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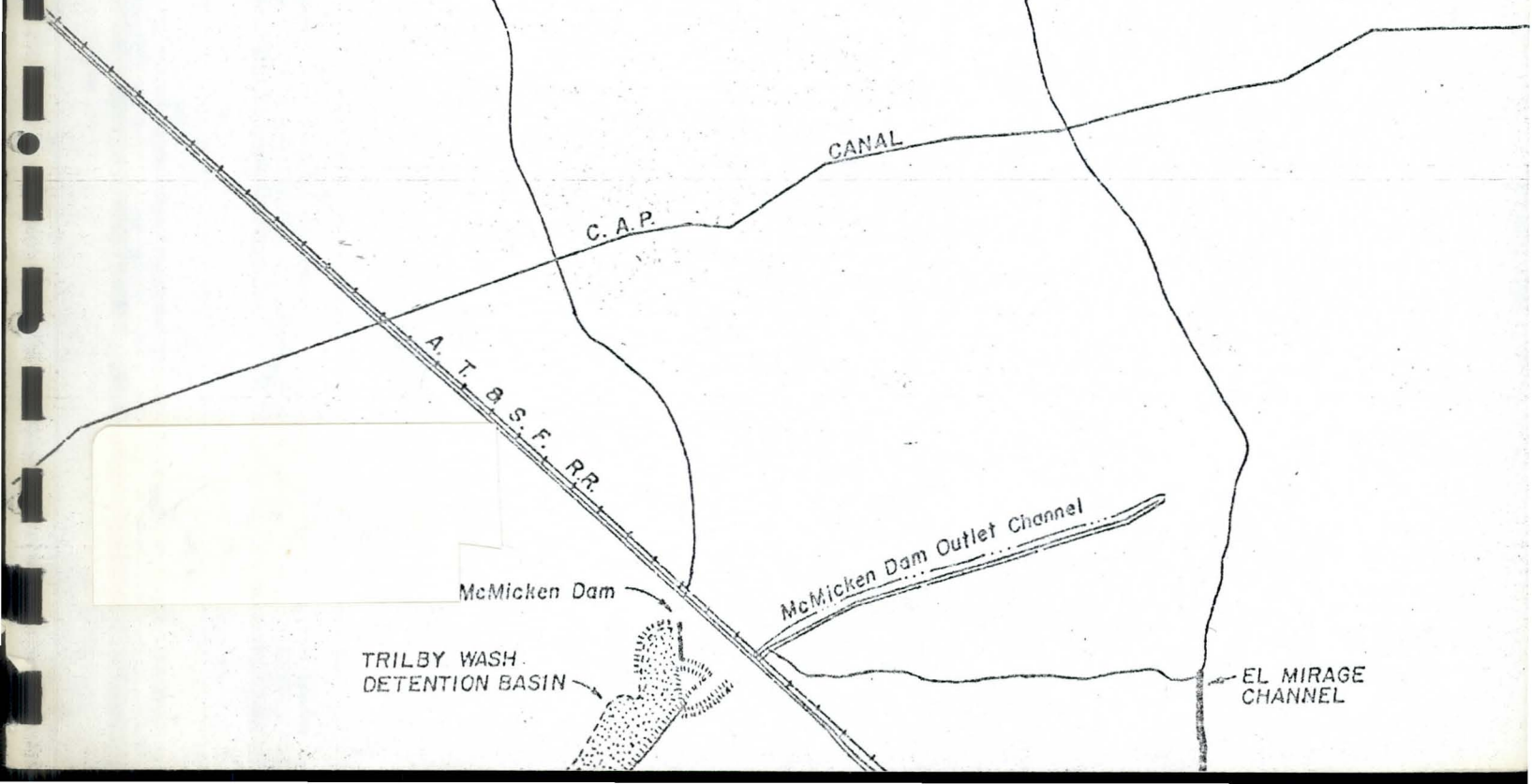
EL MIRAGE CHANNEL

W. S. GOOKIN & ASSOC.

W. S. GOOKIN & ASSOCIATES
CONSULTING ENGINEERS

MARCH 3, 1980



REPORT ON LANDS

IN SECTIONS 24 & 25

TOWNSHIP 4 NORTH, RANGE 1 EAST

GILA AND SALT RIVER BASE AND MERIDIAN,

PREPARED FOR:

ARIZONA STATE LAND DEPARTMENT

Registered Professional Engineer (Civil)
CERTIFICATE NO.
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THOMAS ALLEN
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Date Signed 3/6/80
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BY

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March 7, 1980

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SUMMARY

On January 2, 1980 the engineering firm of W. S. Gookin & Associates was employed by the State Land Department to investigate and evaluate a drainage channel (El Mirage Channel) which had been constructed in Township 4 North, Range 1 East on the east side of Sun City West.

The purpose of the investigation was to determine the reasons for the size of the El Mirage Channel, its impact and potential damage to State lands, and to suggest engineering solutions to ameliorate any potential adverse effects of this Channel.

Based upon the field and office investigations of W. S. Gookin & Associates the following conclusions were made:

1. Basic correspondence indicates that the El Mirage Channel was allegedly designed to handle the Standard Project Flood outflow from the McMicken Outflow Structure and the 100-year flood from unregulated tributary drainage. Due to settlement problems, the McMicken Dam has been temporarily lowered in reaches. It is assumed the McMicken Dam will soon be restored. If the McMicken Dam breeches, the sudden release of water from the Trilby Wash Detention Basin would have a serious impact on Sun City West but most of the flow resulting from such a failure would not reach the channel under investigation.

2. The El Mirage Channel as designed does provide marginally adequate capacity to handle a 100-year storm assuming no erosion. The channel does not, however, have

adequate freeboard in all cases nor is it capable of handling any flows significantly greater than a 100-year storm.

3. The channel as designed does not provide adequate protection against flows entering the channel from the sides. Such flows result in severe erosion in the unprotected earthen banks.

4. The velocities of the water that will be experienced during a 100-year flood will be much higher than the earthen banks will be capable of handling. The cementation that was counted on to protect the channel is, according to the soil tests and visual inspection virtually nonexistent. The erosion that has already occurred from side drainage is an indication of the inadequacies of the channel to sustain the flood flows.

5. Flood flows will peak at a slightly lower level than had been anticipated, however, the high velocity flows will continue for a long duration.

6. The channel cannot be expected to remain in its existing location. The situation has been changed from its natural state under which where the lands of Sun City West would have been flooded to a condition where the State lands have the high risk of flooding.

7. As a result of the large amounts of erosion that will probably occur, the channel will change course and probably erode State Land Department lands.

8. The solution to the problem will involve lining the canal system. There are several design alternatives in the lining that should be examined before a final remedial design is made.

AUTHORITY

Under date of January 2, 1980, W. S. Gookin & Associates entered into a contract with the Arizona State Land Department designated as A. G. Contract #0012 SLD No. 1-80. The contract provided for: (1) An investigation and evaluation of a drainage channel, which had been constructed in Sections 25 and 36 of Township 4N and Range 1E; (2) An assessment of the impact of said drainage channel; (3) A determination as to whether said channel had created incurable damage to the potential use of State lands, by reason of increased flooding; (4) A determination as to whether the existing channel has impaired the State Land Department's ability to use the land for commercial and residential purposes; (5) Suggestions as to reasonable engineering options to cure the adverse effects of this channel, if in fact the channel were found to have adverse effects; and (6) A determination of the reasons, if any, for the size and the drainage channel as constructed.

CLIMATE

The climate of the area is characterized by short mild winters and long hot summers. The average annual precipitation totals about 8 inches. Although rain may fall at any time during the year, for the most part this precipitation falls in two periods, i.e. the summer season of July, August, and September and the winter season which extends from December through March.

Typically, about 45% of the precipitation occurs during the winter season, about 35% during the summer season with the remainder being scattered throughout the year.

Winter precipitation is generally characterized by gentle storms that move into Arizona from the north and west. The winter storms generally extend over wide areas of the State. However, occasionally, and particularly during wet winters, storms enter the area from the south and west. The winter storm characteristically covers a wide area and compared to the summer storms are of long duration. Not infrequently, a series of winter storms will follow one another a few days apart. It is this type of storm that causes the major floods that have ravaged southern Arizona, particularly along the Salt River and its major tributaries during the winters of 1978, 1979, and 1980.

Most of the summer rains fall in evening thunderstorms that are caused by the flow of moist air from the Gulf of Mexico. This moist air rises as it encounters the mountains and condenses into rain which falls as precipitation upon the area. The summer

storms are of short duration and relatively small areal extent. These storms sometimes occur with great intensity and cause localized flood problems as opposed to flood flows over wide spread areas.

LOCATION

Sun City West is located in Maricopa County, Arizona about 20 miles northwest of Phoenix, west of the Agua Fria River and north of the Salt River. Bell Road forms the southern boundary of Sun City West. Current plans call for development of housing in Townships 3 and 4 North, Ranges 1 and 2 West Gila and Salt River Base and Meridian.

Sun City West is designed as a separate community but is essentiallv a westerly extension of a spectacularly successful retirement community known as Sun City. Sun City lies east of Agua Fria River.

Sun City West lies north of Luke Air Force Base, one of the largest military Air Force bases in the country.

The channel to be investigated under the terms of the contract previously mentioned has been excavated along the eastern edge of Sun City West.

Although field surveys were beyond the scope of this contract, it appears from an examination of the aerial photography that some portions of the channel in question lie outside of the right-of-way granted by the Arizona State Land Department to the Maricopa County Flood Control District under Right-of-Way Permit No. 18-1368. For example, the excavated channel appears to straddle line which separates Section 24 and Section 23.

According to the map made available to this office by the Arizona State Land Department, designated as Sheet #3, of County #7, with ownership of Bureau of Land Management, lands indicated as of June 25, 1979, and ownership of State lands indicated as of July 2, 1979, the west half of the northwest quarter and the northwest quarter of the southwest quarter of Section 24 appears to be owned by the Bureau of Land Management. Some portions of the excavated channel lie within that area.

Moreover, neither the State Land Department Right-of-Way Permit nor the ownership map indicate any State land ownership in Sections 23 or 26. A portion of the channel seems to have been constructed in those sections.

The channel in question does, however, appear to lie within the area licensed to the Del E. Webb Development Company by the Flood Control District of Maricopa County under terms of License #~~79802~~ 79002.

CHANNEL LOCATION

The channel in question is between 6500 and 7500 feet in length and for the most part is located along the West line of Section 24, Township 4 North, Range 1 West Gila and Salt River Base and Meridian, beginning near the northwest corner of that Section and proceeding south along the Section line to Section 25. The channel is not located in Sections 25 and 36 as was originally thought to be the case when the contract was drafted. For the most part, the channel is located in Sections 24, 25 although some portions have apparently been constructed in Sections 23 and 26. An accurate determination of the exact location of the channel would require a survey of the Section lines and surveys of this magnitude and type were beyond the scope of this contract. However, it appears from an examination of the aerial photographs that some portions of the channel in question lie outside of the right-of-way granted by the Arizona State Land Department to the Maricopa County Flood Control District under right-of-way permit 18-368. For example, the excavated channel appears to straddle a line which separates Section 24 and Section 31.

According to the map made available to this office by the Arizona State Land Department designated as Sheet #3 of County #7 with ownerships of the Bureau of Land Management indicated as of June 25, 1979 and ownerships of State Lands indicated as of July 2, 1979, the West half of the Northwest quarter and the Northwest quarter of the Southwest quarter of

Section 24 appears to be owned by the Bureau of Land Management. Some portions of the excavated channel lie within that area.

Moreover, neither the State Land Department right-of-way permit nor the land ownership map previously referenced indicate any State or Federal ownership in Sections 23 or 26. It appears that a portion of the channel has been constructed in those Sections.

The channel in question does, however, appear to lie within the area licensed to the Del E. Webb Corporation by the Maricopa County Flood Control District of Maricopa County under license #~~79802.~~ 79002

NATURAL SETTING

Under natural conditions the area proposed for development at Sun City West was a relatively flat desert sloping from the northwest to the southeast. All runoff in the area was tributary to the Agua Fria or Salt Rivers. With the exception of the Agua Fria River, there were no other major drainage channels unless Trilby Wash would be so classified.

For the most part surface runoff occurred either as sheet flow or in multitudinous small drainage channels which flowed in a southeasterly direction toward the Agua Fria River. All of these smaller channels were typical desert type ephemeral streams which were completely dry except during brief periods of intense rainfall or immediately thereafter.

During World War II a large air facility was constructed in the Northwest portion of Township 2 North, Range 1 West. This facility was and is designated as United States Luke Air Force Base. Sporatic flooding constituted a serious problem for Luke Air Force Base.

Canals and irrigated lands of the Maricopa County Municipal Water Conservation District No. 1 received periodical damage from the uncontrolled floods. That District which is frequently referred to as the Beardsley Project, is located in portions of Township 1 and 2 North and 1 and 2 South, Ranges 5, 6 and 7 East. Approximately 35,000 acres are irrigated with water from both the Agua Fria River impounded in Lake Pleasant, which has a capacity of 163,820 acre feet, regulated by Carl Pleasant (Waddell) Dam and also by

pumpage from 61 District owned pumps and a similar number of privately owned pumps.

On August 7, 1953, the 83rd Congress, First Session approved Public Law #209 which authorized construction of the Trilby Wash Detention Basin and Outlet Channel. This structure is commonly known as the McMicken Dam. Construction of this facility was completed by the U. S. Army Corp of Engineers in July 1956 and the structure was dedicated in November of the same year.

McMicken Dam consists of an earthen structure 5200 feet in length with a maximum heigth of 34 feet. The total impound capacity is 19,300 acre feet. McMicken Dam is located in Section 18 of Township 4 North, Range 1 West and Township 13 of Range 4 North 2 West. It controls a drainage area of 228 square miles. The Dam intersects a number of small tributaries, the largest of which is Trilby Wash. There was and is no natural channel with a substantial carrying capacity below the detention basin. For that reason, the outflow is conveyed through an outlet channel extending for 5-8/10 miles into an unnamed wash, sometimes incorrectly referred to as Trilby Wash, which extends approximately parallel to but converging with the Agua Fria River, while transporting the water from the detention basin the outflow channel also intercepts and transports numerous washes. The small wash extends the discharge of the McMicken Dam outlet channel in Section 11 of Township 4 North, Range 1 West to a convergence with the Agua Fria River in Section 25 of Township 4 North, Range 1 West.

On the downstream side of the outlet channel there was a levee with an average height of about 8.5 feet designed to prevent overflow. Initially, this entire project was

designed to protect Luke Air Force Base, the Litchfield Park Naval Air Base, Goodyear Aircraft Company Plant and the Towns of Goodyear, Litchfield Park and Avondale and the surrounding agricultural lands. The facility was designed to reduce the Standard Project Flood which was estimated to have a peak inflow of 35,000 cfs to a peak outflow of 4,450 cfs.

The Standard Project Flood is defined in Circular No. 4262, Civil Works No. 65, dated 20 November 1946, from office, Chief of Engineers as follows:

> "*** The Standard Project Flood) is intended to represent a flood that would be exceeded in magnitude only on rare occasions, and thus to constitute a standard for design of structures that would provide a high degree of flood protection as determined by flood potentialities of the drainage system involved, without regard to localized economic or other practical limitations of individual projects. *** In most cases, the Standard Project Flood should equal or approximate the flood that would result under existing or specified conditions of basin development, if the most critical storm of record in the region should occur over the drainage area involved when hydrologic conditions were reasonably favorable for flood run-off.***"

The evaluation of the hydrology of the channel east of El Mirage Road on the east side of Sun City West was and is predicated on the assumption that McMicken Dam and outlet channel will function as intended.

Within the last few years, McMicken Dam began to develop cracks which indicated that the Dam might fail in time of emergency. While the exact cause of these fissures has not been authoritatively established, it is generally believed they are due to settlement of the foundation due to excessive pumping from the groundwater reservoirs. As

a result that groundwater levels in the area have lowered rather dramatically. In an effort to avoid a failure of the McMicken Dam during a critical period such as a Standard Project Flood with the resultant sudden release of more than 19,000 acre feet of water, the Dam has been lowered in certain areas.

This lowering of the Dam will impair its ability to function as designed. While this office has made no studies of the Dam, it would appear to be almost a foregone conclusion that since the Dam has lowered capacity it would not control the Standard Project Flood. The Maricopa County Flood Control District recognizes this fact and has appealed to the Corps of Engineers to restore the Dam to its original condition. Although there has been no commitment made by the Corps of Engineers, it is hoped that the Corps of Engineers will soon restore the Dam to its original height, particularly because of the greatly increased downstream development which has occurred and will presumably continue to occur downstream from the Dam.

In the studies in this report, it has been assumed that the McMicken Dam will be restored before a major flood occurs.

It must be recognized however, that due to the catastrophic floods of 1978, 1979 and 1980 on the Salt and Gila Rivers there will be a tremendous demand for flood control money. To be realistic, there is no possibility that flood control funds in the amounts necessary to ameloriate

the flood hazards of Southern Arizona can be made available for several years. Competition for the available funds will be intense.

It is interesting to note that the rains of January and February 1980 did not even approximate the Standard Project Flood in terms of either peak or volume at McMicken Dam.

The McMicken Dam has two provisions for the release of storage. The first is a low flow outlet which can handle up to 4,450 cfs before the spillway is utilized. The 4,450 cfs outflow was calculated by the Army Corps of Engineers in 1953 as being sufficient to contain the Standard Project Flood. The second outlet structure is a spillway which begins to release when the storage capacity exceeds 17,000 acre feet. This spillway is designed to handle a peak outflow of 22,000 cfs which would occur during the maximum probable flood (maximum possible flood).

There is also an area which lies east of the McMicken Reservoir which is unregulated and has a potential for flooding.

Crossing this drainage area there are three structures which might be considered to alter the runoff characteristics. The first of these is the main canal of the Maricopa County Municipal Water Conservation District No. 1, more commonly known as the Buckeye Canal. The second is U. S. Highway 60, 70 and 89 which leaves Phoenix as Grand Avenue and proceeds in a northwesterly direction toward Wickenburg. There is in addition to the Highway a branch of the Atchinson, Topeka &

Santa Fe Railroad. A more recent development which crossed the drainage area is the Granite Reef Aqueduct recently constructed to convey water from the Colorado River to Central Arizona. Representatives of this office studied the foregoing features and have ascertained that in each case they permit flow over or under the structure in question. In numerous places some of the flows will cross on overshoots, others in siphons and others in culverts. In any event, the number of structures involved is so great as to lead this office to the conclusion that the practical effect of the Buckeye main canal, the named U. S. Highways and the Railroad would not substantially alter the amount of water reaching the subject channel nor would these structures materially change the time of concentration. To the extent that the time of concentration would be changed it would be slightly reduced.

The Granite Reef Aqueduct was found to have a slight restrictive effect upon the peak flows passing through the area. Data concerning this retardation effect was gathered from the Project's office of the Water and Power Resources Service.

In addition there are, of course, numerous roads and other man made developments which would increase the rate of runoff and the time of concentration an indeterminate amount. However, in the computations which were made, this office did not give consideration to the hydrologic effect of any of the man made developments on the uncontrolled watershed.

INVESTIGATIONS

Pursuant to the terms of the contract, representatives of W. S. Gookin & Associates made field inspections of the drainage channel and adjacent areas on February 5, during a period when the weather was dry and there was no flow in the Agua Fria River. A second investigation was made on February 18, during a time when it was raining and there was heavy flow in the Agua Fria and Salt Rivers. The first investigation was prior to the failure of the Agua Fria Bridge on Bell Road and the second was subsequent to the failure of that bridge.

There was no flow in the channel in question on February 5, 1980. The flow in the referenced channel was very small on February 18, and there was no evidence of excessive flows since construction of the channel.

In addition, representatives of W. S. Gookin and Associates: (1) Met with officials of the Maricopa County Flood Control District (MCFCD) and obtained data from the files of that agency; (2) Visited and were furnished with data from the offices of the Arizona State Land Department; (3) Conferred with representatives of the Water and Power Resources Service (WAPRS); (4) Visited the office of Landis Aerial Surveys and Photography to inspect both old and new aerial photographs of the area; (5) reviewed plans and profiles of the main canal of

the Maricopa County Municipal Water Conservation District #1; (6) Studied plans and profiles of the Granite Reef Aqueduct of the Central Arizona Project; (7) Analyzed numerous maps and aerial photographs; and (8) Analyzed the design of McMicken Dam and Outlet works.

Field Inspection, February 5, 1980

On Tuesday, February 5, 1980, Mr. W. S. Gookin, P. E. and Mr. T. A. J. Gookin, P. E., traveled to Sun City West to view the excavated channel and begin formulation of a report. They proceeded west on Bell Road until they crossed the bridge on the Agua Fria, which was at that time undamaged. They then turned north to El Mirage Road as depicted on the key map for Sun City West, Unit 12, Sheet 2 of 29, by HDR, dated 6/12/79. There was a sign on this road prohibiting trespassers.

They had in their possession aerial photographs from which they were able to orient themselves on the ground. They drove north along the east side of Sun City West. The road paralleled the east side of Sun City West, where a fence existed, and continued north paralleling the obvious location of a northerly extension of the fence on the east side of Sun City West.

East of the road on which they were traveling a channel had been excavated. They proceeded north along El Mirage Road to a point near the northwest corner of Section 24 where El Mirage Road terminated. At about that point, the newly excavated channel tapered into natural ground level. At that point there was a small channel, apparently designed to intercept drainage coming from the north and convey it eastward toward the main drain. The east-west channel apparently had not been completed on February 5. They presumed that ultimately the east-west channel would be extended westerly to provide protection for the north boundary of

Sun City West. Maps and aerial photographs indicate that at least partial protection from flood water entering Sun City West from the north is provided by an east-west ditch which runs along the southern edge of Sections 13, 14, 15 and 16. This drain ditch was apparently constructed to protect an irrigation ditch located in the north edge of Sections 21, 22 and 23 parallel to the drain ditch.

The east-west excavation was relatively minor at that time and there is no indication as to what the ultimate plans are for that particular area; however, it would seem logical to presume protection from flood flows would be provided along the north line of the subdivision, and the logical place for disposal of this water would be through the north-south channel excavated to the east of El Mirage Road.

The plans issued by HDR indicate that the bottom of the excavated channel is about 180 ft., with variable depth and variable side slopes. Where unlined, the side slopes are indicated to be approximately 3 feet horizontal and 1 foot vertical. Detailed measurement of these slopes would have to be made to confirm this if this becomes an important factor.

A few reaches of the excavated channel had cobble rip-rap on the west side, but the side slopes were for the most part unlined. The bottom of the channel was completely unlined, except for approximately 1200 feet. The north edge of the lined portion is shown on the plans prepared by HDR as being 3,625 feet north of the point where

the channel turns eastward to join the Agua Fria River. The lined portion is shown on the plans extending southerly for a distance of 475 feet. This lined portion also shows on the aerial photographs.

It is obviously a concrete-lined energy dissipator. A cross-section of this dissipator is shown on Sheet 4A of 29 of the HDR Paving and Grading Plans.

The lined section as constructed does not conform to the configuration depicted on the plans. On February 25 there was water ponded in the bottom of the energy dissipator which was not draining through the "V" channel designed for that purpose. Sheet 26 of 29 of the HDR Paving and Grading Plans shows the concrete dissipator and indicates that it is approximately 475 feet long. Visual inspection indicates this is not unreasonable. However, the plans indicate that the channel will be so graded as to provide for a graded inlet to the channel and a complete draining of the dissipator after flows have occurred.

Visual inspection seems to indicate that the dissipator was placed on top of the excavated channel after the excavated channel was completed. In any event the dissipator does not drain as designed and would impound water and thereby constituting a possible breeding point for mosquitoes and other insects.

Downstream from the energy dissipator there is a deep gunite-lined gulley through which water will drain from

Sun City West and enter the channel through a culvert from the west. The gulley has near vertical sides which, in their opinion, represents a distinct hazard to human and animal life.

At the time of their visit, there was no evidence of excavation in progress in the newly dug channel paralleling El Mirage Road.

It was obvious that large amounts of fill material had been excavated from the newly dug channel and then supplements of large amounts of similar material from another source had been added to the land encompassed in Sun City West.

Water flowing in the excavated channel will impact against a near vertical, unprotected bank, as shown on sheet 24 of 29 of the plans by HDR, revised 6/29/79.

There is a small exit through which water flowing in the newly constructed drainage channel will turn eastward and join the Agua Fria; however, this exit has a far lesser capacity than that of the drainage channel.

In the newly excavated channel near its south terminus is a saddle with low dirt dike. There was no evidence of any compaction on this dirt dike.

If water were to flow in any quantity in the newly excavated channel, the near vertical unprotected bank referenced above would be eroded and the dike would be overtopped and washed away. The water thereof released would flow in an easterly direction to the main channel of the Agua Fria River.

At the time of their visit to the site, there was extensive hauling in process from gravel pits that are located in the southern half of Section 24, which, according to the maps furnished by the State Land Department, may or may not be located in whole or in part on State land. Aerial photographs seem to indicate that the gravel pits lie just to the east of State land. A survey would be required to verify whether or not this is ture.

They noted a number of trucks, some full and some empty, heading to and from the gravel pits of Sun City West. They also noticed some empty trucks coming from Sun City West toward the gravel pits.

Upon exploring the area along Bell Road, they noticed two extremely steep-sided gunite lined ditches north of and parallel to Bell Road, each heading toward the Agua Fria River. In times of flood these ditches would discharge water into the Agua Fria River north of the breach.

These drainage ditches appear to represent a distinct hazard to automotive, pedestrian, and all other traffic regardless of whether these ditches contain water or not.

Although no measurements were taken it was obvious at the time of their visit that construction of the excavated channel had not been completed in accordance with the plan and correspondence. For example, on Sheet 24 of 29, there is shown a short stretch of channel lining which is further detailed as Section AA on Sheet 4. There is no evidence of this lining as of February 5, 1980.

Both the correspondence and the plans speak extensively of rip-rap and lining which was not in place on February 5, 1980. For example, on Sheet 4A of 29, the typical channel Section from Station 32+00 to Station 36+25 shows a concrete lining on both side slopes and bottom. Moreover, the drawing indicates that the top of the lining is 9'6" above the drainage channel flow line whereas the drawing also shows that the depth of the excavation from the shoulder of El Mirage Road to the bottom varies from 27.5 to 5.3 feet.

The typical channel section from Station 64+00 to 67+50 is shown to have a 3" concrete channel lining with a minimum height above flow line of 8'0" and extending below the flow lining a vertical distance of 4'0". There was no evidence of this type of lining on Feb. 5, 1980. Essentially, the same thing holds true of the typical channel section which shows a lining on the West bank from Station 0+00 to Station 2+50 extending 9'6" above the hinge line and downward 4'0" below the hinge line.

Very little cobble protection was noted on the west bank of the excavated channel and no cobble protection was noted on the east bank.

As noted elsewhere in this report, the velocities expected in this newly excavated channel will be such as to cause severe erosion in the main channel and the inflow from side drainage will severely erode the side slopes unless protective measures are taken.

Field Inspection - February 18, 1980

On February 18, 1980, Mr. T. A. J. Gookin, P. E. went to the site at approximately 11:00 a.m. At that time there was a very light rainfall and no significant runoff was occurring.

At the time of his visit, the Bell Road Bridge crossing the Agua Fria River was reportedly washed out. Bell Road was closed so that he could not get close enough to the Bell Road Bridge to see it. He had to enter Sun City West, west of the Agua Fria River and drive through the subdivision until he found an exit to El Mirage Road. He then drove north and south on El Mirage Road several times along the entire length of the newly excavated channel.

During his inspections the following phenomena were observed.

1. There was no evidence of substantial flows of water in the newly excavated channel since its construction.

2. The channel was flowing at a very small fraction of its overall capacity. Channel flows were slow and sluggish due to the minimal amounts of water being transported.

3. For the flow which was occurring, the dikes in the exit portion were not restraining the water.

4. The Energy Dissipator had been so constructed as to cause a ponding of the water upstream from

it. It is probable that this water will sink into the river bed until silt and fine sand from upstream portions reduce the infiltration rate to a point where the water will effectively stagnate.

5. The south outlet from the subdivision into the channel did not have erosion protection devices on it that were desperately needed. The side of the river channel experienced severe erosion cuts approximately five feet deep with some undercutting of the outlet structure itself apparent.

6. On the far side of the channel, there were numerous cases where small rivulets of water entering into the channel from the land immediately East of the channel had caused the beginning of significant erosive actions. If any major amounts of water were allowed to come in from this source, or if this damage is allowed to continue without immediate repair after every rain storm, the east bank of the channel could be significantly endangered.

In the case of the 100 year storm, a tributary drainage area of two acres, which, according to the data contained on the U. S. Geological Survey Maps does exist in some areas, could yield a side flow drainage of 1.25 cfs. This runoff going down the side of an unprotected slope of approximately 3:1 as now exists on the sides of the newly excavated channel,

would theoretically achieve velocity of 200 feet per second. As a matter of practicality the water could not be achieve velocities that high and the large amounts of energy that could not be dissipated would exert their forces in the erosion of the east bank of the channel.

This phenomena also occurs, although to a significantly lesser extent, on the West bank of the channel.

The significance of these gullies is that during a 100 year storm the time of concentration for the creation of these side flows is only 15 minutes, whereas the time of concentration for the 100 year flow that will go through the channel would be five hours. Therefore, the side flows would peak and erosion would cut gullies into the side slopes of the channel before the peak flows from the 100 year flood would arrive in the main channel. When the peak flows did arrive, they would not find a channel with smooth sides of compacted dirt as had been anticipated, but would instead find a channel riddled with side gullies which would increase the turbulence of the water going through the channel, greatly multiplying the erosive effects of the large flows and all likelyhood washing away the east bank of the drainage channel and washing into and over the other State lands in Sections 25 and 24.

The west bank of the channel would be endangered due to both its own gullies and the effects cause by the turbulance caused by the east side of the channel. It is very probable that undercutting of El Mirage Road, the fence on

the east side of Sun City West and conceivably some residential foundations could occur.

7. Rip-rap protection and concrete protection with the exception of the Energy Dissipator as evidenced in the Specifications were not in evidence at any point in the channel.

FLOOD CONTROL COMPUTATIONS

Soil Conservation Service Method II

The Arizona Highway Dept., Bridge Division has published a book entitled "Hydrologic Design for Highway Drainage in Arizona" dated December 1, 1968 to which there have been periodic updates of precipitation tables and other data contained therein. In this book there are three methods delineated for the computation of flood flows. The most common method used is the so called rational method. The rational method is a simplified technique for use in very small drainage areas where the shape and slope and other factors are not of particular significance. This method is generally applicable only to areas of less than one square mile.

The second method is the Soil Conservation Service Method I which is designed for drainage areas of less than 10 square miles.

The third method is the Soil Conservation Service Method II designed for drainage areas greater than ten square miles and is the procedure used by this office in computation of flood flows.

The design frequencies selected were for a 100-year storm, with durations of 1 hour, 2 hours, 3 hours and 6 hours.

Rainfall amounts for the various durations were determined by precipitation maps prepared by the U. S. Weather Bureau for the Soil Conservation Service in March, 1967, and revised by the Arizona Highway Department in 1970. As the area studied had more than one precipitation value within it, for purposes of our report the values were averaged.

Precipitation values for 6 and 24-hour durations were plotted as a straight line on the rainfall depth-duration diagram prepared by the Weather Bureau, Technical Paper No. 40. By use of this diagram, intercepting values of 1 hour, 2 hour, and 3-hour rainfall depths in inches were determined.

The precipitation values obtained from U. S. Weather Bureau maps depicted point precipitation values; therefore, a reduction of these values would show a uniformly distributed rainfall depth over the entire drainage area. This reduction is necessary because a value for a storm centered over a large area would not be constant over the total area. Reduction of the point values was obtained by use of Figure 2-6, Area-Depth Curves prepared by the Weather Bureau. These reduced point values will be called areal reductions.

The drainage area studies were plotted on the General Highway Map of Maricopa County, Arizona, dated 1968, prepared by The Arizona State Highway Department on a scale of 2 miles to the inch. Where necessary, an adjoining map of the same series was

used to determine the areas in Yavapai County.

By use of a planimeter, the areas were determined. The overall dimensions of the drainage basin were acquired from the same maps. Changes in elevation within the various drainage basins were determined from the appropriate United States Geological Survey Quadrangle Maps. The overall slope was generally constant in all cases with no significant breaks, so in each case, the area studied was studied as a whole.

The vegetative cover type of the drainage area was determined to be Desert Bush, with a "fair" cover density of 20%.

The hydrologic soil cover complex was determined by the use of the hydrologic soil map prepared by the Soil Conservation Service. The drainage area studies included three groups of soils: types B, C, and D. The majority of the area was group D, or soils having a very slow infiltration rate and a high run-off potential. The amount of each type was determined separately for each study.

By use of the hydrologic soil cover and the vegetation type determined for the area, a weighted curve number was found.

Using these data, it is possible to compute a peak flow along with a time at which that peak flow would occur. This may be done for a one hour, two hour, three hour and six hour storm. The storm that would cause the maximum flow for a given storm frequency varies upon the size, shape, and slope of the drainage area under consideration. It is, therefore,

necessary to compute each of the storm floods to determine which yields the maximum peak runoff.

There are many different frequencies of storms that may be designed for. This method allows for the design of a two year, five year, 10 year, 25 year, 50 year, and 100-year storm. The frequency of the storm means that it may be expected that over the long term, a storm equal to or greater than the computed magnitude will occur in one year out of the number of years indicated. For example, a two-year storm would be that storm that would be expected to occur one year out of two. A 100-year storm would be that storm that would be expected to occur in one year out of 100. However, this is not to say that it is not possible for two 100-year storms to occur in one year. The event frequency is merely a statement of the probability and not a prediction as to when any given individual storm would occur.

HDR Calculations

Henningson, Dunham & Richardson, Inc. of Arizona (HDR) based their computation of the flood flow upon the procedures described above. As nearly as we could ascertain, HDR did not compute the flows from the Trilby Wash Dentention Basin, but accepted the outflow conclusions of the Army Corps of Engineers that in a Standard Project Flood, there would be releases at the rate of 4,450 cfs.

The remainder of the drainage area that is not contained by the McMicken Dam was apparently calculated, utilizing the above, referenced techniques. A similar calculation by this office checked the computations of HDR for a two-hour, 100-year storm to be within 8 percent. In view of the many variables involved such as the differences in maps available and the necessary judgment factors in developing the peak discharge, it is the opinion of this office that the two figures checked as closely as could be reasonably expected.

However, the methodology presented by the Highway Department is to compute the 100-year storm for a 1, 2, 3, and 6-hour duration and determine the peak inflow from whichever storm creates the greatest peak.

Because of the configuration of the watershed, which was found to have a time of concentration of five hours, the peak flood occurred from the six hour storm. The time of concentration is a figure which indicates the time required for a drop of water originating in the head-

waters of the drainage area to reach the outlet portion of the drainage area. The time of concentration is important, because it indicates at what point the entirety of the watershed can be expected to begin its contribution to the flood flow. The time of concentration is related to but is not synonomous with the time at which the peak flow from any given storm occurs.

When the computations were made for all storms and the Standard Project Flood for the McMicken discharge of 4,450 cfs was added on, it was found that the channel flow using the above methodology would be 21,000 cfs.

W. S. Gookin & Associates' Calculations

As indicated earlier, there are several factors which would effect the actual flood flow through this region that were not taken into account by HDR. The primary of these factors is the Granite Reef Aqueduct. This would effect not only the unregulated portion of the drainage basin but, to some extent, the Trilby Wash Detention Basin. Therefore, it was decided to divide the drainage area into four drainage areas, which were referred to as A, B, C, and D.

Subarea A consisted of that portion of the watershed unregulated by McMicken Dam and the south of the CAP Aqueduct.

Subarea B constituted that portion of the basin area north of the Granite Reef Aqueduct not regulated by McMicken Dam.

Subarea C constituted that area regulated by the Trilby Wash Detention Basin but south of the Granite Reef Aqueduct.

Subarea D constituted that portion of the drainage basin regulated by the McMicken Dam and north of the CAP Aqueduct.

The computations leading to our conclusions as to the flows out of each of the drainage areas can been seen on the attached Appendix "A" on the Hydrologic Data Sheets for Soil Conservation Method: Part II.

As can be seen from these sheets, the peak flows occurred on a 3-hour storm; however, due to the minor

differences in the peak flow between a 3 and 6-hour storm and the lengthy lag times involved, and the fact that the peak flows from the four areas were not coincident, the 6-hour storm was utilized since it would give the maximum overlap and peak flows.

Flood Routing

Using the methodology presented in the notes to the Hydrologist and Hydrologic Training Sessions dated October 16, 17, 18 of 1972 and revised in December 1973 by the Arizona Highway Department, there is a methodology presented for the design of hydrographs based upon the SCS Method II peak flow computation used earlier. Utilizing this technique the Hydrograph was constructed for Area D which is regulated by the Granite Reef Aqueduct to a maximum outflow of 15,377 cfs. The graphical technique is shown on Graph 1, Appendix A in which the water was routed through the CAP structures up to their maximum capacity. The outflow was then held constant at that capacity until the storage which had occurred behind the canal had dissipated. At that point the remaining flows were conveyed through the canal structures.

Graph 2 shows the routing of the resultant flow from Area D as regulated by the CAP canal along with the flows that would occur in Area C. It was computed that the flow from Area D would not arrive to the outlet point in Area C until 2.91 hours after the beginning of the runoff; therefore, on the Graph, the Area D curve is offset by 2.91 hours. These two flood flows were then added to create the overall flow shown in a heavy outline on Graph 2 into the Trilby Wash Detention Basin. Utilizing the data as to the outlet constructions contained in the 1953 Army Corps of Engineers report, the retention basin would not spill and would have a maximum and relatively continuous outflow of

4,200 cfs. This is the outflow from a 100-year flood based on local data which makes this computation consistent with that used for all other areas. The use of a Standard Project Flood for the Tribly Wash Detention Basin based on data transposed from Texas and a 100-year flood for the rest of the drainage area based on local data, presents an inconsistency that is hard to justify. Preliminary analysis creates serious questions as to whether the Tribly Wash Detention Structure could contain a Standard Project Flood using current methodology and local data for analysis without exceeding the 4,450 cfs discharge and spilling over the spillway. This can be seen as the lower curved line on Graph 2.

Graph 3 shows the flood flow from Area B and the effect of the Granite Reef Aqueduct upon that flow. The flows in excess of the maximum peak outflow over the Aqueduct were treated in a manner similar to that in Graph 1.

Graph 4 shows the computation of all inflows into the reach. There are three contributing curves depicted. The first curve shows the inflow from Area A. The second graph shows the inflow from Area B as regulated by the Granite Reef Aqueduct after a lag time of 2.28 hours. The third line shows the inflow from the Trilby Wash Detention Basin after its lag of 4.8 hours. It may be noted that on Graph 4 this flow only appears after approximately 7 hours. This is due to the 2 hour lag that occurred in Graph 2 between the time when inflow started into the detention

basin and the outflow finally began to leave the basin. These three lines were added to give the resultant 100-year flood that may be expected to impact the El Mirage channel.

The peak discharge that will occur will be 13,000 cfs which will be accompanied by a long period, approximately 9 hours, of 12,000 cfs flows. This long period of flow is significant in that any erosive damage which might begin at those flows will have a considerable length of time to exacerbate the damage. After the nine hour major flood has passed, there will be a flow of approximately 4,000 cfs which will linger on for several days at gradually decreasing rates as the Trilby Wash Detention Basin slowly empties itself.

EROSION

Permissible Velocities

In designing a canal for the transportation of water for either flood control or other purposes, one of the factors that must be considered is the velocities which are being developed through the structure and the silt or other content of the water entering that ditch. If the designed velocity is too slow, there is a tendency for the water entering the reach under consideration to deposit the heavier materials which would result in blockages and reduction in the overall capacity. This is referred to as the silting velocity. The silting velocity constitutes the lower end of the speeds which may be permitted in the reach being designed.

It usually is impractical to design a structure so that the water passing through it will never drop below the silting velocity because the velocity varies with the quantity of flow. Velocities often drop below silty velocity during low flow periods, but rise above silty velocity during high flow periods. However unless the flow drops below silting velocity on a regular basis, any deposits that might occur would be quickly washed out during a major flood flow.

The other extreme is the scouring velocity. This is the velocity at which the water will begin to erode the channel lining and given sufficient time, destroy the integrity of the channel. The more important of the two velocities is the scouring velocity.

The maximum permissible canal velocities are primarily dependent upon the material constituting the lining. In the worst case of fine sand a water velocity of 1.5 feet per second is the maximum that can be tolerated. In extreme cases where reinforced concrete linings have been installed, velocities of 150 feet per second have been designed. This was the design velocity for the spillway tunnels at Hoover Dam; however, when tested at this velocity problems were discovered in cavitation which indicates that the maximum should be held below that speed. Concrete lined canals are commonly designed with velocities of 20 to 40 feet per second, with scattered examples existing that show 80 feet per second. The suggested design standards for this canal were gleaned from the following sources: Flow of Water in Irrigation and Small Canals, by Mred C. Scobey of the U. S. Department of Agriculture; Irrigation Principles and Practices, by Orson Israelson and Vaughn Hansen; Drainage and Flood Control Engineering, by George Pickels; Linings for Irrigation Canals, by the Bureau of Reclamation; and Design, by Elwin E. Seelye.

In examining the sources and recognizing that the water that will be entering the channel will not be clear water but will be transporting silts, sands, gravel and probably even rock fragments, the following maximum velocities were determined. For the earth-lined portions of the canal, assuming a graded mixture of silts to cobbles as indicated by the soil tests a maximum velocity of five feet per second is allowable. For a concrete non-reinforced lining, a maximum velocity of

8 ft. per second is allowable. A velocity greater than that can lead to the following effect:

> "Hard-surface linings permit higher water velocities than do earth sections. Usually, these velocities in unreinforced lining should be less than 8 feet per second to avoid the possibility of lifting the lining should the velocity head be converted to pressure head through a crack that slopes upstream, or into the current." (Linings for Irrigation Canals, by the U. S. Department of the Interior, Bureau of Reclamation.)

Reinforced concrete may have a velocity as high as 40 ft. per second; however, if this does occur a provision must be made to slow the water down before it enters a section with less protection.

Natural River Channel Velocities

In the letter dated November 8, 1978, by Mr. Jason Burgess of HDR to Mr. Les Bond, Chief Hydrologist for the Maricopa County Flood Control District, there is an indication that prior to the construction of the new channel, during the 100-year, 2-hour flood, the old natural drainage channel was capable of handling velocities of between 7 and 12 ft. per second. While these computations have not been independently checked, we seriously doubt the validity of this assumption.

We believe the natural channel could never have contained flows of this magnitude and velocity. The old natural channel has never experienced the flood flows that are being projected. The reason for this is that the wash in its natural state only carried a very minor portion of the waters that are currently being intended for it. The construction of the Trilby Detention Basin and the McMicken Dam outlet have added many hundreds of square miles of drainage area and undoubtedly multiply the flood flows many times over. Hence the computation of high flood flow velocities through this channel are not surprising, but are not indicative of the ability of the ground to withstand velocities of that magnitude. In fact, it is our expectation that if a major flood does occur in the channel upstream from Sun City West that upstream channel will erode severly and possibly change its position.

DESIGNED VELOCITIES

Based upon an examination of the correspondence between Maricopa County Flood Control District and Henningson, Durham and Richardson, Inc. of Arizona, it was found that the basic agreement for the design of the channel was as follows:

"1. The channel will be lined at all areas where velocities are supercritical.

2. When plain soil is encountered upon excavation, the west bank will have properly designed bank protection for velocities between 7 fps. and 12 fps. For velocities greater than 12 fps the west

bank will have to be lined.

3. Except in areas where velocities are supercritical, no bank protection or channel lining will be required when the channel is in cemented conglomerate material." (Letter from Nick Karan, Chief, Engineering Division, Maricopa County Flood Control District, to Mr. Jason Burgess, Henningson, Durham & Richardson, Inc., January 31, 1979.)

The basis for this agreement, according to the correspondence, was that the natural channel was going to be handling velocities of 12 ft. per second; and thus, it would be only necessary to protect against flows higher than 12 ft. per second. There was also discussion that most of the material involved would be "cemented conglomerate." When cemented conglomerate is tightly cemented, it has many of the characteristics of concrete and, hence, is highly resistent to erosion. The soil boring tests made the following comments concerning cementation: Test #1 - no cementation mentioned; Test #2 - light cementation; Test #3 - light cementation; Test #4 - light to moderate cementation; Test #5 - traces of cementation nodules; Test #6 - traces of cementation nodules.

It is apparent from the soil borings and our visual inspection of the sites that the cementation is minimal and cannot be expected to have any significant retardation of the

projected erosion. In fact, during the inspection of Feb. 18, 1980, during the relatively light rainfall where some water was spilling down the side, it was evident from the relatively small trickles which had occurred that significant erosion had already begun.

At the outflow from Sun City West, where the culvert passes under El Mirage Road, the entirety of the canal bank had been eroded away by the water entering from that source. These erosive factors are small forewarnings of what may happen when a high velocity flood goes through the entirety of the channel. If the 100-year flood does occur and the anticipated velocities result, then the flood control channel would be seriously damaged and probably would allow the wash to erode into adjoining areas, causing damage to any development that might exist nearby.

Predicted Velocities

Based upon the canal configuration shown in the specifications provided to us, and employing Mannings Equation computations of the velocities that will occur applying a roughness coefficient of .025 as recommended by the Army Corps of Engineers, the following velocities were computed:

For a 100-year flood as determined by this office (13,000 cfs), the peak velocity would be 11.4 ft. per second through the unlined sections. For a period of approximately 9 hours, the velocity would hold at 11 feet per second (12,000

cfs) and then decrease to 7.4 feet per second which would continue for several days.

For a 2 year storm a peak flow of 5,500 cfs was estimated giving a velocity of 8.30 feet per second. As can be seen from these data, the velocities of the water going through the unlined portions of the ditch are such that extensive maintenance will be required to handle the damage caused by the more frequent floods and during a major flood, it is very possible for the structure to fail altogether.

As described in the section concerning flood flows, the 100-year storm is a minimal flood protection when a residential area is concerned. In addition, the amounts of flood flow computed will increase as upstream development occurs on the watershed. These two factors could lead to even greater damage, since the even higher flows would reach velocities that would escalate the scouring to an even greater degree.

IMPACT ON STATE LANDS

Channel Instability

The major impact on the State Lands from the location and construction of El Mirage Channel is the lack of adequate protection against the erosive forces of the water that will be traversing it.

The wash under the natural conditions was relatively small and would have yielded minimal amounts of water. The construction of the Trilby Wash Detention Basin added the regulated flows of a large area north and west of this Wash. The construction of the McMicken Dam outlet intercepted numerous washes and conveyed their flows along with those from the Trilby Wash Detention Basin into this small wash.

If a large flood had occurred under these relatively new conditions, the channel would have eroded considerably and broken free of its banks to create a new channel capable of handling larger flows. Based upon an examination of the topography under natural conditions, we are convinced that the flood flows which would have broken free of the channel would have inundated large portions of the then lower lands contained in Sections 23 and 26. This break would probably have occurred at the meander which historically existed in Section 23.

Since the construction of the El Mirage Channel and the accompanying subdivision, the lands in Section 23 and 26 have been raised to a point where the waters will encroach upon the State lands in Section 24 and particularly in Section 25 before inundating Sections 23 and 26.

As indicated in the discussion on erosion both from side drainage on the east bank and from the large prolonged flows which will occur in the channel during major storms, the channel as constructed would be unstable and will begin to meander into the adjacent Sections. Due to the additional elevation on the west bank, it is expected that a large part of this meandering would occur on the State lands in Sections 24 and 25.

Flooding

Since the construction of Sun City West, the west bank of the channel has, in all but a few places, a depth of nine feet or more. On the east bank where the State Land Department's lands are located the Paving and Grading Plans of HDR seem to indicate the land surface is between eight and nine feet above the El Mirage Channel. There is a 500-foot stretch in the southern portion where the depth of the east bank drops to 7-1/2 feet. Near the outlet portion on the east side of the southern end of the El Mirage Channel the depths vary from a high of 5-1/2 feet to a low of 3-1/2 feet. The outlet itself, of course, has no elevation above the bottom of the channel. Given the expected depth of flow for the 100-year flood of 7+ feet, the amount of free board or safety margin that will exist on the west bank will be approximately 1-1/2 feet in most places. On the east bank, it will vary considerably but will drop down to less than six inches; and, in the outflow, the channel will be overtopped.

It should be recognized that the 13,000 cubic feet per second, 100-year flood is a minimum standard for flood control in a heavily developed area. A more common standard in dealing with fairly sizeable flood flows, as in this case, is to use the Standard Project Flood.

A Standard Project Flood does not have a specific flood frequency and would require a significant amount of study to determine. However, the Standard Project Flood

is, as a rule of thumb, on the order of magnitude of a 300 to 500-year flood.

The flows that would occur from the Standard Project Flood would be of such volumes that the channel would be overtopped in many locations, virtually all of them being on the east bank. The choice of whether to use a 100-year flood or a Standard Project Flood is primarily an economic choice, taking into consideration the amount of development that would be affected should the flood control structure fail. Under current development conditions, Sun City West would suffer some minor adverse effects and the relatively undeveloped State lands that would be badly inundated. If the State lands are to be developed for anything besides recreation uses, the problem of flooding would have to be seriously examined.

Although a detailed study would be required to develop a plan and assure its viability a cursory examination indicates that since the State lands east of the El Mirage Channel are approximately 40 feet above the Agua Fria River these lands could be developed into extremely valuable residential and/or commercial sites. As the situation now stands, these lands have little value except as possible locations for recreational facilities of the type which infrequent inundation would not severely damage.

Ponding Water

As discussed in the observations of both inspection trips, the channel as designed will, after any flow, create stagnate ponds of water throughout the channel. These ponds could lead to significant health hazards

If the Specifications had been followed, these ponds would not exist. However, the outlet from the energy dissipator was apparently built too shallow and the lead edge of the energy dissipator was apparently built above ground level. In both cases a rigid adherence to the design Specifications would eliminate the problem.

Downstream Damage

No protection has been given against the impact of the waters being released from the channel onto the State lands in transit to the Agua Fria River. Due to the large velocities involved, it is reasonable to expect that the south bank of the turn will be subject to severe erosion and that the channel will attempt to straighten itselt and will impinge upon the State lands on the western side of Section 25. Heavy reinforcement and lining will be required on the southern bend to insure the integrity of the channel at that point.

ENGINEERING SOLUTIONS

General

Consideration has been given to various solutions to the problems created by the channel excavated on the east side of Sun City West.

Basic to all of the engineering solutions considered is criteria that any channel design must be capable of conveying a peak flow of not less than 13,000 cfs and must be capable of conveying flows well in excess of 10,000 cfs for periods of longer than 10 hours.

One solution considered was to flatten the slope or grade of the channel and thereby reduce the velocity of the water in the El Mirage Channel to the extent necessary to prevent erosion.

Another solution would be to line the canal using alternative designs to prevent erosion.

A third possibility is to rip-rap the canal, thereby protecting against erosion and increasing the friction losses to reduce the velocity of the flow.

An additional alternative would be some combination of the foregoing solutions.

Any method which involves paving of the channel would necessitate complicated and careful design. Essential to such a design is an analysis of the static and dynamic loads which the paving would be required to withstand. Paving must have a substantial foundation or base. Normally this requires careful compaction and/or ABC

as well as reinforcing of the concrete itself. All paving alternatives would be extremely expensive.

In any program lining it is necessary to provide that the lining extend above the designed water depth in order to allow some freeboard to accomodate waves and other unforeseen occurrences. For example, if the water depth is determined to be slightly in excess of 9 feet, the depth of lining should be slightly in excess of 10 feet.

In appraising the practicality of such a program it is necessary to examine the plans and profiles submitted by HDR to make sure that the east edge of the roadway is sufficiently above the drainage channel flow line to prevent overflow. For example, on Sheet #26 of 29 at Station 37 and 25, the drainage channel flow line is at elevation 1232.5. The profile grade of El Mirage Road is at elevation 1240 which is to say 7-1/2 feet above the drainage channel flow line. Obviously, if a water depth of 9.2 feet were determined to be appropriate and a 1 foot free board adopted, it would be necessary to lower the flow line or raise the grade of El Mirage Road.

In addition, the contours on the west side of the channel would have to be considered in order to insure that the channel would be sufficiently deep to prevent overflow onto the State lands which lie to the West of the excavated channel. A preliminary examination of these contours indicated that such is not the case in all reaches of the canal. For example, a typical channel section shown on

4A of 29 shows in every case where a dimension is given, that the west bank is lower than 10 feet. Under certain of the engineering solutions described in the following paragraphs, this depth would be inadequate. On several of the drawings, it is indicated that the east bank is lower than the west bank. Careful study and design would be prerequisite to any engineering solution of the problem.

A major problem with flattening the grade of the drainage channel is that as the grade decreases, the depth of water for any given discharge in the channel necessarily increases. If the velocity of the water is decreased so as to prevent erosion, the cross sectional area of the water will have to be increased to permit carriage of a fixed amount of water. In computing the capacity of a channel the discharge per unit of time is equal to the cross sectional area of the channel times the velocity of the water. Reducing the velocities to non-erosive speeds without lining the channel would necessitate increasing the depths of water in the channel to more than 11 feet. This would require extensive lowering of the channel bottom or inverts in some portions of the channel and raising banks in other portions. In some portions both lowering the invert and raising the banks would be necessary.

Any type of lining must be so constructed that water entering the canal from the side slopes will not flow underneath the lining. The Paving and Grading Plans of HDR illustrate this type of construciton in Detail A on Sheet 3 of 29 and Detail X on Sheet 4 of 29. These details provide

that at the top of the concrete lining be extended horizontally for 12 inches away from and at right angles to the channel. At the end of the 12 inch horizontal extension there is an eight inch vertical concrete extension downward into the embankment. This protection should also be provided for rip-rap.

In any reaches of the channel where there is no lining or where the embankment extends above the top of the lining or rip-rap, collector ditches should be constructed and periodic chutes provided to convey the water into the channel. These are essential to prevent erosion of the unprotected side slopes.

One modification which should be implemented along with any design is a fence along both sides. The velocities that are being conveyed under any of the designs would be dangerous to any person who might enter them. Additionally, escape ladders should be provided to give the victim the best change of getting out safely.

Flattening the Unlined Channel

In the absence of lining, it would be necessary to flatten the slope of the El Mirage Channel to reduce the velocities to not exceed five feet per second in order to prevent serious and excessive erosion. To accomplish this, it would be necessary to reduce the slope of the channel invert to less than 0.0005. To better understand what this means, that would represent a slope of approximately 2-1/2 feet per mile.

As shown on the Paving and Grading Plans of HDR, the invert elevations of the drainage channel as it currently exists are approximately 0.004 or slightly more than 21ft. per mile. To flatten the grade of the existing channel without some type of lining so that it would function without excessive erosion would be impractical.

Paving the Channels Chute Construction

An alternative solution would be to line the entire channel utilizing the existing slope of 0.004 throughout the entire length of the existing channel, except where the existing energy dissipator has been constructed. The existing energy dissipator would be incorporated into the lining, with some modification.

This would result in a water velocity of in excess of 19 feet per second and a water depth of approximately 3.6 feet. Water throughout the entire channel would be flowing at what is termed super-critical velocity. Critical velocity is defined as that point at which the flow changes from a tranquil flow rate state to a rapid or shooting flow rate state. The critical velocity is that point at which the inertial forces of the water equal the gravitational forces on the water. When the inertial force exceeds the gravitational force, the water decreases its depth and increases its velocity dramatically. This is super-critical flow.

There are numerous problems which occur when water is flowing at super-critical velocities. Such water is extremely turbulent; and, if it contains any sand or silt, it is highly erosive. Super-critical flows have a tendency to develop standing waves, which can cause the water surface to raise very significantly at unpredictable points in the channel. Another problem with super-critical velocities is that when water passes from super-critical to sub-critical velocity, a phenomena known as a hydraulic jump occurs.

The hydraulic jump may occur in a shooting channel wherever there is an obstruction or a rapid change in cross sectional areas. When the jump occurs, the flow changes from shooting flow to a tranquil flow having a greater depth. In addition, super-critical flow presents particular problems when the direction of flow is changed, such as would be the case on the lower end of the existing channel. Extreme precaution and measures would have to be taken to induce the super-critical flows to turn and head toward the Agua Fria River. Moreover, flows in excess of 19 feet per second would be extremely hazardous in the event that any person or animal were to fall into this flow and would probably prove fatal. As a practical matter, we would not recommend a lined channel throughout the length of the existing channel.

gy Dissipators and/or Drop Structures

Other alternatives would be to provide a series of nergy dissipators and/or drop structures combined with a lined section to reduce the flow velocities to sub-critical speeds. The lining would prevent erosion and the flatter grades would prevent super-critical flow. In effect, this would be a stair-stepping device.

If the slope of the channel between the energy dissipators and/or drop structures were reduced to 0.0008, the depth of the flow would be reduced to approximately 5.3 feet, with a discharge of 13,000 cfs and a velocity of 11½ feet per second. Although this is a very rapid velocity, it could be maintained in sub-critical flow conditions and provide adequate capacity.

Where such dissipators and/or drop structures are installed, there should be provision made to drain the standing water out of the dissipator during times of no flow. Such provisions should be made in the existing dissipator in any event.

Rip-Rap

One additional alternative would be to rip-rap the sides and bottom of the canal. The rip-rapping would provide a rougher surface for the canal and, consequently, a greater friction loss. It is estimated that, if the channel were rip-rapped with 6-inch cobbles held in place by wire mesh and gunite, the velocity could be reduced to 9.2 feet per second and the water to approximately 7 feet. This seems to be the most practical solution.

A P P E N D I X

A

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County Maricopa
Location North of El Mirage Channel - Area "A"
Project No. 890 Station _____
Name of Stream El Mirage

DESIGN DATA:

Design Frequency	100	years
Drainage Area	34.28	square miles
Drainage Length	31,680	feet
Elevation		
Top of Drainage area	1,540	feet
At Structure	1,257	feet
Drainage Area Slope		%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	20	%
Soil Group	50% B - 50% C	
Precipitation		
P = 6 hour =	3.3	inches
P = 24 hour =	4.1	inches

DESIGN COMPUTATION:

Curve Number	86	
Time of Concentration	2.28	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	8967	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	2.45	84.4	2.07	.90	7994
2 hour	2.67	87.7	2.34	1.10	7707
3 hour	2.85	92.3	2.63	1.55	8967
6 hour	3.30	94.3	3.11	1.77	6723

Computed by: _____ Date _____

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County Maricopa
Location North of El Mirage Channel - Area "B"
Project No. 890 Station _____
Name of Stream El Mirage

DESIGN DATA:

Design Frequency	100	years
Drainage Area	56.25	square miles
Drainage Length	84,480	feet
Elevation		
Top of Drainage area	4,200	feet
At Structure	1,540	feet
Drainage Area Slope		%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	20	%
Soil Group	75% D - 25% B	
Precipitation		
P = 6 hour =	3.3	inches
P = 24 hour =	4.1	inches

DESIGN COMPUTATION:

Curve Number	89	
Time of Concentration	3.00	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	12,622	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	2.45	78.5	1.92	.99	11719
2 hour	2.67	83.5	2.23	1.25	12154
3 hour	2.85	89.0	2.54	1.53	12622
6 hour	3.30	92.0	3.04	1.96	11117

Computed by: _____ Date _____

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County Maricopa
Location Trilby Wash - Area C
Project No. 890 Station _____
Name of Stream Trilby Wash

DESIGN DATA:

Design Frequency	100	years
Drainage Area	91.37	square miles
Drainage Length	52,800	feet
Elevation		
Top of Drainage area	1,800	feet
At Structure	1,350	feet
Drainage Area Slope		%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	20	%
Soil Group	50% B - 25% D - 25% C	
Precipitation		
P = 6 hour =	3.3	inches
P = 24 hour =	4.1	inches

DESIGN COMPUTATION:

Curve Number	86	
Time of Concentration	3.44	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	14518	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	2.45	72.8	1.78	.69	11901
2 hour	2.67	79.0	2.11	.92	13278
3 hour	2.85	85.0	2.42	1.17	14518
6 hour	3.30	89.3	2.95	1.60	13973

Computed by: _____ Date _____

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
SCS METHOD: PART II

LOCATION DATA:

Highway _____ County Maricopa
Location Trilby Wash - Area D
Project No. 890 Station _____
Name of Stream Trilby Wash

DESIGN DATA:

Design Frequency	100	years
Drainage Area	137.93	square miles
Drainage Length	97,680	feet
Elevation		
Top of Drainage area	4,200	feet
At Structure	1,500	feet
Drainage Area Slope		%
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	20	%
Soil Group	90% B - 10% D	
Precipitation		
P = 6 hour =	3.3	inches
P = 24 hour =	4.1	inches

DESIGN COMPUTATION:

Curve Number	84	
Time of Concentration	3.53	hours
Peak Design Discharge		
$Qp = \frac{484AQ}{\frac{D}{2} + .6Tc} =$	18452	cfs

Storm Duration	Point Precipitation	Areal Reduction	Areal Precipitation	Direct Runoff	Peak Discharge
1 hour	2.45	69.2	1.70	.53	13,515
2 hour	2.67	75.6	2.02	.73	15,630
3 hour	2.85	82.0	2.34	1.00	18,452
6 hour	3.30	87.0	2.87	1.40	18,261

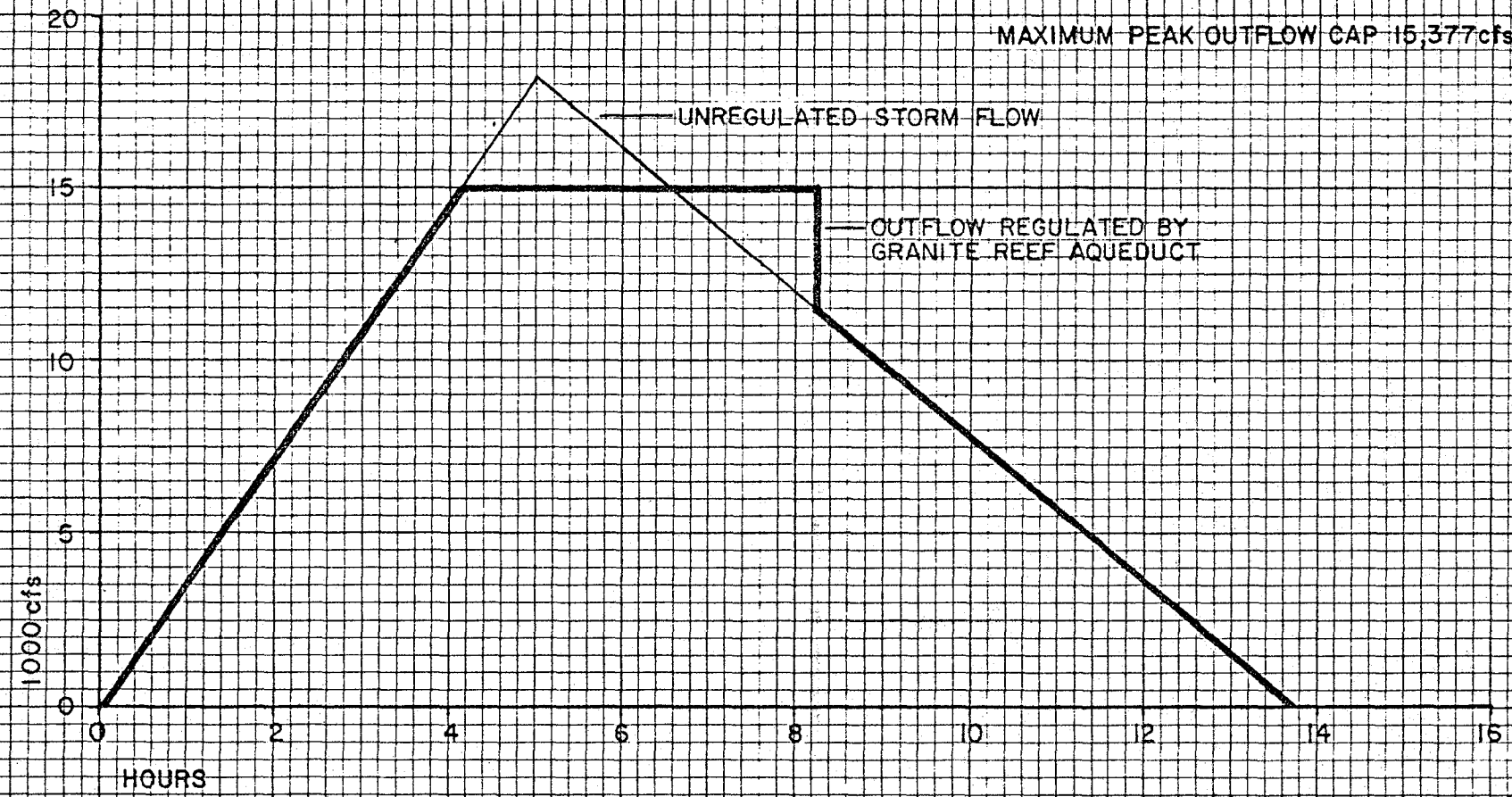
Computed by: _____ Date _____

GRAPH NO. 1

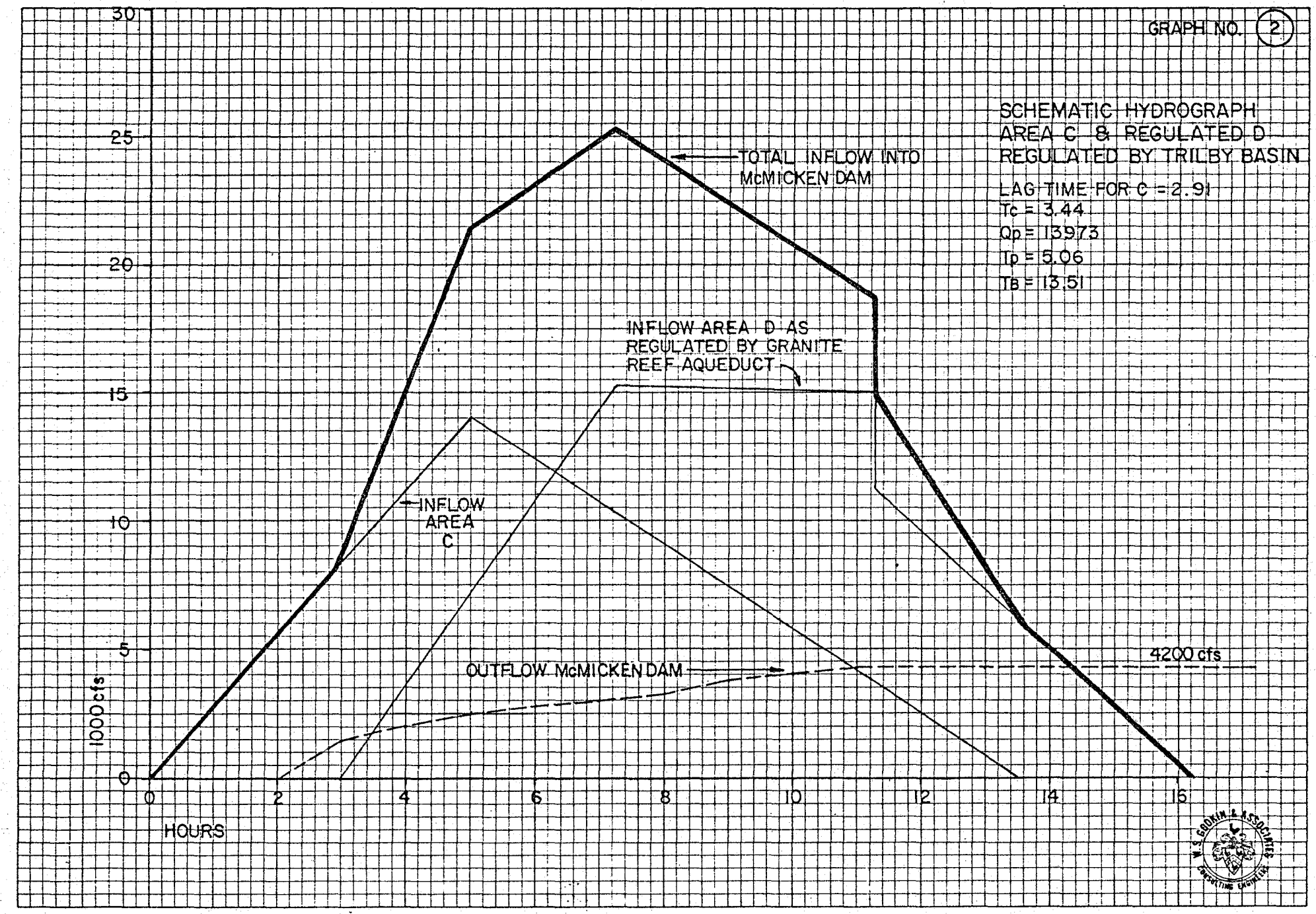
SCHEMATIC HYDROGRAPH
AREA D
REGULATED BY CAP CANAL

Tc = 3.53
Qp = 18261 cfs
Tp = 5.12
TB = 13.67

MAXIMUM PEAK OUTFLOW CAP 15,377 cfs



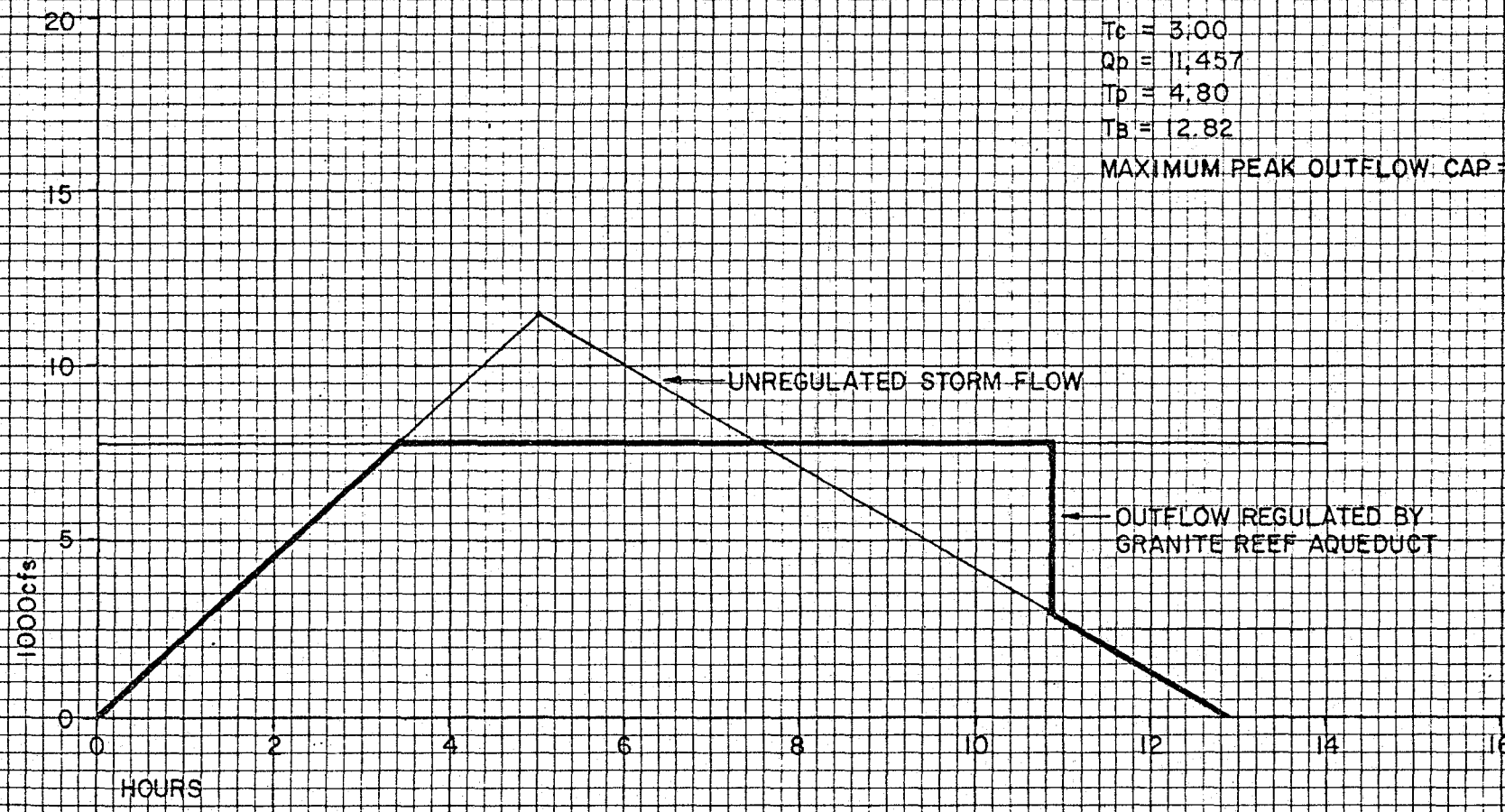
W. S. GODWIN & ASSOCIATES
CONSULTING ENGINEERS


GRAPH NO. 2
SCHEMATIC HYDROGRAPH
AREA C & REGULATED D
REGULATED BY TRILBY BASIN
LAG TIME FOR C = 2.91
Tc = 3.44
Qp = 13973
Tp = 5.06
TB = 13.51
TOTAL INFLOW INTO McMICKEN DAM
INFLOW AREA D AS REGULATED BY GRANITE REEF AQUEDUCT
INFLOW AREA C
OUTFLOW McMICKEN DAM
4200 cfs
1000 cfs
HOURS
0
5
10
15
20
25
30
0
2
4
6
8
10
12
14
16
W. S. GODKIN & ASSOCIATES
CONSULTING ENGINEERS

GRAPH NO. 3

SCHEMATIC HYDROGRAPH
AREA B
REGULATED BY CAP CANAL

T_c = 3.00
Q_p = 11,457
T_p = 4.80
T_B = 12.82
MAXIMUM PEAK OUTFLOW CAP = 7839

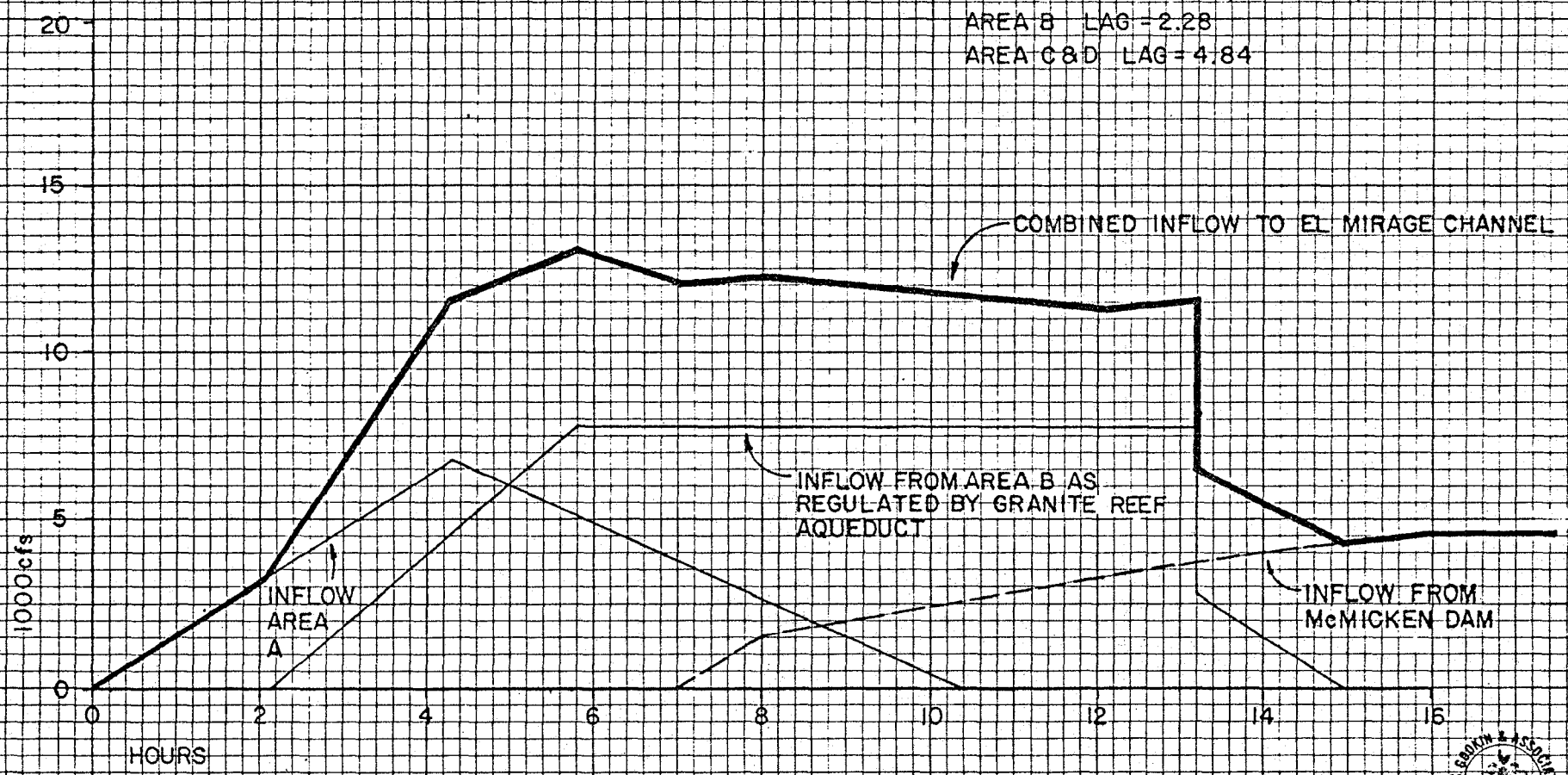


10 X 10 PER INCH

GRAPH NO. 4

SCHEMATIC HYDROGRAPH
EL MIRAGE FLOWS

AREA A $T_c = 2.28$ $Q_p = 6723$ $T_p = 4.37$ $T_B = 11.67$
AREA B LAG = 2.28
AREA C & D LAG = 4.84



W. S. SHOKIN & ASSOCIATES CONSULTING ENGINEERS