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TRACIE K. LINDEMAN
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IN THE SUPREME COURT OF THE STATE OF NEVADA

JOHN AND MELISSA FRITZ,

Plaintiff-Appellants,

CASE NO. 67660

vs.

WASHOE COUNTY,

Defendant-Respondent,
______________________________/

JOINT APPENDIX

Volume 2

Appellants John and Melissa Fritz and Respondent Washoe County, by and through the undersigned counsel, respectfully submit Volume 2 of the Joint Appendix to the briefs for the above captioned proceeding.

1. Opposition to Motion for Summary Judgment Exhibit 9: Bates No. 228-277
2. Opposition to Motion for Summary Judgment Exhibit 10: Bates No. 278-285
3. Opposition to Motion for Summary Judgment Exhibit 11: Bates No. 286-293
4. Opposition to Motion for Summary Judgment Exhibit 12: Bates No. 294-316
5. Opposition to Motion for Summary Judgment Exhibit 13: Bates No. 317-343
6. Opposition to Motion for Summary Judgment Exhibit 14: Bates No. 344-365
7. Opposition to Motion for Summary Judgment Exhibit 15: Bates No. 366-389
8. Opposition to Motion for Summary Judgment Exhibit 16: Bates No. 390-411
9. Opposition to Motion for Summary Judgment Exhibit 17: Bates No. 412-432

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15-20123

10. Opposition to Motion for Summary Judgment Exhibit 18: Bates No. 433-454

Respectfully submitted this Monday, June 29, 2015.

By: /s/ Luke Busby
Luke Busby, Esq.
216 East Liberty St.
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By: /s/ Michael Large
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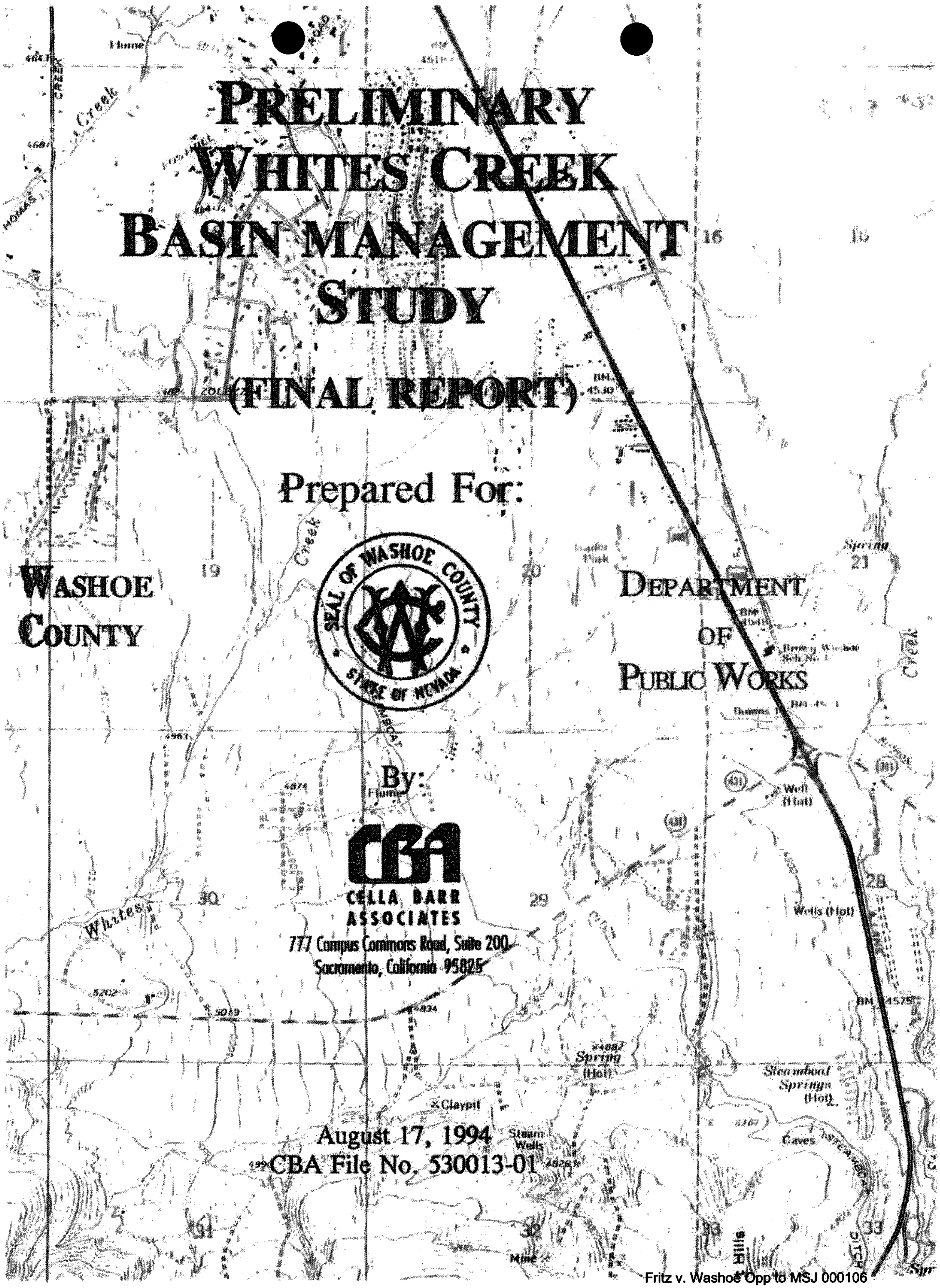
Exhibit 9

Exhibit 9



PRELIMINARY WHITES CREEK BASIN MANAGEMENT STUDY

(FINAL REPORT)

Prepared For:

WASHOE COUNTY

DEPARTMENT OF PUBLIC WORKS

By:

CBA
CELLA BARR ASSOCIATES
777 Campus Commons Road, Suite 200
Sacramento, California 95825

August 17, 1994
CBA File No. 530013-01

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August 17, 1994
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PROFESSIONAL ENGINEER-STATE OF NEVADA
JAMES H. NELSON
CIVIL
No. 8810

James H Nelson
8-17-94

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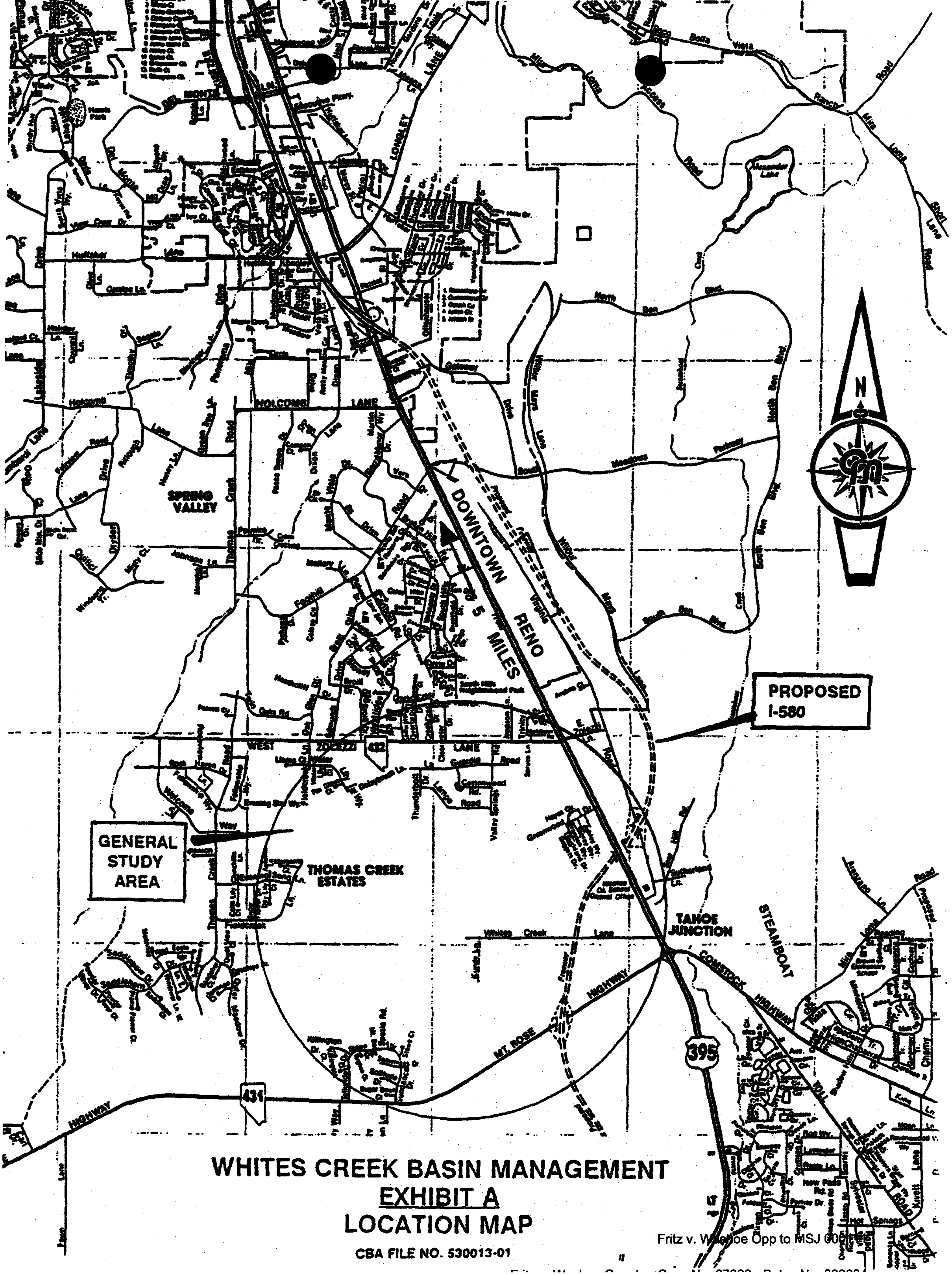
LIST OF EXHIBITS

INTRODUCTION

This document is a Preliminary Basin Management Study performed for the lower Whites Creek watershed located approximately five (5) miles south of downtown Reno, Nevada (see Exhibit A, Location Map). This Preliminary Basin Management Study has been formulated in response to active new development and infrastructure construction occurring within the area and the existence of a unique set of flood hazards. Conclusions and recommendations provided herein have been based upon a review of available information, discussions with several key individuals, workshops, field reconnaissance and cursory calculations.

The purpose of this Preliminary Basin Management Study is to derive a unified set of conclusions with respect to existing flood hazards and develop interim policies for new development and infrastructure improvements within the watershed. Conceptual flood control measures are also recommended, as appropriate.

Much of the information presented herein is envisioned to be subsequently enhanced and supplemented by more detailed studies, which will undoubtedly serve to revise some of its conclusions and recommendations. Until such studies are performed or until other factors impact the information presented in this document, the interim policies shall be utilized for regulating the drainage design of new development and infrastructure projects.



WHITES CREEK BASIN MANAGEMENT
EXHIBIT A
LOCATION MAP

CBA FILE NO. 530013-01

I. DATA COLLECTION AND RECONNAISSANCE

A. Literature Review

In accordance with the Whites Creek Basin Management Scope of Work, the studies, reports and plans listed below were reviewed. Following each listing is a brief and general description of the pertinent information contained therein.

- **Regional Water Study: Concept Level Report - Washoe County Flood Control Master Plan, Volumes I and II; prepared by Kennedy/Jenks/Chilton in association with Kato & Warren, Inc. and FCS Group, Inc.; January, 1991.**
    - Conceptual level flood control master plan for Washoe County intended to provide an estimate of the overall program costs, establish the general level of long-term capital needed, and develop a recommended institutional structure and funding plan.

    - Existing hydrologic data were used to develop a regional relationship between watershed area, average stream slope, 100-year rainfall depth, and 100-year peak discharge, resulting in a 100-year peak discharge of 3100 cfs for the Whites Creek watershed. Flood control improvements identified include a detention site on Whites Creek at the location where Whites Creek divides into four (4) distinct channels, and replacement of existing structures with improved culverts at Thunderbolt Street, La Guardia Road, Zolezzi Lane, U.S. 395 and Old Virginia Road for a total cost of $345,000.

- **I-580 Concept Drainage Study prepared for the Nevada Department of Transportation (NDOT); Plans for I-580 north of Highway 341.**
    - CBA has had several discussions with the Hydraulics Division of NDOT regarding the status of drainage structure design for I-580 along the base of the Whites Creek watershed and has reviewed current Plans for I-580. At this time the drainage design has not been finalized; however, it is proposed that several structures will be provided beneath I-580 to pass the projected 100-year flows resulting from splitting the total 100-year flow amongst the four (4) branches of Whites Creek.

- **Feasibility Study for Huffaker Detention Facility near the City of Reno, Washoe County, Nevada; prepared for Washoe County Public Works in cooperation with City of Reno Engineering by Nimbus Engineers; February, 1990.**
    - Examination of the feasibility of constructing a detention dam at the Huffaker Narrows, upstream of the proposed Mira Loma crossing of Steamboat Creek. A study of alternatives, resulting in the proposed detention site, was originally undertaken to provide all-weather access to the Truckee Meadows area east of Reno, including the Hidden Valley area. The analysis included development of detailed hydrology for the

109-square-mile Steamboat Creek watershed, which includes Whites Creek. The study states that the majority of flow from Whites Creek occurs as sheet flow across meadow or pasture land, with velocities ranging from one (1) to three (3) feet per second.

- **Whites Creek Detention Facility Feasibility Study, Washoe County, Nevada; prepared for the Nevada Department of Transportation by Nimbus Engineers; revised June, 1993.**
  - Evaluation of the benefits of a detention basin on Whites Creek at the existing major flow split at Shadowridge Park, including detailed development of a 100-year peak discharge and runoff hydrograph using the Corps of Engineers' hydrologic computer model, HEC-1.

    The resulting 100-year peak discharge of 5100 cfs at the flow split was distributed amongst the four downstream branches of Whites Creek based on a ratio of available conveyance. This ratio, in turn, was based on cross-sectional channel geometries, slopes, and resulting water surface elevations derived from the Corps of Engineers water surface program, HEC-2. One-hundred year peak discharges divided among the four branches were estimated as follows:

    Channel #1: 700 cfs (14%)
    Channel #2: 1950 cfs (38%)
    Channel #3: 1100 cfs (22%)
    Channel #4: 1350 Cfs (26%)

- **Hydrologic Analysis of Thomas Creek, Dry Creek and Evans Creek, Washoe County, Nevada; prepared for the Federal Emergency Management Agency by Nimbus Engineers; August, 1990.**
  - Evaluation of existing hydrology studies and development of rainfall-runoff models for Thomas Creek, Dry Creek and Evans Creek. The discharges resulting from these models were recommended for use in a Flood Insurance Restudy for Thomas Creek, Dry Creek, and Evans Creek in Washoe County and the City of Reno, instead of discharges previously developed by FEMA and the Corps of Engineers.

- **Thomas Creek Detention Basin Study; prepared for the Technical Advisory Committee, Washoe County Regional Flood Control Master Plan by Kennedy/Jenks/Chilton; May, 1990.**
  - Development of specific hydrologic modeling for the Thomas Creek drainage basin and analysis of several stormwater detention/debris basin sites within the watershed for the Washoe County Regional Flood Control Master Plan. The purpose of this study was threefold: 1) to determine whether detention could be utilized in the watershed to reduce the sizes of planned drainage conveyance structures for U.S. 395 and I-580; 2) to analyze the potential for reclassifying the FEMA-based designation of the Thomas Creek Watershed as an alluvial fan; and 3) to prepare preliminary

design parameters for the detention dam/debris basin and channel improvements.

- **Flood Insurance Study for Washoe County, Nevada Unincorporated Areas; prepared by the Federal Emergency Management Agency (FEMA); revised April 16, 1990.**
  - This Flood Insurance Study (FIS) establishes peak discharges, water surface elevations, and floodplain and floodway limits for portions of the Truckee River, Steamboat Creek, Bailey Canyon Creek, Boynton Slough, North Truckee Drain, Dry Creek, and the four playas in Lemmon Valley. The FEMA alluvial fan methodology was used to study Galena Creek, Thomas Creek and Evans Creek. Approximate methods were utilized to study flooding caused by several creeks along the northern shore of Lake Tahoe and to study those areas having a low development potential or minimal flood hazards. The resulting Flood Insurance Rate Maps are used to set local flood insurance rates and to guide land development with respect to flood hazards. In this study, the peak discharge - frequency relationships for Steamboat Creek and tributaries were determined from regional analyses based on 18 moderate-sized, natural drainage basins in the Truckee River and Carson River basins.

- **Washoe County Flood Control Master Plan - Draft Final Report on Meteorological Analysis; prepared for Kennedy/Jenks Consultants by Henz Meteorological Services; September 29, 1993.**
  - A detailed meteorologic analysis whose purpose was to provide a 100-year precipitation event for Washoe County to use in HEC-1 rainfall-runoff modeling. A review of the study has been performed by HYDMET, Inc. and states that it actually provides the following: 1) Annual and seasonal depth-duration-frequency (DDF) precipitation maps and intensity-duration-frequency analyses; 2) Areal Reduction Factors for 100-year summer thunderstorm events; and 3) Orographic and temporal variations in rain/snow line and snowpack for 100-year winter rain-on-snow events. Values represented are higher than depicted on current NOAA atlases. The study has not been accepted by Washoe County at present.

- **Flood Plain Information - Southwest Foothills Streams (Evans, Thomas, and Whites Creeks & Skyline Wash), Reno, Nevada; prepared for the Regional Planning Commission of Reno, Sparks and Washoe County by the Department of the Army, Sacramento District Corps of Engineers; June, 1974.**
  - Information on past floods, and maps, profiles, and cross sections that indicate the approximate extent and depth of inundation of Evans, Dry, Thomas and Whites Creeks and Skyline Wash from the Intermediate Regional and Standard Project Floods.

  - Intermediate Regional Flood values (equivalent to the 100-year discharge) for Whites Creek, developed by the Corps of Engineers from available streamflow and precipitation records and synthesized from records of other similar watersheds, are as follows:

At Canyon Mouth:	3,000 cfs
At Divide (mile 4.99):	2,000 cfs
At Highway 395:	2,300 cfs

- **Water and Related Land Resources - Central Lahontan Basin, Truckee River Subbasin, Nevada...California: Flood Chronology, 1861-1976; based on a Cooperative Survey by the Nevada Department of Conservation and Natural Resources, the Resources Agency of California, and the United States Department of Agriculture; September, 1977.**
  - Presentation of a flood history of the Truckee River Subbasin of the Central Lahontan Basin, 1861-1976. This history is based on research of newspaper files and other historical archives and is concerned with three types of flood phenomena that have inflicted flooding and flood damage through the years of record: wet-mantle and rain-on-snow or frozen-ground events characteristic of late winter or early spring, and the dry-mantle event typical of localized summer thunderstorms.

- **Truckee River, California and Nevada - Hydrology; Office Report prepared by the Department of the Army, Sacramento District, Corps of Engineers; February, 1980.**
  - Presentation of basic hydrologic data and criteria for the Truckee River Basin for use in flood protection feasibility studies for the Truckee Meadows area near Reno, Nevada. The hydrologic characteristics of the basin are discussed, followed by analysis of flow frequencies and development of the Standard Project and Probable Maximum Floods resulting from winter type rain storms and summer-fall type cloudbursts. The peak flow for Whites Creek at Steamboat Ditch resulting from a Cloudburst Standard Project Flood, was estimated to be 8,700 cfs.

- **Flood Plain Information, Truckee River - Reno-Sparks-Truckee Meadows, Nevada; prepared for the Regional Planning Commission of Reno, Sparks, and Washoe County by the Department of the Army, Sacramento District, Corps of Engineers; October, 1970.**
  - Presentation of information on past floods, and maps, profiles and cross sections that indicate the depth and extent of flooding resulting from the Intermediate Regional and Standard Project Floods along the floodplains of the Truckee River; Steamboat Creek and its tributaries; Alum, Hunter, and Peavine Creeks; and the North Truckee Drain. The area covered extends northward from Huffaker Hills.

- **Flood Plain Information, Steamboat Creek and Tributaries, Steamboat & Pleasant Valleys, Nevada; prepared for the Regional Planning Commission of Reno, Sparks and Washoe County by the Department of the Army, Sacramento District Corps of Engineers; June, 1972.**
  - This report presents information on existing flood hazards along Steamboat Creek and tributary streams in Pleasant and Steamboat Valleys, including the portion of Steamboat Creek that drains Whites Creek and immediately downstream, and the Upper Truckee Meadows area of Washoe County, Nevada. The flood hazard maps produced are those resulting from the Intermediate Regional and Standard Project Floods.

- **Draft Development Standards and Design Guidelines; prepared for the Washoe County Department of Comprehensive Planning; July 6, 1993.**
  - Presentation of draft development standards and design guidelines for Washoe County, including Article 420, Storm Drainage Standards. This article provides general requirements regarding 10-year and 100-year storm runoff improvements; detention requirements; required drainage report contents for land development projects; and design requirements for different types of storm drainage systems. Emergency access roadway design requirements are contained in Article 408, Street Design Standards.

- **Flooding in Douglas County - Making Tough Choices (A Guide for Public Policy Dialogue); prepared by the Citizens Task Force on Flood Control.**
  - A publication written to serve as an educational guide for residents of Douglas County. Its purpose is to educate citizens about hazards from alluvial fan and riverine flooding; to pose alternative policy directions for citizens to consider and debate; and to serve as a basis for gathering public input and setting future County direction.

- **Pertinent Letters and Memoranda from Washoe County Files:**
  - 4/11/93 Memorandum and attachments from Craig V. McConnell, Public Works Director, to the Washoe County Commissioners and County Manager regarding actions taken concerning public discussion of the Whites Creek Detention Basin project at the location of the four-branch flow split. Attachments include the April, 1993 Agenda for the Southwest Truckee Meadows Citizens Advisory Board (CAB); the Presentation Agenda to the Southwest Truckee Meadows CAB regarding the detention basin; notification letter to local property owners regarding discussions held concerning the detention basin and schedule of subsequent meetings; and a description of key factors to consider regarding feasibility of the basin.

  - 4/23/93 Letter from the Southwest Truckee Meadows CAB to the Washoe County Commissioners informing them of the Board's unanimous denial of the Whites Creek Detention Basin project.

  - 4/28/93 Letter from Craig McConnell to Garth Dull, Director of the Nevada Department of Transportation (NDOT), stating the County Commissioners' vote to not proceed with a joint County-NDOT detention basin on Whites Creek.

  - 5/11/93 Letter from the Office of the Washoe County Clerk to Craig McConnell stating the Washoe County Commissioners' discussion and negative vote on the Whites Creek Detention Basin project.

  - 5/11/93 Letter from Ronald W. Hill, Deputy Director of NDOT, to Mr. Brian Walters regarding factors considered in proposing the Whites Creek Detention Basin project.

  - 7/26/93 Agenda for the 7/26/93 meeting of the Regional Water Planning and Advisory Board of Washoe County. Agenda Item No. 5 is a "Discussion on the Need for Whites Creek Drainage Basin Study".

  - 7/29/93 Letter from David R. Roundtree, Regional Water Manager, to Mr. Keith Kellison, Chairman of the Southwest Truckee Meadows CAB regarding involvement of the CAB in development of a Whites Creek Basin Management Program.

  - 8/17/93 List of private and public property owners within the Whites Creek Basin.

  - 8/20/93 Sample Request for Proposals and schedule to consultants for the following items: (1) Formulation of an approach to stormwater management planning of the Whites Creek basin and its connection to Steamboat Creek; and (2) Development of interim policies for managing the basin.

- **Report on the February 1986 Flood in Western Nevada; prepared by Michael W. Ekern, National Weather Service Forecast Office; March 21, 1986.**
  - Summary of the meteorological conditions leading up to the mid-February, 1986 flooding along the Carson and Truckee Rivers, including precipitation records, and a description of the impacts of the flooding, including National Weather Service bulletins.

- **Current Plan Development Report, Truckee Meadows (Reno-Sparks-Metropolitan Area) Nevada; prepared by the Army Corps of Engineers, Sacramento District; July, 1990.**
  - Description of the "Current Plan" being developed by the Corps of Engineers for the Truckee River and tributaries from Reno downstream through Sparks and the Truckee Meadows area in Washoe County north of Huffaker Hills. The Plan includes the Huffaker Hills Dam, a

downstream high-flow channel, levees, floodwalls, excavation, and bridge replacements.

- **Refinement Study, Truckee Meadows (Reno-Sparks Metropolitan Area), Nevada; prepared by the Army Corps of Engineers, Sacramento District; February 1, 1989.**
  - A discussion of potential refinements to the Truckee Meadows project to be studied during the Preconstruction Engineering and Design phase of the project. The project refinements considered include: assessment of the consideration given the Brown Plan; incorporation of the UNAES detention basin into the project; possible reduction of levee freeboard; elimination of Standard Project Flood structural features; and location of marsh enhancement features. Discussion is also provided regarding the Corps' responsibilities in fulfilling requirements of the National Historic Preservation Act of 1966, an assessment of the downtown Reno floodwalls, and local cost share credit requests.

- **Hydrology Office Report Update for the Truckee Meadows, Nevada General Design Memorandum - Spanish Springs and Huffaker Hills Detention Facilities Site Evaluations; prepared by the Army Corps of Engineers, Sacramento District; January, 1989.**
  - A memorandum presenting the results of the revised hydrology for Spanish Springs Valley, including evaluation of two reservoir sites in Spanish Springs Valley and one at the Huffaker Hills Narrows.

- **Office Report for the Truckee Meadows, Nevada General Design Memorandum - Hydrology Review and Update; prepared by the Army Corps of Engineers, Sacramento District; May, 1989.**
  - Results of the hydrology review and update for the Truckee Meadows area and for Spanish Springs Valley, evaluation of the two reservoir sites in Spanish Springs Valley, and a project-level evaluation of the Huffaker Hills Dam site on Steamboat Creek.

- **Office Report: Truckee Meadows (Reno-Sparks Metropolitan Area), Nevada Project; prepared by the Army Corps of Engineers, Sacramento District; May, 1992.**
  - Update to prior reports dealing with proposed flood control and recreation improvements. New evaluations indicated that the project was economically unfeasible with a benefit-to-cost ratio (BCR) of 0.42 to 1. The project was correspondingly reclassified from an active to a deferred category.

- **Major Drainageways Plan, City of Reno**
  - This Plan identifies critical drainage areas in the City of Reno and surrounding area and presents strategies for their treatment and maintenance. The focus of the Plan is to address the visual appearance and uses of specific major drainageways. Of particular concern are those

drainageways that are important to public health, safety and welfare and those that retain additional public values. The document includes a resource analysis, policy analysis, implementation strategies and recommendations designed to preserve and improve these public resource areas.

- **"Draft" Preliminary Feasibility Analysis, Whites and Thomas Creeks Flood Control Detention Basins; prepared by Nimbus Engineers; March, 1994.**
  - Preliminary feasibility study for the construction of regional detention basins near the base of Mt. Rose at Timberline Road to attenuate flood discharges experienced in downstream reaches of Whites Creek and Thomas Creek.

B. Contacted Parties

The following individuals have been contacted on one or more occasions to discuss existing information and present preliminary findings and approaches:

- Craig McConnell, Washoe County Public Works
- David Price, Washoe County Public Works
- Leonard Crowe, Washoe County Comprehensive Planning
- Kirk Nichols, Washoe County Public Works
- David Roundtree, Regional Water Management Agency
- Peggy Bowker, Nimbus Engineers
- Mark Forest, Kennedy/Jenks Consultants
- Amir Soltani, NDOT
- Chris Miller, NDOT
- Paul Frost, NDOT
- Robert Sader, Attorney
- Alex Fittinghoff, CFA
- Samuel Chacon, CFA
- Participants of four (4) Workshops

Several meetings have been held with the staff of Washoe County cited above, and a First Draft of the Preliminary Whites Creek Basin Management Study was prepared and submitted to Washoe County on December 7, 1993. The First Draft was refined based on input received from Washoe County staff and workshop participants, and a Second Draft was prepared and submitted to Washoe County on April 4, 1994. Refinements have also been made to the Second Draft and are now represented in this final version of the study.

C. Hydrologic and Hydraulic Reports for Development Projects

Numerous hydrologic and hydraulic reports prepared for existing and proposed development projects within the lower Whites Creek watershed have been reviewed, and information provided in said documents has been incorporated into the evaluation of existing conditions and formulation of interim policies.

D. Base Map

A base map has been prepared that assimilates the location of existing and proposed development projects, highway improvements, drainage structures, FEMA floodplain boundaries and other significant features within the primary study area comprising the lower Whites Creek watershed. The underlying information on the map consists of five foot (5′) contour interval topography developed in 1966 by NDOT. Though the topography has been altered locally by improvements related to land development since 1966, much of the topographic features have essentially remained unchanged since that time, and the general overall topography of the lower watershed is substantially correct on the base map. This base map and pertinent information is represented as Exhibit B.

E. Geologic Mapping

The Nevada Bureau of Mines and Geology was contacted to determine the nature and extent of geologic mapping that has been performed in the lower Whites Creek watershed. In response, CBA acquired Map 4BG, the Mt. Rose NE Quadrangle Geologic Map prepared in 1983 by H.F. Bonham, Jr. and David K. Rogers. This map includes most of the Whites Creek watershed north of Mount Rose Highway and west of U.S. 395. Geologic units delineated on the map in the study area consist primarily of the Upper Pleistocene (greater than 10,000 years old) Tahoe Outwash-Mount Rose Fan Complex and Donner Lake Outwash-Mount Rose Fan Complex adjacent to the flow split near Shadowridge Park and covering large areas downslope, and younger Alluvial Bajada deposits of the Holocene age (less than 10,000 years old) along two of the four primary channels (Channels #2 and #4, Exhibit B) and adjacent to U.S. 395. Exhibit C depicts generalized surface geologic characteristics derived from soils information.

F. Field Investigations

Several field investigations have been performed within various portions of the Whites Creek watershed, with particular emphasis on the primary study area of the lower Whites Creek watershed. Information derived from these field investigations, as well as from the data collection effort and discussions with Washoe County staff and other key individuals, have facilitated the formulation of conclusions presented in this Preliminary Basin Management Study.

II. OPINIONS, ACCEPTANCE AND CONCURRENCE PERTINENT TO EXISTING STUDIES

Based upon a review of existing studies and reports, field reconnaissance and discussions with Washoe County staff and other key individuals, the following fundamental conclusions have been drawn with regard to the lower Whites Creek watershed.

A. **Magnitude of the 100-Year Discharge for Whites Creek** - CBA reviewed the hydrologic analyses and various calculated values for the 100-year discharge for Whites Creek as presented in the background materials provided by Washoe County in an effort to establish a value that would be most appropriate for use in basin management planning activities. After completion of our review, we have concluded that the 100-year discharge magnitude of **5100** cfs for Whites Creek at Shadowridge Park should be utilized for the current basin management planning activities, at least until such time that a detailed and comprehensive hydrologic analysis is performed. Our rationale for this recommendation is as follows:

1. The HEC-1 analysis presented in the Whites Creek Detention Feasibility Study for NDOT appears to be reasonable.

2. Although technically outside of CBA's Scope of Work for this Preliminary Basin Management Study, CBA modified selected parameters in the HEC-1 analysis cited above to determine their impact upon the calculated discharge for Whites Creek at Shadowridge Park. These modifications included the use of normal depth calculations with varying roughness values along routing reaches, adjustments to impervious cover and adjustments to lag time calculations. The result of these various modifications was that the calculated 100-year discharge for Whites Creek at Shadowridge Park was lowered by as much as 1000 cfs under certain sets of assumptions and elevated by as much as 1000 cfs under other sets of assumptions. Within this range of impacts it appears that the 5100 cfs value is reasonable.

3. Downstream drainage structures along I-580 are being sized in consideration of an upstream discharge of 5100 cfs at Shadowridge Park, thus providing support to this value in terms of system compatibility.

4. In the absence of detailed analyses that would be pertinent to the preparation of the actual Basin Management Plan or a specific and comprehensive hydrologic investigation, it is more prudent to utilize conservative base assumptions in the development of interim basin management policies. The 5100 cfs value appears to be reasonable, yet conservative, and it is the highest of the values calculated from the prior studies reviewed by CBA.

Updated meteorological analyses are currently being performed as a part of the Washoe County Flood Control Master Plan. Upon completion of the updated meteorological analyses and their acceptance by Washoe County, it may be advantageous to revisit the adopted 5100 cfs value to determine if a revision is warranted.

B. **Distribution of the 100-Year Discharge for Whites Creek Downstream of Shadowridge Park** - Whites Creek at Shadowridge Park represents the location where flows are initially distributed across the lower Whites Creek watershed area under investigation. Flow is distributed into one or more of essentially four (4) channels that traverse the lower Whites Creek watershed, ultimately delivering proportionate runoff to the Steamboat Creek area east of U.S. 395. The flow distribution in the Shadowridge Park vicinity is impacted by the following:

1. The magnitude of the discharge collected at said location.
2. The extent to which existing vegetation within the channel becomes denuded by flood flows.
3. The existence of debris flow during a characteristic flood event.
4. The topographic definition of flow paths that exists immediately downstream prior to and during a given flood event.

During a 100-year flood event, it is CBA's opinion that, under existing conditions, it is not possible to accurately predict the distribution of the total discharge that will be allocated to each of the channels forming downstream of the Shadowridge Park area. Perhaps the most significant variable that limits the predictability of the distribution is the potential occurrence of debris flow within Whites Creek. Evidence of prior debris flows is readily identifiable in the field and is characterized by numerous residual large boulders that have been transported from the defined channel upstream of Shadowridge Park to various locations along channels and other areas downstream within the lower Whites Creek watershed. The occurrence of a debris flow will result in a slug of concentrated boulders, sediment and vegetation moving down the defined channel to be distributed at varying locations downstream of the defined channel as flow depth and velocities are diminished through expansion of the flow width.

The potential for debris flow can significantly impact the initial flow distribution originating at Shadowridge Park by effectively diverting flows in a random manner from one downstream channel to another and blocking some of the available flow areas during a given flooding event. For this reason, it is most appropriate to examine the flow distribution in terms of preferential values of proportional discharges to be applied to each downstream channel, from a future planning perspective for new development and infrastructure improvements. The flow distribution presented in the Whites Creek Detention Feasibility Study for NDOT would appear to be reasonable in this regard, as proportional discharges are somewhat equitably allocated to each of the four (4) downstream flow paths and as these distributions have been applied to the design of downstream drainage structures at I-580.

The distribution recommended for adoption by CBA for each of the four primary channels is represented below:

Channel	Allocated Discharge
#1	700 cfs
#2	1950 cfs
#3	1100 cfs
#4	1350 cfs
Total	5100 cfs

These values may be applied to each channel as a future design capacity goal, but are not representative of actual existing conditions due to the dynamic unpredictability of the flow distribution and potential for debris flow. For floodplain management purposes, a probabilistic approach must also be applied to facilitate the selection of a 100-year discharge rate that may enter each of the four (4) channels downstream of Shadowridge Park under existing conditions.

Based on an assessment of probability, CBA has concluded that a flow of approximately 3000 cfs has a one percent (1%) chance of being delivered to any of the four (4) available flow paths in any given year (i.e., a 100-year event). This conclusion was derived as follows:

- 5100 cfs has a 1 in 100 chance of occurring at Shadowridge Park (100-year event).
- Conservatively, there is a 1 in 4 chance of the entire flow at Shadowridge Park being delivered to any of the four (4) downstream flow paths.
- 3000 cfs has a 1 in 25 chance of occurring at Shadowridge Park (25-year event).
- The product of the probabilities of the 1 in 4 chance (flow paths) and the 1 in 25 chance (25-year discharge at Shadowridge Park) is a 1 in 100 chance for 3000 cfs to be delivered to any of the four (4) flow paths, or a 100-year event.

CBA derived the 3000 cfs value for the 25-year discharge at Shadowridge Park by applying 25-year precipitation values represented on available NOAA atlases to the HEC-1 model presented in the Whites Creek Detention Feasibility Study for NDOT. Since the standard for floodplain management in Washoe County and per FEMA is the 100-year event, floodplain conditions along each of the four (4) flow paths downstream of Shadowridge Park need to be established under the assumption that 3000 cfs is initially delivered to them. Until such time as structural measures are implemented that will serve to establish the flow distribution desired for 5100 cfs at Shadowridge Park, a flow of 3000 cfs being delivered to each flow path must be considered in the design of development projects within the lower Whites Creek watershed.

C. **Existing Problem Areas** - As a part of the field investigations performed by CBA staff and the review of available information, several problem areas or potential problem areas were identified within the lower Whites Creek watershed in terms of flooding potential associated with development projects and existing infrastructure improvements. The following listing represents a preliminary identification of potential problem locations that may merit further investigation as a part of future studies. It must be noted that CBA's conclusions are not substantiated by detailed calculations, but have been based upon engineering judgement; hence, the following listing may not be complete and/or some of the listed locations may be determined to not have problems from a flood hazard or capacity perspective upon closer, more detailed examination.

1. **Existing Culverts Along U.S. 395** - All of the existing drainage structures that drain Whites Creek flows are substantially inadequate to convey distributed discharges underneath the roadway during a 100-year flood event. The existing highway will cause upstream ponding of stormwater runoff and, when ponded flood waters reach sufficient levels, sheet flooding across the highway will occur.

2. **Old Virginia Street Culverts** - Inadequate drainage structures exist across Old Virginia Street, and similar conditions will prevail as described for U.S. 395.

3. **Zolezzi Lane Drainage Structures** - The drainage structure crossing of Zolezzi Lane that serves Channel #1 is of substantially insufficient capacity to pass the proportioned 100-year discharge. The existing roadway will divert some of the flow east along the south side of Zolezzi Lane and some of the flow will spill northerly across the roadway. At the intersection of Zolezzi Lane and U.S. 395, there is virtually no provision for accommodating runoff originating from Channel #2 (with some spillover flow from Channel #3), and flooding of this intersection will occur during a 100-year event.

4. **Existing Residential Structures Immediately Downstream of the Defined Channel at Shadowridge Park** - Several existing residential structures at this location are subject to a high flood and debris flow hazard during a 100-year flood event.

5. **Whites Creek Estates** - Some of the existing residential structures adjacent to Channel #1 have a potential for flooding during a 100-year event as induced by spillover from the channel at subdivision street crossings or by limitations in channel capacity.

6. **Lancers Estate** - Some of the residential lots backing up adjacent to the south of Channel #4 have a potential for flooding during a 100-year event.

7. **Existing Residential Structures South of Whites Creek Lane, West of the Proposed Pine Tree Ranch Subdivision** - Several of these structures have a potential for flooding from Channels #2 and #3 during a 100-year flooding event.

8. **Wedge Parkway** - Wedge Parkway is elevated from one to several feet above existing grade and crosses the lower Whites Creek watershed somewhat transversely to the direction of drainage flow. The newly constructed segment of Wedge Parkway between the Mt. Rose Highway and Whites Creek Lane will have a tendency to impound runoff in excess of the proportioned discharge of 1350 cfs for Channel #4 on the upstream side of the roadway and divert flow northeasterly along the west side of the roadway toward Whites Creek Lane. The existing drainage structure under construction across Channel #4 appears to have adequate capacity for the proportioned discharge for this flow path, provided the flow is delivered to the drainage structure itself. Currently, it is proposed that the proportioned flow within Channel #4 be channelized and delivered to the drainage structure as a part of the future development of Sterling Ranch.

It should be reiterated that the above observations and conclusions of system capacity problems are based upon preliminary investigations, only, and will require further substantiation as additional more detailed studies are performed.

III. QUALITATIVE EVALUATIONS OF FLOODING CONDITIONS

To date, floodplain administration within the lower Whites Creek watershed has been based primarily upon floodplain information presented on the FEMA Flood Insurance Rate Maps for Washoe County, Panel Numbers 1501 (Effective date: August 1, 1984) and 1463 (Effective date: April 16, 1990). The floodprone areas depicted for the lower Whites Creek watershed are represented as "Zone A" which indicates that they were originally studied using <u>approximate methods only</u>. Based upon CBA's experience as a Flood Insurance Study Contractor with FEMA, the degree of detail that would have been inherent to these approximate Zone A designations was undoubtedly minimal and, per FEMA guidelines, would have been limited to a cursory review of USGS quad sheets, aerial photographs, and primary low flow paths. It is CBA's professional opinion that the extent of the floodplains represented on these FEMA Flood Insurance Rate Maps for the lower Whites Creek watershed is significantly understated.

In order to accurately delineate the extent and characteristics of flood hazard areas within the lower Whites Creek watershed, a detailed hydrologic and hydraulic analysis will be needed, which is outside the scope of the current study. Such an analysis will need to include the following:

1. Refinement of the total 100-year discharge value of 5100 cfs for Whites Creek at Shadowridge Park, if appropriate.
2. Acquisition of current topographic mapping of the lower Whites Creek watershed with a minimum contour interval of two feet (2').
3. Hydraulic evaluations of flow characteristics across the lower Whites Creek watershed utilizing a combination of HEC-2 evaluations, normal depth calculations, weir flow calculations and culvert capacity calculations.

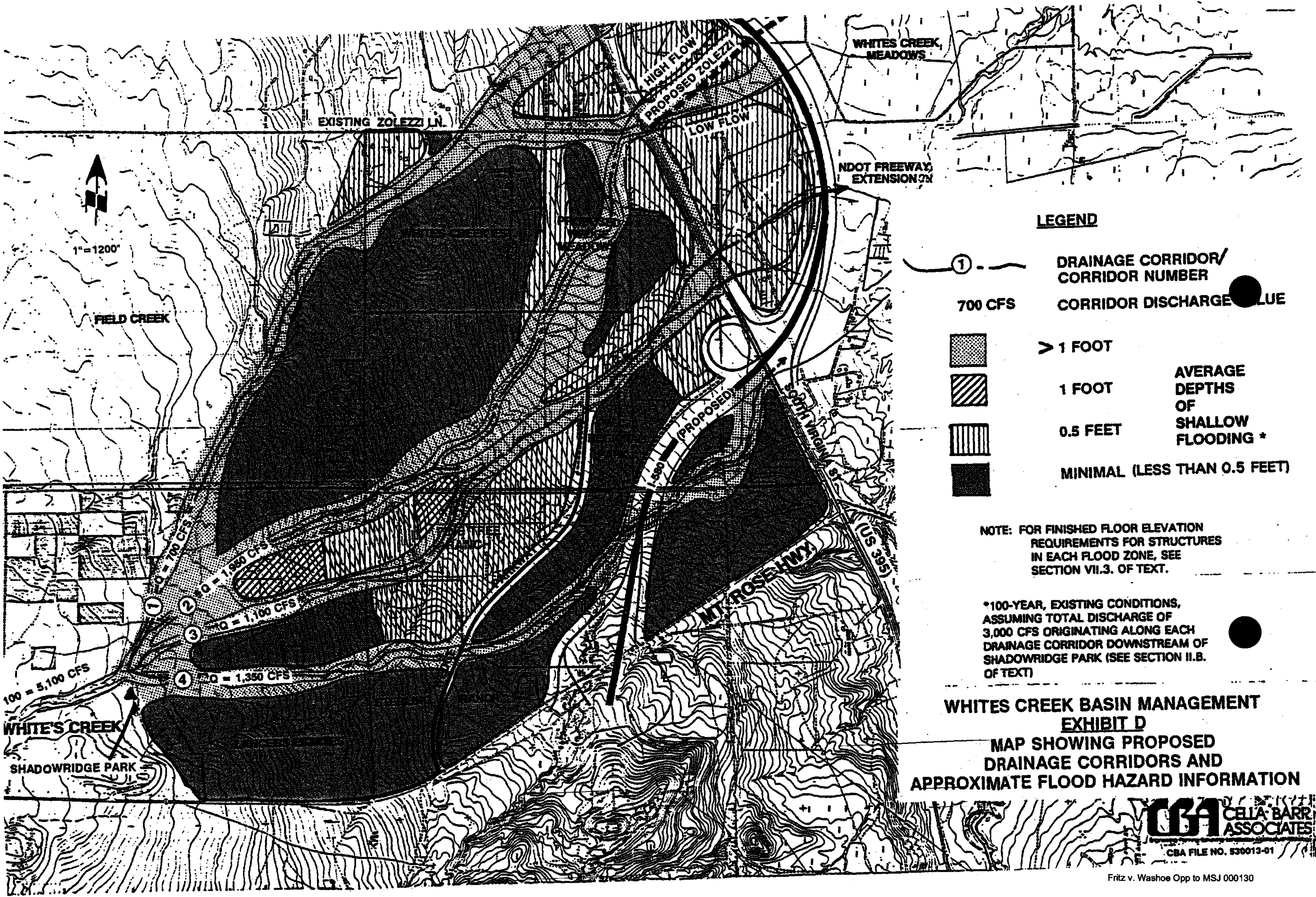
The detailed floodplain analysis should be performed at the earliest possible date in order to supplement the information contained in the current study; to more accurately define floodplain limits and characteristics; and to provide better information to be utilized in the design of new development and infrastructure projects. The analysis should consider both of the following assumptions pertinent to the flow distribution originating at Shadowridge Park:

- The existing conditions which create a potential for the total discharge of 3000 cfs (or a revised number, if applicable) being delivered to any of the four (4) downstream channels (see Section II.B.).
- Future conditions that would prevail if the flow distribution becomes fixed at Shadowridge Park through the implementation of structural measures or if the overall flow in Whites Creek is attenuated through implementation of other upstream structural measures.

As a part of this study, CBA performed a very preliminary analysis to estimate the extent and magnitude of flooding that currently has a potential of occurring within the lower Whites Creek watershed during a 100-year storm event. This analysis utilized USGS quad sheets, current aerial photographs, field investigations, the 1966 topographic mapping acquired from NDOT and rough normal-depth calculations performed across hypothetical flat cross sections of varying widths and slopes. Based on evaluations of the above, it is CBA's opinion that, under existing conditions, much of the lower Whites Creek watershed would be subject to "shallow sheet flooding" during a 100-year event. Approximate flood zones and average 100-year flooding depths have been delineated and are represented on Exhibit D. The flood zone designations that have been utilized in the approximate floodprone area mapping represented on Exhibit D are:

- Minimal Flooding Potential, Average Depth Less Than 0.5 feet
- Sheet flow, Average Depth = 0.5 feet
- Sheet flow, Average Depth = 1 foot
- Sheet flow, Average Depth Greater Than 1 foot

The approximate floodprone areas have attempted to account for the impacts of the construction of Wedge Parkway and I-580. In determining the shallow flooding zones, CBA assumed that a discharge of 3000 cfs may be directed to any of the four (4) primary channels originating downstream of Shadowridge Park. At such time as structural measures are implemented to attenuate the total flow or define the flow distribution for the downstream flow paths originating near Shadowridge Park, the extent and severity of flooding for the downstream areas within the lower watershed will be appreciably reduced.



LEGEND

─①─	DRAINAGE CORRIDOR/ CORRIDOR NUMBER	
700 CFS	CORRIDOR DISCHARGE [illegible]LUE	
▨	> 1 FOOT	AVERAGE DEPTHS OF SHALLOW FLOODING *
▨	1 FOOT	
▥	0.5 FEET	
■	MINIMAL (LESS THAN 0.5 FEET)	

NOTE: FOR FINISHED FLOOR ELEVATION REQUIREMENTS FOR STRUCTURES IN EACH FLOOD ZONE, SEE SECTION VII.3. OF TEXT.

*100-YEAR, EXISTING CONDITIONS, ASSUMING TOTAL DISCHARGE OF 3,000 CFS ORIGINATING ALONG EACH DRAINAGE CORRIDOR DOWNSTREAM OF SHADOWRIDGE PARK (SEE SECTION II.B. OF TEXT)

WHITES CREEK BASIN MANAGEMENT
EXHIBIT D
MAP SHOWING PROPOSED DRAINAGE CORRIDORS AND APPROXIMATE FLOOD HAZARD INFORMATION

CBA CELLA BARR ASSOCIATES
CBA FILE NO. 530013-01

IV. QUALITATIVE GEOMORPHOLOGY

CBA has performed a qualitative assessment of the types of fluvial processes that occur within the lower Whites Creek watershed downstream of the flow split at Shadowridge Park, in order to assist in the development of design requirements and policies for continued land development activities and infrastructure improvements proposed within the area. This assessment is based on field reconnaissance; the Soil Survey of Washoe County, Nevada, South Part prepared by the United States Department of Agriculture, Soil Conservation Service (August, 1983); geologic mapping of the Mt. Rose NE Quadrangle prepared by H.F. Bonham, Jr. and David K. Rogers (1983) and published by the Nevada Bureau of Mines and Geology; aerial photographs; and 1966 topography obtained from the Nevada Department of Transportation. In addition, two papers have been consulted extensively: "Alluvial Fan: Proposed New Process-Oriented Definitions for Arid Southwest" by Richard H. French, Jonathan E. Fuller, and Steve Waters (Journal of Water Resources Planning and Management, Vol.119, No. 5, September/October, 1993); and "Geologic Insights into Flood Hazards in Piedmont Areas of Arizona" by Philip A. Pearthree (Arizona Geology, Vol. 21, No. 4, Winter 1991, Arizona Geological Survey).

Alluvial fans are complex landforms. They are typically cone-shaped features containing boulders, gravel, sand and fine sediments that have been eroded from mountain watersheds and deposited on the adjacent piedmont or valley floor. In general, alluvial fans in the Southwest can be classified as active alluvial fans, distributary flow areas, and inactive alluvial fans (French, et al, 1993). A brief description of each type of fan is provided below to aid in understanding the geomorphic characteristics of the lower Whites Creek watershed.

Processes associated with **active alluvial fans** include rapid channel migration, debris flows, hyper-concentrated sediment transport, channel bank erosion, local bed scour and flash flooding. These fans are characterized by the following:

- Drastic changes in channel pattern and frequent channel movement;
- Bifurcating channel patterns that radiate outward in the downstream direction and that may be discontinuous;
- Low channel capacities with channel flow changing to sheetflow in the downstream direction;
- Recent and relatively uniform deposition of sediment across the fan surface;
- Debris flow levees;
- Weak soil development;
- Immature vegetative communities;
- Limited topographic relief; and,
- Lack of bedrock exposure.

In contrast, **inactive alluvial fans** are subject to sheet flooding, local deposition and scour within a stable channel pattern, extensive sediment transport, and flash flooding. Landforms associated with inactive alluvial fans include:

- Tributary drainage networks;
- Channel and/or overbank capacities adequate for significant flood events, and that increase in capacity in the downstream direction;
- Lack of recent deposition of sediment on the fan surface;
- No recent debris flow activity;
- Extensive soil profile development;
- Mature vegetative communities;
- Significant topographic relief; and,
- Bedrock outcropping within or between channels.

Distributary flow areas exhibit a channel pattern similar to active alluvial fans, but experience hydraulic processes more like those of inactive alluvial fans. Processes that occur in distributary flow areas include local scour and fill, divergent flow, stream capture, flash flooding, hyper-concentrated sediment transport, and shifting of runoff among existing channels. These areas can be identified according to the following characteristics:

- Bifurcating channels that radiate outward;
- Lack of channel capacity for significant flood events;
- Channels that are poorly defined and that may be discontinuous downstream;
- Sheet flooding;
- No debris flow activity below the fan apex;
- Broad floodplain with no apparent stream terraces;
- Low to variable topographic relief;
- Variable soil development;
- Immature and mature vegetation;
- Stable, although not completely predictable, flow paths.

Whites Creek originates on the eastern flank of Mount Rose (elevation 10,778 feet), from which it delivered to the base of the mountain front, at an elevation of approximately 6000 feet. From this location flow expands for a distance of approximately 3500 feet downstream from the mountain front, then becomes re-confined into a channel that is entrenched into an old alluvial fan surface. This alluvial fan surface is probably of Pleistocene age (greater than 10,000 years old), as upper piedmont areas near mountain ranges throughout the Southwest are often dominated by abandoned alluvial fans of this age. The entrenched Whites Creek channel continues in the downstream direction until it reaches a concrete, low flow splitter structure at Shadowridge Park. At this location flow exits the defined channel onto the lower Whites Creek basin, which is characterized by a radial, distributary flow network dominated by four channels. These channels are characterized by low, but variable flow capacity, resulting in generally unconfined distributary flow and alluvial-fan activity downstream of the concrete flow splitter.

Using the classification scheme outlined briefly above, the Whites Creek basin, below the flow split at Shadowridge Park, exhibits characteristics of both an active alluvial fan and a distributary flow system. Based on field reconnaissance, the lower Whites Creek basin displays the following characteristics:

- Radiating channel pattern from the apex (Shadowridge Park area) to the toe of the fan;
- Relatively stable channel pattern; we did not see any evidence of recently abandoned channels indicative of channel migration or avulsion (sudden changes in the course of a channel);
- Generally low channel capacities with no definite trend towards increases in channel capacity in the downstream direction; confinement of flow varies greatly, depending upon fan topography and Quaternary geologic faulting.
- Recent debris flow activity, as evidenced by debris flow deposits at the apex and downstream. One boulder train at the apex, between Channels #1 and #3, is located on a geologically young (Holocene) surface;
- Sheetflooding, increasing in the downstream direction and particularly adjacent to U.S. 395, resulting from poor channel definition and detention of flow created by U.S. 395 and adjacent development;
- Variable topographic relief across the fan;
- Relatively weak soil development throughout most of the fan.

Soil profile development provides a tool to use in determining how old an alluvial surface is, as such factors as silt, clay and calcium carbonate content tend to increase with age. Soils can be used, therefore, to determine approximate ages of surfaces and, therefore, which surfaces have been subject to recent flooding, erosion and deposition. The Soil Survey maps produced by the Soil Conservation Service depict much of the Whites Creek basin below the fan apex at Shadowridge Park as being occupied by Oest soils, described primarily as bouldery or sandy loams. Additional soil units adjacent to and immediately west of U.S. 395, the Surprise sandy loam and the Dithod sandy loam, are described mainly as coarse sandy loams that are subject to flooding. Based on the soil descriptions, the Oest, Surprise and Dithod units can be interpreted as being young soils of Holocene age (less than 10,000 years old) and younger (see Exhibit C).

The Whites Creek fan also contains remnants of Leviathan and Spasprey stony sandy loams, which make up the higher alluvial fan surface into which Whites Creek has entrenched its channel upstream of Shadowridge Park and which also exist on topographically high areas of the lower Whites Creek basin. These latter soil units can be interpreted as being of Pleistocene age (greater than 10,000 years) or older, and therefore, have not been subject to any significant flooding for at least 10,000 years (see Exhibit C). This corroborates well with the approximate floodplain information presented on Exhibit D.

With the exception of the Pleistocene-age alluvial deposits upstream of and adjacent to the fan apex, and the relatively high Pleistocene-aged remnants on the lower fan, it is our opinion that most of the lower Whites Creek basin has been and is currently subject to flooding, erosion and sediment deposition. This is in distinct contrast to the geologic mapping of the Whites Creek watershed published by the Nevada Bureau of Mines and Geology. As previously stated, this mapping shows most of the lower basin to be covered by Pleistocene-age Tahoe Outwash - Mount Rose Fan Complex and Donner Lake - Mount Rose Fan Complex alluvial deposits, with Holocene deposits located primarily along the toe of the fan adjacent to U.S. 395. It is our professional opinion, based on field reconnaissance, that the Soil Survey more accurately reflects current geomorphic processes within the lower basin than the geologic map.

In summary, the lower Whites Creek basin displays some characteristics typical of active alluvial fans and some characteristics typical of distributary flow areas. It is subject primarily to relatively unconfined flooding and sheetflow, and debris flow activity that will be most prevalent in the vicinity of the fan apex and immediately downstream. In our opinion, during significant flow events large quantities of sediment varying in size from small particles to boulders and other debris are likely to be carried by Whites Creek onto the alluvial surface downstream of the concrete flow splitter. Where this sediment and debris are deposited will impact where flooding occurs. It is likely that flow will spread out across the upper fan area immediately downstream of the concrete flow splitter, distributing itself initially among the three channels immediately below the fan apex (Channels #1, #3 and #4) and areas in between. (Channel #2 begins as a divergence from Channel #1 a short distance downstream from the apex.) Within a short distance downfan, topographic relief increases and likely constrains the extent of flooding until the toe of the fan is reached. Because the existing channel pattern appears to be fairly stable, in comparison to a classic, active alluvial fan, rapid channel migrations or avulsion are not anticipated. Shallow sheetflooding will dominate the lowermost part of the basin adjacent to U.S. 395 because of the lack of topographic relief in this area and because of the current detention effect produced by the roadway.

V. DOWNSTREAM CONDITIONS

CBA examined downstream channel, floodplain and riparian conditions along Steamboat Creek, including field review. This qualitative assessment was necessitated by the fact that different approaches to resolving flooding concerns within the Whites Creek watershed may impact downstream conditions along Steamboat Creek.

Steamboat Creek is the largest tributary to the Truckee River in the south Reno area. It originates from Washoe Lake, about 15 miles south of Reno, and drains the southern and eastern part of Truckee Meadows, entering the Truckee River near Vista about six (6) miles downstream from Huffaker Hills. The valley floor area is mostly improved meadowlands used for pasture, hay production, and other agricultural purposes. Rural residences are scattered throughout the area, primarily in the vicinity of U.S. 395 and at the higher elevations along the east side of Truckee Meadows. Existing commercial development is very limited.

Per the Washoe County Flood Control Master Plan, Volume I, Steamboat Creek is well defined until it reaches Highway 341. Downstream of this point flow becomes much shallower and wider. The portion of the Truckee Meadows area traversed by Steamboat Creek is subject to severe flooding during periods of high runoff.

Steamboat Creek appears to contain some level of runoff on a perennial basis, which has resulted in the development of wetlands adjacent to the stream channel and within portions of the Truckee Meadows. Approaches to controlling flows within the Whites Creek watershed will have to be examined closely from a water quantity <u>and</u> quality perspective, in order to have as little impact as possible on the existing wetlands and the larger Truckee Meadows area and in order to avoid increasing downstream flooding of existing roadways and structures.

There are two (2) large scale development proposals that cover properties east of I-580 downstream of the primary study area, including Steamboat Creek north to Huffaker Hills. These proposed development projects are named Damonte Ranch and Double Diamond Ranch. The drainage designs for these development projects, as they relate to the Whites Creek basin, will be facilitated by the concentration of runoff at known locations along proposed I-580 and will not be appreciably impacted by variable sheet flooding conditions that currently prevail upstream of proposed I-580.

VI. CONCEPTUAL APPROACHES TO FLOOD CONTROL

Based upon the review of available information and evaluations of existing conditions, it is CBA's recommendation that implementation of all or a combination of the following flood control measures will most effectively simplify continued development and infrastructure improvements within the lower watershed with a reasonable probability of local and community acceptance:

Flow Distribution Structure

Under existing conditions, the distribution of the 100-year discharge to channels downstream of Shadowridge Park is highly unpredictable. This condition produces a greater potential for flooding along and adjacent to each of the downstream channels within the lower Whites Creek watershed. Channels #1 and #4 are currently reasonably well defined or will become well defined with development and infrastructure improvement projects proposed in the near future downstream of Shadowridge Park. Significant co-mingling of flows between Channels #2 and #3 occurs downstream of the initial flow distribution at Shadowridge Park, and this condition is not foreseen to be corrected in the near future.

The establishment of a predictable flow distribution just downstream of Shadowridge Park to allocate applicable percentages of the total 100-year discharge of 5100 cfs to each of the four (4) primary downstream channels will serve to appreciably reduce the flood potential within the entire lower Whites Creek watershed. The greatest immediate benefit in flood hazard reduction will be realized along Channels #1 and #4 and adjacent areas. Channels #2 and #3 will also experience a significant reduction in flood hazard, initially, with further benefits being gained in the future as the co-mingling of flows between these two primary flow paths becomes eliminated as continued development occurs within the lower watershed.

It is recommended that a **flow distribution structure** be considered at the approximate location depicted on Exhibit E1 as soon as such a structure may be designed and funded, in order to proportionately distribute the total discharge for Whites Creek to each of the downstream channels at rates consistent with the values represented on Exhibit D and per the Whites Creek Detention Facility Feasibility Study prepared for NDOT. This flow distribution structure is recommended to consist of a reinforced ring levee with incremental openings at each of the four (4) primary channel areas. A typical schematic cross section of this ring levee is depicted on Exhibit E2.

Although the design cross section and height of the ring levee will need to be determined as a part of a detailed design process, it is our opinion that the required height and proposed slope reinforcement will be relatively visually unobtrusive once constructed. The slope treatment of soil cement depicted on Exhibit E2 is capable of having an earth-colored finish and natural appearance while providing a monolithic barrier that provides significant stabilization against erosion and impact by large boulders and other debris. This concept will also serve to maintain the integrity of the existing perennial nature of Channels #1 and #3, as all four (4) channels would be allowed to pass through the ring

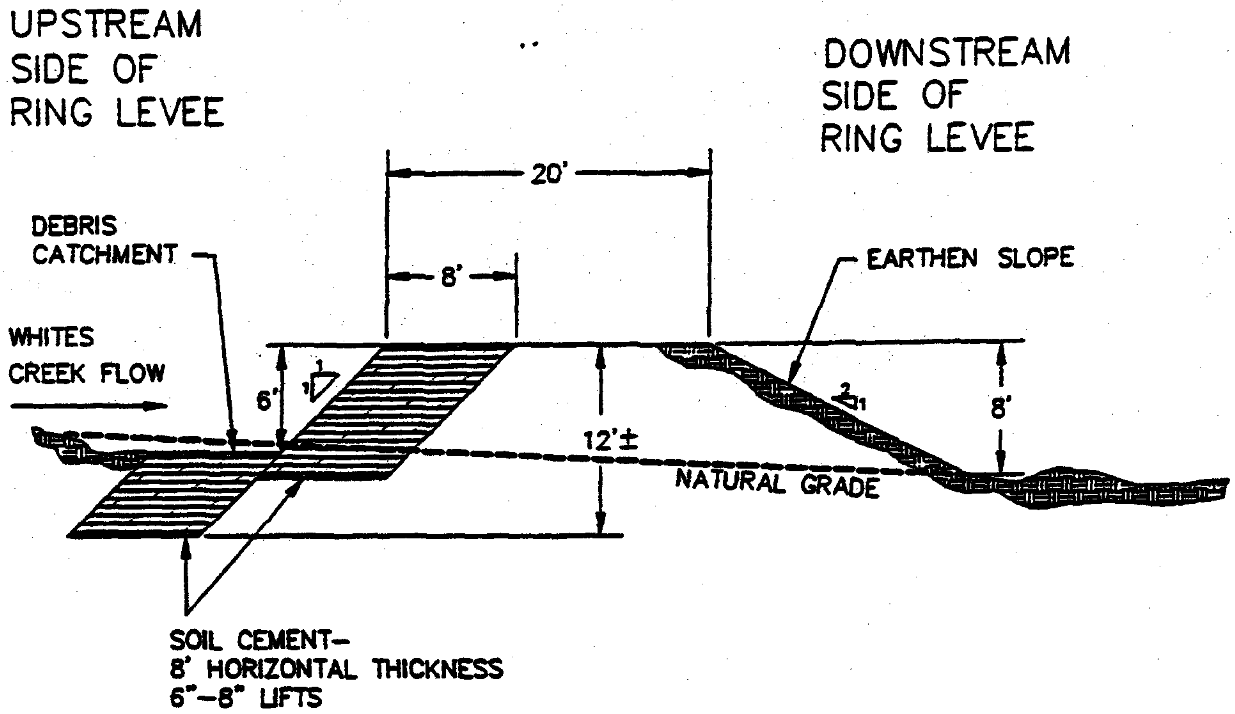


EXHIBIT E2

CROSS-SECTION OF RING LEVEE COMPRISING FLOW DISTRIBUTION STRUCTURE

CBA FILE No. 530013-01-0930

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levee individually via designated openings. By avoiding structural obliteration of riparian zones inherent to Channels #1 and #3, construction of the ring levee will not fall under the jurisdiction of Section 404 of the Clean Water Act and essentially will allow for the preservation of this existing riparian feature and habitat.

It is envisioned that construction of a ring levee system to serve as a flow distribution structure will allow for an effective desired distribution of flows to occur, if stormwater runoff is designed to pass through the designated openings in the levee system as an equalized and distributed weir flow. In order for this to be accomplished, the alignment of the ring levee will need to be parallel with the existing contours downstream of Shadowridge Park as approximately located on Exhibit E1. Use of a flow distribution structure as described will provide appreciable flood relief for downstream properties at a cost that is significantly less than previous proposals, including the Whites Creek Detention Facility Feasibility Study proposal applicable to this location. It will also be much less visually obtrusive than the detention basin option and will not require the obliteration of existing riparian areas. Actual construction costs, right-of-way/easement requirements and design parameters associated with the flow distribution structure will be developed as a part of subsequent design activities if this approach to flood control is deemed acceptable; however, the total cost is expected to be less than $1,000,000.

Local, Sub-Regional Stormwater Detention Basins

As continued development occurs within the lower Whites Creek watershed, the introduction of impervious surfaces and improved flow conveyance mechanisms (such as streets and excavated channels) will cause increases in rates of runoff experienced downstream of the lower Whites Creek watershed. The quality of runoff, particularly "first flush" runoff, will also diminish as pollutants inherent to land development (such as petroleum products, heavy metals, etc.) will also increase. These increases may have an adverse impact upon flooding and upon existing wetland areas present downstream along Steamboat Creek.

The majority of new development that is expected to occur within the lower Whites Creek watershed will ultimately drain toward primary Channels #2 and/or #3, with little new development draining toward Channels #1 and #4. One approach to addressing the impacts of continued development upon runoff rates and water quality is to require on-site detention of stormwater runoff with each new development project. However, until such time as the flow distribution at the Shadowridge Park area becomes structurally defined and downstream flow paths become predictable, the potential exists for flooding (drowning out) and breaching of local on-site detention facilities during a major storm event that causes overflow of primary channels to occur, and this will tend to have a potential of exacerbating downstream flooding problems. Further, the construction of local on-site detention facilities with new development does not guarantee that the combined timing of regulated flows released from said facilities will provide a reduction in downstream discharges, and thus, the local on-site detention approach as a requirement for new development projects is not an ideal solution.

Instead, it is CBA's recommendation that **local, sub-regional stormwater detention basins** be considered at the approximate locations shown on Exhibit E1 as a more effective means of compensating for increases in runoff rates and for water quality issues associated with new development within upstream portions of the lower Whites Creek watershed. Hence, with the construction of such facilities, development within the lower Whites Creek watershed may occur without consideration of any on-site detention facilities, with the need for such detention being provided by local, sub-regional facilities that serve all of the contributing projects.

The cost, sizing, design requirements and permitting requirements for these local, sub-regional stormwater detention facilities will need to be established as a part of a subsequent detailed design process.

Upstream Regional Detention Basins

Another conceptual approach to providing flood control for the lower Whites Creek watershed is the construction of **upstream regional stormwater detention facilities**. An option under this approach is presented in the "Draft" Preliminary Feasibility Analysis, Whites and Thomas Creeks Flood Control Detention Basins report prepared by Nimbus Engineers (March, 1994). The "Draft" report examines a location that would capture flows from both Whites Creek and Thomas Creek on a 120 acre site near the base of Mt. Rose at Timberline Drive (see Exhibit E3 Location Maps).

The overall concept presented by Nimbus Engineers is to capture and attenuate the peak flows for Whites Creek and Thomas Creek and release them into the existing downstream channels at more manageable rates. The concept also includes a multi-use approach that incorporates passive recreation features, wetlands creation and a waterfowl and wildlife refuge into the flood control design. Groundwater recharge and fisheries enhancements are also being investigated.

Nimbus Engineers has made contact with a number of regulatory agencies and interested parties. All of the agencies contacted have given a positive response to the concept of the project. The agencies contacted to date are:

- U.S. Army Corps of Engineers (COE)
- Nevada State Historic Preservation Office
- Nevada Department of Wildlife
- Nevada Department of Environmental Protection
- Nevada Division of Water Resources
- Washoe County Public Works
- Washoe County Department of Comprehensive Planning
- Regional Water Board

Further input from these agencies and others will be sought as the concept continues to be refined by Nimbus Engineers. The project concept will also be presented to the Southwest Area Citizens Advisory Board (CAB) and the Regional Water Board Technical Advisory Committee (RWBTAC) for their review and comment. A Section 404 Permit

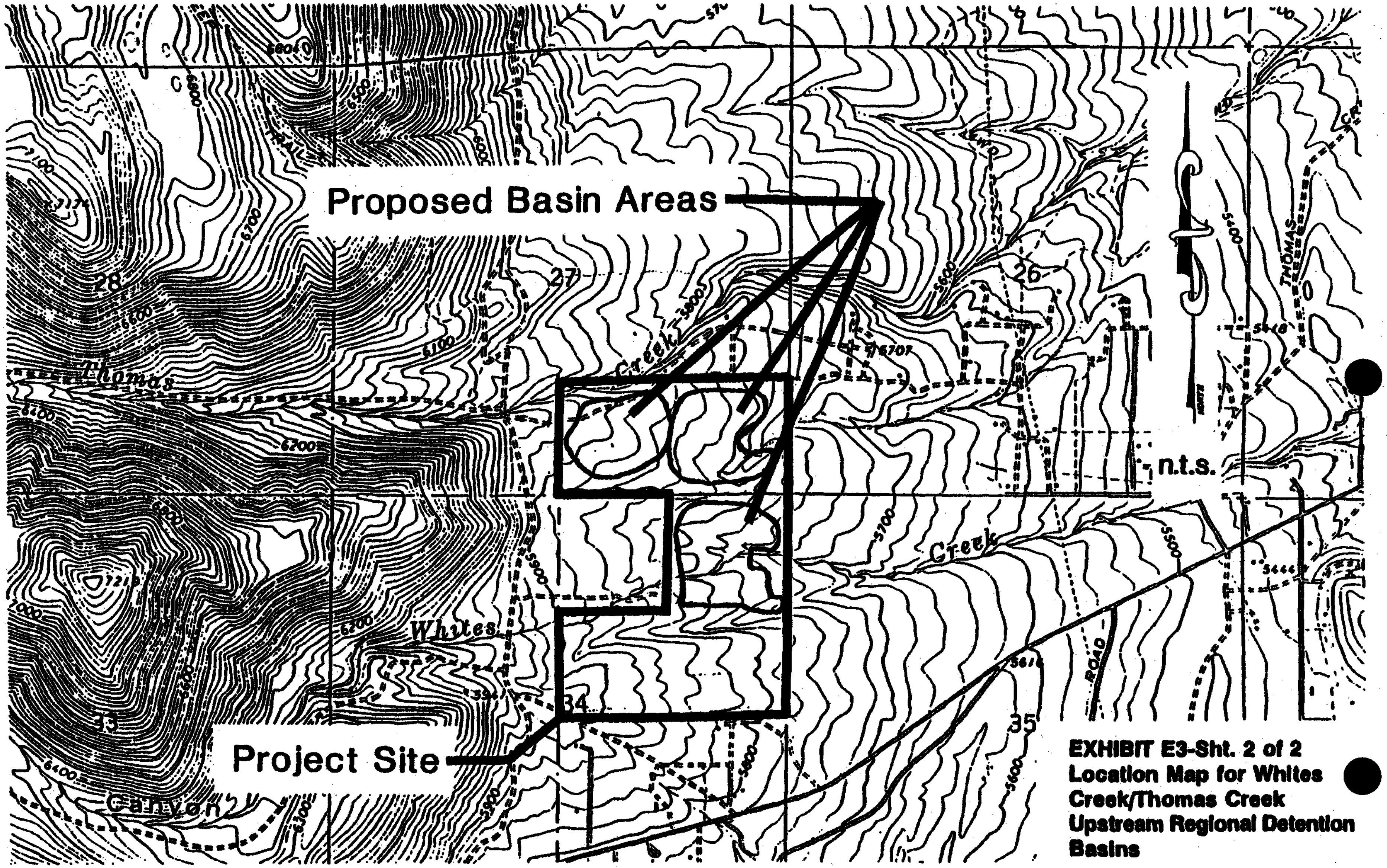


EXHIBIT E3-Sht. 2 of 2
Location Map for Whites Creek/Thomas Creek Upstream Regional Detention Basins

Nimbus Engineers

Figure 1
Project Site

preapplication meeting was scheduled with the COE for April or early May to discuss the project.

Previously developed hydrologic studies of Whites Creek and Thomas Creek were utilized to develop a preliminary size of facilities. The studies used were the Thomas Creek Flood Insurance Study developed for FEMA and the Whites Creek Detention Facility Feasibility Study prepared for NDOT. The hydrologic models for these studies were slightly modified to determine the volume of runoff which would impact the Timberline Road area during a 100-year event.

A preliminary facility size and configuration was developed using the entire volume of flow at Timberline Road and considering the physical constraints of the available site. An initial configuration of three basins, one for Whites Creek and two in series for Thomas Creek was used as a basis for a further analysis and for developing quantities and costs.

The hydraulic characteristics of the regional detention facilities determined from the Nimbus Engineers analysis are as follows:

	Whites Creek	Thomas Creek
Maximum Stage	17.3 ft.	13.8 ft.
Maximum Volume	317 Ac-ft.	308 Ac-ft.
Maximum Outlet Discharge	301 cfs	256 cfs

The estimated 100-year peak flows experienced downstream for the with and without regional detention conditions are given below:

	Without Detention	With Detention
Thomas Creek at Virginia Street	2544 cfs	880 cfs
Whites Creek at Shadowridge Park	5115 cfs	589 cfs

The investigated regional detention basins will require a maximum excavation of 3.9 million cubic yards of material and an estimated construction cost of roughly $12,500,000. Indications are that the excavation quantities could be significantly reduced (and consequently the costs) with several iterations of cost/benefit analyses and better topographic information.

Additional information regarding this conceptual approach to flood control is provided in the Nimbus Engineers' report.

Drainage Crossings of Existing Roadways

Several existing drainage crossings of roadways should be enlarged or have drainage structures provided, in response to development activities and/or reducing current flood hazards in selected locations. The primary locations requiring drainage structure enlargement or new structure installation include:

- Zolezzi Lane crossing of Channel #1.
- U.S. 395 crossing of Channel #1.
- Zolezzi Lane and U.S. 395 Intersection; Drainage structure and outfall channel needed to accommodate flows from Channel #2.
- U.S. 395 crossing of Channel #3.

VII. INTERIM POLICIES FOR MANAGING THE BASIN

As a result of the reviews, discussions, evaluations and investigations performed as a part of this Preliminary Basin Management Study, several proposed interim policies have been formulated relating to new development and infrastructure improvement projects within the lower Whites Creek watershed. It is proposed that these interim policies be utilized until such time as more detailed basin management planning activities or structural improvements are completed at a later date.

1. Drainage Corridors

Open space will be established and retained along each of the four (4) drainage corridors represented on Exhibit D. The purpose of establishing these drainage corridors shall be twofold:

A. To provide a continuous means of conveyance of the proportional discharge for each of the primary channels originating from the flow split at Shadowridge Park downstream to I-580 or the limit of the primary study area.

B. To provide open space linkages and opportunities for passive recreation within the primary study area.

At locations where channel definition and/or capacity is insufficient to convey the desired proportionalized flow, a combination of excavation and adjacent filling will be needed to create a defined channel or conveyance area.

There are several issues associated with the establishment of drainage corridors that require resolution. They are:

- Who will retain ownership of drainage corridors?
- Will they be retained as easements or fee title right-of-way?
- What mechanism will be utilized to convey drainage corridors or easements to an appropriate authority?
- Who is responsible for maintenance?
- Should drainage corridors be natural to the extent feasible or modified by excavation and grading?
- What stabilization measures are deemed appropriate when needed?
- Should establishment of drainage corridors occur on a piecemeal basis in conjunction with new development or should an overall drainage improvement district be established?

2. Discharges

The following discharges shall be applied as the required design capacities, or <u>incremental</u> discharges, for each drainage corridor:

Drainage Corridor	Design Capacity
#1	700 cfs
#2	1950 cfs
#3	1100 cfs
#4	1350 cfs

The value of the total 100-year discharge for Whites Creek at Shadowridge Park is 5100 cfs.

Until such time as flows are predictably distributed downstream of Shadowridge Park through the construction of a structural flow distribution facility or until upstream attenuation is provided, the design for downstream development projects and the elevating of building finished floors must consider the possibility of **3000 cfs** entering any one of the four (4) drainage corridors (see Section II.B.). After construction of a flow distribution structure, the incremental discharges for individual drainage corridors will be applied. However, in certain instances, i.e., drainage corridors #2 and #3, the effect of co-mingling of flows will need to be considered for applicable downstream areas until such time as continuity exists along the applicable drainage corridors to a location downstream of a given point of interest.

3. Finished Floor Elevations

Finished floor elevations of new individual structures <u>where mass grading has not occurred</u> shall be established based upon the average flood depths represented on Exhibit D, until such time as more detailed floodplain mapping is performed for the lower Whites Creek watershed. The flood depths represented on Exhibit D may also be revised at any given location if substantiated by an acceptable site-specific engineering analysis. Average flooding depths represented on Exhibit D have been established under the assumption that **3000 cfs** may enter any of the four (4) drainage corridors downstream of Shadowridge Park, causing flooding of the corridor itself and adjacent areas. Finished floor elevations of individual structures where no mass grading has occurred shall be set a minimum of one foot (1′) above the estimated shallow flooding depths represented on Exhibit D for areas within, between or adjacent to drainage corridors. The one foot (1′) criteria applies to the upstream side of a given structure (see Exhibit F1).

For structures that are integrated into development projects where mass grading is proposed or has occurred, finished floors will be elevated a minimum of one foot (1′) above the applicable water surface elevations calculated via a site specific engineering analysis. In such instances, spillover from drainage corridors will need to be conveyed in streets and/or drainage easements around and adjacent to structures. Provisions must be made to accept spillover runoff, convey it safely, and release it downstream in essentially the same manner as for existing conditions. The one foot (1′) criteria applies to the upstream side of each structure. The impact of fences must be taken into consideration in the analysis. These concepts are graphically represented on Exhibit F2.

In areas of "minimal" flooding depicted per Exhibit D, finished floor elevations for structures shall be set a minimum of one foot (1′) above the highest adjacent natural grade (individual building sites) or the adjacent top of curb (mass graded condition). These requirements may be waived if a site specific engineering analysis demonstrates that no flood hazard exists. Requirements for the elevating of structures in areas of "minimal" flooding are represented on Exhibit F3.

4. Street Alignments

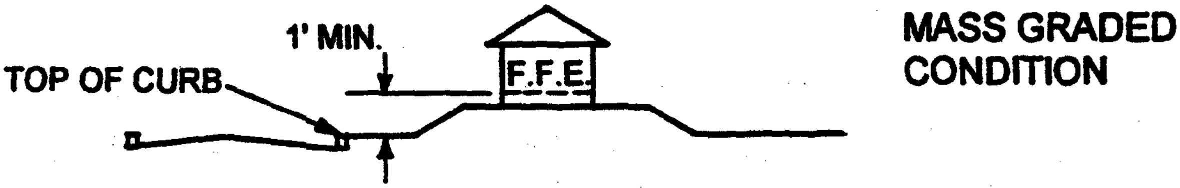
In areas of "minimal" flooding, no special requirements apply pertinent to street alignments. In areas having flood depth designations on Exhibit D, an appropriate amount of streets will be aligned with the direction of existing grades to provide conveyance for shallow flooding (see Exhibit G), at least until such time as incremental discharges for individual drainage corridors become established through upstream structural measures. Appropriate means for inflow and outflow to and from the internal street conveyance systems for development projects shall be provided and applicable shallow flooding in excess of the corridor discharge must enter and exit developed properties in essentially the same manner as under existing conditions. Where possible, the outfall for runoff generated on-site within a development project should be the nearest drainage corridor.

5. Depth of Flow in Streets

Streets utilized for overflow conveyance from drainage corridors shall have a maximum allowable depth of one foot (1′) and must consider the flooding conditions that would be present assuming that 3000 cfs has entered the drainage corridor downstream of Shadowridge Park, until such time as the distribution of flows becomes fixed or attenuation occurs through upstream structural measures. Once upstream structural measures are implemented to distribute the flow, the incremental corridor discharges will govern, the potential for shallow flooding in streets will be appreciably reduced or eliminated, and this requirement will be waived, if appropriate.

WHITES CREEK BASIN MANAGEMENT

IN AREAS OF "MINIMAL" FLOODING PER EXHIBIT D, F.F.E.'S FOR STRUCTURES SHALL BE SET 1' OR MORE ABOVE THE HIGHEST ADJACENT NATURAL GRADE (INDIVIDUAL BUILDING SITES) OR 1' OR MORE ABOVE ADJACENT TOP OF CURB (MASS GRADED CONDITION).



THE ABOVE SPECIAL REQUIREMENTS MAY BE WAIVED IF A SITE SPECIFIC ENGINEERING STUDY DEMONSTRATES THAT NO FLOOD HAZARD EXISTS.

EXHIBIT F3

FINISHED FLOOR ELEVATION REQUIREMENTS IN ZONES OF "MINIMAL" FLOODING.

6. Drainage Structures

Drainage structures for new roadways crossing drainage corridors will be sized to accommodate the applicable incremental corridor discharge. Where possible, a depressed section shall be provided within the roadway over the structure. Reinforcement of the adjacent fill slopes will also be required to minimize damage to the structure in the event that the roadway is overtopped, until such time as corridor discharges become predictably established through upstream structural measures.

7. Transverse Roadway Grades

Elevated roadways that extend perpendicular to flow directions are discouraged and will require prior approval of Washoe County, with consideration being given to any potential for obstructing, retarding or diverting said drainage flows when compared with existing conditions.

8. Grading

Lowering of existing grades for new development projects between or adjacent to drainage corridors will only be allowed if it can be demonstrated that additional flows are not diverted into the development project during a 100-year event as a result of site grading.

9. Detention

Based upon the evaluations and opinions discussed in Section VI of this Preliminary Basin Management Study, it has been concluded that attenuation of increased runoff produced by new development is needed to preclude the potential of significant increases in flooding and a deterioration in water quality experienced downstream within Steamboat Creek. It is also recommended that a preferred approach to providing attenuation of runoff and water quality storage is the construction of local sub-regional stormwater detention facilities, as opposed to requiring local on-site detention with each new development project.

Local, sub-regional detention facilities offer preferred benefits in terms of consolidated flood control and water quality treatment and the removal of requirements for setting aside lands within individual development projects to provide local on-site detention facilities. Also, until such time as incremental flows are successfully assigned to drainage corridors via upstream structural measures, the local on-site detention concept may serve to increase flood hazards due to a potential for overflow and breaching of said facilities during a major storm event. If the requirement that new development projects include provisions for local on-site stormwater detention is established by Washoe County, these concerns must be taken into consideration as a part of the design process.

Until such time as local sub-regional detention facilities are built, the following options may be considered as an interim means of accounting for adverse impacts associated with the construction of development projects in the lower Whites Creek watershed:

- Impact fees
- Phased basin excavation/construction
- Local on-site detention facilities that do not have a potential for overflowing induced by drainage corridor spillovers
- Hold harmless agreements with downstream property owners

The approximate locations for local, sub-regional stormwater detention facilities are represented on Exhibit E1. Further evaluations will be necessary to design, size and prepare a cost estimate for these facilities.

Funding mechanisms to be considered for construction of these facilities may include:

- Drainage improvement district
- Impact fees for new development
- Property taxes
- Drainage utility
- Other alternatives presented in the Washoe County Flood Control Master Plan

10. Site-Specific Engineering Analyses

There are a number of circumstances where a site-specific engineering analysis will be required to supplement or amend the information contained in this study prior to commencing with a given development or infrastructure improvement project. The following situations will require such an analysis:

- A development project that includes mass grading in a portion of the watershed having a flood hazard designation other than "minimal" on Exhibit D.
- A development project that includes basements. Basements will not be allowed in flood hazard areas.

- Any design proposal to amend or that would otherwise alter the flood hazard information represented on Exhibit D.
- Any design proposal to waive the finished floor elevation requirements set forth for areas of "minimal" flooding per Exhibit F3.
- Any project that proposes modification to, constriction to, or realignment of a drainage corridor.
- Individual building sites or subdivisions which include fences that are likely to appreciably alter surrounding flooding characteristics.
- Any roadway design project that impacts existing drainage patterns.
- Any other applicable set of circumstances where such an analysis is deemed appropriate by Washoe County.

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Exhibit 10

Exhibit 10

STORM DRAIN ANALYSIS

LANCER ESTATES UNITS 6 & 7

Introduction

Lancer Estates Units 6 and 7 are two subdivision projects consisting of 33 lots with a minimum size of 15,000 square feet. The projects are a continuation of the on-going Lancer Estates Project which is located in the South One-Half (S 1/2) of Section Thirty (30), Township Eighteen (18) North, Range Twenty (20) East, Mount Diablo Meridian. The site is bordered on the South by Mount Rose Highway. Approximately 138 lots have been developed in the project, which is approved for an ultimate buildout of approximately 309 units.

The purpose of this study is to inventory and analyze the existing storm drainage facilities and flow patterns with respect to the proposed storm drain improvements in Units 6 and 7.

Existing Storm Drain Patterns & Flows

The Lancer Estates Site slopes generally from the Southwest to the Northeast with a average gradient of about six percent. The Mount Rose Highway borders the site on its South side, and the North one-half of the roadway adjacent to the project drains onto Lancer Estates. The west central part of the project site contains a high rocky bluff (Lancer Hill) that is not a part of Lancer Estates, but does drain through the project.

REGISTERED PROFESSIONAL ENGINEER STATE OF NEVADA
GERALD J. WALSH
CIVIL
No. 6475

4/24/94

White's Creek crosses the Northwest corner of the Lancer Estates Site. Near the intersection of White's Creek and the North boundary of Lancer Estates, White's Creek splits into four channels. The Southernmost of these, which has been designated as Channel No. 4 in the Basin Management Study which is currently being conducted by Cella Barr Associates, intercepts most of the drainage from Lancer Estates. Channel No. 4 is designated as Zone A Flood Hazard Area by FEMA. (Firm Community Panel No. 320019-1501B). The Cella Barr Study further delineates the areas of Lancer Estates which are outside of Channel No. 4, and East of Lancer Hill, as areas subject to minimal flooding from over the 100 year storm runoff in White's Creek. Their study defines minimal flooding as less than 0.5' deep.

The enclosed map, Sheet 1 of 2, shows the project site and features described above. The map also shows drainage subareas as they will exist after development of Lancer Estates Units 6 and 7.

Storm drain runoff from the subareas has been calculated using the Rational Method. The parameters used for these calculations are as follows:

Runoff Coefficient ~ C

C = 0.40

Was used in the existing and proposed 15,000 square foot lot portion of the project.

C = 0.30

Was used in the westerly open area of the site. This area will ultimately be developed into one-half acre plus lots.

Time of Concentration ~ tc

Time of concentration was calculated using a flow velocity of four feet per second. A minimum time of concentration of ten minutes was assumed.

Rainfall Intensity ~ I

The City of Reno Intensity Duration - Frequency Curves where used, as published in the City of Reno Public Works Design Manual.

The parameters used for each subarea and the calculated 10 year and 100 year frequency storm runoff are listed in Table I on enclosed Plan Sheet 1 of 2.

Existing Development & Drainage Facilities

As discussed previously, approximately 138 lots have been developed in Units 1, 2, 3 and 4 of the project. Nine lots are presently being developed in Unit 5.

Plans for the existing units have been reviewed to determine the locations and sizes of the storm drain facilities which were constructed. These facilities are shown on Plan Sheet 2 of 2.

In Units 1 and 2, the street's were constructed with asphalt berms instead of curb and gutter. In many areas the berms are depressed with downsloping driveways. In these areas, the street has essentially no capacity to carry storm water. Since the amount of runoff carried by the street cannot be determined, we have not attempted to estimate the inlet capacity of these basins. Catch basin types and locations are listed in Table II. Storm drain pipe sizes and capacities are listed in Table III.

In Units 3 & 4, concrete curb and gutter, and concrete driveway approaches were constructed. These improvements are also proposed in Units 5, 6 & 7. The construction of these improvements allow the street to contain storm runoff and direct it to inlets. Therefore it is possible to calculate inlet capacities for catch basins in these Units. These facilities and capacities are also listed in Tables II and III on Sheet 2 of 2.

Proposed Unit 6 & 7 Drainage System

The construction of Units 2 and 3 has blocked the natural drainage path from the Westerly part of the site to Drainage Channel No. 4. The drainage facilities that were constructed with those units have a limited capacity. Therefore, the storm drain system in Units 5, 6 and 7 has been designed to intercept much of the Westerly site drainage and transport it to Drainage Channel No. 4. The site topography will make construction of this system difficult, with cuts in excess of twelve feet deep in some areas and construction of the roadway in fill in order to maintain pipe cover in other areas.

The Westerly site drainage will enter Units 6 and 7 in two locations. Drainage from Subarea U will enter Unit 6 at the rear of Lot 18. A temporary rip rap swale will be constructed to intercept the drainage at this point and carry it to a twenty-four inch pipe inlet near DeerValley Drive. The calculated 100 year storm flow at this time is 34.59 CFS. The pipe inlet will allow 30 CFS and approximately 4.59 CFS will flow southward on DeerValley Drive.

Drainage from Subarea M will enter Unit 7 near the Southwest corner of Lot 5. A twenty-four inch storm drain will be extended to intercept this flow. The calculated 100 year flow is 26.98 CFS and the inlet capacity will be 30 CFS. The pipe will contain the 100 year flow.

Table II on Sheet 2 of 2 shows the calculated inlet capacities for all inlets in Units 3, 4, 5, 6 and 7. The last column of this table also lists the calculated cumulative 100 year storm flow which will bypass each inlet. The last inlet on the Unit 5, 6 and 7 Storm Drain System is Inlet No. 18 at the corner of DeerValley Court and Solitude Drive. The 100 year bypass flow is much less than the existing condition flow at this point.

Table III on Sheet 2 of 2 lists the proposed storm drain pipes in Units 5, 6 and 7, along with free flow capacities, 10 year storm flows and 100 year storm flows. As can be seen by comparing the 100 year storm flows with the pipe capacity, the 100 year flow exceeds the free flow capacity of the pipes in several instances. In these cases the pipes will function under pressure.

Conclusion

The construction of the storm drain system as proposed in Lancer Estates Units 6 and 7 will provide drainage protection for the proposed homes in these units, and will also greatly improve the drainage protection for the existing units of Lancer Estates. The runoff which will bypass these new units during the 100 year storm event will be substantially less than what the existing condition runoff would be. The streets in the newer units will be capable of carrying the 100 year storm overflow. The streets in Units 1 and 2, however, will not contain any substantial storm water flows. In order to provide protection for homes in these areas, some reconstruction of streets, driveways and storm drain improvements would be required.

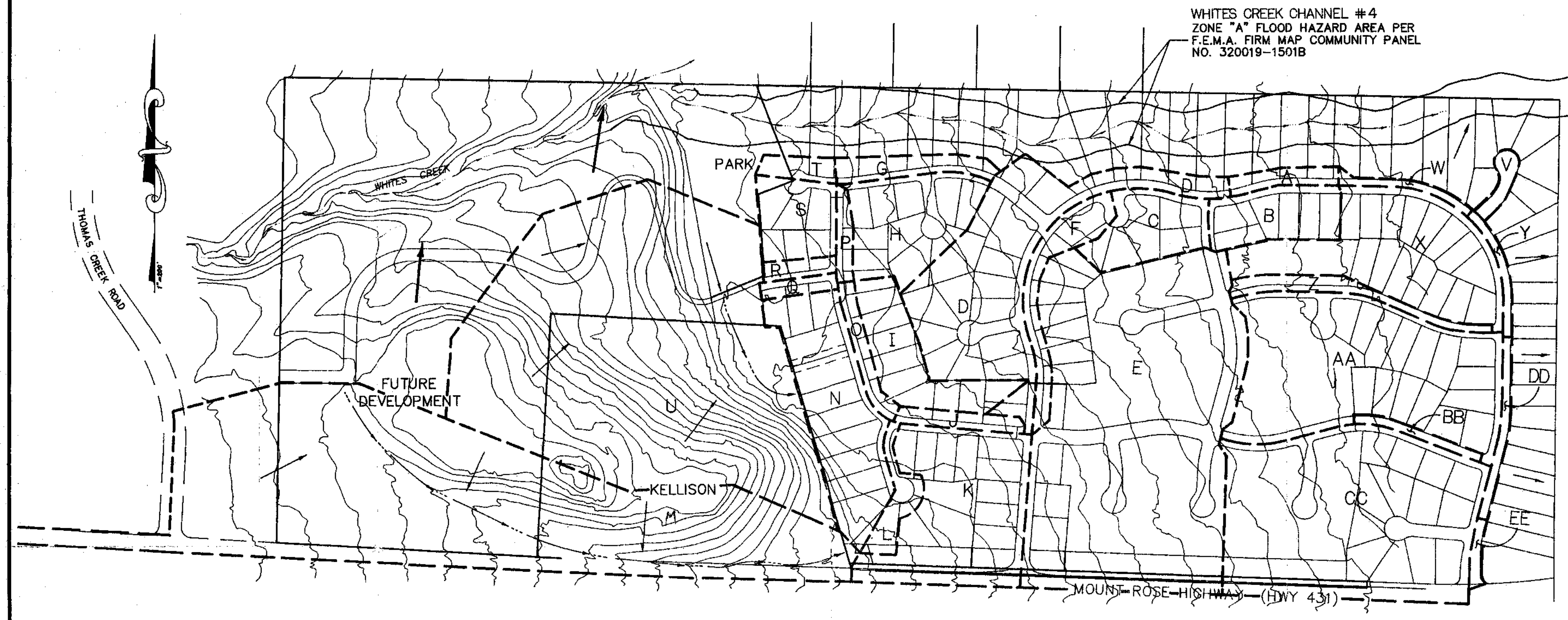

TABLE I

RATIONAL METHOD STORM DRAIN CALCULATIONS
FOR
INDIVIDUAL SUBAREAS

DRAINAGE SUBAREA	AREA (ACRES)	C	tc MIN	I10 IN/HR	Q10 CFS	I100 IN/HR	Q100 CFS
A	0.39	0.40	10	1.39	0.51	3.80	1.05
B	2.00	0.40	10	1.85	1.92	3.80	5.95
C	2.75	0.40	10	1.59	2.04	3.80	4.10
D	6.18	0.50	10	1.85	5.78	3.80	13.95
E	23.55	0.40	10	1.85	17.43	3.80	35.80
F	2.08	0.50	10	1.85	1.84	3.80	3.16
G	1.18	0.40	10	1.85	0.87	3.80	1.79
H	4.21	0.40	10	1.85	3.13	3.80	6.39
I	3.00	0.40	10	1.85	2.22	3.80	4.85
J	0.71	0.40	10	1.85	0.52	3.80	1.08
K	3.54	0.40	10	1.85	5.58	3.80	11.46
L	1.60	0.40	10	1.85	0.79	3.80	1.57
M	32.12	0.30	10	1.40	13.46	2.60	26.08
N	5.84	0.46	10	1.85	4.31	3.80	8.68
O	0.82	0.40	10	1.85	0.61	3.80	1.25
P	0.65	0.40	10	1.85	0.41	3.80	0.84
Q	0.60	0.30	10	1.85	0.33	3.80	0.68
R	0.55	0.30	10	1.85	0.31	3.80	0.63
S	2.30	0.40	10	1.85	1.72	3.80	3.53
T	0.80	0.40	10	1.85	0.49	3.80	1.00
U	30.34	0.30	10	1.85	16.83	3.80	34.59
V	0.42	0.40	10	1.85	0.31	3.80	0.63
W	0.50	0.40	10	1.85	0.37	3.80	0.76
X	7.50	0.80	10	1.85	5.56	3.80	11.40
Y	0.45	0.40	10	1.85	0.33	3.80	0.68
Z	1.21	0.40	10	1.85	0.89	3.80	1.84
AA	12.10	0.40	10	1.85	8.95	3.80	18.39
BB	0.38	0.40	10	1.85	0.28	3.80	0.57
CC	14.87	0.40	10	1.85	11.00	3.80	22.00
DD	0.52	0.40	10	1.85	0.38	3.80	0.79
EE	0.30	0.60	10	1.85	0.22	3.80	0.46

DATE PRINTED
APR 2 5 1994
ODYSSEY ENGINEERING INC.

LANCER ESTATES
STORM DRAINAGE SUB-AREAS & FLOWS

lancdran.dwg

REV.	DATE	DESCRIPTION	BY	APP'D

DATE: RCH 1994
DRAWN BY: D.B.
DESIGNED BY: J.W.
CHECKED BY: J.W.

LANCER ESTATES
STORM DRAINAGE
SUB-AREAS & FLOWS
WASHOE COUNTY, NEVADA

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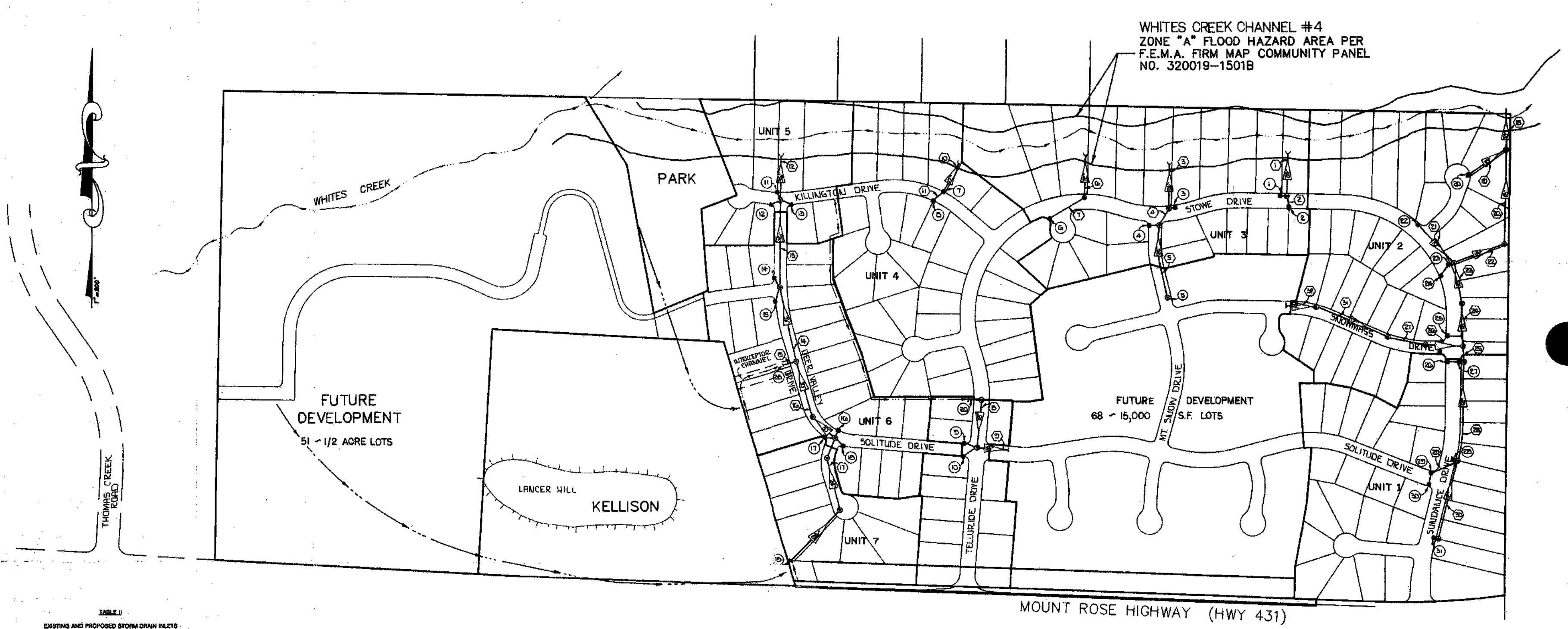


TABLE II

EXISTING AND PROPOSED STORM DRAIN INLETS

STORM DRAIN INLET	TYPE	INLET CAPACITY	LOCATION	SUBAREA SERVED	100 YEAR STORM BYPASS FLOW
1	TYPE 4-R 5.2%	6.5 CFS	UNIT 3 – N. SIDE OF STOWE DR.	A	15.09
2	TYPE 4-R 6.2%	2.2 CFS	UNIT 3 – S. SIDE OF STOWE DR.	B	7.33
3	TYPE 4-R 5.4%	6.8 CFS	UNIT 3 – N. SIDE OF STOWE DR.	D	20.04
4	TYPE 1 SUMP	3.4 CFS	UNIT 3 – N.W. CORNER OF STOWE DR. & MT. SNOW	C	6.78
5	24" CMP SUMP 3' HEAD	29 CFS	UNIT 3 – S. END OF MT. SNOW	E	5.80
6	TYPE 1 SUMP	3.4 CFS	UNIT 3 – TARGHEE CT.	F	6
7	TYPE 4-R 5%	1.79 CFS	UNIT 4 – N. SIDE OF KILLINGTON DR.	G	0
8	TYPE 4-R 5%	4.6 CFS	UNIT 4 – S. SIDE OF KILLINGTON DR.	H	1.49
9	TYPE 4-R 6.5%	1.8 CFS	UNIT 4 – N.W. CORNER OF SOLITUDE DR. & TELLURIDE DR.	J	0
10	TYPE 4-R 8.5%	5.4 CFS	UNIT 4 – S.W. CORNER OF SOLITUDE DR. & TELLURIDE DR.	K	11.40
11	TYPE 4-R 4.6%	1.00 CFS	UNIT 5 – N. SIDE OF KILLINGTON DR.	T	0
12	TYPE 4-R SUMP	6.2 CFS	UNIT 5 – S.W. CORNER OF KILLINGTON DR. & DEERVALLEY DR.	S	0
13	TYPE 4-R 6%	0.94 CFS	UNIT 5 – S.E. CORNER OF KILLINGTON DR. & DEERVALLEY DR.	P	0
13	TYPE 4-R 4%	0.83 CFS	UNIT 5 – N.W. CORNER OF DEERVALLEY DR. & WHISTLER MTN.	Q	0
15	TYPE 4-R R%	2.56 CFS	UNIT 6 – S.W. CORNER OF DEERVALLEY DR. & WHISTLER MTN.	Q	3
14	TYPE 4-R .5%	1.2 CFS	UNIT 5 – N. SIDE OF DEERVALLEY DR. & SOLITUDE DR.	O	6
17	TYPE 4-R SUMP	6.2 CFS	UNIT 5 – S.W. CORNER OF DEERVALLEY DR. & DEERVALLEY CT.	N	7.27

STORM DRAIN INLET	TYPE	INLET CAPACITY	LOCATION	SUBAREA SERVED	100 YEAR STORM BYPASS FLOW
18	TYPE 4-R .4%	3.5 CFS	UNIT 6 – S.E. CORNER OF DEERVALLEY CT. & SOLITUDE DR.	L	5.34
19	24" RCP 3' HEAD	29 CFS	UNIT 7 – COMMON AREA AT MT. ROSE HIGHWAY	M	0
20	18" RCP SUMP 1.5' HEAD	12 CFS	UNIT 4 – W. SIDE OF TELLURIDE DR.	I	0
21	TYPE 1 SUMP	*	UNIT 3 – N. END OF STOWE CT.	V	*
22	TYPE 1 6.6%	*	UNIT 2 – N. CORNER OF STOWE CT. & STOWE DR.	W	*
23	TYPE 1 SUMP	*	UNIT 3 – E. SIDE OF SUNDANCE DR.	Y	*
24	TYPE 1 SUMP	*	UNIT 3 – W. SIDE OF SUNDANCE DR.	X	*
25	TYPE 1 2.6%	*	UNIT 3 – N.W. CORNER OF SUNDANCE DR. & SNOWMASS DR.	Z	*
26	2 - TYPE 1 .4% & 7%	*	UNITS 1 & 2 – S.W. CORNER OF SUNDANCE DR. & SNOWMASS DR.	AA	*
27	TYPE 1 HI PT.?	*	UNIT 1 – S.E. CORNER OF SUNDANCE DR. & SNOWMASS DR.	BB	*
28	TYPE 1 2%	*	UNIT 1 – E. SIDE OF SUNDANCE DR.	DD	*
29	TYPE 1 5%	*	UNIT 1 – N.W. CORNER OF SUNDANCE DR. & SOLITUDE DR.	BB	*
29	TYPE 1 5%	*	UNIT 1 – S.W. CORNER OF SUNDANCE DR. & SOLITUDE DR.	CC	*
31	TYPE 1 8.3%	*	UNIT 1 – E. SIDE OF SUNDANCE DR.	EE	*
32	24" RCP SUMP 3.0' HEAD	30 CFS	STUB TO KELLISON PROPERTY	U	4.29

* DUE TO IRREGULAR STREET CROSS SECTION, ASPHALT BERM CURB, AND MULTIPLE CURB OPENINGS TO DOWNHILL DRIVEWAYS, IT IS NOT POSSIBLE TO ESTIMATE CATCH BASIN CAPACITIES IN UNITS 1 & 2.

TABLE III

STORM DRAIN CAPACITIES AND CALCULATED FLOWS

STORM DRAIN PIPE	SIZE & SLOPE	FREE FLOW CAPACITY	SUBAREA SERVED	COMBINED AREA (ACRES)	COMBINED C	COMBINED tc (MINUTES)	COMBINED I15 (CFS)	COMBINED Q15 (CFS)	COMBINED I100 (CFS)	COMBINED Q100 (CFS)
1	12" RCP @ .75%	9.1 CFS	A,B	3.29	0.4	16	1.85	2.43	3.6	* 6.2 (1)
2	12" RCP @ .94%	5.4 CFS	A	0.69	0.4	16	1.85	0.51	3.6	3.2 (1)
3	24" RCP @ 2.04%	31 CFS	C,D,E,I,J,K	46.79	0.4	13.6	1.55	29	3.2	* 49.2 (1)
4	24" RCP @ .62%	15 CFS	C,E,I,J,K	37.91	0.4	13	1.6	* 24.07	3.25	* 45.9 - 6.58 = 42.32 (1)
5	24" RCP @ .56%	17 CFS	E,I,J,K	34.85	0.4	12.5	1.65	* 23	3.3	* 46.0 - 3.9 = 39.2 (1)
4	12" RCP @ 3.15%	5.2 CFS	F	2.00	0.4	18	1.95	1.56	3.6	* 3.16
7	12" RCP @ 4.26%	7.3 CFS	F	2.99	0.4	16	1.90	1.54	3.6	* 5.16
3	13" RCP @ .50%	7.5 CFS	I	3	0.4	16	1.85	2.22	3.6	4.56
5	10" RCP @ .50%	7.3 CFS	I,J,K	11.25	4.4	18	1.85	* 8.33	3.6	* 11.0 (1)
10	12" RCP @ 3.5%	7 CFS	G,H	5.36	8.4	16	1.84	3.99	3.6	6.7 (1)
11	12" RCP @ 2.6%	5.8 CFS	H	4.21	8.4	16	1.85	3.1S	3.6	4.9 (1)
12	36" RCP @ .40%	42 CFS	L,M,N,O,P,Q,R,S,T,U	74.64	0.32	20	1.2	28.73	2.4	* 57.45 - 5.34 = 52.14 (1)
13	36" RCP @ .40%	48 CFS	L,M,N,O,Q,R,U	71.3	0.31	18	1.38	27.14	2.5	* 55.25 - 5.34 = 49.91 (1)
14	36" RCP @ .30%	36 CFS	L,M,N,O,U	70.16	0.52	18	1.6	25.18	2.66	* 59.43 - 5.34 = 24.15
13	24" RCP @ 2.5%	34 CFS	U	30.34	0.3	10	1.85	16.66	3.6	56.56 - 4.58 = 30.00 (1)
16	30" RCP @ .30%	23 CFS	L,M,N,O	39.41	0.52	17	1.85	17.2	2.75	* 38.00
17	24" RCP @ 3.4%	41 CFS	M	32.12	0.3	16	1.4	13.8	2.6	26.98
1S	21" RCP @ .65%	13 CF4	V,W,X,Y,Z,AA,BB,CC,DD,EE	38.26	0.4	18	1.85	29.80 (2)	3.6	58.14 (2)
16	10" CP @ 6%	5.4 CFS	V	0.42	0.4	16	1.85	0.31 (2)	5.9	0.64 (2)
20	18" RCP @ .65%	16 CFS	V,W,X,Y,Z,AA,BB,CC,DD,EE	37.00	0.4	10	1.85	27.69 (2)	3.6	57.56 (2)
21	10" CP @ 5.6%	4.4 CF3	W	0.5	[illegible]	18	1.85	0.37 (2)	3.6	0.76 (2)
27	18" RCP @ 4%	21 CFS	V,W,X,Z,AA,BB,CC,DD,EE	37.99	0.4	16	1.85	27.99 (2)	3.6	57.50 (2)
34	18" RCP @ 2.7%	17.5 CFS	Z,AA,BB,CC,DD,EE	29.38	0.4	16	1.85	21.74 (2)	3.6	44.66 (2)
24	18" RCP @ 2.7%	17.3 CFS	Z,AA,BB,CC,DD,EE	29.38	0.4	18	1.86	21.74 (2)	3.8	44.68 (2)
28	12" RCP @ 1.8%	4.5 CFS	AA,BB,CC,DD,EE	28.17	0.4	10	1.85	20.65 (2)	5.6	42.62 (2)
26	18" RCP @ 8.4%	23 CFS	AA	12.1	0.4	16	1.86	6.95 (2)	3.5	16.35 (2)
27	12" RCP @ 9%	11 CFS	BB,CC,DD,EE	16.07	0.4	16	1.86	11.90 (2)	3.9	24.43 (2)
28	12" RCP @ 1%	3.5 CFS	BB,CC,DD,EE	16.07	0.4	16	1.85	11.56 (2)	3.6	24.45 (2)
29	12" RCP @ 4.3%	7.5 CFS	BB,CC	15.20	0.4	16	1.85	11.29 (2)	3.6	23.16 (2)
30	12" RCP @ 3.3%	5.5 CFS	EE	5.9	0.4	16	1.86	0.22 (2)	3.6	0.45 (2)
31	12" RCP @ 4%	7 CFS	O		NO	INLET				
32	12" RCP @ 2%	5 CFS	6		NO	INLET				

* PIPE WILL OPERATE UNDER PRESSURE FLOW

(1) FLOW IN PIPE ASSUMING INLET CAPACITIES PER TABLE II

(2) UNIT 1 & 2 Q15 & Q100 YEAR FLOWS IF DRAINAGE GETS INTO INLETS

DATE PRINTED
APR 25 1994
ODYSSEY ENGINEERING INC.

LANCER ESTATES
EXISTING & PROPOSED STORM DRAINAGE FACILITIES

lancdran.dwg

REV.	DATE	DESCRIPTION	BY	APP'D

DATE: ...RCH 1994
DRAWN BY: D.B.
DESIGNED BY: J.W.
CHECKED BY: J.W.

LANCER ESTATES
EXISTING AND PROPOSED
STORM DRAINAGE FACILITIES
WASHOE COUNTY, NEVADA

odyssey ENGINEERING INCORPORATED
2105 CAPURRO WAY, SUITE E, SPARKS, NV 89431
(702) 359-3303 FAX (702) 359-3329

SCALE
HORIZ. 1"=200'
VERT. X
JOB NO. 1178

SHEET
2
OF
2

F I L E D
Electronically
2015-02-13 04:49:23 PM
Jacqueline Bryant
Clerk of the Court
Transaction # 4818450 : melwood

Exhibit 11

Exhibit 11

Lancer Estates Unit 9

HYDROLOGY REPORT

FOR

LANCER ESTATES UNIT 8 & 9

PREPARED BY

2105 CAPURRO WAY, SUITE F. SPARKS, NV 89431
(702) 359-3303 FAX (702) 359-3329

Odyssey ENGINEERING INCORPORATED

APRIL 1995

STORM DRAIN ANALYSIS

LANCER ESTATES UNIT NO.'S 8 & 9

INTRODUCTION

Lancer Estates Unit No.'s 8 and 9 are two subdivision projects consisting of 68 lots with a minimum size of 15,000 square feet. The projects are a continuation of the on-going Lancer Estates Project which is located in the South One-Half (S 1/2) of Section Thirty (30), Township Eighteen (18) North, Range Twenty (20) East, Mount Diablo Meridian. The site is bordered on the south by Mount Rose Highway. Approximately 180 lots have been developed in the project, which is approved for an ultimate buildout of approximately 309 units.

The purpose of this study is to inventory and analyze the existing storm drain facilities and flow patterns with respect to the proposed storm drain improvements in Units 8 and 9.

EXISTING STORM DRAIN PATTERNS AND FLOWS

The Lancer Estates site slopes generally from the southwest to the northwest with an average gradient of about six percent. The Mount Rose Highway borders the site on it's south side, and the north one-half of the roadway adjacent to the project drains onto Lancer Estates. Whites Creek crosses the northwest corner of the Lancer Estates site. Near the intersection of Whites Creek and the north boundary of Lancer Estates, Whites Creek splits into four channels. The southern most of these, which has been designated channel No. 4 in the Basin Management Study conducted by Cella Barr Associates, intercepts most of the drainage from Lancer Estates. Channel No. 4 is designated as zone A Flood Hazard Area by FEMA (Flood Community Panel No. 320019-1501B). The Cella Barr Study further delineates the area of Lancer Estates which are outside of Channel No. 4, and east of Lancer Hill, as area subject to minimal flooding from over the 100 year storm runoff in Whites Creek. Their study defines minimal flooding as less than 0.5' deep.

The enclosed map, sheet 1 of 2, shows the project site and features described above. The map also shows drainage subareas as they will exist after development of Lancer Estates Unit No.'s 8 and 9.

Storm drain runoff from the subareas has been calculated using the Rational Method.

The parameters used for these calculations are as follows:

Runoff Coefficient = C
C = 0.40
was used in existing and proposed areas

Time of Concentration = tc
Time of Concentration was calculated using a flow velocity of five feet per second and the following equation tc = 20 + L/60xV
Where L = Length in feet from top of the watershed to the inlet
V = overland velocity

Not new standard
10 or L/60xV

The parameters used for each subarea and the calculated 10 year and 100 year frequency storm runoff are listed in Table 1 on enclosed Plan Sheet 1 of 2.

EXISTING DEVELOPMENT AND DRAINAGE FACILITIES

As discussed previously, approximately 180 lots have been developed in Units 1 through 7 of the project. With reference to the Storm Drain Analysis for Lancer Estates 6 and 7, prepared by Odyssey Engineering Incorporated dated April 1994, plans for the existing units were reviewed to determine the locations and sizes of the storm drain facilities which were constructed. These facilities are shown on sheet 2 of 2.

In Units 1 and 2, the streets were constructed with asphalt berms instead of curb and gutter. In many areas the berms are depressed with downsloping driveways. In these areas, the streets have essentially no capacity to carry water. Since the amount of runoff carried by the street cannot be determined, we have not attempted to estimate the inlet capacity of these catch basins. Catch basin types and sizes are listed in table III.

In Units 3 through 7, concrete curb and gutter and driveway approaches were constructed. These improvements allow the street to contain storm runoff and direct it to inlets. The capacities of these inlets are also listed in Tables II and III on Sheet 2 of 2.

PROPOSED UNIT 8 AND 9 DRAINAGE SYSTEM

The construction of Units 2 and 3 have blocked the natural drainage path from the westerly part of the site to drainage channel No. 4. The drainage facilities that were constructed with these units have a limited capacity. The storm drain system in Units 5, 6 and 7 were assigned to intercept much of the westerly site drainage and transport it to Drainage Channel No. 4 (Ref. Storm Drain Analysis for Lancer Estates Units 6 and 7, prepared by Odyssey Engineering Incorporated dated April 1994).

Drainage from subareas I, J and K enter the site at the west end of Solitude Drive within Unit No. 9. A temporary drainage swale was constructed through Unit No.'s 8 and 9 to carry this water to a temporary inlet at Snowmass and Mt. Snow Drive. This temporary ditch will be rerouted with construction of Unit No. 8 and will be eliminated with construction of Unit 9.

A portion of Units 3 and 4 currently drains across Units 8 and 9 and eventually to Units 1 and 2. With construction of Units 8 and 9, the 10 year flows from these units will be piped through Unit No. 3 and eventually to drainage Channel No. 4 via a 24 inch storm drain stub within Mt. Snow Drive. A portion will also be piped through Unit No. 2 via a 12" diameter stub within Snowmass Drive.

Due to the existing six percent cross slope of Units 8 and 9, a small portion of Unit 8 in the northeast corner will drain into Unit No. 2. This is the westerly portion of subarea "X". The flow will enter an existing drainage easement and earth V-ditch at the northeast corner of lot 6.

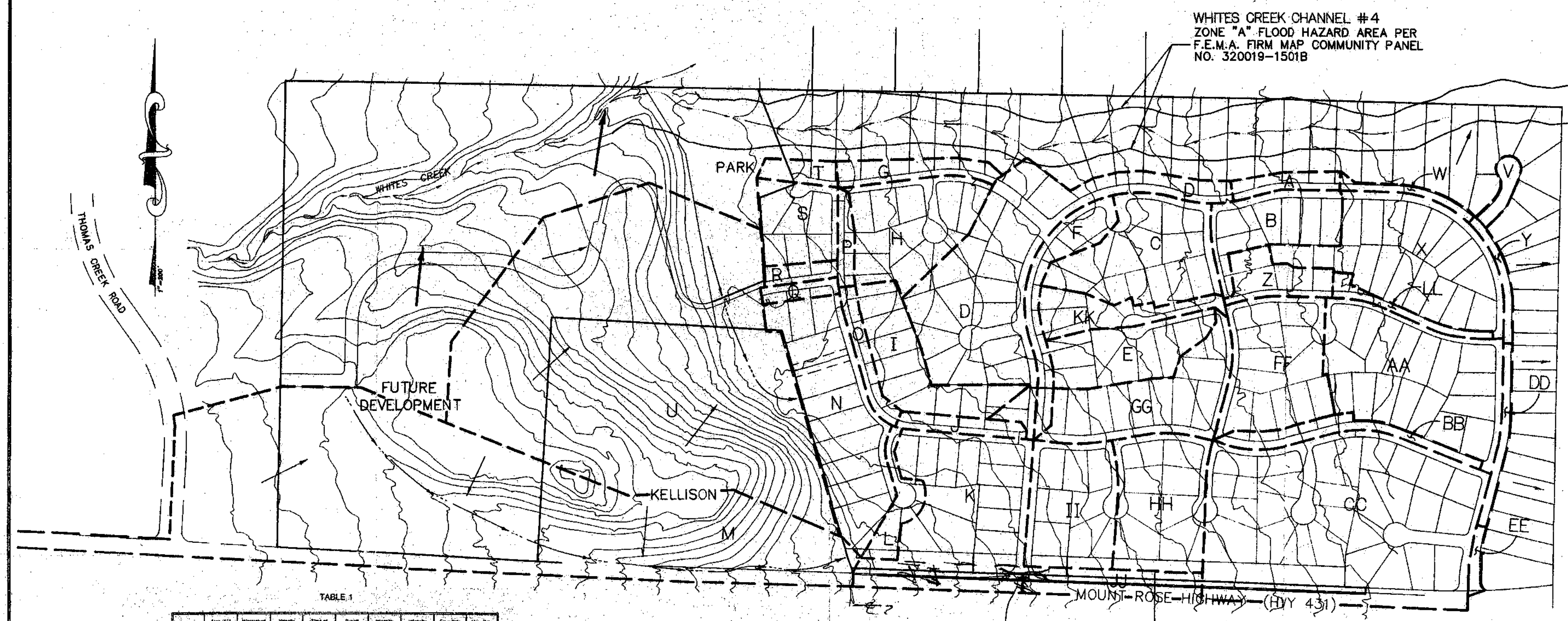
A portion of Unit 9 will drain into Unit No. 1. The west half of subarea "CC", a small portion of subarea "AA" and the west portion of subarea "BB" are the areas draining to Unit No. 1/. The flows from these areas will be contained within the streets of Unit 9 and will enter Unit No. 1 through Solitude Drive.

Currently the majority of Unit No.'s 8 and 9 drain through Unit No.'s 1 and 2. With construction of these units, the flows will be greatly decreased because of the street and storm drain system.

Table III on sheet 2 of 2 lists the proposed storm drain pipes in Units 1 through 9, along with free flow capacities, 10 year storm flows and 100 year storm flows. With reference to the Hydrology Study for Lancer 6 and 7, the flows and capacities that were not affected by Lancer 8 and 9 were not changed. As can be seen by comparing the 100 year storm flows with the pipe capacities, the 100 year flows exceeds the free flow capacity in several instances. In these cases the pipes will function under pressure. — Hydraulic Grade Line?

CONCLUSION

The construction of the storm drain system as proposed in Lancer Estates Units 8 and 9 will provide drainage protection for the proposed homes in these units, and will greatly improve the drainage protection for the existing units of Lacer Estates. The runoff which will bypass these new units during the 100 year storm event will be substantially less than what the existing condition runoff would be. The streets in the newer units will be capable of carrying the 100 year storm overflow. The streets in Units 1 and 2, however, will not contain any substantial storm water flows. In order to provide protection for homes in these areas, some reconstruction of streets, driveways and storm drain improvements would be required.



E?

18" CMP Under Telluride

TABLE 1

Drainage Area	Area (AC)	Watershed Length (FT)	Velocity (FT/Sec.)	Time of Concentration (Min.)	Runoff Coefficient	Intensity (10)	Intensity (100)	Flow Rate (10)	Flow Rate (100)
A	0.48	656	5	21.80	0.4	2.00	3.40	0.55	0.94
B	2.75	380	5	31.83	0.4	2.00	3.40	2.20	3.74
C	4.41	2000	5	28.67	0.4	1.75	2.90	3.09	5.12
D	9.38	2150	5	27.17	0.4	1.75	2.90	6.43	10.65
E	3.18	1600	5	26.00	0.4	1.87	2.95	2.29	3.75
F	2.08	900	5	23.00	0.4	1.95	3.30	1.62	2.75
G	3.10	*							
H	4.71	*							
I	3.00	1000	3	23.33	0.4	1.95	3.30	2.34	3.96
J	0.71	600	5	23.00	0.4	1.95	3.30	0.55	0.94
K	7.54	580	5	22.83	0.4	1.95	3.30	5.88	9.95
L	1.07	*							
M	32.12	*							
N	5.84	*							
O	0.82	*							
P	5.55	*							
Q	3.60	*							
R	5.55	*							
S	2.55	*							
T	0.68	*							
U	10.34	*							
V	0.42	2050	5	28.83	0.4	1.75	2.90	0.29	0.49
W	0.30	1600	5	26.00	0.4	1.85	2.95	0.30	0.54
X	6.85	1700	5	25.67	0.4	1.80	2.95	5.00	8.20
Y	0.45	1750	5	30.83	0.4	1.80	2.85	0.32	0.53
Z	1.31	1350	5	24.5	0.4	1.90	3.15	1.00	1.65

* Existing System (Ref. Lancer 6 and 7)

TABLE 1 CONTINUED

Drainage Area	Area (AC)	Watershed Length (FT)	Velocity (FT/Sec.)	Time of Concentration	Runoff Coefficient	Intensity (10)	Intensity (100)	Flow Rate (10)	Flow Rate (100)
AA	6.34	1500	5	25.00	0.4	1.85	3.00	4.84	7.65
BB	0.91	1200	5	24.00	0.4	1.90	3.15	0.69	1.15
CC	16.25	1200	5	24.00	0.4	1.90	3.10	12.35	20.48
DD	0.52	1600	5	25.00	0.4	1.85	3.00	0.38	0.62
EE	0.30	1100	5	23.67	0.4	1.90	3.17	0.23	0.38
FF	4.67	1200	5	24.00	0.4	1.90	3.15	3.70	6.14
HH	4.50	1500	5	25	0.4	1.85	3.00	3.34	5.41
II	3.92	1150	5	23.83	0.4	1.90	3.15	2.98	4.94
JJ	3.61	1400	5	24.67	0.4	1.85	3.00	1.20	1.94
KK	1.82	1800	3	38	0.4	1.8	2.85	1.31	2.15
LL	0.40	1550	5	25.17	0.4	1.85	3.00	0.30	0.48
GG	3.65	1800	5	26.00	0.4	1.8	2.95	2.77	4.54

A-5?

No. 10504

4/17/95

LANCER ESTATES

STORM DRAINAGE SUB-AREAS & FLOWS

lancdran.dwg

REV.	DATE	DESCRIPTION	BY	APP'D

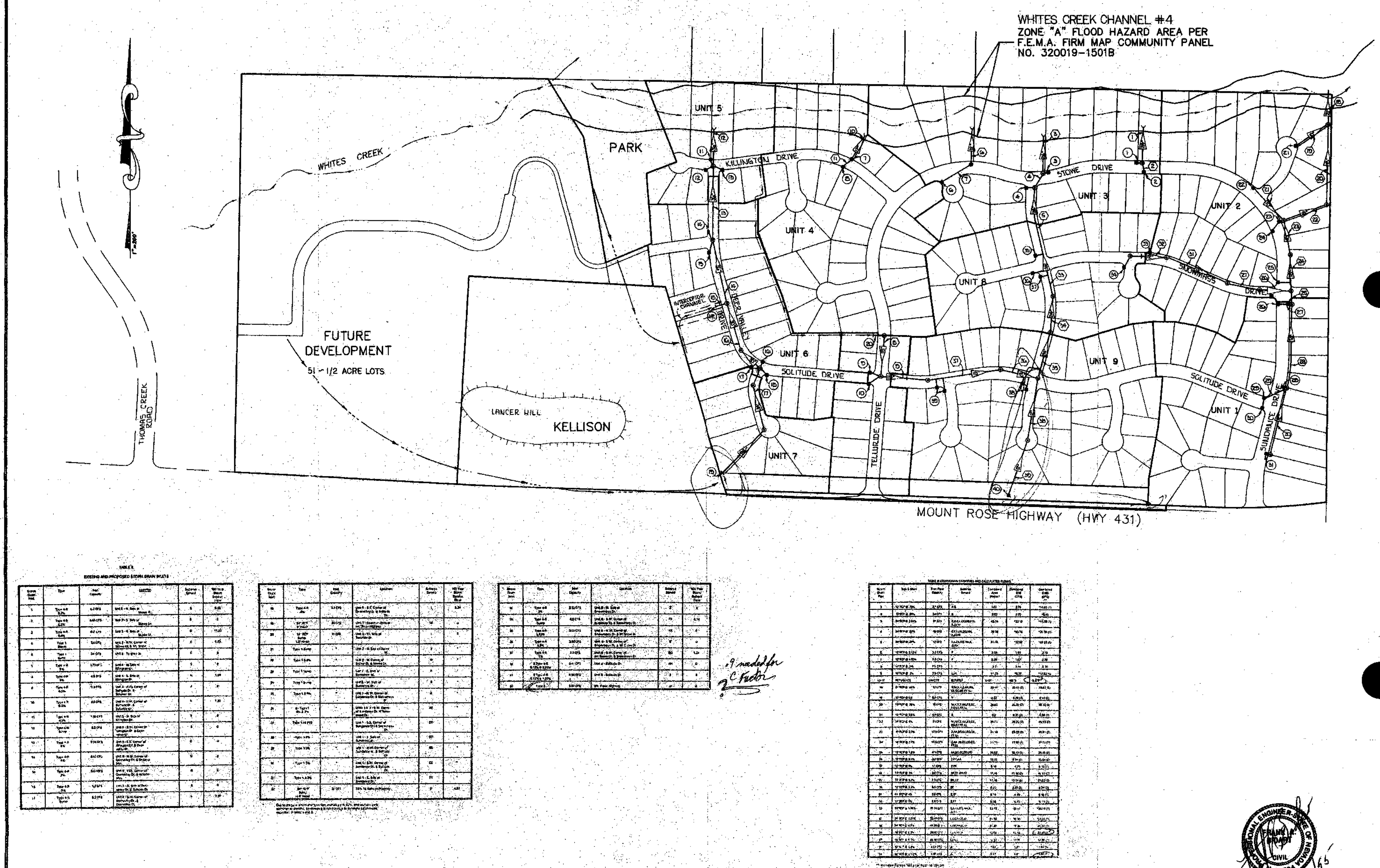
DATE: ...H 1994
DRAWN BY: D.B.
DESIGNED BY: J.W.
CHECKED BY: J.W.

LANCER ESTATES
STORM DRAINAGE
SUB-AREAS & FLOWS
WASHOE COUNTY, NEVADA

2105 CAPURRO WAY, SUITE F, SPARKS, NV 89431
(702) 359-3303 FAX (702) 359-3329
Odyssey ENGINEERING INCORPORATED

SCALE
HORIZ. 1"=200'
VERT. X
JOB NO. 1176

SHEET
1
OF
2



.9 needed for
C Factor
2

No. 10504

4/7/95

LANCER ESTATES
EXISTING & PROPOSED STORM DRAINAGE FACILITIES

lancdran.dwg

REV.	DATE	DESCRIPTION	BY	APP'D

DATE: ~~~~RCH 1994
DRAWN BY: D.B.
DESIGNED BY: J.W.
CHECKED BY: J.W.

LANCER ESTATES
EXISTING AND PROPOSED
STORM DRAINAGE FACILITIES
WASHOE COUNTY, NEVADA

2105 CAPURRO WAY, SUITE F, SPARKS, NV 89431
(702) 359-3303 FAX (702) 359-3329
Odyssey ENGINEERING INCORPORATED

SCALE
HORIZ. 1"=200'
VERT. X
JOB NO. 1176

SHEET 2 OF 2

F I L E D
Electronically
2015-02-13 04:49:23 PM
Jacqueline Bryant
Clerk of the Court
Transaction # 4818450 : melwood

Exhibit 12

Exhibit 12

Lancer Estates Unit 9

LANCER ESTATES UNIT 9

HYDROLOGY REPORT

Prepared for:

LANCER LTD., A JOINT VENTURE
C/O BARNESON INVESTMENTS INC.
4971 LAKERIDGE TERRACE WEST
RENO, NEVADA 89509

Prepared by:

FPE Engineering & Planning
4600 Kiezke Lane, Suite H-182
Reno, Nevada 89502

September 8, 1997

LANCER ESTATES UNIT 9

HYDROLOGY REPORT

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4971 LAKERIDGE TERRACE WEST
RENO, NEVADA 89509

Prepared by:

FPE Engineering & Planning
4600 Kiezke Lane, Suite H-182
Reno, Nevada 89502

REGISTERED PROFESSIONAL ENGINEER STATE OF NEVADA
HARLAN F. FRICKE
CIVIL
05508
9-15-97

September 8, 1997

INTRODUCTION

The proposed subdivision is located along Mount Rose Highway just west of the new Galena Creek Shopping Center. This site is a portion of SE1/4, SECTION 30, T.18N., R.20E., M.D.M.

Lancer Estates Unit 9 is a seven lot subdivision covering 3.32+/- acres. The native vegitation is typical of the area with slopes in excess of 3.00%.

A previous study of the area has been prepared by Odyssey Engineering, *"Hydrology Report for Lancer Estates Unit 8 & 9."*

HISTORIC CONDITION

The site is not currently developed with natural vegetation. The flows, as shown below, are overland flows to Solitude Drive where they enter an existing storm drain system at the intersection of Solitude Drive and Sundance Drive. Flow were calculated using the Rational Method.

$$Q_{10} = 14.65 \text{ cfs}$$

$$Q_{100} = 30.90 \text{ cfs}$$

PROPOSED CONDITION

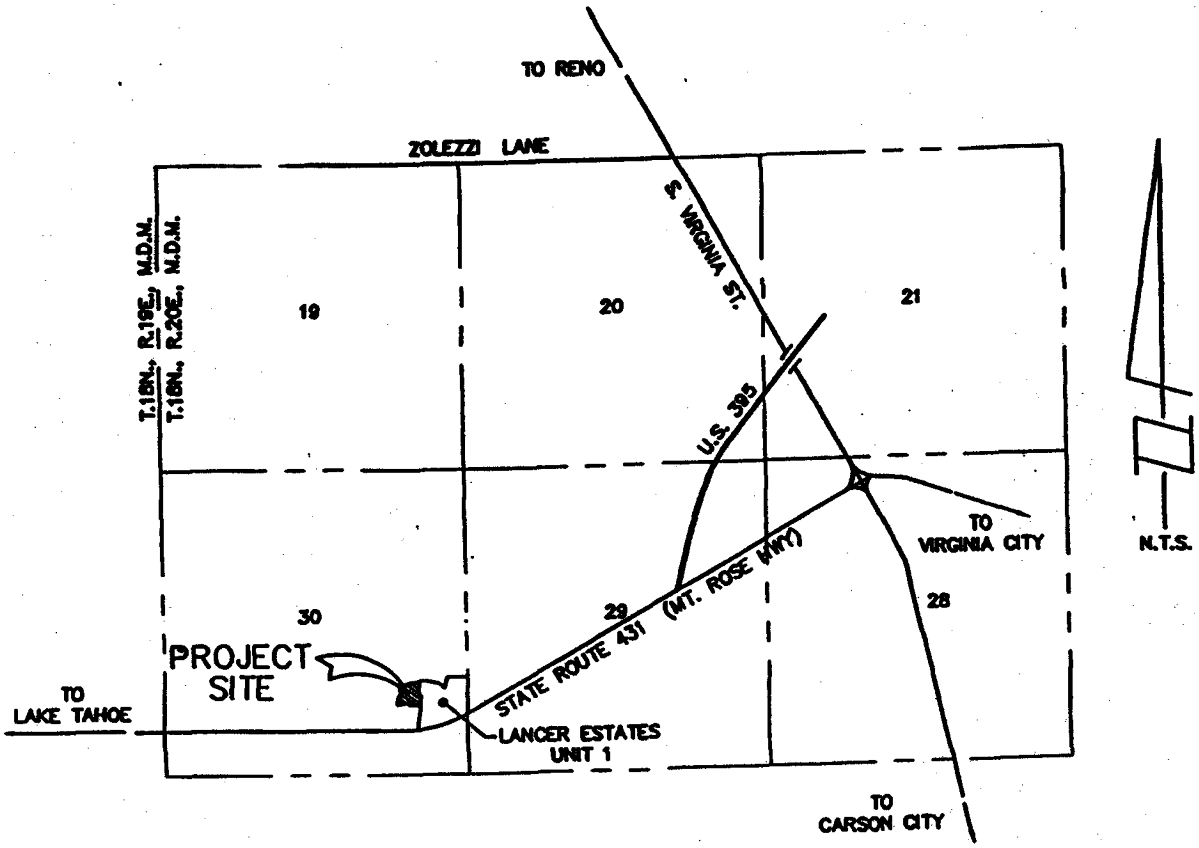
Proposed flows will from areas #2 and #3 will use the existing curb in Solitude Drive to direct it to the intersection of Solitude Drive and Sundance Drive where it will be collected by two recently constructed catch basins. Area #1 will follow an existing swale along a rear lot line of Lancer Estates Unit 1. See the attached calculations and Figure 2

CONCLUSION

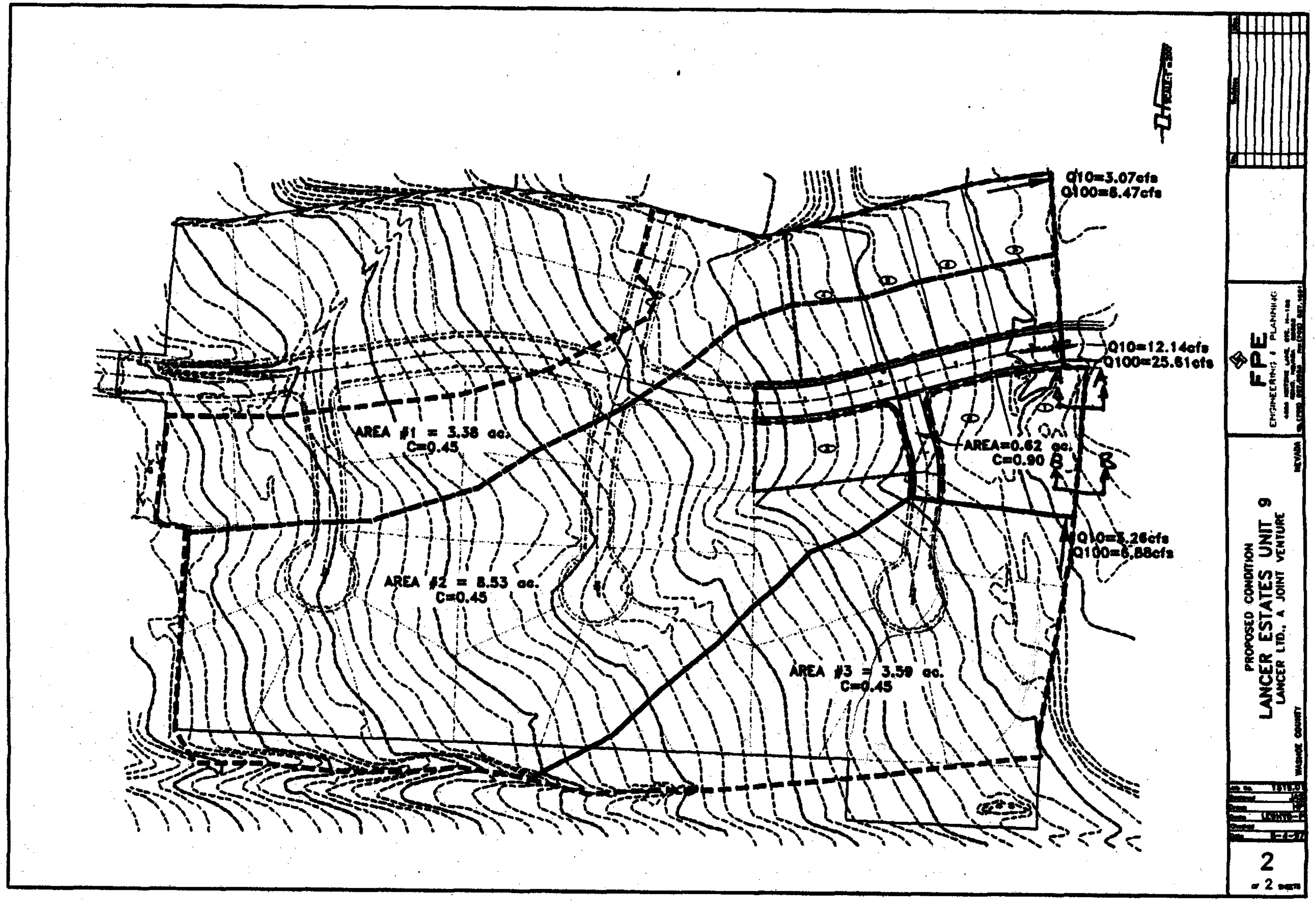
The flows generated by the proposed project will not be significantly increased. The development of Lancer Estates Unit 9 will ulitmately direct flows more efficiently to existing drainage facilities.

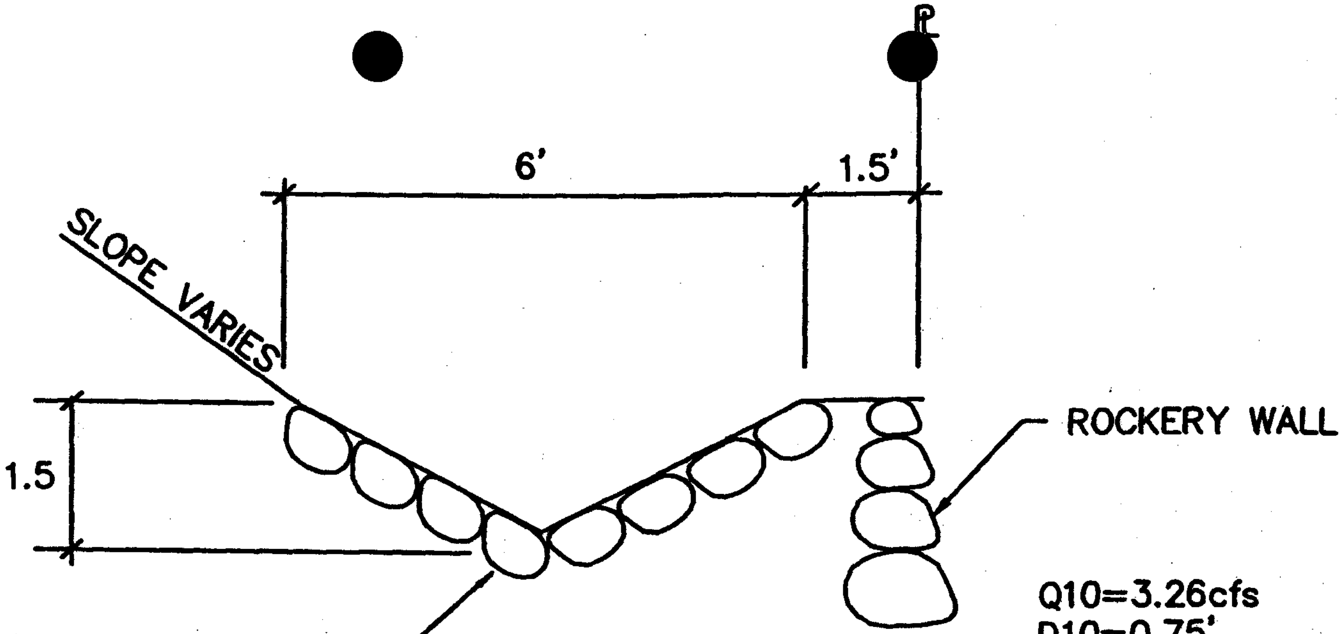
August 4, 1997
1519.01
LE9HYDRO.WPD

APPENDIX



VICINITY MAP

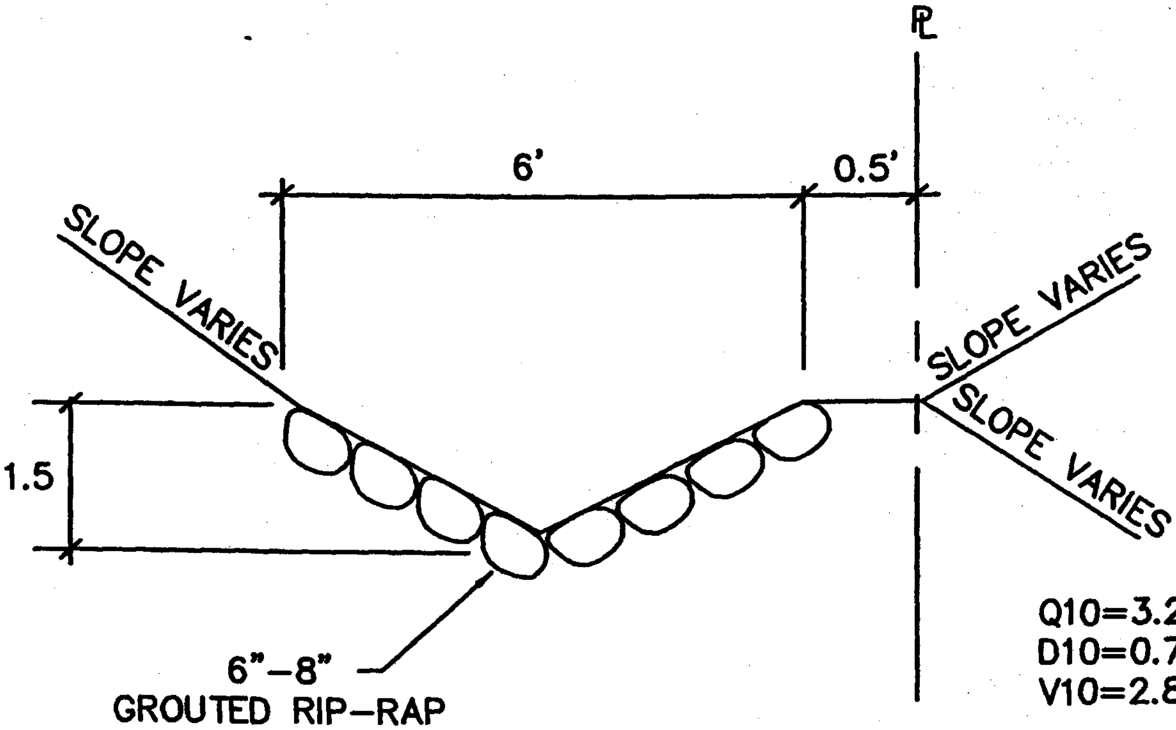

Q10=3.07cfs
Q100=8.47cfs
Q10=12.14cfs
Q100=25.61cfs
AREA=0.62 ac.
C=0.90
Q10=3.26cfs
Q100=6.88cfs
AREA #1 = 3.38 ac.
C=0.45
AREA #2 = 8.53 ac.
C=0.45
AREA #3 = 3.59 ac.
C=0.45
FPE
ENGINEERING & PLANNING
PROPOSED CONDITION
LANCER ESTATES UNIT 9
LANCER LTD., A JOINT VENTURE
WASHOE COUNTY
NEVADA
1518.01
2
of 2 sheets



Q10=3.26cfs
D10=0.75'
V10=2.87fps

Q100=6.88cfs
D100=1.00'
V100=3.46fps

SECTION A–A



Q10=3.26cfs
D10=0.75'
V10=2.87fps

Q100=6.88cfs
D100=1.00'
V100=3.46fps

SECTION B–B

Q10 Cross Section
Worksheet for Triangular Channel

Project Description	
Project File	t:\joe\docs\general.fm2
Worksheet	V-DITCH
Flow Element	Triangular Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.028	
Channel Slope	0.012500	ft/ft
Left Side Slope	2.000000	H : V
Right Side Slope	2.000000	H : V
Discharge	3.26	cfs

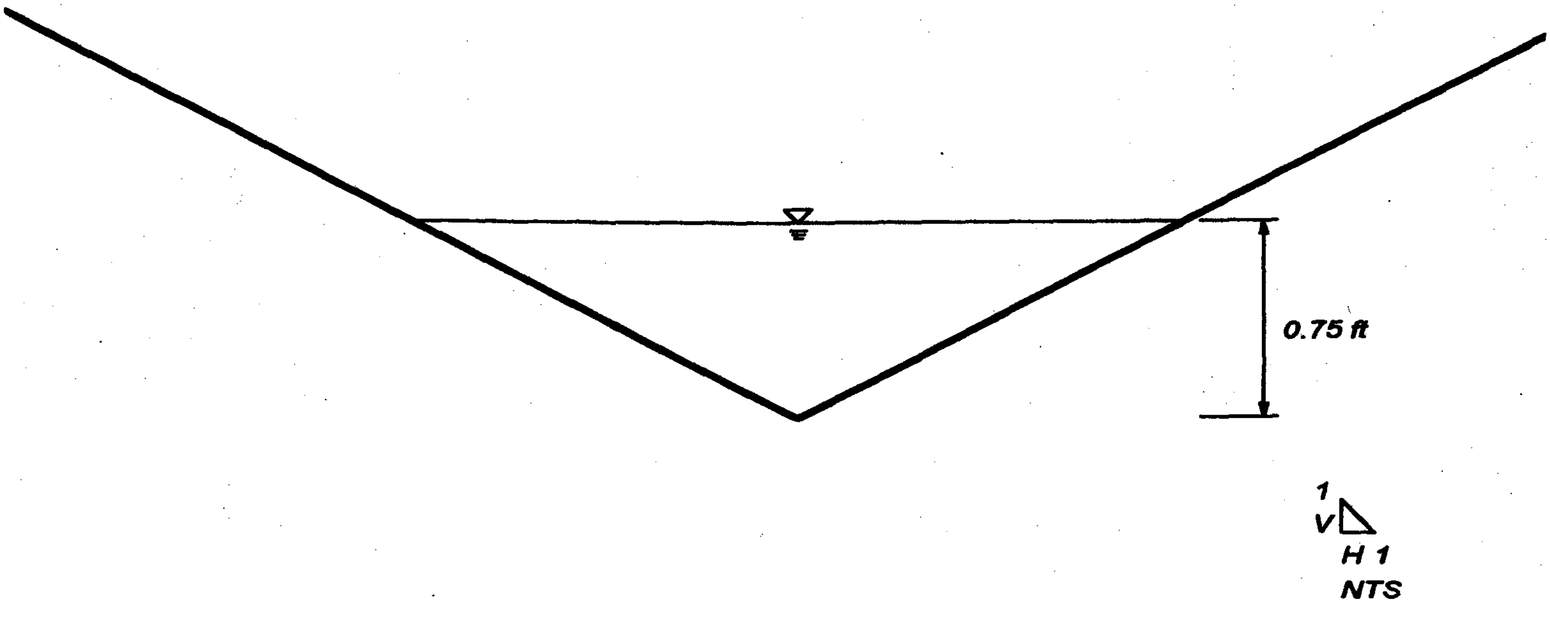
Results		
Depth	0.75	ft
Flow Area	1.13	ft²
Wetted Perimeter	3.37	ft
Top Width	3.01	ft
Critical Depth	0.70	ft
Critical Slope	0.018834	ft/ft
Velocity	2.87	ft/s
Velocity Head	0.13	ft
Specific Energy	0.88	ft
Froude Number	0.83	
Flow is subcritical.		

Q10 Cross Section

Cross Section for Triangular Chann

Project Description	
Project File	t:\joe\docs\general.fm2
Worksheet	V-DITCH
Flow Element	Triangular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.028	
Channel Slope	0.012500	ft/ft
Depth	0.75	ft
Left Side Slope	2.000000	H : V
Right Side Slope	2.000000	H : V
Discharge	3.26	cfs



Q100 Cross Section
Worksheet for Triangular Channel

Project Description	
Project File	t:\joe\docs\general.fm2
Worksheet	V-DITCH
Flow Element	Triangular Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.028	
Channel Slope	0.012500	ft/ft
Left Side Slope	2.000000	H : V
Right Side Slope	2.000000	H : V
Discharge	6.88	cfs

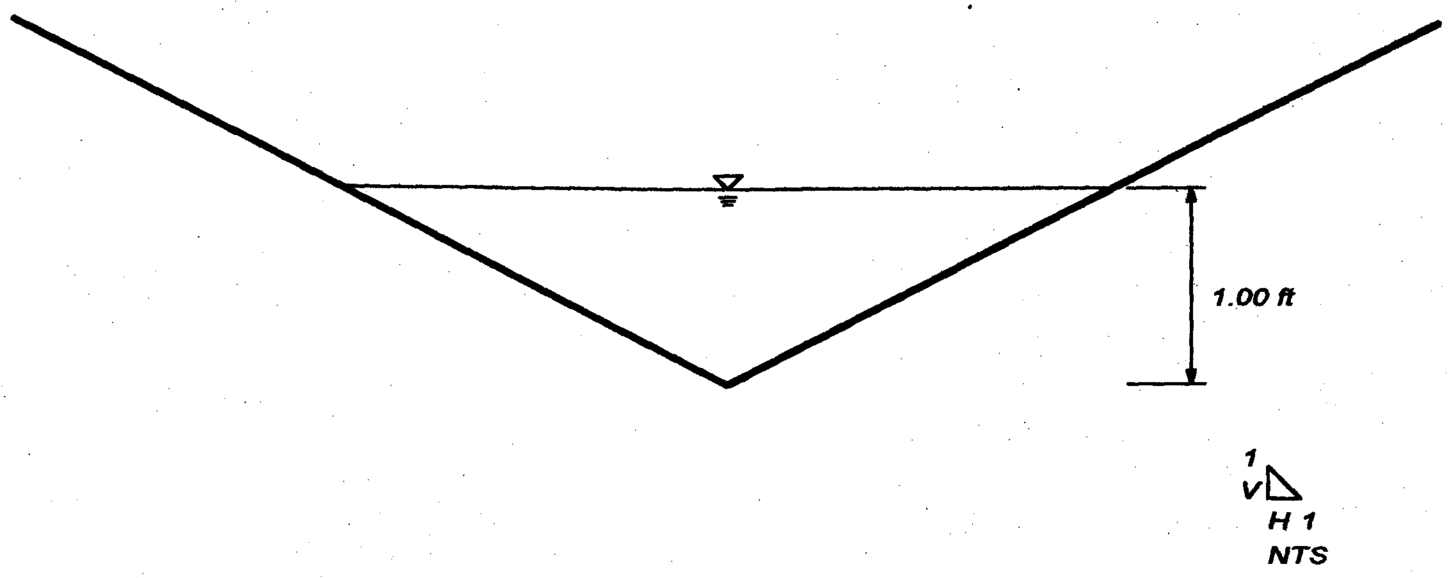
Results		
Depth	1.00	ft
Flow Area	1.99	ft²
Wetted Perimeter	4.46	ft
Top Width	3.99	ft
Critical Depth	0.94	ft
Critical Slope	0.017048	ft/ft
Velocity	3.46	ft/s
Velocity Head	0.19	ft
Specific Energy	1.18	ft
Froude Number	0.86	
Flow is subcritical.		

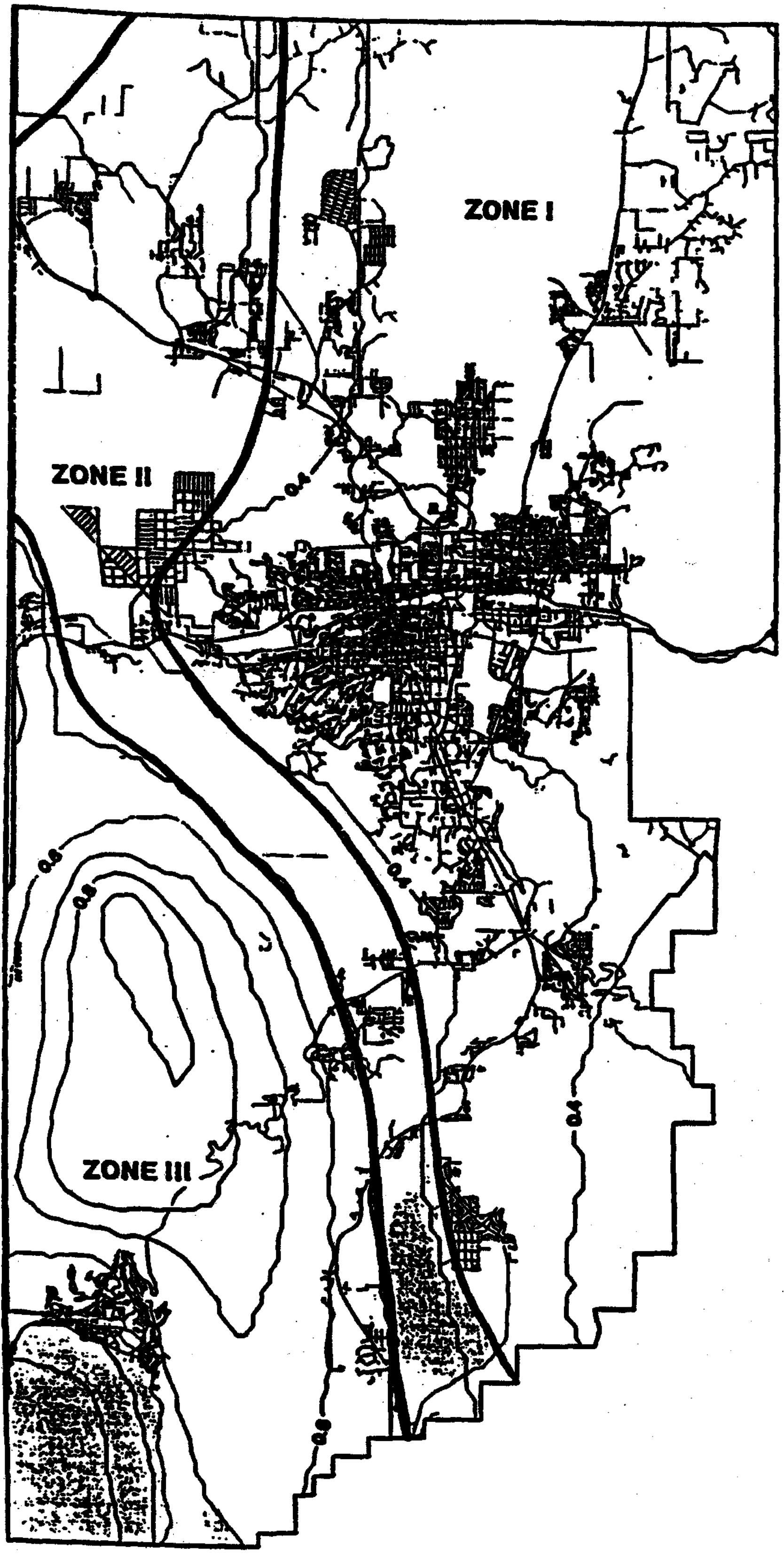
Q100 Cross Section

ross Section for Triangular Channe

Project Description	
Project File	t:\joe\docs\general.fm2
Worksheet	V-DITCH
Flow Element	Triangular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.028	
Channel Slope	0.012500	ft/ft
Depth	1.00	ft
Left Side Slope	2.000000	H : V
Right Side Slope	2.000000	H : V
Discharge	6.88	cfs





DRAFT

SOUTHERN
WASHOE COUNTY

PRECIPITATION FREQUENCY STUDY OF THE UNITED STATES, NOAA ATLAS 14, VOLUME 1 - SEMI-ARID SOUTHWEST UNITED STATES

2 YEAR 1 HOUR
PRECIPITATION
EVENTS (in inches)

— 0.4
— 0.5
— 0.6
— 0.7
— 0.8
— 0.9

SCALE IN FEET

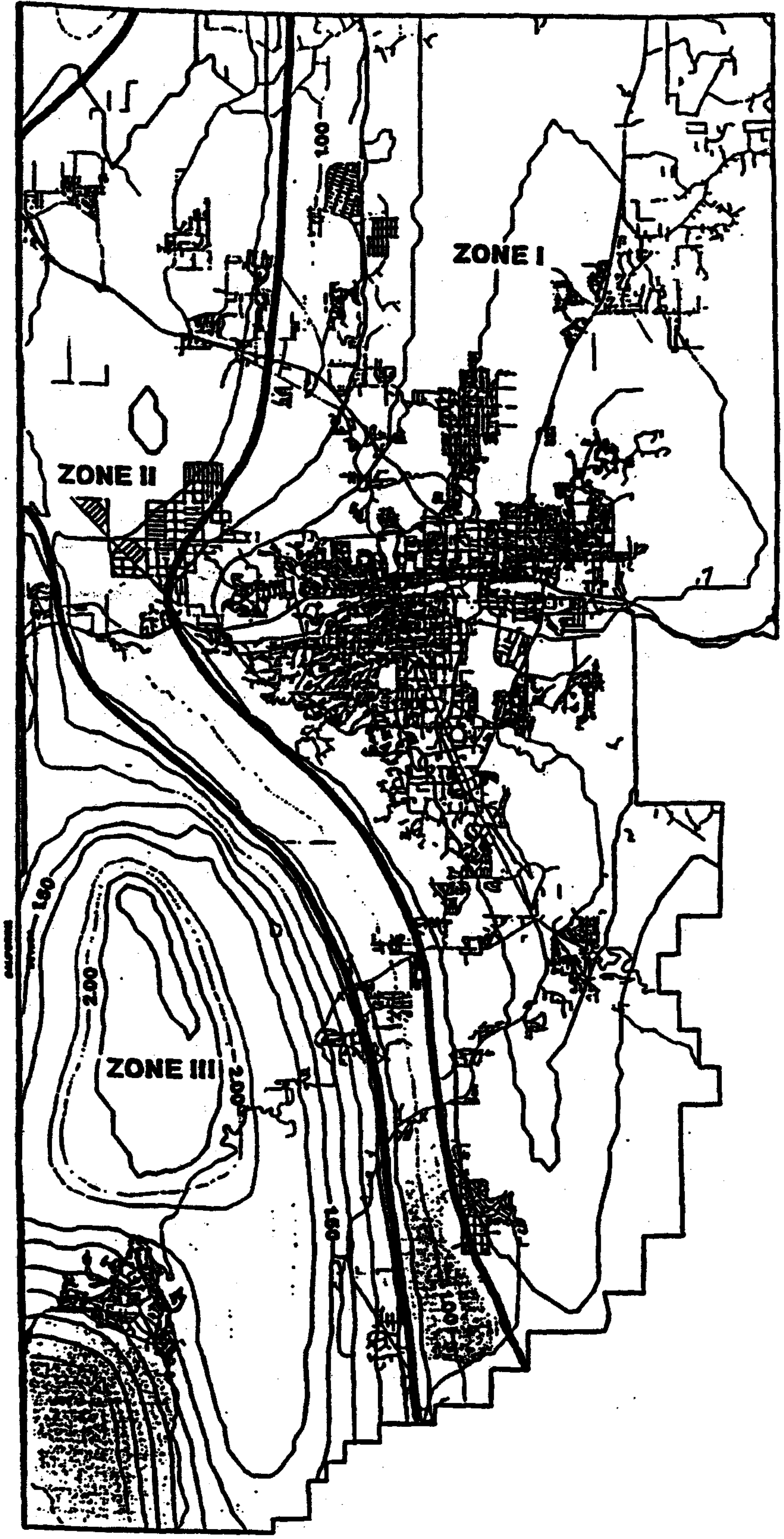
Department of Water Resources

**WASHOE COUNTY
NEVADA**

SOURCE: NATIONAL WEATHER SERVICE - SOUTHWEST SEMIARID PRECIPITATION FREQUENCY STUDY GROUP

DATE: MAY 1996

VERSION: 06-18-1996	REFERENCE:	FIGURE
WRC ENGINEERING INC		601



DRAFT

SOUTHERN
WASHOE COUNTY

PRECIPITATION FREQUENCY STUDY OF THE UNITED STATES, NOAA ATLAS 14, VOLUME 1 - SEMI-ARID SOUTHWEST UNITED STATES

2 YEAR 6 HOUR
PRECIPITATION
EVENTS (in inches)

—	0.70	—	1.50
—	0.80	—	1.60
—	0.90		1.70
	1.00		1.80
—	1.10	—	1.90
—	1.20	--	2.00
—	1.30	—	2.10
—	1.40	—	2.20

[illegible]

SCALE IN FEET

Department of Water Resources

WASHOE COUNTY NEVADA

[illegible]

SOURCE: NATIONAL WEATHER SERVICE - SOUTHWEST SEMIARID PRECIPITATION FREQUENCY STUDY GROUP

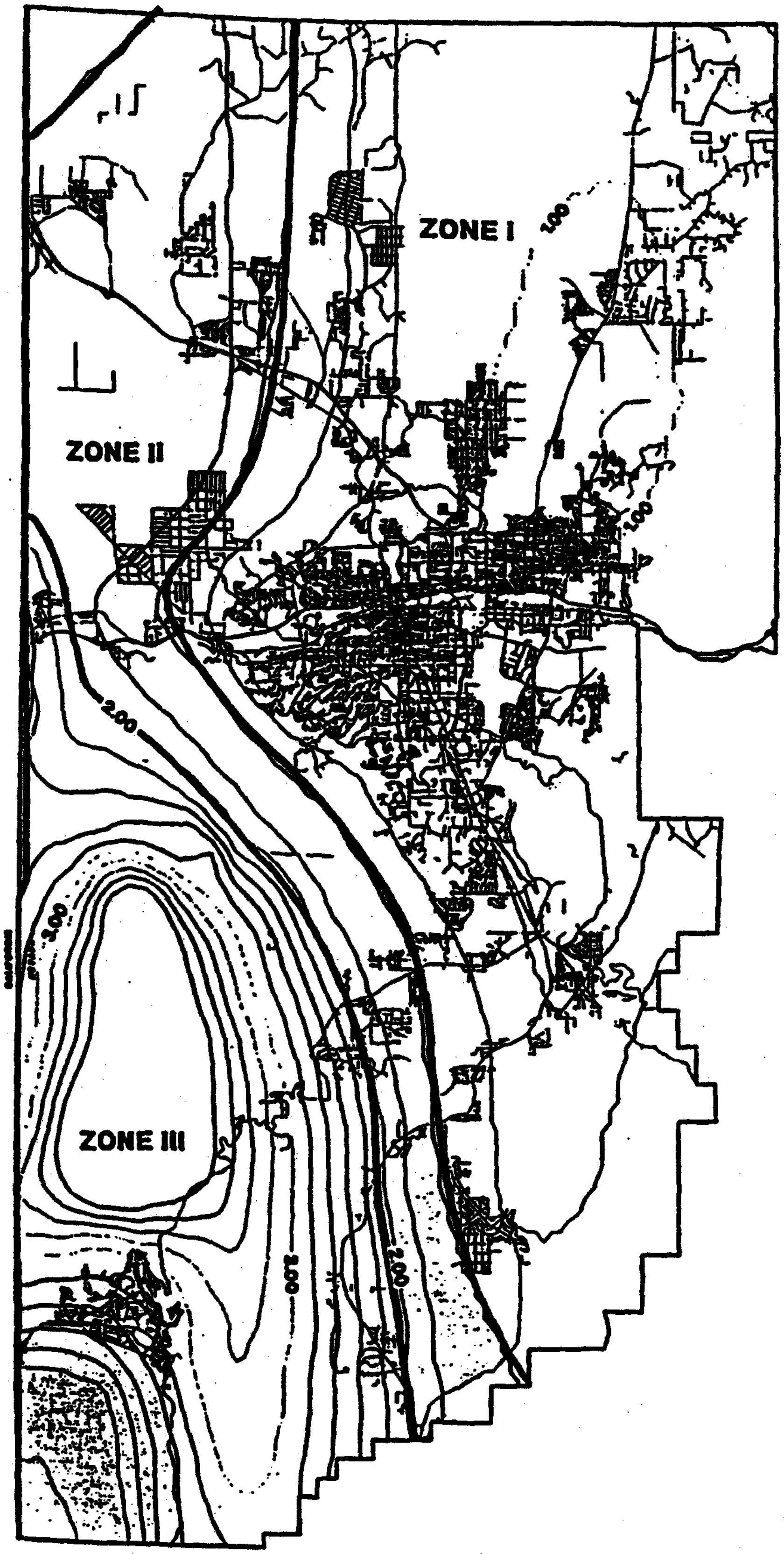
DATE: MAY [illegible]

VERSION: 06-18-1996

WRC ENGINEERING, INC.

REFERENCE:

FIGURE 602



DRAFT

SOUTHERN
WASHOE COUNTY

PRECIPITATION FREQUENCY STUDY OF THE UNITED STATES, NOAA ATLAS 14, VOLUME 1 - SEMI-ARID SOUTHWEST UNITED STATES

2 YEAR 24 HOUR
PRECIPITATION
EVENTS (in inches)

- - -	1.00	—	2.60
—	1.20	—	2.80
—	1.40		3.00
—	1.60		3.20
—	1.80	—	3.40
- - -	2.00	—	3.60
—	2.20	—	3.80
—	2.40		

SCALE IN FEET

Department of Water Resources

WASHOE COUNTY NEVADA

SOURCE: NATIONAL WEATHER SERVICE - SOUTHWEST [illegible] PRECIPITATION FREQUENCY STUDY GROUP

DATE: MAY [illegible]

VERSION: 06-18-1996	REFERENCE:
WRC ENGINEERING, INC.	

FIGURE 603

CALCULATIONS

PROJECT

JOB NO SHEET 2 OF

CALCULATED BY DATE

CHECKED BY DATE

EXISTING CONDITION

$C = 0.45$

$i_{10} = 2.02$ in/hr

$i_{100} = 4.26$ in/hr

$A = 16.12$ ac

$Q_{10} = (0.45)(2.02)(16.12) = 14.65$ cfs

$Q_{100} = (0.45)(4.26)(16.12) = 30.90$ cfs

PROPOSED CONDITION

$C = 0.45$ (NATIVE/LANDSCAPED AREAS)

$C = 0.90$ (PAVED AREAS)

$i_{10} = 2.02$ in/hr

$i_{100} = 4.26$ in/hr

— AREA #1

$A = 3.38$ ac.

$Q_{10} = (0.45)(2.02)(3.38) = 3.07$ cfs

$Q_{100} = (0.45)(4.26)(3.38) = 6.47$ cfs

— AREA #2

$A_{NATURAL} = 8.53$ ac

$A_{PAVED} = 0.62$ ac

$Q_{10} = [(0.45)(8.53) + (0.90)(0.62)](2.02) = 8.88$ cfs

$Q_{100} = [(0.45)(8.53) + (0.90)(0.62)](4.26) = 18.73$ cfs

— AREA #3

$A = 3.59$ ac

$Q_{10} = (0.45)(2.02)(3.59) = 3.26$ cfs

$Q_{100} = (0.45)(4.26)(3.59) = 6.88$ cfs

FPE
ENGINEERING & PLANNING
4600 KIETZKE LANE, STE. H-182 RENO, NV 89502
TEL (702) 827.8833 FAX (702) 827.1831

PROJECT

JOB NO. SHEET 4 OF

CALCULATED BY DATE

CHECKED BY DATE

– CONTRIBUTIONS OF PROPOSED CONDITION TO EXISTING SUBDIVISION.

AREA #1 WILL CONTRIBUTE 3.07cfs AND 6.47cfs TO AN EXISTING SWALE LOCATED ALONG THE REAR LOT LINE OF LOTS IN LANCER ESTATES UNIT ONE. FOR THE 10 & 100-yr STORMS.

AREA #3 WILL FEED A 6' SWALE ALONG THE EAST PROPERTY LINE AND ADD ITSELF TO THE FLOWS CREATED IN AREA #2. COMBINED FLOWS OF 12.14 cfs AND 25.61 cfs FOR THE 10 & 100-YR STORMS, RESPECTIVELY.

F I L E D
Electronically
2015-02-13 04:49:23 PM
Jacqueline Bryant
Clerk of the Court
Transaction # 4818450 : melwood

Exhibit 13

Exhibit 13

LANCER ESTATES UNIT 10
HYDROLOGY REPORT

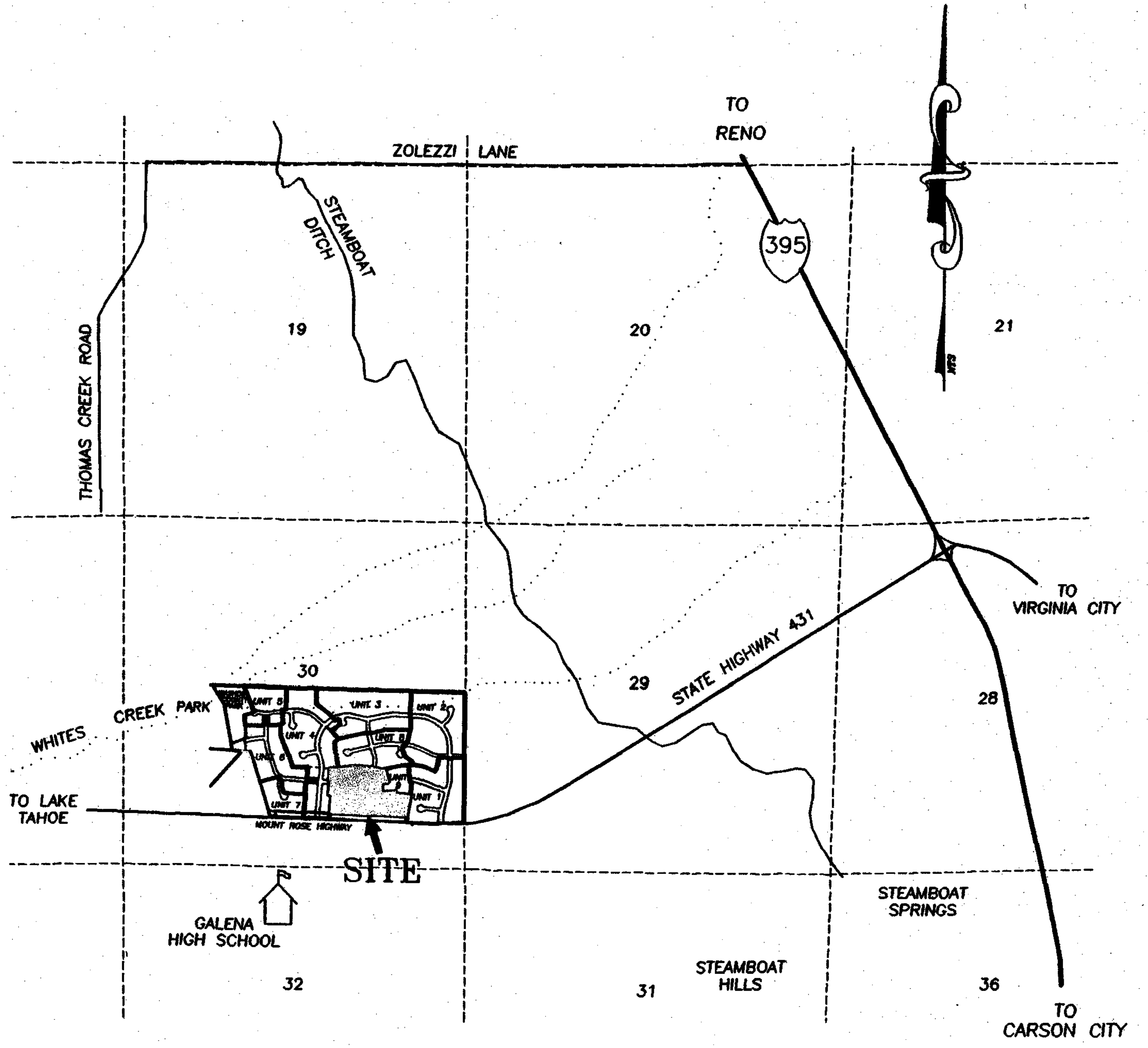
INTRODUCTION

Lancer Estates Unit 10 is a subdivision project consisting of 34 lots with a minimum size of 15,000 square feet. The project is a continuation of the on-going Lancer Estates Project which is located in the South One-Half (S ½) of Section Thirty (30), Township Eighteen (18) North, Range Twenty (20) East, Mount Diablo Meridian. The existing units border unit 10: Unit 4 on the west, Unit 8 on the north, and Unit 1 on the east. The site is bordered on the south by Mount Rose Highway (reference Figure 1). Approximately 200 lots have been developed in the project, which is approved for an ultimate buildout of approximately 309 units. The total drainage area for unit 10 is 23.27 acres.

The purpose of this study is to inventory and analyze the existing storm drain facilities and flow patterns with respect to the proposed storm drain improvements in Unit 10.

EXISTING (ON SITE) STORM DRAIN SYSTEM

The Lancer Estates site generally slopes from the southwest to the northeast with an average gradient of about six percent. The Mount Rose Highway borders the site on its south side, and the north one-half of the roadway adjacent to the project drains onto Lancer Estates. Whites Creek crosses the northwest corner of the Lancer Estates site. Near the intersection of Whites Creek and the north boundary of Lancer Estates, Whites Creek splits into four channels. The southern most of these, which has been designated channel No. 4 in the Basin Management Study conducted by Cella Barr Associates, intercepts most of the drainage from Lancer Estates. Channel No. 4 is designated as zone A Flood Hazard Area by FEMA (Flood Community Panel No. 320019-1501B). The Cella Barr Study further delineates the area of Lancer Estates, which are outside of Channel No. 4, and east of Lancer Hill, as area subject to minimal flooding from over the 100 year storm runoff in Whites Creek. Their study defines minimal flooding as less than 0.5' deep.



VICINITY MAP

FIGURE 1

Unit 10 like the entire project slopes from the southwest to the northeast with an average gradient of about 6.6 percent. About 1/3 of the existing drainage area currently drains northeast into the existing storm drain system of Unit 1. Due to drainage swales, most of the remaining area drains into the storm drain system of Unit 8. A berm between Mount Rose Highway and Lancer Estates captures the remaining runoff from the Unit 10 drainage area as well as runoff from the highway. This berm forces the runoff in an easterly direction along the south portion of the site. The runoff continues into Unit 1, but does not enter the storm drain system of Lancer Estates. It enters a culvert, which transfers the flow to the south side of Mt. Rose Highway. The drainage area between the highway and berm originates at Unit 7 (the southwest corner of the entire site) extends through Unit 4, and then goes into Unit 10 via a culvert under Telluride Drive.

At the northwest corner of Unit 10, an existing 18" storm drain releases flows into a temporary drainage swale approximately where proposed Solitude Drive will match existing Solitude Drive. These flows are collected from Unit 4 and Unit 6 by three catch basins and are 8.77 cfs. and 14.85 cfs. for the 10 year and 100 year storms respectively. The temporary drainage swale transports the flows through Unit 10 to an 18" storm drain inlet that was stubbed with the construction of Unit 8. The storm drain inlet is located on the north border of Unit 10 where existing Mt. Snow Drive (Unit 8) will match proposed Mt. Snow Drive (Unit 10).

The existing hydrology and constructed storm drain information was obtained from Storm Drain Analysis for Lancer Estates Unit 8 and 9 prepared by Odyssey Engineering Incorporated dated January 1996. Refer to Figure 2 for a map and details of the existing storm drain and flows pertaining to Unit 10.

PROPOSED (ON-SITE) STORM DRAIN SYSTEM

The proposed storm system throughout the Unit 10 site is designed to perpetuate flows through the project. All on-site storm water flows were calculated using the Rational Method in conjunction with Manning's Formula. Underground storm facilities will carry the 10 year flows and the streets will carry the 100 year storm flows.

All 10 and 100 year storm flows listed in Table 3, represent future developed condition flows and were calculated using a 1' = 40" scale grading plan prepared by Odyssey Engineering Incorporated dated February, 1999.

Catch basin locations, areas contributing flows to each catch basin, and the pipes transporting the flows are shown in Figure 3. Summaries for the catch basins are provided in Table 2. The 10 year flows west of Mt. Snow Drive which includes all of the flows to the 8 catch basins, flows from Sub-Area L, and the existing flows from Unit 4 and 6 will enter into the existing storm drain system of Unit 8. This will account for 16.13 acres of the total drainage area (23.27 acres). The remainder of the flows will be released into future Lancer Unit 9 or the existing Unit 1.

STORM DRAINAGE CALCULATION METHODOLOGY

As mentioned previously, the Rational Method was used in conjunction with Manning's Formula for all flow calculations. The parameters are as follows:
Rational Method:

Design Flow = Q = CIA Where:

Q = Runoff (cubic feet per second)
C = Runoff Coefficient
I = Rainfall Intensity (inches per hour)
A = Watershed Area (acres)

The land use for the proposed Lancer Estates Unit 10 site will be residential and the existing area is undeveloped. The C values obtained bellow were taken from the Washoe County Design Manual for the 10 year and 100 year storms (Ref. Table 701). Values for the 10 year and 100 year storms are as follows:

	10 year C_{10}	100 year C_{100}
<u>Residential:</u>		
(Average Lot Size)		
1/3 Acre	.50	.60
<u>Streets/Roads:</u>		
(Paved)	.90	.93
<u>Undeveloped Areas:</u>		
Range	.25	.50
Forest	.10	.30

Manning's Formula was used to calculate the capacity of each catch basin. The parameters are as follows:

$$\text{Design Capacity} = Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \text{ Where:}$$

A = Area of Gutter (square feet)
R = Hydraulic Radius (feet), R = A/WP
WP = Wetted Perimeter
S = Longitudinal Slope (feet/feet)
n = Manning's Coefficient (for concrete 0.014)

Per the Washoe County Hydrologic Criteria and Drainage Design Manual, rainfall intensity curves were used to determine the average intensity (Ref. Figure 605). The time of concentration values with a minimum buildup time of ten minutes for existing and proposed areas were calculated from the data on Tables 4 and 5 in the appendix and are expressed as follows:

Tc = Time of Concentration at Calculation Point (minutes)
L = Length of Watershed (feet)
V = Flow Velocity (feet per second)

CATCH BASIN ANALYSIS

Utilizing the above calculation method, flows were calculated at each basin for the entire project. Calculated flows are listed in Table 2 and descriptions for each catch basin are listed below.

Catch basin No. 1 is located on the south side of Solitude Drive at the intersection with Winter Park Court. The catch basin is a Type 4-R and will contain flows from Sub-Area "A". The 10 year and 100 year flows to this catch basin are 1.31 cfs. and 3.31 cfs. respectively. This catch basin has a free flow capacity of 3.28 cfs., therefore containing the 10 year flow.

Catch basin No. 2 is located on the west side of Winter Park Court at the intersection with Solitude Drive. The catch basin is a Type 4-R and will receive flows from Sub-Area "B". The 10 year and 100 year flows to this catch basin are 2.61 cfs. and 6.58 cfs. respectively. This catch basin has a sump capacity of 5.60 cfs., therefore containing the 10 year flow.

Catch basin No. 3 is at the end of the drainage swale between Winter Park Court and Mt. Snow Court and is adjacent to Solitude Drive. The catch basin is a Type 1-A storm drain inlet with a 36" diameter opening and a 36" barrel. It will contain flows from Sub-Area "C". The 10 year and 100 year flows to this catch basin are 1.19 cfs. and 3.00 cfs. respectively. This catch basin has a free flow capacity of 6 cfs., therefore containing the 10 year flow.

Catch basin No. 4 is located on the south side of Solitude Drive at the intersection with Mt. Snow Court. The catch basin is a Type 4-R and will contain flows from Sub-Area "D". The 10 year and 100 year flows to this catch basin are 1.15 cfs. and 2.91 cfs. respectively. This catch basin has a free flow capacity of 2.93 cfs., therefore containing the 10 year flow.

Catch basin No. 5 is located on the west side of Mt. Snow Court at the intersection with Solitude Drive. The catch basin is a Type 4-R and will receive flows from Sub-Area "E". The 10 year and 100 year flows to this catch basin are

2.19 cfs. and 5.52 cfs. respectively. This catch basin has a sump capacity of 5.60 cfs., therefore containing the 10 year flow.

Catch basin No. 6 is located on the south side of Solitude Drive at the intersection with Mt. Snow Court. The catch basin is a Type 4-R and will contain flows from Sub-Area "F". The 10 year and 100 year flows to this catch basin are 0.69 cfs. and 1.74 cfs. respectively. This catch basin has a free flow capacity of 2.88 cfs, therefore containing the 10 year flow.

Catch basin No. 7 is located on the north side of Solitude Drive at the intersection with Mt. Snow Drive. The catch basin is a Type 4-R and will contain flows from Sub-Area "G". The 10 year and 100 year flows to this catch basin are 0.66 cfs. and 1.66 cfs. respectively. This catch basin has a free flow capacity of 2.68 cfs., therefore containing the 10 year flow.

Catch basin No. 8 is located south of Mt. Snow Court in the small detention pond. The catch basin is a Type 1-A storm drain inlet with a 36" diameter opening and a 36" barrel. It will contain flows from Sub-Area "H". The 10 year and 100 year flows to this catch basin are 2.38 cfs. and 6.29 cfs. respectively. This catch basin has a free flow capacity of 6 cfs., therefore containing the 10 year flow.

DRAINAGE SUB-AREA DESIGNATION

Drainage Sub-Area "A" is 1.296 ac. and drains to Catch Basin No. 1.

Drainage Sub-Area "B" is 2.580 ac. and drains to Catch Basin No. 2.

Drainage Sub-Area "C" is 1.177 ac. and drains to Catch Basin No. 3.

Drainage Sub-Area "D" is 1.141 ac. and drains to Catch Basin No. 4.

Drainage Sub-Area "E" is 2.164 ac. and drains to Catch Basin No. 5.

Drainage Sub-Area "F" is 0.683 ac. and drains to Catch Basin No. 6.

Drainage Sub-Area "G" is 0.651 ac. and drains to Catch Basin No. 7.

Drainage Sub-Area "H" is 3.665 ac. and drains to Catch Basin No. 8 and to the culvert in Unit 1.

Drainage Sub-Area "I" is 0.988 ac. and drains to the culvert in Unit 1.

Drainage Sub-Area "J" is 1.741 ac. and drains to Lancer Estates Unit 9.

Drainage Sub-Area "K" is 3.512 ac. and drains to Lancer Estates Unit 9.

Drainage Sub-Area "L" is 2.769 ac. and drains to the storm drain of Unit 8.

Drainage Sub-Area "M" is 0.903 ac. and drains to Lancer Estates Unit 9.

Refer to Figure 3 for a further description of drainage Sub- Areas.

CONCLUSION

Construction of the Lancer Estates Unit 10 project can be accomplished with the construction of a new storm drain branch that is to be connected to the existing storm drain system of Lancer Estates. The 10 year flows along with the majority of the 100 year flows west of Mt. Snow Drive will be contained by the constructed storm drain system of Unit 10. The capacity of the existing system of Unit 8 is able to accommodate these flows. It was possible for us to pick up 6 cfs. from the MT. Rose Highway drainage area (existing Sub-Area E) and bring it through Unit 10.

In 1993 it was decided between NDOT and Washoe County that all flows south of the existing berm between Telluride Dr. and Sundance Dr. exceeding 10 cfs. would be conveyed northerly through the Lancer Estates property (Ref. NDOT letter in the appendix). Proposed Areas "H" and "I" are south of the existing berm and have 100 year flows of 6.29 cfs. and 2.10 cfs. respectively. Catch Basin 8 located in Area "H", diverts 6 cfs. of the 6.29 cfs. (100 year flow) underground and into the storm drain system of Lancer Estates Unit 10. The remaining 0.29 cfs. from Area "H" along with the 2.10 cfs. from Area "I" are released to the Mt. Rose Highway drainage system. This is a combined 100 year flow of 2.39 cfs., which is less than NDOT's maximum of 10 cfs.

Some flows from Unit 10 will be released as street flow to future Unit 9 and existing Unit 1. It is well known by the County and Odyssey Engineering that the streets and storm drain system of Unit 1 was not built to accommodate the flows of future Lancer Estates projects. The system of Unit 10 is designed to alleviate as much flow as possible that will be released to Unit 1 by putting flows underground and into Unit 8.

TABLE 1
EXISTING CONDITION FLOWS
LANCER UNIT 10

Sub-Area	Area (acres)	Tc (min.)	C_{10}[a]	I_{10}[b] (in/hr)	Q_{10} (cfs)	C_{100}[a]	I_{100}[b] (in/hr)	Q_{100} (cfs)
A	1.935	15.24	0.35	1.62	1.097	0.40	3.41	2.639
B	1.184	10.73	0.40	1.98	0.938	0.50	4.07	2.409
Flow released in sub-area B by existing 18" storm dra					8.77			14.85
				Total in B=	9.708		Total in B=	17.259
C	7.290	19.43	0.35	1.45	3.700	0.40	3.05	8.894
D	8.208	16.09	0.35	1.60	4.596	0.40	3.38	11.097
E	4.653	20.12	0.40	1.42	2.643	0.50	3.00	6.980
TOTAL	23.27				21.744			46.869

FOOTNOTES

- a Runoff Coefficients were obtained from table 701
- b Intensities were obtained from figure 605

TABLE 2
CATCH BASIN CAPACITIES
LANCER UNIT 10

Catch Basin No.	Catch Basin Type	Q_{10} (cfs)	Q_{cap} (cfs)
1	4-R	1.31	3.28
2	4-R	2.61	5.60
3	1-A	1.19	6
4	4-R	1.15	2.93
5	4-R	2.19	5.60
6	4-R	0.69	2.88
7	4-R	0.66	2.68
8	1-A	2.37	6

TABLE 3
PROPOPOSED FLOWS
LANCER UNIT 10

Sub-Area	Area (acres)	Tc (min.)	C_{10}[a]	I_{10}[b] (in/hr)	Q_{10} (cfs)	C_{100}[a]	I_{100}[b] (in/hr)	Q_{100} (cfs)
A	1.296	10.00	0.50	2.02	1.309	0.60	4.25	3.305
B	2.580	10.00	0.50	2.02	2.606	0.60	4.25	6.579
C	1.177	10.00	0.50	2.02	1.189	0.60	4.25	3.001
D	1.141	11.05	0.50	2.02	1.152	0.60	4.25	2.910
E	2.164	10.00	0.50	2.02	2.186	0.60	4.25	5.518
F	0.683	10.00	0.50	2.02	0.690	0.60	4.25	1.742
G	0.651	10.00	0.50	2.02	0.658	0.60	4.25	1.660
H	3.665	15.35	0.40	1.62	2.375	0.50	3.43	6.285
I	0.988	10.00	0.40	2.02	0.798	0.50	4.25	2.100
J	1.741	10.00	0.50	2.02	1.758	0.60	4.25	4.440
K	3.512	10.00	0.50	2.02	3.547	0.60	4.25	8.956
L	2.769	10.00	0.50	2.02	2.797	0.60	4.25	7.061
M	0.903	10.00	0.50	2.02	0.912	0.60	4.25	2.303
TOTAL	23.27				21.976			55.858

FOOTNOTES

[a] Runoff Coefficients were obtained from table 701

[b] Intensities were obtained from figure 605

REFERENCES

1. Odyssey Engineering, Hydrology Report for Lancer Estates Unit No.'s 8 and 9, January 1996.
2. Washoe County, Washoe County Hydrologic Criteria and Drainage Design Manual, December 1996.

APPENDIX

TABLE 4
TIME OF CONCENTRATION FOR EXISTING SUB-BASINS
LANCER UNIT 10

Sub-	Area	INITIAL TIME (t_i)[a]				TRAVEL TIME (t_t)				Tc[d]	Tc[e]
Area		R (=C)[b]	L_o	SLOPE	t_i	L	SLOPE	VEL.[c]	t_t		(minimum)
	(acres)		(ft)	%	(min)	(ft)	%	(fps)	(min)	(min)	(min)
A	1.935	0.35	300	6.6	12.47	300	7	1.8	2.78	15.24	**15.24**
B	1.184	0.90	400	4.0	4.54	650	6.6	1.75	6.19	10.73	**10.73**
C	7.290	0.35	500	6.6	16.09	350	6.6	1.75	3.33	19.43	**19.43**
D	8.208	0.35	500	6.6	16.09	--	--	--	0	16.09	**16.09**
E	4.653	0.90	50	2.0	2.02	1900	6.5	1.75	18.10	20.12	**20.12**
TOTAL	23.27										

FOOTNOTES

- a $ti=1.8(1.1-R)L^{1/2}/S^{1/3}$
- b For Rational Method $R=C_{10}$
- c Obtained from figure 701 (short grass pasture)
- d $Tc=t_i+t_t$
- e The minimum Tc for non-urban and urban watersheds is 10 minutes

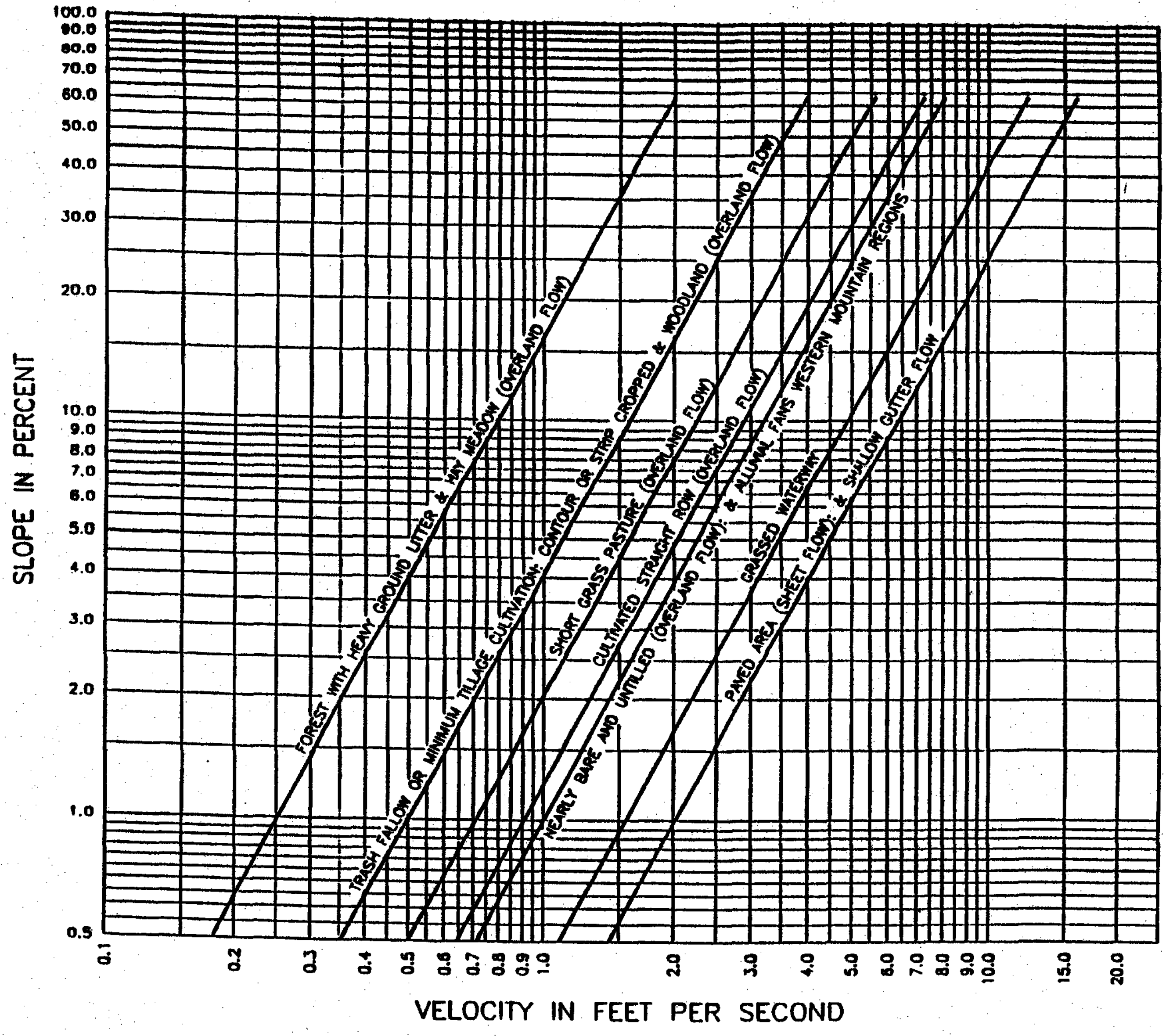
TABLE 5
TIME OF CONCENTRATION FOR PROPOSED SUB-BASINS
LANCER UNIT 10

Sub-	Area	INITIAL TIME (t_i)[a]				TRAVEL TIME (t_t)				Tc^d	Tc^e
Area		R (=C)[b]	L_o	SLOPE	t_i	L	SLOPE	VEL.[c]	t_t		(minimum)
	(acres)		(ft)	%	(min)	(ft)	%	(fps)	(min)	(min)	(min)
A	1.296	0.50	140	5	7.47	300	5.1	4.5	1.11	8.58	**10.00**
B	2.580	0.50	140	5	7.47	400	2	2.8	2.38	9.85	**10.00**
C	1.177	0.50	80	9	4.64	400	2.13	2.9	2.30	6.94	**10.00**
D	1.141	0.50	100	2	8.57	580	3.8	3.9	2.48	11.05	**11.05**
E	2.164	0.50	160	8	6.83	280	2.25	3.0	1.56	8.39	**10.00**
F	0.683	0.50	50	2	6.06	300	2.25	3.0	1.67	7.73	**10.00**
G	0.651	0.50	40	4	4.30	480	6.18	5.0	1.60	5.90	**10.00**
H	3.665	0.90	50	2.0	2.02	1400	6.6	1.75	13.33	15.35	**15.35**
I	0.988	0.90	50	2.0	2.02	200	6.6	1.75	1.90	3.93	**10.00**
J	1.741	0.50	140	8	6.39	290	3	3.5	1.38	7.77	**10.00**
K	3.512	0.50	170	7.5	7.19	160	4	4.0	0.67	7.86	**10.00**
L	2.769	0.50	150	5	7.74	450	7	5.3	1.42	9.15	**10.00**
M	0.903	0.50	180	6	7.97	--	--	--	0	7.97	**10.00**
TOTAL	23.27										

FOOTNOTES

- a $ti=1.8(1.1-R)L^{1/2}/S^{1/3}$
- b For Rational Method $R=C_{10}$
- c Obtained from figure 701 (paved area) or (short grass pasture)
- d $Tc=t_i+t_t$
- e The minimum Tc for non-urban and urban watersheds is 10 minutes

TRAVEL TIME VELOCITY



VERSION: 12-02-1996

WRC ENGINEERING, INC.

REFERENCE:

Soil Conservation Service, 1985 (Modified)

FIGURE

701

1076, 1076-701.DWG 12-02-96 1:11AS WRC.PCP CAN

RATIONAL FORMULA METHOD RUNOFF COEFFICIENTS

Land Use or Surface Characteristics	Aver. % Impervious Area	Runoff Coefficients	
		10-year (C_{10})	100-year (C_{100})
<u>Business/Commercial</u>:			
Downtown Areas	85	.88	.89
Neighborhood Areas	70	.70	.80
<u>Residential</u>:			
(Average Lot Size)			
⅛ Acre or Less (Multi-Unit)	65	.68	.78
¼ Acre	38	.55	.65
⅓ Acre	30	.50	.60
½ Acre	25	.45	.55
1 Acre	20	.40	.50
<u>Industrial</u>:	72	.72	.82
<u>Open Space</u>:			
Lawns, Parks, Golf Courses)	5	.10	.30
<u>Undeveloped Areas</u>:			
Range	0	.25	.50
Forest	0	.10	.30
<u>Streets/Roads</u>:			
Paved	100	.90	.93
Gravel	20	.40	.50
<u>Drives/Walks</u>:	95	.88	.89
<u>Roofs</u>:	90	.85	.87

Notes:

1. Composite runoff coefficients shown for Residential, Industrial, and Business/Commercial Areas assume irrigated grass landscaping for all previous areas. For development with landscaping other than irrigated grass, the designer must develop project specific composite runoff coefficients from the surface characteristics presented in this table.

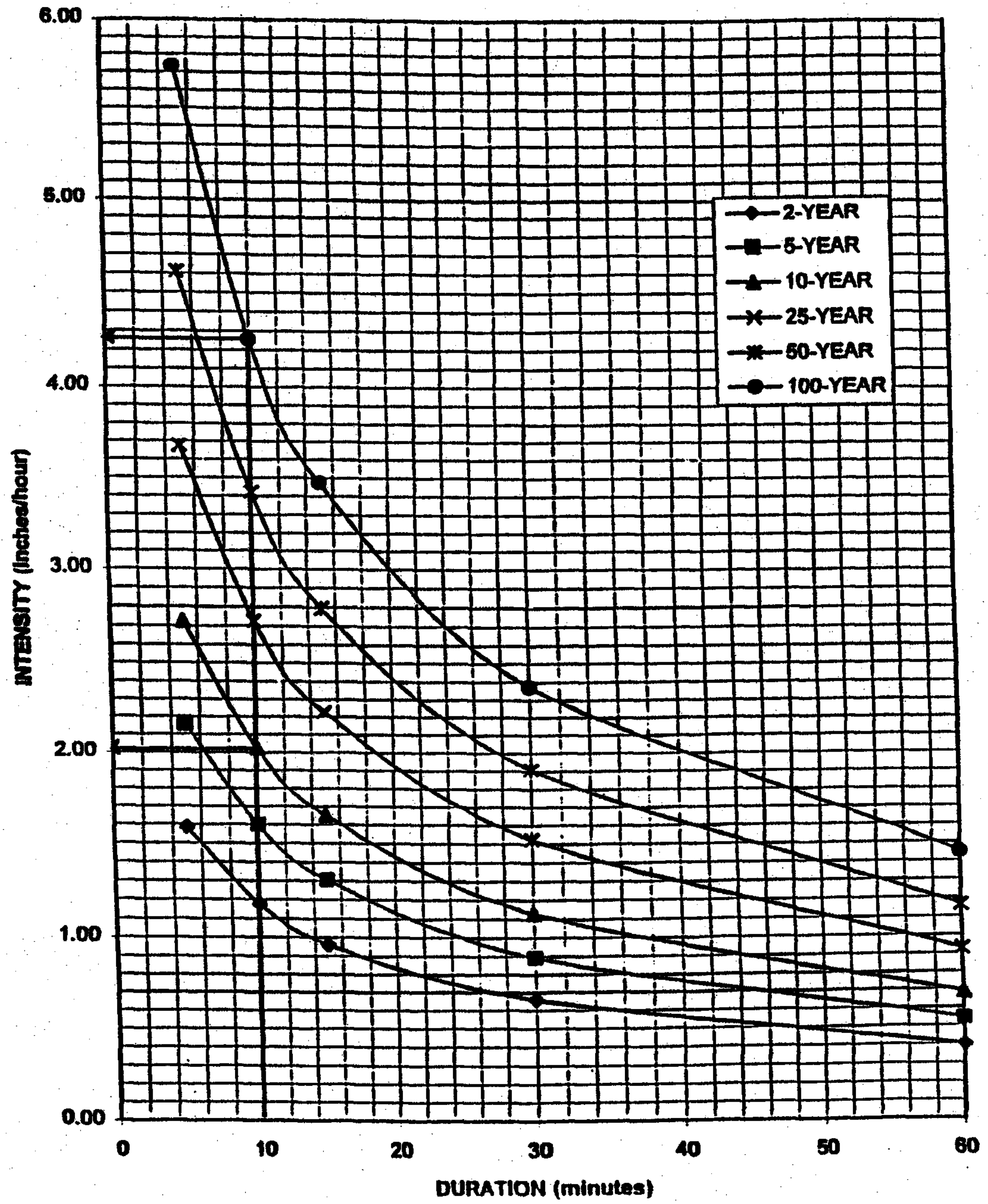
VERSION: December 2, 1996

WRC ENGINEERING, INC.

REFERENCE:
USDCM, DRCOG, 1969
(with modifications)

TABLE
701

ZONE I TIME-INTENSITY-FREQUENCY CURVES



VERSION: 00-00-0000

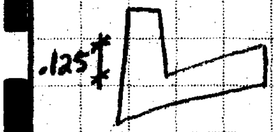
WRC ENGINEERING INC

REFERENCE:

FIGURE
605

dyssey ENGINEERING INCORPORATED
2105 Capurro Way Suite F
Sparks, NV 89431
(775) 359-3303 [redacted]ax (775) 359-3329

JOB [illegible]
SHEET NO. 1 OF 3
CALCULATED BY D.[redacted] DATE 3/1/99
CHECKED BY DATE
SCALE



Catch Basin Capacities

Lancer Unit 10

CB # 1 Type 4-R

$Q_{10} = 2.212$ cfs

Gutter Flow $= 1.48(S)^{1/2} = 1.48(0.0618)^{1/2} = 0.368$ cfs (s = street slope)

Adj. Flow $= 2.212 - 0.368 = 1.844$ cfs

Depth From Neenah $= 0.13$ ft

Adj. Depth $(D) = 0.125 + 0.13 = 0.255$ ft ✓

$K = 32.2$

$Q_{cap} = K(D)^{5/3} = 32.2(.255)^{5/3} = \underline{3.30 \text{ cfs}}$

CB # 2 ~~Type 4-R~~ Now a sump condition head = 0.41 ft capacity = 5.60 cfs

~~$Q_{10} = 4.070$ cfs~~

~~Gutter Flow $= 1.48(s)^{1/2} = 1.48(0.0156)^{1/2} = 0.185$ cfs~~

~~Adj Flow $= 4.070 - 0.185 = 3.885$ cfs~~

~~Depth From Neenah $= 0.222$ ft~~

~~Adj Depth $(D) = 0.125 + 0.222 = 0.347$ ft~~ ✓

~~$K = 20$~~

~~$Q_{cap} = K(D)^{5/3} = 20(0.347)^{5/3} = \underline{3.43 \text{ cfs}}$~~

CB # 3 (36" dia)

$Q_{10} = 2.12$ cfs

slope = 2.13%

Estimate

$Q_{cap} = \underline{6 \text{ cfs}}$

JOB Lancer Unit 10
SHEET NO. 2 OF 3
CALCULATED BY D. DATE 3/1/99
CHECKED BY DATE
SCALE

CB # 4 Type 4-R

Q_{10} = 1.707 cfs

street slope = 0.0618

Gutter flow = $1.48(s)^{1/2} = 1.48(0.0618)^{1/2}$ = 0.368 cfs

Adj flow = 1.707 − 0.368 = 1.339 cfs

Depth from Neenah = 0.1125 ft

Adj depth (D) = 0.125 + 0.1125 = 0.2375 ft ✓

K = 32.2

Qcap = $K(D)^{5/3} = 32.2(0.2375)^{5/3}$ = 2.93 cfs

~~**CB # 5** Type 4-R~~

~~Q_{10} = 3.664~~

~~Street Slope = 0.0225~~

~~Gutter flow = $1.48(s)^{1/2} = 1.48(0.0225)^{1/2}$ = 0.222 cfs~~

~~Adj flow = 3.664 − 0.222 = 3.442 cfs~~

~~Depth from Neenah = 0.188 ft~~

~~Adj depth (D) = 0.125 + 0.188 = 0.313 ft~~ ✓

~~K = 24~~

~~Qcap = $K(D)^{5/3} = 24(0.313)^{5/3}$ = 3.46 cfs~~

Now a sump condition
Head = 0.41 ft
capacity = 5.60 cfs

CB # 6 Type 4-R

Q_{10} = 1.203

Street Slope = 0.0705

Gutter flow = $1.48(s)^{1/2} = 1.48(0.0705)^{1/2}$ = 0.393 cfs

Adj flow = 1.203 − 0.393 = 0.81 cfs

Depth from Neenah = 0.095 ft

Adj Depth (D) = 0.125 + 0.095 = 0.220 cfs ✓

K = 36

Qcap = $K(D)^{5/3} = 36(0.22)^{5/3}$ = 2.88 cfs

2105 Capurro Way Suite F
Sparks, NV 89431
(775) 359-3303 [redacted]ax (775) 359-3329
Odyssey ENGINEERING INCORPORATED

JOB Lancer Unit 10
SHEET NO. 3 OF 3
CALCULATED BY D. [redacted] DATE 3/1/99
CHECKED BY DATE
SCALE

CB # 7 Type 4-R
$Q_{10} = 1.204$
Street Slope = 0.0618
Gutter flow = $1.48(S)^{1/2} = 1.48(0.0618)^{1/2} = 0.368$ cfs
Adj Flow = 1.204 - 0.368 = 0.836 cfs
Depth from Neenah = 0.0998 ft
Adj Depth (D) = 0.125 + 0.0998 = 0.2248 ft
K = 32.2
Qcap = $K(D)^{5/3} = 32.2(0.2248)^{5/3}$ = 2.68 cfs

CB # 8 Type 3 Beehive grate
$Q_{10} = 4.76$ cfs
Estimate
Qcap = 6 cfs

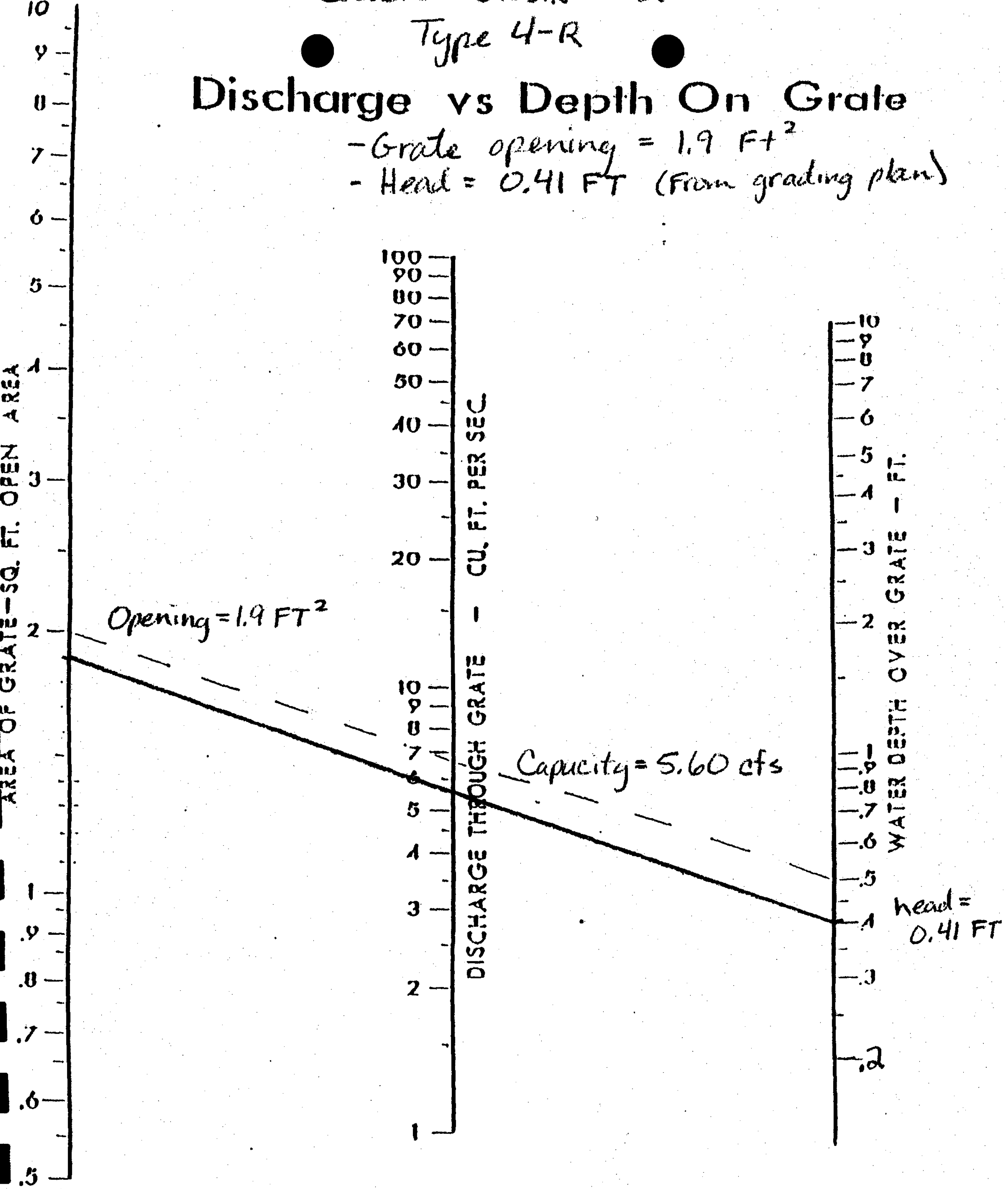
Catch Basin #2

Type 4-R

Discharge vs Depth On Grate

-Grate opening = 1.9 Ft^2

- Head = 0.41 FT (From grading plan)



Opening = 1.9 FT^2

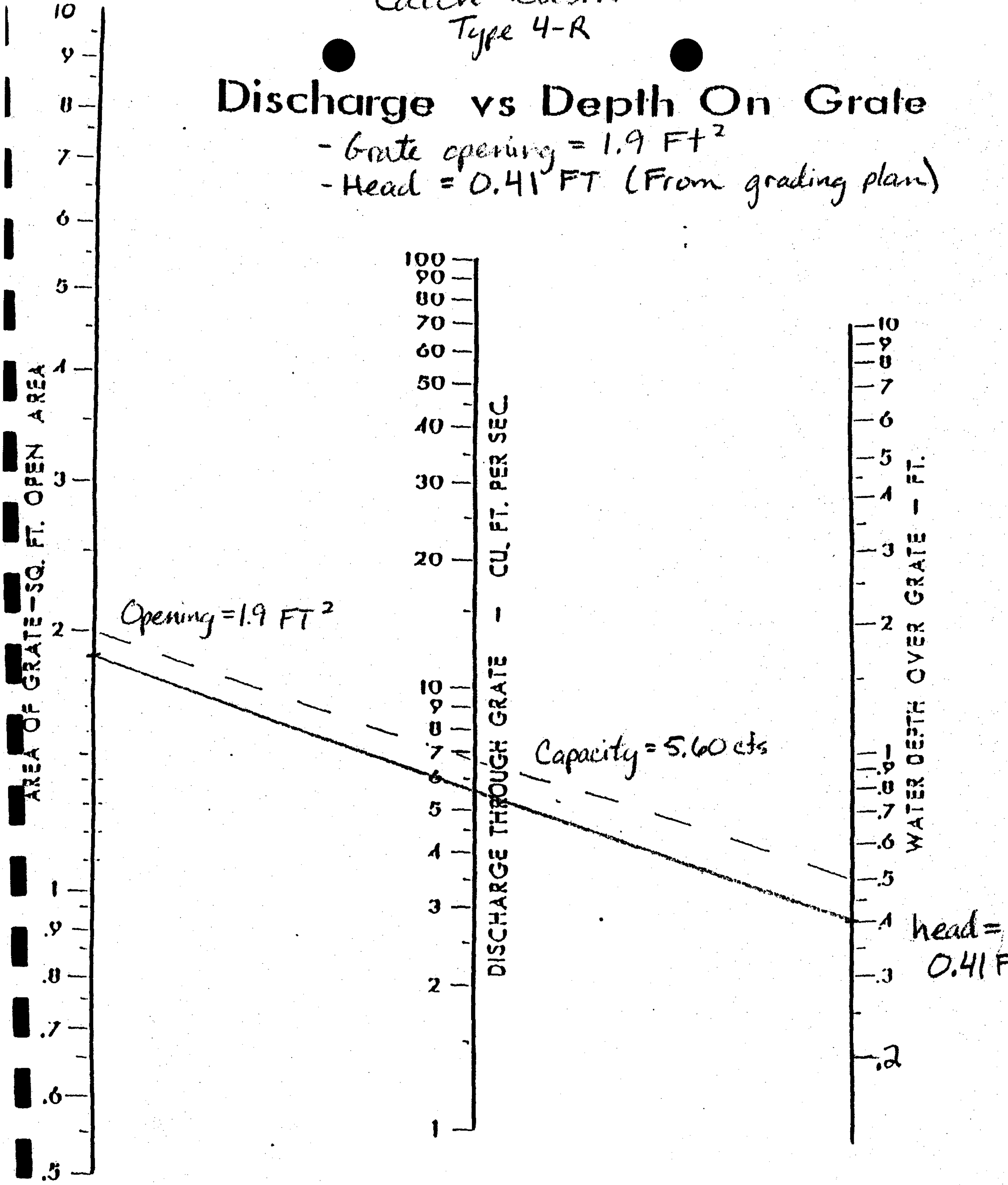
Capacity = 5.60 cfs

head = 0.41 FT

Catch Basin #5
Type 4-R

Discharge vs Depth On Grate

- Grate opening = 1.9 Ft^2
- Head = 0.41 FT (From grading plan)



STATE OF NEVADA
D[illegible]ARTMENT OF TRANSPORTATI[illegible]
District II
310 Galletti Way
Sparks, NV 89431
(702) 688-1250 FAX (702) 688-1189

BOB MILLER, *Governor*
TOM STEPHENS, P.E., *Director*

June 13, 1996

RECEIVED
JUN 18 1996
OFFICE OF
WASHOE COUNTY ENGINEER

David T. Price
Washoe County Engineer
Department Of Public Works
P.O. Box 11130
Reno, NV 89520

Re: Roadway Surface Drainage on SR-431

Dear Mr. Price:

The department is requesting the assistance of Washoe County in correcting a drainage problem on the north side of SR-431 (Mount Rose Highway) between Telluride Dr. and Sundance Dr..

During discussions in April of 1993 it was decided between the department and Washoe County that all flows between Telluride Dr. and Sundance Dr. exceeding 10 cfs would be conveyed northerly through the Lancer Estates property. Currently, there is a large berm constructed on the Lancer Estates property that prevents all roadway surface drainage from the highway to flow northerly as agreed. Meetings between the department, Washoe County and McMillan Homes has not brought a resolution to this problem. We are asking that Washoe County direct McMillan Homes to construct facilities that will convey all drainage above the 10 cfs across their property per the April 1993 discussions.

Please contact me at 688-1250 to discuss correcting this problem.

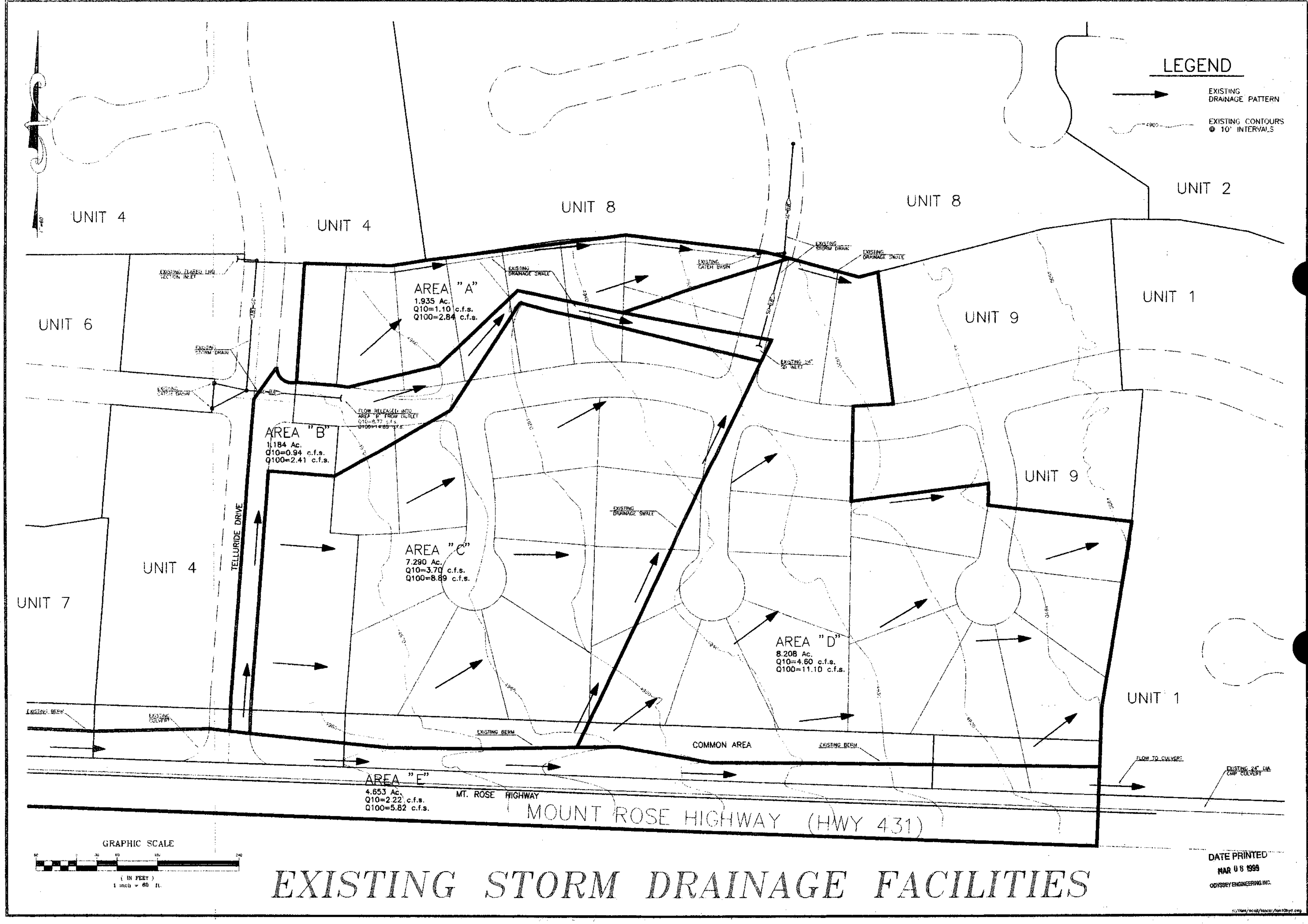
Sincerely,

George E Jordy

George E. Jordy, P.E.
Assistant District Engineer

GEJ:nd
Enclosures

cc: Chris McMillan, McMillan Homes
Norm Lindeman, Washoe County



EXISTING STORM DRAINAGE FACILITIES

REV.	DATE	DESCRIPTION	BY	APP'D

DATE: ...CH. 1999
DRAWN BY: D.P.
DESIGNED BY: D.P.
CHECKED BY:

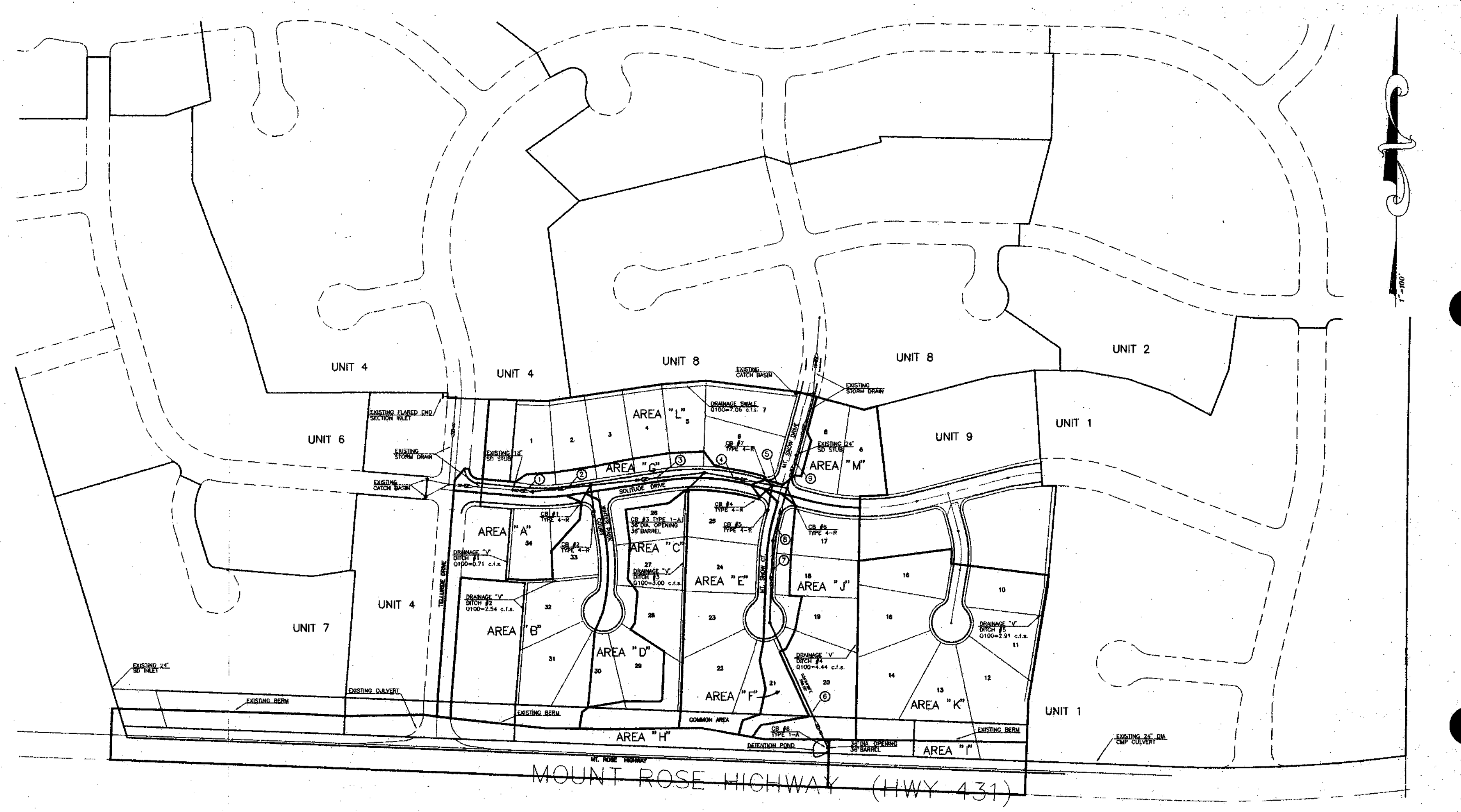
LANCER ESTATES UNIT 10
EXISTING STORM DRAIN

WASHOE COUNTY, NEVADA

odyssey ENGINEERING INCORPORATED
2105 CAPURRO WAY, SUITE ... KS, NV 89431
(702) 359-3303 FAX (702) 359-3329

SCALE
HORIZ. 1"=60'
VERT. NA
JOB NO. 2238

SHEET
1
OF
2

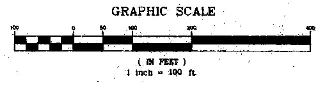


PROPOSED FLOWS

SUB-AREA	AREA Acres	Q10 c.f.s.	Q100 c.f.s.	DESCRIPTION
A	1.298	1.309	3.305	FLOWS TO CB#1 Qcap=3.28 c.f.s.
B	2.580	2.606	6.579	FLOWS TO CB#2 Qcap=5.60 c.f.s.
C	1.177	1.189	3.001	FLOWS TO CB#3 Qcap=6 c.f.s.
D	1.141	1.152	2.910	FLOWS TO CB#4 Qcap=2.93 c.f.s.
E	2.164	2.186	5.518	FLOWS TO CB#5 Qcap=5.60 c.f.s.
F	0.663	0.690	1.742	FLOWS TO CB#6 Qcap=2.88 c.f.s.
G	0.651	0.658	1.660	FLOWS TO CB#7 Qcap=2.68 c.f.s.
H	3.665	2.375	6.285	FLOWS TO CB#8 Qcap=6 c.f.s.
I	0.986	0.798	2.100	FLOWS INTO UNIT 1 AND THEN INTO CULVERT FOR MT. ROSE HIGHWAY
J	1.741	1.758	4.440	FLOWS INTO UNIT 9
K	3.512	3.547	8.956	FLOWS INTO UNIT 9
L	2.769	2.797	7.061	FLOWS EXISTING STORM DRAIN OF UNIT 8
M	0.903	0.912	2.303	FLOWS INTO UNIT 9
TOTAL	23.27	21.479	54.485	

PROPOSED FLOWS IN PIPES

STORM DRAIN	SIZE (dia.)	Q10 c.f.s.	Q100 c.f.s.	Qcap c.f.s.
1	18"	6.77	14.85	7.50
2	18"	8.77	14.85	24.40
3	18"	12.69	24.73	23.97
4	16"	13.86	27.74	23.81
5	18"	18.55	39.57	23.00
6	18"	2.38	6.29	11.67
7	18"	2.38	6.29	12.44
8	18"	2.38	6.29	13.05
9	16"	20.93	45.86	44.56



PROPOSED STORM DRAINAGE FACILITIES

h:/files/acad/lancer/lan10hyd.dwg

REV.	DATE	DESCRIPTION	BY	APP'D

DATE: CH, 1999
DRAWN BY: D.P.
DESIGNED BY: D.P.
CHECKED BY:

LANCER ESTATES UNIT 10
PROPOSED STORM DRAIN
WASHOE COUNTY, NEVADA

2105 CAPURRO WAY, SUITE RKS, NV 89431
(702) 359-3303 FAX (702) 359-3329
odyssey ENGINEERING INCORPORATED

SCALE
HORIZ. 1" =100'
VERT. NA
JOB NO. 2238

SHEET
2
OF
2

FILED
Electronically
2015-02-13 04:49:23 PM
Jacqueline Bryant
Clerk of the Court
Transaction # 4818450 : melwood

Exhibit 14

Exhibit 14

STORM DRAIN ANALYSIS
LANCER ESTATES UNIT NO. 11

INTRODUCTION

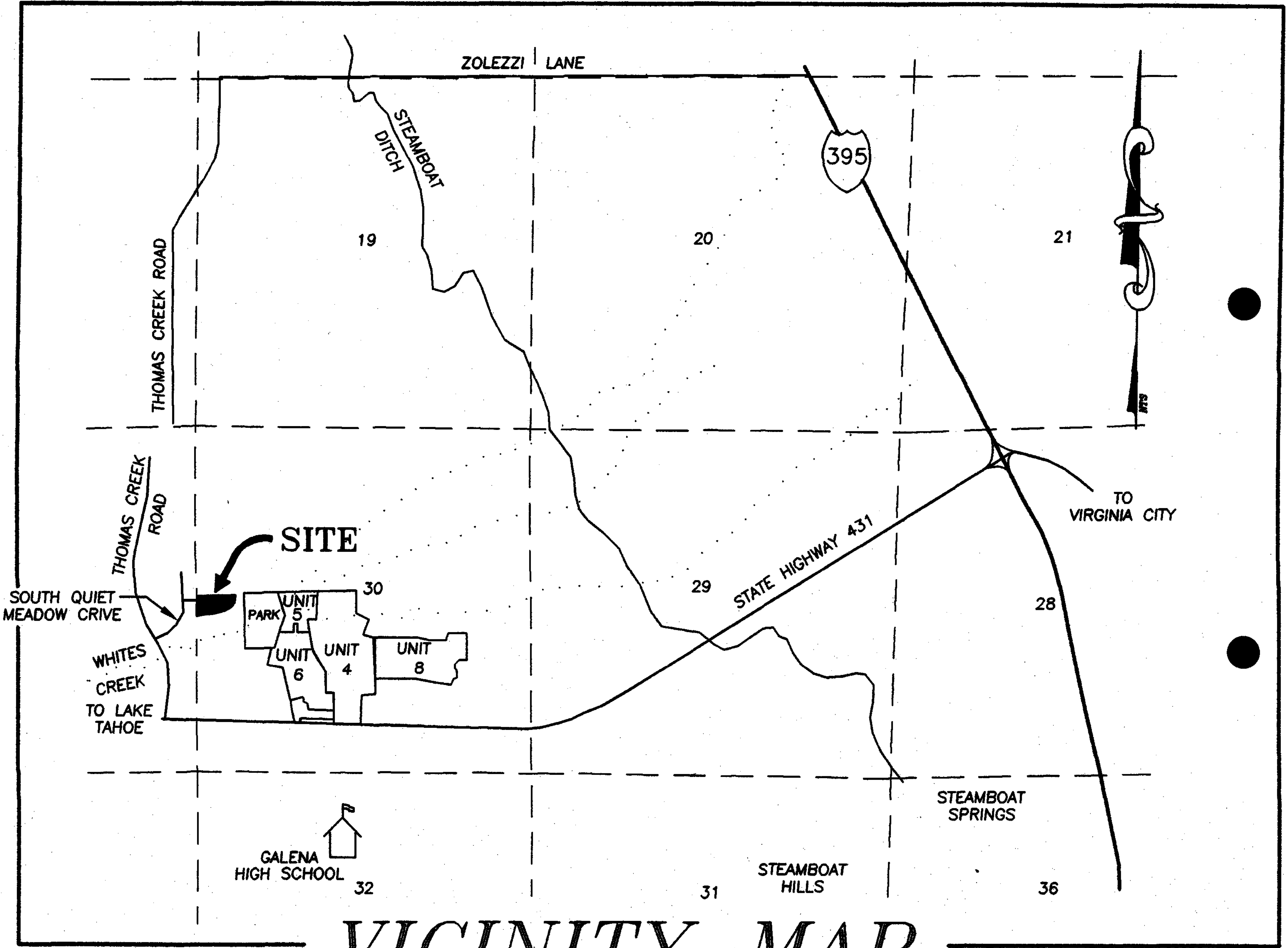
Lancer Estates Unit No. 11 is a single family residential subdivision consisting of 12 lots on 8.123 acres. The minimum lot site is 21,780 square feet (1/2 acre). Lancer Unit 11 is the next phase of the Lancer Estates Subdivision project. The site is located in the Southwest One-Quarter (1/4) of Section Thirty (30), Township Eighteen (18) North, Range Twenty (20) East, Mount Diablo Meridian. The site is bordered on the south by Whites Creek, the north by vacant property and the west by the Saddlehorn Development. Approximately 250 lots have been or are currently being developed within the development, which is approved for a total of 309 units. The proposed building envelopes are located in unshaded Zone X, area of minimal flooding. Whites Creek is located in Zone A flood hazard area by FEMA map no. 32031C370E, effective September 30, 1994.

EXISTING STORM DRAIN SYSTEM

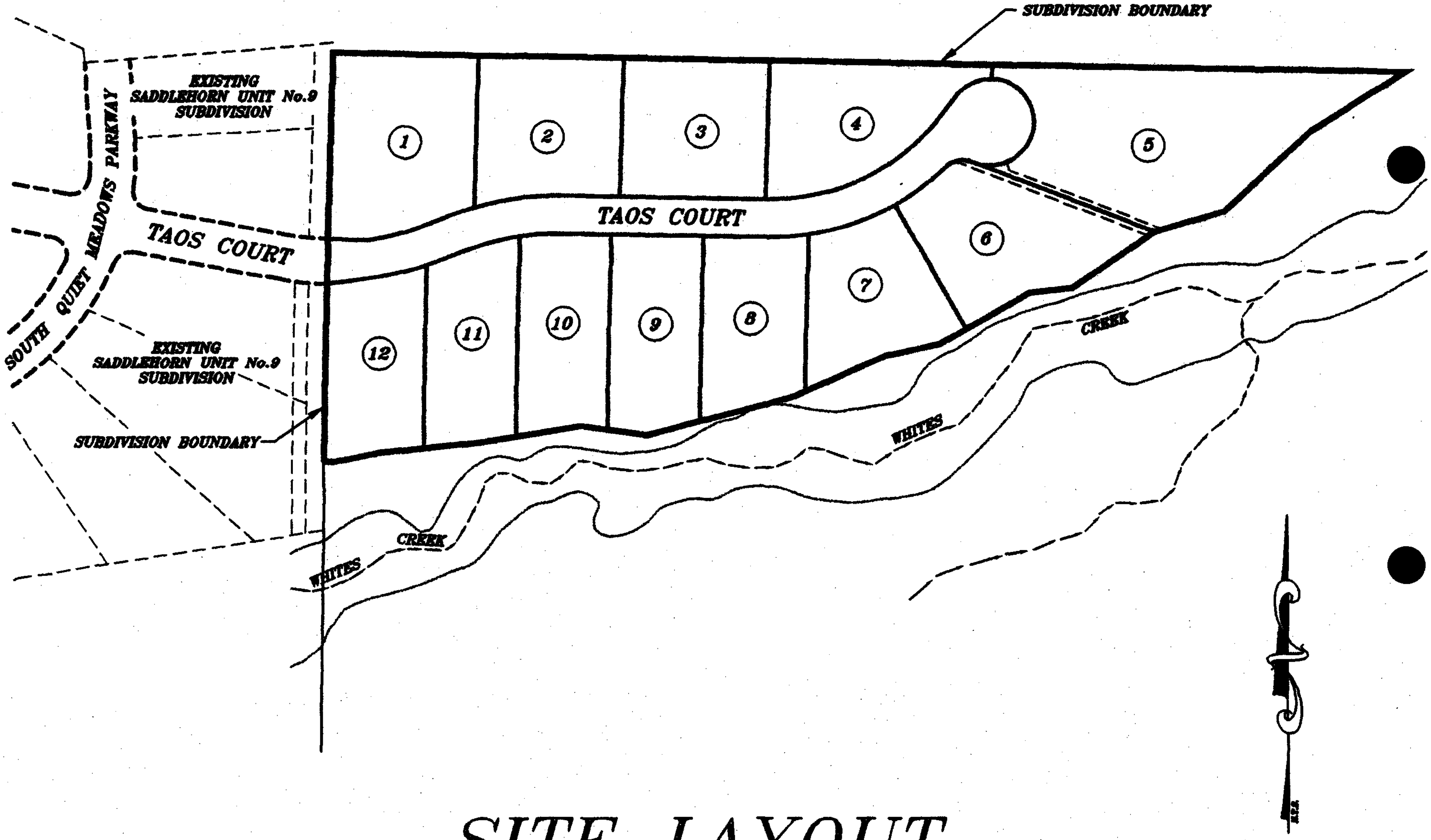
The existing topography of the site traverses down in a west to east direction. The south end of the site drains down in a north to south direction into Whites Creek. The site is covered with native grasses, ground cover and sage brush. The average slope of the site from west to east is approximately 6% and from north to south, the slope is relatively flat, breaking at the south edge to approximately 50% into Whites Creek. Due west, there is an existing storm drain facility which was constructed with Saddlehorn Unit No. 9 which will remain with development of Lancer Estates Unit No. 11. By connecting Taos Court into existing South Quiet Meadows Parkway, portions of the storm water which traveled down South Quiet Meadows Parkway into the existing low point will now be contained within a drop inlet located approximately 200 feet from the proposed intersection, which connects into an existing 18" diameter storm drain main. All other existing storm drain facilities will remain undisturbed.

PROPOSED STORM DRAIN SYSTEM

The proposed storm drain system throughout Lancer Estates Unit No. 11 is designed to perpetuate flows through the project and to maintain existing flow patterns. All storm water flows were calculated using the Rational Method. The flow rates listed as Q10 year and Q100 year as shown on the improvement drawings represent final developed conditions. Catch basin flow rates are for their particular sub-area and storm drain mains represent flows at ultimate buildout (ref. figure no. 2 for sub-area designations and flows).



VICINITY MAP


SUBDIVISION BOUNDARY
EXISTING SADDLEHORN UNIT No.9 SUBDIVISION
SOUTH QUIET MEADOWS PARKWAY
TAOS COURT
TAOS COURT
EXISTING SADDLEHORN UNIT No.9 SUBDIVISION
SUBDIVISION BOUNDARY
1
2
3
4
5
6
7
8
9
10
11
12
CREEK
WHITES
CREEK
WHITES

SITE LAYOUT

The storm drain system was designed to carry all 10 year flows within the pipe systems. All 10 year and 100 year flows and 10 year catch basin capacities are shown on the improvement plans. The 10 year flow from Unit 1 will be contained at a low point in Taos Court and will discharge into Whites Creek via a 15" dia. concrete pipe. A 6' wide x 1' deep earth "V"-ditch will be constructed at the north property line of lot no., 5 to provide for the 100 year overflow. This "V"-ditch will also discharge into Whites Creek. As mentioned previously, a drop inlet will be constructed approximately 200 feet from the intersection of Taos Court with South Quiet Meadows Parkway to contain portions of the existing flows produced by Saddlehorn Unit No. 9. This drop inlet will connect into an existing 48" dia. storm drain manhole in Taos Court, which was constructed with the Saddlehorn Development.

The finish grading of lots 5 through 12 is designed such that the high point is located near the center of the building envelope. This allows the front half of the lot to drain into Taos Court, and the rear half will drain into Whites Creek (sub-areas "B" and "C"). Lots 1 through 4 will drain directly onto Taos Court (sub-area "A").

STORM DRAINAGE CALCULATION METHODOLOGY

As mentioned previously, the Rational Method was used for all flow calculations.

Design Flow = Q = CiA

Where:
Q = Runoff (cubic feet per second)
C = Runoff Coefficient
i = Rainfall intensity (inches per hour)
A = Watershed Area (acres)

Since the site land use will be single family residential averaging 1.48 units per acre, a C value of 0.45 was used.

Per the Washoe County Development Code, Art.420, Storm Drainage Standards, rainfall intensity curves were used to determine the average intensity. The time of concentration with a minimum buildup time of ten minutes is expressed as follows:

Tc = 10 or L/(VX60) whichever is greater

Where:
Tc = Time of Concentration at calculation point (minutes)
L = Length of Watershed (feet)
V = Flow Velocity (feet per second)

Since the time of concentration values calculated were less than 10 minutes in every case, 10 minutes was used. The 10 year storm rainfall intensity is i10 = 1.85 in/hr, and the 100 year intensity is i100 = 3.8 in/hr.

CATCH BASIN ANALYSIS

Utilizing the above calculation method, flows were calculated at each catch basin. Calculated flows and descriptions for each catch basin are listed below.

Catch basin no. 1 is located approximately 200 feet east of the intersection of Taos Court and South Meadows Parkway on the south side of the street. All flows contributing to this catch basin are from the Saddlehorn Development with exception of a small portion of Taos Court. With reference to the improvement plans prepared for Saddlehorn Unit No. 9, prepared by Jeff Codega Planning Design, Inc., dated August 1994, the 10 year and 100 year flows are 1.50 cfs and 2.40 cfs, respectively. The street slope entering this catch basin is 4.84 percent. The catch basin is a Type 4-R with a capacity of 2.80 cfs, therefore containing all excess runoff. This catch basin will connect into the existing drainage system within Saddlehorn Unit No. 9.

Catch basin no. 2 is located at a low point in Taos Court (north side), adjacent to lot no. 4. The 10 year and 100 year flows are 2.45 cfs and 5.04 cfs respectively. This catch basin is a Type 4-R in a sump condition. Using a headwater depth of 0.5 feet, this catch basin has a capacity of 6.40 cfs, therefore containing all excess runoff.

Catch basin no. 3 is located at a low point in Taos Court (south side), adjacent to lot no. 5. The 10 year and 100 year flows are 1.65 cfs and 3.35 cfs respectively. This catch basin is also a Type 4-R in a sump condition. Using a headwater depth of 0.5 feet, this catch basin has a capacity of 6.40 cfs, therefore containing all excess runoff.

DRAINAGE SUB-AREA DESIGNATION

Drainage sub-area "A" is 2.95 acres and drains to catch basin no. 2.

Drainage sub-area "B" is 1.98 acres and drains to catch basin no. 3.

Drainage sub-area "C" is 3.59 acres and drains into Whites Creek.

Catch basin no. 1 contains existing flows within the Saddlehorn Development.

CONCLUSION

With development of the Lancer Estates Unit No. 11 Subdivision, the proposed storm drainage system is designated to carry all 10 year flows which will be generated by development and will discharge into acceptable drainage ways. The runoff will be increased by approximately 12% or 0.8 cfs (10 year). This increase will have a minimal effect on downstream properties.

The drainage and grading design for this subdivision will provide drainage protection for the homes within the development, and will maintain existing drainage patterns within the watershed area.

TABLE I
EXISTING CONDITIONS

SUB-AREA	AREA	C	Tc (Min.)	i10 (in/hr)	Q10 (cfs)	i100 (in/hr)	Q100 (cfs)
A	2.95	0.40	10	1.85	2.18	3.8	4.48
B	1.98	0.40	10	1.85	1.47	3.8	3.01
C	3.59	0.40	10	1.85	2.66	3.8	5.46
TOTAL	8.52				6.31		12.95

TABLE II
PROPOSED CONDITIONS

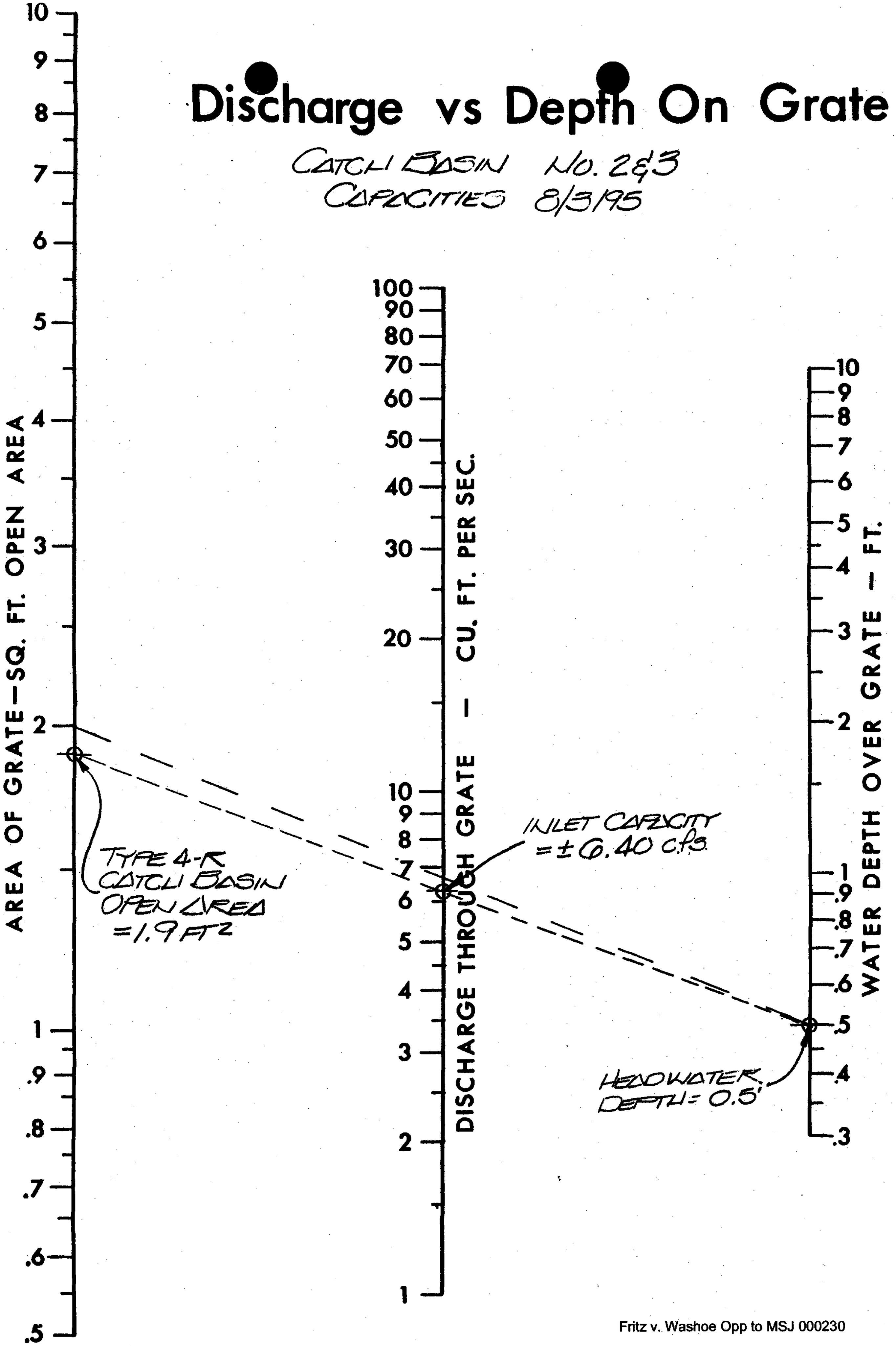
SUB-AREA	AREA	C	Tc (Min.)	i10 (in/hr)	Q10 (cfs)	i100 (in/hr)	Q100 (cfs)
A	2.95	0.45	10	1.85	2.45	3.8	5.04
B	1.98	0.45	10	1.85	1.65	3.8	3.35
C	3.59	0.45	10	1.85	2.99	3.8	6.14
TOTAL	8.52				7.09		14.53

TABLE III
CATCH BASIN CAPACITIES

CATCH BASIN NUMBER	TYPE	Q10 (CFS)	Q CAP (CFS)
1	4-R	1.50	2.80
2	4-R	2.45	6.4
3	4-R	1.65	6.4

Discharge vs Depth On Grate

Catch Basin No. 2&3
Capacities 8/3/95



Type 4-R
Catch Basin
Open Area
=1.9 FT²

Inlet Capacity
=±6.40 cfs

Headwater
Depth = 0.5'

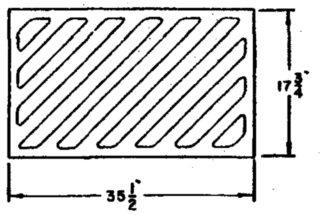
2105 Capurro Way Suite F
Sparks, NV 89431
(702) 359-3303 Fax (702) 359-3329
Odyssey ENGINEERING INCORPORATED

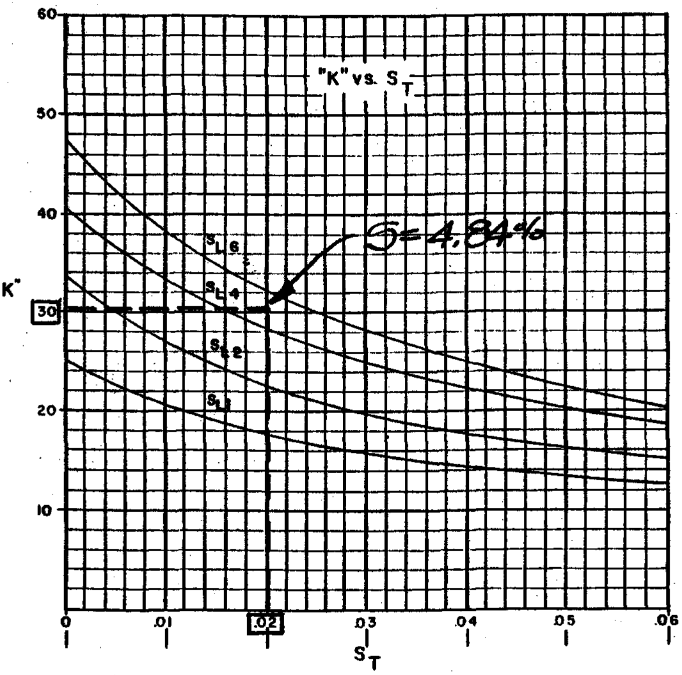
JOB LANCER ESTATES UNIT No. 11
SHEET NO. 3 OF 3
CALCULATED BY HAMMOND DATE 9/9/95
CHECKED BY ________ DATE ________
SCALE ________

CAT. NO.- R-3067
DESCRIPTION- DIAGONAL REVERSIBLE
COMR CODE- 3067-0004

TYPE 4-R INLET GRATE

FLOW ⟶



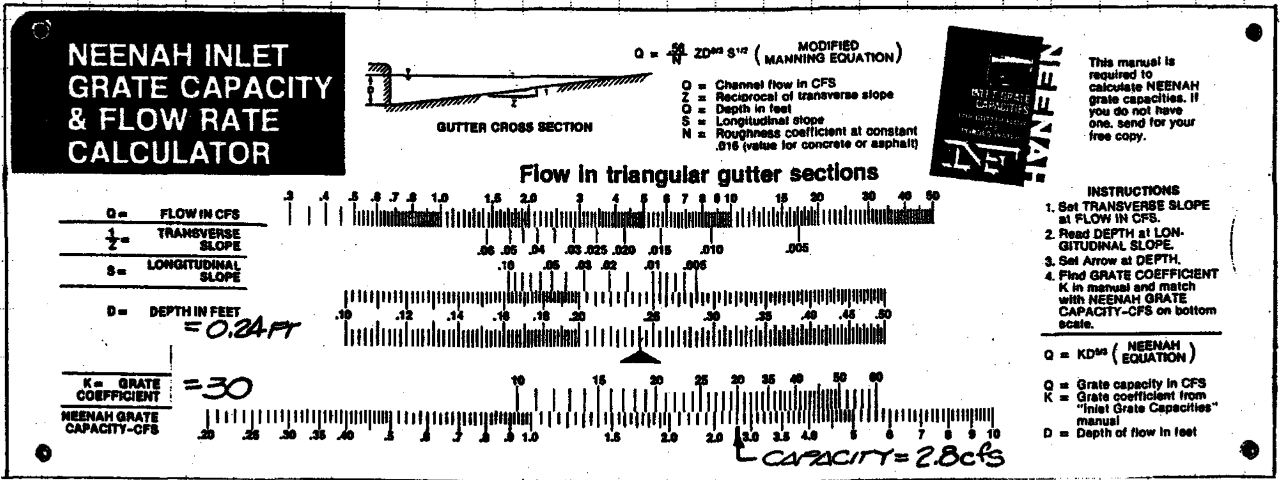


S_T = TRANSVERSE GUTTER SLOPE

S_L = LONGITUDINAL GUTTER SLOPE

K = GRATE INLET COEFFICIENT

GRATE COEFFICIENT GRAPH "K"



NEENAH INLET GRATE CAPACITY & FLOW RATE CALCULATOR

$Q = \frac{.56}{N} Z D^{8/3} S^{1/2}$ (MODIFIED MANNING EQUATION)

Q = Channel flow in CFS
Z = Reciprocal of transverse slope
D = Depth in feet
S = Longitudinal slope
N = Roughness coefficient at constant .016 (value for concrete or asphalt)

This manual is required to calculate NEENAH grate capacities. If you do not have one, send for your free copy.

INSTRUCTIONS
1. Set TRANSVERSE SLOPE at FLOW IN CFS.
2. Read DEPTH at LONGITUDINAL SLOPE.
3. Set Arrow at DEPTH.
4. Find GRATE COEFFICIENT K in manual and match with NEENAH GRATE CAPACITY-CFS on bottom scale.

$Q = KD^{5/3}$ (NEENAH EQUATION)

Q = Grate capacity in CFS
K = Grate coefficient from "Inlet Grate Capacities" manual
D = Depth of flow in feet

=0.24 FT

=30

CAPACITY = 2.8cfs

GRATE CAPACITY ~ cfs

```
                               NATURAL CHANNELS

RIABLES LIST:
        Y - FLOW ELEVATION      Q - FLOWRATE      S - CHANNEL SLOPE

RIABLE TO BE SOLVED (Y,Q OR S) ? Q        Enter up to 20 cross-section points.
                                         Enter <Return> only for distance to end.
 (FT)      ? 99.755
 (FT/FT)   ? .0484                                 CROSS-SECTION POINTS
                                         DIST  ELEV   COEFF    DIST  ELEV   COEFF
                                         ------------------    ------------------
                                         0     100    .016
                                         0.1   99.5   .016
     RESULTS                             1.5   99.625.016
================                         16    99.915.016
        3.24 CFS
        0.70 SF
        8.17 FT
=       4.64 FPS
        2.76   SUPER-CRITICAL FLOW

hift> <Prt Sc> print         <Return> repeat        <Space Bar> back to menu
-------------------------------------------------------------------------------
```

```
                              NATURAL CHANNELS

ARIABLES LIST:
        Y - FLOW ELEVATION      Q - FLOWRATE      S - CHANNEL SLOPE

ARIABLE TO BE SOLVED (Y,Q OR S) ? Q        Enter up to 20 cross-section points.
                                          Enter <Return> only for distance to end.
Y (FT)      ? 99.755
S (FT/FT)   ? .06                                CROSS-SECTION POINTS
                                          DIST  ELEV  COEFF    DIST  ELEV  COEFF
                                          -----------------    -----------------
                                          0     100    .016
                                          0.1   99.5   .016
    RESULTS                               1.5   99.625.016
=================                         16    99.915.016
Q=        3.60 CFS
 =        0.70 SF
 =        8.17 FT
V=        5.16 FPS
 =        3.07   SUPER-CRITICAL FLOW

Shift> <Prt Sc> print          <Return> repeat          <Space Bar> back to menu
---------------------------------------------------------------------------------
```

```
                              NATURAL CHANNELS

ARIABLES LIST:
        Y - FLOW ELEVATION      Q - FLOWRATE      S - CHANNEL SLOPE

ARIABLE TO BE SOLVED (Y,Q OR S) ? Q        Enter up to 20 cross-section points.
                                         Enter <Return> only for distance to end.
Y (FT)      ? 100
 (FT/FT)    ? .0484                          CROSS-SECTION POINTS
                                         DIST  ELEV  COEFF   DIST  ELEV  COEFF
                                         -----------------   -----------------
                                         0     100    .016
                                         4.5   99.91 .016
    RESULTS                              4.51  99.41 .016
=================                        6     99.535.016
 =      110.74 CFS                       20.5  99.825.016
A=       11.27 SF                        35    99.535.016
P=       42.00 FT                        36.5  99.41 .016
 =        9.83 FPS                       36.51 99.91 .016
 =        3.30   SUPER-CRITICAL FLOW     41    100    .016

hift> <Prt Sc> print        <Return> repeat        <Space Bar> back to menu
---------------------------------------------------------------------------
```

```
                              NATURAL CHANNELS
ARIABLES LIST:
        Y - FLOW ELEVATION      Q - FLOWRATE      S - CHANNEL SLOPE

ARIABLE TO BE SOLVED (Y,Q OR S) ? Q         Enter up to 20 cross-section points.
                                        Enter <Return> only for distance to end.
 (FT)      ? 100
 (FT/FT)   ? .06                                 CROSS-SECTION POINTS
                                        DIST  ELEV  COEFF   DIST  ELEV  COEFF
                                        -----------------   -----------------
                                        0     100   .016
                                        4.5   99.91 .016
    RESULTS                             4.51  99.41 .016
===============                         6     99.535.016
=      123.30 CFS                       20.5  99.825.016
=       11.27 SF                        35    99.535.016
        42.00 FT                        36.5  99.41 .016
        10.94 FPS                       36.51 99.91 .016
=        3.68   SUPER-CRITICAL FLOW     41    100   .016

Shift> <Prt Sc> print        <Return> repeat        <Space Bar> back to menu
-------------------------------------------------------------------------------
```

```
           Triangular Channel Analysis & Design
              Open Channel - Uniform flow

Worksheet Name: LANCER 11

Comment: 100 YEAR OVERFLOW DITCH CAPACITY

Solve For Discharge

Given Input Data:
        Left Side Slope..     3.00:1 (H:V)
        Right Side Slope.     3.00:1 (H:V)
        Manning's n......     0.035
        Channel Slope....     0.0470 ft/ft
        Depth............     1.00 ft

Computed Results:
        Discharge........    16.80 cfs
        Velocity.........     5.60 fps
        Flow Area........     3.00 sf
        Flow Top Width...     6.00 ft
        Wetted Perimeter.     6.32 ft
        Critical Depth...     1.14 ft
        Critical Slope...     0.0231 ft/ft
        Froude Number....     1.40 (flow is Supercritical)


  Open Channel Flow Module, Version 3.42 (c) 1991
  Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708
```

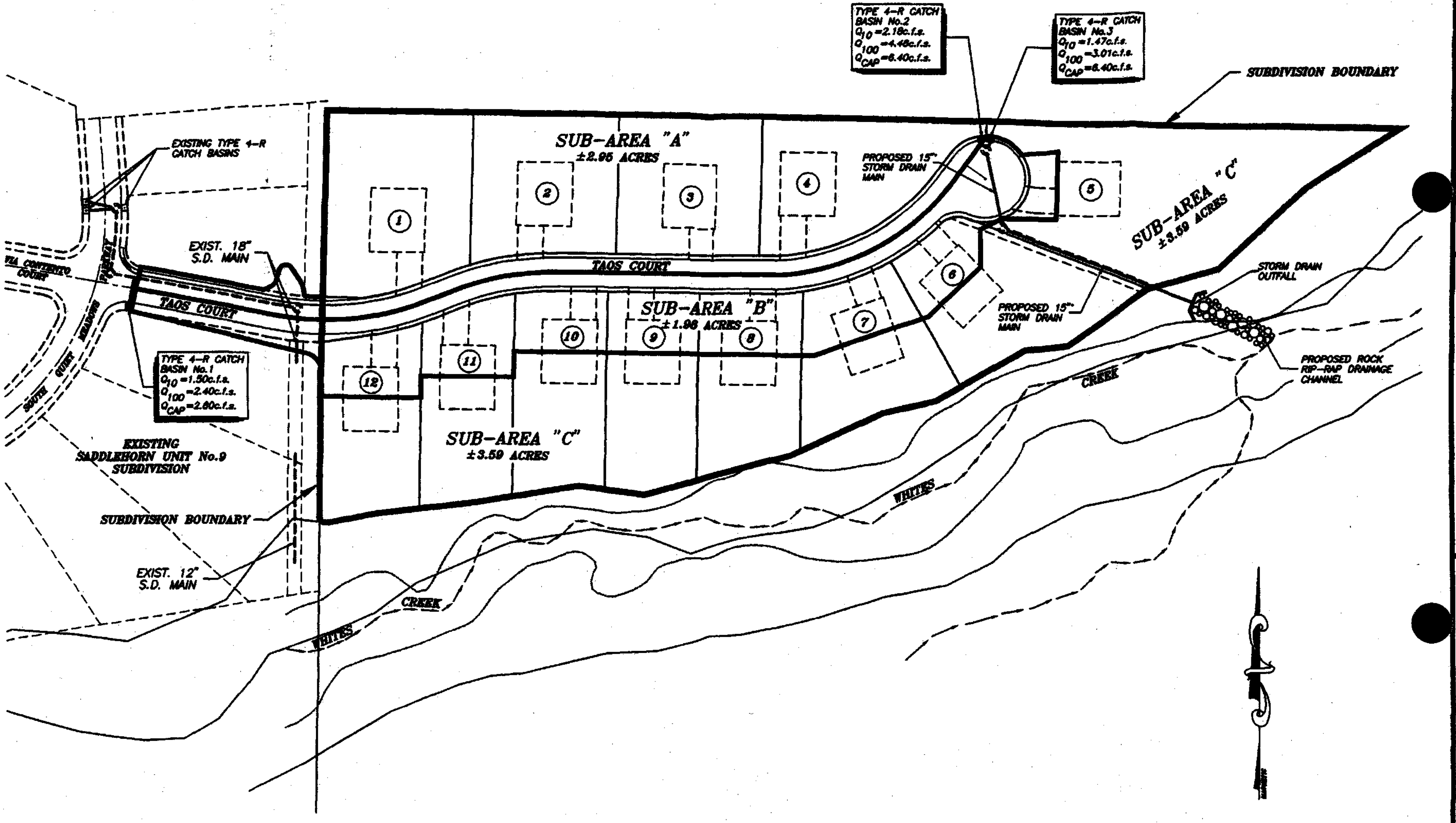


FIGURE No.2
PROPOSED DRAINAGE CONDITION

LANCER ESTATES UNIT No. 11
PROPOSED CONDITIONS

odyssey ENGINEERING INCORPORATED

DATE: AUGUST 1996
DRAWN BY: D.B.
DESIGNED BY: B.B.
CHECKED BY:

SCALE
HORIZ. N/A
VERT. N/A
JOB NO. 2014

SHEET
2
OF
2

Exhibit 15

Exhibit 15

TECHNICAL DRAINAGE REPORT

The Estates at Mt. Rose Unit 3A
A Single Family Home
Residential Community

Prepared for:

Reynen and Bardis
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December 29, 2005

GARY M. PROBERT
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CIVIL
No. 7721

1-12-06

WOOD RODGERS INC.
ENGINEERING PLANNING MAPPING SURVEYING

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Modified Form 2 (Tc Calculations)
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Introduction

This report presents the storm water management plan for The Estates at Mount Rose Subdivision Unit 3A. The Project site, Unit 3A encompasses 147 acres and is bounded to the south by Mount Rose Highway (Nevada State Route 431), and to the north Whites Creek. Access to the site is off of Mount Rose Highway and Callahan Ranch Road. Callahan Ranch Road runs north-south through the project and intersects Mount Rose Highway. The far northern parcel (the area north of White's Creek) will become Unit 3B. The Existing Unit 3B parcel is currently accessed by Mountain Ranch Road. The project site is contained in Section 35, Township 18 North, Range 19 East, M.D.M., in Washoe County, Nevada.

The proposed development consists of 59 single-family residential lots of approximately 46,000 sq. ft each. The total area for the project consists of 228 acres. The purpose of this report is to show the drainage plan for The Estates at Mount Rose Unit 3A complies with the criteria set forth in the Washoe County Drainage Design Manual.

Previous Studies

Black Eagle Consulting conducted a geotechnical investication in June of 2003. The findings of this investigation are included in the report entitled, "Geotechnical Investigation The Estates at Mount Rose Callahan Ranch Road and Mount Rose Highway Washoe County, Nevada."

Nimbus Engineers had done a Master Report "Flood Control Master Plan Mt. Rose Estates" Revised Oct. 24, 2003. Unit 3A is in compliance with their Master Report.

Hydrologic and Hydraulic Analysis

The Rational Method (Q=CIA) was used for computing flows contributing to the storm drainage system as outlined in section 704 of the Washoe County Drainage Design Manual (Manual).

The drainage system was designed using the 5-year and 100-year storm events. Table 603 (see Appendix) of the Manual lists the rainfall depth, duration, and frequency data for Region 2 for both the 5-year and 100-year storm events.

Hydrologic parameters for the analysis were determined as shown in section 704 of the Manual. The Rational Equation requires an area, intensity, and runoff coefficient to determine the flow at each inlet location. The time of concentration was calculated for each sub-basin using a modified version of the Standard Form 2. The intensity for each sub-basin was derived from linear interpolation using the calculated time of concentration and the listed values from Table 603. The runoff coefficients were derived from Table 701 (see Appendix) of the Manual for 1-acre lots (~43,560sq. ft.), for open space, and undeveloped rangeland based on the storm event. Areas for each sub-basin were determined from tracing polylines along sub-basin boundaries and listing the polyline properties to obtain the area.

Hydraulic computations were performed to analyze the storm drain system to ensure compliance with section 900 of the Drainage Manual. Section 902.1 requires the HGL to be one foot below the final grade above the storm sewer at all locations for the design storm. The design storm is typically the minor (5-year) event unless the design of the storm drain system meets one of the conditions noted in Section 901 of the Manual. StormCAD Version 5.0 was utilized to model the pipe network including; energy losses due to pipe friction, and at junctions, inlets, and outlets and to draw the hydraulic grade line. StormCAD also provides flow rates and velocities at outlets so outlet protection can be appropriately sized based on Section 807.3 of the manual.

Each catch basins amount of intake to the storm drain system was determined from inputting parameters (taken from figures 907 and 906 of the Washoe County Hydrologic Criteria and Drainage Design Manual) into StormCAD where orifice or weir equations are used based on sump or on grade conditions to solve for capacity and performance of the inlet. Figures 907 and 906 are located in the appendix.

Maximum catch basin capacities were also determined from figures 906 and 907 of the Washoe County Hydrologic Criteria and Drainage Design Manual. Street slopes and depth of flow (taken from street spread calculations using Flow Master) were used to determine from the figures what their maximum capacities were.

Section 304.4 of the Manual stipulates the requirements for flooding of streets. For the minor storm event a local street must maintain a 12-foot centered dry travel lane, and the velocity must be less than 6fps. For a major storm the street may be flooded and the velocity must be less than 6fps. To analyze the street capacity a depth of flow and velocity of flow was determined by using Flow Master. In Flow Master a custom half street section was created. Slopes, coefficients, and flows were inputted to determine velocities and depths of flows at each catch basin. This has been done for both the 5 and 100 year storm events and included in there respected locations of this report.

Detention for this project will be provided by a series of detention basins that will be constructed with each phase of the overall Estates at Mt. Rose project. The SCS Method was utilized in conjunction with HEC-1 Version 4.0.1E to determine pre-development hydrographs for the 5 yr and 100 yr storm events. Detention ponds were sized to reduce post-development peak flow rates to below the pre-development peak flow rates for both the 5 yr and 100 yr storms. Emergency Overflow route for Pond 2 in the HEC-1 Model is to discharge into a special overflow grate drain which carries additional storm water, above the 100 year storm, to Whites Creek via a 24" pipe. Storm water will only spill into the overflow grate when the storm water elevation in the pond reaches above the 100 year elevation. This 100 year surface elevation is 5570.90'. Additionally a weir has been created to discharge additional flows that may occur with storms greater than the 100 year storm event to Whites Creek on the north side of Pond 2. The overtopping point elevation for the weir is 5571'. This is above the 100 year storm water elevation and lower then the top pond height.

The post flows for this project are below the pre flow amounts. The table below shows the Pre and Post Development flows for the 5 and 100 year storm events. Also included below is Pond 2's figures which include size in (Ac-ft), and water surface elevations.

PRE and POST PEAK FLOWS

	5YR		100YR	
	Peak Flow	Peak Time	Peak Flow	Peak Time
PRE	64 cfs	15.42 hrs	302 cfs	15.42 hrs
POST	63 cfs	15.25 hrs	298 cfs	15.33 hrs

POND 2 FIGURES

Peak flow in (5yr):	34.22 cfs
Peak flow in (100yr):	130.41 cfs
Peak flow out :	70.00 cfs
100yr surface elevation:	5570.90'
Peak 100yr storage:	5.00 ac-ft
Peak time:	15.33 hrs

ELEVATION	5564'	5566'	5568'	5570'	5572'
AREA (acres)	0.01	0.55	1.08	1.23	1.26
Storage (ac-ft)	0.00	0.42	2.02	4.33	6.82
DISCHARGE (cfs)	0.00	8	12	35	113

Near the south east corner of the project lays Pond 2. Before entering the pond, storm water must be conveyed through three 36" RCP's where the sewer and emergency access easement crosses Ditch 5. This culvert was designed using Culvert Master. Two scenarios have been planned and designed for. The proposed pond has a bottom elevation of 5464'. The top of the pond has an elevation of 5572'. However, the 100 year flood event should only raise the storm water elevation to a level of 5570.90'. This proposed triple 36" culvert has been designed to convey storm water when the pond is filling and while the detention pond is flooded and tail water becomes an issue. The IE out of these 36" pipes is 5569.22. When the pond is flooded the tail waters increase; thus two scenarios have to be designed to. These calculations have there own places in this report under Triple 36" Culvert for Sewer/Emergency Access Easement.

Historic Drainage System

The historic drainage for this parcel consists of overland sheet flow that travels in an easterly direction where it is either intercepted by Whites Creek (a 4' to 8' wide perennial stream) or the drainage ditch that borders the Mount Rose Highway.

Proposed Drainage System

The proposed drainage system consists of overland flow in drainage swales to the curb and gutter system to direct the flow to catch basins and into pipes that direct the water to a detention pond and eventually release the water into Whites Creek.

The drainage basin plans are included in the Appendix (see Map Pocket 1). The drainage plans shows each storm drain line and the corresponding inlets and the areas associated with each inlet. Overland flow equations were used to calculate the initial travel time, and Manning's Equation was used to calculate the channelized flow (see the Modified Form 2 in the 5 and 100 year sections).

The proposed storm drain system is shown as a tree structure with element annotation on three 24" by 36" sheets for both the 5-year and 100-year storms (see Map Pocket 2). Pertinent output for each element is shown on these sheets. Additionally, storm drain profiles are included to show the hydraulic grade lines for the pipe systems in both the 5-year and 100-year storms.

Sediment transport and erosion will be controlled through sizing of outlet and inlet protection, slope stabilization with riprap and vegetation, and through conformance of the Storm Water Pollution Prevention Plan (SWPPP) that has been prepared for this site. The SWPPP includes Best Management Practices (BMPs), a maintenance schedule, and a list of the responsible parties for maintenance to insure the storm drain system operates correctly to prevent excessive sediment transport. Calculations for outlet protection and erosion control using riprap are included in the Appendix of this report.

Flood Hazard Areas

The proposed project is found mostly within flood zone X on the FEMA Flood Insurance Rate Maps (FIRM) 32031C3165E and 32031C3170E, dated September 30th, 1994. Flood Zone X indicates areas that are outside of the 500-year floodplain. Two FIRMettes are included in the Appendix of this report. A small portion of the project lies within Flood Zone A. No homes are designed to be within the FEMA Flood Zone A, shown on the same maps. The Basin Maps located in the appendix of the report show a better proximity of Flood Zone A to our project.

Conclusions

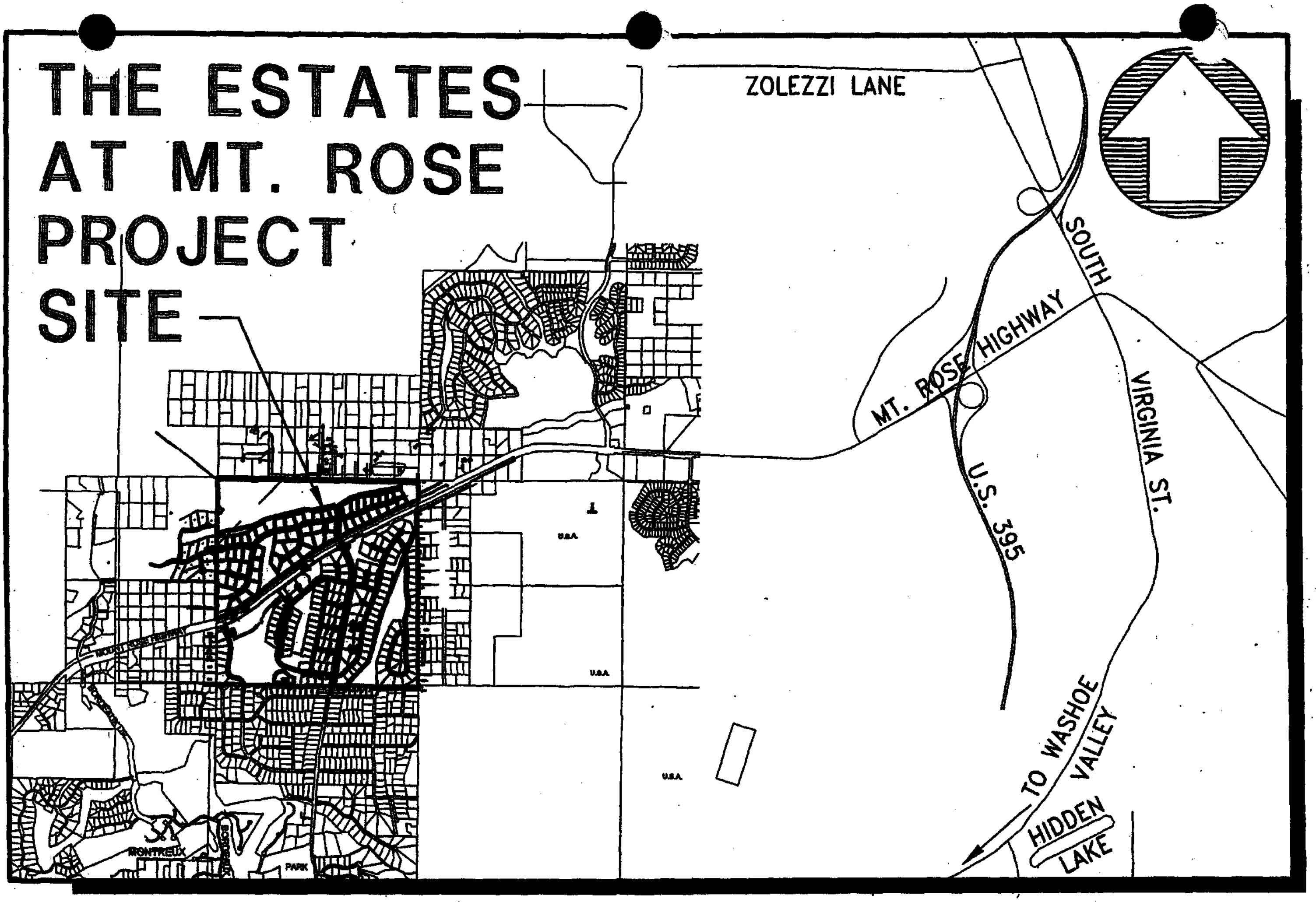
The proposed drainage system is designed in compliance with all manual policies and requirements. The proposed design is also in compliance with Washoe County Requirements. The proposed drainage facilities will reduce the amount of existing sedimentation through erosion control measures.

References

Geotechnical Investigation The Estates at Mount Rose Callahan Ranch Road and Mount Rose Highway. Washoe County, Nevada. June 2003.

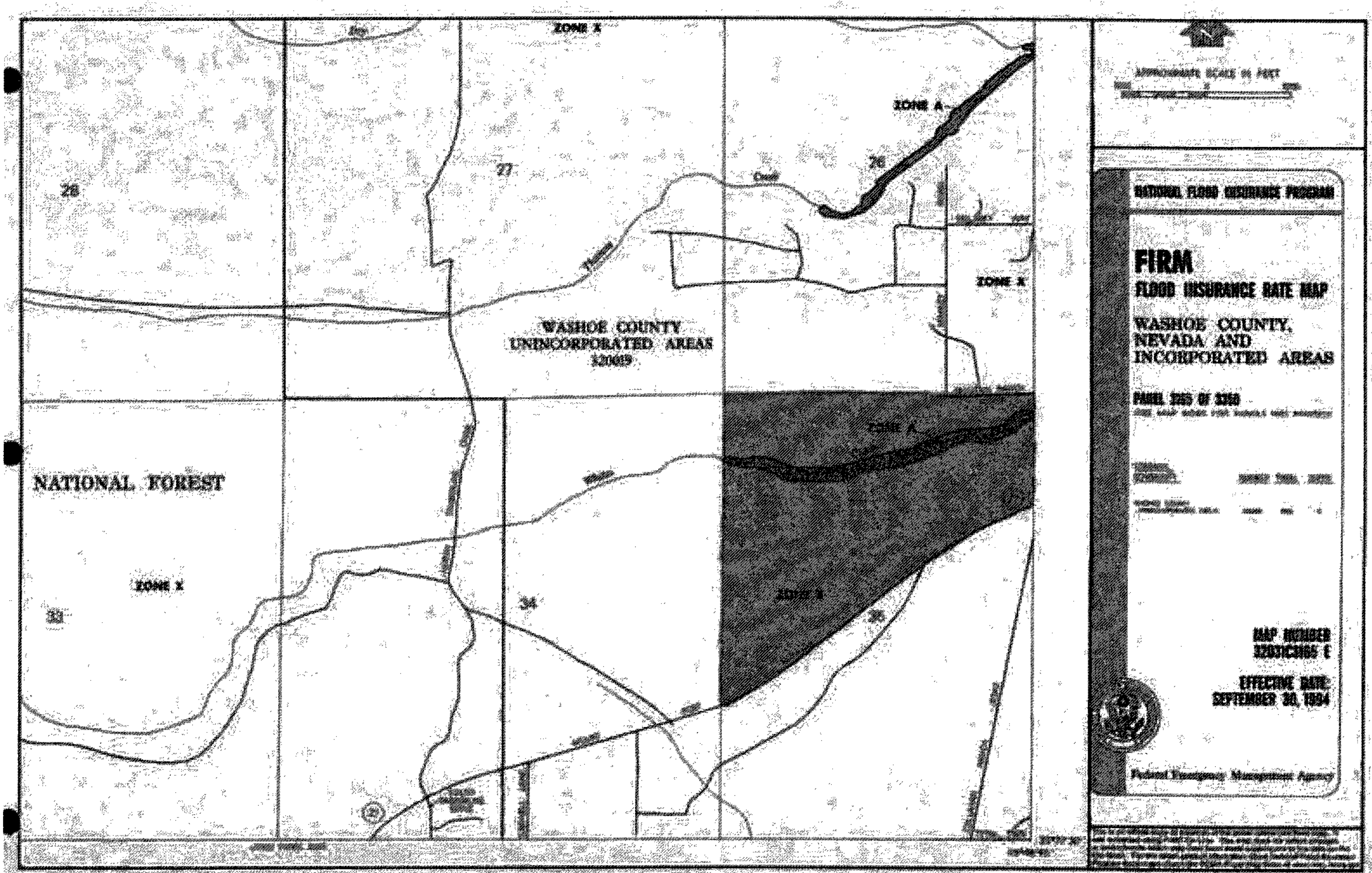
Washoe County Hydrologic Criteria and Drainage Design Manual, July 1998

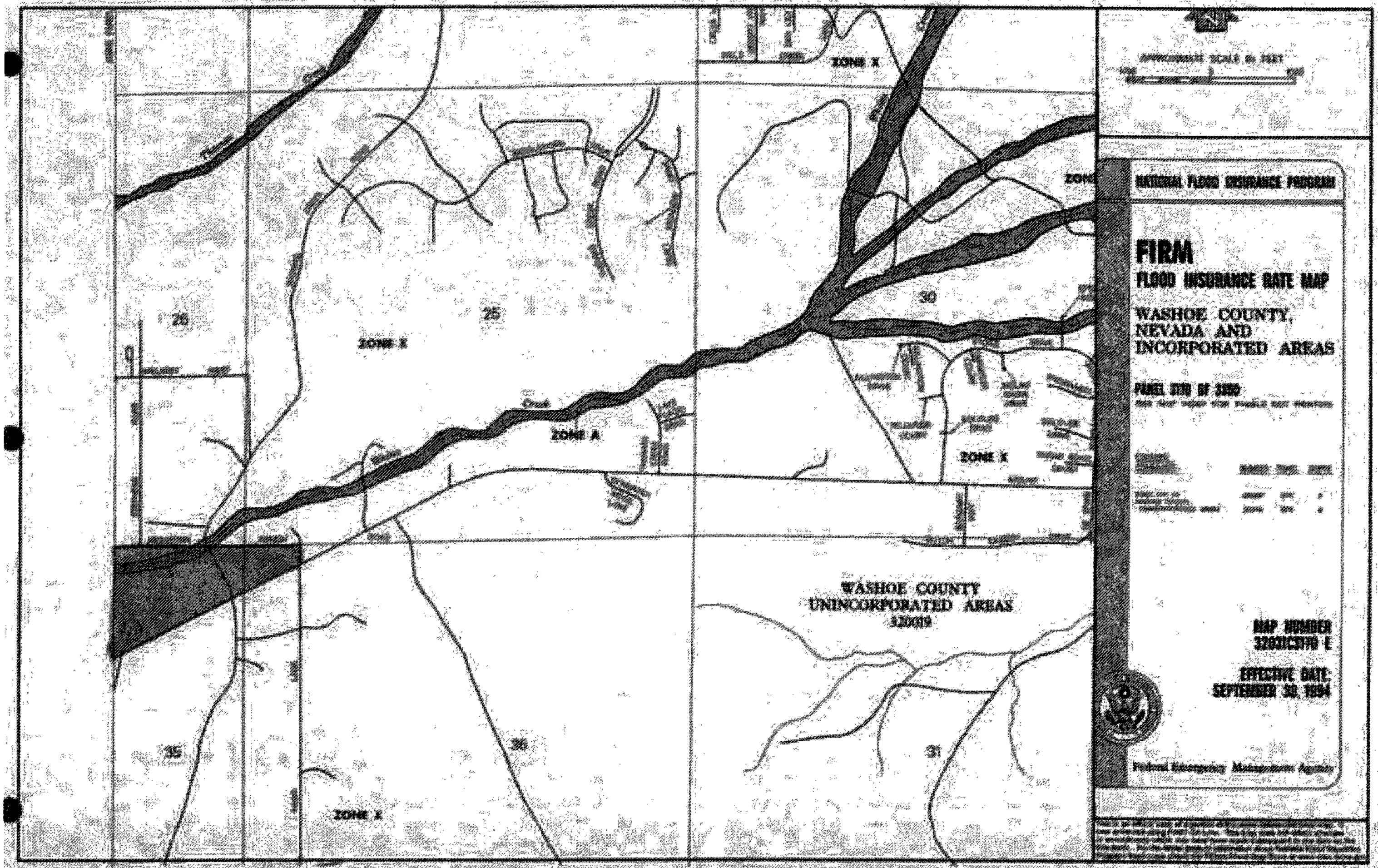
<u>Appendices</u>



VICINITY MAP

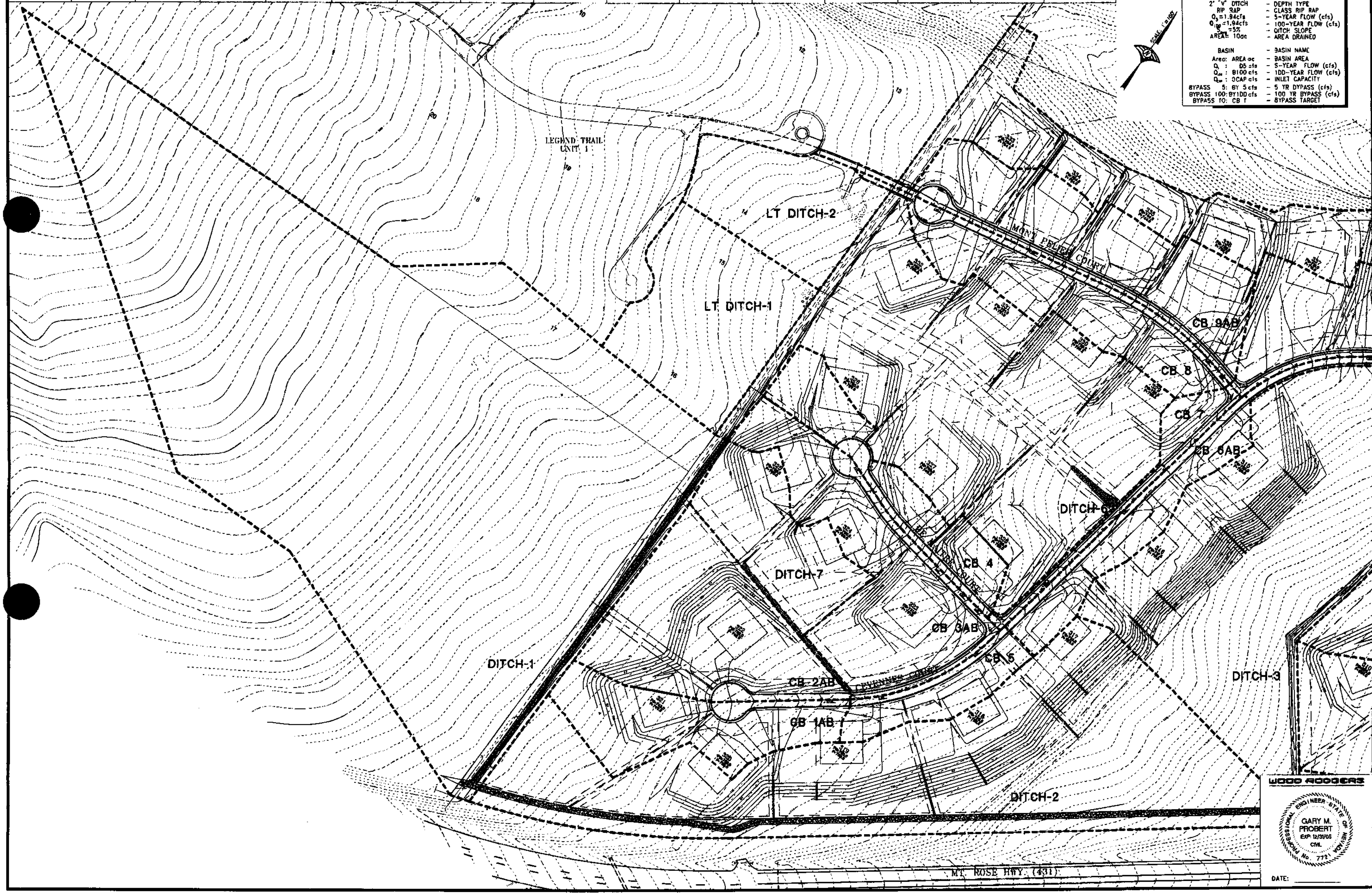
NOT TO SCALE


ZONE X
ZONE A
ZONE X
NATIONAL FOREST
ZONE X
WASHOE COUNTY
UNINCORPORATED AREAS
320019
28
27
26
33
34
35
Creek
APPROXIMATE SCALE IN FEET
NATIONAL FLOOD INSURANCE PROGRAM
FIRM
FLOOD INSURANCE RATE MAP
WASHOE COUNTY, NEVADA AND INCORPORATED AREAS
PANEL 3165 OF 3350
MAP NUMBER
32031C3165 E
EFFECTIVE DATE:
SEPTEMBER 30, 1994
Federal Emergency Management Agency


APPROXIMATE SCALE IN FEET
ZONE X
ZONE A
ZONE X
ZONE X
ZONE X
Creek
25
26
30
31
35
36
WASHOE COUNTY
UNINCORPORATED AREAS
320019
NATIONAL FLOOD INSURANCE PROGRAM
FIRM
FLOOD INSURANCE RATE MAP
WASHOE COUNTY,
NEVADA AND
INCORPORATED AREAS
PANEL 3170 OF 3350
MAP NUMBER
32031C3170 E
EFFECTIVE DATE:
SEPTEMBER 30, 1994
Federal Emergency Management Agency

DITCH-1 3' "TRAP" DITCH 2' BOTTOM 2:1 SIDE SLOPES 0.5' CL150 RIP RAP Q_5=13.18cfs Q_{100}=46.80cfs S_{avg}=1.50% AREA= 27.9ac	DITCH-2 3' "TRAP" DITCH 2' BOTTOM 2:1 SIDE SLOPES 3.6' CL550 RIP RAP Q_5=17.54cfs Q_{100}=62.29cfs S_{avg}=5.00% AREA= 39.04ac	DITCH-3 3' "V" DITCH 2:1 SIDE SLOPES NO RIP RAP Q_5=4.89cfs Q_{100}=17.08cfs S_{avg}=1.25% AREA= 10.53ac	DITCH-6 2.5' "TRAP" DITEH 1' BOTTOM 2:1 SIDE SLOPES 1.6' CL300 RIP RAP Q_5=7.71cfs Q_{100}=27.40cfs S_{avg}=6.30% AREA= 15.24ac	DITCH-7 1' "V" DITCH 2:1 SIDE SLOPES 1' CL150 RIP RAP Q_5=1.18cfs Q_{100}=4.19cfs S_{avg}=7.7% AREA= 2.12ac	LEGEND TRAILS DITCH-1 2.2' "V" DITCH NO RIP RAP Q_5=1.75cfs Q_{100}=6.23cfs S_{avg}=1.25% AREA= 3.52ac	LEGEND TRAILS DITCH-2 2.0' "V" DITCH NO RIP RAP Q_5=1.23cfs Q_{100}=4.37cfs S_{avg}=1.25% AREA= 2.31ac

CB 1AB	CB 2AB	CB 3AB	CB 4	CB 5	CB 6AB	CB 7	CB 8	CB 9AB
Area: 2.25 ac	Area: 3.56 ac	Area: 3.69 ac	Area: 1.76 ac	Area: 1.00 ac	Area: 1.53 ac	Area: 0.70 ac	Area: 2.35 ac	Area: 3.51 ac
Q_5 : 1.08 cfs	Q_5 : 1.53 cfs	Q_5 : 2.37 cfs	Q_5 : 0.96 cfs	Q_5 : 0.57 cfs	Q_5 : 0.97 cfs	Q_5 : 0.43 cfs	Q_5 : 1.19 cfs	Q_5 : 1.75 cfs
Q_{100} : 2.76 cfs	Q_{100} : 3.80 cfs	Q_{100} : 10.01 cfs	Q_{100} : 3.40 cfs	Q_{100} : 1.85 cfs	Q_{100} : 4.68 cfs	Q_{100} : 1.53 cfs	Q_{100} : 4.22 cfs	Q_{100} : 6.20 cfs
Q_{cap} : 9.60 cfs	Q_{cap} : 9.60 cfs	Q_{cap} : 11.00 cfs	Q_{cap} : 5.50 cfs	Q_{cap} : 4.25 cfs	Q_{cap} : 11.00 cfs	Q_{cap} : 5.50 cfs	Q_{cap} : 5.50 cfs	Q_{cap} : 11.00 cfs
BYPASS 5: 0.15 cfs	BYPASS 5: 0.42 cfs	BYPASS 5: 0 cfs	BYPASS 5: 0 cfs	BYPASS 5: 0.15 cfs	BYPASS 5: 0 cfs	BYPASS 5: 0 cfs	BYPASS 5: 0 cfs	BYPASS 5: 0 cfs
BYPASS 100: 1.62 cfs	BYPASS 100: 3.13 cfs	BYPASS 100: 0 cfs	BYPASS 100: 0 cfs	BYPASS 100: 1.78 cfs	BYPASS 100: 0 cfs	BYPASS 100: 0 cfs	BYPASS 100: 0 cfs	BYPASS 100: 0 cfs
BYPASS TO: CB 5	BYPASS TO: CB 3AB	BYPASS TO: CB 4	BYPASS TO: CB 3AB	BYPASS TO: CB 6AB	BYPASS TO: CB 7	BYPASS TO: CB 6AB	BYPASS TO: CB 9AB	BYPASS TO: CB 8



LEGEND

▬ ▬	DRAINAGE SHED
→	OVERLAND RELEASE DIRECTION

DITCH D-D# - DITCH NAME
2' "V" DITCH - DEPTH TYPE
RIP RAP - CLASS RIP RAP
Q_5=1.94cfs - 5-YEAR FLOW (cfs)
Q_{100}=1.94cfs - 100-YEAR FLOW (cfs)
S_{avg}=5% - DITCH SLOPE
AREA= 10ac - AREA DRAINED

BASIN - BASIN NAME
Area: AREA ac - BASIN AREA
Q_5 : Q5 cfs - 5-YEAR FLOW (cfs)
Q_{100} : Q100 cfs - 100-YEAR FLOW (cfs)
Q_{cap} : QCAP cfs - INLET CAPACITY
BYPASS 5: BY 5 cfs - 5 YR BYPASS (cfs)
BYPASS 100: BY100 cfs - 100 YR BYPASS (cfs)
BYPASS TO: CB T - BYPASS TARGET

IMPROVEMENT PLANS FOR
THE ESTATES AT MT. ROSE
PHASE 3A
HYDROLOGICAL BASINS MAP
WASHOE COUNTY NEVADA

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DATE: DECEMBER, 2005
SCALE: H: 1"=200'
DRAWN BY: JFF
DESIGNED BY: GMP
CHECKED BY: GMB

GARY M. PROBERT
EXP. 12/31/05

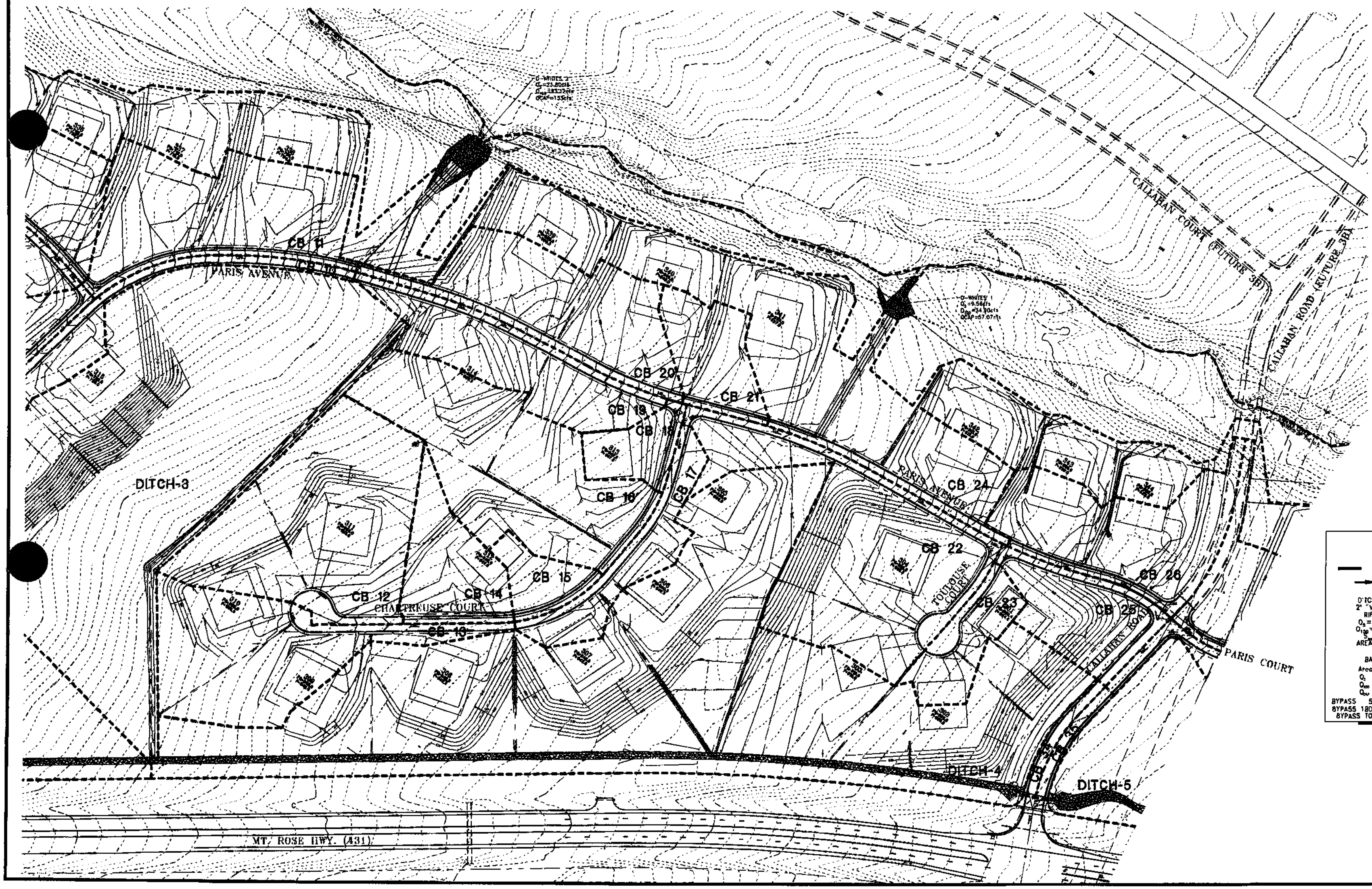
DATE:

PROJECT NO. 1558.001
DRAWING
H-1
SHT 1 OF 3

	CB 10	CB 11	CB 12AB	CB 13	CB 14	CB 15	CB 16AB	CB 17AB	CB 18
Area:	0.28 ac	2.64 ac	2.53 ac	2.53 ac	4.80 ac	0.50 ac	3.22 ac	0.97 ac	0.58 ac
Q_5 :	0.17 cfs	1.28 cfs	1.50 cfs	0.92 cfs	0.78 cfs	0.50 cfs	1.48 cfs	0.93 cfs	0.79 cfs
Q_{100} :	0.60 cfs	2.73 cfs	3.99 cfs	2.23 cfs	2.38 cfs	1.98 cfs	5.07 cfs	4.35 cfs	5.96 cfs
Q_{CAP} :	6.42 cfs	6.42 cfs	8.35 cfs	4.25 cfs	4.25 cfs	4.25 cfs	7.50 cfs	11.00 cfs	5.50 cfs
BYPASS 5:	0 cfs	0.18 cfs	0.43 cfs	0.42 cfs	0.30 cfs	0.10 cfs	0.31 cfs	0 cfs	0 cfs
BYPASS 100:	0 cfs	2.17 cfs	2.88 cfs	2.52 cfs	2.88 cfs	2.01 cfs	2.83 cfs	0 cfs	0 cfs
BYPASS TO:	CB 18	CB 19	CB 14	CB 16AB	CB 15	CB 16AB	CB 18	CB 18	CB 17AB

	CB 19	CB 20	CB 21	CB 22AB	CB 23	CB 24	CB 25	CB 26AB	CB 34	CB 35
Area:	1.79 ac	2.27 ac	0.50 ac	3.81 ac	0.77 ac	2.32 ac	1.23 ac	2.23 ac	0.17 ac	0.16 ac
Q_5 :	0.89 cfs	1.14 cfs	0.66 cfs	1.89 cfs	0.44 cfs	1.13 cfs	0.68 cfs	1.42 cfs	0.10 cfs	0.18 cfs
Q_{100} :	2.13 cfs	3.13 cfs	2.51 cfs	6.71 cfs	1.58 cfs	3.33 cfs	2.40 cfs	8.26 cfs	0.37 cfs	0.35 cfs
Q_{CAP} :	6.00 cfs	6.00 cfs	5.25 cfs	11.00 cfs	5.59 cfs	6.00 cfs	5.50 cfs	11.00 cfs	5.50 cfs	5.50 cfs
BYPASS 5:	0.84 cfs	0.18 cfs	0.09 cfs	0 cfs	0 cfs	0.18 cfs	0 cfs	0 cfs	0 cfs	0 cfs
BYPASS 100:	1.18 cfs	1.18 cfs	2.81 cfs	0 cfs	0 cfs	3.69 cfs	0 cfs	0 cfs	0 cfs	0 cfs
BYPASS TO:	CB 18	CB 21	CB 24	CB 23	CB 22AB	CB 26AB	CB 26AB	CB 25	DITCH 5	DITCH 5

DITCH-3	DITCH-4	DITCH-5
3' "V" DITCH	3.1' "TRAP" DITCH 2' BOTTOM	3.3' "TRAP" DITCH 2' BOTTOM
2:1 SIDE SLOPES NO RIP RAP	2:1 SIDE SLOPES 3.0' CL550 RIP RAP	2:1 SIDE SLOPES 3.0' CL550 RIP RAP
Q_5=4.81cfs	Q_5=26.69cfs	Q_5=29.98cfs
Q_{100}=17.08cfs	Q_{100}=94.76cfs	Q_{100}=106.46cfs
S_{avg}=1.25%	S_{avg}=5.00%	S_{avg}=5.32%
AREA= 10.53ac	AREA= 62.61ac	AREA= 73.55ac



LEGEND

▬ ▬	DRAINAGE SHED
→	OVERLAND RELEASE DIRECTION
DITCH D-0#	- DITCH NAME
2' "V" DITCH	- DTPTH TYPE
RIP RAP	- CLASS RIP RAP
Q_5=1.94cfs	- 5-YEAR FLOW (cfs)
Q_{100}=1.94cfs	- 100-YEAR FLOW (cfs)
S_{avg}-5%	- DITCH SLOPE
AREA= 18ac	- AREA DRAINED
BASIN	- BASIN NAME
Area: AREA ac	- BASIN AREA
Q_5 : Q5 cfs	- 5-YEAR FLOW (cfs)
Q_{100} : Q100 cfs	- 100-YEAR FLOW (cfs)
Q_{CAP} : QCAP cfs	- INLET CAPACITY
BYPASS 5: BY 5 cfs	- 5 YR BYPASS (cfs)
BYPASS 100: BY100 cfs	- 100 YR BYPASS (cfs)
BYPASS TO: CB T	- BYPASS TARGET

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EXP. 12/31/05
CIVIL
No. 7721

DATE: ____________

IMPROVEMENT PLANS FOR
THE ESTATES AT MT. ROSE
PHASE 3A
HYDROLOGICAL BASINS MAP
WASHOE COUNTY NEVADA

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DATE: DECEMBER, 2005
SCALE: H: 1"=200' V:
DRAWN BY: JFF
DESIGNED BY: GMP
CHECKED BY: GMP

NO.	DESCRIPTION	ENGR	BY	DATE

PROJECT NO.
1558.001

DRAWING
H-2

SHT 2 OF 3

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

Nevada 39.38 N 119.83 W 5688 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003
Extracted: Wed Dec 21 2005

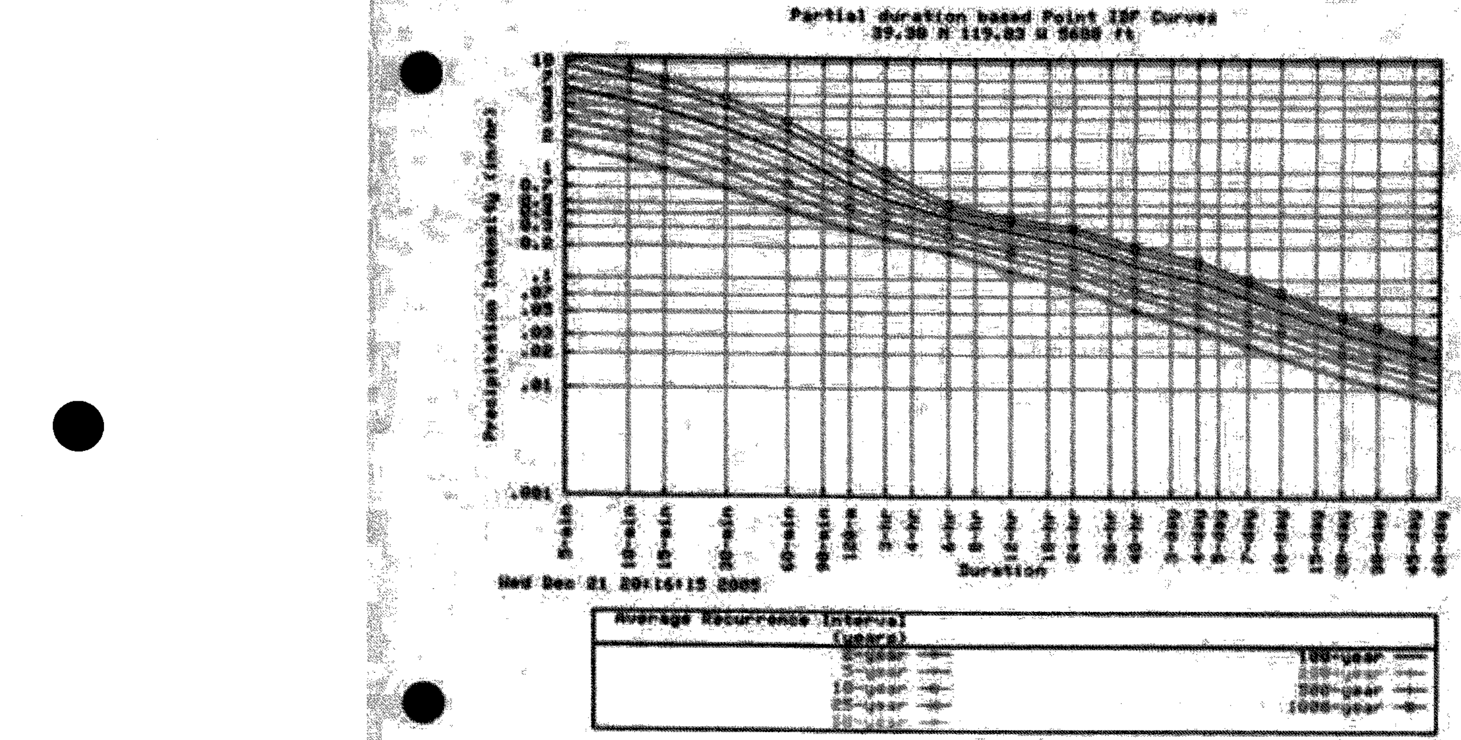
Confidence Limits | Seasonality | Location Maps | Other Info. | GIS data | Maps | Help

Precipitation Intensity Estimates (in/hr)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.67	1.27	1.05	0.71	0.44	0.29	0.24	0.17	0.12	0.08	0.05	0.04	0.02	0.02	0.01	0.01	0.01	0.01
5	2.29	1.74	1.44	0.97	0.60	0.38	0.30	0.22	0.15	0.11	0.07	0.05	0.03	0.03	0.02	0.01	0.01	0.01
10	2.86	2.17	1.80	1.21	0.75	0.45	0.36	0.26	0.18	0.13	0.08	0.06	0.04	0.03	0.02	0.02	0.01	0.01
25	3.78	2.88	2.38	1.60	0.99	0.57	0.43	0.30	0.22	0.16	0.10	0.07	0.05	0.04	0.03	0.02	0.02	0.01
50	4.66	3.54	2.93	1.97	1.22	0.67	0.49	0.34	0.24	0.18	0.12	0.08	0.06	0.05	0.03	0.02	0.02	0.02
100	5.69	4.33	3.58	2.41	1.49	0.78	0.55	0.37	0.27	0.21	0.14	0.10	0.07	0.05	0.03	0.03	0.02	0.02
200	6.95	5.29	4.37	2.94	1.82	0.94	0.65	0.41	0.30	0.24	0.16	0.11	0.08	0.06	0.04	0.03	0.02	0.02
500	8.99	6.84	5.65	3.81	2.35	1.21	0.83	0.46	0.34	0.27	0.18	0.13	0.09	0.07	0.04	0.03	0.03	0.02
1000	10.91	8.30	6.86	4.62	2.86	1.46	1.00	0.51	0.38	0.31	0.21	0.15	0.10	0.08	0.05	0.04	0.03	0.02

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.
Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.



Confidence Limits -

* Upper bound of the 90% confidence interval Precipitation Intensity Estimates (in/hr)																		
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.97	1.49	1.24	0.83	0.52	0.33	0.27	0.20	0.13	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01	0.01
5	2.72	2.08	1.71	1.15	0.71	0.43	0.34	0.25	0.17	0.13	0.08	0.06	0.04	0.03	0.02	0.02	0.01	0.01
10	3.37	2.57	2.12	1.43	0.89	0.52	0.40	0.29	0.20	0.15	0.10	0.07	0.05	0.04	0.02	0.02	0.02	0.01
25	4.51	3.43	2.84	1.91	1.18	0.66	0.48	0.35	0.25	0.19	0.13	0.08	0.06	0.05	0.03	0.02	0.02	0.02
50	5.65	4.30	3.56	2.39	1.48	0.79	0.56	0.39	0.28	0.22	0.15	0.10	0.07	0.05	0.03	0.03	0.02	0.02
100	7.06	5.37	4.44	2.99	1.85	0.97	0.68	0.43	0.32	0.25	0.17	0.11	0.08	0.06	0.04	0.03	0.02	0.02
200	8.92	6.79	5.61	3.77	2.34	1.20	0.84	0.48	0.36	0.29	0.20	0.13	0.09	0.07	0.04	0.04	0.03	0.02
500	11.93	9.08	7.50	5.05	3.13	1.60	1.11	0.57	0.42	0.34	0.24	0.16	0.11	0.08	0.05	0.04	0.03	0.03
1000	14.96	11.39	9.41	6.34	3.92	2.00	1.37	0.70	0.47	0.39	0.27	0.18	0.12	0.09	0.06	0.05	0.03	0.03

* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.
** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.
Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

* Lower bound of the 90% confidence interval

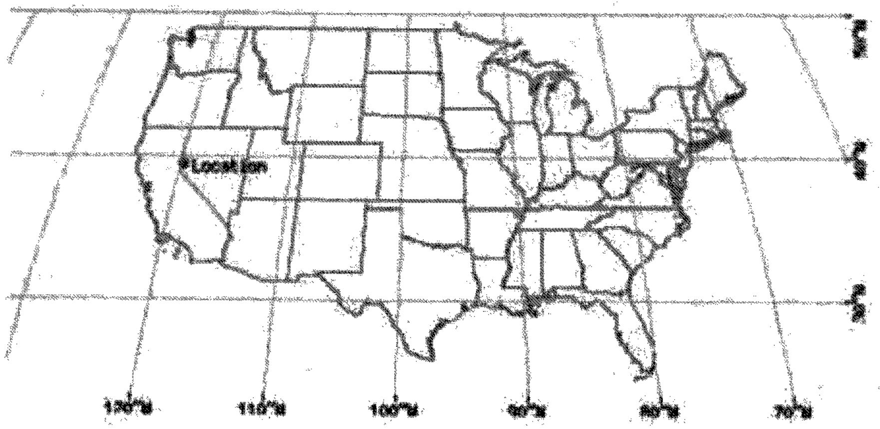
Precipitation Intensity Estimates (in/hr)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.44	1.10	0.91	0.61	0.38	0.26	0.21	0.15	0.11	0.07	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.01
5	1.98	1.51	1.24	0.84	0.52	0.34	0.27	0.19	0.14	0.09	0.06	0.04	0.03	0.02	0.02	0.01	0.01	0.01
10	2.44	1.85	1.53	1.03	0.64	0.40	0.32	0.22	0.16	0.11	0.07	0.05	0.03	0.03	0.02	0.01	0.01	0.01
25	3.11	2.37	1.96	1.32	0.82	0.49	0.37	0.26	0.19	0.13	0.09	0.06	0.04	0.03	0.02	0.02	0.01	0.01
50	3.68	2.80	2.31	1.56	0.96	0.55	0.42	0.29	0.21	0.15	0.10	0.07	0.05	0.04	0.02	0.02	0.02	0.01
100	4.31	3.28	2.71	1.82	1.13	0.62	0.46	0.31	0.23	0.17	0.11	0.08	0.06	0.04	0.03	0.02	0.02	0.01
200	5.02	3.81	3.15	2.12	1.31	0.72	0.54	0.33	0.25	0.19	0.12	0.09	0.06	0.05	0.03	0.03	0.02	0.02
500	6.01	4.57	3.78	2.55	1.57	0.88	0.66	0.36	0.27	0.21	0.14	0.11	0.07	0.05	0.04	0.03	0.02	0.02
1000	6.84	5.20	4.30	2.90	1.79	1.02	0.78	0.41	0.29	0.23	0.15	0.12	0.08	0.06	0.04	0.03	0.02	0.02

* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than.
** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -



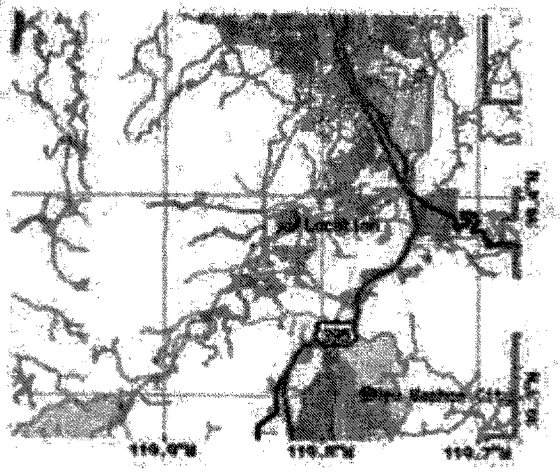
These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.

LEGEND

— State	— Connector
— County	▨ Stream
▨ Indian Resv	▨ Military Area
▨ Lake/Pond/Ocean	▨ National Park
— Street	▨ Other Park
— Expressway	▨ City
— Highway	— County

Scale 1:228583 0 2 4 6 8 mi / 0 2 4 6 8 10 km
*average--true scale depends on monitor resolution



Other Maps/Photographs -

View USGS digital orthophoto quadrangle (DOQ) covering this location from TerraServer; USGS Aerial Photograph may also be available
from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilts has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the National Digital Orthophoto Program (NDOP) for more information.

Watershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information
about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study,
please refer to our documentation.

Using the National Climatic Data Center's (NCDC) station search engine, locate other climate stations within:

+/-30 minutes ...OR... +/-1 degree of this location (39.38/-119.83). Digital ASCII data can be obtained directly from NCDC.

Find Natural Resources Conservation Service (NRCS) SNOTEL (SNOwpack TELemetry) stations by visiting the Western Regional Climate Center's state-specific SNOTEL station maps.

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
Silver Spring, MD 20910
(301) 713-1669
Questions?: HDSC.Questions@noaa.gov

Disclaimer

RATIONAL FORMULA METHOD
RUNOFF COEFFICIENTS

Land Use or Surface Characteristics	Aver. % Impervious Area	Runoff Coefficients 5-Year (C_5)	Runoff Coefficients 100-Year (C_{100})
Business/Commercial:			
Downtown Areas	85	.82	.85
Neighborhood Areas	70	.65	.80
Residential: (Average Lot Size)			
⅛ Acre or Less (Multi-Unit)	65	.60	.78
¼ Acre	38	.50	.65
⅓ Acre	30	.45	.60
½ Acre	25	.40	.55
1 Acre	20	.35	.50
Industrial:	72	.68	.82
Open Space: (Lawns, Parks, Golf Courses)	5	.05	.30
Undeveloped Areas:			
Range	0	.20	.50
Forest	0	.05	.30
Streets/Roads:			
Paved	100	.88	.93
Gravel	20	.25	.50
Drives/Walks:	95	.87	.90
Roofs:	90	.85	.87

Notes:

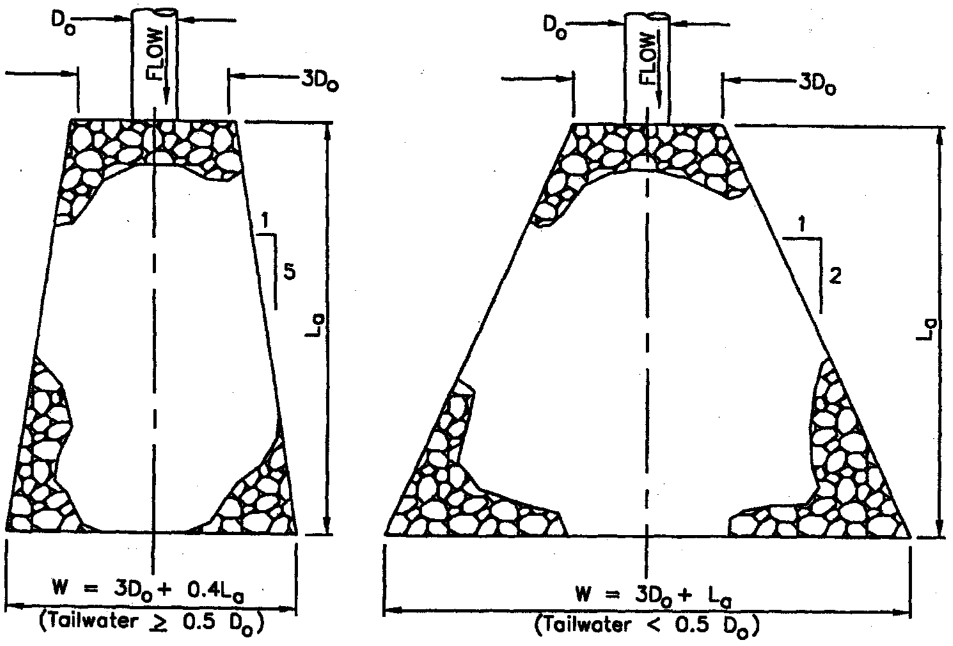
1. Composite runoff coefficients shown for Residential, Industrial, and Business/Commercial Areas assume irrigated grass landscaping for all previous areas. For development with landscaping other than irrigated grass, the designer must develop project specific composite runoff coefficients from the surface characteristics presented in this table.

VERSION: December 2, 1996	REFERENCE: USDCM, DROCOG, 1969 (with modifications)	TABLE 701
WRC ENGINEERING, INC		

WASHOE COUNTY

HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

CONFIGURATION OF CULVERT OUTLET PROTECTION

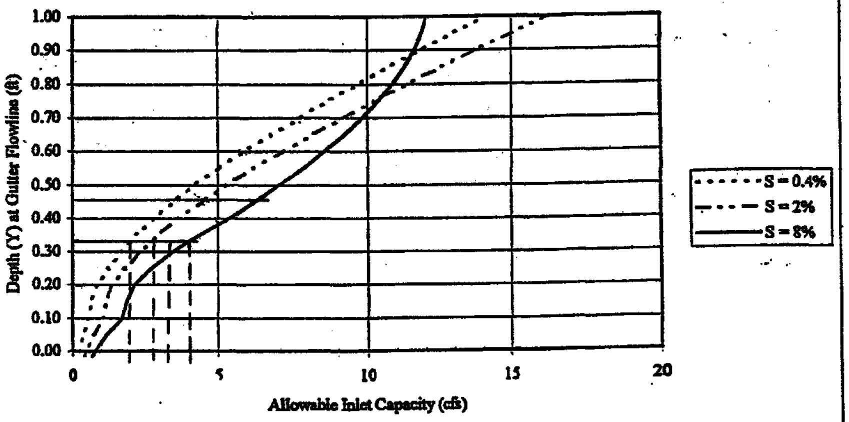


VERSION: December 2, 1996 WRC ENGINEERING, INC	**REFERENCE:** U.S. EPA, 1976	**FIGURE 821**

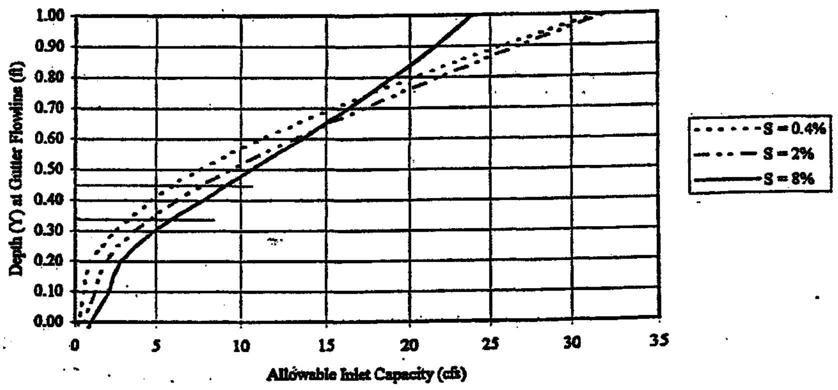
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

ALLOWABLE INLET CAPACITY
CONTINUOUS GRADE CONDITION
CATCH BASIN TYPE 4-R

TOTAL FOR CURB AND GRATE OPENING (L = 33")



TOTAL FOR CURB AND GRATE OPENING (L = 33" x 2)



NOTE: 1) Includes a capacity reduction factor of 0.70 for grate and 0.80 for curb opening

VERSION: 00-00-0000 WRC ENGINEERING, INC	REFERENCE:	FIGURE 906 1 OF 2

F I L E D
Electronically
2015-02-13 04:49:23 PM
Jacqueline Bryant
Clerk of the Court
Transaction # 4818450 : melwood

Exhibit 16

Exhibit 16

TECHNICAL DRAINAGE REPORT

The Estates at Mt. Rose Unit 3B
A Single Family Home
Residential Community

Prepared for:

Reynen and Bardis
1380 Greg Street, ste 230
Sparks, Nevada 89431
(775) 355-0507

Prepared by:

Wood Rodgers Inc.
575 Double Eagle Ct
Reno, NV 89521
(775) 823-4068

August 23, 2007

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

August 7, 2007

Kris Klein, P.E.

Washoe County Engineering
1001 E 9th Street
Reno, NV 89520

Kris Klein,

Wood Rodgers has revised the pre- and post-condition hydrologic models for the Estates at Mount Rose, Unit 3B in response to your comments. Except for a few minor wording changes the sections of the Technical Drainage Report addressing on-site flow conveyance (ditches, catch basins, and pipes) were not revised. Wood Rodgers made every attempt to address each of your concerns and comments in the revised Technical Drainage Report as well as within this letter.

To account for upstream flows, subbasin parameters for all basins upstream of the Estates at Mount Rose Subdivision were obtained from the Legend Trail Hydrology Master Plan from Nimbus Engineering, May 2004. Wood Rodgers developed a HEC-HMS model of pre- and post-conditions for on-site and off-site flows. The following discussion should address each of your comments and concerns.

The comments are based on a review of the Technical Drainage Report for The Estates at Mt. Rose Unit 3B, prepared by Wood Rodgers, Inc., dated 12/22/06.

1. ***Report page 5, last paragraph. Explain why a dummy basin is necessary in your model. Couldn't you just combine hydrographs at this location without the creation of a dummy basin?***

Wood Rodgers has revised the pre- and post-conditions hydrologic models. HEC-HMS was utilized as outlined in the revised report. In the revised models, the dummy basin was removed and was replaced by a concentration point.

2. ***Report page 6, last table. Is this table for Pond 1? Clarify in text.***

The table was edited and a discussion about the Ponds has been added to the report.

3. ***Report page 8, Conclusions. Is the increase for the 5-year storm significant? Why or why not? Provide mitigation if the 5-year increase is significant.***

A slight increase in peak flows leaving the site and at the downstream concentration point (C2 and C1 respectively) occurs during the 5-year event. The 5-year increase in peak flow is minimal (8.6 cfs (0.70%) at C2 and 5.27 cfs (0.44%) at C1) and the water surface elevation on Whites Creek is raised by no more than 0.01 ft, which is easily contained entirely within the existing Whites Creek channel. An increase in peak flows of 10.6 cfs (0.2%) at C2 is expected for the 100-year event. However, a 14.6 cfs reduction in peak

Rec 8/23/07

flows occurs at C1. The increased flows at C2 result in a rise of the water surface elevation within Whites Creek of no more than 0.01 ft. The increased flows can be easily contained within the existing Whites Creek channel. The anticipated increases in peak flows of less than 1% in Whites Creek are in compliance with those outlined in the approved Flood Control Master Plan for Mt. Rose Estates by Nimbus Engineers.

4. ***Report page 8, Conclusions. Better explain what is meant by "…the peaks of the outflow of the detention basin and that of Whites Creek will not be the same."***

Through the capture of flows into the two detention ponds, the peak flows exiting the ponds are delayed compared to that of Whites Creek. The peak flow on Whites Creek occurs prior to the release of peak flows from the detention basins. Language has been clarified in the report.

5. ***Appendix, HEC-1 Analysis, Pre-Basin and Post-Basin Quad Maps. How were the existing culverts under Mt. Rose Highway addressed? These culverts divert flow to the south and away from convergence point 1C. Is your existing flow at convergence point 1C larger than it should be if the culverts under Mt. Rose were considered? Also, the pre-development flow seems to have been reduced by about 1/3 from the previous report. What caused this flow reduction?***

Wood Rodgers revised the hydrologic modeling and addressed the concern of flow diversion from existing culverts under Mount Rose. The existing culverts were addressed in the pre-conditions hydrologic model by diverting a proportion (10%) of the flows out of the model from subbasin NDOT. In post-conditions, a ditch has been constructed with 100-year capacity diverting all flows away from the culverts under Mount Rose. The pre-condition NDOT subbasin has been reconfigured into subbasin B2 in the post-conditions model. All flows from subbasin B2 will be conveyed to Pond 2.

6. ***Appendix, HEC-1 Analysis, Time of Concentration Calculations. What method was used to calculate the channelized flow portion of the time of concentration? Where did the velocities come from for the channelized flows? Based on your drainage basin maps the length of channelized flow seems long for pre-development basins North and NDOT and post-development basin Whites. Also the length of channelized flow for post-development basin B1 seems short. We need to discuss how your flow lengths were generated.***

Lag times have been recalculated to more accurately represent what is occurring on-site in both pre- and post-conditions. To approximate travel time velocities for concentrated flow, Figure 701 in the Washoe County Hydrologic Criteria and Drainage Design Manual was applied. For the Whites subbasin, velocities of 17 ft/s and 24 ft/s were applied in 5-year and 100-year models, respectively for both pre- and post-conditions. These velocities were determined from FlowMaster by applying cross-sectional information and flows from the concentration point located at the upstream boundary of the Whites subbasin.

Flow lengths were re-calculated in the revised pre- and post-conditions models. Auto-CAD was utilized with the existing topography and the proposed grading to develop the lengths of initial overland, natural channels, and urbanized channel flows.

7. ***Appendix, HEC-1 Analysis, Pond Outlets. The Pond 2 outlet shown in the report does not match the approved Unit 3A plans: why?***

The County has a revised plan for Unit 3A showing the final design for the outlet structure of Pond 2. However, through the finalization of the HEC-HMS modeling it was determined that slight modifications to the outlet structure of Pond 2 will be required during construction of Pond 1. The modification consists of adding a 8" orifice plate to the 24" inlet pipe and leave the size of the existing 8" orifice as such on the 15" inlet pipe.

8. ***Appendix, HEC-1 Analysis, HEC-1 5-yr Pre. Color routing diagram compared to routing diagram generated by HEC-1. Why aren't these diagrams identical? Both 3C and 3R show up in one diagram but not the other.***

The HEC-1 routing diagram and the Color routing diagram from the December 2006 report were removed. HEC-HMS basin diagrams for the pre- and post-conditions models have been included in the revised report to demonstrate basin routing.

9. ***Appendix, HEC-1 Analysis, HEC-1 5-yr Pre input. Explain the purpose of input lines such as "KK 3R CNAME 3C". This example appears in line 13 of the pre-development 5-year model, but similar lines are contained in all HEC runs.***

The model has been changed from a HEC-1 to HEC-HMS format for ease of use. Therefore, all HEC-1 annotation has been removed.

10. ***Appendix, HEC-1 Analysis, HEC-1 5-yr Post, Color and HEC-1 generated routing diagrams. Based on your drainage basin maps, WHITES routes to concentration point 2C, not 4C. Explain?***

Wood Rodgers has revised the flow routing in the pre- and post-conditions models. The Whites subbasin routes directly to concentration point 2C. Concentration point 4C has been removed from the models.

11. ***Runoff routing. The routing used for both the pre- and post-development flows is still not clear. Explain why the routing to point 1C is accurate, since you have not included all overland flows that arrive at this point. Flow does not exit your site at one point. What are the increases and decreases in flow exiting your site at all points, and is mitigation required?***

The pre- and post-conditions models have been revised to incorporate all areas contributing overland flows to concentration point 1C. The revisions have included the addition of flows from upstream drainage areas on Whites Creek and subbasin W8R.

In pre-conditions a portion of the off-site and on-site flows travel though the project area and concentrate downstream of the property boundary at concentration point 1C. Under post-conditions, all off-site and on-site flows are collected and conveyed to Whites Creek prior to the downstream property boundary. Peak flows at 2C are slightly increased in the post-condition design events. Cross sections of the existing Whites Creek geometry were modeled in FlowMaster to determine the increase in water surface elevations resulting from increased flows immediately downstream of concentration point 2C. The 5- and 100-year peak flow normal depth calculations in FlowMaster demonstrate that increases in peak flows do not raise the water surface elevation more than 0.01 ft under flood conditions. Therefore, the slight flow increases do not overwhelm the existing configuration of the Whites Creek channel and flood hazard to surrounding properties is not increased in post-conditions.

If you have any questions please call me at 823-4068.
Sincerely:

Mary C. Horvath. P.E., CFM
Wood Rodgers, Inc.

TECHNICAL DRAINAGE REPORT

The Estates at Mt. Rose Unit 3B
A Single Family Home
Residential Community

Prepared for:

Reynen and Bardis
1380 Greg Street, ste 230
Sparks, Nevada 89431
(775) 355-0507

Prepared by:

Wood Rodgers Inc.
575 Double Eagle Ct
Reno, NV 89521
(775) 823-4068

August 23, 2007

GARY M. PROBERT
Exp. 12-31-07
CIVIL
No. 7721

8-23-07

MARY C. HORVATH
CIVIL

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

Rec 8/23/07

TABLE OF CONTENTS

Introduction

This report presents the drainage design for The Estates at Mount Rose Subdivision Unit 3B. The project site, Unit 3B, encompasses 82 acres and is bounded to the south by Whites Creek and Unit 3A. To the north, the site is bounded by Mountain Ranch Road and to the west by Legend Trail. Access to the site is off of Mount Rose Highway at Callahan Road. Callahan Road runs north-south through the project and connects Mount Rose Highway and Mountain Ranch Road. The project site is contained in Section 35, Township 18 North, Range 19 East, M.D.M., in Washoe County, Nevada. A vicinity map has been included in the Appendix.

The proposed development consists of 23 single-family residential lots with an average size of approximately 55,000 sq. ft each. This is the final development phase of The Estates at Mount Rose Unit 3. The total combined area for Estates at Mt. Rose Units 3A & 3B consists of 228 acres. The purpose of this report is to demonstrate that the drainage plan for The Estates at Mount Rose Unit 3B/3A complies with the criteria set forth in the Washoe County Hydrologic Criteria and Drainage Design Manual (Manual).

Previous Studies

Black Eagle Consulting conducted a geotechnical investication in June of 2003. The findings of this investigation are included in the report entitled, "Geotechnical Investigation The Estates at Mount Rose Callahan Ranch Road and Mount Rose Highway Washoe County, Nevada."

Wood Rodgers prepared a Technical Drainage Report for "The Estates at Mt Rose Unit 3A" dated December 29, 2005, which was approved by Washoe County.

Flood Hazard Areas

The proposed project is found mostly within Flood Zone X on the FEMA Flood Insurance Rate Maps (FIRM) 32031C3165E and 32031C3170E, dated September 30, 1994. Flood Zone X indicates areas that are outside of the 500-year floodplain. Two FIRMettes covering the project area are included in the Appendix of this report. A small portion of the project area lies within Flood Zone A. No structures are proposed to be located within the FEMA Flood Zone A. Map H1 located in the Appendix of the report shows the proximity of Flood Zone A to the Unit 3B project.

Historic Drainage System

The historic drainage for the project area consists of overland sheet flow traveling in an easterly and southerly direction until it is intercepted by Whites Creek, a small perennial stream flowing through the project site. As indicated in the report, Preliminary Whites Creek Basin Management Study, by Cella Barr Associates, Whites Creek has a peak flow of roughly 5,100 cfs at the diffluence of Whites Creek downstream of the Estates at Mount Rose Subdivision project site. Flow routing of the historic drainage is shown on the Pre-Conditions Basin maps within the Appendix.

Within the subbasin NDOT, as identified on the Pre-conditions maps in the Appendix, flows in historic drainage conditions are conveyed along a small, natural drainage channel. This drainage channel flows perpendicular to a set of two culverts which convey a portion of flows under the Mount Rose Highway. The remaining flows are carried downstream where they are intercepted by Whites Creek.

Proposed Drainage System

The on-site proposed drainage system consists of overland flow in drainage swales to the curb and gutter system or directly to the pipe system through a culvert pipe stubbed into a common area or open space. Flows within the curb and gutter flow to catch basins and then into the storm drain system. The pipes discharge either into an open space/common area and then to Whites Creek via a ditch or into one of two detention ponds which release the water into Whites Creek. Additional offsite flows originating from the north and northwest are collected via ditches. Flows west of lots 363 and 378 flow directly to Whites Creek via a ditch and bypass the detention ponds. Flows picked up and/or generated east of lots 363 and 378 flow to the detention basin where storm drainage is detained and released to Whites Creek in a controlled fashion. Post-conditions flow routing is shown on the Post-Conditions Basin maps included in the Appendix.

The drainage basin plans are included in the Appendix (see Map Pocket 1). The drainage plans show each storm drain line, the corresponding inlets, the areas associated with each inlet, ditches, and off-site areas. Overland flow equations were used to calculate the initial travel time, and Manning's Equation was used to calculate the channelized flow (see the Modified Form 2 in the 5-year and 100-year sections of the appendices).

The proposed storm drain system is shown as a tree structure with element annotation on 24"x 36" sheets for both the 5-year and 100-year storms (see Map Pocket 2 and 3 respectively). Pertinent output for each element is shown on these sheets. Additionally, storm drain profiles are included to show the hydraulic grade lines for the pipe systems in both the 5-year and 100-year storms (see Map Pocket 2 and 3 respectively). Washoe County requires that the hydraulic grade lines be greater than one foot (1') below rim elevations for each manhole for the design storm. The Hydraulic grade line of each catch basin can be viewed through the storm drain profiles and in the Junction Report included in both the 5-year and 100-year appendices of this report. In no case does the hydraulic grade line encroach within the one foot (1') separation requirement.

Sediment transport and erosion will be controlled through sizing of outlet and inlet protection, slope stabilization with riprap and vegetation, and through conformance with the Storm Water Pollution Prevention Plan (SWPPP) that has been prepared for this site. The SWPPP includes Best Management Practices (BMPs), a maintenance schedule, and a list of the responsible parties for maintenance to ensure the storm drain system operates correctly to prevent excessive sediment transport. Calculations for outlet protection and erosion control using riprap are included in the Appendix of this report. A check was performed and some rip rap sizes were adjusted from that indicated in the previous version of this report for those channels located on steep slopes.

Hydrologic and Hydraulic Methods

On-site Drainage Facility Analysis

The Rational Method (Q=CIA) was used for computing on-site and off-site flows contributing to the storm drainage system as outlined in section 704 of the Manual. The rainfall intensities were obtained from the NOAA Atlas 14 data.

Ditches were sized using the rational method flows and using a custom Wood Rodgers Manning's equation spreadsheet that calculates required rip rap sizing as well. The rip rap sizing algorithms match those methods in the Manual (Section 800). Rip rap sizing was also checked for steep slopes. Per Washoe

County, and steep slope calculation is to be used when the D50 size is larger than the depth of flow. In this case we used calculations from Figure 813 (see Appendix) from the Manual.

On-site Design

Hydrologic parameters for the analysis were determined as shown in section 704 of the Manual. The Rational Equation requires an area, intensity, and runoff coefficient to determine the flow at each inlet location. The time of concentration was calculated for each subbasin using a modified version of the Standard Form 2. The intensity for each subbasin was derived from linear interpolation using the calculated time of concentration and the listed values from NOAA Atlas 14. The runoff coefficients were derived from Table 701 (see Appendix) of the Manual for 1-acre lots (~43,560sq. ft.), for open space, and undeveloped rangeland based on the storm event. Consideration was given towards using runoff coefficients listed in the TR-20 manual as there are additional choices. Those runoff coefficients listed in the Manual were ultimately selected as they were in all cases more conservative than those in the TR-20 manual. Areas for each subbasin were determined from creating polylines in AutoCAD along subbasin boundaries and listing the polyline properties to obtain the area.

Hydraulic computations were performed to analyze the storm drain system to ensure compliance with Section 900 of the Manual. Section 902.1 requires the HGL to be one foot below the final grade above the storm drain at all locations for the design storm. The design storm is typically the minor event unless the design of the storm drain system meets one of the conditions noted in Section 901 of the Manual. StormCAD Version 5.0 was utilized to model the pipe network including: energy losses due to pipe friction, and at junctions, inlets, and outlets and to draw the hydraulic grade line. StormCAD also provides flow rates and velocities at outlets so outlet protection can be appropriately sized based on Section 807.3 of the Manual.

Each catch basin's contribution to the storm drain system was determined from inputting parameters (taken from Figures 907 and 906 of the Manual) into StormCAD where orifice or weir equations are used based on sump or on grade conditions to solve for capacity and performance of the inlet. Figures 907 and 906 are located in the Appendix.

Maximum catch basin capacities were also determined from Figures 906 and 907 of the Manual. Street slopes and depth of flow (taken from street spread calculations using FlowMaster) were used to determine from the figures what their maximum capacities were.

Section 304.4 of the Manual stipulates the requirements for inundation of streets. For the minor storm event a local street must maintain a 12-foot centered dry travel lane, and the velocity must be less than 6 fps. For a major storm the street may be flooded and the velocity must be less than 6 fps. To analyze the street capacity a depth of flow and velocity of flow was determined by using FlowMaster. In FlowMaster a custom half street section were created. Slopes, coefficients, and flows were inputted to determine velocities and depths of flows at each catch basin. This was done for both the 5- and 100-year storm events and included in the respective locations of this report. All street spreads are within allowable tolerances.

HEC-HMS Watershed Modeling

Pre- and post-conditions hydrology were modeled with HEC-HMS (Version 3.1.0) to determine peak flows both on-site and at the downstream project boundary for the 5-year and 100-year design events. This analysis was used to size the detention basins for the purpose of mitigating increases in peak flows.

HEC-HMS Model Parameters

The input parameters from the Nimbus Engineers, May 2004, Legend Trail Hydrology Master Plan HEC-1 model (see Appendix) were applied to represent the watersheds upstream of the Estates at Mount Rose project boundary. The Nimbus Engineers model has been approved by Washoe County and computes a peak 100-year flow on Whites Creek that is comparable to that identified in the report titled "Preliminary Whites Creek Basin Management Study" prepared by Cella Barr in 1994. Subbasin Area, Runoff curve numbers, lag times and precipitation depths for all upstream off-site subbasins were taken from the Nimbus model.

The rainfall run-off methods used for the HEC-HMS analysis for the pre- and post-condition on-site subbasins are as follows:

- The Soil Conservation Service (SCS) method was used to compute rainfall loss. To determine curve numbers for each pre- and post-condition subbasin, the Soil Survey of Washoe County, Nevada, South Part was overlain onto the pre- and post-conditions subbasins. A vegetation cover type of Shrub/brush was used for the undeveloped areas in the pre- and post-condition models and an average lot size of 1 acre was used for the post-condition developed areas. Weighted curve numbers were then determined based upon hydrologic soil group and cover type according to Table 702 in the Manual.

- The SCS dimensionless unit hydrograph method was used for translation of excess precipitation into runoff. Time of concentration for the pre- and post-condition subbasins was calculated through application of methods outlined in the Manual. Velocities were determined from Figure 701. The lag time, TLAG, was calculated by multiplying the time of concentration by a factor of 0.6. Refer to the Appendix for time of concentration calculations.

- The Muskingum routing method was applied to route the flows in the pre- and post-conditions HEC-HMS models. The Muskingum method was applied to be consistent with the methodology used in Legend Trail Hydrology Master Plan. In addition, the Muskingum method is frequently used where stream gauge data is sporadic, channel dimensions are inconsistent and flood flows are maintained within the stream bank as they are in Whites Creek.

- Precipitation depths for off-site basins were obtained from the Nimbus Report for the 5-year and 100-year, 24-hour events, which utilized the NOAA Atlas 14 precipitation depths for the 5-year and 100-year, 24 hour events. The precipitation depths for on-site basins were obtained from NOAA Atlas 14 for the 5-year and 100-year, 24 hour events. The precipitation distribution from the Nimbus report was applied for both the 5- and 100-year events in both pre- and post-conditions. A depth-area reduction factor of 0.98 was applied to the precipitation to account for the overall size of the watershed.

HEC-HMS Model Routing

Model routing schemes for both the pre- and post-conditions are shown in the HEC-HMS models within the Appendix.

Subbasins upstream of the HEC-HMS junctions WCOUT and WCOUT2 represent the Whites Creek watershed upstream of the Estates property boundary and are identical in both the pre- and post-condition models. In the pre-conditions model, on-site subbasins Whites, North and off-site LT7 are routed and combined with upstream flows at concentration point 2C. Flows from subbasin Mid are routed to a junction with flows from the NDOT subbasin. A portion of flows from subbasins NDOT, off-site LT8 and off-site LT5a1 are diverted by the culverts under the Mt. Rose highway. Due to the alignment of the existing culverts (90 degrees to the direction of flow), 10% of the total flows from the NDOT, LT8 and LT5a1 subbasins were estimated to be accepted into the culverts and therefore diverted out of the model. The flows from subbasins Mid, NDOT, LT8, and LT5a1 are combined with flows from W8R (an off-site subbasin to the north of Whites Creek) and concentration point 2C at concentration point 1C, located approximately 2000 feet downstream of the project boundary.

In the post-conditions model, subbasin B1 is routed to Pond 1. Pond 2 collects and mitigates flows from subbasins B2, LT8 and LT5a1. At concentration point 2C, flows from Pond 1, Pond 2, LT7 and the Whites subbasin are combined. Identical to the pre-conditions model, concentration point 1C accounts for all upstream flows including subbasin W8R and is located downstream of the project boundary.

Detention Ponds

On-site and a portion of off-site peak flows in post-conditions are attenuated through the development of two detention ponds. Pond 2 was designed, approved, and constructed in conjunction with project phase Unit 3A. Pond 1 will be constructed during project phase Unit 3B. The ponds were sized to mitigate peak flows within Whites Creek at the downstream project boundary to be equal to or lesser than the pre-conditions peak flow rates for both the 5-year and 100-year storms.

The details for the detention ponds are shown in the Pond Outlets Figure within the Appendix. A standard headwall with a trash rack will serve as the entrance for a 15" pipe fitted with a 6" orifice plate (invert elevation 5568.00 ft) to control flows out of Pond 1. A 48" diameter standpipe structure will be constructed with a 24" outlet pipe fitted with a 6" orifice plate located roughly 6' above the top of the 15" pipe (at an invert elevation 5574.00 ft). A grate is located on the top of the 48" standpipe at an elevation of 5576.20 ft. This elevation is 0.13' above the 100-year design water surface elevation of 5576.07 ft. A 24" pipe drains the 48" standpipe. The 24" outlet pipe has adequate capacity for the 100-year peak flow exiting Pond 1. There are two emergency outlets to Pond 1. The grated 48" standpipe lid is set 0.13 above the 100-year water surface elevation. An emergency overflow weir with an invert elevation 0.90' above the 100-year water surface elevation and 1.00' below the top of bank has been designed for the pond. The two emergency structures should be capable of handling storms greater than the 100-year event and/or plugging of the inlets during storm conditions. The emergency overflow weir discharges directly to Whites Creek on the southeast side of Pond 1 via a riprap channel. The capacity over the emergency weir alone is greater than that of the major storm event entering Pond 1.

Pond 2 has a similar outlet structure to that of Pond 1. Details on the structure and design of Pond 2 are shown in the Pond Outlets Figure in the Appendix. A slight modification to the existing outlet structure of Pond 2 will be required in conjunction with the construction of Pond 1. The modification consists of adding a 8" orifice plate to the 24" inlet pipe and adding a 8" orifice plate to the 15" inlet pipe.

The ponds were included in the 5-year and 100-year post condition HEC-HMS models. HEC-HMS used the orifice and weir equations for the discharge of each outlet structure, corresponding to the computed elevation in the pond for each time step. HEC-HMS generates the final peak flows and times for the overall post-development conditions of the project.

Summary of Flows

The pre- and post-condition peak flows for the 5- and 100-year storm events for this project are tabulated below. Points of interest are shown in bold text. Refer to Figures in the Appendix.

Table 1. - Summary of Peak Flows

Subbasin	5-year		100-year	
	Flow (cfs)	Time (hrs)	Flow (cfs)	Time (hrs)
Pre-Conditions				
WCIN1	1161.1	13:39	5347.1	13:27
WCOUT*	1176.4	13:39	5415.1	13:27
WCOUT2*	14.6	12:03	74.9	12:33
North	1.8	12:48	15.8	12:36
Whites**	8.5	12:12	99.4	12:06
2C	**1189.8**	13:42	**5482.4**	13:30
NDOT	2.8	12:54	40.4	12:36
Mid	0.5	13:06	16.7	12:24
W8R*	1.1	13:21	20.0	12:39
1C	**1194.23**	13:42	**5516.6**	13:30
Post-Conditions				
WCIN1	1164.3	13:36	5347.1	13:27
WCOUT*	1180.2	13:36	5415.1	13:27
WCOUT2*	14.6	12:03	74.9	12:33
Whites**	23.2	12:12	155.6	12:09
B1	7.7	12:15	59.1	12:12
B2	5.8	12:12	85.4	12:09
2C	**1198.4**	13:39	**5493.0**	13:30
W8R*	1.1	13:21	20.0	12:39
1C	**1199.5**	13:39	**5502.0**	13:30

*Off-site tributary subbasins of a consistent size.

**On-site subbasins with an increase in contributing area in post-conditions.

Table 2. - Summary of Ponds 1 and 2

Flow Summary	Pond 1	Pond 2
Peak flow in (5-year)	7.7 cfs	12.4 cfs
Peak flow in (100-year)	59.1 cfs	89.8 cfs
Peak flow out (5-year)	1.2 cfs	2.4 cfs
Peak flow out (100-year)	4.0 cfs	10.6 cfs
5-year surface elevation	5569.79 ft	5466.32 ft
100-year surface elevation	5576.07 ft	5471.0 ft
Peak 5-year storage	0.5 ac-ft	0.6 ac-ft
Peak 100-year storage	3.36 ac-ft	5.56 ac-ft

Table 3. – Summary of Detention Basin Elevation, Storage, and Discharge

Pond 1				Pond 2			
Elevation (ft)	Area (acres)	Storage (ac-ft)	Discharge (cfs)	Elevation (ft)	Area (acres)	Storage (ac-ft)	Discharge (cfs)
5568	0.21	0.0	0.0	5464	0.00	0.0	0.0
5570	0.30	0.5	1.25	5466	0.55	0.4	2.35
5572	0.40	1.2	1.82	5468	1.07	2.0	3.20
5574	0.51	2.1	2.26	5470	1.24	4.3	5.83
5576	0.63	3.3	4.0	5471	1.31	5.75	10.62

Conclusions

The proposed drainage system is designed in compliance with the Washoe County drainage policies and engineering requirements. The proposed drainage facilities will control the amount of downstream sedimentation through erosion control measures. Construction of Ponds 1 and 2 provide significant mitigation of peak flows from the 5-year and 100-year design events.

A slight increase in the peak flows for the 5-year event of 8.6 cfs at concentration point 2C is due to the redistribution of flows in the drainage basins as a result of the development of the site. The 5-year increase of 0.70% in peak flows is not significant enough to raise the water surface elevation in Whites Creek more then a hundredth of a foot (0.01 ft). Cross-sections have been provided which show both the pre- and post-conditions water surface elevations downstream of concentration point 2. Additional flows from the project site and from off-site areas are directed to the detention basin (Pond 2) located in Phase 3A. Historically, some of these flows were directed toward the ditch along the NDOT Right-of-way and a portion went under the Mount Rose Highway. The increase in flows at the location where Whites Creek leaves the site (concentration point 2C) is 8.6 cfs or (0.7%) in the 5-year event and becomes a 5.27 cfs (0.44%) increase 2000 ft downstream at concentration point 1C. Flows for the more critical 100-year event are reduced downstream from the site as a result of the project mitigation measures.

The peak discharge in Whites Creek for the 100-year event has been reduced at concentration point (1C) located roughly 2000 feet downstream of the site by 14.6 cfs. Peak flows at concentration point (2C) at the outlet of the site are 10.6 cfs greater in post-conditions for the 100-year event. The increase in peak flows at concentration point 2C account for only 0.2% of the flows and thus can be considered negligible. In addition, the increase in peak flow results in an increase in the water surface elevation of no more than 0.01 ft (see Appendix) within Whites Creek. Due to the timing of flows leaving the site, this 10.7 cfs increase becomes a 14.6 cfs decrease as flows proceed to a point 2000 ft downstream.

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APPENDIX A

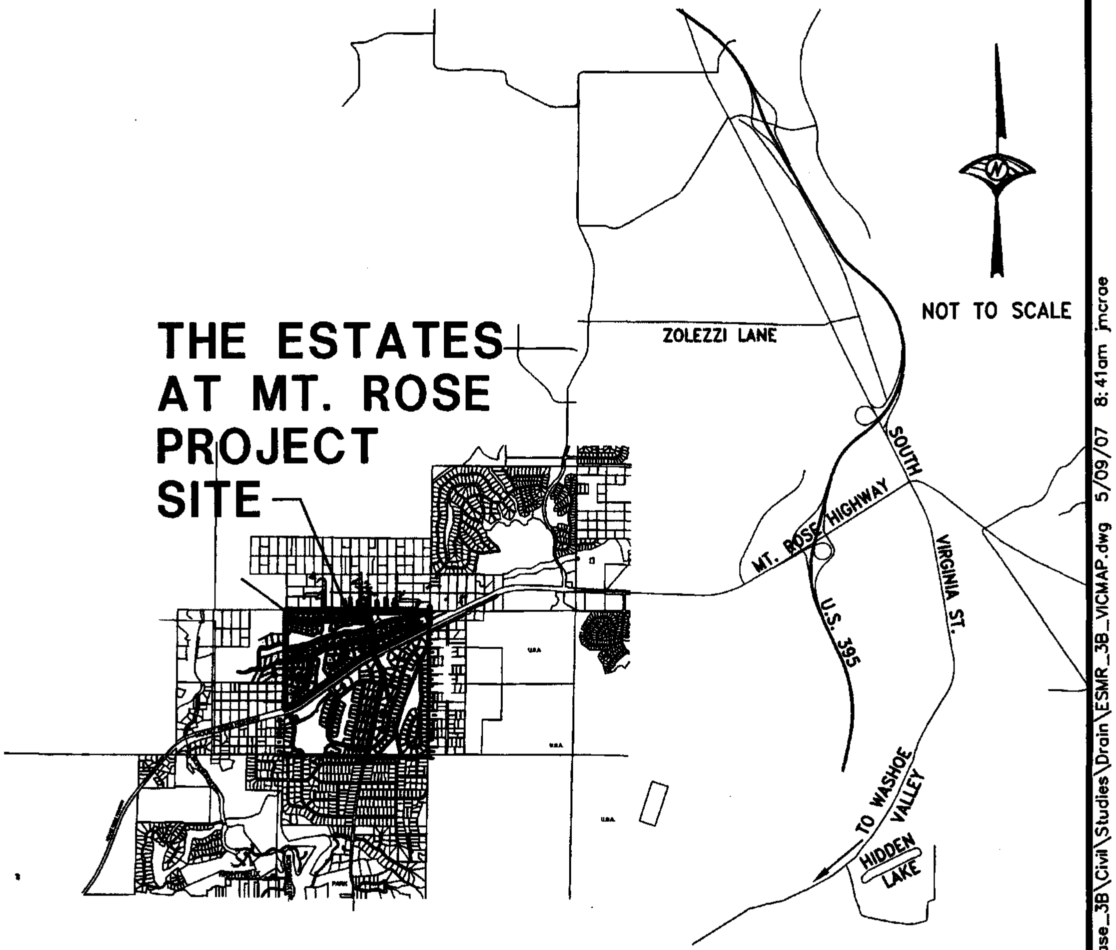
VICINITY MAP

THE ESTATES AT MOUNT ROSE

UNIT 3B

WASHOE COUNTY **NEVADA**

AUGUST 8, 2005



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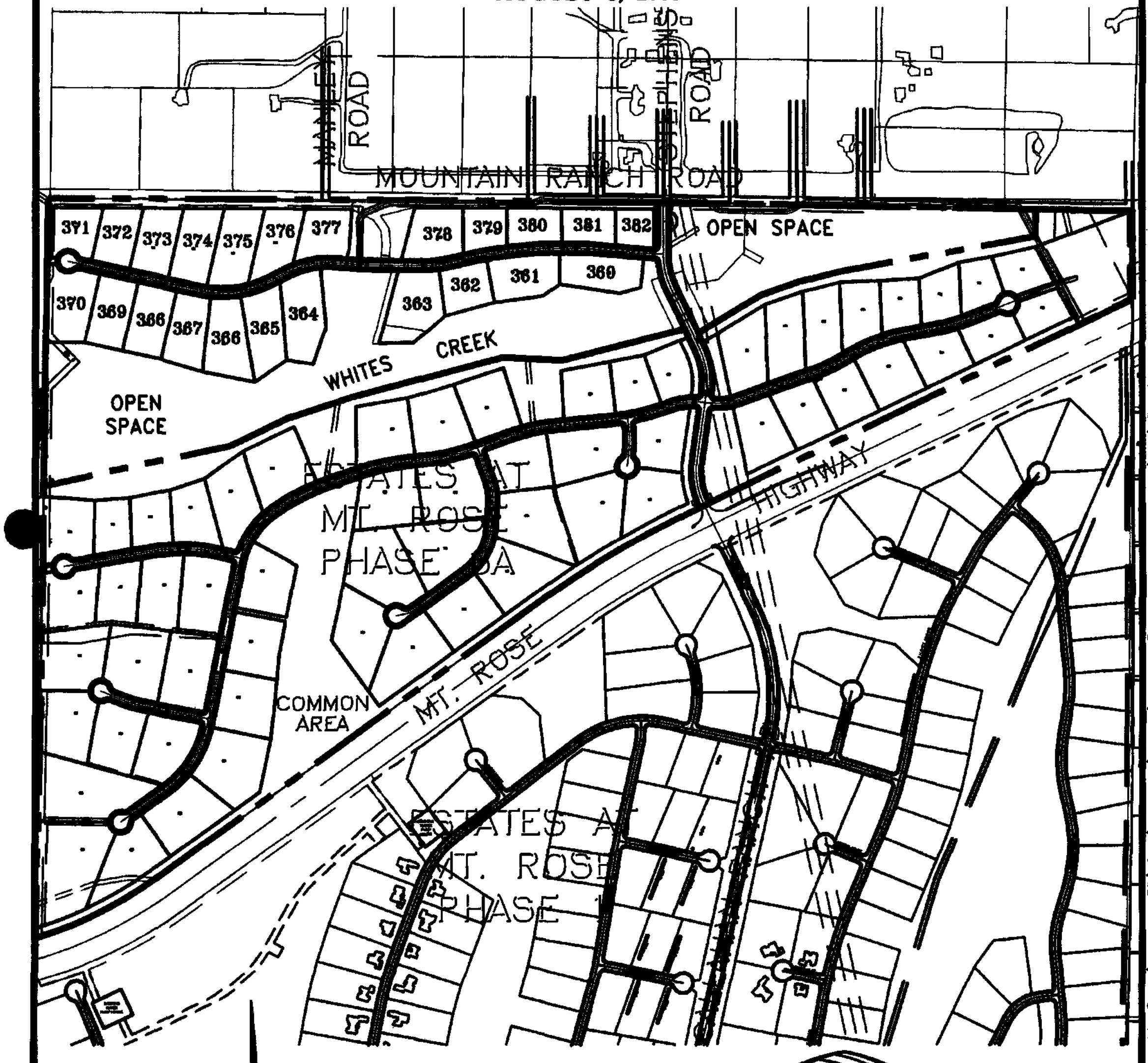
OVERALL PROJECT SITE MAP
THE ESTATES AT MOUNT ROSE
UNIT 3B

WASHOE COUNTY NEVADA

AUGUST 8, 2005



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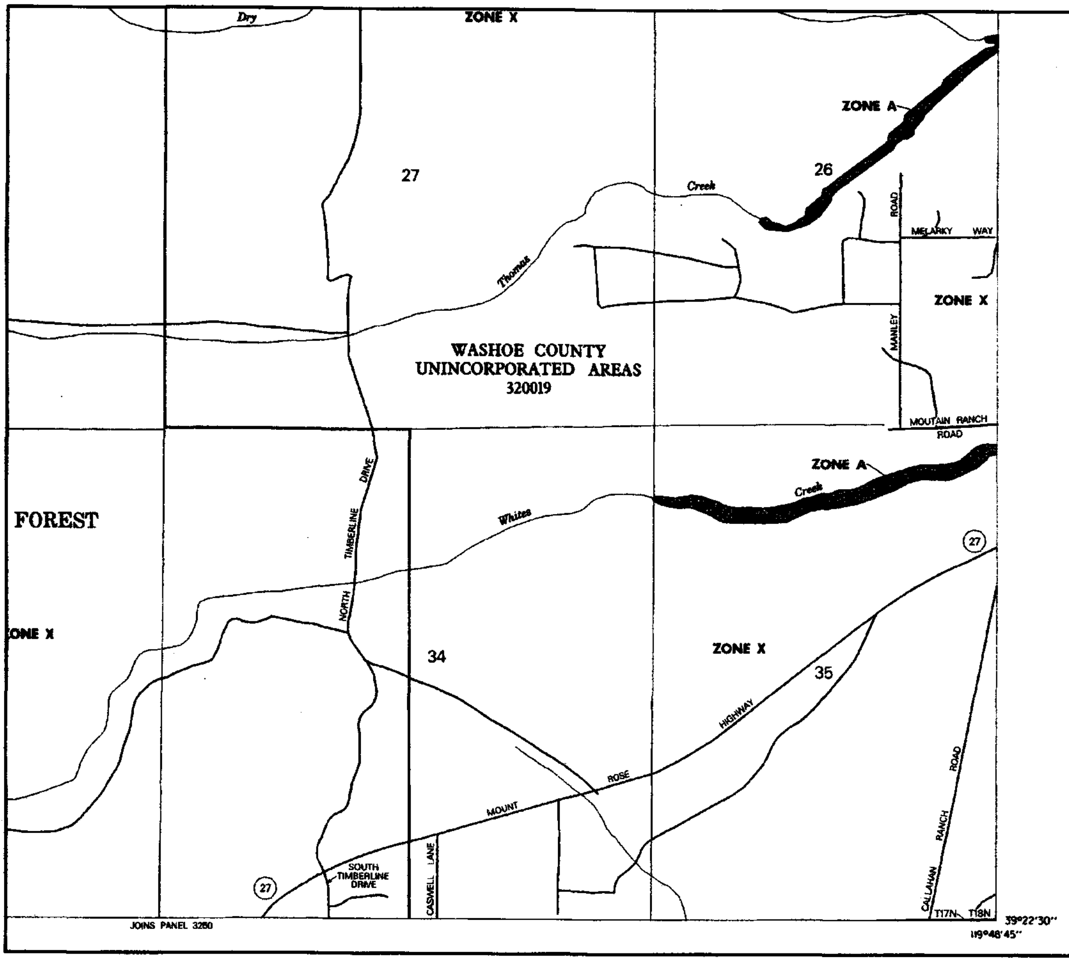

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NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP
WASHOE COUNTY, NEVADA AND INCORPORATED AREAS

PANEL 3165 OF 3350
(SEE MAP INDEX FOR PANELS NOT PRINTED)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
WASHOE COUNTY, UNINCORPORATED AREAS	320019	3165	E

MAP NUMBER
32031C3165 E

EFFECTIVE DATE:
SEPTEMBER 30, 1994

Federal Emergency Management Agency

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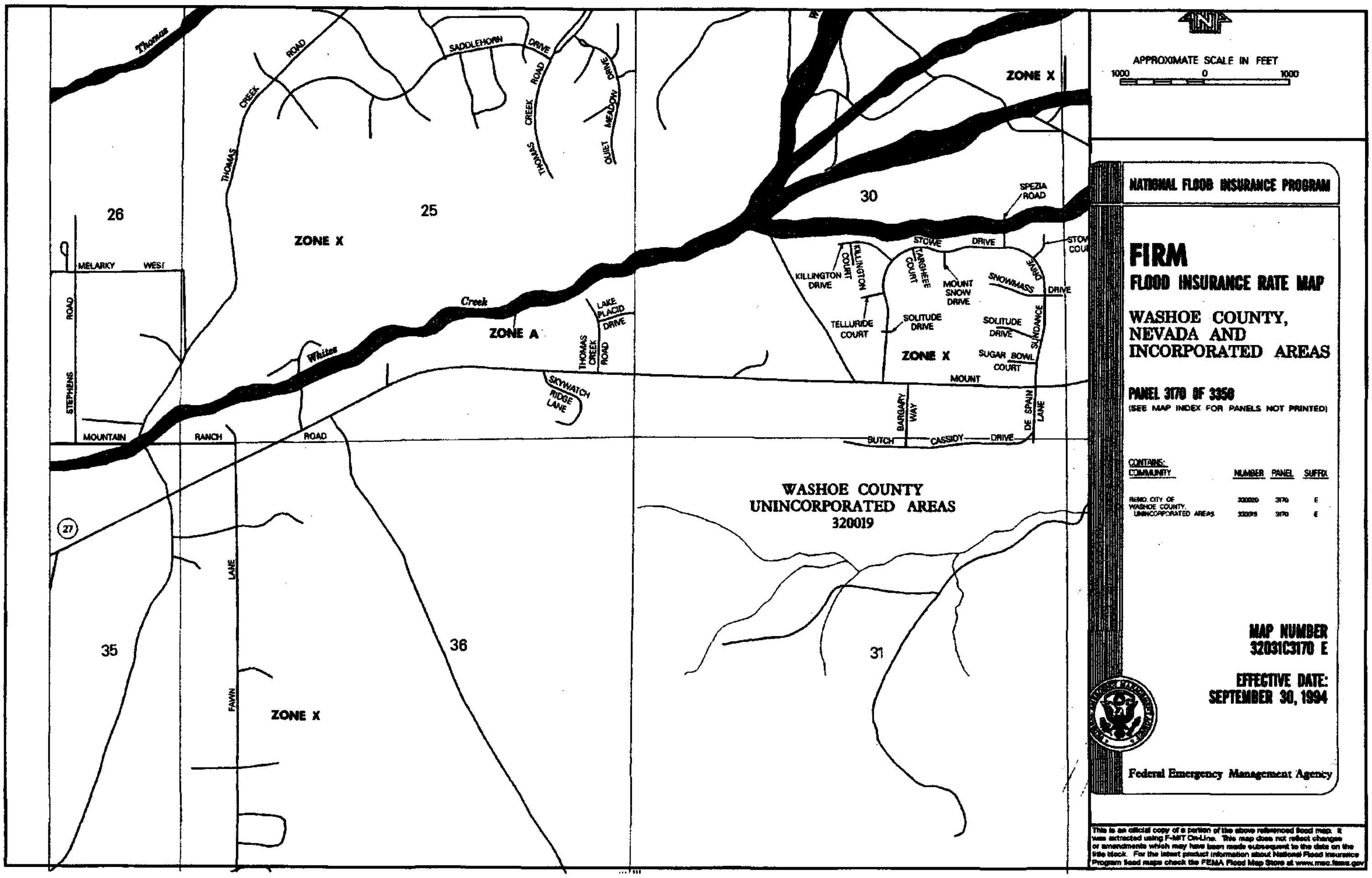

NATIONAL FLOOD INSURANCE PROGRAM

FIRM

FLOOD INSURANCE RATE MAP

WASHOE COUNTY, NEVADA AND INCORPORATED AREAS

PANEL 3170 OF 3350

(SEE MAP INDEX FOR PANELS NOT PRINTED)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
RENO, CITY OF	320020	3170	E
WASHOE COUNTY, UNINCORPORATED AREAS	320019	3170	E

MAP NUMBER
32031C3170 E

EFFECTIVE DATE:
SEPTEMBER 30, 1994

Federal Emergency Management Agency

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Exhibit 17

Exhibit 17

TECHNICAL DRAINAGE REPORT

The Reserve at Monte Rosa
Phase I
A Single Family Home Residential Community

Prepared for:

Monte Rosa LLC.
Alan Means
6121 Lakeside Dr. Suite 236

Prepared by:

Wood Rodgers Inc.
6774 S. McCarran Blvd
Reno, NV 89509
(775) 823-4068

April 24, 2005

WOOD RODGERS INC.
ENGINEERING PLANNING MAPPING SURVEYING

TECHNICAL DRAINAGE REPORT

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April 24, 2005

PROFESSIONAL ENGINEER-STATE OF NEVADA
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Introduction

This report presents the storm-water management plan for Phase I of The Reserve at Monte Rosa, a single-family home residential development. Phase I consists of 32 single- family residential lots ranging from one-half acre to over an acre in size. The site is bounded by the Saddlehorn South-Phase Five to the west, the Mt. Rose Highway (U.S. 431) to the south, White's Creek to the north, and Lancer Estates Unit Six to the east. An aerial photo of the site and the surrounding subdivisions and land features is included in the Appendix of this report. The site is contained within Section 30, Township 18 North, and Range 20 East. The approximate Latitude and Longitude is 39.39°N 119.8°W.

The topography of the site ranges from steep rocky terrain to gentle sloping areas with an elevation range from 5020 ft to 5175ft. The existing-conditions drainage map included in the Appendix shows an aerial photo of the site with contours at 10-ft intervals. Existing drainage facilities and site conditions are detailed on this map.

Jeff Codega Inc prepared a previous hydrology report for Saddlehorn South Phase Seven that is adjacent and west of the subject parcel. Basin parameters for the upstream and offsite areas were derived from this study and included in the HEC-1 model prepared for this report. A list of basin parameters is included in the Appendix of this report for both onsite and offsite areas.

The purpose of this report is to show the drainage plan conforms to *Article 420 of the Washoe County Development Code* and the *Conditions for The Reserve at Monte Rosa Tentative Subdivision Map* dated January 5th 2005. This report includes analysis for Phase I of The Reserve at Monte Rosa, and a subsequent report will be submitted with the improvement plans for Phase II of the development.

Pre-Development Drainage System

The existing drainage system consists of three basins that cover the entire property (both Phase I and Phase II) with confluence points in three locations. Basin One includes the southern-most portion of the property including runoff from Lancer Hill and a portion of the Mt Rose Highway. A roadside ditch directs runoff from Basin One to a 24" reinforced concrete pipe in the southeast corner of the site. A small area of offsite runoff enters the subject parcel via the roadside ditch along the Mt Rose Highway at the southwest corner of the property. The majority of the Phase I development is included within Basin One. Basin Two encompasses a large central portion of the site. Runoff from Basin Two consists of overland flow to a cutoff ditch on the eastern boundary of the property that directs the flow to a 24" reinforced concrete pipe in the Lancer Estates Unit 6 Subdivision. Basin Three includes the northern portion of the property that flows to Whites Creek. Offsite flows from an existing 18" RCP enter Basin Three at the northwest property boundary and are directed through the site in a drainageway before entering Whites Creek.

The pre-development drainage system was modeled using the SCS Method and HEC-1 Version 4.0.1E, and WMS Version 6.0. The parameters utilized for modeling the existing condition are included in the Appendix of this report. A composite curve number was derived for each basin from hydrologic soil groups and landuse data. The design storm was taken from the NOAA Atlas 14 Point Precipitation Frequency Estimates for the subject parcel.

Table 1.0 shows runoff computed for both the 5-yr storm event and 100-yr storm event for each of the three confluence points shown on the pre-development drainage system map.

Confluence Point	5-yr Runoff (cfs)	100-yr Runoff (cfs)
1	1.2	19.9
2	1.7	29.0
3	3.2	38.9

Table 1.0

Proposed (Developed) Drainage System

The proposed drainage system was analyzed using the SCS Method for sizing of detention ponds and to compare with the pre-development peak-flow rates at each of the three confluence points. The major basin areas were modeled with composite curve numbers that reflect the developed-condition. The detention ponds were sized to reduce the developed-peak flow rates to below the pre-development peak flow rates. One-detention basin will be constructed with Phase I of the development, and additional detention facilities will be analyzed and modeled with Phase II of the project.

The following table shows the peak-flow rate for each of the three confluence points in the developed condition, and the peak flow rate with the proposed detention facilities. Confluences 2 and 3 have been modeled with approximate pond sizes. With the Phase II submittal the HEC-1 model will be updated and revised with exact pond sizes.

Confluence Point	5-yr Runoff (cfs) Without Detention	100-yr Runoff (cfs) Without Detention	5-yr Runoff (cfs) With Detention	100-yr Runoff (cfs) With Detention
1	7.5	42.0	1.3	11.1
2	4.8	34.2	2.5	23.6
3	3.4	51.4	3.5	35.5

Table 2.0

Peak flow rates have been reduced in each of the three confluence points for the 100-yr storm event.

The detention facility to be constructed with Phase I includes 1.6ac/ft of capacity. A 24" standpipe will be constructed with a grate invert at 2' below the top of the pond. An 8" low flow orifice will be constructed as the low flow outlet. An emergency spillway will be constructed to mitigate flows greater than the 100-yr storm event.

The Rational Method was utilized to calculate peak-flow rates for sub-basin areas contributing to the on-site storm drainage system. The following table includes a list of the basin areas, intensities, and runoff coefficients for each basin. A minimum time of concentration of ten minutes was used for each sub-basin.

Catch Basin ID	Area (ac)	Runoff C 100yr, 5yr	Intensity (in/hr) 100yr, 5yr	5-yr Runoff (cfs)	100-yr Runoff (cfs
1	0.14	0.85, 0.85	3.72, 1.52	0.18	0.44
2	4.2	0.5, 0.35	3.72, 1.52	2.23	7.81
3	2.0	0.5, 0.35	3.72, 1.52	1.06	3.72
4	2.9	0.5, 0.35	3.72, 1.52	1.54	5.39
5	1.5	0.5, 0.35	3.72, 1.52	0.80	2.79
6A	3.0	0.5, 0.35	3.72, 1.52	1.60	5.58
6B	2.25	0.5, 0.35	3.72, 1.52	1.20	4.19
7	0.76	0.85, 0.85	3.72, 1.52	0.98	2.40

Table 3.0

Street capacity calculations were completed for each of the roadways in the Phase I development. Rating tables from Flowmaster version 7.0 were prepared for a range of street slopes at the allowed depth of flow for the 5-yr and 100-yr storm events. The following table shows the flow in the street and street capacity for each section of roadway with different grades for both Boulder Patch Dr. and Nature Trail Dr.

STREET NAME	FROM STA	TO STA	GRADE %	[illegible] (CFS)	STREET [illegible] (CFS)	STREET [illegible] (CFS)	STREET [illegible] (CFS)
NATURE TRAIL	0+00	4+00	-1%	0.9	3.7	1.14	12.67
	4+00	9+00	-4%	1.2	4.8	2.27	18
	9+00	14+18	-2.50%	1.5	6.1	1.8	18
	14+18	17+57	8%	1.4	5.8	3.21	18
	17+57	END	2.90%	1.4	5.8	1.9	18
BOULDER PATCH	4+00	5+18	-5.00%	0.2	0.8	2.54	18
	5+18	7+25	-2.00%	0.2	0.8	1.61	17.92
	7+25	8+75	1.00%	1.3	5.3	1.14	12.67
	8+75	13+22	4.00%	1.3	5.3	2.27	18
	13+22	18+50	-3.00%	1.4	5.8	1.97	18
	18+50	END	-8.00%	1.4	5.8	3.21	18

NOTES:

1. *THE 5YR STREET CAPACITY IS BASED ON MAINTAINING A 12' CENTER TRAVEL LANE IN THE 5YR STORM EVENT AND VELOCITIES BELOW 6FPS.*
2. *THE 100YR STREET CAPACITY IS BASED ON CONTAINING ALL RUNOFF WITHIN THE ROW AND VELOCITIES BELOW 6FPS.*

Table 4.0

The storm sewer system was analyzed using StormCAD version 5.5. The system is designed to convey the 5-yr and 100-yr storm events in the drainage pipes. The Appendix includes pipe profiles for both the 5-yr and 100-yr storm events.

The storm sewer system discharges directly to the detention facility in the southeast corner of the property. Riprap sizing calculations were prepared to size the riprap apron for the pipe outlet and are included in the Appendix of this report.

Areas Within Flood Hazard Zone

The Phase I site is entirely within flood zone X on the FEMA Flood Insurance Rate Map (FIRM) 32031C3170E, dated September 30th, 1994. This zone indicates areas that are outside of the 500-year floodplain. A FIRMette is included in the Appendix of this report showing the project boundary in relation to the various flood zones.

Conclusions

The development of the Reserve at Monte Rosa meets the requirements of Article 420 of the Washoe County Development Code and this drainage report addresses the requirements set forth in the Tentative Map Conditions of Approval. The improvements to this site will reduce peak flow rates at each of the confluence points in the 100-yr storm event. Additionally, erosion control and best management practices will ensure sedimentation and erosion of the existing site will be greatly reduced.

Drainage Report Appendices

Vicinity Map
Pre-Development Drainage Map
Post-Development Drainage Map
FIRMette
Table 701-Rational Formula Method Runoff Coefficients
NOAA Atlas 14 Precipitation Frequency Estimates
Figure 907-Allowable Inlet Capacity Sump Condition
Figure 907-Allowable Inlet Capacity Continuous Grade

5-Year

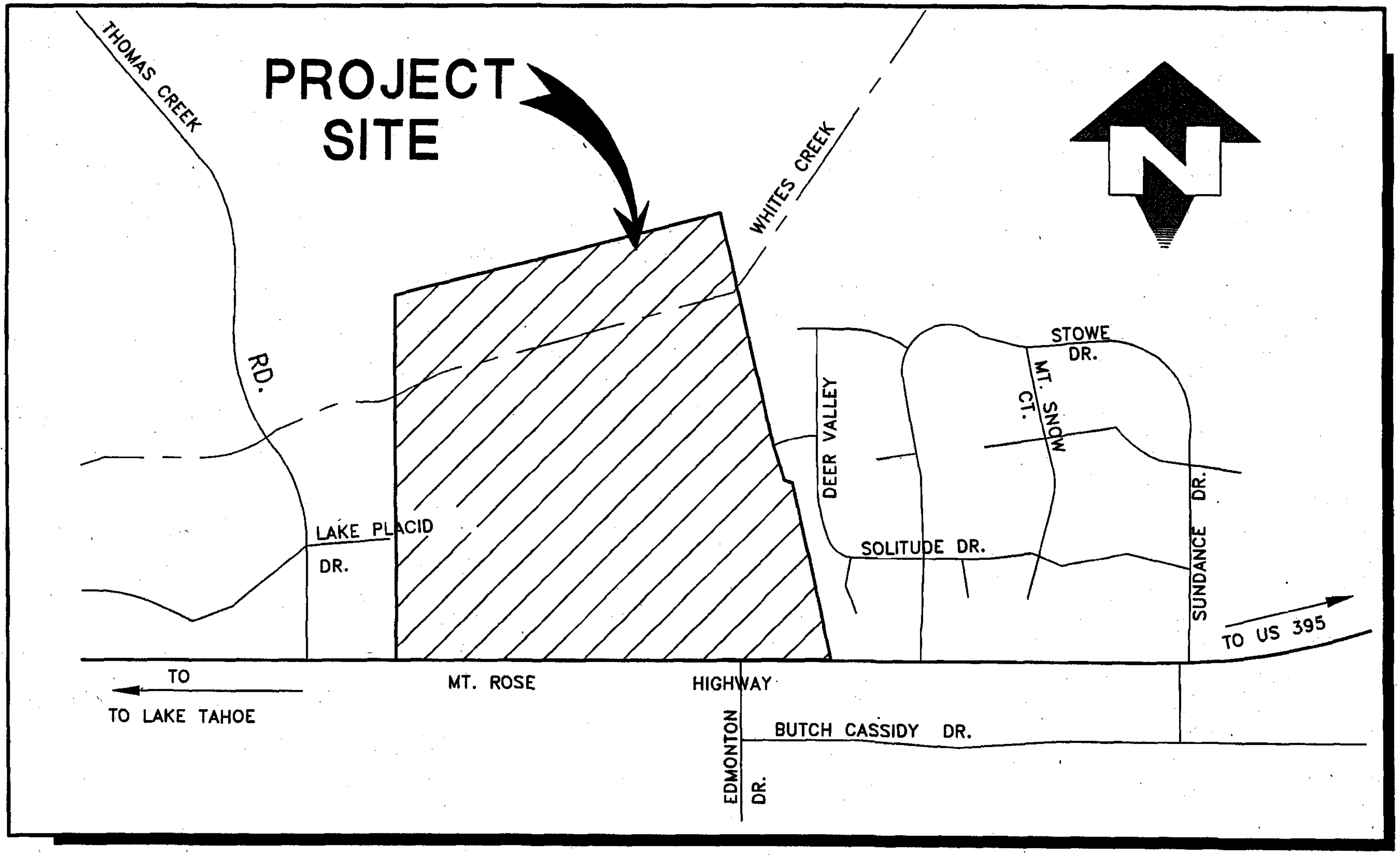
Basin Schematic-StormCAD Plan View
Hydraulic Grade Lines-Storm Drain Profiles
Pipe Analysis Table
Inlet Analysis Table
Junction Analysis Table
Street Spread

100-Year

Basin Schematic-StormCAD Plan View
Hydraulic Grade Lines
Pipe Analysis Table
Inlet Analysis Table
Junction Analysis Table
Street Spread
Erosion Control, and Riprap Sizing
Ditch Sizing Calculations

HEC-1 Analysis

SCS Basin Parameters
5-yr HEC-1 Pre-Development Condition
5-yr HEC-2 Post-Development Condition
100-yr HEC-1 Pre-Development Condition
100-yr HEC-1 Post-Development Condition



VICINITY MAP

NOT TO SCALE

E-DEVELOPMENT-DRAINAGE STUDY

IONTE ROSA PHASE I

ONTE ROSA LLC.

HOE COUNTY NEVADA

APRIL, 2005

LEGEND:

—	DIRECTION OF FLOW
— —	BASIN BOUNDARY

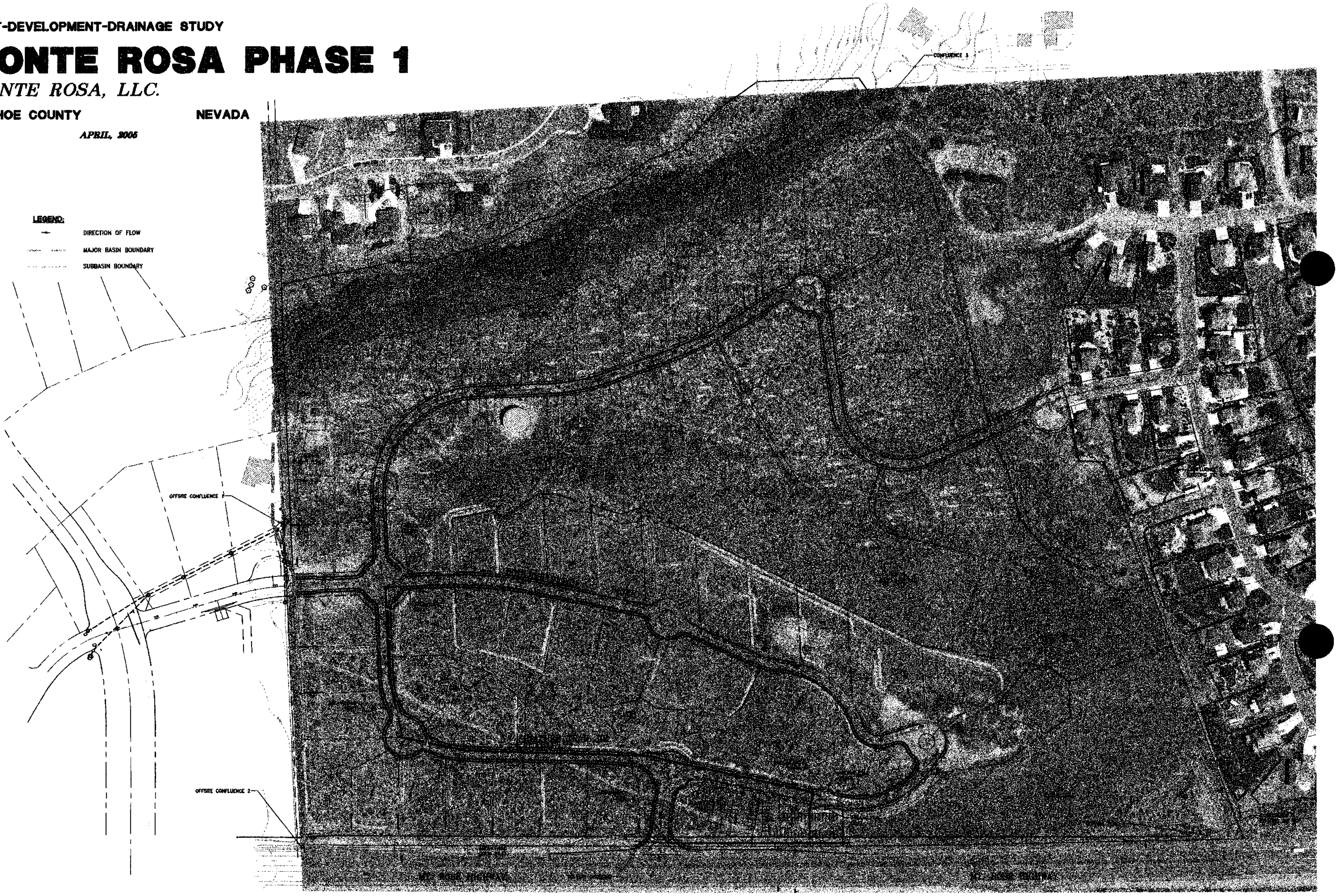
POST-DEVELOPMENT-DRAINAGE STUDY

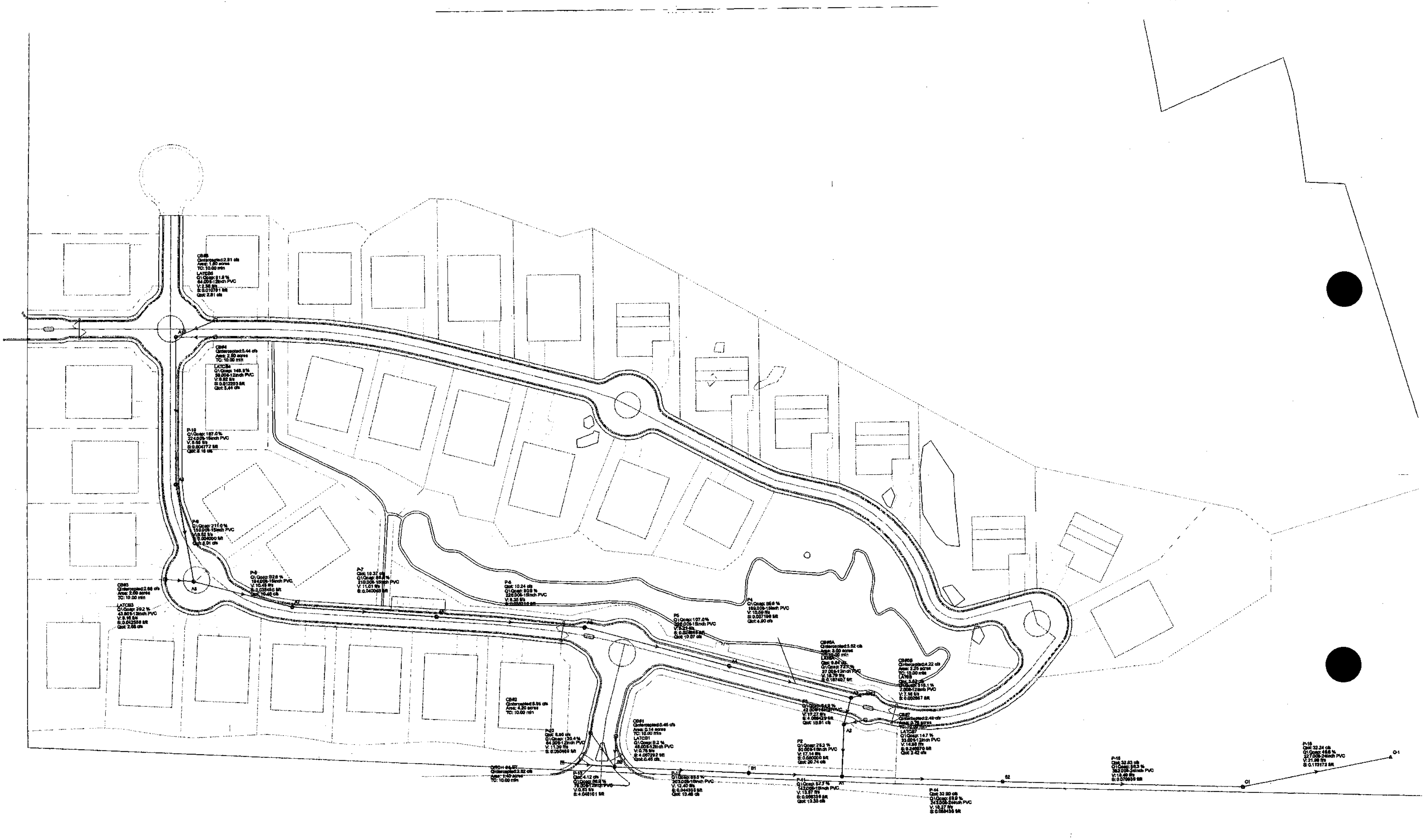
MONTE ROSA PHASE 1

MONTE ROSA, LLC.

WASHOE COUNTY NEVADA

APRIL, 2005



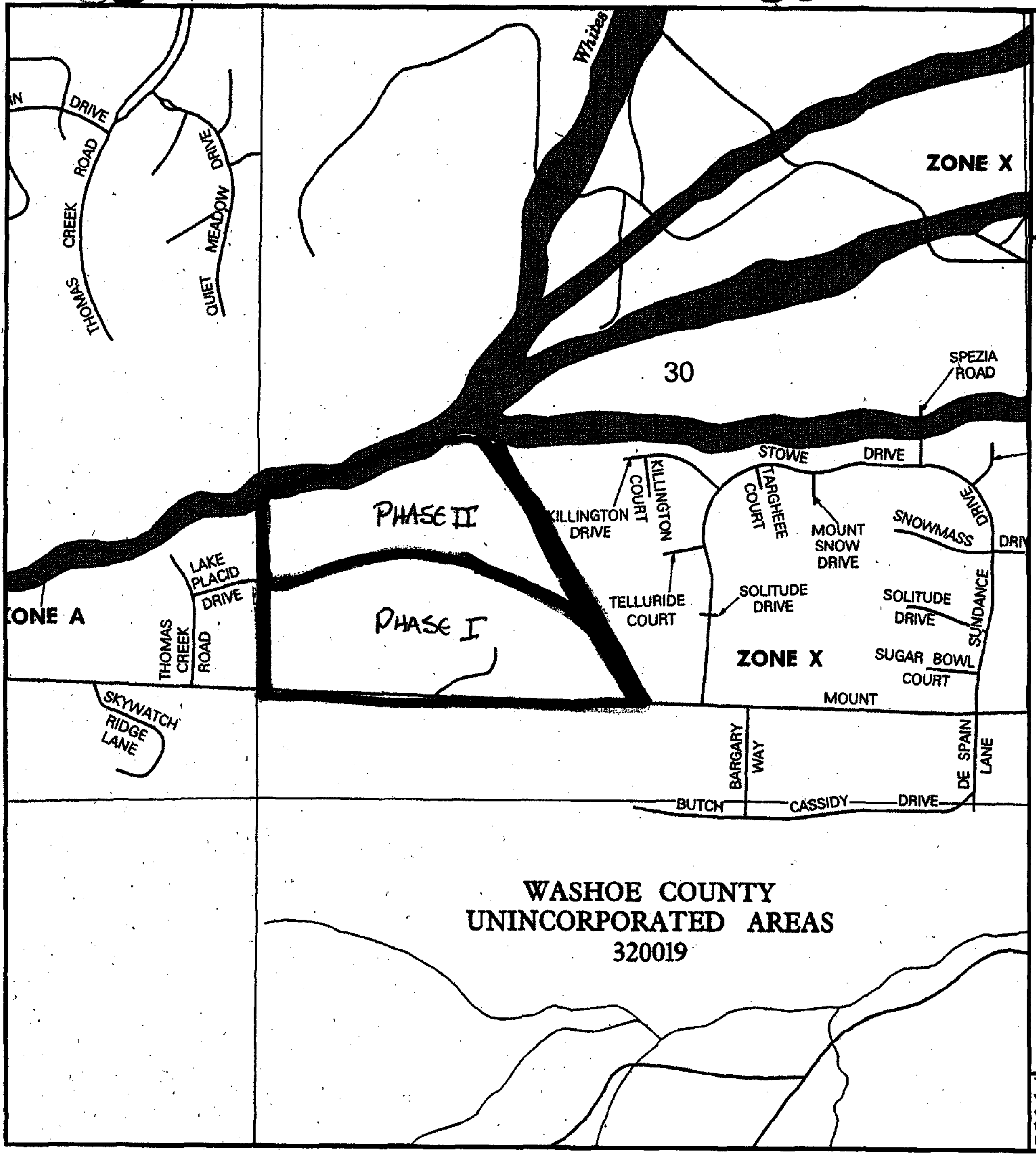


Title: MONTE ROSA UNIT ONE
j:\...\drain\stormcad\monte rosa unit 1 100yr.stm

Wood Rodgers Inc

Project Engineer: WOOD RODGE
StormCAD v5.5 [5.50

~~Fritz v. Washoe Opp to MSJ 000302~~



NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

WASHOE COUNTY,
NEVADA AND
INCORPORATED AREAS

PANEL 3170 OF 3350
(SEE MAP INDEX FOR PANELS NOT PRINTED)

CONTAINS: COMMUNITY	NUMBER	PANEL	SUFFIX
RENO, CITY OF	320020	3170	E
WASHOE COUNTY, UNINCORPORATED AREAS	320019	3170	E

MAP NUMBER
32031C3170 E

EFFECTIVE DATE:
SEPTEMBER 30, 1994

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

RATIONAL FORMULA METHOD
RUNOFF COEFFICIENTS

Land Use or Surface Characteristics	Aver. % Impervious Area	Runoff Coefficients 5-Year (C_5)	100-Year (C_{100})
Business/Commercial:			
Downtown Areas	85	.82	.85
Neighborhood Areas	70	.65	.80
Residential: (Average Lot Size)			
⅛ Acre or Less (Multi-Unit)	65	.60	.78
¼ Acre	38	.50	.65
⅓ Acre	30	.45	.60
½ Acre	25	.40	.55
1 Acre	20	.35	.50
Industrial:	72	.68	.82
Open Space: (Lawns, Parks, Golf Courses)	5	.05	.30
Undeveloped Areas:			
Range	0	.20	.50
Forest	0	.05	.30
Streets/Roads:			
Paved	100	.88	.93
Gravel	20	.25	.50
Drives/Walks:	95	.87	.90
Roofs:	90	.85	.87

Notes:

1. Composite runoff coefficients shown for Residential, Industrial, and Business/Commercial Areas assume irrigated grass landscaping for all previous areas. For development with landscaping other than irrigated grass, the designer must develop project specific composite runoff coefficients from the surface characteristics presented in this table.

VERSION: December 2, 1996	**REFERENCE:** USDCM, DROCOG, 1969 (with modifications)	**TABLE** **701**
WRC ENGINEERING, INC		

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

Nevada 39.4 N 119.8 W 5285 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M. Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003
Extracted: Thu Apr 21 2005

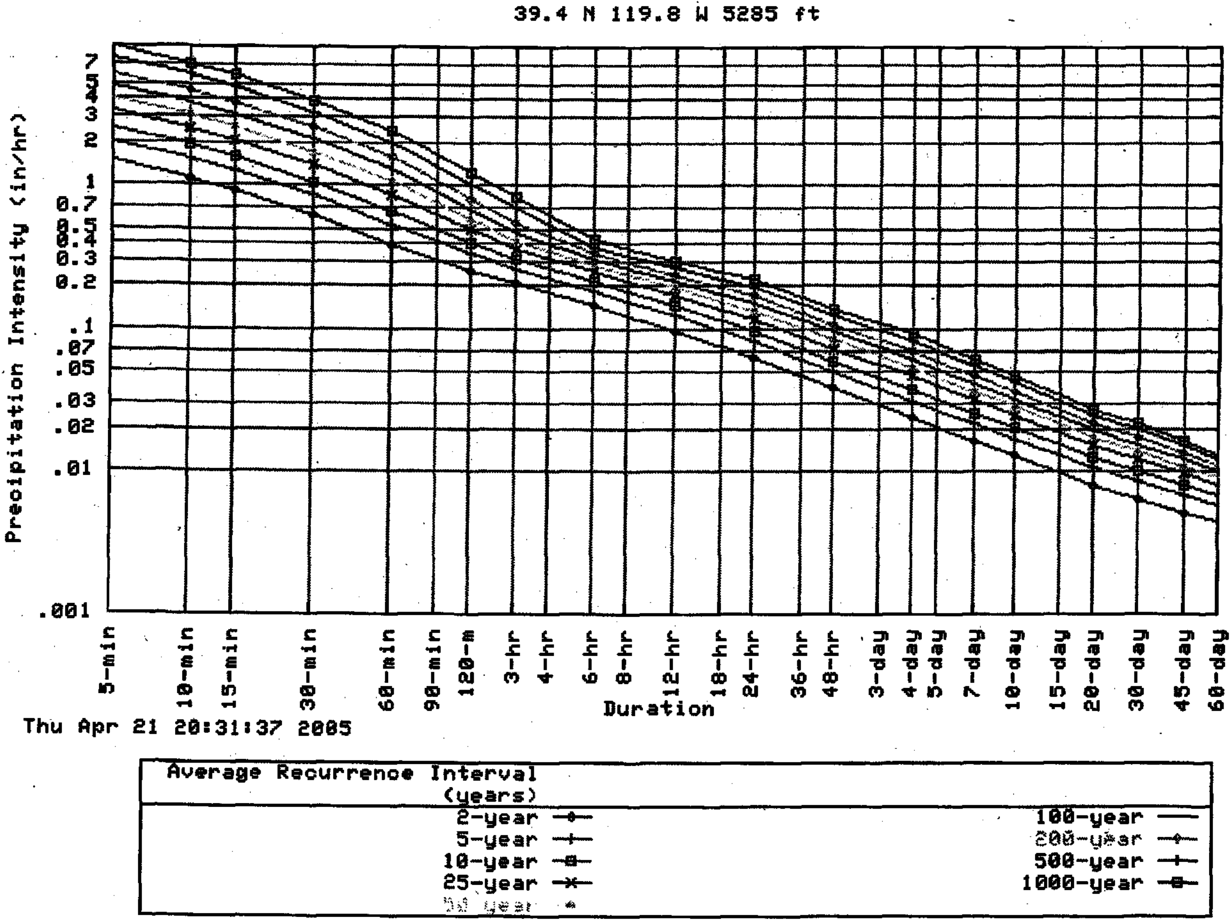
Confidence Limits | Seasonality | Location Maps | Other Info. | Grids | Maps | Help | Docs

Precipitation Intensity Estimates (in/hr)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.45	1.10	0.92	0.62	0.38	0.26	0.21	0.15	0.10	0.06	0.04	0.02	0.02	0.01	0.01	0.01	0.01	0.00
5	1.99	1.52	1.26	0.85	0.52	0.33	0.26	0.18	0.12	0.08	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.01
10	2.48	1.89	1.56	1.05	0.65	0.40	0.30	0.21	0.14	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01	0.01
25	3.28	2.50	2.06	1.39	0.86	0.49	0.36	0.25	0.17	0.12	0.07	0.05	0.03	0.03	0.02	0.01	0.01	0.01
50	4.02	3.06	2.53	1.70	1.05	0.58	0.41	0.28	0.19	0.13	0.08	0.05	0.04	0.03	0.02	0.01	0.01	0.01
100	4.90	3.72	3.08	2.07	1.28	0.67	0.47	0.31	0.22	0.15	0.10	0.06	0.04	0.03	0.02	0.02	0.01	0.01
200	5.94	4.52	3.74	2.52	1.56	0.80	0.55	0.34	0.24	0.17	0.11	0.07	0.05	0.04	0.02	0.02	0.01	0.01
500	7.63	5.81	4.80	3.23	2.00	1.02	0.69	0.37	0.27	0.20	0.12	0.08	0.06	0.04	0.03	0.02	0.02	0.01
1000	9.22	7.01	5.79	3.90	2.41	1.22	0.83	0.42	0.29	0.22	0.14	0.09	0.06	0.05	0.03	0.02	0.02	0.01

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.



Confidence Limits -

* Upper bound of the 90% confidence interval Precipitation Intensity Estimates (in/hr)																	
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.72	1.31	1.08	0.73	0.45	0.29	0.23	0.16	0.11	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.01
5	2.36	1.80	1.49	1.00	0.62	0.38	0.29	0.21	0.14	0.09	0.06	0.04	0.02	0.02	0.01	0.01	0.01	0.01
10	2.93	2.23	1.84	1.24	0.77	0.45	0.34	0.24	0.16	0.11	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.01
25	3.91	2.98	2.46	1.66	1.02	0.57	0.41	0.28	0.20	0.13	0.09	0.05	0.04	0.03	0.02	0.01	0.01	0.01
50	4.87	3.71	3.06	2.06	1.28	0.68	0.48	0.32	0.22	0.15	0.10	0.06	0.04	0.03	0.02	0.02	0.01	0.01
100	6.02	4.59	3.79	2.55	1.58	0.83	0.57	0.35	0.25	0.18	0.11	0.07	0.05	0.04	0.02	0.02	0.01	0.01
200	7.55	5.74	4.75	3.20	1.98	1.01	0.69	0.39	0.28	0.20	0.13	0.08	0.06	0.04	0.03	0.02	0.02	0.01
500	10.07	7.66	6.33	4.26	2.64	1.34	0.91	0.48	0.33	0.24	0.15	0.10	0.07	0.05	0.03	0.02	0.02	0.01
1000	12.53	9.53	7.88	5.30	3.28	1.66	1.13	0.58	0.36	0.27	0.17	0.11	0.07	0.06	0.03	0.03	0.02	0.02

* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.

** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

* Lower bound of the 90% confidence interval

Precipitation Intensity Estimates (in/hr)

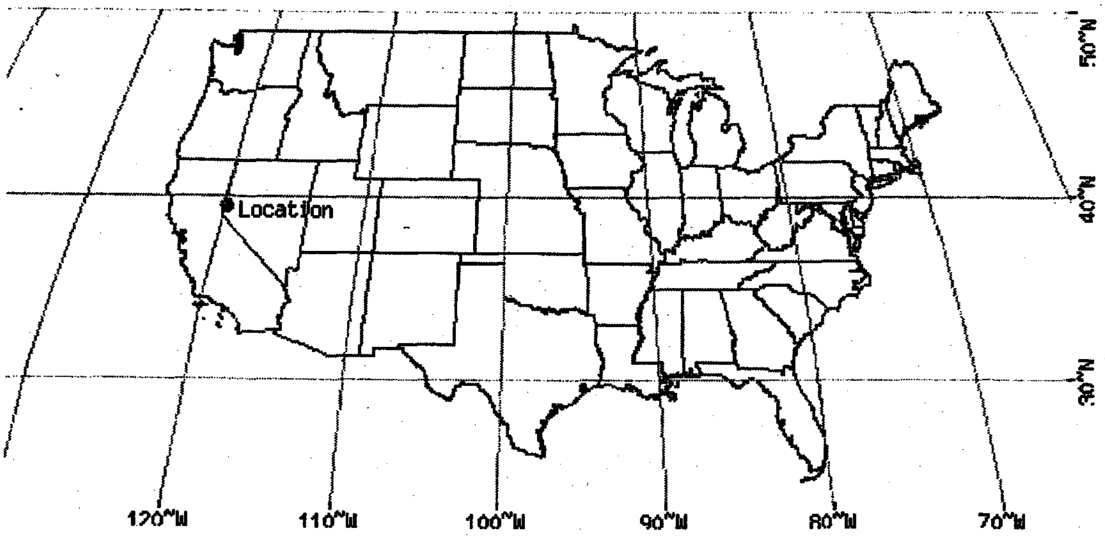
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.26	0.95	0.79	0.53	0.33	0.23	0.19	0.13	0.09	0.06	0.03	0.02	0.01	0.01	0.01	0.01	0.00	0.00
5	1.72	1.30	1.08	0.73	0.45	0.29	0.23	0.16	0.11	0.07	0.04	0.03	0.02	0.02	0.01	0.01	0.01	0.01
10	2.11	1.61	1.33	0.89	0.55	0.34	0.27	0.19	0.13	0.08	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.01
25	2.70	2.05	1.70	1.14	0.71	0.42	0.32	0.22	0.15	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01	0.01
50	3.17	2.41	2.00	1.34	0.83	0.47	0.35	0.24	0.17	0.11	0.07	0.05	0.03	0.02	0.02	0.01	0.01	0.01
100	3.71	2.82	2.33	1.57	0.97	0.53	0.39	0.26	0.18	0.13	0.08	0.05	0.04	0.03	0.02	0.01	0.01	0.01
200	4.30	3.27	2.70	1.82	1.13	0.61	0.45	0.28	0.19	0.14	0.09	0.06	0.04	0.03	0.02	0.01	0.01	0.01
500	5.12	3.90	3.22	2.17	1.34	0.74	0.55	0.30	0.21	0.16	0.10	0.07	0.05	0.04	0.02	0.02	0.01	0.01
1000	5.84	4.45	3.67	2.47	1.53	0.85	0.64	0.34	0.22	0.17	0.11	0.07	0.05	0.04	0.02	0.02	0.01	0.01

* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than.

** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -



These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.

LEGEND

— State	— Connector
— County	▆ Stream
▨ Indian Resv	▆ Military Area
▆ Lake/Pond/Ocean	▆ National Park
— Street	▆ Other Park
— Expressway	▨ City
— Highway	— County

Scale 1:228583 0 2 4 6 8 mi / 0 2 4 6 8 10 km
*average--true scale depends on monitor resolution

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Jacqueline Bryant
Clerk of the Court
Transaction # 4818450 : melwood

Exhibit 18

Exhibit 18

TECHNICAL DRAINAGE REPORT

The Reserve at Monte Rosa Unit 2

Project # 1460.004
Washoe County, Nevada

Prepared for:
MONTER ROSA, LLC
ALAN MEANS
6121 LAKESIDE DRIVE, SUITE #230
RENO, NEVADA 89511

APRIL 21, 2006

Prepared by:
Jon Bailey, EI
Logan Carter, EI

PROFESSIONAL ENGINEER-STATE OF NEVADA
SCOTT A. CHRISTY
Exp. 12-31-06
CIVIL
No. 14498

4/21/06

Scott Christy, P.E.

WOOD RODGERS
ENGINEERING • PLANNING • MAPPING • SURVEYING

575 Double Eagle Court
Reno, Nevada 89521
(775) 823-4068

TECHNICAL DRAINAGE REPORT

The Reserve at Monte Rosa
Phase II
A Single Family Home Residential Community

Prepared for:

Monte Rosa LLC.
Alan Means
6121 Lakeside Dr. Suite 236

Prepared by:

Wood Rodgers Inc.
6774 S. McCarran Blvd
Reno, NV 89509
(775) 823-4068

June 28, 2006

WOOD RODGERS INC.
ENGINEERING PLANNING MAPPING SURVEYING

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Drainage Report Appendices

HEC-1 Analysis

SCS Basin Parameters
5-yr HEC-1 Pre-Development Condition
5-yr HEC-1 Post-Development Condition
100-yr HEC-1 Pre-Development Condition
100-yr HEC-1 Post-Development Condition

Introduction

This report presents the storm-water management plan for Phase II of The Reserve at Monte Rosa, a single-family home residential development. Phase II consists of 32 single- family residential lots ranging from one-half acre to over an acre in size. The site is bounded by the Saddlehorn South-Phase Five to the west, the Mt. Rose Highway (U.S. 431) to the south, White's Creek to the north, and Lancer Estates Unit Six to the east. An aerial photo of the site and the surrounding subdivisions and land features is included in the Appendix of this report. The site is contained within Section 30, Township 18 North, and Range 20 East. The approximate Latitude and Longitude is 39.39°N 119.8°W.

The topography of the site ranges from steep rocky terrain to gentle sloping areas with an elevation range from 5020 ft to 5175ft. The existing-conditions drainage map included in the Appendix shows an aerial photo of the site with contours at 10-ft intervals. Existing drainage facilities and site conditions are detailed on this map.

Jeff Codega Inc prepared a previous hydrology report for Saddlehorn South Phase Seven that is adjacent and west of the subject parcel. Basin parameters for the upstream and offsite areas were derived from this study and included in the HEC-1 model prepared for this report. A list of basin parameters is included in the Appendix of this report for both onsite and offsite areas.

The purpose of this report is to show the drainage plan conforms to *Article 420 of the Washoe County Development Code* and the *Conditions for The Reserve at Monte Rosa Tentative Subdivision Map* dated January 5th 2005. This report includes analysis for Phase II of The Reserve at Monte Rosa.

Pre-Development Drainage System

The existing drainage system consists of three basins that cover the entire property (both Phase I and Phase II) with confluence points in three locations. Basin One includes the southern-most portion of the property including runoff from Lancer Hill and a portion of the Mt Rose Highway. A roadside ditch directs runoff from Basin One to a 24" reinforced concrete pipe in the southeast corner of the site. A small area of offsite runoff enters the subject parcel via the roadside ditch along the Mt Rose Highway at the southwest corner of the property. The majority of the Phase I development is included within Basin One. Basin Two encompasses a large central portion of the site. Runoff from Basin Two consists of overland flow to a cutoff ditch on the eastern boundary of the property that directs the flow to a 24" reinforced concrete pipe in the Lancer Estates Unit 6 Subdivision. Basin Three includes the northern portion of the property that flows to Whites Creek. Offsite flows from an existing 18" RCP enter Basin Three at the northwest property boundary and are directed through the site in a drainageway before entering Whites Creek.

The pre-development drainage system was modeled using the SCS Method and HEC-1 Version 4.0.1E, and WMS Version 6.0. The parameters utilized for modeling the existing condition are included in the Appendix of this report. A composite curve number was derived for each basin from hydrologic soil groups and landuse data. The design storm was taken from the NOAA Atlas 14 Point Precipitation Frequency Estimates for the subject parcel.

Table 1.0 shows runoff computed for both the 5-yr storm event and 100-yr storm event for each of the three confluence points shown on the pre-development drainage system map.

Confluence Point	5-yr Runoff (cfs)	100-yr Runoff (cfs)
1	1.2	19.9
2	1.7	29.0
3	3.2	38.9

Table 1.0

Proposed (Developed) Drainage System

The proposed drainage system was analyzed using the SCS Method for sizing of detention ponds and to compare with the pre-development peak-flow rates at each of the three confluence points. The major basin areas were modeled with composite curve numbers that reflect the developed-condition. The detention ponds were sized to reduce the developed-peak flow rates to below the pre-development peak flow rates. One-detention basin will be constructed with Phase I of the development, and additional detention facilities will be analyzed and modeled with Phase II of the project.

The following table shows the peak-flow rate for each of the three confluence points in the developed condition, and the peak flow rate with the proposed detention facilities. Confluences 2 and 3 have been modeled with approximate pond sizes. With the Phase II submittal the HEC-1 model will be updated and revised with exact pond sizes.

Confluence Point	5-yr Runoff (cfs) Without Detention	100-yr Runoff (cfs) Without Detention	5-yr Runoff (cfs) With Detention	100-yr Runoff (cfs) With Detention
1	7.5	42.0	1.3	11.1
2	4.8	34.2	2.5	23.6
3	3.4	51.4	3.5	35.5

Table 2.0

Peak flow rates have been reduced in each of the three confluence points for the 100-yr storm event.

The detention facility to be constructed with Phase I includes 1.6ac/ft of capacity. A 24" standpipe will be constructed with a grate invert at 2' below the top of the pond. An 8" low flow orifice will be constructed as the low flow outlet. An emergency spillway will be constructed to mitigate flows greater than the 100-yr storm event.

The Rational Method was utilized to calculate peak-flow rates for sub-basin areas contributing to the on-site storm drainage system. The following table includes a list of the basin areas, intensities, and runoff coefficients for each basin. A minimum time of concentration of ten minutes was used for each sub-basin.

Table 3.0

Catch Basin ID	Area (ac)	Runoff C 100yr, 5yr	Intensity (in/hr) 100yr, 5yr	5-yr Runoff (cfs)	100-yr Runoff (cfs)
1	1.80	0.55, 0.55	4.26, 1.60	1.60	4.25
2	1.20	0.55, 0.55	4.26, 1.60	1.06	2.83
3	1.20	0.55, 0.55	4.26, 1.60	1.27	4.43
4	1.24	0.55, 0.55	4.26, 1.60	1.15	3.71
6	1.52	0.55, 0.55	4.26, 1.60	1.38	4.71
7	0.86	0.55, 0.55	4.26, 1.60	0.84	3.74
5	0.20	0.55, 0.55	4.26, 1.60	0.23	2.01
9	0.65	0.55, 0.55	4.26, 1.60	0.58	2.68
8	0.36	0.55, 0.55	4.26, 1.60	0.32	1.14
11	0.10	0.55, 0.55	4.26, 1.60	0.09	0.78
10	1.14	0.55, 0.55	4.26, 1.60	1.01	2.71
13	0.10	0.55, 0.55	4.26, 1.60	2.45	3.51
12	2.66	0.55, 0.55	4.26, 1.60	1.98	6.81
14	1.36	0.55, 0.55	4.26, 1.60	1.21	3.21

Street capacity calculations were completed for each of the roadways in the Phase II development. Rating tables from Flowmaster version 7.0 were prepared for a range of street slopes at the allowed depth of flow for the 5-yr and 100-yr storm events. The following table shows the flow in the street and street capacity for each section of roadway with different grades for Aspen Hollow.

Street Capacity

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
STREET NAME	Sta.	Sta.	Street	5-yr	5-yr	100-yr	100-yr	Velocity
	(from)	(to)	Slope	Flow	Capacity	Flow	Capacity	(100-yr)
Aspen Hollow	(ft)	(ft)	(%)	(cfs)	(cfs)	(cfs)	(cfs)	(fps)
CB#1,2 @3+22	0	3+77	8.0%	1.6	3.22	4.25	12.85	8
CB#3,4 @5+89	3+77	7+50	4.4%	1.27	2.37	4.43	20.65	8
CB#6 @9+36	7+50	14+00	6.4%	1.38	2.88	4.71	15.28	8
CB#5,7 @10+96			6.4%	0.84	2.88	3.74	15.28	8
CB#8,9 @13+87			6.4%	0.58	2.88	2.68	15.28	8
	14+00	17+00	3.7%	1.07	2.19	2.86	32.6	8
CB#10,11 @22+96	17+00	23+50	8.0%	2.36	3.22	6.81	12.85	8
	23+50	END	2.8%	N/A	N/A	N/A	N/A	N/A

(1) Street name with Catch Basin's and their stations.

(2) Station from which that street slope begins

(3) Station from which that street slope ends

(4) Actual 5-yr flow in street computed from StormCAD

(5) 5-yr flow capacity computed from FlowMaster using a flow depth of 0.28ft. This is the depth of water in the channel allowed before it cross's half the lane.

(11ft/2)*(0.02ft/ft) + (0.17ft) = 0.28

(6) Actual 100-yr flow in street computed from StormCAD

(7) 100-yr capacity computed from FlowMaster using a max velocity of 8fps and a max channel depth of 0.6ft. 0.6ft is the max depth before the water cross's the Right of Way.

(8) Max velocity for 100-yr flow

The storm sewer system was analyzed using StormCAD version 5.5. The system is designed to convey the 5-yr and 100-yr storm events in the drainage pipes. The Appendix includes pipe profiles for both the 5-yr and 100-yr storm events.

The storm sewer system discharges directly to the detention facility in the southeast corner of the property. Riprap sizing calculations were prepared to size the riprap apron for the pipe outlet and are included in the Appendix of this report.

Areas Within Flood Hazard Zone

The developed portion of Phase II site is entirely within flood zone X on the FEMA Flood Insurance Rate Map (FIRM) 32031C3170E, dated September 30th, 1994. This zone indicates areas that are outside of the 500-year floodplain. A FIRMette is included in the Appendix of this report showing the project boundary in relation to the various flood zones.

Conclusions

The development of the Reserve at Monte Rosa meets the requirements of Article 420 of the Washoe County Development Code and this drainage report addresses the requirements set forth in the Tentative Map Conditions of Approval. The improvements to this site will reduce peak flow rates at each of the confluence points in the 100-yr storm event. Additionally, erosion control and best management practices will ensure sedimentation and erosion of the existing site will be greatly reduced.

Table of Contents

Hydrologic Analysis with SCS using HEC-1

APPENDIX A

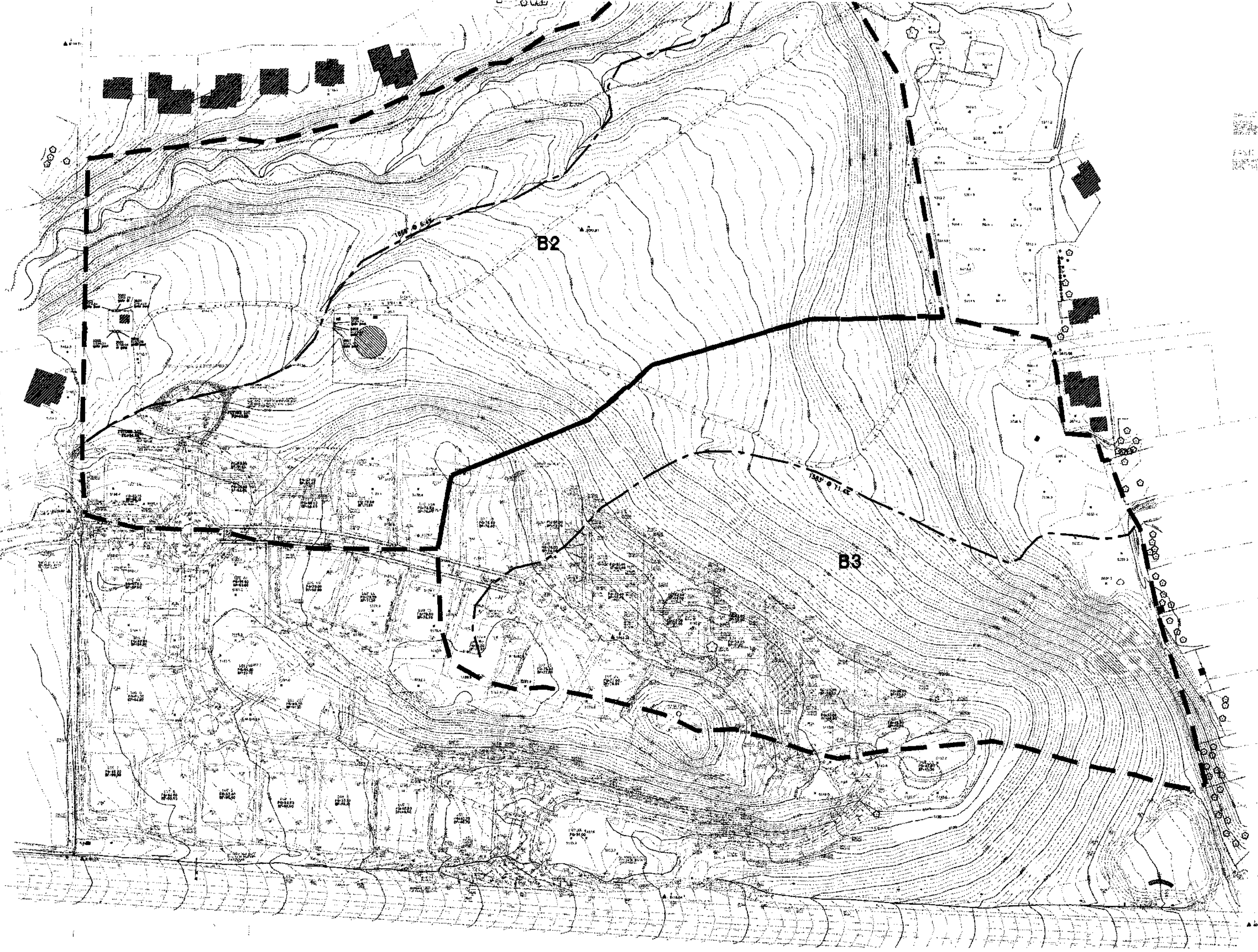
PRE-DRAINAGE EXHIBIT

THE RESERVE @ MONTE ROSA UNIT 2

MONTE ROSA, LLC.

RENO **NEVADA**

DECEMBER, 2005



WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS
575 Double Eagle Court Tel 775.823.4068
Reno, NV. 89521 Fax 775.823.4066

APPENDIX B

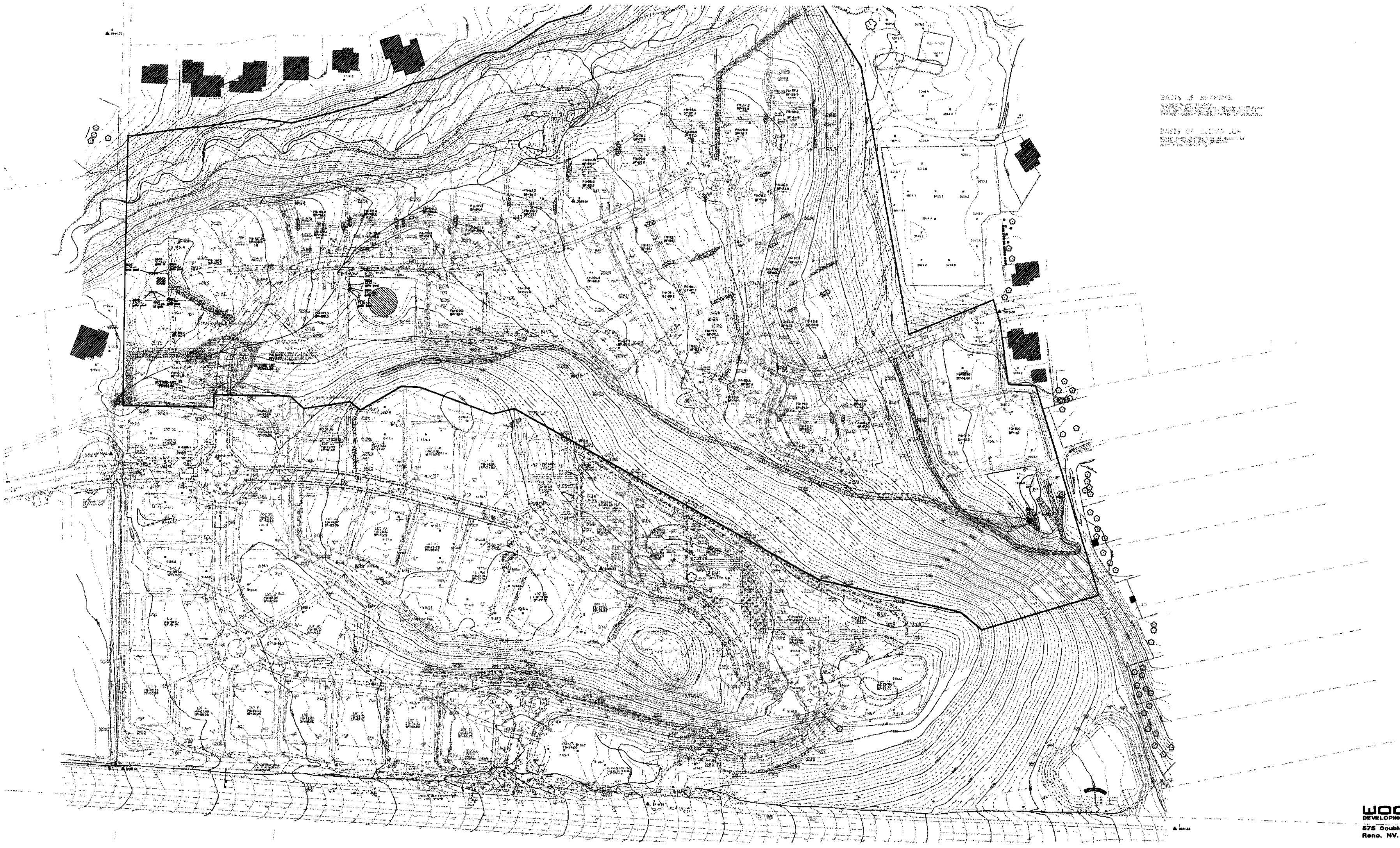
POST DRAINAGE EXHIBIT

THE RESERVE @ MONTE ROSA UNIT 2

MONTE ROSA, LLC.

RENO **NEVADA**

DECEMBER, 2005



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APPENDIX C

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

Nevada 39.4 N 119.8 W 5285 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003
Extracted: Wed Feb 16 2005

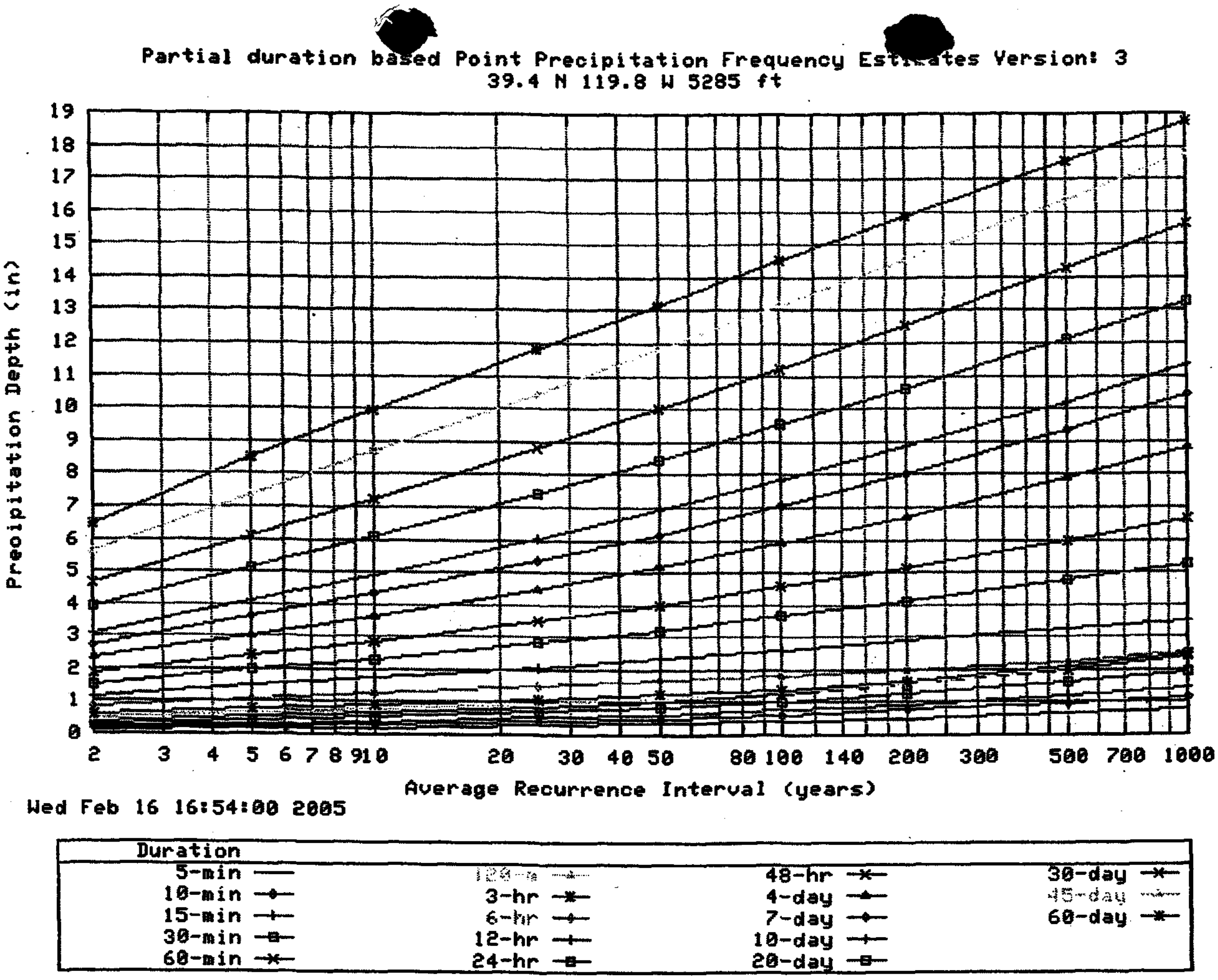
Confidence Limits | Seasonality | Location Maps | Other Info. | Grids | Maps | Help | Docs

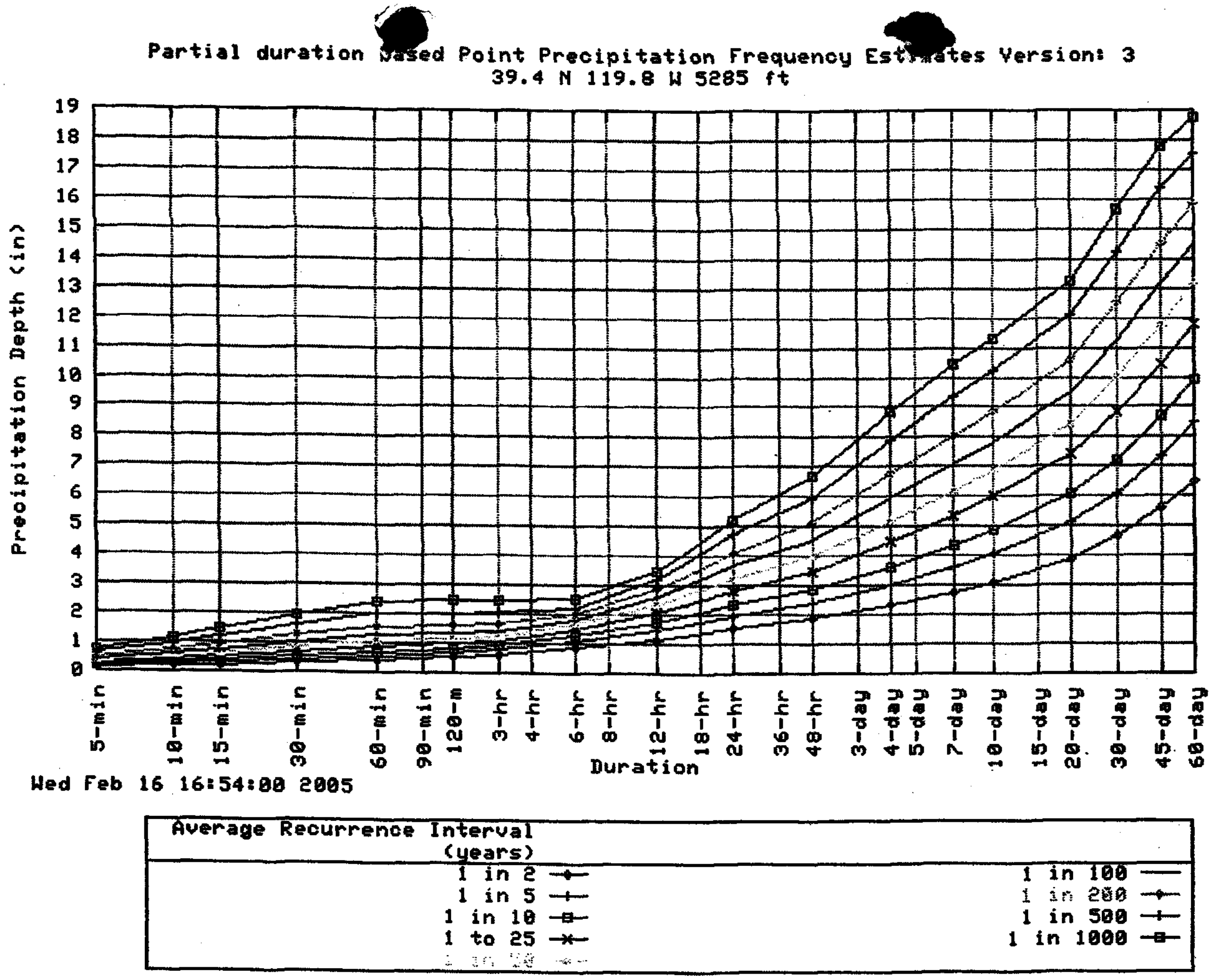
Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.12	0.18	0.23	0.31	0.38	0.51	0.62	0.87	1.16	1.53	1.86	2.33	2.74	3.10	3.90	4.65	5.61	6.46
5	0.17	0.25	0.31	0.42	0.52	0.66	0.79	1.10	1.49	1.97	2.41	3.04	3.62	4.11	5.14	6.12	7.37	8.48
10	0.21	0.32	0.39	0.53	0.65	0.79	0.91	1.27	1.74	2.32	2.85	3.63	4.33	4.90	6.11	7.25	8.69	9.93
25	0.27	0.42	0.52	0.69	0.86	0.99	1.09	1.50	2.08	2.82	3.49	4.48	5.35	6.02	7.42	8.80	10.46	11.79
50	0.34	0.51	0.63	0.85	1.05	1.16	1.24	1.66	2.33	3.22	4.01	5.17	6.17	6.91	8.44	10.00	11.80	13.17
100	0.41	0.62	0.77	1.04	1.28	1.34	1.41	1.83	2.60	3.65	4.56	5.92	7.06	7.86	9.50	11.25	13.17	14.52
200	0.49	0.75	0.93	1.26	1.56	1.59	1.65	2.01	2.87	4.10	5.15	6.72	8.00	8.84	10.59	12.53	14.54	15.84
500	0.64	0.97	1.20	1.62	2.00	2.03	2.09	2.24	3.23	4.74	5.99	7.88	9.34	10.21	12.10	14.27	16.37	17.53
1000	0.77	1.17	1.45	1.95	2.41	2.44	2.49	2.52	3.51	5.25	6.66	8.83	10.43	11.32	13.27	15.63	17.76	18.76

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.


Partial duration based Point Precipitation Frequency Estimates Version: 3
39.4 N 119.8 W 5285 ft
Precipitation Depth (in)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
2
3
4
5
6
7
8
9
10
20
30
40
50
80
100
140
200
300
500
700
1000
Average Recurrence Interval (years)
Wed Feb 16 16:54:00 2005
Duration
5-min
10-min
15-min
30-min
60-min
120-m
3-hr
6-hr
12-hr
24-hr
48-hr
4-day
7-day
10-day
20-day
30-day
45-day
60-day



Confidence Limits -

* Upper bound of the 90% confidence interval
Precipitation Frequency Estimates (inches)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.14	0.22	0.27	0.36	0.45	0.59	0.70	0.98	1.29	1.73	2.15	2.64	3.13	3.54	4.44	5.31	6.29	7.29
5	0.20	0.30	0.37	0.50	0.62	0.76	0.88	1.24	1.67	2.23	2.79	3.45	4.14	4.69	5.83	6.98	8.26	9.55
10	0.24	0.37	0.46	0.62	0.77	0.91	1.03	1.43	1.95	2.63	3.32	4.13	4.96	5.61	6.94	8.28	9.75	11.16
25	0.33	0.50	0.61	0.83	1.02	1.15	1.25	1.70	2.36	3.23	4.09	5.10	6.14	6.90	8.44	10.06	11.77	13.30
50	0.41	0.62	0.77	1.03	1.28	1.37	1.43	1.90	2.68	3.71	4.73	5.91	7.11	7.93	9.63	11.47	13.31	14.90
100	0.50	0.77	0.95	1.28	1.58	1.66	1.72	2.12	3.04	4.27	5.45	6.79	8.15	9.05	10.89	12.95	14.90	16.50
200	0.63	0.96	1.19	1.60	1.98	2.02	2.08	2.37	3.39	4.87	6.21	7.76	9.30	10.25	12.22	14.52	16.54	18.05
500	0.84	1.28	1.58	2.13	2.64	2.68	2.75	2.87	3.93	5.74	7.33	9.20	10.98	11.97	14.11	16.68	18.76	20.12
1000	1.04	1.59	1.97	2.65	3.28	3.32	3.38	3.48	4.36	6.49	8.33	10.42	12.38	13.38	15.59	18.45	20.55	21.70

* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.

** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

* Lower bound of the 90% confidence interval
Precipitation Frequency Estimates (inches)

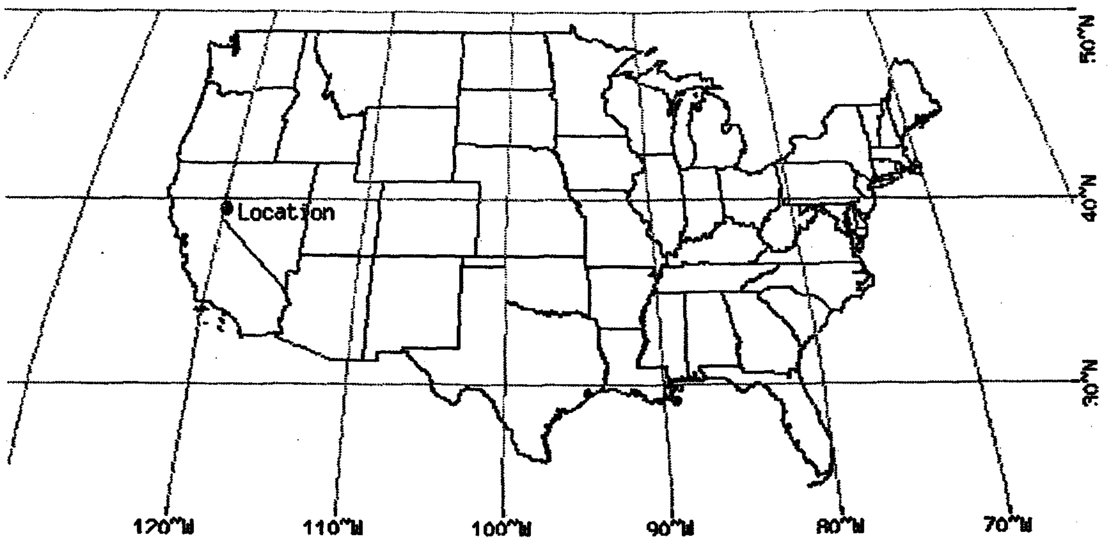
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	[illegible] day	20 day	30 day	45 day	60 day
2	0.10	0.16	0.20	0.27	0.33	0.46	0.56	0.78	1.04	1.37	1.63	2.07	2.42	2.74	3.47	4.13	4.98	5.70
5	0.14	0.22	0.27	0.36	0.45	0.58	0.70	0.98	1.33	1.74	2.10	2.70	3.18	3.61	4.55	5.40	6.52	7.45
10	0.18	0.27	0.33	0.45	0.55	0.69	0.81	1.12	1.53	2.04	2.47	3.21	3.79	4.29	5.38	6.37	7.66	8.71
25	0.23	0.34	0.42	0.57	0.71	0.83	0.95	1.30	1.80	2.43	2.98	3.92	4.63	5.22	6.48	7.67	9.17	10.31
50	0.26	0.40	0.50	0.67	0.83	0.95	1.06	1.42	1.99	2.74	3.39	4.48	5.30	5.95	7.32	8.65	10.29	11.46
100	0.31	0.47	0.58	0.79	0.97	1.07	1.18	1.54	2.17	3.04	3.79	5.07	6.00	6.70	8.18	9.62	11.40	12.56
200	0.36	0.55	0.68	0.91	1.13	1.21	1.35	1.65	2.35	3.35	4.21	5.67	6.70	7.44	9.02	10.61	12.49	13.61
500	0.43	0.65	0.81	1.09	1.34	1.47	1.66	1.78	2.55	3.74	4.76	6.49	7.67	8.44	10.12	11.91	13.91	14.96
1000	0.49	0.74	0.92	1.24	1.53	1.70	1.92	2.02	2.70	4.04	5.17	7.15	8.44	9.21	10.96	12.88	14.94	15.91

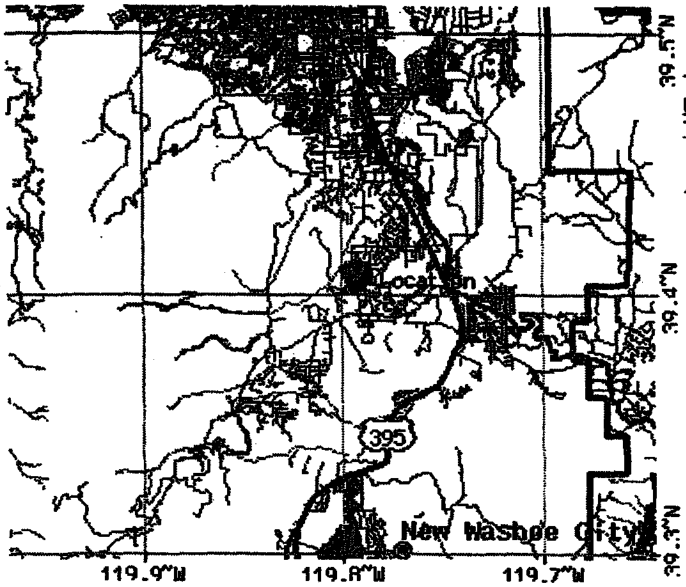
* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than.

** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -





These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.

LEGEND

— State	— Connector
— County	■ Stream
▨ Indian Resv	■ Military Area
■ Lake/Pond/Ocean	■ National Park
— Street	■ Other Park
— Expressway	☐ City
— Highway	— County

Scale 1:228583 0 2 4 6 8 10 12 14 16 18 mi; 0 2 4 6 8 10 km
*average--true scale depends on monitor resolution

Other Maps/Photographs -

View USGS digital orthophoto quadrangle (DOQ) covering this location from TerraServer; USGS Aerial Photograph may also be available
from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilts has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the USGS for more information.

Watershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information
about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study,
please refer to our documentation.

Using the **National Climatic Data Center's (NCDC)** station search engine, locate other climate stations within:

+/-30 minutes ...OR... +/-1 degree of this location (39.4/-119.8). Digital ASCII data can be obtained directly from **NCDC**.

Find Natural Resources Conservation Service (NRCS) SNOTEL (SNOwpack TELemetry) stations by visiting the **Western Regional Climate Center's state-specific SNOTEL station maps**.

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
Silver Spring, MD 20910

(301) 713-1669
Questions?: HDSC.Questions@noaa.gov

Disclaimer