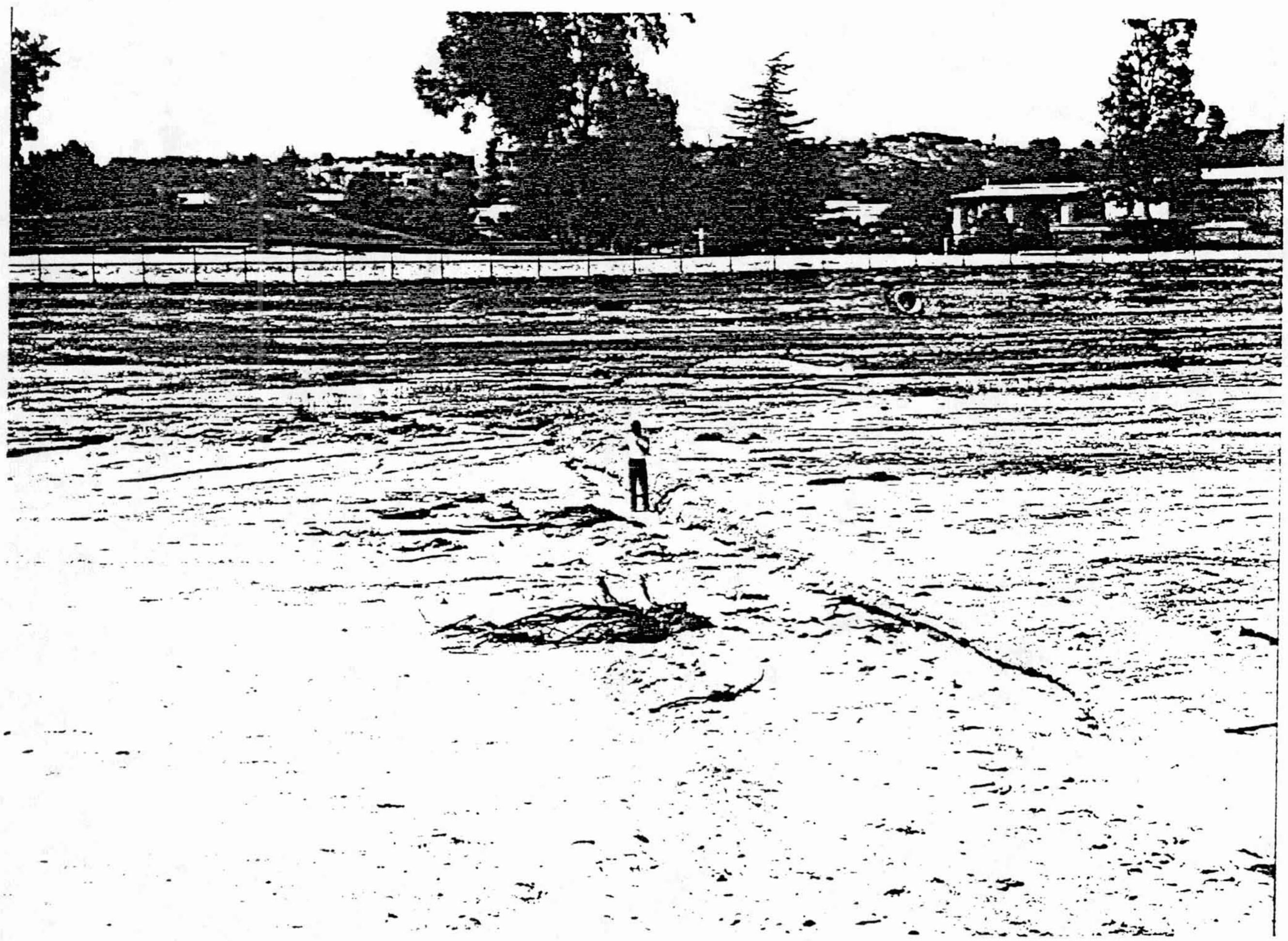
Julian Wash concrete lined channel at Palo Verde Road



Julian Wash concrete lined channel at Palo Verde Road



Soil cement bank protection and grade control, Canada Del Oro Wash

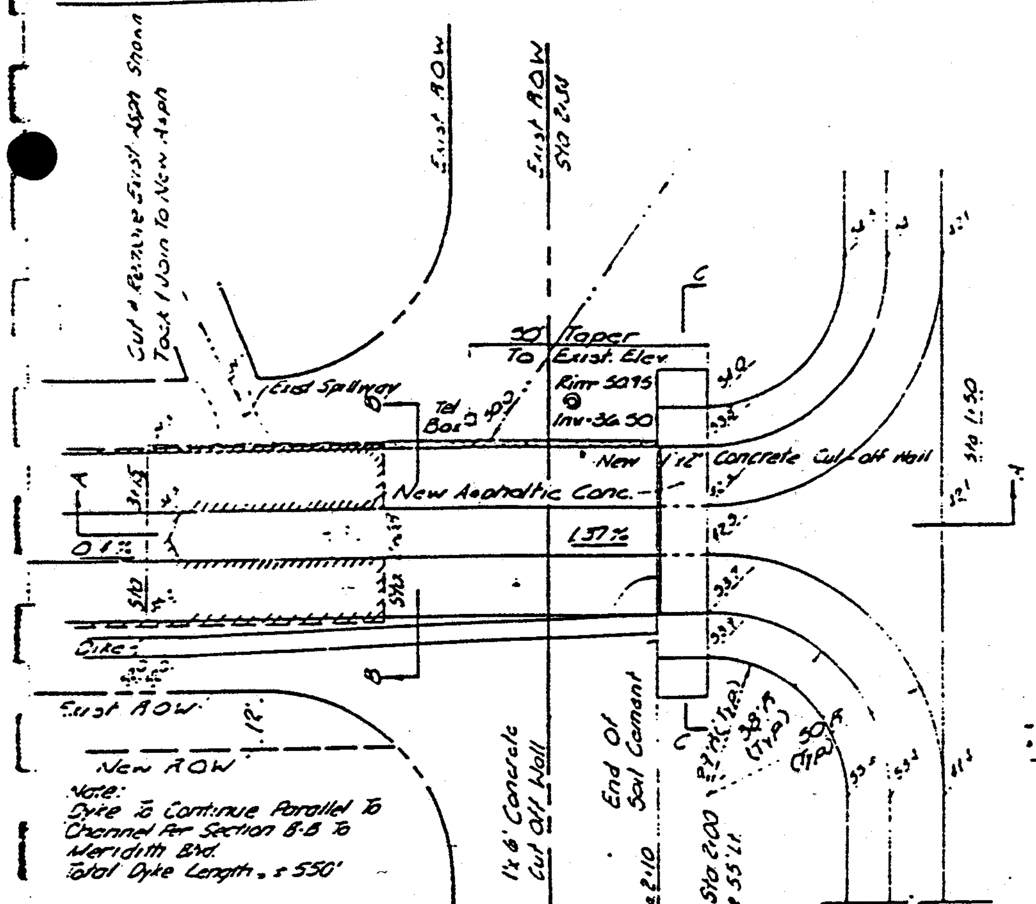


Soil cement bank protection and grade control, Canada Del Oro Wash

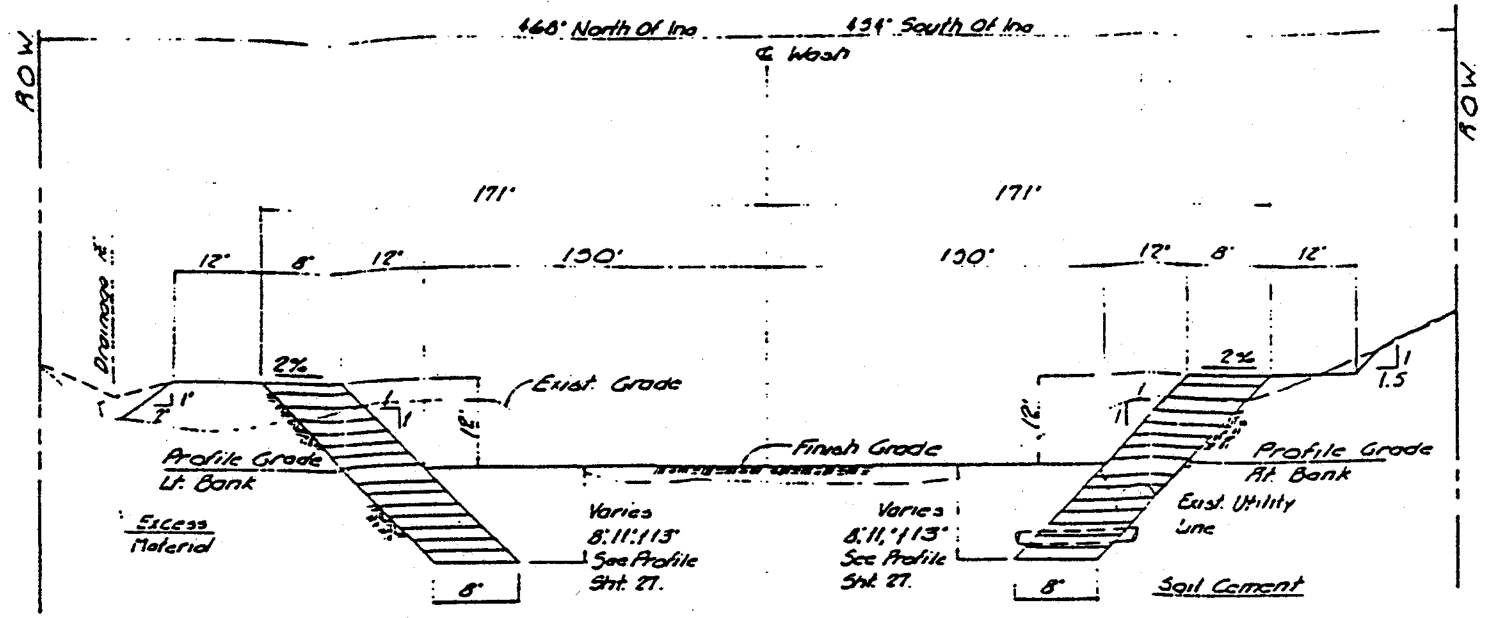
GRADE CONTROL DROP STRUCTURE DESIGN

EXAMPLE MAGEE ROAD

AT CANADA DEL ORO WASH



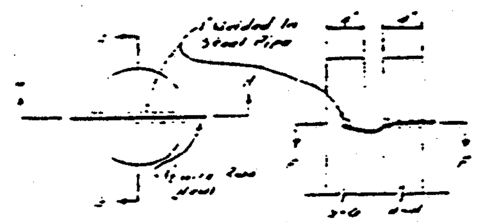
DETAIL I
No Scale



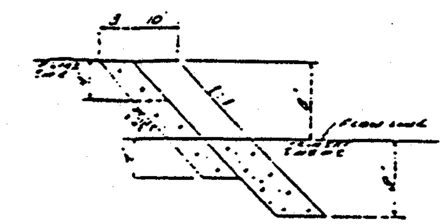
CAÑADA DEL ORO SECTION
No Scale

CAÑADA DEL ORO WASH CHANNELIZATION APPROXIMATE EARTHWORK QUANTITIES	
Excavation *	* Embankment
Drainage Exc. 8,993 C.Y.	Drainage Embankment 1,954 C.Y.
	Expanded 2,299 C.Y. (15% Average Shrinkage)
TOTAL EXCAVATION 8,993 C.Y.	EXCESS ** 6,694 C.Y.
	Soil Cement Required 10,035 C.Y.

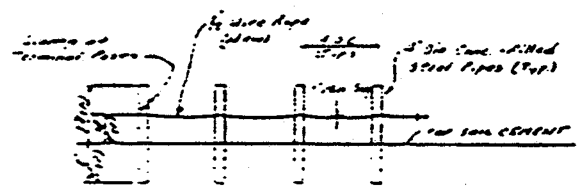
* These quantities are exclusive of soil cement bank protection requirements.
** Excess material shall be used for roadway embankment as required - any additional excess material shall be placed as shown on these drawings.



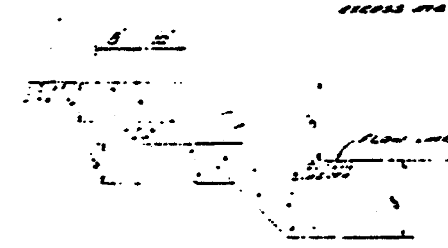
SECTION F-F



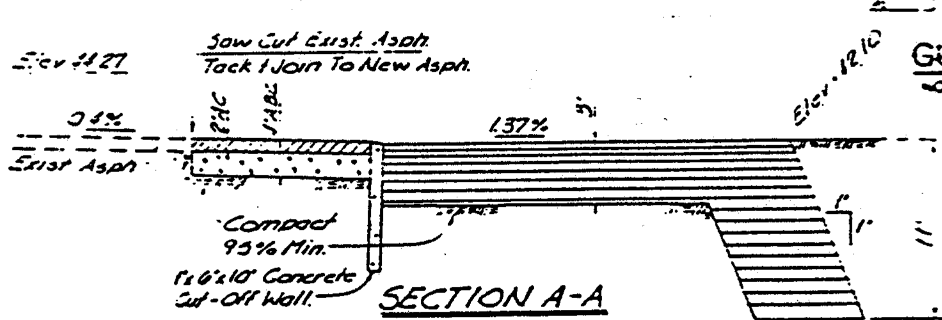
SECTION D-D
NO SCALE



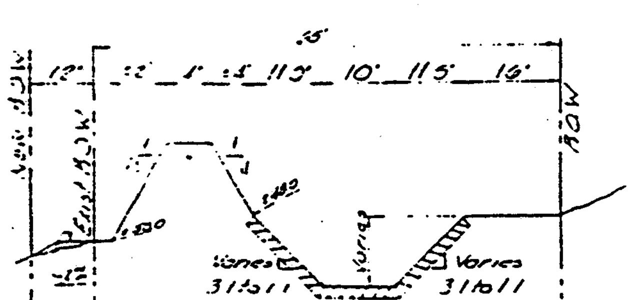
GUARD POST DETAIL
NO SCALE



SECTION E-E
NO SCALE

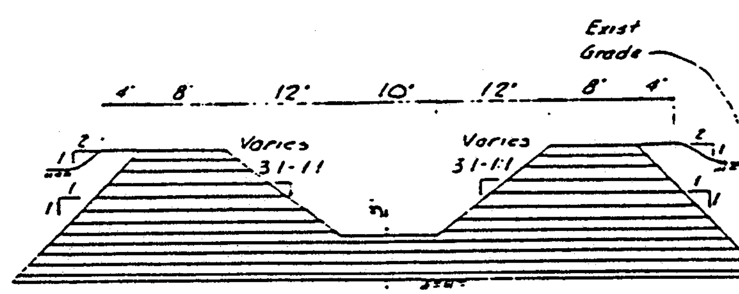


SECTION A-A
No Scale



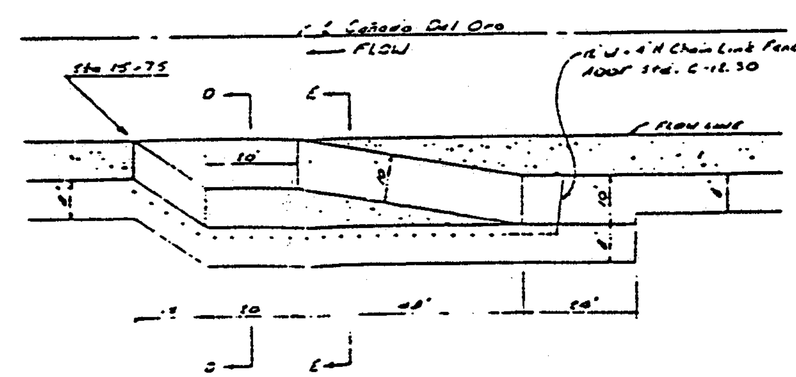
SECTION B-B
No Scale

* Top Dike Elev = 20 (Continuous) Compact To 95% (South Bank Only)

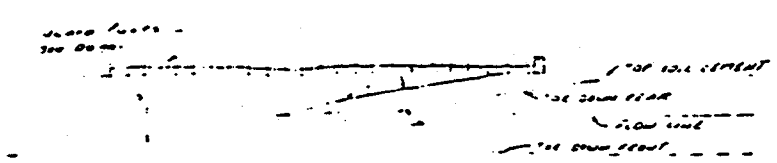


SECTION C-C
No Scale

NOTES:
1. Embankment To Be Compacted To 95% Of Maximum Density As Determined By Ariz. Highway Department Test Method 225 & 227a.
2. Excess Material To Be Placed As Shown On Plans Shall Be Compacted To 90% Maximum Density.
3. Construct Soil-Cement Bank Protection In Max. 8" Lifts (Compacted). Trim Face To A Smooth 1:1 Surface.
4. Refer To Sheet 32 Of The Bridge Plans For Soil Cement Bank Protection Thru Bridge Abutment Area.
5. All Quantities Are Approximate And Should Be Verified By The Contractor To His Own Satisfaction.
6. Soil Cement Shall Be Hand Placed & Compacted In Vicinity Of Any Existing Utilities Which Will Protrude Through Proposed Bank Protection. Hand Placement & Compaction Shall Take Place To A Minimum Of 1' Above Top Utility Line.
7. Soil Cement Shall Be Placed 9' Wide (Pay Width) and Trimmed To 8' Width



PLAN



ELEVATION

CHANNEL ACCESS RAMP
NO SCALE

INA ROAD - THORNYDALE RD. TO MONA LISA RD., W.O. 4B INA 3

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

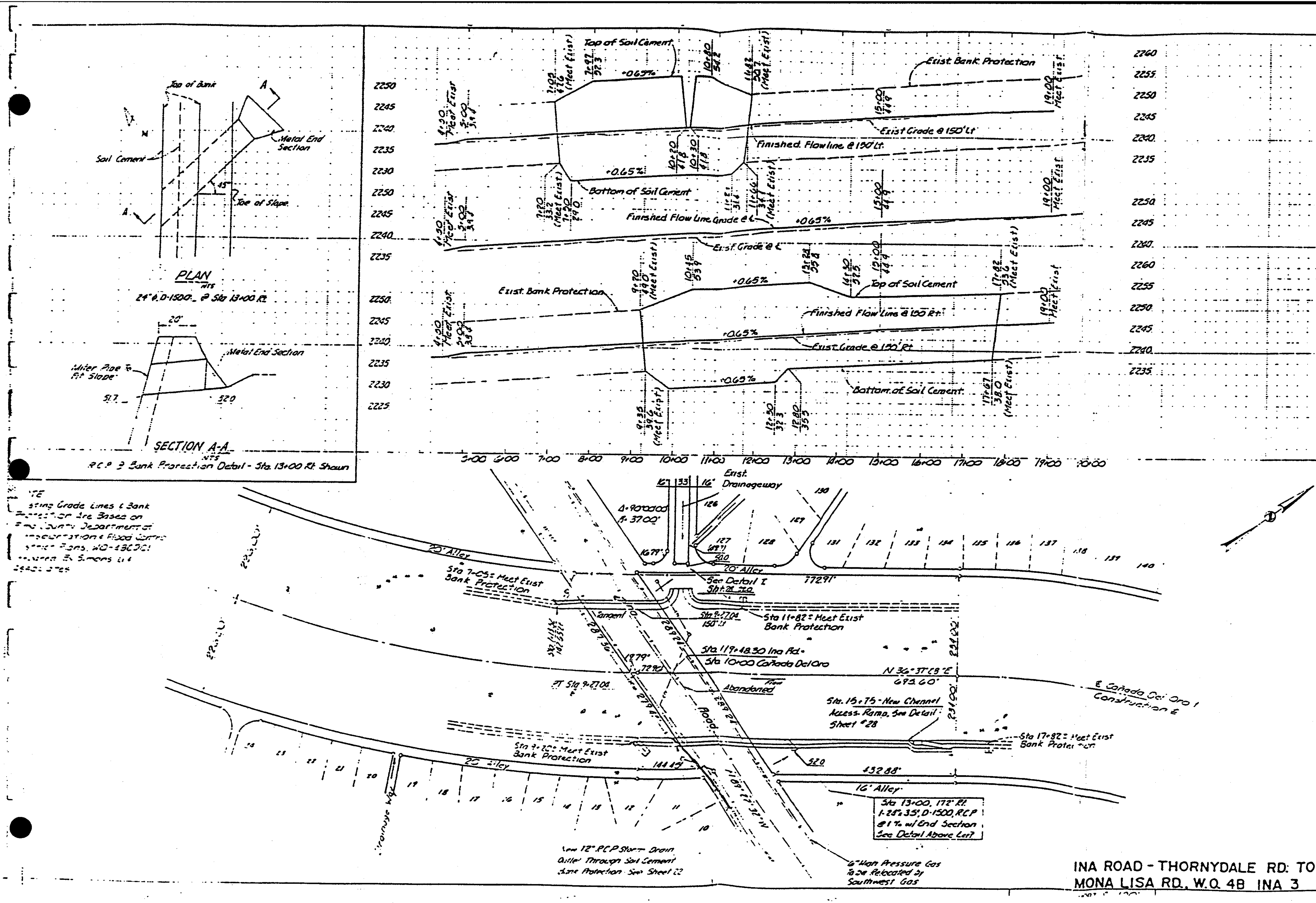
C. H. HUCKELBERRY R.L.S., P.E. DIRECTOR

SOIL CEMENT BANK PROTECTION DETAILS

NO	REVISION	DESCRIPTION	DIV/SECT ENGR.	DATE

		DATE
DESIGNED		
DRAWN		
CHECKED		
PROJ. ENGR.		

FINICAL & DOMBROWSKI ARCHITECTS & ENGINEERS TUCSON ARIZONA

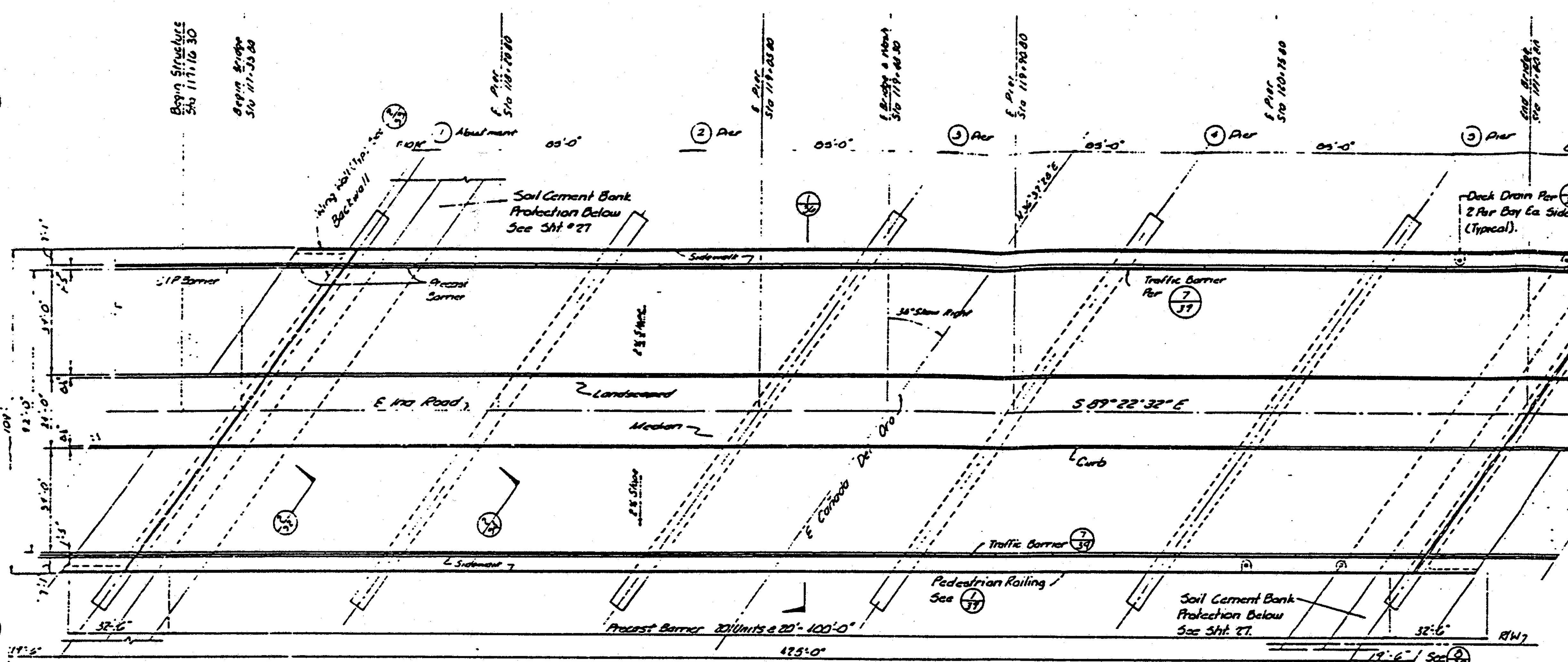


INA ROAD - THORNYDALE RD. TO MONA LISA RD. W.O. 4B INA 3

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

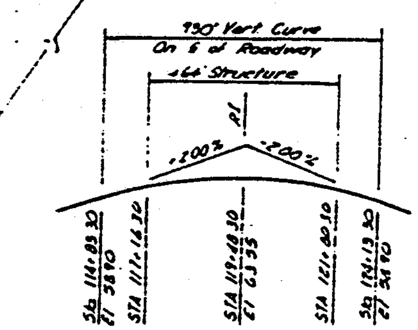
SOIL CEMENT BANK PROTECTION PLAN

C. H. HUCKELBERRY, M.S., P.E. DIRECTOR



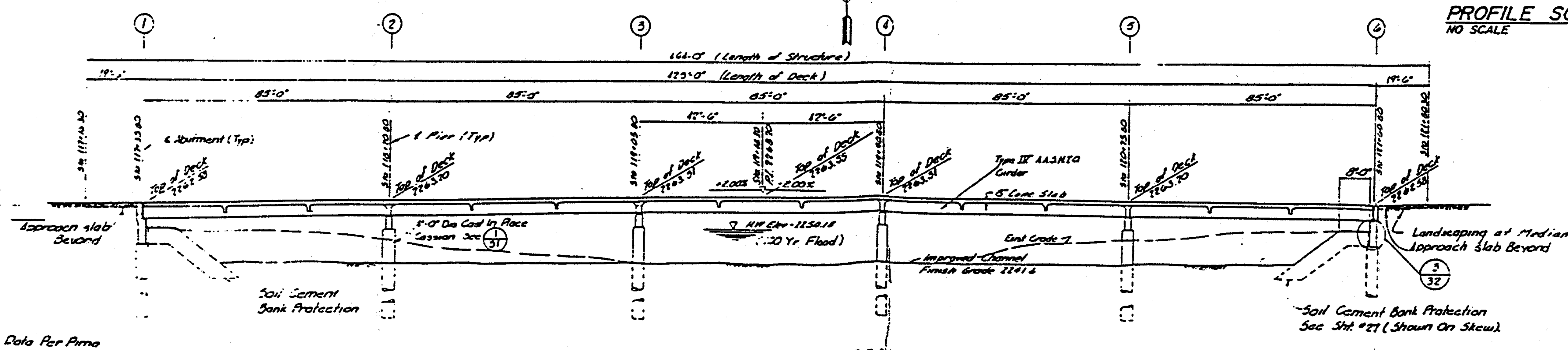
BRIDGE PLAN

SCALE: 1" = 20'



PROFILE SCHEMATIC

NO SCALE



PROFILE ON CENTERLINE OF ROADWAY

SCALE: 1" = 20'

Stream Data Per Pima County D.O.T. / F.C.D.
Design: 100 Year Flood
Flow: 33,000 c.f.s.
Average Velocity 11.06 ft/Sec

INA ROAD - THORNYDALE RD. TO MONA LISA RD., W.O. 48 INA 3

SHEET 29 OF 40

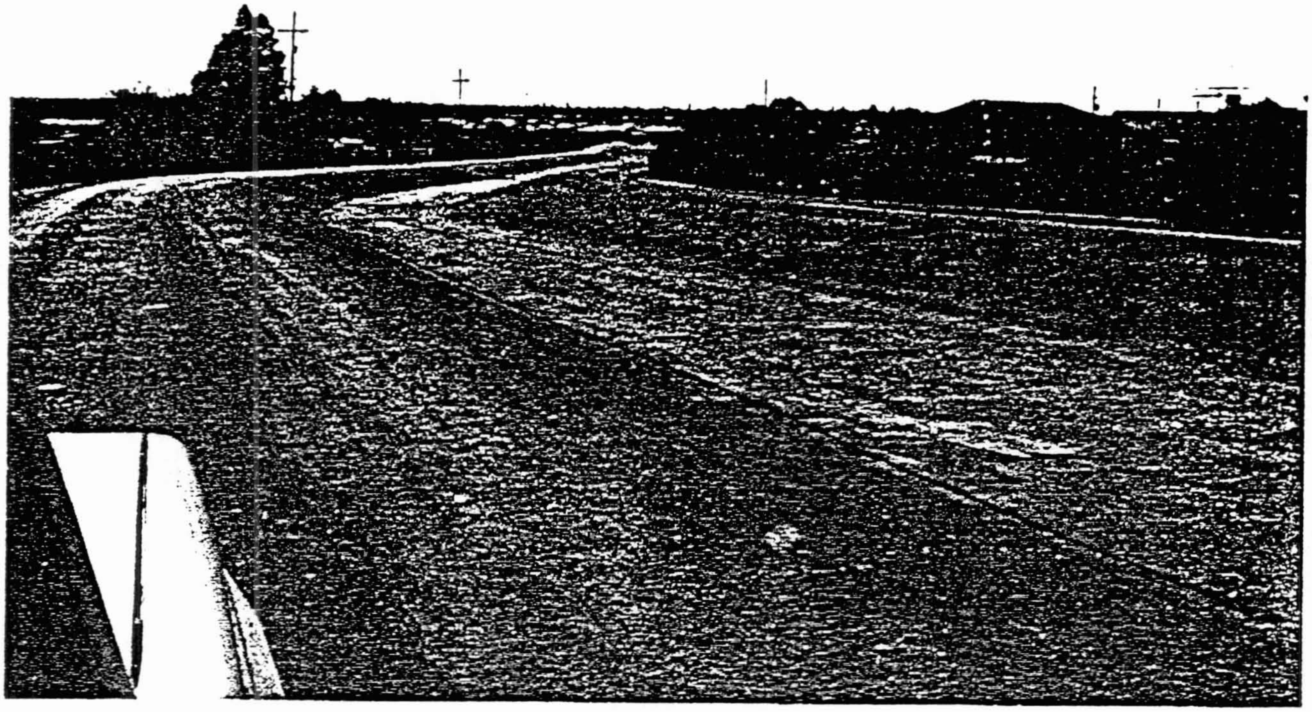
PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

C.H. HUCKELBERRY R.L.S.,PE DIRECTOR

PLAN & PROFILE

FINCAL & DOMBROWSKI ARCHITECTS & ENGINEERS TUCSON ARIZONA

TYPICAL LOCAL DRAINAGE
CHANNELS CONSTRUCTED
WITH SOIL CEMENT

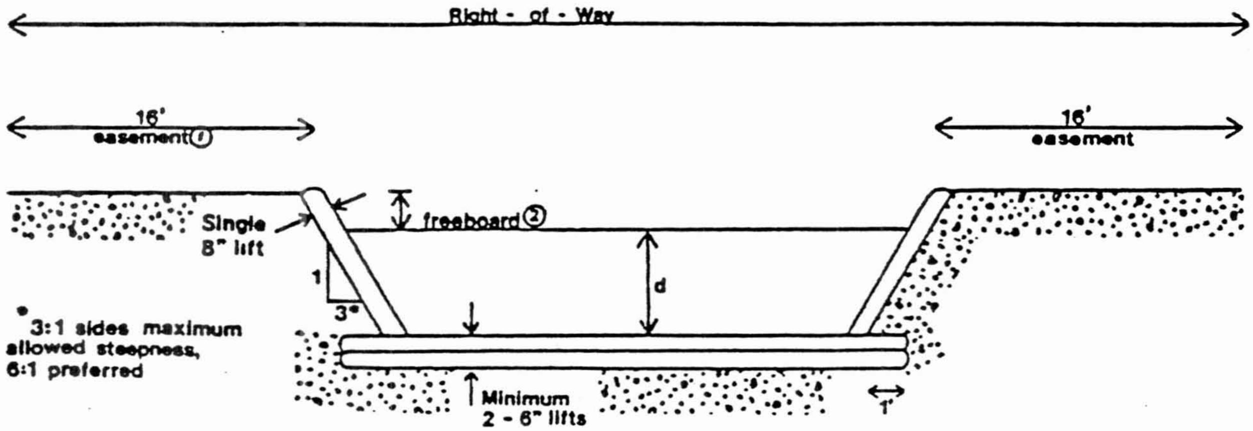


Soil cement drainageway at Sellarole Road

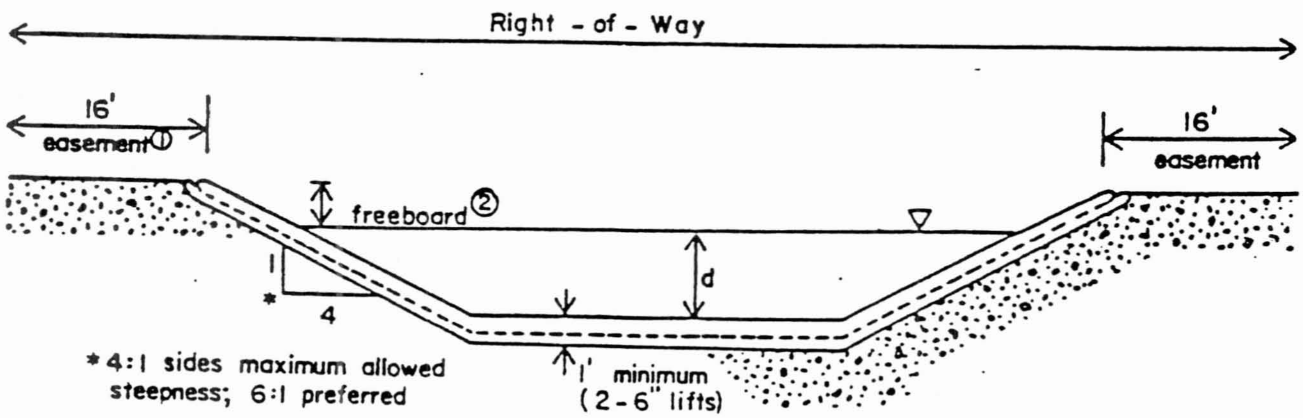
Soil cement drainageway at Sellarole Road

LOCAL DRAINAGE
EXAMPLES OF SOIL CEMENT

CHANNEL CROSS-SECTIONS



Soil Cement Lined Channel
For $v_{100} \leq 15$ fps,
Frequency Of Flow Less Than
5 to 6 Times A Year



Soil Cement Lined Channel
(Excluding Control Structures)

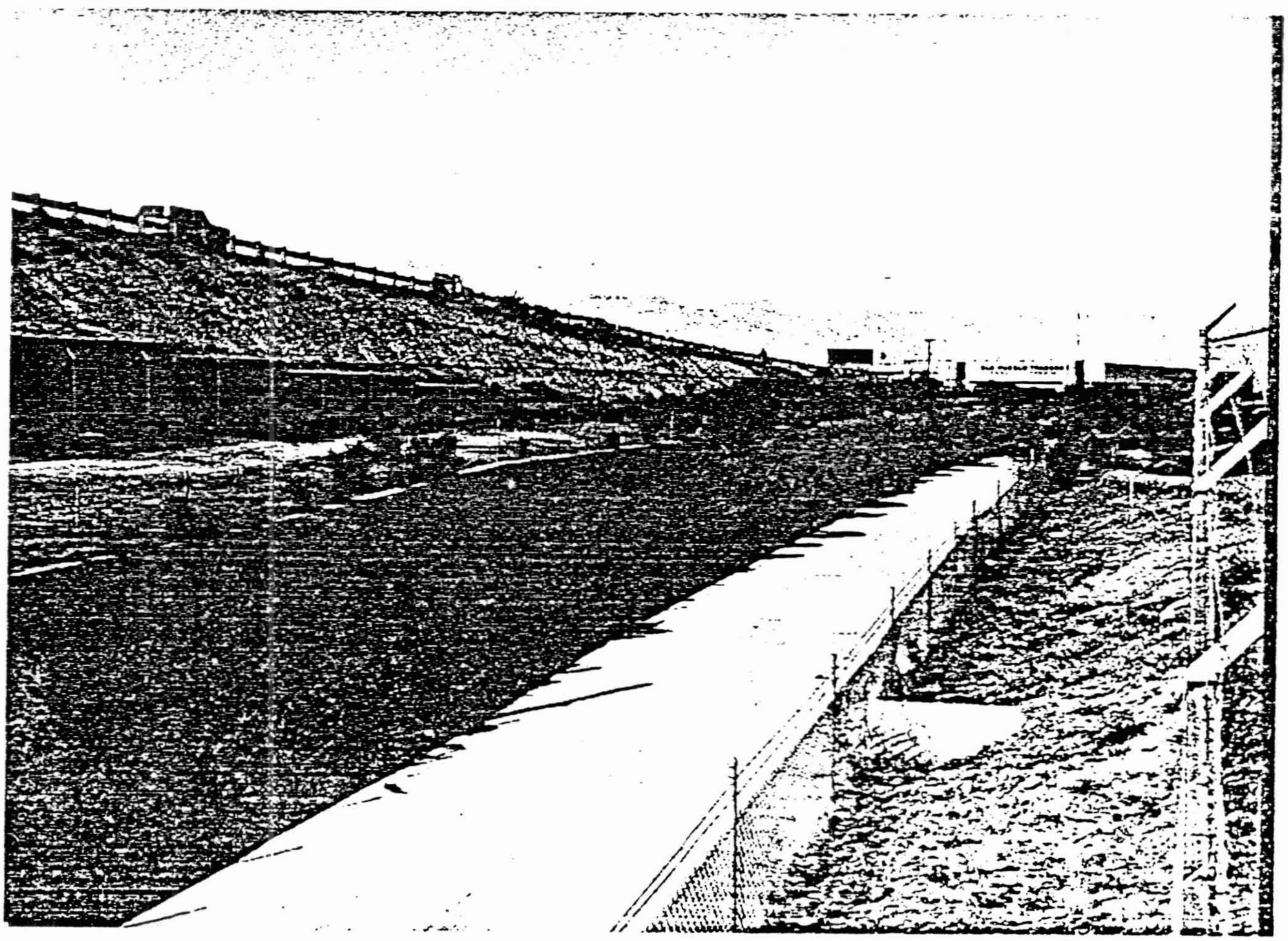
① Where bottom width > 20' only one 16' easement is required.

② $F < 0.86$ Freeboard = 1' minimum*

$F \geq 0.86$ Freeboard = $\frac{1}{6}\left[d + \frac{v^2}{2g}\right]$; 1' minimum

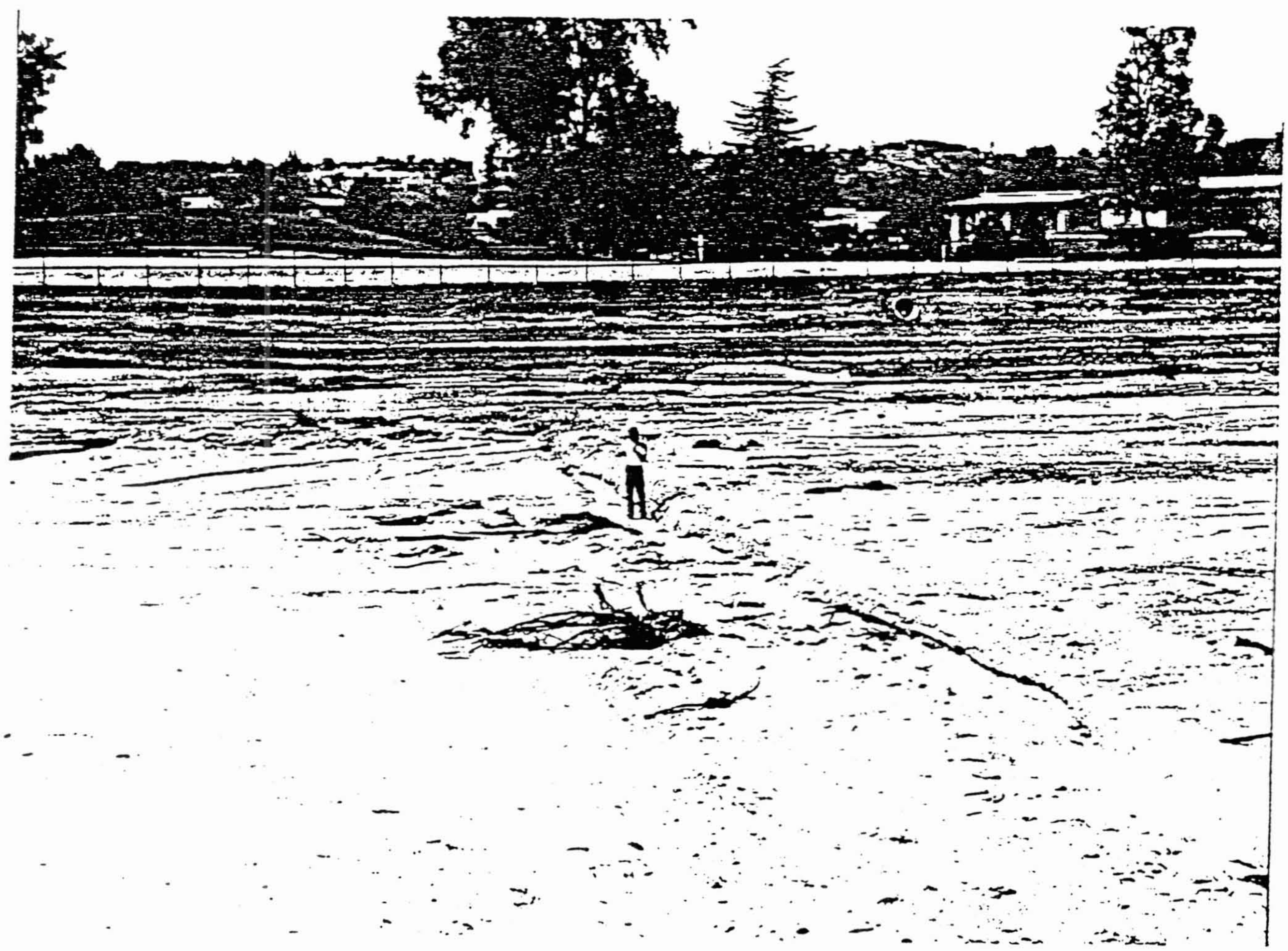
*For tranquil flow, Freeboard = 0.25d, when $d \leq 4$ feet.

Julian Wash concrete lined channel at Palo Verde Road



Julian Wash concrete lined channel at Palo Verde Road

Soil cement bank protection and grade control, Canada Del Oro Wash

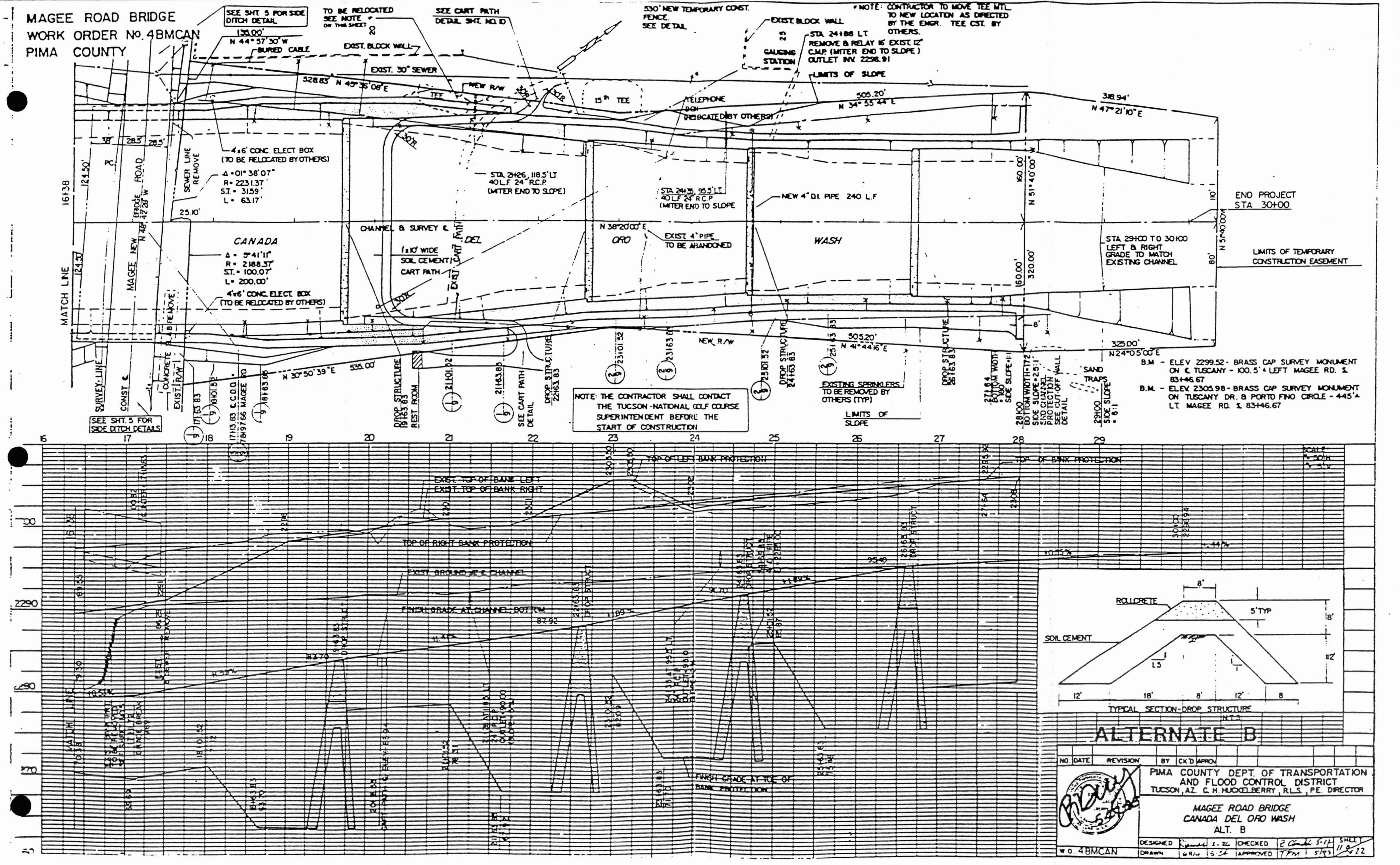


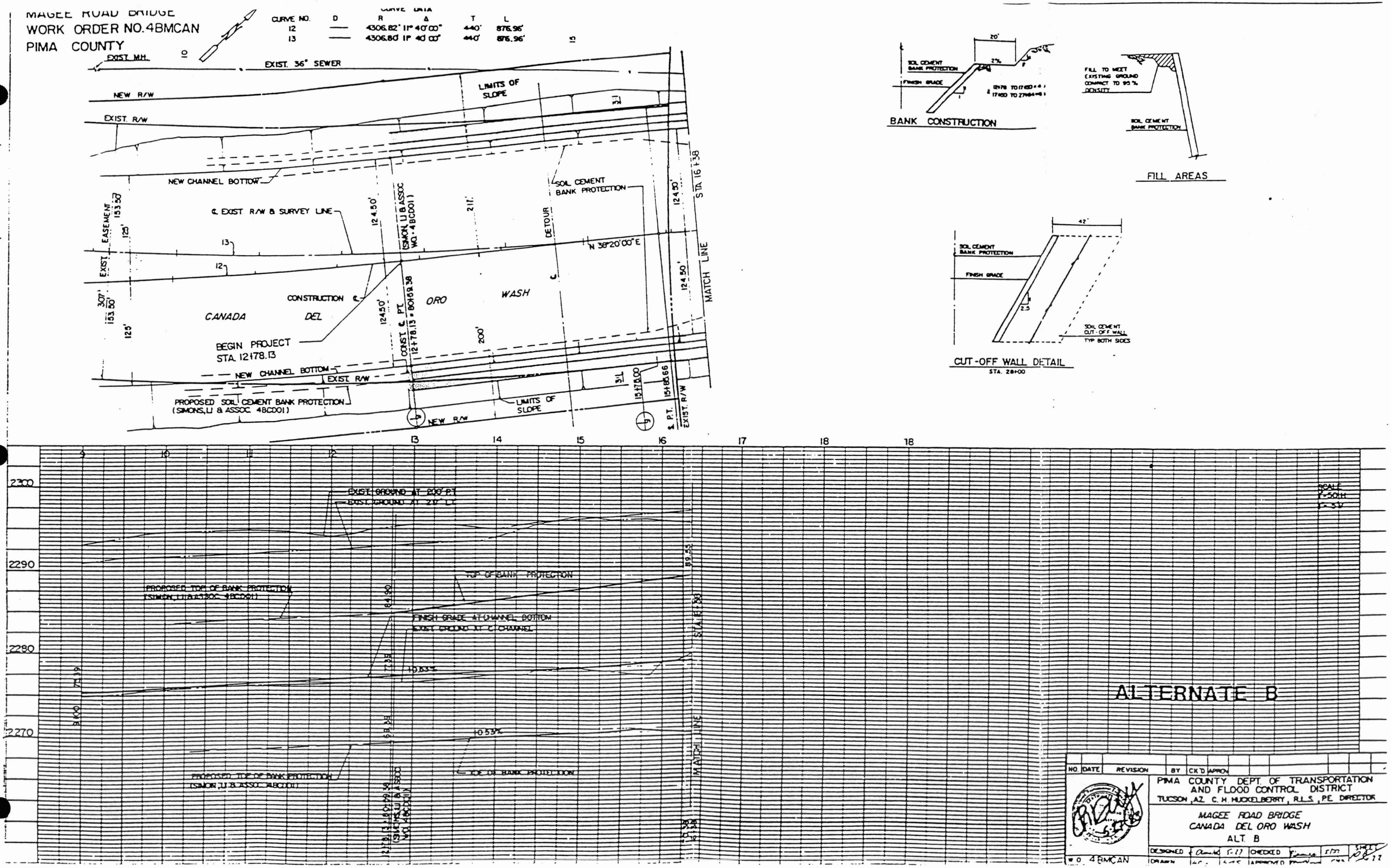
Soil cement bank protection and grade control, Canada Del Oro Wash

GRADE CONTROL DROP STRUCTURE DESIGN

EXAMPLE MAGEE ROAD

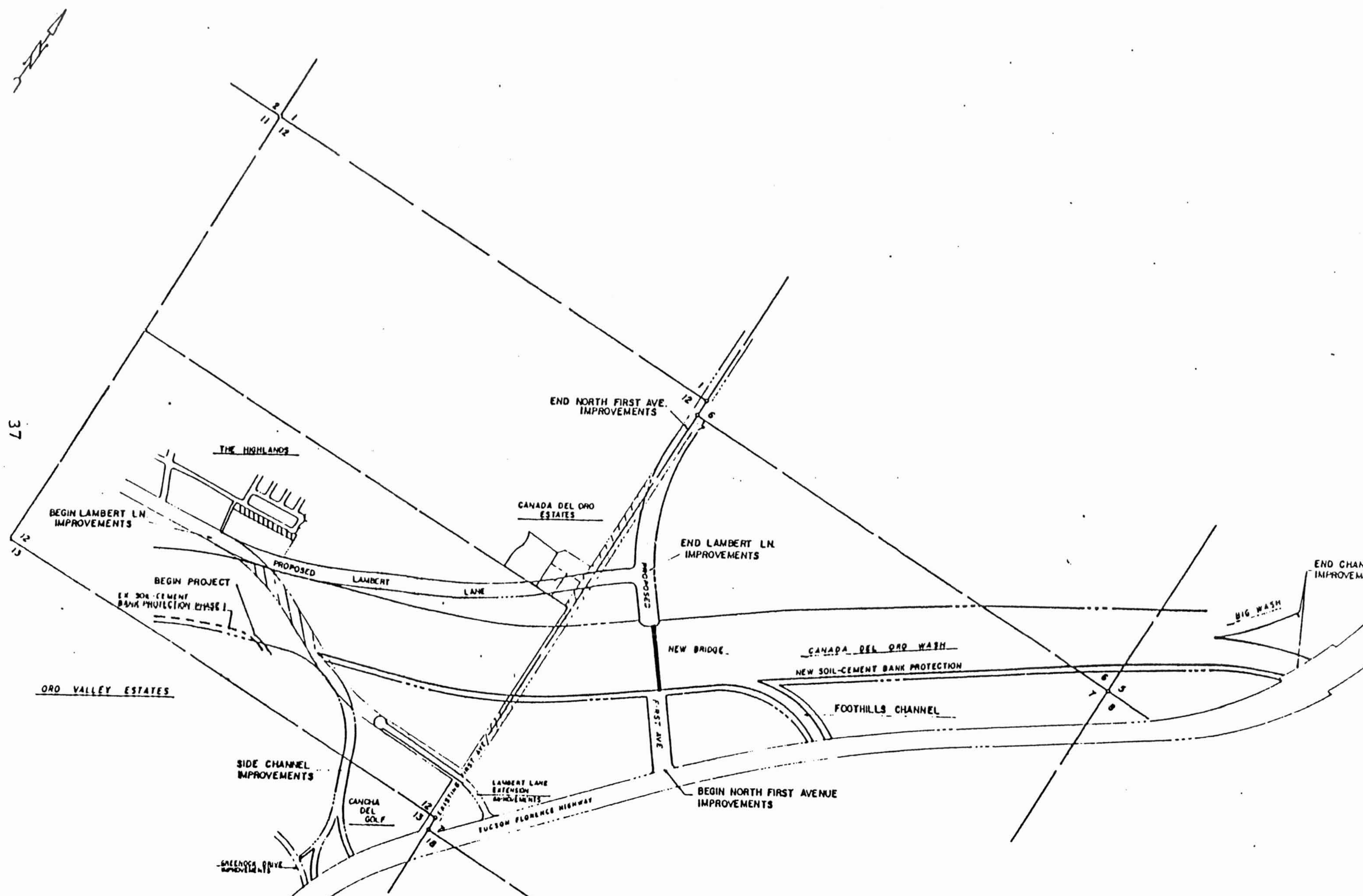
AT CANADA DEL ORO WASH


MAGEE ROAD BRIDGE
WORK ORDER No. 4BMCAN
PIMA COUNTY
SEE SHT. 5 FOR SIDE DITCH DETAIL
TO BE RELOCATED SEE NOTE ON THIS SHEET
SEE CART PATH DETAIL SHT. NO. 10
530' NEW TEMPORARY CONST. FENCE. SEE DETAIL
NOTE: CONTRACTOR TO MOVE TEE MTL. TO NEW LOCATION AS DIRECTED BY THE ENGR. TEE CST. BY OTHERS.
EXIST. BLOCK WALL
STA. 24+88 LT REMOVE & RELAY 16 EXIST. 12" C.M.P. (MITER END TO SLOPE) OUTLET INV. 2298.91
GAUGING STATION
LIMITS OF SLOPE
BURIED CABLE
EXIST. 30" SEWER
NEW R/W
15th TEE
TELEPHONE (RELOCATED BY OTHERS)
4'x6' CONC. ELECT. BOX (TO BE RELOCATED BY OTHERS)
Δ = 01° 38' 07"
R = 2231.37'
S.T. = 31.59'
L = 63.17'
STA. 21+26, 118.5' LT 40 L.F. 24" R.C.P. (MITER END TO SLOPE)
STA. 24+35, 95.5' LT 40 L.F. 24" R.C.P. (MITER END TO SLOPE)
NEW 4" D.I. PIPE 240 L.F.
END PROJECT STA 30+00
CHANNEL & SURVEY ℄
CANADA DEL ORO WASH
Δ = 5° 41' 11"
R = 2188.37'
S.T. = 100.07'
L = 200.00'
4'x6' CONC. ELECT. BOX (TO BE RELOCATED BY OTHERS)
1'x10' WIDE SOIL CEMENT CART PATH
EXIST. 4" PIPE TO BE ABANDONED
STA 29+00 TO 30+00 LEFT & RIGHT GRADE TO MATCH EXISTING CHANNEL
LIMITS OF TEMPORARY CONSTRUCTION EASEMENT
MATCH LINE
NEW R/W
B.M. - ELEV. 2299.52 - BRASS CAP SURVEY MONUMENT ON ℄ TUSCANY - 100.5' LEFT MAGEE RD. STA. 83+46.67
B.M. - ELEV. 2305.98 - BRASS CAP SURVEY MONUMENT ON TUSCANY DR. & PORTO FINO CIRCLE - 44.5' LT. MAGEE RD. STA. 83+46.67
NOTE: THE CONTRACTOR SHALL CONTACT THE TUCSON NATIONAL GOLF COURSE SUPERINTENDENT BEFORE THE START OF CONSTRUCTION
EXISTING SPRINKLERS TO BE REMOVED BY OTHERS (TYP)
LIMITS OF SLOPE
SAND TRAPS
DROP STRUCTURE
REST ROOM
SEE CART PATH DETAIL
TOP OF LEFT BANK PROTECTION
EXIST. TOP OF BANK LEFT
EXIST. TOP OF BANK RIGHT
TOP OF RIGHT BANK PROTECTION
EXIST. GROUND AT ℄ CHANNEL
FINISH GRADE AT CHANNEL BOTTOM
FINISH GRADE AT TOE OF BANK PROTECTION
TOP OF BANK PROTECTION
ROLLCRETE
SOIL CEMENT
5' TYP
TYPICAL SECTION - DROP STRUCTURE N.T.S.
ALTERNATE B
NO. DATE REVISION BY CK'D APPROV.
PIMA COUNTY DEPT. OF TRANSPORTATION AND FLOOD CONTROL DISTRICT
TUCSON, AZ. C. H. HUCKELBERRY, R.L.S., P.E. DIRECTOR
MAGEE ROAD BRIDGE
CANADA DEL ORO WASH
ALT. B
W.O. 4BMCAN
DESIGNED
DRAWN
CHECKED
APPROVED
SHEET


MAGEE ROAD BRIDGE
WORK ORDER NO. 4BMCAN
PIMA COUNTY
CURVE DATA
CURVE NO. D R Δ T L
12 — 4306.82' 11° 40' 00" 440' 876.96'
13 — 4306.80 11° 40' 00" 440' 876.96'
EXIST. M.H.
EXIST. 36" SEWER
NEW R/W
EXIST. R/W
LIMITS OF SLOPE
NEW CHANNEL BOTTOM
SOIL CEMENT BANK PROTECTION
℄ EXIST. R/W & SURVEY LINE
EXIST. EASEMENT
N 38°20'00" E
DETOUR
CANADA DEL ORO WASH
CONSTRUCTION ℄
BEGIN PROJECT STA. 12+78.13
CONST. ℄ P.T. 12+78.13 = BO 69.38
SIMON, LI & ASSOC. WO. 4BCD01
MATCH LINE
STA. 16+38
EXIST. R/W
PROPOSED SOIL CEMENT BANK PROTECTION (SIMONS, LI & ASSOC. 4BCD01)
UNITS OF SLOPE
NEW R/W
BANK CONSTRUCTION
SOIL CEMENT BANK PROTECTION
FINISH GRADE
FILL TO MEET EXISTING GROUND COMPACT TO 95% DENSITY
FILL AREAS
CUT-OFF WALL DETAIL
STA. 28+00
SOIL CEMENT CUT-OFF WALL TYP BOTH SIDES
2300
2290
2280
2270
EXIST. GROUND AT 200' RT.
EXIST. GROUND AT 20' LT.
TOP OF BANK PROTECTION
PROPOSED TOP OF BANK PROTECTION (SIMON, LI & ASSOC. 4BCD01)
FINISH GRADE AT CHANNEL BOTTOM
EXIST. GROUND AT ℄ CHANNEL
+0.53%
SCALE 1" = 50' H 1" = 5' V
ALTERNATE B
NO. DATE REVISION BY CK'D APPROV.
PIMA COUNTY DEPT. OF TRANSPORTATION AND FLOOD CONTROL DISTRICT
TUCSON, AZ. C. H. HUCKELBERRY, R.L.S., P.E. DIRECTOR
MAGEE ROAD BRIDGE
CANADA DEL ORO WASH
ALT. B
DESIGNED DRAWN CHECKED APPROVED
W.O. 4BMCAN

COMPREHENSIVE FLOOD CONTROL

ORO VALLEY FLOOD CONTROL LEVEE



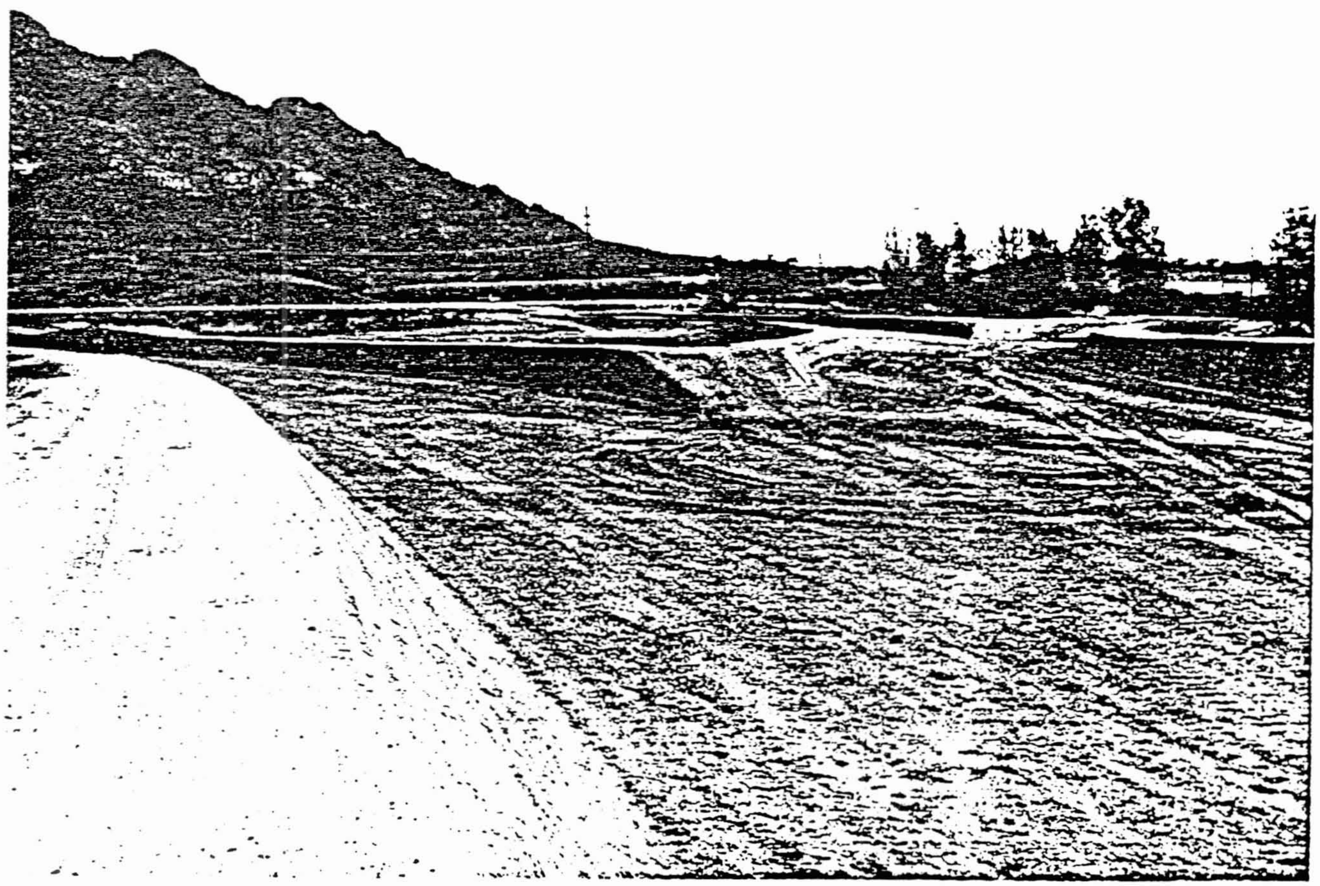
PROJECT OVERVIEW
-Phase II

37

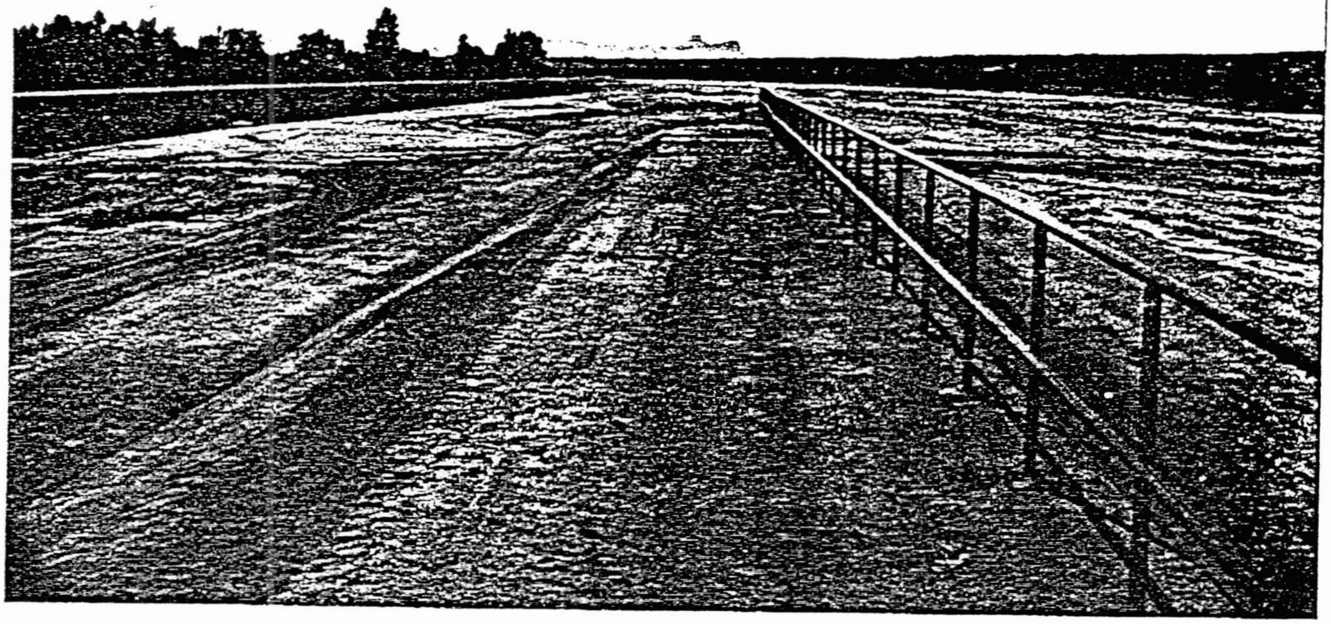
Original unprotected channel banks, Canada Del Oro Wash and Oro Valley Wash

Erosion along original earthen levee at Oro Valley and Canada del Oro Wash, prior to improvements

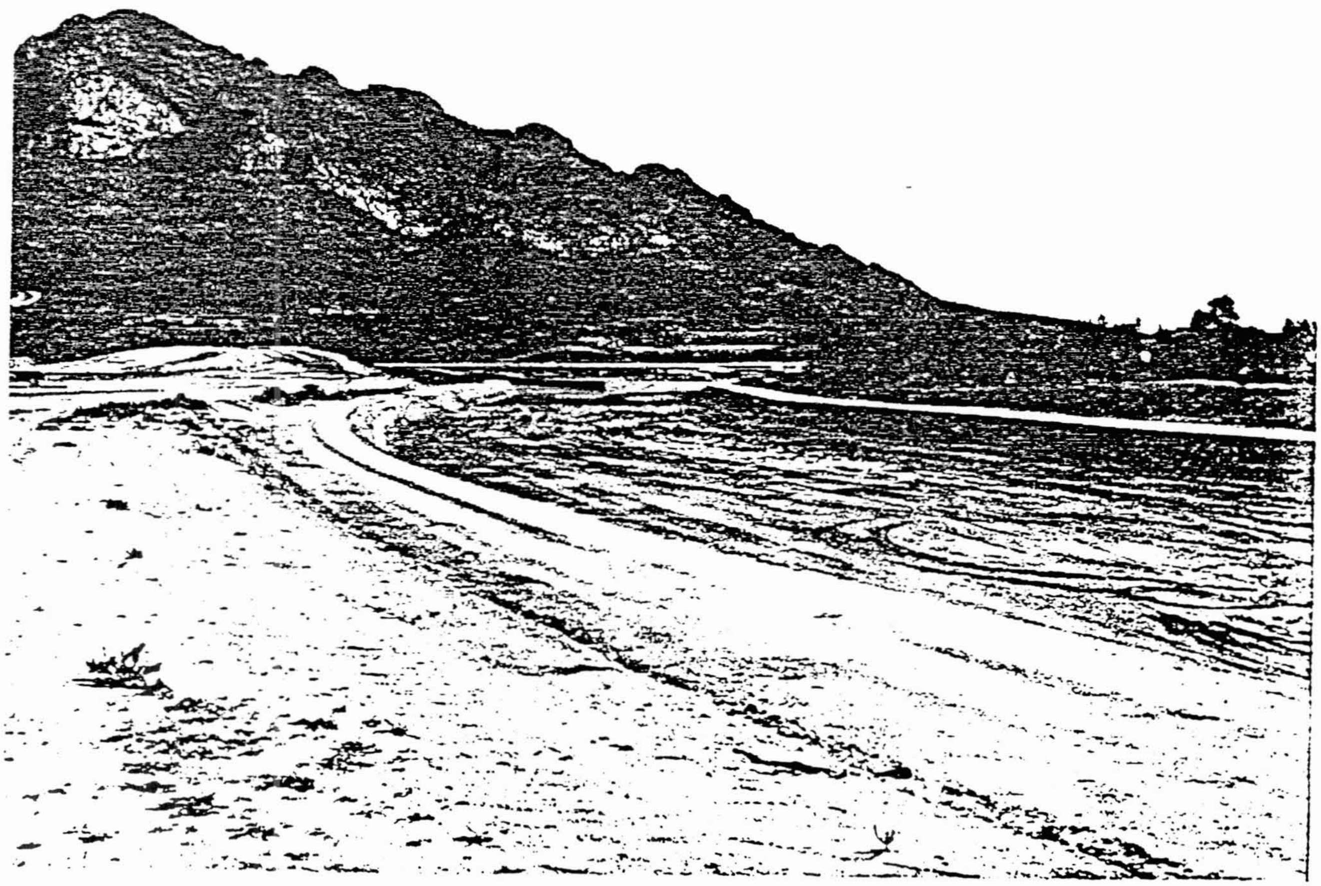
Soil Cement bank protection at confluence of Rooney and Pucsh Ridge Washes



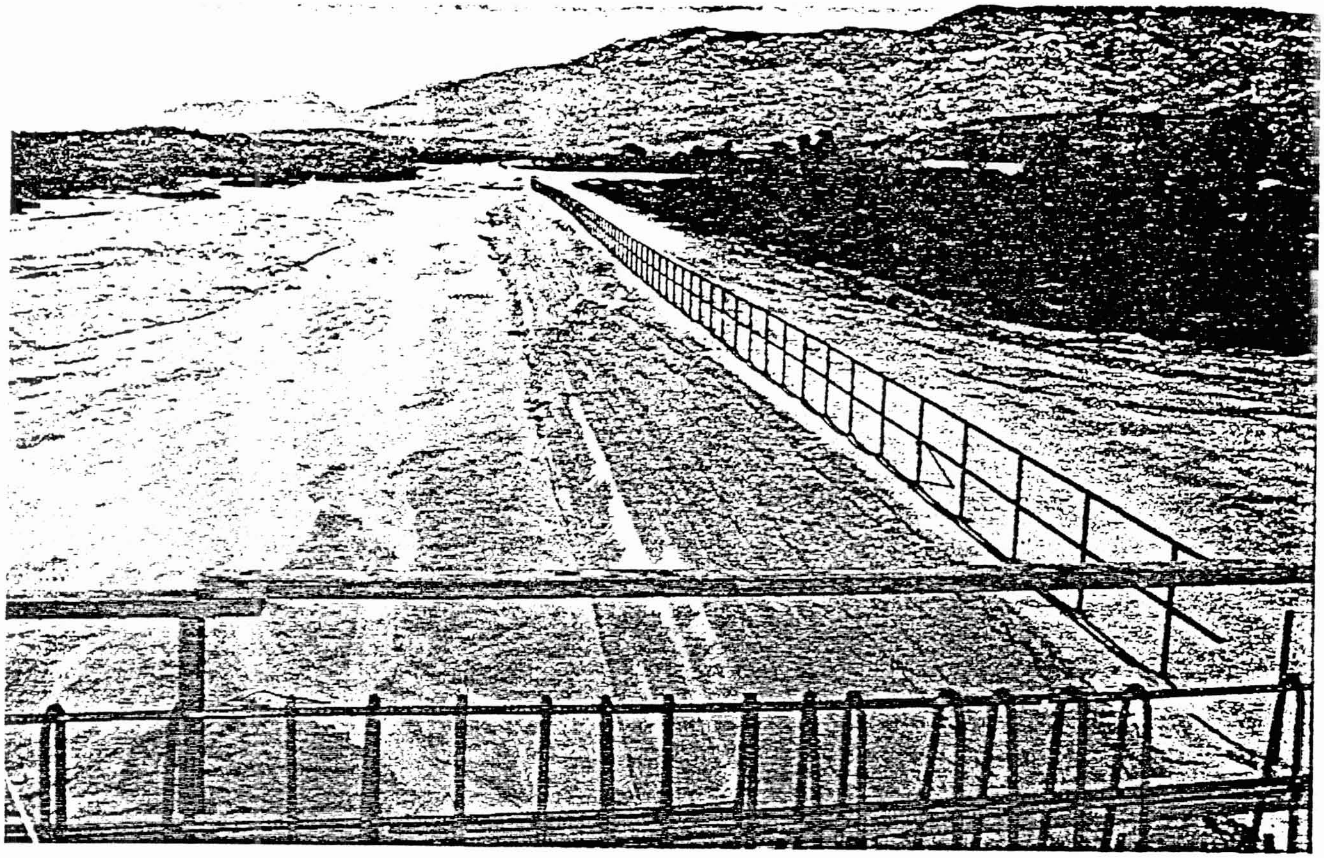
Soil Cement bank protection at confluence of Rooney, Pucsh Ridge and Mutterer Washes



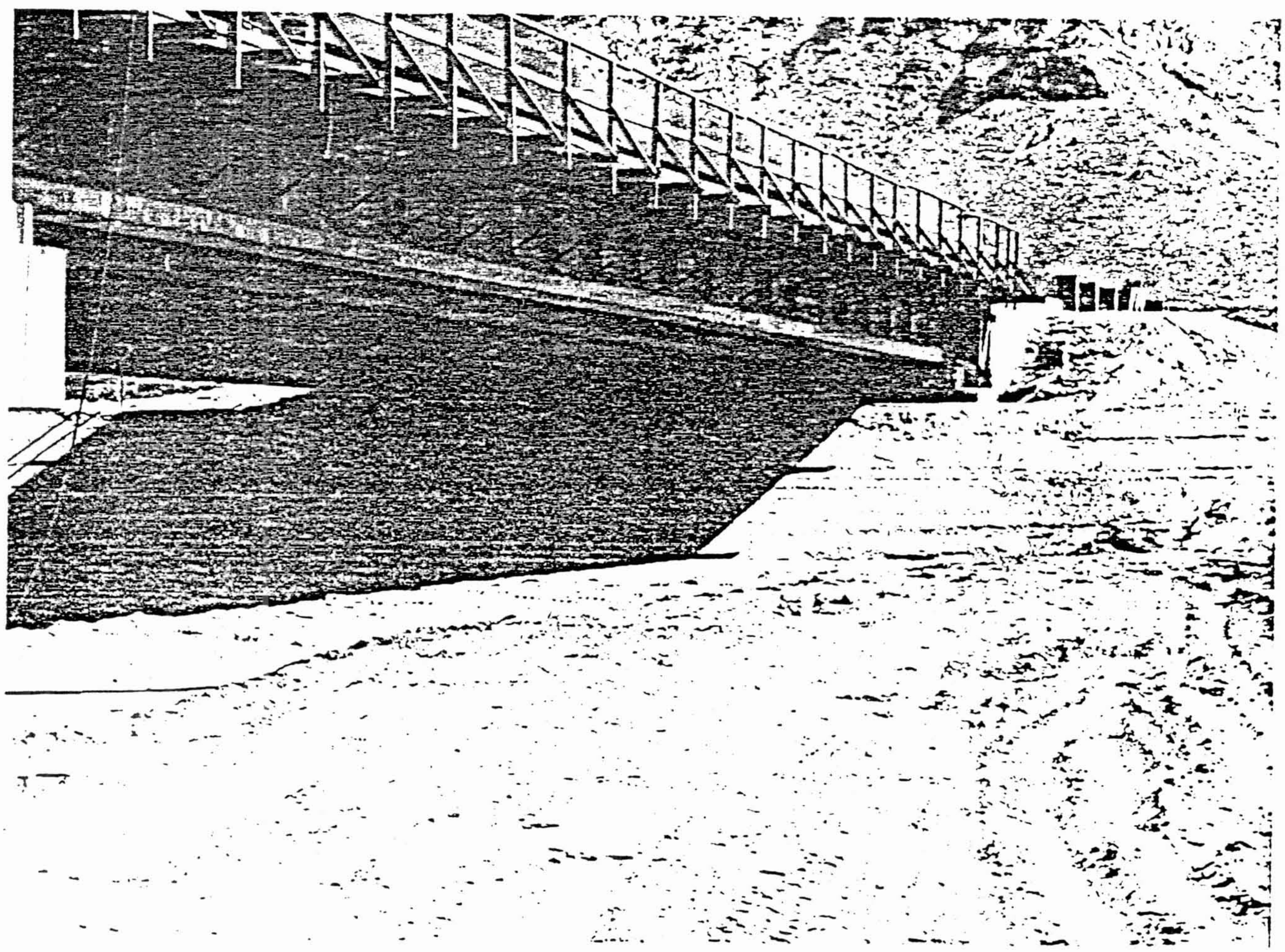
Soil cement bank protection at confluence of Canada Del Oro and Oro Valley Washes



Soil cement bank protection at confluence of Rooney and Pucsh Ridge Washes



Soil cement bank protection, Canada Del Oro at First Avenue



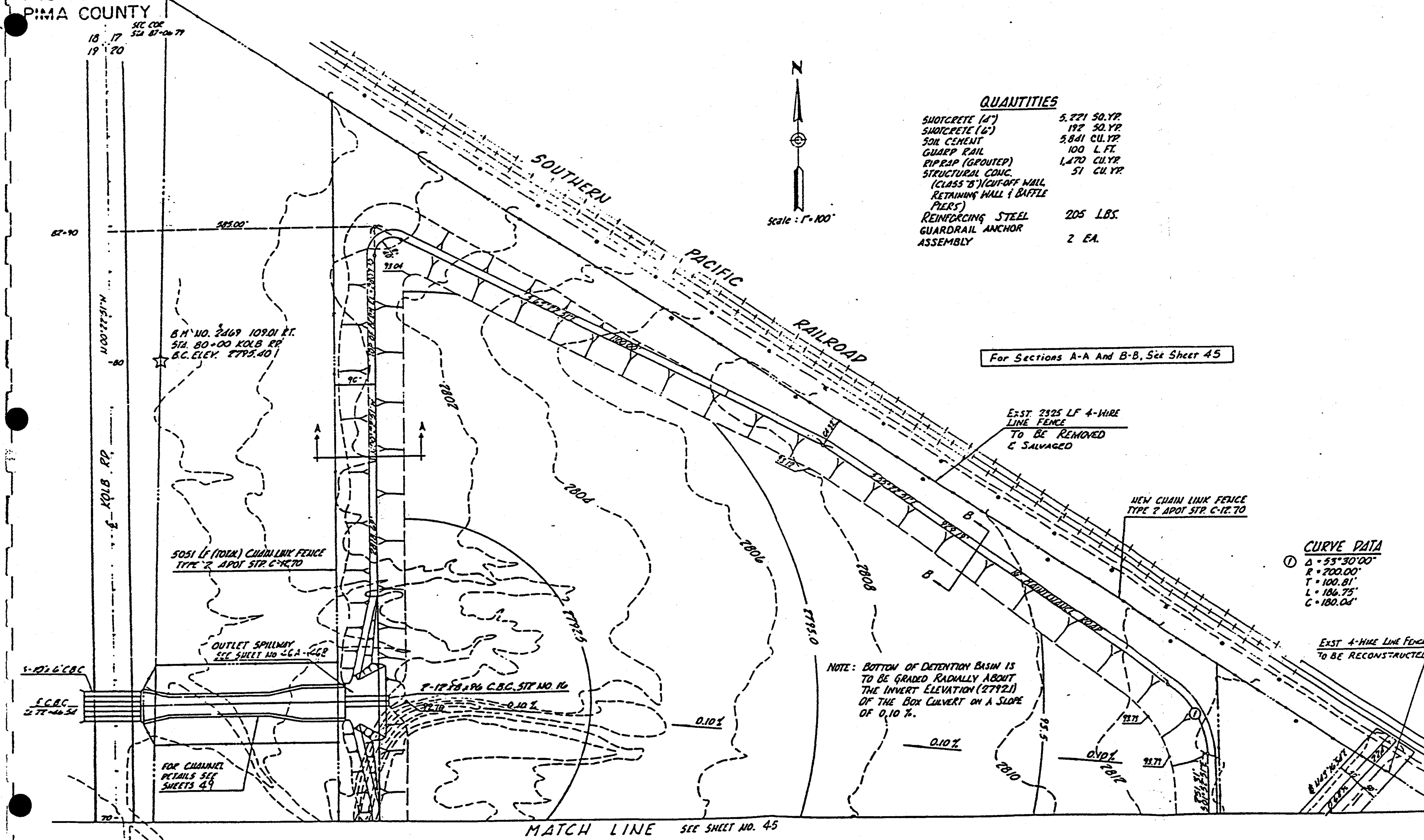
Soil cement abutment protection, Canada Del Oro at First Avenue

DETENTION BASIN

SOIL-CEMENT SPILLWAY DESIGN

EXAMPLE PROJECT: JULIAN WASH

KOLB ROAD
I-10 TO VALENCIA RD.
PIMA COUNTY



JULIAN WASH DETENTION BASIN

SHEET 46 OF 129

EF-084-1(9)

AS BUILT

C.H. HUCKELBERRY R.L.S., P.E. DIRECTOR

DESIGNED
DRAWN
CHECKED
PROJ ENGR

DATE

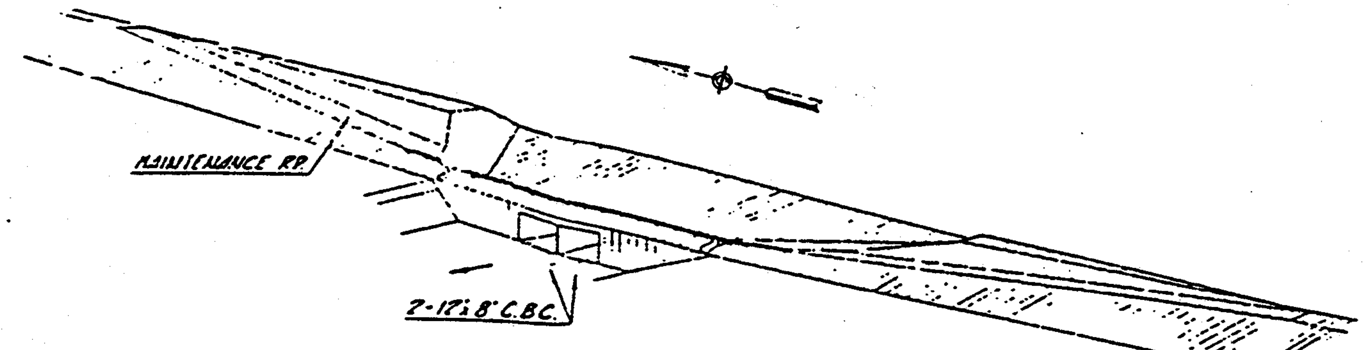
NO. | REVISION | DESCRIPTION | DIV/SECT ENGR. | DATE

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

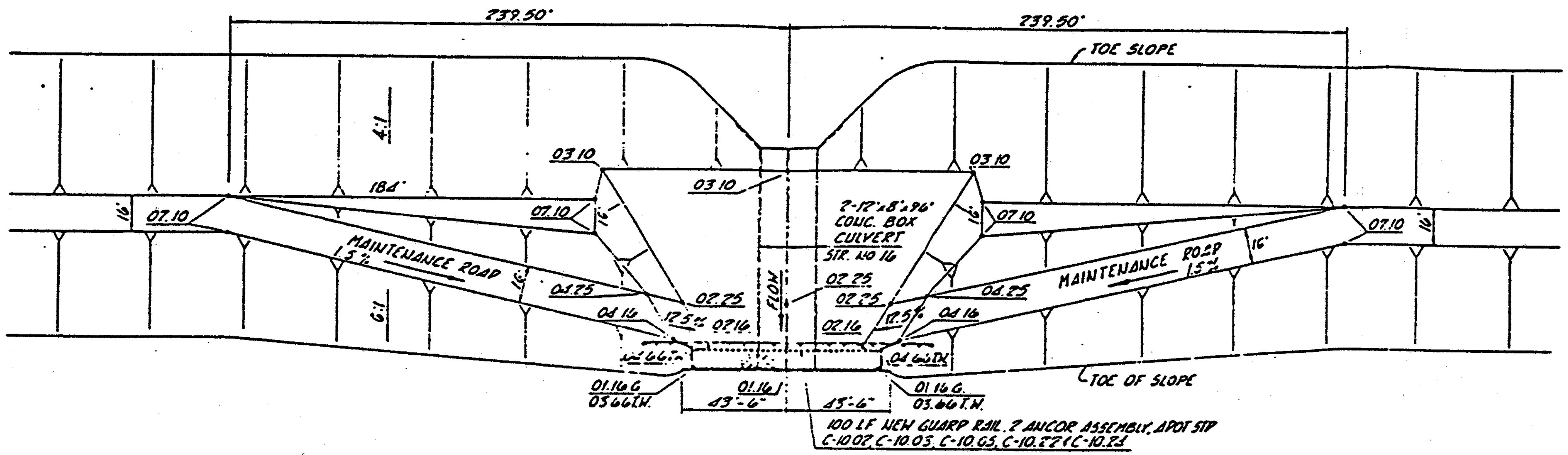
CELIA BARR ASSOCIATES

PROJ. NO. 02012-02

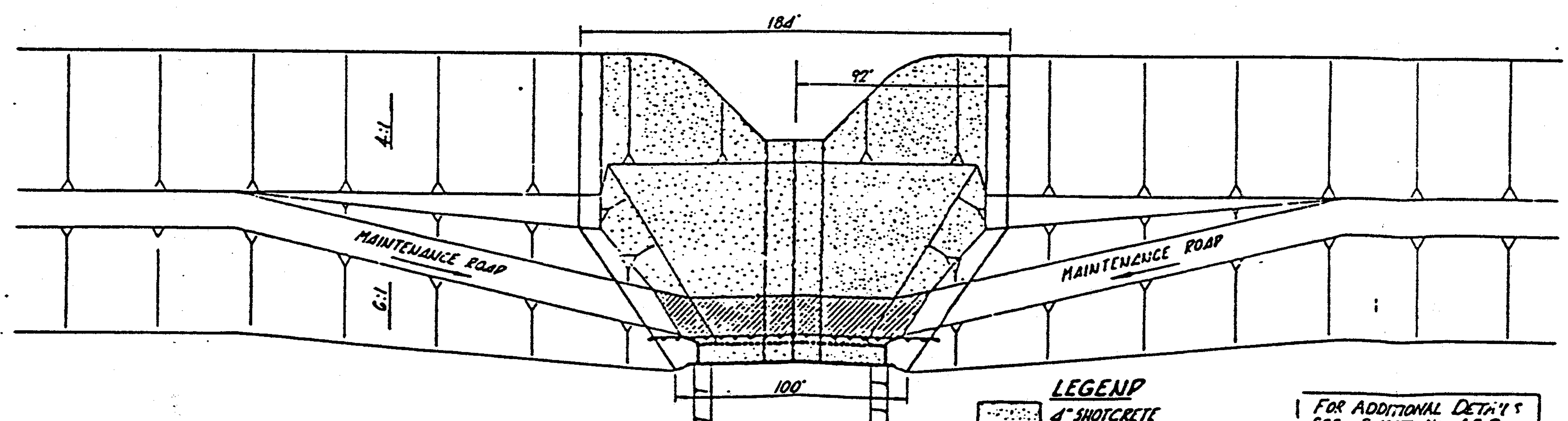
KOLB ROAD
I-10 TO VALENCIA RD.
PIMA COUNTY



OUTLET SPILLWAY PERSPECTIVE



SPILLWAY PLAN
SCALE: 1" = 30'



LEGEND
- 4" SHOTCRETE
- 6" SHOTCRETE
- SOIL CEMENT

FOR ADDITIONAL DETAILS SEE SHEET No. 46B

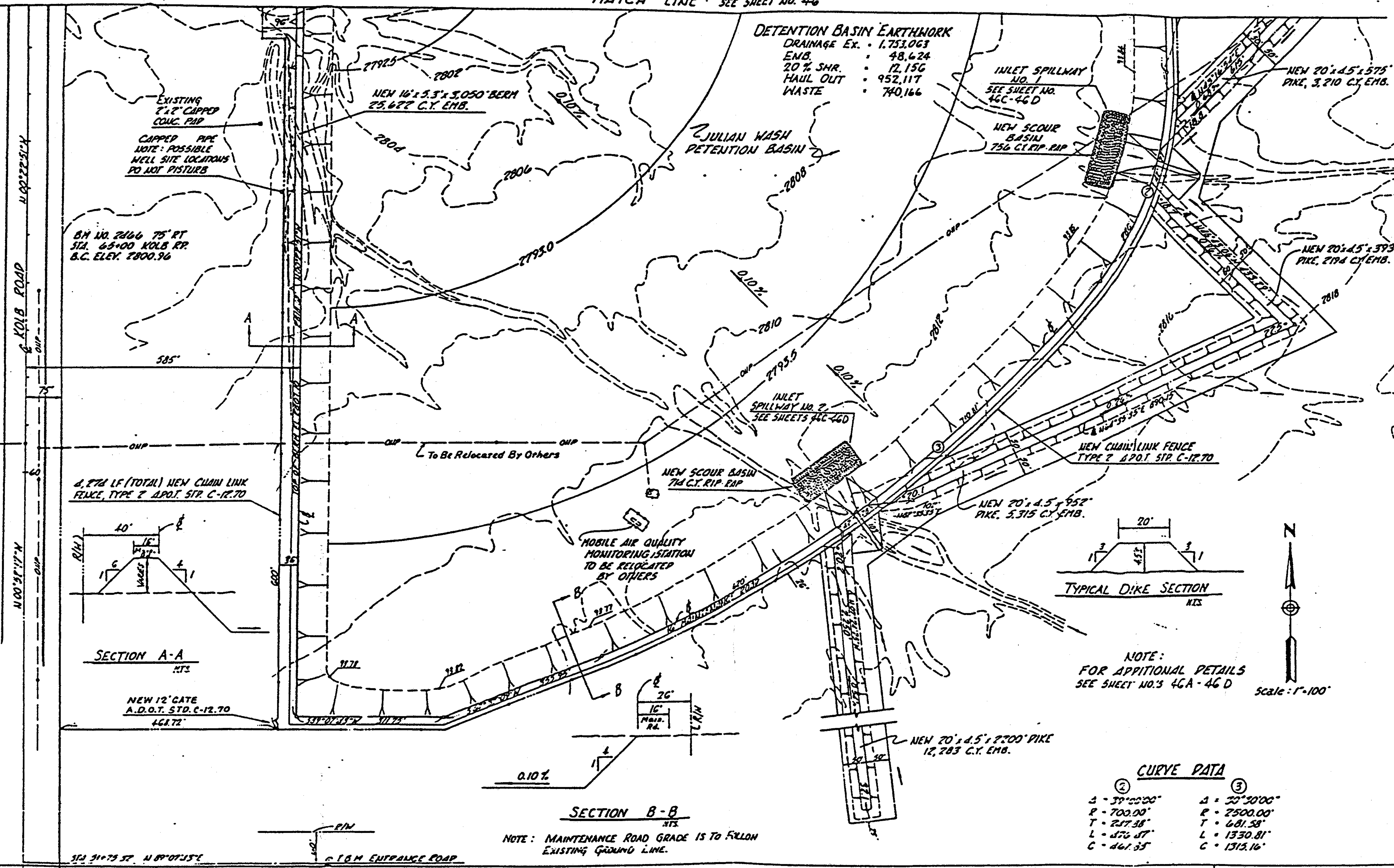
SOIL CEMENT PLAN
SCALE: 1" = 30'

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT
C. H. HUCKELBERRY R.L.S., P.E. DIRECTOR

NO	REVISION DESCRIPTION	PROJECT ENGR	DATE

CELLA BARR ASSOCIATES

KOLB ROAD
I-10 TO VALENCIA RD.
PIMA COUNTY



JULIAN WASH DETENTION BASIN

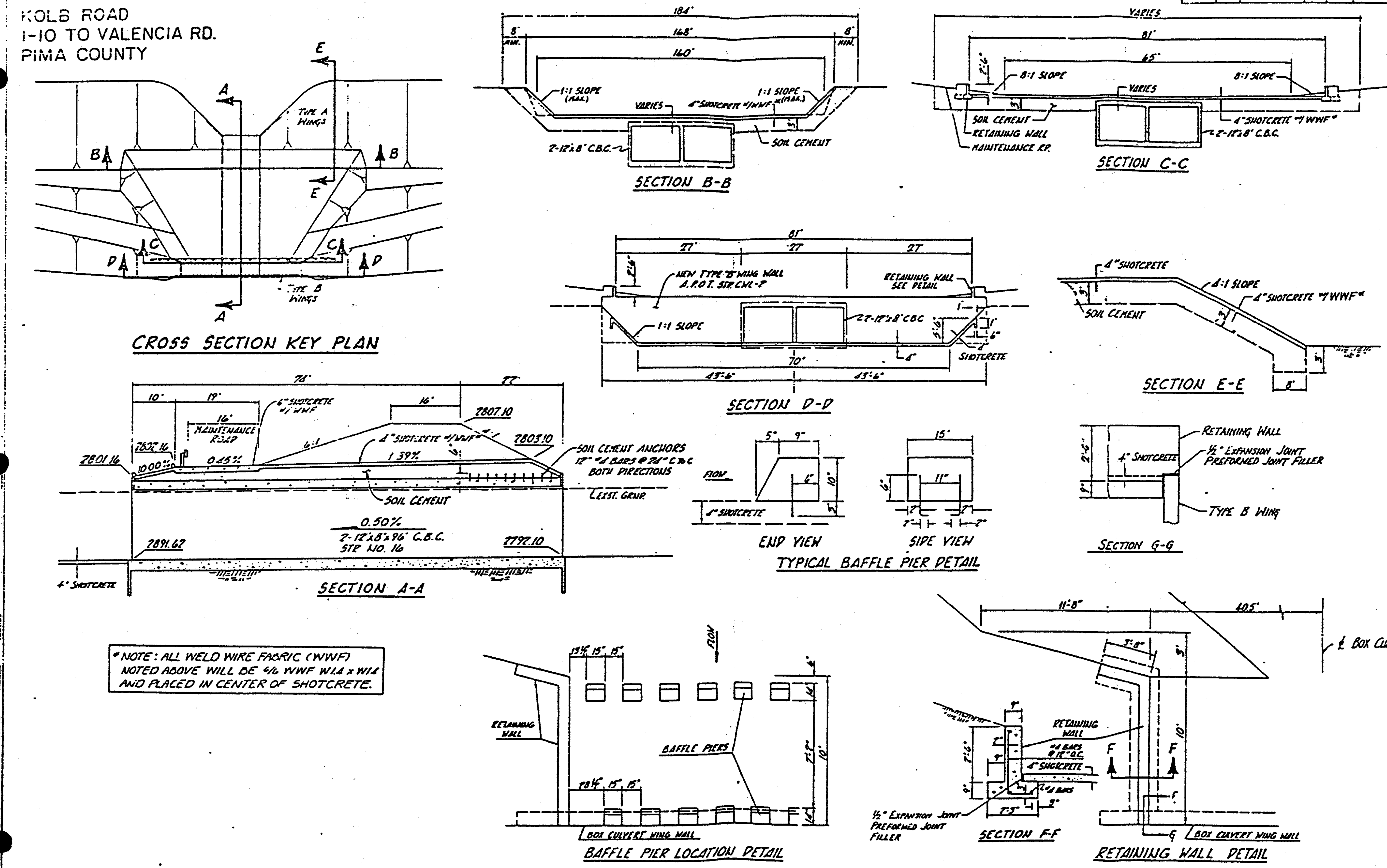
PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

C.H. HUCKELBERRY R.L.S., P.E. DIRECTOR

CELLA BARR ASSOCIATES

PROJ. NO. 02012-02

KOLB ROAD
I-10 TO VALENCIA RD.
PIMA COUNTY



PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

C H HUCKELBERRY R.L.S.,P.E. DIRECTOR

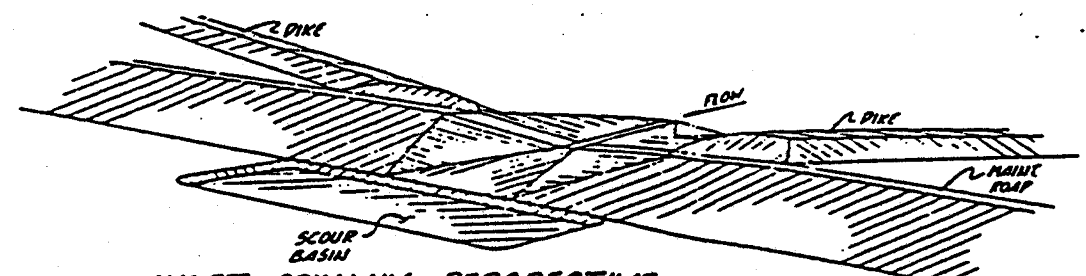
CELLA BARR ASSOCIATES

PROJ. NO. 02012-02

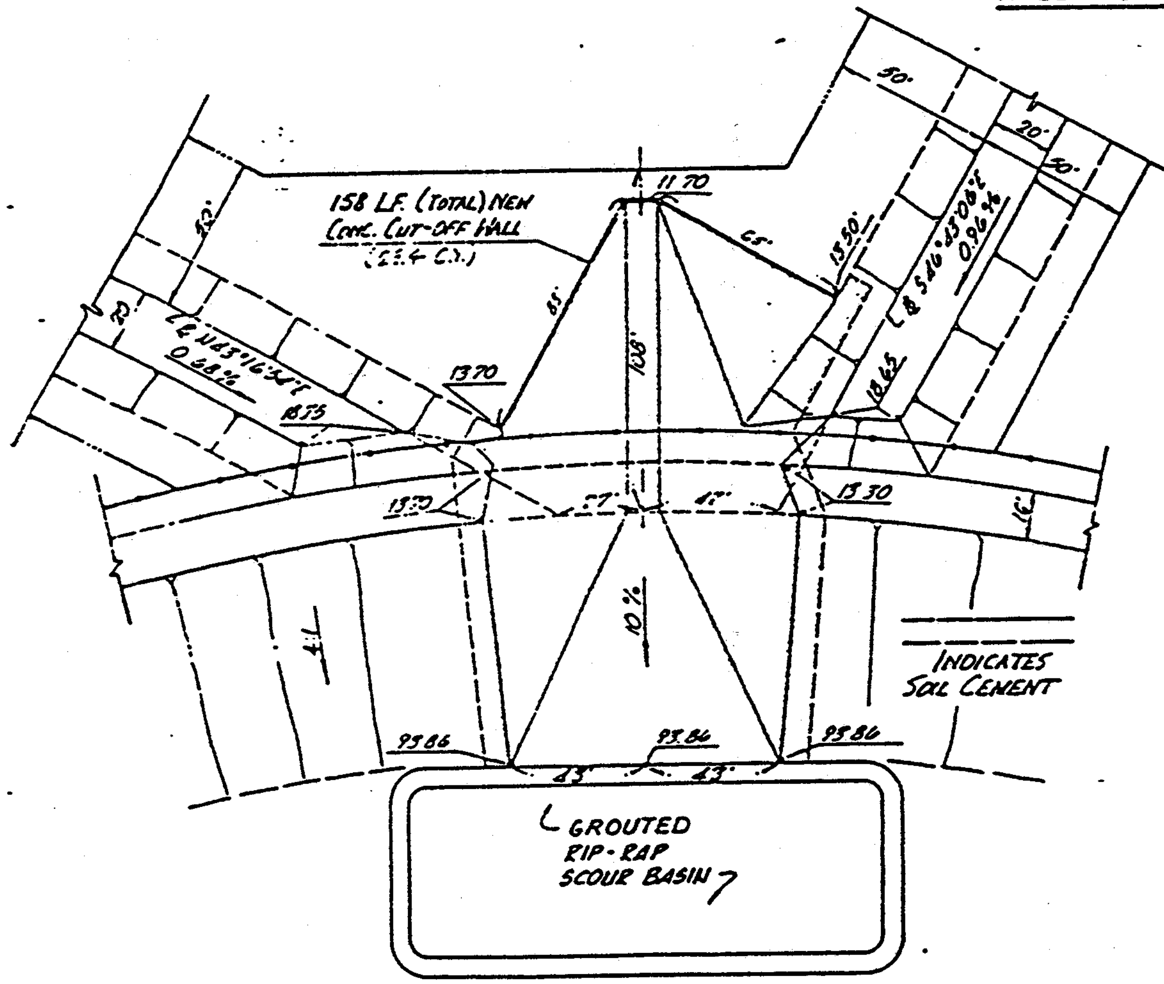
OUTLET SPILLWAY

SHEET 46 B OF 129

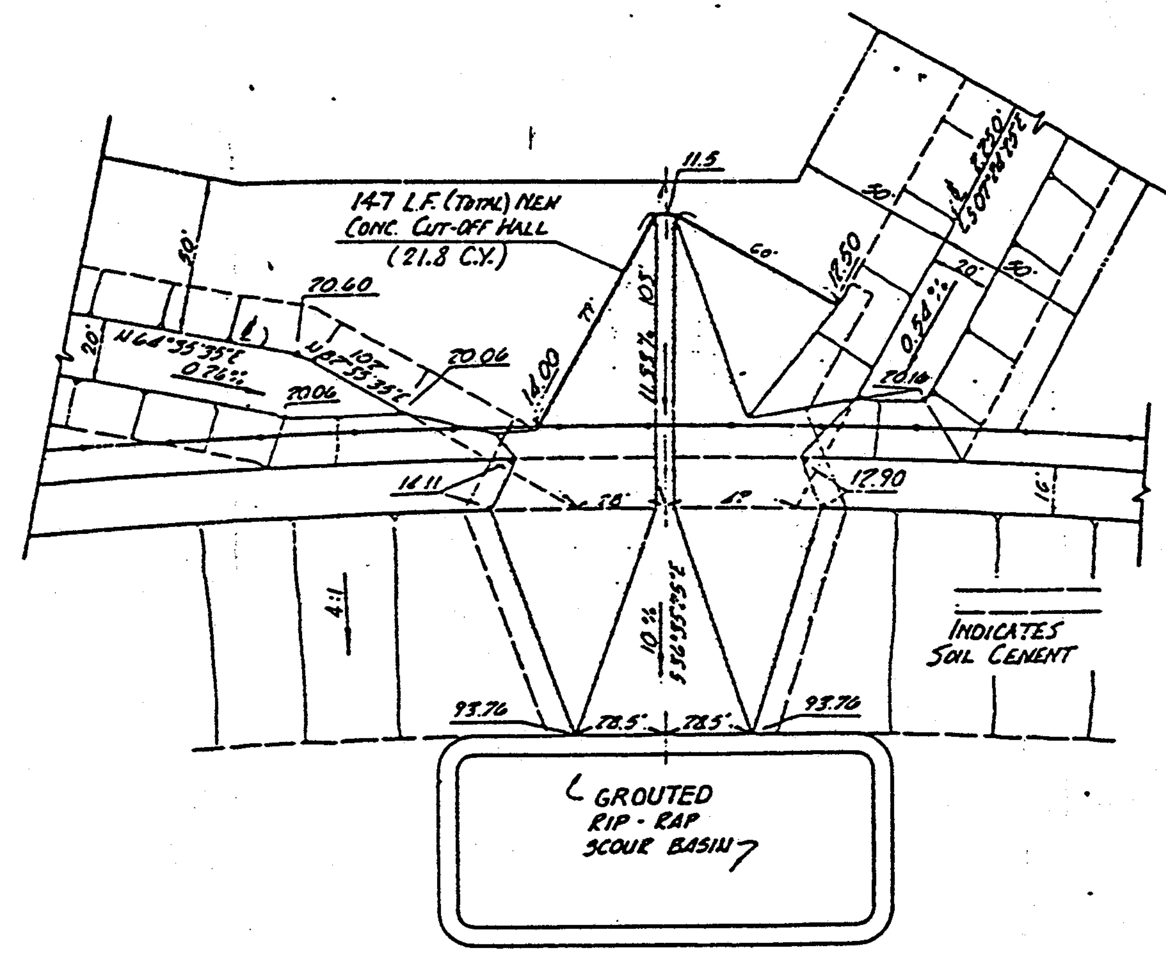
KOLB ROAD
I-10 TO VALENCIA RD.
PIMA COUNTY



INLET SPILLWAY PERSPECTIVE



INLET SPILLWAY NO. 1
PLAN VIEW
scale: 1"=30'



INLET SPILLWAY NO. 2
PLAN VIEW
scale: 1"=30'

FOR ADDITIONAL DETAILS
SEE SHEET No. 46 D

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

C H HUCKELBERRY R.L.S., P.E. DIRECTOR

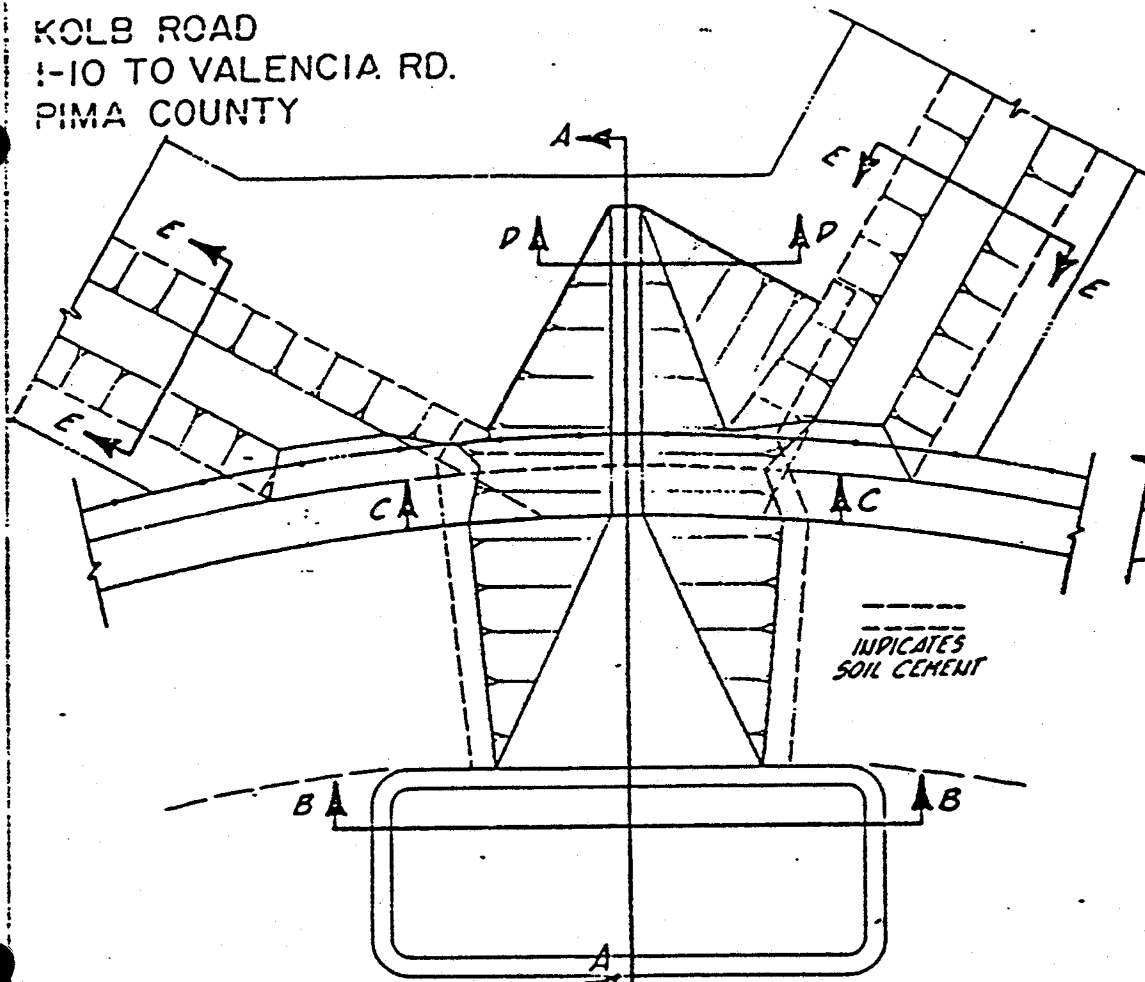
NO	REVISION DESCRIPTION	DIV/SECT ENGR.	DATE

CELLA BARR ASSOCIATES

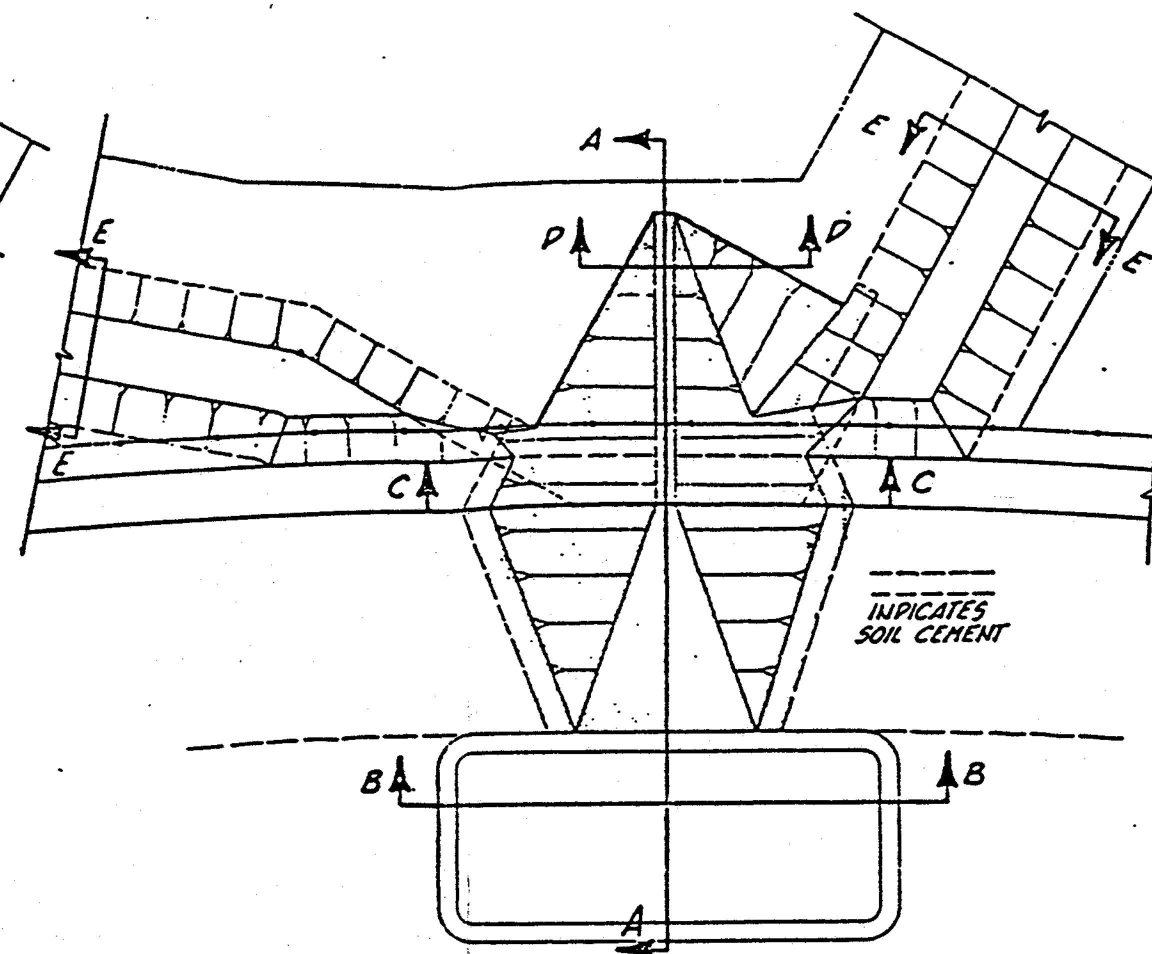
PROJ. NO. 02012-02

INLET SPILLWAY | SHEET 46 C OF 159

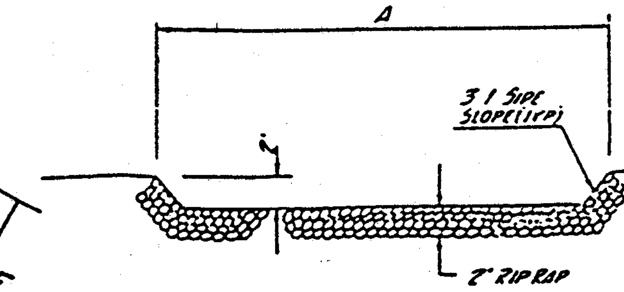
KOLB ROAD
I-10 TO VALENCIA RD.
PIMA COUNTY



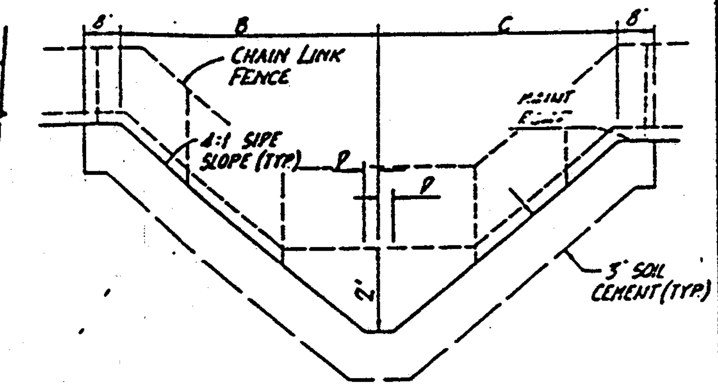
INLET SPILLWAY NO. 1
CROSS SECTION KEY PLAN
Scale: 1" = 30'



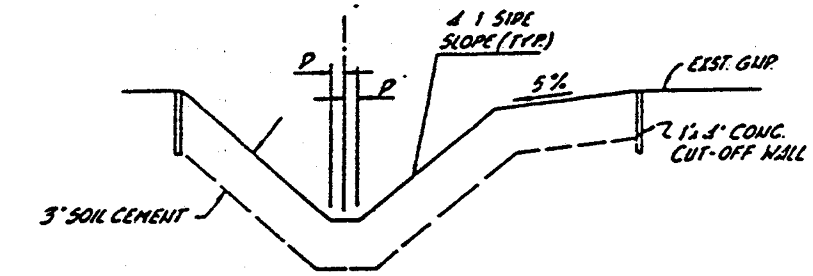
INLET SPILLWAY NO. 2
CROSS SECTION KEY PLAN
Scale: 1" = 30'



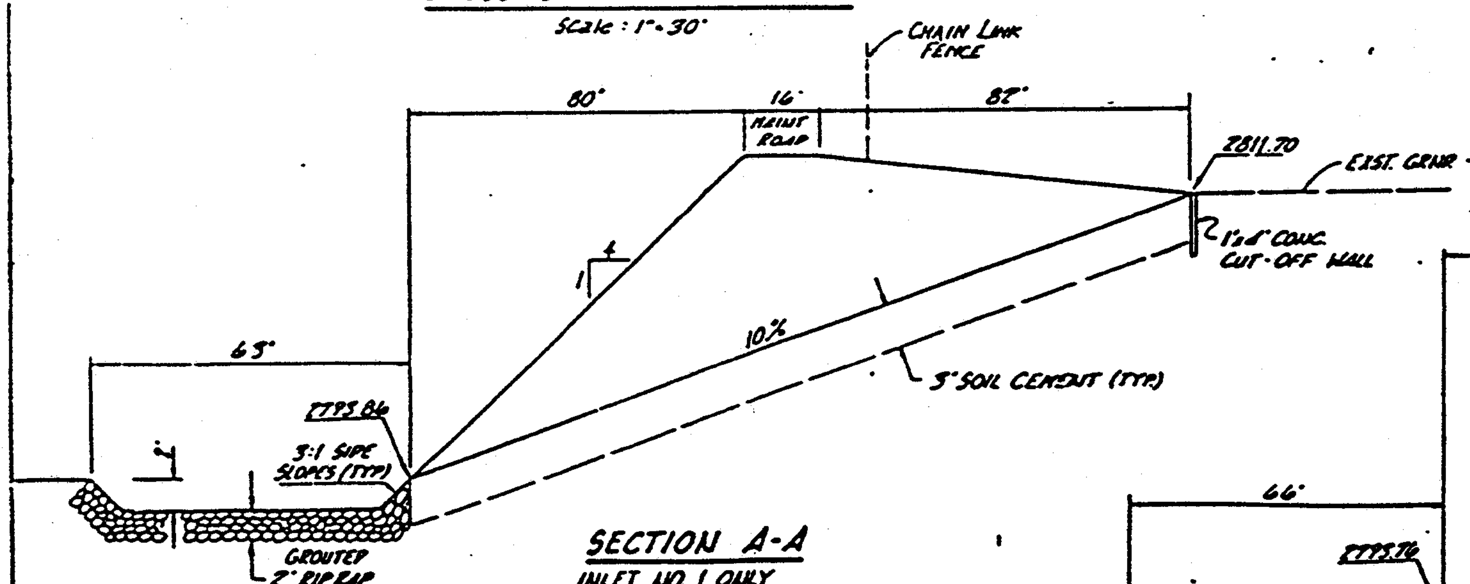
SECTION B-B
N.T.S.



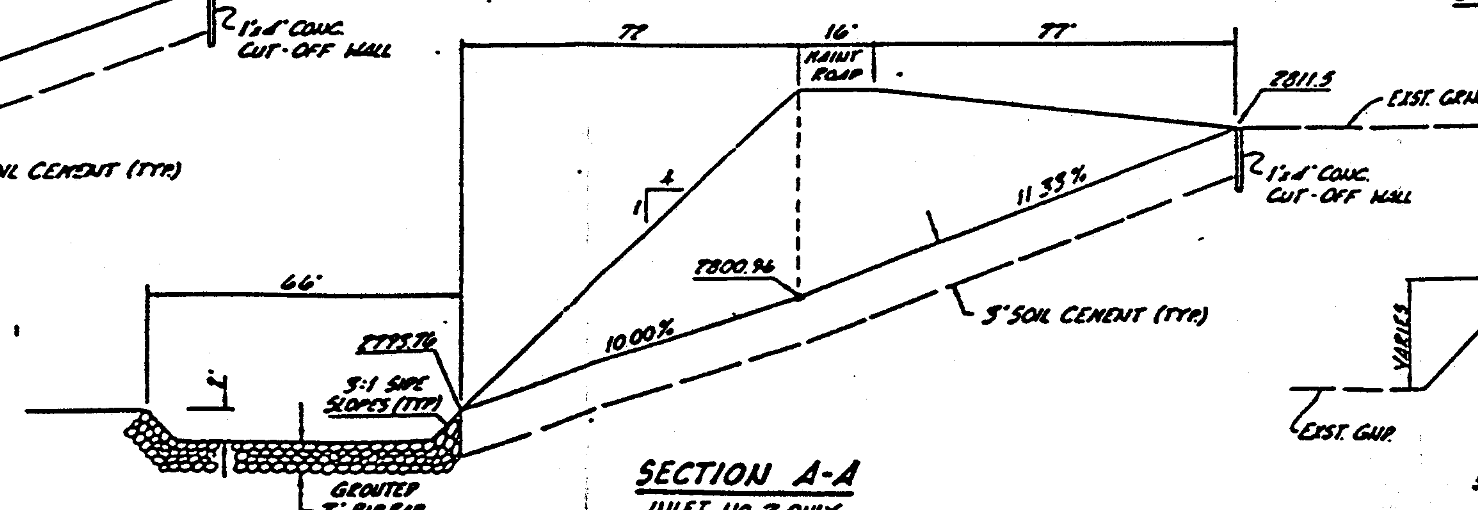
SECTION C-C
N.T.S.



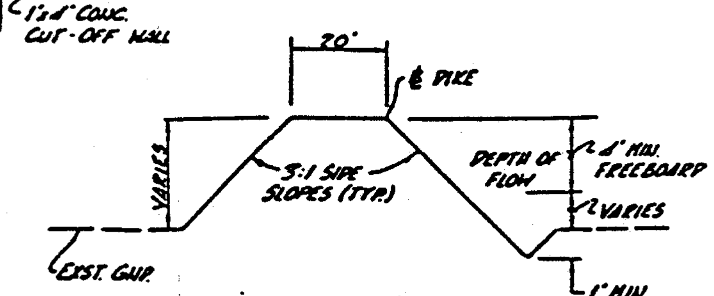
SECTION D-D
N.T.S.



SECTION A-A
INLET NO. 1 ONLY
N.T.S.



SECTION A-A
INLET NO. 2 ONLY
N.T.S.



SECTION E-E
N.T.S.

DIMENSION TABLES

	A	B	C	D
INLET SPILLWAY NO. 1	162'	52'	51'	5'
INLET SPILLWAY NO. 2	146'	56'	51'	3'

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

CELLA BARR ASSOCIATES

PROJ. NO. 02012-02

INLET SPILLWAY

SHEET 46 D OF 124

When necessary, the soil cement shall be protected from freezing for seven (7) days after its construction by a covering of loose earth, straw, or other suitable material approved by the Engineer.

Construction Joints

At the end of each day's work, or whenever construction operations are interrupted for more than two (2) hours, a transverse construction joint shall be formed by cutting back into the completed work to form a full-depth vertical face.

Maintenance

The Contractor shall be required, within the limits of his contract, to maintain the soil cement in good condition until all work is completed and accepted. Maintenance shall include immediate repairs of any defects that may occur. This work shall be done by the Contractor at his own expense and repeated as often as necessary. Faulty work shall be replaced for a full depth of the layer.

Measurement

This work shall be measured in: (1) cubic yards of completed in place soil-cement slope protection as determined by the specified lines, grades, and cross sections shown on the Plans, and (2) in tons of cement incorporated into the soil cement used for tests and for the slope protection in accordance with the instructions of the Engineer. Any waste of cement and/or soil cement by the Contractor during the handling, mixing, placing, etc. operations shall not be paid for.

Payment

This work shall be paid for at the Contract Unit Price per cubic yard of soil-cement protection and at the Contract Unit Price per ton of cement furnished, multiplied by the quantities obtained in accordance with Measurement herein. Such payment shall constitute full reimbursement for all work necessary to complete the soil-cement slope protection, dewatering, trench excavation, watering, mixing, placing, compacting, curing, inspection and testing assistance, and all other incidental operations.

5.2 Inspection and Testing

GENERAL

The Engineer, with the assistance and cooperation of the Contractor, shall make such inspections and tests as he deems necessary to insure the conformance of the work to the contract documents. These inspections and tests may include, but shall not be limited to: (1) the taking of test samples of the soil cement and its individual components at all stages of processing and after completion, and (2) the close observation of the operation of all equipment used on the work. Only those materials, machines, and methods meeting the requirements of the contract documents shall be approved by the Engineer.

All testing of soil cement or its individual components, unless otherwise provided specifically in the Contract Documents, shall be in accordance with the latest applicable ADOT, ASTM or AASHTO Specifications in effect as of the date of advertisement for bids on the project.

Testing for proper compaction shall be done on at least every other lift of compacted soil cement at any location chosen by the testing personnel. If the lift being tested does not pass the minimum 98 percent density requirements, it must be reworked as directed by the Engineer until it passes or be removed at the Contractor's expense. The Contractor shall not be permitted to continue placing lifts of soil cement of any lift which has failed the compaction tests until such time as that lift has been reworked, retested, and accepted as meeting density requirements.

5.2.1 Testing Procedure for Cement Content
(A Modification of Arizona 220 Test Method)

Description

(A) This method of test is intended for determining the percentage of portland cement required in developing soil cement mixtures by the determination of the compressive strength of molded specimens at varying cement contents.

(B) Equipment Required:

(1) Mold - A cylindrical metal mold having a capacity of 1/30 Cubic foot with an internal diameter of 4.0 plus/minus .005 inch and a height of 4.585 plus/minus .005 inch equipped with a detachable collar approximately 2-1/2 inches in height.

(2) Rammer - A metal rammer manually or mechanically operated having a 2-inch diameter circular face and weighing 5.5 pounds. The rammer shall be equipped with a suitable arrangement to control the height of drop to a free fall of 12 inches above the elevation of the cement-treated mixture.

(3) Balance - A balance or scale of at least 5 kg. capacity sensitive to 0.5 gm.

(4) Drying Oven - A thermostatically controlled drying oven capable of maintaining a temperature of 230^{o} plus/ minus 9^{o} F (110^{o} plus/minus 5^{o} C).

(5) Straightedge - A rigid steel straightedge 12 inches in length having one beveled edge.

(6) Sieve - 3/4 inch sieve conforming to the requirements of the Specifications for sieves for Testing Purposes (ASTM E11-81 and AASHTO M 92).

(7) Miscellaneous mixing tools and plans.

(8) Speedy Moisture Tester (optional).

(9) Equipment required for the determination of the Compressive Strength of Cylindrical Concrete Specimens (ASTM C39, C42, C511).

Sample Preparation

(A) If the sample is damp when received, it shall be dried until it becomes friable under a trowel. Drying may be accomplished by air drying or by the use of a drying apparatus such that the temperature of the sample does not exceed 140° F (60° C).

(B) After drying, prepare the sample for testing by separating the aggregate retained on the 3/4 inch sieve and breaking up the remaining soil aggregations to pass the 3/4 inch sieve in a manner which will avoid reducing the natural size of individual particles.

(C) Select and prepare eight separate test charges of dry soil cement of approximately 2500 gm. each. Two samples are to be made for every cement percentage selected.

(D) Add the first of the cement contents to be used and mix thoroughly together.

Example: If the percent cement selected is 10%

Dry Soil Weight	=	2250 gms.	(90%)
Portland Cement	=	250 gms.	(10%)
TOTAL		2500 gms.	(100%)

(E) The moisture content to be added to each test charge is determined by making a maximum density-optimum moisture determination with the anticipated required cement content (according to AASHTO T 99-74 Method C) and using this developed optimum-moisture thereafter for all specimens prepared.

Compaction

(A) Form a specimen by compacting the prepared mixture in the mold with the collar attached in three equal layers to give a total compacted depth of 5 inches Compact each layer by applying 25 uniformly distributed blows from a 5.5 pound (2.5 kg.) rammer dropping free from a height of 12 inches (305 min). Following compaction, remove the extension collar, carefully trim the compacted mixture even with the top of the mold by means of a straightedge and weigh. Multiply the weight of the specimen (in gms.) by 0.06614 to obtain the wet weight per cubic foot. The factor 0.06614 is valid only if the volume of the mold is 1/30 cubic foot. If calibration shows any change in volume, a new factor shall be calculated.

NOTE: Assuming the mold has a volume of 1/30 (0.0333) cubic foot the factor is derived as follows:

$$.06614 = \frac{1}{0.0333 \text{ cu. ft.} \times 453.6 \text{ g./lb}}$$

In case of a change in volume of the mold, 0.0333 cubic foot shall be replaced by the decimal fraction for the new volume.

(B) Compact a duplicate specimen in the same manner as step (A) above.

(C) Extrude both samples from their respective molds using caution and place on glass or non-absorptive plates and store for curing in a moist condition; i.e., a moist cabinet or a moist room meeting the requirements of ASTM C-511-80.

(D) Determine the moisture content of the prepared samples from the residue.

(E) Determine the wet density and dry density of the samples.

(F) Repeat steps (A) - (D) on additional samples with increased cement content (in 2% increments) until a complete bracketing of specification requirements is met.

Determination of Compressive Strength

(A) All specimens must be cured as specified in a moist condition for six days and then immersed for a period of 24 hours in water maintained at 73.4 plus/minus 3^{o} F (23 plus/minus 1.7^{o} C).

(B) Specimens shall then be prepared for the compression test in accordance with ASTM C 617.

(C) The compressive strength of the cylinders shall then be determined in accordance with ASTM C-39 and ASTM C-42.

(D) The results shall be reported in a format similar to that shown in the following report form.

5.3 Cost and Aesthetics

A major advantage in the use of soil cement is the natural look obtained from using the high percentage of native soils. Soil cement bank stabilization blends into the natural river environment as computed to concrete lined channels and banks. The aesthetic value of soil cement aids in its use with multi-use flood control projects that also include recreation facilities. Horse ramps can be designed into the soil-cement bank protection, bike paths, jogging tracks, etc., can easily accompany soil-cement projects while blending into the natural river system.

The cost of soil cement varies for each project depending upon toe-down limits, design height, percentage of portland cement and length of project. The following illustrate the basic computations for determining the cost per cubic yard for placement of soil cement.

Example: Large scale soil-cement bank stabilization

Given: $ 8.00/cubic yard construction
$ 76.00/ton Portland Cement
10% cement content by weight

Therefore: 140 lbs./cubic foot x 27 = 3,780 lbs/cubic yard
x 10% = 378 divide by 2,000 lbs./10
= .19 ton/cubic yard.
.19 x 76 = $ 14.36 cubic yard soil cement

$14.36 + $8.00 = $22.36 complete in-place soil cement.

SECTION 6 - CURRENT RESEARCH

6.1 Groundwater Recharge Impact

The construction of bank stabilization and grade-control structures creates a potential for altering the relationship with and interaction between surface flow and groundwater. The primary factors in this relationship are the rate of water infiltration from the river through the stream bed into the underlying porous material, and the characteristics of subsurface movement of this infiltrated water. Several controlling factors determine the parameters which include hydraulic conductivity of the streambed and underlying porous matrix, hydraulic head in the river corresponding to the event under study, river width, and lateral and vertical variability of subsurface inhomogeneties (perching layers, caliche, clay, or gravel lenses, etc.).

Pima County is concerned with the potential impact of soil-cement bank stabilization and grade control structures on stream infiltration and groundwater recharge. Primary areas of concern include: 1) evaluation of stream infiltration rates under existing natural channel bank conditions versus stabilized channel bank conditions (soil cement); and 2) evaluation of the effects of grade control structures on subsurface flow, infiltration, and/or groundwater recharge. The University of Arizona Water Resources Research Center was selected to: 1) perform model studies, 2) evaluate potential integration of flood control priority and ground water recharge, and 3) evaluate recharge in the Tucson Basin. The approach was to adapt the UNSAT II numerical model to simulate infiltration and recharge for a sequence of boundary and flow conditions. The model has a distinct advantage over classical flow net models in that it accounts for both saturated and unsaturated flow regions in the vadose zone. The model will be developed using actual vadose zone flow data obtained at a research site abutting the Santa Cruz River. After calibrating and validating the model using these field data, a sequence of simulations will be employed to determine the effect of boundary conditions (i.e., bank protection and cut-off walls, channel widths, etc) in infiltration and recharge. The model will also be adapted to other stream reaches in an attempt to evaluate regional effects.

6.2 Physical Model of Grade Control Structures

The construction of soil-cement grade control structures to control the streambed profile in eroding ephemeral streams have been utilized throughout Pima County. Estimation of the scour depth has been a difficult task due to the complicated nature of the flow phenomenon as well as the lack of appropriate equations. Experience has indicated that one of the major factors contributing to the cost of the construction is the necessity to provide adequate burial depth in order to prevent the failure due to the scour hole that develops at the downstream face. To date, the prediction of scour depth has been made based upon equations developed for scour resulting from a free overfall flow condition. The conditions under which the grade control/ drop structure is performing are significantly different from those under which the equation was developed. Particularly, due to the limited drop height of the soil-cement grade control structure at the channel invert, compared to the depth of stream flow, the flow over the drop will be partially or fully submerged. This submergence of the overfall reduces the scour to below the values otherwise indicated by the existing equations. Unfortunately, literature to date does not provide the quantitative information to substantiate the extent of the scour reduction. The use of the more conservative free overfall scour calculation results in significant increased costs for the additional buried depth required. Pima County has initiated a physical model study to provide an adequate prediction of scour depth and the stability of the grade control structures. A range of various hydraulic parameters will be modeled in a steel flume 90 feet long, 11 feet high and 3 feet wide. These parameters include drop height, flow width, flow depth, side slope on downstream face and discharge (up to 200 cfs/ft). The results of this study will provide quantitative answers in the form of equations and charts which will prevent overdesign for scour and consequently will significantly reduce construction costs.

6.3 Design Life

Soil cement bank stabilization has been in use for 20 years at various locations within Pima County. Limited testing of long-term erosion resistance has been performed to determine the life expectancy of soil cement. Erosion tests have been conducted where soil cement has been utilized as part of dam construction or diversion work structures. Results of these tests failed to show any significant erosion once the outside loose and uncompacted material was washed away. Pima County has 20 years of experience with the design, construction and maintenance of soil cement which has led to several refinements of the design criteria. For example, the 1978 flood indicated that 500 pounds per square inch (psi) was inadequate but 750 psi was sufficient to withstand erosion. This erosion strength was further substantiated in the October, 1983 flood which was the largest flood on record for many washes within Pima County. In order to refine the mix design and strength requirements to resist erosion and to provide a long service life, quantitative data must be developed. Pima County is currently reviewing proposals to perform laboratory investigations to determine the resistance of various mix designs of soil cement to erosion by sediment laden flow. The tests will evaluate the use of soil cement both parallel and normal to the direction of flow. Various parameters will be considered including discharge, percent of sediment laden material, size fraction, velocity of flow and duration of flow versus the various percentage of cement content. This type of testing based on literature reviewed to date, has not been performed anywhere else. Tests of moderately strong portland cement concrete in spillways stilling basins, bridge piers and abutments show evidence of abrasion by sediment-laden flow. It is anticipated that the mix design will be refined based on several criteria: 1) desired of sufficient strength, 2) resistance to abrasion and weathering, and 3) service life duration for the least cost. The results of the study will be in the form of design charts.

6.4 Quality Control

Mix design and daily field control specimens currently being made in accordance with the Pima County construction specifications for soil cement are performed using a mold 4 inches in diameter by 4.5 inches high compacted in three (3) lifts utilizing a 5.5 pound, 12 inch drop standard proctor hammer.

The specimens are tested in compression in accordance with Arizona 300; i.e. concrete testing, as specified in Arizona 220. For consistency in the compressive strength results of the cylindrical samples, the L/D correction (where L = height of cylinder and D = diameter of cylinder) is performed in accordance with ASTM-C42-82. This procedure is applied because the length or height (L) of the specimens utilized in the Pima County procedure is less than twice their diameter (D).

For the purpose of evaluating the correction factor and its effect on the test results, a new mold size and compaction procedure was developed and is currently under testing. The mold is 4 inches in diameter and 8 inches high and the material is compacted in five equal lifts with 26 blows applied to each life utilized the 5.5 pound and 12 inch drop standard proctor hammer. Initial test results are very positive with excellent correlations to the original procedure concerning compacted dry density and compressive strengths. The procedure will be further tested and compared to the one currently being utilized before its final adoption.

6.5 Additives and Cement Types

Evaluation of our current design criteria is being considered to obtain a more cost effective design for soil cement bank protection.

Ongoing research on soil cement by Pima County is currently being conducted in two major areas of mix design in order to optimize the construction soil cement at the least cost while maintaining the required high quality. The main and most costly component of soil cement is Portland Cement, therefore, any savings received by reducing the percentage of cement content without jeopardizing the quality of the soil cement will result in direct project savings.

Pima County's Materials Section is currently investigating the possibilities of utilizing fly ash (Class F) as a replacement for Portland Cement in percentage varying from 10 to 30 percent of the total cement utilizing the same mix design and testing procedures currently in use. Also, being investigated is the use of IP (portland-Pozzuolan) cement.

Interrelated properties such as moisture content, dry densities, fly ash content, portland cement content, portland-Pozzuolan cement content, 24-hour compressive strength, and soaked seven (7) day compressive strength are all currently being evaluated with test results expected shortly.

PIMA COUNTY DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT
FIELD ENGINEERING DIVISION
MATERIALS LAB

SOIL CEMENT PROJECT CONTROL AND RESEARCH DATA

********* RILLITO RIVER S/C W. OF ORACLE RD. *********

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
1	----	----	----	----	654.000000	1291.00000	50.658404	
2	----	----	----	----	477.000000	1045.00000	45.645933	
3	----	----	----	----	632.000000	1266.00000	49.921011	
4	----	----	----	----	480.000000	895.000000	53.631285	
5	----	----	----	----	618.000000	1175.00000	52.595745	
6	----	----	----	----	634.000000	1157.00000	54.796889	
7	----	----	----	----	677.000000	1186.00000	57.082631	
8	----	----	----	----	533.000000	958.000000	55.636743	
9	----	----	----	----	568.000000	1196.00000	47.491639	
10	----	----	----	----	489.000000	864.000000	56.597222	
11	----	----	----	----	383.000000	726.000000	52.754821	
12	----	----	----	----	349.000000	767.000000	45.501956	
13	----	----	----	----	414.000000	736.000000	56.250000	
14	----	----	----	----	636.000000	1182.00000	53.807107	
15	----	----	----	----	288.000000	559.000000	51.520572	
16	----	----	----	----	754.000000	1299.00000	58.044650	
17	----	----	----	----	800.000000	1647.00000	48.573163	
18	----	----	----	----	669.000000	1565.00000	42.747604	
19	----	----	----	----	724.000000	1273.00000	56.873527	
20	----	----	----	----	651.000000	1274.00000	51.098901	
21	----	----	----	----	700.000000	1312.00000	53.353659	
22	----	----	----	----	606.000000	930.000000	65.161290	
23	----	----	----	----	682.000000	1052.00000	64.828897	
24	----	----	----	----	728.000000	1318.00000	55.235205	
AVERAGE	----	----	----	----	589.416667	1111.87500	53.325369	
STD DEV	----	----	----	----	134 963414	266.124001	5.891611	

* FIRST PROJECT CONTROLLED BY THE RELATIONSHIP BETWEEN THE COMPRESSIVE STRENGTH AFTER 24 HRS OF HUMID CURED SAMPLES AND THE CORRESPONDING 6 DAY HUMID CURED AND 24 HR. SOAKED SAMPLES (I.e. 7 DAY SAMPLES)

********* RILLITO RIVER @ FIRST AVE. BANK PROTECTION (S.C.S.) ********* 4FRRFA *********

29,150 CU.YDS. OF S/C

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
MIX DESIGN	6/29/84	6.000000	7.400000	132.400000	632.000000	987.000000	64.032421	RECOMMENDED CEMENT CONTENT 8.0%
" "	"	8.000000	7.400000	133.700000	875.000000	1460.00000	59.931507	
" "	"	10.000000	7.400000	135.100000	1158.00000	2106.00000	54.985755	
1	9/25/84	8.370000	9.400000	130.300000	678.000000	1324.00000	51.208459	
2	"	8.420000	9.400000	129.800000	728.000000	1288.00000	56.521739	
3	"	8.580000	9.600000	128.300000	445.000000	990.000000	44.949495	
4	"	8.600000	9.700000	129.800000	469.000000	972.000000	48.251029	
5	10/01/84	8.410000	9.100000	139.700000	762.000000	1318.00000	57.814871	
6	"	8.600000	9.000000	140.200000	748.000000	1312.00000	57.012195	
7	10/02/84	8.800000	9.500000	129.800000	770.000000	1408.00000	54.687500	
8	"	8.760000	9.300000	128.900000	843.000000	1441.00000	58.501041	
9	"	8.790000	9.500000	129.500000	468.000000	1009.00000	46.382557	
10	"	8.720000	9.200000	128.400000	626.000000	1430.00000	43.776224	
13	10/04/84	8.420000	10.500000	129.800000	935.000000	1445.00000	64.705882	
14	"	8.400000	10.300000	129.800000	920.000000	1353.00000	67.997044	
15	10/08/84	8.210000	8.400000	129.450000	843.000000	1420.00000	59.366197	
16	"	8.150000	8.100000	127.550000	808.000000	1437.00000	56.228253	
17	10/09/84	8.180000	9.000000	126.150000	654.000000	1239.00000	52.784504	
18	"	8.180000	8.600000	125.600000	758.000000	1483.00000	51.112610	
19	"	8.180000	8.800000	127.050000	719.000000	1283.00000	56.040530	
20	10/10/84	7.920000	8.100000	126.950000	748.000000	1158.00000	64.594128	
21	"	8.090000	9.600000	129.250000	791.000000	1354.00000	58.419498	
22	10/11/84	8.040000	8.000000	124.300000	778.000000	1417.00000	54.904728	
23	"	8.120000	8.500000	126.000000	816.000000	1474.00000	55.359566	
24	"	8.080000	8.200000	124.700000	740.000000	1633.00000	45.315370	
25	10/12/84	7.480000	9.400000	129.050000	635.000000	1351.00000	47.002221	
26	"	7.470000	9.200000	129.150000	794.000000	1646.00000	48.238153	
27	10/15/84	7.630000	9.100000	129.250000	770.000000	1398.00000	55.078684	
28	"	7.640000	9.300000	128.950000	852.000000	1375.00000	61.963636	
29	10/16/84	7.700000	9.400000	127.650000	669.000000	1235.00000	54.170040	
30	"	7.800000	9.800000	128.900000	712.000000	1283.00000	55.494934	
31	10/17/84	7.700000	8.400000	127.350000	829.000000	1454.00000	57.015131	
32	"	7.640000	8.200000	128.500000	830.000000	1582.00000	52.465234	
33	10/18/84	7.770000	8.000000	125.850000	664.000000	1351.00000	49.148779	
34	"	7.680000	8.300000	125.400000	744.000000	1381.00000	53.874004	
35	10/19/84	7.720000	8.000000	123.750000	812.000000	1676.00000	48.448687	
36	"	7.720000	7.500000	124.350000	748.000000	1643.00000	45.526476	
37	10/23/84	7.550000	8.800000	126.600000	629.000000	1454.00000	43.259972	
45	11/05/84	7.630000	8.800000	126.500000	681.000000	1376.00000	49.491279	
46	"	7.600000	8.400000	125.400000	642.000000	1319.00000	48.673237	
47	11/0[illegible]/84	7.460000	9.800000	125.300000	475.000000	1295.00000	36.679537	
48	"	7.370000	10.000000	125.050000	484.000000	1295.00000	37.374517	
49	11/07/84	7.420000	9.200000	126.350000	495.000000	1492.00000	33.176944	
50	"	7.420000	10.000000	129.600000	506.000000	1488.00000	39.381720	
51	11/0[illegible]/84	7.030000	9.000000	130.500000	777.000000	1319.00000	58.908264	
52	"	7.640000	10.000000	131.950000	799.000000	1831.00000	43.637357	
53	[illegible]	7.630000	9.400000	127.500000	641.000000	1597.00000	40.137750	
54	[illegible]	7.500000	8.800000	126.700000	599.000000	1507.00000	39.747843	
11	10/[illegible]/84	8.330000	9.400000	130.400000	N/A	1366.00000	N/A	* 7 DAY ONLY
12	"	8.460000	9.600000	129.450000	N/A	1353.00000	N/A	* 7 DAY ONLY
38	10/23/84	7.540000	9.200000	125.100000	N/A	1477.00000	N/A	* 24 HR VOID / BROKEN
39	10/24/84	7.540000	N/A	N/A	549.000000	1297.00000	42.328450	* NO % MOIST. OR DENSITY
40	"	7.540000	N/A	N/A	535.000000	1283.00000	41.699143	* NO % MOIST. OR DENSITY
41	[illegible]/30/84	7.240000	9.200000	129.300000	N/A	1410.50000	N/A	* AVG. 7 DAY BREAK
42	"	7.340000	9.100000	129.550000	N/A	1416.00000	N/A	* AVG. 7 DAY BREAK
43	10/[illegible]/84	7.610000	9.400000	128.950000	N/A	1447.00000	N/A	* AVG. 7 DAY BREAK

44	"	7.640000	9.600000	130.300000	N/A	1446.50000	N/A	* AVG. 7 DAY BREAK
55	11/12/84	7.240000	10.200000	129.350000	N/A	1420.00000	N/A	* AVG. 7 DAY BREAK
56	"	7.410000	10.400000	129.850000	N/A	1462.00000	N/A	* AVG. 7 DAY BREAK
AVERAGE	----	7.981778	9.053333	128.235555	709.200000	1389.66666	51.218396	* AVG OF FIRST 44 SAMPLES
STD DEV	----	0.431026	0.704724	3.199993	122.865151	170.389103	7.915914	* SDV OF FIRST 44 SAMPLES

*** PANTANO/GOLF LINKS SOIL CEMENT BANK PROTECTION *** 4885TM ***

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
MIX DESN	2/21/85	8.000000	8.800000	131.100000	522.000000	1108.00000	47.119130	START CEMENT CONTENT 9.0%
1	2/26/85	9.260000	7.000000	131.000000	1004.00000	1881.00000	53.375864	
3	2/28/85	8.240000	6.900000	131.800000	1324.00000	2159.00000	61.324687	
4	"	8.240000	7.200000	134.600000	1150.00000	2407.00000	47.777316	
5	"	8.000000	6.100000	133.100000	879.000000	1727.00000	50.897510	
6	3/01/85	8.360000	6.900000	134.400000	1091.00000	2290.00000	47.641921	
7	"	8.750000	6.900000	133.500000	1110.00000	2152.00000	51.579926	
8	"	8.000000	6.600000	133.400000	796.000000	1737.00000	45.826137	
12	3/06/85	8.090000	6.300000	134.000000	904.000000	1571.00000	57.542966	
13	"	7.950000	6.700000	134.500000	801.000000	1433.00000	55.896720	
14	"	7.920000	5.900000	133.900000	975.000000	1780.00000	54.775281	
2	2/26/85	8.300000	7.200000	130.000000	N/A	1621.00000	N/A	* 7 DAY BREAK ONLY
9	3/04/85	7.930000	6.300000	133.800000	998.000000	1512.00000	N/A	W/O CAP COMP 7D
10	"	8.080000	6.800000	134.500000	1102.00000	1223.00000	N/A	W/O CAP COMP 7D
11	"	8.000000	6.800000	135.200000	733.000000	1011.00000	N/A	W/O CAP COMP 7D
AVERAGE	----	8.281000	6.650000	133.420000	1003.40000	1918.70000	52.663833	* AVERAGE OF FIRST 10 SAMPLES
STD DEV	----	0.424564	0.422295	1.186779	167.814845	322.994875	4.880569	* STD DEV OF FIRST 10 SAMPLES

********* PANTANO WASH/TANQUE VERDE SOIL CEMENT ********* 4FPNTV *********

8,825 CU. YDS. OF S/C

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
3	4/17/84	9.200000	8.700000	133.700000	1454.00000	2403.00000	60.507699	
4	"	9.400000	11.000000	131.800000	1447.00000	2545.00000	56.856582	
5	4/18/84	9.100000	10.400000	134.450000	1305.00000	2249.00000	58.025789	
6	"	9.180000	11.200000	130.850000	1346.00000	2350.00000	57.276596	
7	"	9.230000	9.900000	132.650000	1219.00000	2077.00000	58.690419	
8	4/20/84	8.630000	9.400000	133.600000	1491.00000	2385.00000	62.515723	
9	"	8.550000	10.600000	132.300000	1358.00000	2474.00000	49.435748	
10	4/23/84	9.550000	10.400000	130.500000	1203.00000	2317.00000	51.920587	
1	4/16/84	N/A	10.200000	N/A	1464.00000	2414.00000	N/A	
2	"	N/A	11.900000	N/A	1216.00000	2069.00000	N/A	
AVERAGE	----	9.105000	10.200000	132.481250	1352.87500	2884.12500	56.903643	* AVERAGE OF FIRST 8 SAMPLES
STD DEV	----	0.348261	0.833238	1.400239	107.514036	198.627748	4.304119	* STD DEV OF FIRST 8 SAMPLES

********* CANADA DEL ORO FLOOD CONTROL PROJECT ********* 4FOVCD *********

45,200 CU. YDS. S/C

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
MIX DESN	12/20/83	6.000000	0.900000	----	310.000000	572.000000	54.195804	START CEMENT CONTENT 8.0%
"	"	8.000000	8.900000	----	490.000000	1054.00000	46.409564	
"	"	10.000000	8.900000	----	794.000000	1728.00000	46.082414	
1	12/24/83	11.510000	11.000000	----	661.000000	1352.00000	48.890533	
2	"	11.100000	11.600000	----	616.000000	1131.00000	54.465075	
3	12/26/83	11.400000	10.000000	----	941.000000	1540.00000	61.103896	
4	12/26/83	12.000000	9.500000	----	818.500000	1434.00000	57.078103	*AVG 24HR AND 7 DAY BREAK
5	12/27/83	10.600000	11.400000	----	870.000000	1945.00000	44.730077	
6	"	11.320000	11.800000	----	697.000000	1317.00000	52.923311	
7	"	9.970000	9.400000	----	438.000000	813.000000	53.874539	
8	12/28/83	10.000000	11.000000	----	707.000000	1622.00000	43.588163	
9	"	9.580000	9.700000	----	645.000000	1276.00000	50.548589	
10	"	9.500000	8.800000	----	535.000000	1022.00000	52.348337	
11	12/29/83	9.680000	9.400000	----	460.000000	869.000000	52.934407	* LOW BREAKS DUE TO CHANGE IN STOCKPILE MATERIAL FOR #'S 11, 12 AND 13.
12	"	9.540000	12.400000	----	537.000000	822.000000	65.328467	
13	"	9.170000	10.200000	----	383.000000	911.000000	42.041712	
14	12/30/83	9.570000	12.100000	----	613.000000	1198.00000	51.168614	
15	"	9.700000	10.400000	----	608.000000	1218.00000	49.917898	
16	"	9.800000	9.000000	----	613.000000	1098.00000	55.828780	
17	"	10.200000	9.400000	----	605.000000	1094.00000	55.301645	
18	12/31/83	10.200000	8.600000	----	702.000000	1280.00000	54.843750	
19	"	10.200000	7.600000	----	758.000000	1104.00000	68.659420	
20	"	9.928000	8.900000	----	694.000000	1480.00000	46.891892	
21	"	10.260000	10.200000	----	835.000000	1681.00000	49.672814	
22	1/02/84	9.500000	8.800000	----	878.000000	1426.00000	61.570827	
23	"	9.700000	9.200000	----	760.000000	1250.00000	60.800000	
24	"	9.900000	10.700000	----	851.000000	1530.00000	55.620915	
25	"	9.880000	7.600000	----	791.000000	1403.00000	56.379187	
26	1/03/84	10.450000	8.900000	----	1003.00000	1792.00000	55.970982	
27	"	10.100000	8.600000	----	1057.00000	2132.00000	49.577861	
28	"	9.700000	8.200000	----	1246.00000	2700.00000	46.148148	
29	1/04/84	9.300000	9.700000	----	865.000000	1571.00000	55.060471	
30	"	9.500000	9.800000	----	647.000000	1501.00000	43.104597	
31	1/05/84	9.620000	9.800000	----	878.000000	1676.00000	52.386635	
33	1/06/84	9.480000	9.700000	----	778.000000	1792.00000	43.415179	
34	"	9.450000	9.200000	----	749.000000	1404.00000	53.347578	
35	"	10.200000	8.800000	----	662.000000	1255.00000	52.749004	
36	"	9.200000	8.600000	----	736.000000	1482.00000	49.662618	
39	1/09/84	9.660000	9.000000	----	1014.00000	1868.00000	54.282655	
40	"	9.600000	11.000000	----	1014.00000	2127.00000	47.672779	
41	"	9.600000	10.200000	----	813.000000	1821.00000	44.645799	
42	1/10/84	9.660000	9.400000	----	699.000000	1207.00000	57.912179	
43	"	10.460000	9.600000	----	784.000000	1491.00000	52.582160	
44	"	10.000000	9.200000	----	784.000000	1685.00000	46.528190	
45	1/11/84	9.370000	8.600000	----	806.000000	1477.00000	54.570074	
46	"	10.900000	8.800000	----	820.000000	1246.00000	65.810594	
47	"	9.510000	9.200000	----	806.000000	1245.00000	64.738956	
48	1/12/84	9.530000	10.200000	----	764.000000	1397.00000	54.688618	
49	"	9.100000	10.400000	----	742.000000	1511.00000	49.106552	
50	"	9.550000	10.600000	----	613.000000	1310.00000	46.793893	
51	1/13/84	9.500000	9.000000	----	934.000000	1778.00000	52.530934	
52	"	9.520000	9.500000	----	000.000000	1428.00000	56.022409	
53	"	9.530000	8.400000	----	726.000000	1292.00000	56.191950	
54	1/14/84	9.520000	9.700000	----	1573.00000	1585.00000	99.242902	
32	1/05/84	N/A	9.700000	----	N/A	N/A	N/A	* PLUGS VOID / OLD MATERIAL
37	1/07/84	9.850000	9.400000	----	1421.00000	1460.00000	N/A	
38	"	9.550000	8.800000	----	1312.00000	1221.00000	N/A	
AVERAGE	----	9.915647	9.650980	----	771.166667	1442.92156	53.946170	* AVERAGE OF FIRST 52 SAMPLES
STD DEV	--	0.624195	1.087010	----	197.972026	351.870130	8.860078	* STD DEV OF FIRST 52 SAMPLES

********** THORNYDALE ROAD BRIDGE S.C. BANK PROTECTION ********** 48TDEL *** ******

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
MIX DESN	11/17/82	8.000000	9.400000	---	---	561.000000	---	RECOMMENDED CEMENT CONTENT 11.5%
"	"	10.000000	9.400000	---	---	940.000000	---	
"	"	12.000000	9.400000	---	---	1464.00000	---	
"	"	14.000000	9.400000	---	---	2175.00000	---	
1	11/23/82	11.500000	8.500000	---	758.000000	1449.00000	52.811989	
2	"	11.500000	10.800000	---	720.000000	1418.00000	50.775740	
4	11/24/82	11.500000	9.800000	---	721.000000	1529.00000	47.155003	
5	"	11.500000	10.800000	---	797.000000	1630.00000	45.214724	
6	"	11.500000	9.300000	---	690.000000	1477.00000	46.716817	
7	11/29/82	11.500000	9.700000	---	871.000000	1649.00000	52.819891	
8	"	11.500000	9.200000	---	807.000000	1575.00000	51.238095	
9	"	11.500000	9.800000	---	448.000000	1010.00000	44.356436	
10	11/30/82	11.500000	9.500000	---	819.000000	1785.00000	45.882353	
11	"	11.500000	10.900000	---	786.000000	1489.00000	52.787105	
12	"	11.500000	9.500000	---	905.000000	1764.00000	51.303855	
13	12/02/82	10.500000	9.900000	---	683.000000	1521.00000	44.904668	
14	"	10.500000	10.600000	---	661.000000	1564.00000	42.263427	
15	"	10.500000	10.200000	---	731.000000	1879.00000	38.903672	
16	12/03/82	10.500000	9.900000	---	691.000000	1258.00000	54.928458	
17	"	10.500000	9.800000	---	731.000000	1244.00000	58.762058	
18	"	10.500000	10.20000	---	801.000000	1297.00000	61.757903	
19	12/06/82	10.500000	10.80000	---	651.000000	1119.00000	58.176944	
20	"	10.500000	11.70000	---	581.000000	1091.00000	53.253896	
21	"	10.500000	10.00000	---	844.000000	1550.00000	54.451613	
22	12/07/82	10.500000	9.500000	---	734.000000	1441.00000	50.936849	
23	"	10.500000	9.900000	---	696.000000	1322.00000	52.647504	
24	"	10.500000	10.600000	---	785.000000	1497.00000	49.098196	
25	12/08/82	10.500000	9.900000	---	807.000000	1391.00000	58.015816	
26	"	10.500000	10.100000	---	796.000000	1470.00000	54.149660	
27	"	10.500000	10.500000	---	897.000000	1824.00000	49.177632	
28	12/09/82	10.500000	11.500000	---	674.000000	1155.00000	58.354978	
29	"	10.500000	11.700000	---	592.000000	1220.00000	48.524590	
31	12/13/82	10.500000	9.800000	---	754.000000	1420.00000	53.098592	
32	"	10.500000	10.600000	---	821.000000	1432.00000	57.332402	
33	"	10.500000	10.400000	---	645.000000	1301.00000	49.577248	
34	12/14/82	10.500000	11.200000	---	610.000000	1298.00000	46.995378	
35	"	10.500000	9.800000	---	840.000000	1671.00000	50.269300	
36	"	10.500000	9.100000	---	908.000000	1496.00000	60.695187	
37	12/15/82	10.500000	10.400000	---	866.000000	1623.00000	53.357979	
38	"	10.500000	9.100000	---	938.000000	1693.00000	55.404607	
39	"	10.500000	10.400000	---	759.000000	1572.00000	48.282443	
3	11/23/82	10.500000	9.600000	---	N/A	1652.00000	N/A	* 7 DAY ONLY
30	12/09/82	10.500000	10.800000	---	N/A	397.000000	N/A	* SET DAMAGED
40	12/16/82	10.500000	9.300000	---	N/A	1470.00000	N/A	* AVG. 7 DAY BREAKS
41	"	10.500000	9.800000	---	N/A	1505.00000	N/A	* AVG. 7 DAY BREAKS
42	"	10.500000	10.800000	---	N/A	1662.50000	N/A	* AVG. 7 DAY BREAKS
43	12/17/82	10.500000	9.400000	---	N/A	1914.50000	N/A	* AVG. 7 DAY BREAKS
44	"	10.500000	10.900000	---	N/A	1005.50000	N/A	* AVG. 7 DAY BREAKS
45	"	10.500000	10.400000	---	N/A	1301.50000	N/A	* AVG. 7 DAY BREAKS
AVERAGE		10.744444	8.791111	---	748.864865	1462.81081	51.45[illegible]	
STD DEV		0.434613	9.523094	---	104.063592	207.660492	5.206393	

********* RUTHRAUFF ROAD PROJECT C.T.B. (LA CHOLLA BLVD TO I-10) ********* 4BRAUF *********

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS PSI	COMP STR 7DAY PSI	STR GAIN (%)	REMARKS
MIX DESN	12/10/84	5.000000	8.600000	133.100000	342.000000	619.000000	55.250404	
1	2/14/85	5.100000	6.700000	132.700000	216.000000	324.000000	66.666667	
2	"	4.900000	6.900000	132.700000	303.000000	432.000000	70.138889	
3	"	4.900000	6.900000	133.300000	502.000000	853.000000	58.851114	
4	"	5.000000	7.200000	134.000000	389.000000	585.000000	66.495726	
5	2/15/85	4.800000	8.000000	132.000000	294.000000	647.000000	45.440495	
6	"	5.000000	8.000000	132.800000	421.000000	776.000000	54.252577	
7	"	4.800000	7.100000	133.300000	459.000000	792.000000	57.954545	
8	"	5.100000	7.600000	132.900000	296.000000	541.000000	54.713494	
9	2/16/85	5.000000	7.400000	135.100000	281.000000	518.000000	54.247104	
10	"	5.100000	8.100000	132.500000	361.000000	606.000000	59.570957	
11	2/18/85	5.000000	6.600000	133.000000	311.000000	571.000000	54.465849	
12	"	5.000000	6.300000	131.600000	420.000000	671.000000	62.598145	
13	"	4.800000	7.200000	134.500000	556.000000	881.000000	63.110102	
14	2/19/85	4.900000	6.500000	134.600000	469.000000	672.000000	69.791667	
15	3/1/85	4.900000	7.100000	134.400000	287.000000	455.000000	63.076923	
16	"	5.100000	6.900000	134.100000	332.000000	615.000000	53.983740	
17	"	5.000000	7.000000	133.900000	339.000000	551.000000	61.524501	
18	3/4/85	5.100000	7.200000	134.100000	318.000000	606.000000	52.475248	
19	"	4.900000	7.400000	133.000000	311.000000	548.000000	56.751825	
20	"	4.900000	7.200000	134.900000	448.000000	885.000000	50.621469	
21	"	5.100000	7.200000	133.700000	439.000000	910.000000	48.241758	
22	3/5/85	5.200000	7.400000	133.900000	491.000000	939.000000	52.289670	
23	"	5.000000	8.100000	133.900000	556.000000	1114.00000	49.910233	
24	"	4.900000	6.900000	133.700000	388.000000	619.000000	62.681745	
25	3/6/85	4.900000	7.200000	133.100000	426.000000	657.000000	64.840183	
26	"	5.200000	7.200000	134.400000	449.000000	868.000000	51.728111	
27	"	5.000000	7.400000	134.400000	454.000000	857.000000	52.975496	
28	"	5.000000	6.800000	135.400000	418.000000	701.000000	59.629101	
29	3/7/85	4.900000	8.000000	132.500000	273.000000	381.000000	71.653543	
30	"	5.100000	6.700000	135.700000	542.000000	852.000000	63.615023	
31	"	5.100000	7.500000	132.900000	252.000000	425.000000	59.294118	
32	"	5.100000	7.000000	135.100000	275.000000	504.000000	54.563492	
AVERAGE	----	4.993750	7.234375	133.690625	383.625000	667.375000	58.379641	
STD DEV	----	0.110534	0.475604	1.010065	94.488009	186.609261	6.696505	

********* C.D.D. AT FIRST AVENUE BANK PROTECTION ********* 4FOVCD *********

COMPARISON OF 4in.X 4.5in. CYLINDERS TO 4in.X 8in. CYLINDERS

SAMP NO	DATE SAMP	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF) 4 x 4.5in.	COMP STR 7DAY PSI 4 x 4.5in.	DRY DENS (PCF) 4 x 8in.	COMP STR 7DAY PSI 4 x 8in.	REMARKS
1	7/26/84	8.860000	10.800000	127.500000	1540.00000	127.800000	1600.00000	
2	"	9.200000	11.300000	131.200000	1886.00000	132.100000	1739.00000	
3	7/27/84	8.570000	11.300000	128.200000	1130.00000	128.000000	1179.00000	
4	"	8.600000	10.600000	128.800000	1128.00000	130.200000	1046.00000	
6	7/30/84	7.980000	10.900000	125.700000	1083.00000	127.700000	1060.00000	
7	"	8.730000	10.000000	122.600000	937.000000	122.600000	976.000000	
8	7/31/84	9.290000	9.500000	127.300000	1345.00000	128.400000	1582.00000	
9	"	9.860000	9.800000	129.700000	1751.00000	129.800000	1392.00000	
12	8/01/84	8.120000	10.200000	126.200000	1456.00000	128.000000	1450.00000	
13	"	8.680000	10.700000	127.800000	1237.00000	126.700000	1218.00000	
17	8/02/84	8.340000	10.500000	128.700000	1621.00000	128.100000	1621.00000	
19	"	8.900000	10.900000	130.000000	1132.00000	129.600000	1030.00000	
21	8/03/84	8.420000	10.200000	125.200000	1609.00000	126.600000	1320 00000	
22	"	8.680000	9.400000	126.100000	1211.00000	125.200000	1247.00000	
25	8/06/84	8.400000	10.600000	127.200000	1533.00000	129.400000	1485.00000	
26	"	8.350000	10.800000	126.200000	985.000000	127.100000	1028.00000	
28	8/07/84	8.230000	9.200000	124.900000	1102.00000	125.600000	1228.00000	
29	"	8.490000	10.000000	127.100000	1196.00000	128.400000	1199.00000	
31	8/08/84	8.280000	9.400000	122.300000	829.000000	122.100000	810.000000	
34	8/09/84	7.520000	9.200000	127.400000	1293.00000	128.900000	1304.00000	
35	"	9.100000	10.400000	128.400000	1593.00000	130.000000	1612.00000	
AVERAGE	----	8.571429	10.271429	127.071429	1314.14285	127.728571	1291.71428	
STD DEV	----	0.521529	0.660411	2.207067	284.060607	2.399613	253.024533	

NOTE: 4x4.5in. CYLINDERS, 3 LIFTS, 25 BLOWS PER LIFT, 5.5lb. HAMMER, 12in. DROP

4x8in. CYLINDERS, 5 LIFTS, 26 BLOWS PER LIFT, 5.5lb. HAMMER, 12in. DROP

SECTION 7 - KEY PERSONNEL

Charles H. Huckelberry
Director/County Engineer
Pima County Department of Transportation
& Flood Control District

EDUCATION

Bachelor of Science in Mining Engineering, 1972, University of Arizona, Tucson, Arizona

Master of Science in Civil Engineering, 1976, University of Arizona, Tucson, Arizona

PROFESSIONAL LICENSES

Registered Professional Engineer (Civil) in the State of Arizona, Registration No. 10502

Registered Land Surveyor in the State of Arizona, Registration No. 8901

Licensed Land Surveyor in the State of California, Registration No. 4049

JOB DESCRIPTION

Director/County Engineer
Chief Engineer of Flood Control District
Pima County Department of Transportation
& Flood Control District
Tucson, Arizona
March,1979 - Present

Responsible for planning, directing, organizing and managing the activities of the Pima County Department of Transportation and Flood Control District, an organization of Pima County government with 350 employees and an annual operating 1984/85 budget of approximately 27 million dollars and a capital improvement budget of 45 million dollars.

Suzanne J. Shields
Division Manager
Planning & Development Division

EDUCATION

Bachelor of Science in Hydrology from the College of Earth Science at the University of Arizona: December, 1976.

Master of Science in Hydrology from the University of Arizona: graduation date of December, 1982. Grade point average of 3.4. Courses taken for the degree were
Hydrology for 23 units; Soil and Water Engineering for 6 units; Math for 6 units; System Engineering for 3 units; Civil Engineering for 6 units; Water Resources for 3 units:

Continuing Education Courses:
- "Ground Water Modeling"
  Colorado State University, 1980, CSU 3 units
- "Hydraulic, Erosion, and Sedimentation Analysis in Arid Areas"
  Simons, Li and Associates, 1981, U of A 2.1 units
- "Fundamentals of Urban Stormwater Management"
  Water Resources Management Science, 1982
- "Floodplain Management and National Flood Insurance Workshop"
  ADWR and FEMA, 1982
- "Disaster Assistance Seminar"
  FEMA, 1983
- "Management for Engineers"
  Battelle Corporation, 1983, 1.2 units

PROFESSIONAL REGISTRATION

Registered Professional Engineer (Civil) in the State of Arizona, Registration No. 15610

JOB DESCRIPTION

Division Manager - Planning & Development Division
Pima County Department of Transportation
& Flood Control District
Tucson, Arizona
May, 1984 to Present

Manager for the Planning & Development Division. Responsibilities include Subdivision Coordination, Subdivision Engineering, Transportation Planning, Flood Plain Management and Flood Control Planning which include approximately 35 employees. As the manager the work responsibilities include providing coordination of subdivision development and rezoning requirements; project

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manager for transportation and flood control planning studies; review and approval of street and drainage improvement plans; preparation of the Capital Improvement Program for Pima County Department of Transportation and Flood Control District and management of all aspects of the Pima County Flood Control District (i.e. budget, flood plain management, planning, etc.)

PUBLICATIONS

Drainage Development and Channel Design Standards, Pima County Department of Transportation and Flood Control District, 1982

Evapotranspiration in a Desert Environment, University of Arizona, Thesis, 1982

Existing Capabilities for Measurement and Forecasting Soil Moisture by Remote Means, co-authored with D. Evans and L. Onyskow, U.S. Department of Army Contract, DACA-39-77-M-0179, 1977

Lawrence E. Maucher
Division Manager
Design & Field Engineering

EDUCATION

Mayville Central School - Mayville, New York through 10th grade
Casa Grande Union High School - Casa Grande, Arizona
graduated 1964
University of Arizona - Tucson, Arizona graduated 1969
Bachelor of Science in Civil Engineering

REGISTRATION

Registered Professional Engineer (Civil) in the State of Arizona, Registration No. 9370

JOB DESCRIPTION

Manager of Engineering & Field Engineering Divisions
Pima County Department of Transportation
& Flood Control District
Tucson, Arizona
August 1984 - Present

Responsible for managing the operation and progress of two Divisions of approximately 140 personnel with the following duties:

A. Design Engineering Division
In-house design of approximately $10 million of major arterials, bridges, drainage improvements, river stabilization, safety and signalization projects annually. The majority of these projects are in the Tucson, Arizona, metropolitan area. Other projects such as drainage, access and paving improvements are also undertaken in the rural areas of a County of 9,240 square miles.

Consultant design of in excess of $50 million annually of projects similar in nature to those listed above but also including airport, Federal and State aid projects.

Included under the Design Engineering Division are the following sections:

1. Computer Applications Section
Responsible for computer support of the entire Department.

2. Records Section
Duties include road establishment and abandonment proceedings, storing, microfilming and retrieval of all Departmental files, records, plans and plats.

3. Contracts and Specifications Section
Negotiates and monitors all consultant contracts as well as acting as Departmental liaison on all Federal-aid projects ($30-40 million) while under design and construction. Also responsible for development of all construction specifications, special provisions and contract documents.

B. Field Engineering Division
Responsible for the inspection and testing of all Pima County Transportation and Flood control construction projects totaling $20 to $30 million annually. Major sections include:

1. Inspections Section
Performs all construction inspection on those projects designed or administered by the Design Section (currently at $48 million).

2. Survey Section
Responsible for design, construction and property surveys for the Department as well as County-wide mapping and establishment and maintenance of all survey monuments within Pima County.

3. Materials Section
Performs all quality control on construction and maintenance materials within the Department such as embankment, aggregate base course, asphalt, asphaltic cement concrete, portland cement concrete, concrete aggregates, soil-cement, slurry and chip seals, reinforcing steel, etc.

4. Permit Section
Issue all permits for work within County right-of-way and perform inspection on same. This includes all new subdivision roadway work which averages $10 million.

David A. Smutzer
Manager
Flood Control Planning Section
Planning & Development Division

E D U C A T I O N

Bachelor of Science in Physical Geography, 1977, Pennsylvania State University

Flood Plain Hydraulic, HEC-II Seminar, 1980, Pennsylvania State University

Flood Plain hydrology, HEC-I Seminar, 1982, University of Texas

Open-Channel Hydraulics, 1983, New Mexico State University

Applied Sedimentation and River Engineering, 1984, San Diego State University

J O B D E S C R I P T I O N

Hydrologist, Manager, Flood Control Planning Section
Pima County Department of Transportation
& Flood Control District
Flood Control Planning
Tucson, Arizona
May 1984 - Present

Evaluate scope of services and cost estimates of hydrologic and hydraulic analyses of proposed County projects submitted by local engineering firms. Define study areas to determine existing flood hazards and establish need studies to evaluate possible drainage improvements. Review reports and plans for proposed drainage improvements and flood control projects. Coordinate FEMA restudy efforts, including defining areas of restudy, upgrading existing area plans, defining existing flooding problems, and establishing flood control policies relating to development. Supervise review of rezoning applications, flood warning system and KAVOURAS Radar System.

P U B L I C A T I O N S

Modeling Unsteady Flows in Large Basins, The Santa Cruz Experience
(with V. Miguel Ponce, Zbig Osmolski)
Tucson, Arizona, 1985

Computer-Based Design of River Bank Protection
(with Howard H. Change, Zbig Osmolski)
Tucson, Arizona, 1985

Ahmed A. Taji-Farouki
Manager
Materials Section
Field Engineering Division

EDUCATION

El Nasr School (Ex. The English School) - Heliopolis, Cairo Egypt through High School - Graduated June, 1970

Bachelor of Science Degree in Materials Engineering, January, 1976, The American University in Cairo - Cairo, Egypt

Master of Science in Civil Engineering, September, 1977

JOB DESCRIPTION

Materials Lab Supervisor - Field Engineering Division
Pima County Department of Transportation
& Flood Control District
November, 1983 - Present

Responsibilities included:

1. Supervising and managing the field and laboratory testing of construction materials relative to highway, drainage and subdivision construction projects. The materials section is currently comprise of a total of 14 personnel.

2. Supervising and conducting preliminary soil investigation and preparation of soil and foundation investigation reports.

3. Developing and control of Asphalt Concrete, Portland Cement Concrete, Soil Cement mix design and job mix formulae.

4. Performing research and experimentation on construction materials and providing consultation on complex matters of materials and soils engineering.

5. Formulating, developing and upgrading of utilized construction specifications.

Zbigniew Osmolski
Manager
Flood Control Design Section
Design Engineering Division

EDUCATION

Bachelor of Science, Civil Engineering, 1967, Central School of Agriculture, Warsaw Poland

Master of Science, Civil Engineering, 1969, Central School of Agriculture, Warsaw, Poland

Doctoral Studies in Watershed Management and Hydrology, 1969 - 1972, Warsaw Technical University, Warsaw, Poland

Faculty of Mathematics, 1974, University of Warsaw, Warsaw, Poland

Doctor of Philosophy Candidate, 1985, Watershed Hydrology, University of Arizona, Tucson, Arizona

REGISTRATIONS

Registered Professional Engineer (Civil) in the State of Arizona, No. 14487

Registered Land Surveyor in the State of Arizona, No. 17409

JOB DESCRIPTION

Manager, Flood Control Design Section
Design Engineering Division
Pima County Department of Transportation
& Flood Control District
Tucson, Arizona
May 1984 - Present

Directly responsible for technical review of private consulting Engineering design contracts, including reports and plans associated with roadways, bridges, storm drains, flood detention/retention facilities, river mechanics and river management plans, flood control structures, such as grade controls and bank protection.

In house: design projects, including roadways, storm drains, bridges and flood control structures. Typical project included the design of roadways, bridges and flood control improvements for the Canada del Oro Flood Control Improvement Project, Phases I and II (estimated construction cost of 7.3 million dollars).

P U B L I C A T I O N S

Methodes de Calcul des Reseaux E'Egouts
Brussels, Belguim, 1979

Estimating Potential Evapotranspiration in Arid Environments
Tucson, Arizona, 1981

Comparison of Bowen Ratio Estimates from Two sets of Sensors
Fort Collins, Colorado, 1983

Estimating Daily Evapotranspiration from a Single Measurement
(with Lloyd W. Gay, Ph. D.)
Fort Collins, Colorado, 1983

The Surface Energy Budget of A Sonoran Desert Site
(with Lloyd W. Gay, Ph. D.)
Technical Report for the U.S. Army Engineers
Tucson, Arizona, 1983

Daily Rate of Evapotranspiration in Arid Environment, A comparison Between Bowen Ratio Technique and Lysimeter Measurements
(with Lloyd W. Gay, Ph. D.)
Technical Report for Water Conservation laboratory of the United States Department of Agriculture
Phoenix, Arizona, 1983

Limitations of Penman's Equation in Arid Zones
(with Lloyd W. Gay, Ph. D)
Tucson, Arizona, 1985

Modeling Unsteady Flows in Large Basins, The Santa Cruz Experience
(with V. Miguel Ponce, Dave Smutzer)
Tucson, Arizona, 1985

Computer-Based Design of River Bank Protection
(with Howard H. Chang, Zbig Osmolski, David Smutzer)
Tucson, Arizona 1985

SECTION 8 TESTIMONY ON 1983 FLOOD DISASTER BEFORE CONGRESSIONAL SUBCOMMITTEE

Testimony of David Yetman, Vice Chairman of Pima County Board of Supervisors before the House of Representatives Subcommittee on Transportation, Public Works Committee, United States Congress, October 17, 1984.

Pima County has experienced four presidentially declared flood disasters since October of 1977. With each flooding event, it is more and more obvious that present methods of flood damage from inundation simply do not work in the arid Southwest. During the October 1983 flood disaster, there were almost no inundation damages on the Rillito River; yet, five of seven bridges crossing the Rillito River were closed because of erosion damage. Historically, Pima County has received a good deal of study effort from the federal government and the Corps of Engineers regarding flood control and water resources in southern Arizona. Every time flood control is addressed, federal participation is discounted because of the lack of an appropriate cost-benefit ratio. These cost-benefit ratios do not take into account, and never have, the damage component of erosion and sedimentation which is the principal economic damage component of flooding in the arid Southwest. The federal role in flood control must be re-evaluated based on the obvious facts of erosion and sedimentation damage. The extreme soil loss and erosion in the upper Santa Cruz Basin ended up as sedimentation and deposition in the lower Santa Cruz Basin. In the area of Marana, this meant decreasing the natural flood carrying capability of the Santa Cruz River. Federal assistance should be given for the total management of the river system. No single flood control strategy is appropriate in southern Arizona. We must find ways to protect our existing population from the harmful effects of flood waters. We must also present our future residents from unknowingly becoming exposed to flood hazards, and we must protect and preserve our unique riverine environment in the Southwest. Federal recognition that the Southwest has unique flooding problems which require unique solutions is essential. I ask that this sub-committee of the Public Works Committee strive to recognize the unusual flooding problems of the arid Southwest, as traditional approaches simply do not work.

********* SANDERS ROAD BRIDGE SOIL CEMENT BANK PROTECTION ********* 485ASC *********

SAMPLE NUMBER	DATE SAMPLED	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS psi	COMP STR 7DAY psi	STR. GAIN (%)	REMARKS
MIX DESIGN	03/15/85	8.0	11.4	121.9	243	573	42.4	
"	"	10.0	"	122.9	356	794	44.8	RECOMMENDED CEMENT CONTENT
"	"	12.0	"	123.8	485	1076	45.1	TO START 12.0 %
"	03/17/85	14.0	"	125.4	659	1673	39.0	
"	"	16.0	"	126.8	925	2393	38.7	
1	03/25/85	N/A	N/A	N/A	678	1235	54.9	
2	"	12.0	7.5	126.1	871	1701	51.2	
3	"	11.9	8.2	126.2	965	1932	49.9	
4	03/26/85	N/A	N/A	N/A	944	1840	51.3	
5	03/27/85	11.5	7.2	127.6	1041	1719	60.6	
6	"	11.6	7.7	127.7	1189	2072	57.4	
7	"	11.5	8.0	129.0	1247	2512	49.6	
8	03/28/85	11.5	9.5	126.7	1225	1990	61.6	
9	"	11.5	10.2	126.2	1510	2681	56.3	
10	"	11.52	9.9	126.7	1097	2011	54.5	
11	03/29/85	12.0	9.6	124.5	720	1766	40.8	
12	"	11.0	9.0	124.0	759	1634	46.5	
13	"	11.2	9.3	124.3	1001	2125	47.1	
14	04/01/85	11.5	9.5	128.1	996	1701	58.7	
15	"	10.5	9.3	125.4	819	1462	56.0	
16	"	10.5	9.0	122.4	595	1168	50.9	
17	04/02/85	10.5	9.3	125.6	868	1680	51.7	
18	"	10.5	9.6	129.2	855	1562	54.7	
19	"	10.5	9.6	122.9	942	1880	50.1	
20	04/03/85	10.5	9.0	124.9	798	1452	55.0	
21	"	10.5	9.3	122.9	764	1338	57.1	
22	"	10.3	8.2	125.0	625	1162	53.8	
23	04/04/85	10.0	8.8	122.7	761	1204	63.2	
24	"	10.1	9.3	123.5	647	1122	57.7	
25	"	10.1	9.5	124.3	1023	1727	59.2	
26	"	10.2	9.3	123.1	799	1600	49.9	
27	04/05/85	10.0	7.3	121.5	867	1423	60.9	
28	"	10.0	9.5	126.0	817	1799	45.4	
AVERAGE	----------	10.0	8.6	125.0	900	1696	53.8	
STD DEV.	-- -----	3.0	1.9	2.0	210	389	5.4	

********* CAMINO DEL CERRO SOIL CEMENT BANK PROTECTION ********* 4005TE *********

SAMPLE NUMBER	DATE SAMPLED	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS psi	COMP STR 7DAY psi	STR. GAIN (%)	REMARKS
MIX DESIGN	04/16/85	7.0	10.7	127.2	424	757	56.0	RECOMMENDED CEMENT CONTENT
		9.0	10.7	129.2	694	1028	67.5	TO START 9.0%
		11.0	10.7	129.0	879	1644	53.5	RECOMMENDED MOISTURE CONTENT
		9.0	8.7	129.2	1169	1740	66.8	TO START 9.0%
								The following are results
1	04/22/85	9.0	8.2	131.4	1056	1850	57.1	obtained by use of the
2		9.0	6.6	133.2	830	1358	61.1	'modified' proctor method.
3		6.9	7.5	133.3	807	1503	53.7	DRY DEN. 24hr 7day %gain
4	04/23/85	9.0	7.0	132.1	720	1463	49.8	134.0 1103 1918 57.5
5		9.0	7.6	134.9	787	1563	50.4	139.0 650 1311 49.6
6		9.1	6.5	129.8	579	1273	45.5	134.0 961 1968 48.8
7		8.0	6.6	126.9	432	927	46.6	132.5 758 1410 53.8
8	04/24/85	8.0	7.5	130.0	707	1190	59.4	
9		7.9	7.5	127.4	662	1159	57.1	
10		8.1	7.6	134.2	666	1046	63.7	
11	04/25/85	8.0	7.1	131.2	919	1433	63.7	135.3 910 1439 63.2
12		8.0	7.2	135.6	944	1666	56.7	135.6 746 1573 47.4
13		7.9	7.6	135.1	856	1483	57.7	137.2 835 1589 52.5
14	05/15/85	9.1	6.1	131.9	498	1002	49.7	
15		9.0	6.6	127.2	939	1621	57.9	
16		9.0	7.5	124.8	678	1469	46.3	
17		9.0	6.1	132.4	900	1872	48.1	
18	05/16/85	9.0	[illegible]	132.8	915	1730	52.9	
19		9.0	7.5	130.1	752	1546	48.6	
20		9.0	6.3	130.4	805	1694	47.5	
21		9.0	10.0	130.6	744	1806	41.2	
22	05/17/85	9.0	6.5	131.2	1054	2280	46.2	
23		9.0	6.8	129.7	759	1515	50.1	
24		9.0	[illegible]	131.6	994	2269	43.8	
25		9.0	7.6	131.9	742	1948	38.1	
26	05/18/85	8.7	6.5	133.1	774	1560	49.6	
27		8.5	6.0	130.7	905	1564	57.9	
28	05/20/85	8.2	[illegible]	126.7	926	1520	60.9	
29		8.0	[illegible]	122.2	497	1021	48.7	
30		8.0	[illegible]	[illegible]	462	1051	44.0	
31		8.0	[illegible]	127.8	601	1211	49.6	
32	05/21/85	8.5	[illegible]	132.2	491	993	49.4	
33		8.5	[illegible]	129.0	374	760	49.2	
34		8.0	[illegible]	132.4	547	1210	45.2	
35		8.0	[illegible]	128.7	628	1391	45.1	
36	05/22/85	8.5	[illegible]	131.1	528	1182	44.7	
37		8.5	[illegible]	132.2	693	1312	48.2	
38		9.0	6.1	132.7	940	2110	44.9	
39		9.0	6.6	133.4	477	1257	37.9	
40	05/23/85	9.0		[illegible]	[illegible]	1850	[illegible]	
41		9.0	[illegible]	[illegible]	900	1830	[illegible]	
42		9.0	[illegible]	[illegible]	[illegible]	2163	47.3	
43		9.0	6.0	132.4	1151	2669	43.2	
44	05/24/85	9.0	[illegible]	130.9	1405	2492	57.0	
45		9.0	[illegible]	134.3	990	1723	57.9	

46		8.5	6.9	132.5	779	1463	53.2
47		8.5	8.2	133.2	1009	1835	55.0
48	05/28/85	8.5	7.0	133.1	1142	1620	70.5
49		8.5	8.5	136.1	736	1430	51.5
50	05/29/85	8.5	7.2	132.2	972	2066	47.0
51		8.5	9.7	131.0	664	1546	42.9
52		8.5	9.1	134.4	787	1375	57.2
53		8.5	9.1	133.1	759	1425	53.3
54	05/30/85	8.5	7.8	134.1	930	1643	56.6
55		8.5	9.2	133.5	676	1223	55.3
56		8.5	8.5	135.2	615	1076	57.2
57		8.5	7.9	133.2	764	1315	58.1
58	05/31/85	8.3	8.5	134.4	748	1541	48.5
59		8.0	8.1	133.1	744	1456	51.1
60		8.0	6.9	130.1	433	1018	42.5
61		8.0	8.4	131.8	390	997	39.1
62	06/3/85	8.0	8.0	135.2	839	1526	55.0
63		8.0	8.2	136.4	836	1708	48.9
64		8.0	6.0	134.4	808	1322	61.1
65		8.0	7.6	135.9	577	1062	54.3
66	06/4/85	8.0	7.1	135.9	879	1763	49.9
67		8.0	7.6	134.4	757	1416	53.5
68		8.0	6.7	135.2	879	1543	57.0
69		8.0	8.0	135.5	641	1213	52.8
70	06/5/85	8.0	6.8	136.0	885	1546	57.2
71		8.0	8.3	136.2	531	1098	48.4
72		8.0	7.8	135.4	823	1513	54.4
73		8.0	7.5	133.5	816	1517	53.8
74	06/06/85	7.9	6.9	134.5	860	1403	61.3
75		8.0	8.1	135.9	821	1448	56.7
76		8.0	6.1	134.5	698	1275	54.7
77		8.0	6.6	135.7	778	1729	45.0
AVERAGE	--------	8.5	8.1	132.5	774	1501	51.7
STD DEV.	--------	0.4	0.8	2.7	192	361	6.4

QUANTITIES PER	PLANS	S/C =	31,500	CU.YD.	CEMENT =	5,550	TONS
QUANTITIES AS	BUILT	S/C =	33,370	CU.YD.	CEMENT =	5,668	TONS

********* SANTA CRUZ RIVER AT VALENCIA RD. SOIL CEMENT BANK PROTECTION ********* 4BBRGB *********

SAMPLE NUMBER	DATE SAMPLED	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS psi	COMP STR 7DAY psi	STR. GAIN (%)	REMARKS
MIX DESIGN	05/13/85	7.0	9.2	127.1	416	902	46.1	START @ 10.0% CEMENT
"	"	9.0	9.2	127.5	496	1456	34.1	
"	"	11.0	9.2	128.2	845	1914	44.1	
1	05/14/85	12.0	8.2	128.0	611	1189	51.4	
2	"	10.1	8.2	128.1	689	1633	42.2	
3	"	9.5	8.7	127.4	677	1655	40.9	
4	05/15/85	10.2	7.7	130.2	1071	2039	52.5	
5	"	9.5	6.5	128.9	824	1730	47.6	
6	"	9.5	8.5	129.4	686	1416	48.4	
7	05/16/85	9.5	7.7	130.0	1001	3001	35.1	
8	"	9.5	7.1	128.1	891	1564	53.1	
9	05/17/85	9.5	8.6	123.9	726	1496	48.5	
10	"	9.5	9.5	124.4	852	1712	49.8	
11	"	8.5	8.6	119.2	505	1101	45.9	
12 *	05/20/85	8.8	8.3	122.2	957	1872	51.1	
14	"	8.6	7.7	123.4	693	1635	42.4	
15	05/21/85	8.6	6.9	127.1	771	1166	66.1	
16	"	8.6	7.5	125.3	696	1127	61.8	
17	"	8.5	7.7	126.4	468	995	47.0	
18	05/22/85	8.7	8.8	124.9	852	1461	58.3	
19	05/24/85	8.7	7.4	124.2	784	1588	49.4	11 DAY.
20	"	8.5	7.4	120.6	742	1463	50.7	
21	"	8.4	7.6	120.6	829	1690	49.1	
22	05/28/85	8.0	6.8	121.9	994	1944	51.1	
23	"	8.0	8.3	122.1	1072	1943	55.2	
24	"	8.1	7.9	123.2	553	1187	46.6	
25	05/29/85	8.0	7.2	133.6	748	1083	69.1	
26	"	7.9	6.8	124.0	774	1162	66.6	
27	"	7.6	7.6	123.5	779	1307	59.6	
28	"	8.0	9.2	126.6	623	1249	49.9	
29	05/30/85	8.1	8.7	124.4	642	1122	57.2	
30	"	8.1	7.5	127.7	653	1262	51.7	
31	"	7.6	8.7	128.4	795	1485	53.5	
32	"	8.0	8.6	127.6	832	1452	57.3	
33	05/31/85	7.5	8.4	128.9	738	1434	51.5	
34	"	7.5	7.5	128.8	388	1032	37.6	
35	"	7.5	6.9	128.0	611	960	62.3	
36	06/03/85	7.6	7.6	125.0	541	977	55.4	
37	"	7.6	6.4	124.6	467	766	61.0	
38 *	06/04/85	7.6	7.6	131.9	611	1496	40.8	
40	"	7.6	7.8	125.1	527	814	64.7	
41	06/05/85	7.1	7.5	126.8	540	1207	44.7	
42	"	7.1	7.8	125.6	524	954	54.9	
43	"	7.4	7.6	132.6	731	832	87.9	13A & 13B HAVE SILT BALL. BROKE
13	05/20/85	6.6	7.2	126.1	674	686	98.5	13B @ 24 HRS. + 4 HRS. SOAKING
39	06/04/85	7.6	6.6	127.4	714	993	71.9	
AVERAGE	--------	8.4	7.8	126.2	718	1412	52.1	
STD DEV		1.0	0.7	3.2	164	419	[illegible]	

QUANTITIES PER PLANS	S/C =	21,645	CU.YD.	CEMENT =	4,213	TONS
QUANTITIES AS BUILT	S/C =		CU.YD.	CEMENT =		TONS

********* INA RD. BRIDGE SOIL CEMENT BANK PROTECTION ********* 46BRGE *********

SAMPLE NUMBER	DATE SAMPLED	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS psi	COMP STR 7DAY psi	STR. GAIN (%)	REMARKS
MIX DESIGN	06/10/85	5.0	9.8	120.2	215	400	53.7	
"	"	7.0	9.8	122.9	310	683	45.4	
"	"	9.0	9.8	124.5	518	1076	48.1	
"	06/11/85	11.0	9.8	124.5	583	1579	36.9	RECOMENDED CEMENT CONTENT
"	"	13.0	7.8	125.2	900	1808	49.8	TO START = 11.0%
1	06/13/85	10.1	8.4	124.6	998	1876	53.2	
2	"	11.0	8.4	123.4	1147	2057	55.8	
3	"	11.1	9.8	125.7	871	1806	48.2	
4	"	11.0	9.1	125.9	770	1860	41.4	
5	06/14/85	11.0	9.0	125.8	875	1725	50.7	
6	"	10.0	9.1	124.5	778	1386	56.1	
7	"	10.0	9.2	125.9	744	1474	50.5	
8	"	10.0	10.0	126.0	637	1387	45.9	
10	06/17/85	10.0	7.4	126.1	694	1346	51.5	
11	"	9.9	9.8	127.0	571	1065	53.6	
12	"	10.0	7.7	124.8	668	1163	57.4	
13	"	10.0	9.1	125.2	597	1097	54.4	
14	06/18/85	10.0	8.9	125.5	657	1160	56.6	
15	"	10.0	12.5	119.7	309	582	53.1	CHANGE IN MATERIAL, LIFT RE- MOVED
16	06/19/85	10.0	8.1	133.0	1010	1982	51.0	
17	"	10.0	8.2	130.8	1316	2195	60.0	
18	"	10.0	9.1	127.2	960	1835	52.3	
19	"	10.0	9.4	130.7	1706	2768	61.6	
20	"	10.0	8.4	126.7	770	1225	62.9	S/C COMPLETE
9	06/15/85	10.0	9.1	125.5	178	1124	15.8	@ 5 HOURS
AVERAGE	--------	10.2	9.3	126.2	846	1576	53.5	
STD DEV.	--------	0.4	1.0	2.9	308	505	5.2	

********* SPEEDWAY/PANTANO SOIL CEMENT BANK PROTECTION ********* 85PBR *********

SAMPLE NUMBER	DATE SAMPLED	CEM CONT (%)	MOIS CONT (%)	DRY DENS (PCF)	COMP STR 24HRS psi	COMP STR 7DAY psi	STR. GAIN (%)	REMARKS
MIX DESIGN	05/27/85	6.0	9.0	134.0	533	864	61.7	PLUGS WERE PUMPING AT 6% AND 8% CEMENT.
"	"	8.0	9.0	135.6	696	1090	63.9	
"	"	10.0	9.0	135.8	1002	1897	52.8	
1	06/17/85	9.0	7.8	130.3	1194	1729	65.6	
2	"	7.5	8.8	129.7	1114	1997	55.8	
3	"	8.0	8.4	128.9	1026	1850	55.5	
4	"	7.7	9.3	130.8	1082	2016	53.7	
5	06/18/85	7.0	7.2	129.4	1219	1842	66.2	
6	"	8.0	8.0	129.9	1056	1729	61.1	
7	"	8.0	8.4	130.5	926	1756	52.7	
8	"	7.9	6.5	130.7	838	1423	58.9	
9	06/19/85	6.1	7.1	131.2	853	1516	56.3	
10	"	7.9	6.0	128.0	756	1369	55.4	
11	"	6.0	8.7	129.5	843	1489	56.6	
12	"	7.3	7.1	129.2	585	911	64.2	
13	06/20/85	7.7	6.2	130.5	910	1334	68.2	
14	"	7.7	6.2	131.6	864	1526	56.6	
15	"	7.0	6.7	131.4	93[illegible]	1496	62.2	
16	"	7.0	9.0	132.5	834	1450	57.5	
17	06/21/85	7.9	7.2	127.3	1066	1719	62.0	
18	"	7.9	7.8	129.8	943	1531	61.6	
19	"	7.9	6.5	130.8	657	1239	53.0	
20	"	7.9	9.0	133.0	845	1678	50.4	
21	06/22/85	7.4	7.2	129.6	911	1571	58.0	
22	"	7.3	8.0	130.5	1031	1690	61.0	
23	"	7.3	9.2	132.4	979	1838	53.9	
24	"	7.1	8.0	131.2	1069	1715	62.3	
25	06/24/85	7.8	7.7	131.2	955	1631	58.6	
26	"	7.1	8.0	130.9	1025	1626	62.9	
27	"	7.0	6.2	131.4	946	1679	56.3	
28	"	7.0	9.0	133.0	816	1559	52.3	
29	06/25/85	7.2	6.8	132.8	756	1391	54.5	
30	"	7.2	8.2	129.6	835	1594	54.4	
31	"	7.1	7.1	125.6	635	1261	50.4	
32	"	7.0	8.0	125.6	612	1261	48.5	
33	06/26/85	7.0	7.1	129.2	844	1344	63.2	
34	"	6.9	6.7	123.1	940	1549	60.7	
35	"	7.0	7.7	129.1	687	1069	64.3	
36	"	7.0	9.2	131.2	843	1578	53.4	
37	06/27/85	6.8	7.6	134.5	1102	1936	56.9	
38	"	7.2	9.0	131.9	1575	2797	56.3	
39	"	7.7	9.0	132.4	1766	3172	55.7	
40	"	7.0	7.1	128.3	1075	1795	59.9	
41	06/28/85	7.6	7.1	133.9	1155	1665	69.4	
42	"	7.0	7.2	130.6	931	1509	61.7	
43	"	7.1	9.0	135.3	814	1545	52.7	
44	"	7.2	6.2	132.5	866	1514	58.5	
45	07/01/85	6.5	7.1	131.[illegible]	846	1295	65.1	
46	"	8.0	9.0	134.5	1842	3079	59.8	
47	"	6.6	6.0	133.9	785	1373	57.2	
48	"	6.5	8.0	134.0	1424	2502	56.9	
49	07/02/85	7.0	7.2	134.4	1192	2284	52.2	

50	"	7.0	7.4	131.6	1445	2031	71.1	
51	"	7.0	7.4	134.2	1445	2197	65.8	
52	"	7.0	7.6	129.0	900	1464	60.6	
53	07/08/85	7.0	7.4	130.9	886	1465	60.5	
54	"	6.9	4.6	132.5	932	1539	60.6	
55	"	7.0	7.1	130.2	925	1664	55.6	
56	07/09/85	7.1	7.5	132.1	1048	2042	51.9	
57	"	7.0	7.3	132.0	1115	2127	52.4	
AVERAGE		7.8	7.9	131.1	991	1700	58.4	
STD DIV.		0.5	0.9	3.9	254	428	5.1	

QUANTITIES PER	PLANS	S/C =	24,800	CU.YD.	CEMENT =	4,855	TONS	
QUANTITIES AS	BUILT	S/C =		CU.YD.	CEMENT =		TONS	

SECTION 4 - TECHNICAL ANALYSIS OF SCOUR AND EROSIVE FORCES

4.1 General

The complex requirements of water resource and land use planning have stimulated the development of watershed and river system models for predicting the response of fluvial systems to natural and man-induced changes. These models, both physical-process oriented and conceptual, are generally intended to estimate physical qualities and parameter quantities which describe a fluvial system response to precipitation, runoff, changes in system morphology, and transport of sediment and pollutants.

Depending on the data and resources available for analysis, the problem of assessing the response of a watershed and river system to natural inputs and man's activities can be approached in three phases or levels of analysis:

Level I	A qualitative analysis based on general geomorphic parameters.
Level II	A quantitative analysis based on specific geomorphic concepts and basic engineering relationships.
Level III	A physical or mathematical model of watershed and channel processes in the reach or system of concern.

As listed, each level requires an increasing commitment of resources; but, individually, each level of analysis yields meaningful results which range from a purely qualitative assessment of trends to the numerical results and predictive capability of physical process computer modeling. When applied sequentially, this multilevel approach constitutes a powerful methodology for evaluation of short- and long-range response of watershed and river systems.

The fluvial system is a prime example of a highly nonlinear, complex system that includes hydrologic, hydraulic, geologic, soil, climatologic, biologic, and man's influences as subsystems or components. Each is governed by physical processes that often affect other components.

The two primary inputs to the fluvial system are climatic inputs and man's activities. The principal climatic input is precipitation. Man's governing activities for the watershed and river analyses cover the full range of urbanization, resource management, and encroachment into the natural systems.

4.1.1 Qualitative Geomorphic Analysis

Engineers and others concerned with analysis of fluvial systems are mainly interested in hydraulic geometry and other fluvial geomorphic parameters related to watersheds and channels. In addition, investigation of river response to climatic changes and catastrophic flood events provides information and insight into long-term river adjustment to hydrologic conditions.

Aerial photographs provide information valuable to the qualitative analysis of river hydraulics and channel geometry. The availability of aerial photographs taken over a span of many years provides documentation of historical trends and changes in the river. The accuracy of measurements made from aerial photographs is largely dependent on the quality and scale of the photos. Properly applied, photographic interpretation can provide an abundance of accurate and useful information. Evidence of bank cutting, shifting of thalweg, lateral migration meander tendencies, vegetation changes, and sediment deposition can be documented by studying photographs for different years. Changes in stream widths can also be documented by measuring the minimum, maximum, and average stream width in each reach.

Quantitative geomorphic analysis of actual gradation changes (aggradation/degradation) is based on the concept of equilibrium. The qualitative approach assumes that rivers strive, in the long run, to achieve a balance between the products of water flow, channel slope, and the products of sediment discharge and size.

4.1.2 Quantitative Geomorphic Analysis

Quantitative analysis provides only a general understanding of the direction of change. Geomorphic principles can be applied to quantitatively evaluate aggradation, degradation and lateral migration; however, this requires collection and analysis of data for at least several years.

Analysis of gauging stations stage trends is generally easily done and is useful information on long term trends. The U.S.G.S. and U.S. Army Corps of Engineers, especially, have previously performed the analysis, providing excellent records for which some data may extend for decades.

4.1.3 Hydraulic and Geomorphic Relations

Basic engineering relationships together with qualitative and quantitative geomorphic concepts provide solutions to specific problems such as those related to surface profiles and sediment transport rates. Geomorphic principles are useful for establishing a basic understanding of gradation problems prior to an analysis using engineering relationships.

Calculation of water surface profiles is an integral part of sediment transport analysis for gradation changes. After qualitatively classifying the type of flow, either subscribed or supercritical, computer programs for the computation of the water surface elevations and profiles can be used. Most computer programs require a qualitative analysis of general characteristics of backwater curves in order to determine whether the analysis proceeds upstream or downstream. For large and complex situations involving many bridges, culverts, and long reaches of river, it is often necessary to use a computer program, such as the U.S. Army Corps of Engineers HEC-2, to compute a water surface profile. This generally requires a considerable effort to create large input data files describing the river conditions. The HEC-2 program includes a provision for modifying the resistance as the state changes. This is an important consideration since the resistance value used in calculation has a significant affect on the results; however, HEC-2 and similar models are for rigid-boundary systems. For movable bed conditions, computer programs that route water and sediment must be used.

Knowledge of sediment transport conditions is essential to erosion and sedimentation analyses. Evaluation of local scour around a bridge pier or long-term gradation trends requires accurate estimates of sediment transport rates.

Degradation, aggradation, and movement of pollutants are closely related to water and sediment movement. Understanding the physical processes related to water and sediment routing is of fundamental importance for effective analysis of watershed and river response. These considerations are continuity equations of water and sediments, flow momentum equations, flow energy equations, and other supplementary equations such as flow resistance relations, channel geometry equations, and sediment supply equations. These relationships provide the basis for quantitative analysis using physical process computer modeling.

4.1.4 Mathematical Modeling of River Systems

A mathematical model is simply a quantitative expression of a process or phenomenon that is being analyzed. Mathematical simulation of the governing physical process provides a direct estimate of the time depending response of a fluvial system.

During the last decade, significant time and effort have been devoted to the development of a numerical model of flow and sediment transport in moveable bed channels. Only six models have been developed nationwide, and only three (HEC2SR by Simons, Li & Associates, HEC-6 by U.S. Army Corps of Engineers, FLUVIAL-11 by Dr. Howard H. Chang), have been successfully applied nationwide. It is interesting to note that extensive testing and calibration of HEC2SR and FLUVIAL-11 have been applied to numerous watercourses within Pima County. Through the extensive use of the models in Pima County, numerous revisions, modifications, and improvements to the models have been made.

All models utilize similar theoretical principles with the exception of FLUVIAL-11 which incorporates width changes. This is an important advancement over the other sediment transport models, and follows the general recommendations of the National Academy of Science. The Pima County engineering community has extensively utilized HEC2SR and FLUVIAL-11 since their inception. Further advancement to the FLUVIAL-11 model encouraged

Pima County to purchase this particular model. The following is a brief description. While the alluvial stream bed is subject to scour and fill induced by the imbalance in longitudinal (streamwise) sediment discharge, such channel bed development may also be caused by transverse sediment movement due to channel curvature. Despite bank protection, the channel still has certain freedom in width adjustment within the constraints, particularly in the width between rigid banks at different locations. Therefore, scour and fill, due to longitudinal sediment imbalance and curvature effects as well as width changes, need to be considered in the simulation study. The latest version of FLUVIAL-11, for water and sediment routing through curved channels, contains the necessary features for river channel changes during a flood. Briefly, this model for a given flood hydrograph simulates time and spatial variations in flood level, sediment transport, and bed topography. In the prediction of river channel changes, scour and fill are tied in with width variation and the effect of secondary currents under the changing channel curvature. In the model, scour and fill are computed on the basis of longitudinal imbalance in sediment discharge. The effects of secondary currents through a curved reach consist of moving sediment away from the concave bank until the transverse bed slopes balance such sediment movement. At the same time, the variation in channel width is simulated such that the flow moves in the direction of equal power expenditure, i.e. equal energy gradient, subject to the physical constraint of rigid banks. If the energy gradient is approximated by the water surface slope, then the equal energy gradient is equivalent to the straight water surface profile along the channel. In response to any design or control scheme, the river channel evolves in such a way that uniformity in sediment discharge and straight water surface profile are approached subject to the given constraints.

4.2 Design Criteria and Requirements

Natural rivers in and near high urbanized areas need to be stabilized in order to prevent channel bank erosion and lateral migration. A common economical practice in Pima County and elsewhere in the west is to protect channel banks while maintaining a natural alluvial channel bed. Soil-cement bank stabilization is popular due to its stability, cost effectiveness, and natural soil appearance.

4.2.1 Existing Condition Hydraulic and Geomorphic Analysis

Initial analysis of any proposed project involves evaluation of existing hydrologic, hydraulic, and geomorphic conditions to provide the required understanding of the dominant physical processes affecting the system. This should include the water surface profile determination in order to properly delineate the floodplain/floodway boundaries as well as the qualitative and quantitative geomorphic evaluation of the river system. This information will serve as the initial input for the mathematical modeling process.

The results obtained from the above mentioned analysis is essential and of great importance prior to initiating any analysis of proposed structural improvements such as channelization and soil-cement bank protection. Man's activities can have both short- and long-term effects on the fluvial system, and even short-term projects can have prolonged impacts. An integrated analysis approach supported by an interactive data storage and retrieval system is required to predict the response on a basin or sub-basin level to multiple development alternatives.

4.2.2 Design Condition Hydraulic and Geomorphic Analysis

The type of channel improvements and devices used for training and bank stabilization depend on river size (with regard to width, depth, and discharge), type of rivers (meandering, braided, or straight), sediment transport in terms of concentration and size distribution, length of river to be protected, availability of materials, environmental considerations, aesthetics, legal aspects, river use with regard to recreational, agricultural, municipal and industrial purposes and, perhaps, other factors.

Alluvial channel systems are very dynamic and experience significant changes with respect to width, depth, alignment, and stability with time, particularly with floods of long duration.

Design of bank protection and channelization projects must take into consideration the local scour, general scour, and gradation changes in the reach. Adequate freeboard considering the influence of vegetative debris, and water and sand wave movement must also be included in the design. When large scale improvements are planned for a river system, the effects of the changes should be investigated for the entire system. Total scour used for design is the sum of the general scour, local scour, and sand wave movement at the site. Total scour should also be considered as long-term changes in the sediment supply which could redefine equilibrium conditions. Freeboard analysis is the sum of aggradation, sand wave movements, water waves, superelevation, and increases in depth from debris blockage added to a maximum water surface elevation of the design storm. Maximum water surface elevation for freeboard analysis is based on a conservative selection of roughness (n values in the upper range for the channel condition).

Determination of velocity is based on the lower range of roughness for conservative scour computations.

After the proposed improvement has been included within the model, the analysis should insure that the improvements do not adversely affect the river system. Consequently, the water surface profile for the improved condition must be performed in order to insure that federal (FEMA) and local regulations are not violated. The results of the hydraulic, geomorphic, and mathematical modeling generate other design parameters such as total scour, degradation, and aggradation which will be discussed in the following subsections.

4.2.3 Scour and Deposition Analysis

The dynamic nature of watershed and channel systems requires that local problems and their solutions be considered in terms of the entire system. Natural and man-induced changes in a channel frequently initiate responses that can be propagated for long distances both upstream and downstream.

Qualitative analysis methods provide an answer for the long-term river bed changes. Important spatial information includes the plan and profile of the river channel, classification of the channel form (meandering, braided, straight), and the full width and depth of the bank along the river. This information can be obtained easily from aerial photos. The general response of the river system can be assessed qualitatively by studying the profile, plan view, and sediment size distribution variations along the river system.

Equilibrium Slope Analysis

Equilibrium concepts can also be applied to a quantitative analysis of gradation changes in a river system. These quantitative equilibrium relationships are fairly easy to formulate and provide very powerful insights into the behavior of the river system. The relationships are developed from the application of basic engineering concepts.

The equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply. Under this condition, the channel neither aggrades nor degrades.

The calculation of equilibrium slope is accomplished by trial and error. The slope of the study reach is varied until the resulting sediment transport capacity equals the incoming supply. The equilibrium slope analysis is essential to determine the need for grade control structures.

Various sediment transport equations are in use by different models, and will not be the subject of a detailed discussion within this report.

Short Term Channel Bed Changes

Dynamic mathematical modeling of water and sediment routing such as HEC-2, FLUVIAL-11 provides the information for short term channel bed changes during the passage of the design flow (i.e. 100-year event).

Local Scour

Local scour is observed whenever an abrupt change in the direction of flow occurs, such as at bridge piers or embankments. For example, local scour at bridge piers is a result of vortex systems developed at the pier. Local scour occurs when the capacity of flow to remove or transport the bed materials is greater than the rate at which replacement materials are supplied. This suggests a basis for categorizing local scour by considering the sediment-transport condition of the scour area.

During a flood event, the equilibrium condition may never become established. Scour occurs during the rising limb of the hydrograph, and could potentially endanger hydraulic structures such as bridges. After the peak has passed (during the falling limb), the scour hole refills as sediments drop out with the lower flows; therefore, the critical time for structural stability is near the peak flow. Two formulas have been found to be particularly successful based on previous experience. These two involve a relationship developed by Shen and Neill. The relationship for square-nosed piers by Neill (1964) is

$$ds/d_1 = 2.2 \left(\frac{b}{d_1}\right)^{0.65} Fr_1^{0.43}$$

and for a group of circular cylinders

$$ds/d_1 = 2.0 \left(\frac{b}{d_1}\right)^{0.65} Fr_1^{0.43}$$

where ds is the depth of the scour hole, b is the pier width (normal to the flow direction), d_1 is the upstream depth of flow, and Fr is the upstream Froude number.

The equations by Shen, et al. (1966, 1969), are:

$$d_{sc} = 0.00073R^{0.619}$$

and

$$\frac{d_{sc}}{b} = 11.0\ Fr^{2} \quad Fr\ 0.2$$

$$\frac{d_{sc}}{b} = 3.4\ Fr^{0.67}\ Fr\ 0.2$$

Respectively, where d_{sc} = equilibrium depth of scour measured from mean bed elevation, R = pier Reynolds number = V b/v, V = mean velocity of the undisturbed flow, b = width of pier projected on a plane normal to the undisturbed flow, v = kinematic viscosity, and Fr (pier Froude number) is $V/\sqrt{gb}$.

Another important local scour zone at a bridge structure occurs at the abutments. Detailed studies of scour around embankments have been made mostly in laboratories. There are very few field studies of abutment scour. According to the studies of Liu, et al. (1961), the equilibrium local scour depth may be determined by:

$$\frac{s}{d_1} = 1.1\ \left(\frac{a}{d_1}\right)^{0.4} Fr_1^{0.33}$$

in which a is the embankment length (measured normal to the wall of a flume), d_1 is upstream depth, and Fr_1 is the normal upstream Froude number. If the embankment terminates at a vertical wall, such as a wing abutment extended down into the channel bed, then the depth of scour hole almost doubles (Liu et al., 1971, and Gill, 1972), that is:

$$\frac{s}{d_1} = 2.15\ \left(\frac{a}{d_1}\right)^{0.4} Fr_1^{0.33}$$

The lateral extent of the scour hole is nearly always determinable from the depth of scour and the natural angles of repose of the bed material. This equation is also useful for estimating local scour at bank protection, spur dikes, and jetties.

Antidune (Sand Wave Movement)

It is necessary to estimate the height of bed forms moving through the channel for natural or man-made channel segments with upper regime flow, particularly where freeboard requirements are critical. In addition to this consideration, the affect of antidune formation can be seen in the elevation of the bed surface. Consequently, the formation of antidune conditions will not only increase the mean water surface elevation by one-half the wave height, it will also decrease the mean bed elevation by one-half the wave height. When antidunes reach a height of approximately 0.14 times the wavelength, the wave becomes unstable and breaks. Studies on antidune flow (Kennedy 1961) conclude that the minimum wavelength is given by:

$$L - \frac{2 \quad V^2}{g}$$

where V is mean flow velocity and g is gravitational acceleration; thus, the maximum wave height before breaking will be:

$$h = 2 \quad (0.14)\ V^2/g$$

Antidune height cannot exceed the mean flow depth.

It is evident from the previous equation that the presence of antidunes should be considered when conducting an aggradation/degradation analysis.

As mentioned previously, proper consideration of scour at a study site requires a determination of total scour for a safe design analysis. A design which gives adequate support for the structure when the channel bed is at its initial elevation may be inadequate after scour occurs in the channel bed. The physical processes that must be considered are general and local scour as well as the passage of sand waves. The total scour is the sum of all these, and must be subtracted from the initial design elevation to establish the design depth for the structure.

Conversely, total aggradation is the sum of general aggradation and the passage of sand waves. Aggradation will increase the channel bed elevation; hence, the water surface elevation at the site during a flood event. When aggradation is combined with debris blockage and hydraulic effects, such as superelevation of the flow, a structure can be severely threatened. Adequate freeboard must be provided to assure the safety of the structure; therefore, the design height of a structure must consider a combination of the flow depth, superelevation, total aggradation, water waves, and vegetative debris blockage.

The previous paragraphs discuss various hydraulic and geomorphic parameters that must be analyzed prior to design of any soil-cement projects. Local and general scour, and short and long-term degradation are all factors that must be considered in the toe-down depth of soil-cement bank protection. Conversely, depth of flow, aggradation of channel bed, wave height, debris, and superelevation factors affect the design height. Varying toe-down depths and design heights may occur throughout the project length based on the results of the previous calculations.

The final results are multiplied by a safety factor to consider hydrologic and hydraulic uncertainties. Results of the geomorphic and equilibrium slope analyses determine channel bed stability which determine the need and spatial distribution for grade control structures.

4.2.4 Stability Analysis for Soil-Cement Bank Protection and Grade Control Structures

Once hydraulic and geotechnical conditions are obtained, stability analysis should be performed to assure stability of the proposed structure. Stability of the structure is determined by computing the overturning moment caused by the active thrust behind the bank/grade control structure, the hydrostatic load produced by water behind the bank/grade control structure, and a comparison of the results to the resisting moment resulting from the weight of the structure and passive soil resistance. The active/passive soil pressure can be obtained by applying the Rankine or Coulomb equation. While the factor of safety against overturning is satisfactory, the resistance to sliding should be checked. The final check should assure the design that the soil bearing pressure is less than allowable.

Pima County's experience indicates that soil-cement bank protection, eight (8) feet thick with side slopes of 1:1, is perfectly stable and satisfies all conditions listed above, with the exception of collapsible soil. The soil-cement grade control structure needs thorough structural analysis for each individual case. It should be noted that hydraulic evaluation of the grade control structures is based on the free overfall equation (conservative assumption), and is the basis of ongoing research described in Section 6.

SECTION 5 CONSTRUCTION

GENERAL

Pima County soil-cement experience is the result of installing approximately 750,000 cubic yards of bank protection utilizing soil-cement material exclusively.

Construction techniques and specifications continuously undergo modifications as needed based on every new experience that is realized. The major changes, additions, or modifications that have been made are outlined as follows:

a. Aggregate quality, i.e. grading plasticity index (P.I.), and stockpiling requirements.

b. Utilization of stationary plant produced soil cement controlled by direct digital readout belt scales.

c. Establishment of time limits for dumping, spreading, and compacting soil-cement material.

d. Protection from contamination during construction and curing of the soil cement.

e. Finishing, trimming, and shaping of soil cement.

f. Mix design and construction quality control methodology, including utilization of 24-hour compressive strength test results in predicting the 7-day soaked compressive strength for daily production control.

g. Establishing a daily production record which includes information on the time and location represented, percent cement utilized, compaction, moisture content, dry density, and 24-hour and 7-day compressive strengths.

SHOULD NEW WORDING REGARDING FLY ASH BE ADDED?

5.1 Construction Specifications

DESCRIPTION

The work shall consist of the construction of soil-cement bank protection as required by the construction plans, including trench excavation, backfill, and dewatering.

5.1.1 Materials and Equipment

Portland Cement

Portland cement shall comply with the latest specifications as approved by the Engineer, for portland cement (ASTM C150, CSA A-5, or AASHTO M85) Type II.

Water

Water shall be clear and free from injurious amounts of oil, acid, alkali, organic matter or other deleterious substances.

Aggregate

The soil used in the soil-cement mix shall not contain any material retained on a one and one-half inch (1-1/2") sieve, nor any deleterious material. Soil for soil-cement lining shall be obtained from the required excavations, or from other borrow areas approved by the Engineer, and stockpiled on the jobsite as specified herein. The actual soil to be used shall be analyzed by laboratory tests in order to determine the job mix as set forth herein. The distribution and gradation of materials in the soil-cement lining shall not result in lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material.

Aggregate Stockpile

The soil aggregate stockpile shall be constructed on level, firm ground free of brush, trees, stumps, roots, rubbish, debris, and other objectionable or deleterious material, and shall be located so as to provide a distance of not less than fifty feet (50") from the outside bottom edge of the conical stockpile, built up under the processing plant conveyor, or any other existing stockpile. The stockpile shall be constructed in layers; each layer not exceeding two (2) feet in thickness. Ramps formed for stockpile construction shall be of the same material as that being stockpiled, and will be considered a part of the stockpile. Before steepening a ramp, any contaminated surface material shall be removed.

Stockpiled material should be thoroughly mixed throughout its depth, width, and length before utilization. The material should be homogeneous and uniform in color, gradation and moisture throughout.

Stockpile sampling will be done by the Pima County's Material Lab after the required amount of soil aggregate for the entire soil-cement job has been excavated and stockpiled. After the stockpile has been sampled and approved, no material will be added to it without approval of the Engineer.

Stockpile(s) shall be completed and approved at least eight (8) days prior to start of soil-cement production. Mix design shall then be performed by Pima County Materials Lab to determine job mix proportions.

Equipment

The soil-cement bank protection may be constructed with any combination of machines and/or equipment, except as noted herein, that will produce a completed soil-cement lining meeting the requirements for soil pulverization, cement, and water application, mixing, transporting, placing, compacting, finishing, and curing as provided in these specifications.

5.1.2 Mix Design

Methodology

The design requirements for the soil-cement bank protection shall be such that it has a compressive strength of 750 psi at the end of seven (7) days, plus two percent (2%) additional cement added for erosion resistance. The minimum acceptance strength shall be that developed as a result of adding two percent (2%) cement to the base amount determined. For example, if the mix design shows that six percent (6%) cement is required to achieve 750 psi in seven (7) days, the total cement content shall be 6.0% + 2.0% = 8.0%. Hence, the governing strength shall be that strength which is acquired by the mix design at eight percent (8%). A 24-hour test will be run to monitor the mix design on a daily basis. Experience has shown that 24-hour compressive strength results for moist cured samples are approximately 50 to 60 percent of the seven (7) day strength (moist cured for six (6) days and soaked in water for 24 hours). In the example cited herein, once the design strength mix of 6.0% + 2.0% or 8.0% cement is determined, a 24-hour test will be run, using the mix to obtain a 24-hour compressive strength, which will be used to monitor the daily output of the central plant. Seven (7) day samples will also be taken for final acceptance. The amount of cement thus determined by laboratory testing shall continue to be monitored throughout the life of the project with modification as required to meet existing field conditions.

Cement and Additives

For bidding-purposes only, the estimated mix design for a project shall be as shown in the following examples:

Base Cement Content required for 750 psi @ 7 days	8%
Addition for Durability and Erosion	2%
TOTAL CEMENT REQUIRED *	10%

The percent of cement to be used in the mix shall be calculated to be the weight of cement divided by the total weight of the dry compacted soil cement. The actual mix design used on a project shall be determined by laboratory tests on material stockpiled.

*Fly ash may be used at the approval of the Engineer. A maximum of fifteen (15) percent of the total weight of cement may be replaced with fly ash. A minimum of 1.2 pounds of fly ash shall replace each 1.0 pound of portland cement removed. Fly ash shall conform to the requirements of ASTM C 618 Class F, except that the Pozzuolanic activity index with lime shall be reduced to a minimum of 650 pounds per square inch at seven (7) days. The Blaine fineness shall have an average value of at least 2,800 with a minimum value of 2,600 for any one sample. The average value will be determined on the last five (5) consecutive samples. The loss on ignition shall not exceed 3.0 percent. An additional scale shall be required for the fly ash and shall conform to Mixing specifications herein. Fly Ash will be considered upon receipt of a Value Engineering Proposal.

5.1.3 Construction Requirements

Required Contractor Submittals

Prior to the start of construction, the Contractor shall submit in writing, for approval, the following items:

1. The approximate length of soil cement to be placed prior to starting compaction operations.
2. The type of compaction equipment to be used.
3. The number and type of watering equipment to be used.
4. The method used to keep surfaces continually moist until subsequent layers of soil cement are placed.
5. The method used to cure permanently exposed surfaces.
6. The proposed source of soil, if other than required excavations.

Preparation

Before soil-cement processing begins, the area to be lined shall be graded and shaped to lines and grades as shown on the Plans or as directed by the Engineer. The subgrade shall be compacted to a minimum of ninety (90%) percent of maximum density.

Immediately prior to placement of the soil-cement mixture, the subgrade shall be moistened if necessary. Soft or yielding subgrade shall be corrected and made stable before construction proceeds.

Mixing

Soil cement shall be central-plant mixed in an approved twin shaft, continuous-flow or batch-type pugmill. The plant shall be equipped with screening, feeding and metering devices that will add the soil, cement, fly ash (if utilized), and water into the mixer in the specified quantities. In the production of the soil cement, the percent of cement content and the percent of fly ash content,if utilized, shall not vary by more than ± 0.5 percent from the contents specified by the Engineer. The plant should also be equipped with a hydraulically or mechanically operated discharge hopper having a minimum capacity of six (6) cubic yards. Scales are required at both the cement feed and either the soil or total mix feed locations. Each scale shall record weight of the material and have a digital readout such that the total discharged weight per hour is displayed. These shall also be calibrated, certified, and approved by the Engineer at least forty-eight (48) hours prior to the start of production. Each scale shall be calibrated to an accuracy of plus/minus two percent (2.0%). Soil and cement shall be mixed sufficiently to prevent cement balls from forming when water is added.

The mixing time shall be that time which is required to secure a homogeneous, intimate, uniform mixture of the soil, cement, and water.

Free and safe access to the plant must be provided to the Engineer at all times for inspection of the plant's operation and for sampling the soil-cement mixture and its components.

Proportioning

The Contractor shall use the soil aggregate, fly ash content (if utilized), cement content and moisture content determined by the Engineer in accordance with laboratory tests. The Contractor shall allow a minimum of eight (8) days for the cement content results. During the course of the work, the Engineer shall adjust the job mix proportions whenever necessary in order to achieve the minimum design strength shown in subsection Mix

Design. The Contractor may have to blend overbank silty soils with the clean in situ sands to maintain ideal soil properties as specified below and avoid cement overrun. Special blending shall require constructing separate stockpiles for materials to be blended and it shall be performed by the utilization of the separate storage feed bins at the plant to the satisfaction of the Engineer.

Sieve Size	Percent Passing (Dry Weight)
1-1/2"	98 % - 100 %
#4	60 % - 90 %
#200	5 % - 15 %

NOTE: The Plasticity Index shall be a maximum of three (3). Clay and silt lumps larger than one-half inch (1/2) shall be unacceptable, and screening will be required whenever this type of material is encountered.

The amount of cement required shall be determined by test results performed by the Materials Section of the Pima County Department of Transportation & Flood Control District in accordance with the procedure specified in Testing Procedure for Determination of Cement Content required herein. The required cement content is shown in Mix Design for this project herein. Testing during the life of the project may require changes in the cement requirements which shall be made promptly by the Contractor at the direction of the Engineer.

Required Moisture

The moisture content of the mix shall be adjusted as needed at the direction of the Engineer.

Handling

The soil-cement mixture, if transported, shall be transported from the mixing area to the embankment in clean equipment provided with suitable protective devices in unfavorable weather. The total elapsed time between the addition of water to the mixture and the start of compaction shall be the minimum possible. In no case should the total elapsed time exceed thirty (30) minutes. This time may be reduced by the Engineer when the air temperature exceeds 90° F, or when there is a breeze or wind which promotes rapid drying of the soil-cement mixture.

The Contractor shall take all necessary precautions to avoid damage to completed soil cement by the equipment and to avoid the deposition of raw earth or foreign materials between layers of soil cement. Earth ramps crossing completed soil cement must have at least two (2) foot compacted thickness. Where ramps are constructed over soil cement that is not to grade, all foreign materials and the uppermost one (1) inch of the previously placed soil-cement mixture must be removed prior to continuation of the soil-cement construction.

Placing

The mixture shall be placed on the moistened subgrade embankment, or previously completed soil cement with spreading equipment that will produce layers of such widths and thicknesses as are necessary for compaction to the required dimensions of the completed soil-cement layers. The compacted layers of soil cement shall not exceed eight (8) inches, nor be less than four (4) inches in thickness.

Each successive layer shall be placed as soon as practicable after the preceding layer is completed and certified.

All soil-cement surfaces that will be in contact with succeeding layers of soil cement shall be kept continuously moist by fog spraying until placement of the subsequent layer provided that the Contractor will not be required to keep such surfaces continuously moist for a period longer than seven (7) days.

Mixing shall not proceed when the soil aggregate or the area on which the soil cement is to be placed is frozen. Soil cement shall not be mixed or placed when the air temperature is below 45° F (7° C), unless the air temperature is at least 40° F (5° C) and rising.

Compaction

Soil cement shall be uniformly compacted to a minimum of 98 percent (98%) of maximum density as determined by field density tests. Optimum moisture and maximum density shall be determined in accordance with Arizona 221, 222b procedures. Wheel rolling with only hauling equipment shall not be an acceptable method of compaction.

At the start of compaction, the mixture shall be in a uniform, loose condition throughout its full depth. Its moisture content shall be as specified in Required Moisture herein. No section shall be left undisturbed for longer than thirty (30) minutes during compaction operations. Compaction of each layer shall be done in such a manner as to produce a dense surface, free of compaction planes, in not longer than one (1) hour from the time water is added to the mixture. Whenever the Contractor's operation is interrupted for more than two (2) hours, the top surface of the completed layer, if smooth, shall be scarified to a depth of at least one inch (1") with a spike tooth instrument prior to placement of the next lift. The surface, after said scarifying, shall be swept using a power broom or other method approved by the Engineer to completely free the surface of all loose material prior to actual placement of the soil-cement mixture for the next lift.

Finishing

After compaction, the soil cement shall be further shaped to the required lines, grades, and cross sections and rolled to a reasonably smooth surface. Trimming and shaping of the soil cement shall be conducted daily at the completion of each day's production utilizing a smooth blade.

Curing

Temporarily exposed surfaces shall be kept moist as set forth in Placing herein.

Care must be exercised to ensure that no curing material other than water is applied to the surfaces that will be in contact with succeeding layers.

Permanently exposed surfaces shall be kept in a moist condition for seven (7) days, or they may be covered with some suitable curing material subject to the Engineer's approval. Any damage to the protective covering within seven (7) days shall be repaired to satisfaction of the Engineer.

Regardless of the curing material used, the permanently exposed surfaces shall be kept moist until the protective cover is applied. Such protective cover is to be applied as soon as practicable, with a maximum time limit of twenty-four (24) hours between the finishing of the surface and the application of the protective cover or membrane.