FINAL REPORT
GEOTECHNICAL AND ENGINEERING STUDIES FOR THE NEW LOS PADRES WATER SUPPLY PROJECT

MONTEREY PENINSULA WATER
MANAGEMENT DISTRICT

94-1198801.80
March 16, 1995

THE
MARK
GROUP, INC.
ENGINEERS & GEOLOGISTS

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ENGINEERS & GEOLOGISTS

March 16, 1995
94-119801.80

Mr. Andrew M. Bell, P.E.
District Engineer
Monterey Peninsula Water Management District
187 Eldorado Street
Monterey, California 93942-0085

Subject: REPORT - Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

Dear Mr. Bell:

The MARK Group, Inc. (MARK) is pleased to submit ten copies of this draft final report to Monterey Peninsula Water Management District (MPWMD) for the Geotechnical and Engineering Studies of the New Los Padres Water Supply Project.

We appreciate the opportunity to work as a consultant to MPWMD on this interesting project.

Sincerely,

The MARK Group, Inc.

David Rogers

David K. Rogers, P.E., C.E.G.
Principal

DKR:KBK:scd
GEOTCH2.LTR

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PROFESSIONAL CERTIFICATION

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FINAL REPORT
GEOTECHNICAL AND ENGINEERING STUDIES FOR THE NEW LOS PADRES WATER SUPPLY PROJECT
MONTEREY PENINSULA WATER MANAGEMENT DISTRICT

94-1198801.80
March 16, 1995

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FINAL REPORT
GEOTECHNICAL AND ENGINEERING STUDIES FOR THE NEW LOS PADRES WATER SUPPLY PROJECT
MONTEREY PENINSULA WATER MANAGEMENT DISTRICT

94-1198801.80
March 16, 1995

Portions of Sections 2 through 6, 13, Appendix A and Appendix E of this report have been prepared by the staff of William Cotton and Associates, Inc. under the professional supervision of the Principal and senior staff whose seals and signatures appear hereon.

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EXECUTIVE SUMMARY

Geotechnical and engineering studies for the New Los Padres Water Supply Project were performed by the MARK Team in accordance with a contract with the Monterey Peninsula Water Management District dated September 27, 1994. The MARK Team included The MARK Group, Inc., Morrison Knudsen Corporation, William Cotton and Associates and other consultants and subcontractors.

The New Los Padres Dam will be located on the Carmel River approximately 19 miles southeast of the City of Monterey as shown on Figure ES-1. The project includes a 24,000 acre-foot reservoir, a 282 foot high roller compacted concrete (RCC) gravity dam, access roads, and fish passage facilities. Investigations were performed to evaluate borrow area material, the potential for faulting of the Cachagua fault, geotechnical conditions at the dam site, and seismic design parameters. A review also was made of the preliminary design from previous studies for the dam and fish collection and screening facilities. The preliminary design of the dam was modified and cost estimates were prepared for the RCC dam and fish facilities.

Borrow area investigations indicated that there are sufficient amounts of suitable construction materials available on site, upstream of the dam within the reservoir area. The materials consist of terrace gravels, sand, and rock from borrow areas including required excavation for the dam. The location of these materials within the reservoir area significantly reduces the impact of the project on the surrounding area. Approximately 885,000 cubic yards (cy) of material are required for the project and 1.5 to 2 times that volume was proven during the investigations and additional material was identified. A RCC trial mix program was performed on representative samples from the borrow areas to provide a relationship between strength and cement content.

Geologic studies indicate that granitic rock is predominant at the dam site. Portions of the dam site are overlain with alluvial fan and terrace deposits. The granitic rock is weathered from 20 to 80 feet on the abutments. The alluvial fan, terrace gravel and weathered rock material will need to be excavated from the dam foundation to minimize seepage, reduce settlement, and to provide a stable foundation during static, flood and dynamic (earthquake loading) conditions.

The dam is located in a seismically active zone in California, and there are several potentially active faults near the site. The Tularcitos and Cachagua faults are the two faults that are located nearest to the dam site. The Tularcitos fault shows evidence of late Quaternary and probable Holocene movement and is considered active. As part of this study, a regional and site specific investigation was conducted to evaluate the present level of activity of the Cachagua fault. There is compelling geologic and geomorphic evidence that the Cachagua fault has not experienced fault movement since at least the late Pleistocene (85,000 to 213,500 years ago). Based on the findings of this investigation, the Cachagua fault is considered not active, and therefore, the Cachagua fault was not included in the evaluation of the maximum credible earthquake (MCE). The Tularcitos fault is considered the controlling fault for the design earthquake.

Design earthquake properties were estimated through consideration of potentially active faults in the region and a dynamic analysis was performed on the dam to evaluate the required RCC strength and cement contents. Modifications were made to the preliminary design of the dam based on the topography and available geotechnical information. Modifications included providing a straight alignment for the dam to take advantage of dam site topography and improved foundation conditions; and, a vertical intake instead of a sloping intake. A seven phase construction sequence was prepared to show development of the borrow areas upstream of the dam and to provide information for preparation of the cost estimate of the dam. Major quantities of the dam and access roads were calculated for the cost estimates. Construction of the project is envisioned for two construction seasons. An important feature of the construction is that the primary construction staging area is located between the existing Los Padres Dam and the New Los Padres Dam site.

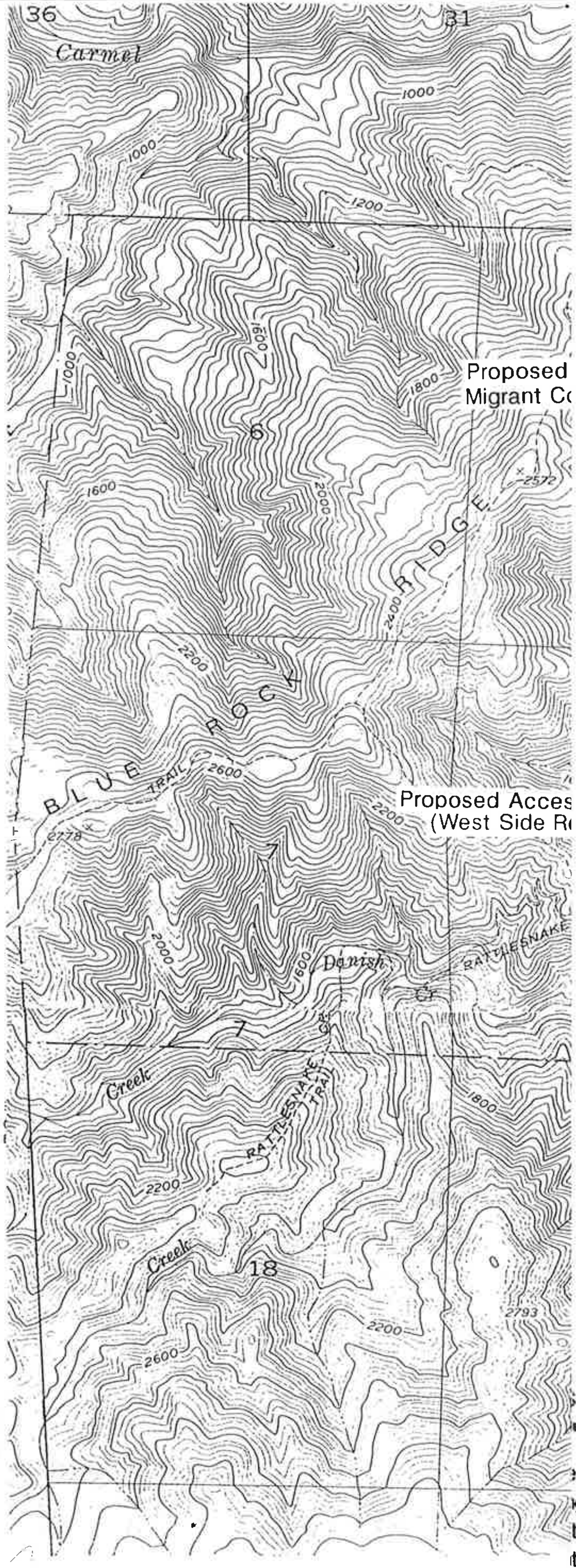
Based on the information obtained in these and previous studies, a 282 foot high RCC dam can be constructed at the site. The cost of the dam and fish facilities with engineering and construction contingencies is estimated to be approximately $81,720,000 in January 1995 dollars. Of this total, the estimated construction cost of the dam alone is $57,369,000. These costs include a 20 percent contingency to allow for unforeseen items and uncertainties inherent in the preliminary level of design.

Additional investigations must be performed for final design. These investigations should be performed to evaluate the geologic and foundation conditions at the dam site and the borrow area materials and RCC construction requirements. Similarly, additional investigations are required for the fish facilities, access roads, and reservoir slope stability. RCC aggregate testing and mix design should be performed conjunctively with the borrow area materials. A two dimensional, dynamic analysis of both static, flood, and seismic conditions is required to evaluate the stresses in the dam and rock foundations. Hydraulic model tests should be performed on the spillway to establish the final configuration of the spillway and stilling basin.

Prepared By JMH

Approved By KBK

Date 2/2/9



Base Maps: U.S.G.S. Carmel Valley (1984) and Ventana C

EXPLANATION

▩ Approximate Location
- New Los Padres Dam

▨ Approximate Limit of
New Los Padres Reservoir

〜 Approximate Alignment
of Access Roads

⊙ Approximate Location of
Migrant Collection Facilities

N

SCALE

0 2000

1"=2000'

ITE MAP

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ter Supply Project
ty, California

PROJECT NO.
94-1198801.80

FIGURE NO.
ES-1

1.0 INTRODUCTION

1.1 General

The MARK Group Team (MARK) is pleased to present this final report to the Monterey Peninsula Water Management District (MPWMD), completing the scope of work outlined under our contract titled "Geotechnical and Engineering Services related to the New Los Padres Dam". The overall scope of work under this contract included:

- Review and evaluation of the Bechtel June 1989 Preliminary Design and Cost Estimate and June 1992 Geotechnical Report for the New Los Padres Dam;
- Integrated exploration programs for the location of the Cachagua fault, aggregate borrow areas dam foundation and fish facilities which included core borings, exploratory trenches, geologic mapping and seismic refraction surveys;
- Detailed geologic mapping and excavation of an exploratory trench along the Cachagua fault;
- Estimation of usable material and testing of samples from borrow areas for use in a roller compacted concrete (RCC) dam and conducting a RCC trial mix program;
- Evaluation of the seismic design criteria with necessary revisions based upon updated seismic hazard characterization;
- Installation of two monitoring wells in Borrow Area A in accordance with the County of Monterey and MPWMD requirements;
- Review and evaluation of the preliminary design and cost estimates presented in the 1989 Bechtel report based on the information obtained from this study and other factors which may effect the cost estimate;
- Update the cost estimate to January 1995 levels; and,
- Prepare this report.

1.2 Authorization

On September 19, 1994, the Board of Directors of MPWMD authorized a contract for geotechnical and engineering services based on MARK's proposal dated July 22, 1994 with a revised scope of work. Written notice to proceed was given to MARK on September 27, 1994. The contract was later amended by the Board of Directors on December 19, 1994.

This amendment expanded the scope of the original contract to modify the drilling in borrow areas, provide for additional studies for the Cachagua fault and to perform additional RCC trial mix testing.

1.3 Project Background

The New Los Padres Dam is proposed to be located on the Carmel River approximately 19 miles southeast of the City of Monterey and 7 miles southeast of Carmel Valley Village (Drawing 1-1). The site is located approximately 2,400 feet downstream of the existing Los Padres Dam (Drawing 1-2). The project will include a 24,000 acre-foot reservoir and a 282-foot high roller compacted concrete (RCC) gravity dam. MPWMD has completed operation studies, and environmental impact assessments, and a significant number of preliminary geotechnical and engineering design studies for the project.

Studies were performed for the New Los Padres, New San Clemente and San Clemente Creek projects in 1989 by Bechtel (Bechtel, 1989). Preliminary designs were developed and cost estimates were prepared. In 1991 Bechtel prepared conceptual design and cost estimates for the fish collection facilities as part of the New Los Padres and New San Clemente projects (Bechtel, 1991). Geotechnical studies for the New Los Padres Dam were performed by Bechtel in 1992 (Bechtel, 1992).

The purpose for the present studies was to refine the design assumptions and the estimated cost of the project so that the project can be presented to the MPWMD Board of Directors and community for approval.

The present studies were undertaken to (1) refine available information on dam foundation materials and conditions; (2) further explore, evaluate, and define quantities of borrow materials available in the project area that are suitable for use as RCC and conventional concrete aggregate; (3) evaluate the seismicity of the dam site, including evaluation of the Cachagua fault zone; (4) review Bechtel Corp's project design and cost estimates; and (5) update and escalate previous project cost estimates to current price levels.

1.4 Acknowledgements

The MARK Group Team and their respective areas of responsibility for Geotechnical and Engineering Services related to these studies was comprised as follows:

- The MARK Group, Inc. - Project Management, Aggregate Source Investigations, Seismic Design Parameter Evaluation.
- Morrison Knudsen Corporation - RCC Trial Mix Design, Aggregate Source Evaluation, Design Review and Cost Estimate Update
- William Cotton and Associates - Geologic Mapping, Seismic Surveys and Fault Investigation
- Greensfelder and Associates - Seismic Hazard Evaluation
- Alan L. O'Neill - Engineering Geology Consultant

with support services provided by:

- Testing Engineers Inc. - RCC trial mix design batching and testing.
- Granite Construction Company - Aggregate crushing for RCC trial mixes.
- Bestor Engineering - Surveying services.

We appreciate the excellent cooperation provided to us by the staff of MPWMD including:

Mr. James R. Cofer - General Manager
Mr. Andrew M. Bell - District Engineer
Mr. Joseph W. Oliver - Senior Hydrogeologist

1.5 Report Organization

The report is organized as follows:

- Section 1.0 - Introduction
- Section 2.0 - Physical and Geologic Setting
- Section 3.0 - Borrow Area and Dam Foundation Field Investigation
- Section 4.0 - Site Conditions
- Section 5.0 - Design Earthquake Characteristics
- Section 6.0 - RCC Materials Testing
- Section 7.0 - Geotechnical Engineering Evaluation
- Section 8.0 - Preliminary Analysis of RCC Dam
- Section 9.0 - Review of Preliminary Design
- Section 10.0 - Construction Planning
- Section 11.0 - Cost Estimate
- Section 12.0 - Conclusions and Recommendations
- Section 13.0 - References

Sections 2.0 through 4.0 present the results of the geologic/geotechnical investigations performed in the project areas. Seismic refraction surveys are documented in Appendix A and the Cachagua fault investigation is documented in Appendix E.

Sections 5.0 through 8.0 present the engineering evaluation used as a basis for review of the preliminary design.

Sections 9.0 and 10.0 present the review of the preliminary design and the phases of construction anticipated for construction of the dam.

Section 11.0 presents the cost estimates for construction of the dam and fish facilities.

Section 12.0 presents conclusions and recommendations as a result of these studies.

1.6 Limitations

The services provided under this contract as described herein include the professional opinion and judgements on the information and data collected and reviewed. This report has been prepared for the exclusive use of the Monterey Peninsula Water Management District for geotechnical and engineering studies of the New Los Padres Water Supply Project. Additional studies must be performed for final design and construction of the project.

The conclusions and recommendations presented in this report are based upon the information developed during these studies, information contained in other consultants reports and information provided by MPWMD.

The conclusions and recommendations included herein may no longer apply if:

- The recommendations for additional studies are not implemented;
- There are significant changes in material or labor costs; and
- There is a significant change in the location of the site.

2.0 PHYSICAL AND GEOLOGIC SETTING

2.1 Terrain

The New Los Padres Dam site is located on the upper Carmel River in the northern Santa Lucia Range, approximately 24 miles upstream from the river mouth (Drawing 1-1). This rugged mountainous region, the westernmost of several ranges forming the southern Coast Ranges physiographic province, extends from the Monterey Bay southeastward for approximately 125 miles, and is bounded on the southwest and northeast by the Pacific Ocean and Salinas Valley, respectively. The topography of the region is characterized by high, narrow ridges, steep-sided hillsides, and incised drainages. Elevations in the northern portion of the range vary from approximately 800 feet at the confluence of the Carmel River and Cachagua Creek (approximately 0.7 miles downstream of the dam site) to nearly 4,800 feet at several peaks located approximately 4 to 6 miles southwest of the existing reservoir. The crests of Blue Rock Ridge and Hennicksons Ridge, located immediately west and southeast of the proposed dam site, respectively, are at elevations of approximately 2,600 feet (Drawing 2-1). Slope gradients in the region are typically in the range of 45 to 90 percent (20 to 40 degrees), but locally vary from gently sloping on the surface of elevated stream terraces to near-vertical along the banks of incised canyons.

The Carmel River flows in a generally northwest direction from its headwaters in the upland area of the northern Santa Lucia Range to the coast at Carmel Bay. From the river mouth to the upper end of Carmel Valley, a distance of 15 miles, the lower Carmel River is characterized as a meandering channel cutting a relatively wide (roughly 0.5 miles), well-developed floodplain. In contrast, the upper portion of the river system is characterized by steep, narrow canyons with relatively high stream gradients, high sinuosity and a lack of extensive floodplains.

Individual river segments in the upper portion of the river locally change direction abruptly between north- and west-flowing courses. In the vicinity of the proposed dam and existing reservoir (i.e., from approximately river mile 28 to 23), the river flows north-northeastward. Approximately 4,000 feet downstream from the proposed dam site, the river enters Cachagua Valley, merges with Cachagua Creek, and turns abruptly to the northwest.

The river flows northwestward through Cachagua Valley for a distance of about 1 mile before it turns abruptly westward and enters a narrow gorge.

The Carmel River drains a watershed area of 45 square miles at the proposed dam site (Bechtel, 1992). The average annual precipitation in the vicinity is 27 inches; however, significant fluctuations in the long-term average have occurred historically. Nearly all of the annual precipitation falls between November and April. With average rainfall, the existing Los Padres Reservoir generally reaches maximum level by mid-December.

2.2 Regional Geology

The Santa Lucia Range is the largest of several northwest-trending mountain ranges of the crystalline basement complex known as the Salinian block. The Salinian basement complex underlies most of the southern Coast Ranges and is composed largely of granitic rocks that locally encompass pendants of metamorphic rocks. It is bounded by two major fault zones: the San Andreas fault on the northeast, and the Sur-Nacimiento fault zone, which may also be a southern branch of the San Gregorio fault, on the southwest (Drawing 2-1).

The major geologic units and structural features of the northern Santa Lucia Range are depicted on Drawing 2-2. The primary geologic units in the vicinity of the proposed dam are Mesozoic crystalline rocks, Tertiary sedimentary rocks, and Quaternary deposits. The general distribution of these geologic units with respect to the study area are shown in more detail on Drawing 2-3, and brief descriptions of the major geologic units are provided in the following sections.

2.2.1 Paleozoic and Mesozoic Crystalline Rocks

The crystalline basement rocks are both granitic (granodiorite, quartz diorite) and lesser amounts of gabbro and metamorphic. The existing Los Padres Dam and proposed dam site are located within a broad belt of metamorphic rock; however, a northwest-trending block of primarily granitic rock extends from Chews Ridge to the Monterey Peninsula (Ross, 1976). The age of the granitic rocks in the area is considered to be middle to late Cretaceous (Compton, 1966; Wiebe, 1970), while the metamorphic rocks are considered to be older (Paleozoic) than the granitic rocks that engulf them.

2.2.2 Tertiary Sedimentary Rocks

Tertiary sedimentary rocks in the vicinity of the dam site include three distinct formations (Drawing 2-3). In order of decreasing age, these formations are referred to as: Unnamed Redbeds (Trb), Marine Sandstone (Tts), and Monterey Formation (Tm). The Redbeds overlie the granitic complex and are overlaid statigraphically by the Marine Sandstone. The Marine unit is, in turn, capped by the Monterey. At one time, these Tertiary deposits were widespread in the northern Santa Lucia Range, but they have since been exhumed due to uplift and resulting erosion, thus leaving behind remnant outcrops primarily along down-faulted blocks.

The primary Tertiary unit of interest to this study is the Marine Sandstone Formation, which is exposed north of the proposed dam site along both the southern and northern margins of Cachagua Valley. This sandstone formation is in fault contact with the basement rocks along the Cachagua fault, which strikes northwestward through the northern portion of the study area (Drawing 2-3).

2.2.3 Quaternary Deposits

Quaternary deposits in the vicinity are unconsolidated stream terraces and alluvial fans that locally cover the crystalline basement and Tertiary sedimentary rocks. The stream deposits, consisting of coarse gravel, sand and silt, include modern fluvial deposits located in the present river channel, as well as elevated terraces that were deposited as the ancestral Carmel River carved a channel through the mountainside. Several levels of terrace deposits are present in the study area, and each level represents the position of the river at different time intervals in the geologic past. Alluvial fans emanate from tributary drainage ravines, and represent the accumulation of episodic pulses of slopewash and debris flow deposition. The fans form a wedge of slope debris that cover the flat-lying terrace deposits.

2.3 Seismotectonic Setting

2.3.1 Regional Geologic Structure

The southern Coast Ranges province is a region of active tectonism associated with movement of the Pacific Plate, on the southwest, relative to the North American Plate, on the northeast, (Drawing 2-1). The boundary between these two tectonic plates is the San Andreas fault; however, tectonic movement is not limited to just the San Andreas fault and immediate vicinity. Rather, plate motion and associated deformation occur over a broad region, and are reflected by an abundance of faults and structural disturbance throughout the southern Coast Ranges.

In a regional sense, the Salinian block generally is considered to behave as a rigid tectonic block, with northwest transfer of the block being accommodated primarily along the northeastern and southwestern block boundaries (Dibblee, 1976; Clark and others, 1994). However, the Salinian block can be divided into seismotectonic domains, or sub-regions, on the basis of physiography and geologic structure. The proposed dam site is situated in the northern Santa Lucia Range domain, which is the most intensely deformed region of the Salinian block. In comparison to surrounding domains, this region is characterized by higher elevations, greater structural complexity, and an abundance of northwest-trending, "intra-Salinian" faults.

The elevated, northern portion of the Santa Lucia Range is the result of compressional deformation due to the transfer of right-lateral movement from the Rinconada-Reliz fault zone to the San Gregorio-Hosgri fault zone across a left (i.e., toward the southwest) stepover (Dibblee, 1976). The result of this compression is manifested by the presence of intra-Salinian faults in the northern Santa Lucia Range. Most of these faults are northwest-trending, steeply dipping reverse faults that have disrupted the Tertiary rock record and elevated the mountain range. Quaternary activity is difficult to adequately assess for many of the intra-Salinian faults because of the relative lack of Quaternary deposits in the rugged interior of the northern Santa Lucia Range, and general lack of detailed study. Traditionally, geologists have considered the intra-Salinian faults incapable of generating significant earthquakes because these faults were formed under an older, compressional stress regime that has now been overshadowed by right-lateral transform movement (Dibblee, 1976; Ross, 1976;). However, data revealed by closer examination of this region indicate that certain intra-Salinian faults (e.g., Tularcitos, Navy and Chupines) may have experienced at least some amount of Quaternary movement (Clark, written comm., 1994; Clark and others, 1974). The type of deformation associated with the most recent movement along these faults is probably reverse-oblique (i.e., a combination of compression and right-lateral strike-slip movement).

2.3.2 Seismogenic Potential of Nearby Faults

The proposed dam will be susceptible to earthquake shaking from several different sources. Historical seismicity in the region is concentrated along the major boundary faults, and is relatively sparse within the interior of the Salinian block (Rosenberg, 1993; Cockerham and others, 1990; Clark and others, 1994). The faults forming the margins of the Salinian block, the San Andreas fault zone and the Sur-Nacimiento/San Gregorio fault zone, clearly are capable of generating relatively frequent, moderate to large earthquakes. In addition, some of the intra-Salinian faults appear to have a potential to generate significant earthquakes. Most notable among these intra-Salinian faults is the Rinconada-Reliz-King City fault zone, which appears to form a structural boundary between the northern Santa Lucia Range and the Salinas Valley-Gabilan Range to the northeast. Intra-Salinian faults in proximity to the proposed dam site are the Cachagua, Tularcitos, Blue Rock, Miller Creek and Chupines faults. Although these local faults are primarily reverse faults, fault plane solutions of several historical micro-earthquakes reveal reverse-oblique and strike-slip motions, indicating that the faults in the region are reacting to the transpressional stress regime associated with right-lateral plate motion. These faults are structurally complex, and are typically shown to be imbricated, braided and segmented on available geologic maps of the region. Distances to significant faults from the proposed dam axis are shown in Table 2-1.

The seismogenic potential of significant nearby faults was previously evaluated by Geomatrix (1985) and Bechtel (1988) in order to assess the Maximum Credible Earthquake (MCE) for seismic design considerations. The conclusions of these reports, and other previous work, are that the Tularcitos and Cachagua faults are the most significant faults in terms of seismic design because of their earthquake potential and proximity to the New Los Padres Dam site. However, as discussed in Appendix E of this report, there is compelling geologic evidence from this investigation that the Cachagua fault has not experienced significant movement in the past several tens to several hundred thousand years. Therefore, the Cachagua fault is no longer considered to pose a significant earthquake potential, and the Tularcitos fault is considered to be the most significant seismogenic source for the proposed dam. Descriptions of the Tularcitos and Cachagua fault zones are presented in the following sections. Evaluation of seismic design parameters is presented in Section 5.0.

2.3.2.1 Tularcitos Fault Zone - Indications of late Quaternary movement along the Tularcitos fault zone include: (1) youthful geomorphic expression along individual fault traces in the Carmel Valley area (McKittrick, 1987; Clark J., pers. comm., 1994), (2) offset stream terrace deposits and colluvium of probable late Quaternary age, and (3) possible connection to the Monterey Bay fault zone (MBFZ), which appears to have been active in the past 11,000 years (Greene and others, 1973). Recently, a sample of charcoal from colluvium displaced by the Tularcitos fault was radiometrically dated to be approximately 7,940 to 7,620 years old (Clark, J., oral comm., 1994). Thus, there is increasing evidence that the Tularcitos has experienced movement during Holocene time (i.e., less than 11,000 years ago). In addition, plots of earthquake epicenters suggest a microseismicity pattern roughly aligned along the Tularcitos-Navy-MBFZ trend (Cockerham and others, 1990).

Assessment of MCE for seismic design requires an estimate of fault rupture length and rupture area, both of which are based on total fault length. The total length of the Tularcitos fault zone is difficult to assess, not only because of the discontinuous, imbricate nature of individual segments within the fault zone, but also because of the uncertainty associated with connection of the Tularcitos to faults mapped to the northwest and southeast. In general, the Navy and MBF zones, located to the northwest, have been included as part of the Tularcitos fault zone, but the short, discontinuous fault segments located east of approximately longitude 121°30' have not been included for reasons discussed in the following sections.

Table 2-2, at the end of this section, lists the measured lengths of the Tularcitos fault according to previous sources. The measured lengths of the Cachagua fault are included for comparison. The two primary segments of the Tularcitos fault zone are discussed below:

Navy-MBFZ

Although some previous workers consider the Navy fault to be independent of the Tularcitos, there is no clear evidence that these faults are not connected beneath Carmel Valley. Published maps do not portray any geologic structures that clearly cross the MBFZ-Navy-Tularcitos fault zone, and recent work (Clark, J., written comm., 1994) can be used to strengthen the argument that the Navy and Tularcitos faults may be connected. Furthermore, the northwest-trending pattern of microseismicity along the general alignment of the MBFZ-Navy-Tularcitos fault zone suggests some interconnection between these faults. Thus, in the absence of data clearly demonstrating that the faults are not connected, and based on similar trend and sense of offset between the two faults, the Navy fault and Tularcitos fault are considered to be the same fault zone for the purpose of seismic hazard analysis. In addition, the Navy fault has been considered to be an extension of the Monterey Bay fault zone for similar reasons.

Some workers have portrayed the Tularcitos fault as a buried fault (concealed beneath Carmel Valley) that extends to the coastline and to Cypress Point (Bowen, 1965 and 1969). To make that fault connection requires several abrupt bends in fault orientation, and supposedly includes a fault with an opposite sense of movement (Greene and others, 1973). Thus, this connection does not appear to be as likely as the Navy-MBFZ connection.

Southeast Extension of the Tularcitos Fault Zone

A series of discontinuous faults located southeast of the recognized Tularcitos fault zone have been mapped by Durham (1974) as connecting the Tularcitos with the Rinconada-Reliz-King City fault zone. However, this connection is considered to be speculative because of the cross-cutting nature of these faults, and change in general fault orientation from northwest to west. The complexity and change in character associated with this zone of faults has been interpreted to reflect a different structural pattern that probably formed under a previous stress regime when compared to the Tularcitos fault.

2.3.2.2 Cachagua Fault - The Cachagua fault is represented by a complex zone of steep, southwest dipping reverse faults that mark the southwestern edge of the Cachagua Valley (Drawing 2-3). It can be traced to the northwest of the site for nearly 9 miles, where it is truncated by a northeast-trending sequence of folded Tertiary sedimentary rocks. To the southeast of the Cachagua Valley, the fault is represented by two fault traces approximately 1,000 feet apart. To the southeast of the Carmel River, as shown on Drawing 2-3, the southern fault strand places granitic and metasedimentary rocks (hgc) against granodiorite rocks (gdh) and an overlying sequence of Quaternary terrace material (Qoa), while the northern strand juxtaposes granodiorite rock (gdh) against middle Miocene marine sandstone (Tts). These two fault strands merge southeast of the Cachagua Valley and continue as a single trace to the southeast for another 2 to 4 miles. To the northwest of the Carmel River, the northern strand merges with the single trace of the Cachagua fault that traverses the base of the steep mountain front that defines the south side of the valley. The total structural length of the Cachagua fault, as measured by various sources, are shown on Table 2-2.

TABLE 2-1: DISTANCE TO FAULTS
New Los Padres Water Supply Project
Monterey, California

Fault Name	Fault Type	Distance (miles/kilometers) and Direction
Cachagua	Reverse-oblique	0.2/0.3 N
Blue Rock[1]	Reverse	2.0/3.2 SW
Miller Creek[1]	Reverse	2.5/4.0 SE
Tularcitos[2]	Reverse/Strike-slip	2.5/4.0 N
Palo Colorado[3]	Reverse-oblique	6.0/13.0 SW
Chupines	Reverse	6.5/10.5 N
Sur-Nacimento[1]	Reverse oblique	11.0/17.7 SW
Rinconada-Reliz-King City	Strike-slip	11.0/17.7 NE
San Andreas	Strike-slip	29.0/46.7 NE

---

[1] The activity and seismogenic potential of the Blue Rock and Miller Creek faults are not known; however, Buchanan-Banks and others (1978) indicate that the Blue Rock fault has not been active in Quaternary time. The earthquake potential of both faults are overshadowed by the longer Tularcitos fault zone.

[2] The Tularcitos fault is a reverse fault; however, recent motion may be strike-slip, as expressed by fault plane solutions and geomorphic lineations (Clark, J., pers. comm., 1994).

[3] Although thrust and reverse motions are expressed geologically, both the Sur-Nacimiento and Palo Colorado fault zones may be extensions of the longer San Gregorio-Hosgri fault zone, which is considered to be strike-slip.

TABLE 2-2: FAULT LENGTHS FOR TULARCITOS AND CACHAGUA FAULTS FROM VARIOUS SOURCES (km)

New Los Padres Dam
Monterey, California

	A Fiedler (1944)	B Dibblee (1972); Dibblee and Clark (1973)	C Greene, et al (1973)	D Buchanan-Banks, et al (1978)	E RJA (1984)	F Bryant (CDMG) (1985)	G Geomatrix (1985)	H McKittrick (1987)	I Bechtel Civil (1988)
Tularcitos[1]									
Tularcitos	22.4	>33.1[2]	24.0	35.0	42.0	>28.7[4]	22[5]	29.7 - 33.4[6]	39.7[7]
Navy-MBFZ	--	16.8	--	27.0	(see Note 3)	10.1 (NF))	(see Note 5)	9.5 (NF)	0.0
TOTAL TFZ:	22.4	>49.9	24.0	62.0	42.0	>38.8	22 - 42	39.2 - 42.9	39.7
Cachagua	12.8	24.3	9.7[2]	20.0	26.0	20.8	12[3]	(not mapped)	23.4[7]

Notes:

1. Attempts to determine a fault length for the Tularcitos fault are complicated by uncertain connections between individual, discontinuous fault traces within the Tularcitos fault zone, and postulated connections to faults mapped to the west-northwest (WNW) and southeast (SE). In addition, some sources have connected the fault to a complex set of faults to the southeast (for example, as mapped by Durham, 1974); however, we discount this connection for reasons cited in the text.

   Tularcitos: Length of the fault segment from the Carmel River (northwest) to about latitude 120°30' (southeast), according to the respective source.

   Navy-MBFZ: Length of the Navy and MBFZ fault zone segment which, according to some sources, may be an extension of the Tularcitos fault zone. Connection of the Navy fault to at least a portion of the MBFZ of Greene, et al (1973) may be made based on similar fault orientation and indications of Holocene activity (historical seismicity and seafloor offset). Consequently, if the Navy fault is an extension of the Tularcitos fault, then the total length of the fault zone must include at least part of the MBFZ.

   Total TFZ: This is the total length of the Tularcitos fault zone that we consider for seismic hazard analysis, and is the sum of the Tularcitos and Navy-MBFZ segments.

2. The Cachagua fault is shown incorrectly on Source C - the actual length should be identical to Source B.

3. Rogers Johnson & Associates (1984) provide a maximum length of 103 km for the Tularcitos (connecting the San Gregorio-Hosgri and Rinconada-Reliz fault zones); and a maximum length of 94 km for the Cachagua (connecting the San Gregorio-Hosgri and Rinconada-Reliz fault zones).

4. Southeastern portion of the Tularcitos fault is not defined, because these sources (B and F) do not portray area east of latitude 120°30' (Jamesburg 15' or Rana Creek 7.5'). Dibblee (1972) is a geologic map of the Jamesburg 15' quadrangle, and Dibblee and Clark (1973) is a geologic map of the Monterey 15' quadrangle.

5. Fault length measurements do not match the sources cited in this report. For the Tularcitos fault, Geomatrix (1985) states that the Tularcitos "proper" is 22 km and, with connections to the WNW and SE, the combined total with all possible connections is 42 km. Geomatrix refers to Dibblee et al (1974), which is a collection of individual 15' quadrangles, including the Monterey and Jamesburg 15' quadrangles (i.e., source B in Table 1). Using these maps, we measure a minimum length of 34.4 km for the main Tularcitos, and a length of 51.2 km when connecting the Tularcitos to the Navy-MBFZ. Geomatrix includes the Buchanan-Banks, et al (1978) map as their fault figure, and we measure lengths of 37.0 to 70.5 km on this map (see source D in Table 1).

   For the Cachagua fault, Geomatrix gives a length of 12 km; however, their source (Dibblee, et al 1974) shows a length of 24.3 km.

6. McKittrick's map does not show a continuous fault trace northwest of Tularcitos Ridge. Rather, several en echelon zones can be interpreted from the discontinuous traces in this area, and these zones are included in the lengths provided. The Del Monte fault does not appear to connect with the Navy fault (length of Del Monte fault zone is 3.2 km on the Seaside 7.5' quadrangle). The Berwick Canyon fault may connect to the Tularcitos, but must make some dramatic turns to connect with the Navy fault (length of Berwick Canyon fault is 4.9 km on the Seaside 7.5' quadrangle). A buried and queried extension of the Tularcitos fault may extend to the Navy fault under Carmel River Valley (length of buried fault zone is 6.6 km on the Seaside 7.5' quadrangle).

   The Navy fault zone is shown to be 9.5 km long on McKittrick (1987).

7. Bechtel Civil (1988) recognizes the measurement errors in the Geomatrix (1985) report, and considers the Navy and Cypress Point faults to be "independent of the Tularcitos fault." The southeastern extension is considered "speculative".

3.0 BORROW AREA AND DAM FOUNDATION FIELD INVESTIGATIONS

3.1 General

An exploration plan was developed to evaluate and document the following:

- The quantity and suitability of terrace deposits for use as RCC aggregate;
- Evaluate bedrock conditions below borrow area terrace deposits for potential use as RCC aggregate; and
- Foundation conditions at the dam site.

The exploration program consisted of detailed engineering geologic mapping, advancing seven core borings through the borrow area terrace deposits, excavation of 19 trenches, and completing 22 seismic refraction surveys (Appendix A). One core boring was drilled and three seismic refraction surveys were performed at the right abutment of the dam to investigate foundation conditions. Water pressure tests were performed in the core boring to evaluate foundation leakage potential. Two seismic refraction surveys were performed at the site of the proposed upstream migrant fish collection facilities.

Table 3-1 summarizes the boring and trench exploration program. Table 3-2 cross references the exploration locations with specific locations (i.e., borrow area and dam site). The locations of borrow areas are shown on Drawing 3-1.

3.2 Borrow Areas

3.2.1 Core Drilling

Core drilling was conducted in Borrow Areas A and G to estimate the thickness of the terrace gravel deposits and lithology of the underlying bedrock (granitic or metasedimentary rock). The boreholes were advanced at least 10 feet into bedrock to evaluate rock quality. While it was difficult to recover samples of the terrace gravel deposits overlying the bedrock, samples recovered when coring through boulders were geologically logged.

Boreholes B-1 through B-4 and B-7 were drilled in Borrow Area A and boreholes B-5 and B-6 were cored in Borrow Area G. The depths cored, the surveyed locations and ground surface elevations are shown in Table 3-1. Borehole locations are illustrated in Drawing 4-1.

Coring in the Borrow Areas A and G began on October 25, 1994 and was completed on November 4, 1994. Boreholes were drilled using rotary-wash methods with a truck-mounted Mobile B-53 drill rig. A five-foot long double tube NX core barrel with a split inner tube was used for coring. Drilling fluid was generally clean water obtained from the Carmel River; however, bentonite clay was used in borehole B-5 to seal a zone of water loss. Due to the very rocky nature of the overburden at most of the coring locations, coring commenced from the surface using a NX diamond bit. To maintain water in the advancing corehole, 3.5 inch casing was advanced in most cases to the terrace/bedrock contact. Core samples were stored in wooden core boxes with wooden blocks to separate core runs. Core samples will be stored at the MPWMD facility in Carmel Valley. Photographs of the recovered cores were taken and have been provided to MPWMD. Geologic Boring logs are included in Appendix B.

Upon completion of coring, observation wells were installed in Boreholes B-1 and B-7, while the remaining boreholes were sealed with cement-bentonite grout. Observation well design was in concurrence with Monterey County and MPWMD requirements at locations requested by the MPWMD. Well construction logs are presented in Appendix B. Installation of these wells were requested by MPWMD. However, water levels were not monitored during this investigation.

3.2.2 Trenching

Trenching was conducted to evaluate the vertical extent of the alluvial fan and terrace deposits, as well as the lithology (granitic or metasedimentary rock) of bedrock beneath the terrace deposits in Borrow Areas A, B, C, G, and H. Depths trenched, surveyed locations and ground surface elevations are shown in Table 3-1. Trenches excavated in specific borrow areas are listed in Table 3-2. Trench locations are illustrated in Drawing 4-1.

Trenching was performed in two phases. The first phase was conducted on October 20 and 21, 1994 and the second phase was conducted on November 18 and 19, 1994. Trenches were excavated and backfilled on the first day using a John Deere 710B backhoe. On all following days a Caterpillar 200B excavator was used. The change to an excavator was necessary due to the large boulders encountered in the trenches. A Caterpillar D-6 dozer was used during the second phase of trenching to backfill trenches, clean up work areas, and for regrading an old access road into Borrow Area C. At the close of trenching, the ground surface in areas of potential erosion was covered with straw.

Trenches were excavated to bedrock, the maximum reach of the equipment, or refusal. Trench depths ranged from 3.5 to 18 feet deep. The trenches were logged, photographed, and bulk samples of the fan and terrace deposits were collected from selected trenches. Approximately 200 pounds of material was collected in plastic-lined burlap bags from each trench. The samples were delivered to Testing Engineers, Incorporated (TEI) in Oakland for testing. Terrace gravel material from Borrow Area A (TP-12) and Borrow Area B (TP-9) was collected during the first phase of trenching in a 10 cubic yard dump truck. This material was delivered to the Granite Construction Company plant in San Jose for crushing, and then delivered to TEI for testing. Geologic trench logs are included in Appendix B.

Native and crushed rock samples were tested in the laboratory for grain-size analysis, specific gravity, absorption, mineral count, sodium sulfate soundness, and Los Angeles Abrasion. Results of these analyses are included in Appendix D, and the test results are discussed in Section 6.0.

3.2.3 Seismic Refraction Surveys

A seismic refraction survey was conducted to: 1) investigate subsurface conditions in the vicinity of the proposed damsite; 2) evaluate the depth to bedrock (and indications of depth of weathering and/or fracturing) as well as the rippability of materials in the vicinity of the proposed dam, borrow and quarry areas, and fish handling facilities; and 3) help constrain the location of the Cachagua fault.

A total of twenty-two (22) individual seismic refraction lines with a combined spread length of 9,100 feet were recorded in the general area of the proposed dam. These seismic refraction lines were recorded from September 28, to November 18, 1994 in the locations shown on Drawing 4-1. A detailed report of the seismic refraction survey and the interpreted results are presented in Appendix A with Figures A-1 through A-20 (in the appendix) illustrating the results. Rippability information based on the geophysical surveys is presented on Figure A-21 (Appendix A).

Seismic refraction lines were surveyed for location and elevation using hand level, Brunton compass and measuring tape methods. Lines were marked with stakes in the field and located on the base map. Locations and relative elevations should be considered approximate.

3.3 Dam Site and Appurtenant Structures

3.3.1 Core Drilling

One boring (RA-2) drilled was conducted in the right dam abutment to estimate the thickness and lateral extent of the overburden deposits (alluvial fan and terrace), as well as the quality of the granitic rock beneath the overburden. Cuttings from the terrace gravel deposits were geologically logged and samples were also recovered when coring through boulders. Borehole RA-2 was advanced approximately 25 feet into the bedrock and two water pressure tests were conducted. RA-2 was drilled at the location shown in Drawing 4-1. The depth cored, the surveyed location and ground surface elevation of RA-2 are shown in Table 3-1.

Coring in the right dam abutment began on November 7, 1994 and was completed on November 8, 1994. The borehole was drilled using rotary-wash methods with a truck-mounted Mobile B-53 drill rig. A five-foot long double tube NX core barrel with a split inner tube was used for coring. Drilling fluid was clean water obtained from the Carmel River. Due to the rocky nature of the overburden, coring commenced from the surface using a NX diamond bit. To maintain water in the advancing borehole, 3.5 inch casing was advanced to the alluvial fan/bedrock contact. Core samples were stored in wooden core boxes with wooden blocks to separate core runs. Photographs of the recovered core were taken and have been provided to MPWMD.

3.3.2 Water Pressure Testing

Two water pressure tests were conducted in the bedrock portion of borehole RA-2, located in the right dam abutment. The first test was conducted after the borehole had been advanced to a depth approximately 15 feet into bedrock. After the first test, the borehole was advanced an additional 10 feet and the second test was performed.

Water pressure test equipment consisted of a water pump, two-way valve, flow meter calibrated in tenths of a cubic foot, a pressure gauge with a 0-100 psi range, and a push packer. The packer was placed, and then water was pumped into the borehole at increasing and then decreasing predetermined pressure stages of 5, 10, 20, 10, and 5 psi. Once the flow rate at the initial pressure stabilized, readings from the flow meter were taken at 2 minute intervals for a period of 10 minutes at each pressure stage. Results of the water pressure test are shown on the borehole log for RA-2 (Appendix B) and in Appendix C.

TABLE 3-1
SUMMARY OF EXPLORATION
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

Excavation I.D.	Northing	Easting	Ground Surface Elevation (NGVD)	Depth to Top of Rock (ft)	Total Exploration Depth (ft)
Boreholes					
B-1	395800.1	1215456.7	965.2	25.1	44.4
B-2	396019.6	1215377.3	968.4	26.0	57.0
B-3	395575.1	1215437.8	967.9	18.0	55.2
B-4	395741.9	1215625.5	961.2	13.0	60.1
B-5	395749.7	1214721.9	1086.9	38.0	63.5
B-6	395805.1	1214902.7	1066.6	65.5	99.9
B-7	395828.9	1215140.3	992.9	46.7	62.5
RA-2	396451.8	1216248.4	1048.1	58.8	85.0
Trenches					
TP-1	394892.4	1215383.4	981.7	11.5	13
TP-2	394945.6	1215731.6	986.7	>12.5	12.5
TP-3	394971.2	1215844.3	991.9	>16.5	16.5
TP-4	395448.1	1214501.4	1097.0	>12	12
TP-5	395972.4	1215176.7	998.5	>9.5	9.5
TP-6	395904.6	1214838.3	1093.2	>8.5	8.5
TP-7	395496.2	1214659.9	1064.1	0	3.5
TP-8	394783.4	1215703.0	988.2	>18	18
TP-9	394599.0	1215520.0	991.5	14	14.5
TP-10	394454.7	1215216.0	993.5	16	17
TP-11	394469.8	1214943.7	992.2	12.5	13
TP-12	395803.9	1215422.8	966.6	>18	18
TP-13	395778.3	1214734.4	1091.5	>6	6
TP-14	395739.4	1214893.3	1057.4	>6	6

TABLE 3-1
SUMMARY OF EXPLORATION (Cont.)
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

Excavation I.D.	Northing	Easting	Ground Surface Elevation (NGVD)	Depth to Top of Rock (ft)	Total Exploration Depth (ft)
TP-15	394707.1	1215167.4	986.9	16	17
TP-16	395178.2	1215798.0	982.7	>18	18
TP-17	396074.1	1215995.5	906.0	>9	9
TP-18	396022.3	1215865.3	895.9	9	10
TP-19	396122.7	1215775.6	893.8	>18	18

TABLE 3-2
SUMMARY OF BORROW AREA EXPLORATION LOCATIONS
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pits	Borings
A	TP-5, TP-12	B-1, B-2, B-3, B-4,
B	TP-1, TP-2, TP-3, TP-8, TP-9, TP-10, TP-11, TP-15, TP-16	None
C	TP-17, TP-18. TP-19	None
G	TP-6, TP-13, TP-14	B5, B6, B-7
H	TP-4, TP-7	None
Dam Site	None	RA-2

4.0 SITE CONDITIONS

4.1 Introduction

This section presents a detailed discussion of the site conditions observed during the field investigations for the dam site, borrow areas and upstream migrant collection facilities. First, descriptions of the site topography, bedrock units, and surface deposits are presented, followed by general descriptions of each borrow area, and foundation conditions at the dam and upstream migrant collection facilities.

4.2 Topography

The topography of the New Los Padres Dam study area is dominated by an established drainage system that is incised into steep mountainous terrain (Drawing 4-1). As a result of long-term downcutting and valley widening, the hillsides have been modified into a series of steep slopes and gentle benches which resemble stair steps in cross sectional view. Elevations are highest along the mountain ridge crests that roughly border the west and east margins of the study area. The highest elevations in the study area are greater than 1,600 feet along the western ridge, and vary from 1,200 to over 1,500 feet along the eastern ridge. The hillsides flanking the ridges slope steeply downward toward the Carmel River, but are locally interrupted by moderately to gently sloping surficial deposits that overlie and conceal eroded bedrock platforms. Borrow Areas A, B, C, D, H, I, and J are situated on these gently sloping surficial deposits; whereas, Borrow Areas F, and G are situated on the steep bedrock slopes (Drawing 3-1).

The Carmel River flows generally northeastward from the existing Los Padres Dam toward Cachagua Valley, a river distance of approximately 6,200 feet. The vertical drop across this distance is about 100 feet; therefore, the average stream gradient across the project area is less than 1 degree (1.6 percent). The incised channel through most of this river section is characterized as a narrow (approximately 300 feet wide), sinuous gorge, with steep to near-vertical side slopes up to 100 feet in height. In most of the area, the river flows over bedrock. Locally, however, the river channel has widened to form small, low-lying fluvial terraces that are up to 15 feet above river level.

Tributary drainages include several moderately incised gullies and a few more prominent canyons. The largest tributaries are relatively narrow (typically less than 100 feet wide) and incised up to 75 feet. They are characterized by steep side slopes, numerous shallow slope failures, and are locally choked with slope detritus. Alluvial fans emanate from the mouths of the ravines, and fan deposits cover most of the flat-lying to gently sloping topographic benches. Some of the drainages open onto elevated benches (i.e., older stream terraces) that do not extend down to the modern river elevation. The incision rates of these drainages are less than those that have been able to keep pace with the base level of the Carmel River. The genesis and significance of the stream terrace deposits and alluvial fans are discussed more fully in Appendix E.

4.3 Bedrock Units and Surface Deposits

Primarily, as discussed in Section 2.0, geologic units in the New Los Padres Dam study area include both bedrock and overlying surficial materials. The bedrock units include the crystalline basement rocks, which underlie the proposed dam axis and borrow areas, and the Tertiary-age Marine Sandstone Formation, which is present in the northern portion of the study area. These two bedrock units locally are overlain by unconsolidated Quaternary stream terrace and alluvial fan deposits. The areal distribution and thickness of the various geologic units were derived from geologic mapping of geomorphic surfaces and rock exposures and are portrayed on the Engineering Geologic Map (Drawing 4-1) and representative Engineering Geologic Cross Sections (Drawing 4-2). Subsurface exploration (boreholes and test pits) and seismic refraction lines were used as an additional aid in assessing the extent, thickness and physical character of various geologic units. Detailed descriptions of the various geologic units are presented below.

4.3.1 Bedrock Units

Crystalline basement rocks. The primary bedrock formation underlying the proposed dam site, existing dam and proposed borrow areas is the crystalline basement complex. Regionally, this basement complex is a heterogeneous mixture of granitic and metamorphic rocks. However, for the purpose of this investigation this formation has been divided into two separate lithologic units characterized by differences in relative percentages of granitic and metamorphic rock:

Granitic rock unit (Kgr) - Granitic intrusive rock, composed primarily of porphyritic granodiorite and diorite, is the primary bedrock unit underlying the study area (Drawing

4-1). Granitic rock is exposed nearly continuously along the banks of the Carmel River canyon for a distance of roughly 0.7 miles, from approximately 1,200 feet downstream from the existing Los Padres Dam to approximately 1,400 feet downstream of the proposed dam axis, although there is a noticeable scarcity of rock exposures in several areas. To the south, the granitic rock is in contact with the metamorphic rock unit (ms). To the north, the Cachagua fault juxtaposes the granitic bedrock against Tertiary sedimentary rocks (Tts).

The granitic lithology is variable throughout the project area; however, the most prevalent outcrop exposure is a coarse-grained, porphyritic biotite-hornblende, granodiorite and diorite. Exploratory boreholes advanced during the current and previous investigations encountered a variety of granitic lithologies, including granodiorite, quartz monzonite, monzonite, quartz diorite and diorite. Gabbro also is locally exposed in the slopes above Borrow Areas B and C.

In general, the granite exposed in the river canyon contains approximately 10 percent metasedimentary bedrock which occurs as highly contorted and weathered tabular and lenticular blocks. Granitic exposures vary from small blocks protruding through a cover of slope debris to bold, pronounced outcrops that occur along the channel section of the proposed New Los Padres Dam site. The granitic rock (Drawing 4-1) exposed at river level is slightly weathered, moderately hard to hard, with closed joints spaced 6 inches to several feet apart. Three pervasive joints sets occur near river level at the proposed dam site; an approximate east-west oriented, near-vertical joint set; an approximate north-south oriented, near-vertical joint set; and a low-angle, east-dipping joint set. A gradual increase in weathering of the granitics, a decrease in joint spacing, and opening of joints occurs upslope from river level. Additionally, a pronounced exfoliation joint set (i.e., a joint set that roughly mimics topography) occurs in the granitic exposures upslope from river level. Localized toppling tends to occur along the near-vertical north-south joint set, resulting in numerous exposures of near-vertical granitic faces in the vicinity of the proposed dam site.

Metamorphic rock unit (ms) - Metamorphic rock is exposed in the southern portion of the study area, in the vicinity of Borrow Areas B, and H, and the existing dam (Drawing 4-1). This unit primarily consists of metasedimentary rock composed of quartzofeldspathic schist. Prominent outcrops of schist exposed near the existing Los Padres Dam exhibit persistent foliation oriented to the northeast with steep to near-vertical dips toward the northwest. A distinct foliation change occurs across a prominent shear zone located approximately 100 feet downstream of the existing Los Padres Dam spillway at this location, the strike of the foliation changes from northeast to northwest, with moderate to near-vertical dips changing from northwest to southwest. Highly variable and contorted foliation is characteristic within small blocks of metasedimentary rock that have been incorporated into the intruding granitic bedrock.

A distinct near-vertical, northwest-trending contact between the schist and the granitic rock to the north occurs approximately 1,200 feet downstream of the existing Los Padres Dam. However, the contact between the two basement rocks is considered to be transitional throughout much of the study area, as the percentage of metamorphic inclusions in the granitic unit increases toward the contact.

The metasedimentary rock outcrops vary from deeply to slightly weathered. The schist is most deeply weathered near the schist-granite contact, in small blocks incorporated within the granite, and in an old shear zone exposed near the spillway. Metasedimentary rocks exposed along the existing dam abutments and spillway excavation appear to be less weathered than these other areas. In general, the metasedimentary rock is much less resistant to weathering and abrasion than the granitic rock. It is estimated that less than 10 percent of the stream terrace deposits (Qt) and modern alluvium (Qal) is comprised of metasedimentary clasts, even though metamorphic rock comprises more than 50 percent of the basement complex in the drainage area.

Marine Sandstone Formation (Tts). The Miocene-age Marine Sandstone Formation is present in the northern portion of the study area (Drawing 4-1). The sandstone is exposed approximately 1,200 feet downstream of the proposed New Los Padres Dam axis along the eastern bank of the Carmel River canyon, and roughly 2,200 feet downstream from the proposed axis on the western bank. These outcrops consist of 10- to 50-foot, near-vertical exposures. This bedrock unit is characterized by moderately weathered, soft to moderately hard, thick bedded to massive, coarse-grained to conglomeratic sandstone. The lithology varies from a poorly sorted, coarse-grained arkosic sandstone to a well-sorted, fine-grained feldspathic arenite. Some of the less weathered exposures contain weak calcareous cementation. The conglomeratic interbeds are multi-lithologic, but contain primarily granitic cobbles and boulders. Stratification within the sandstone strikes to the northwest and dips moderately to the northeast with the dip increasing locally near the Cachagua fault.

4.3.2 Surficial Deposits

Stream Terrace Deposits (Qt). Coarse-grained stream deposits are exposed at various elevations along both sides of the Carmel River, having been deposited in late Quaternary time as the ancestral river eroded nearly horizontal surfaces on the underlying bedrock and deposited its stream load on these erosional surfaces. These deposits are characterized as loosely consolidated, uncemented, clast-supported, sandy gravel dominated by fine- to medium-grained sand with cobbles and abundant boulders. The terrace deposits consist of greater than 50 percent (and locally up to 90 percent) gravel- to boulder-sized clasts, which are typically subround to round, and are up to 5 feet in diameter. The large rounded clasts are composed almost entirely (greater than approximately 90 percent) of granitic rock, with a lessor amount (less than approximately 10 percent) of metamorphic clasts, most of which are a granite gneiss. Metasedimentary rock (schist) clasts are generally absent. Crude, near-horizontal stratification is visible where the terrace deposits are well exposed, and imbrication of clasts occurs within the terrace deposits. These structural features indicate that significant post-depositional deformation (i.e. fault offset, warping, etc.) of these stream terrace deposits has not occurred.

The terrace-bedrock contact is a stratigraphic nonconformity. The older, underlying bedrock unit was essentially leveled off to form a nearly horizontal surface, and younger, stream deposits were deposited over this eroded surface to form a distinct layer of stream gravel. Each terrace deposit has accumulated on a specific, eroded bedrock surface that represents a unique time interval in the geologic past. These terrace-bedrock surfaces (i.e., nonconformities) are now visible along the banks of the modern river channel and in tributary side canyons. The abrupt contrast in materials, from rounded, bouldery clasts resting on top of intact massive bedrock, make the nonconformities easy to recognize in outcrop and in borehole cores. Although generally horizontal, the terrace-bedrock nonconformities locally are irregular due to preferential erosion into zones of relatively weaker bedrock. These relatively weak zones generally correspond to highly fractured and sheared intervals and soft metamorphic inclusions in the granitic basement rocks.

In general, the best preserved terrace surfaces, and most widespread terrace deposits, occur at elevations between approximately 40 and 120 feet above the modern channel. The thickness of these terrace deposits vary between approximately 15 and 25 feet. The two most widespread terrace deposits are the third and fourth highest terraces above the modern Carmel River, and these are shown as Qt_3 and Qt_4 on Drawing 4-2.

Higher terrace deposits (Qt_5 and higher) also are present, at least to elevations of 400 feet above the modern river channel. These higher terrace surfaces are older, more deeply dissected by erosion, and have been more highly modified by subsequent alluvial fan deposition. The older terrace remnants are not as widespread, or as well preserved, as the lower terraces, and are not delineated on Drawing 4-1). The lowermost terrace deposits (Qt_1 and Qt_2), found along the lower banks of the Carmel River, are shown as Quaternary alluvium (Qal) on Drawing 4-1.

Alluvial Fan Deposits (Qf). Quaternary-age alluvial fan deposits are widespread in the study area, having accumulated at the base of steep slopes where the steep hillside gradient transitions abruptly to the flat valley floor or gentle stream terrace surfaces. Alluvial fans are composed of materials that have been eroded from the higher mountain slopes, transported downslope through hillside ravines and drainages, and are deposited onto the more gentle valley floor and terrace surfaces. They are fan-shaped accumulations of fluvial and debris flow deposits that emanate from hillside ravines and side canyons.

Fan deposits consist primarily of poorly to moderately consolidated, matrix-supported sandy silt and silty sand with angular to round cobbles and boulders. These deposits typically contain approximately 5 to 10 percent clasts. Fan deposits also locally include some rounded terrace gravels that have been eroded from upper terraces, become mixed with angular slopewash and then are re-deposited onto lower terrace surfaces. Most of the clasts are composed of angular granitic materials that are hard and moderately weathered.

The thickness of the fan deposits varies from greater than approximately 40 feet at the base of steep mountain front (at the back edge of the terrace surfaces) to just a few inches near the outer (river channel) edge of terrace deposits. Slope gradients of the alluvial fan surfaces in the project area are typically in the range of 20 degrees at the apex of the fan to 5 degrees at the downslope edge of the deposit.

4.4 Borrow Areas

Sources of construction materials in the project area include eleven potential borrow areas (areas A through K), including the proposed dam site and appurtenant facilities, migrant fish collection facilities and access roads. These borrow areas are illustrated in Drawing 3-1. The distribution and thickness of earth materials with respect to these potential sources of construction materials are summarized in Table 4-1 and depicted on the Engineering Geologic Map and Engineering Geologic Cross Sections (Drawings 4-1 and 4-2, respectively).

4.4.1 Borrow Area A

Borrow Area A is an approximate 12 acre, gently sloping area located on the left (west) side of the river roughly 1,000 feet downstream of the existing dam and 600 feet upstream from the proposed dam axis at surface Els. of 950 feet to 975 feet (Drawing 3-1). This area is characterized by a well-preserved, relatively uniform terrace deposit (Qt_3) covering granitic basement rock and locally covered by a thin wedge of alluvial fan (Qf) materials. Conditions in Borrow Area A were explored by boreholes B-1 through B-4, test pits TP-5 and TP-12, test pits T-1 through T-6 and T-9 by Bechtel (1992), and seismic refraction lines S-1 through S-5.

The terrace-bedrock contact, at approximate Els. 940 to 948 feet, is relatively horizontal. Thus, the thickness of the Qt_3 terrace deposits is uniformly about 11 to 16 feet across Borrow Area A. Alluvial fan deposits that overlie the terrace gravels vary in thickness from 30 feet at El. 1,000, adjacent to Borrow Area G, to less than a foot at the eastern margin of the borrow area near the edge of the river canyon.

The basement rock underlying Borrow Area A is composed primarily of diorite and granodiorite which is likely highly to moderately weathered to a depth of 30 to 50 feet based on surface exposures, borehole data and seismic refraction interpretation. The rock is relatively unweathered (i.e., characterized by seismic velocities greater than approximately 11,000 ft/sec) below an approximate depth of 50 feet. Metasedimentary bedrock is present in the southern corner of the borrow area, and isolated metamorphic inclusions are present locally along the 60-foot high river bank (Drawing 4-1). From geologic mapping of the river bank exposures and an analysis of borehole data, it is estimated that roughly 5 to 15 percent of the rock underlying Borrow Area A may be metamorphic.

4.4.2 Borrow Area B

Borrow Area B is a relatively flat-lying area on the right (east) side of the river located adjacent to the dam spillway, and immediately downstream of the existing Los Padres Dam (Drawing 3-1). The borrow area, which is roughly 12.6 acres in area between surface Els. 970 to 990 feet, previously was used as a borrow source for construction of the existing Los Padres Dam. Conditions in Borrow Area B were explored by test pits TP-1 through TP-3, TP-8 through TP-11, test pits T-10 and T-11 by Bechtel (1992), and seismic refraction lines S-6, S-7, S-8 and S-9.

The terrace-bedrock contact, at approximate Els. 970 to 979 feet, is relatively horizontal, and the overlying terrace deposits (Qt_3) are approximately 10 to 15 feet in thickness. Because of previous earth removal in the area for construction of the existing dam embankment, Borrow Area B contains only a relatively thin cover of alluvial fan deposits. At the northern portion of Borrow Area B, a large alluvial fan deposit (Qf) covers the entire surface and extends to the east along a tributary canyon. This fan deposit reaches a maximum thickness of approximately 40 feet at the apex of the fan to the northeast of the borrow area.

It is estimated that more than 80 percent of the bedrock underlying Borrow Area B consists of metasedimentary rock (ms) which is likely moderately to deeply weathered to a depth of 20 to 40 feet based on surface exposures, borehole data and seismic refraction interpretation. The rock is relatively unweathered (i.e., characterized by seismic velocities greater than approximately 10,000 ft/sec) below an approximate depth of 30 to 50 feet. The northern portion of the borrow area appears to be underlain by granitic rock (Kgr). However, the contact between granitic and metamorphic rock is concealed beneath terrace deposits; consequently, the relative percentage of granitic rock in the northern portion of the borrow area, can not be estimated accurately.

4.4.3 Borrow Area C

Borrow Area C is a low, relatively flat-lying area located just upstream of the proposed right abutment on the eastern side of the river (Drawing 3-1). It is approximately 5 to 15 feet above the modern river channel, and has an approximate areal extent of 3.2 acres between Els. 890 and 925 feet. Conditions in Borrow Area C were explored by test pits TP-17 through TP-19 and seismic refraction lines S-14, S-15 and S-18.

The terrace-bedrock contact is relatively horizontal. The overlying terrace gravels have an average thickness of 9 to more than 18 feet, and are characterized at the surface by relatively matrix-free, bouldery clasts from 1 to 4 feet in diameter. A wedge of fan deposits is located in the eastern portion of Borrow Area C, at the base of the steep, west-facing slope representing Borrow Area F. The Qf deposits have a maximum thickness of approximately 25 feet at the base of slope, but only extend about 50 feet into the borrow area. Thus, the bouldery terrace deposits are exposed over most of the borrow area.

Borrow Area C is underlain by granitic bedrock (Kgr), which is likely weathered to a depth of 15 to 30 feet based on seismic refraction interpretation. The rock is relatively unweathered (i.e., characterized by seismic velocities greater than approximately 13,000 ft/sec) below an approximate depth of 30 to 50 feet. Extrapolation of bedrock exposures observed along Nason Road, located roughly 200 feet east and 200 feet above Borrow Area C, and in the access road located just north of the borrow area, indicate that 10 to 20 percent of the bedrock in this area may be composed of metasedimentary rock.

4.4.4 Borrow Area D (dam foundation)

Borrow Area D is the excavation area for the proposed dam (Drawing 3-1). More detailed discussions of the borrow areas is presented in Section 4.5. In general, the abutments extend from the river channel northwestward and southeastward to approximately El. 1,125 feet and covers approximately 11.9 acres. Subsurface conditions in the right abutment area were explored by borehole RA-2, seismic refraction lines S-10, S-12 and S-13, as well as previous borehole RA-1 and previous test pits 12 and 13 by Bechtel (1992). In addition, subsurface conditions in the right abutment are exposed to a depth of approximately 100 feet along the east-west canyon wall escarpment near the southern margin of the proposed dam excavation. Subsurface conditions in the left abutment and center section were previously explored by boreholes LA-1 and C-1, respectively (Bechtel, 1992). Seismic refraction lines S-21 and S-22 were completed in the left abutment. Additionally, prominent granite outcrops exposed along the approximate 100-foot high canyon walls in the dam area, were mapped and characterized with respect to rock type, strength and structure.

4.4.5 Borrow Area E

Borrow Area E (designated Quarry A in Bechtel, 1992) is located just upstream of the proposed left abutment on the western side of the river approximately between El. 885 and 1,130 feet and covers approximately 4.0 acres (Drawing 3-1). This area is characterized by steep to very steep (40 to 80 degrees) east-facing topography dissected by several incised drainage courses. Conditions in Borrow Area E were characterized by geologic mapping of the borrow area and adjacent side canyons, and seismic refraction lines S-19 and S-20.

Borrow Area E is underlain by granitic bedrock (Kgr), which is highly weathered to a depth of 15 to 30 feet and moderately weathered from 30 to 60 feet based on seismic refraction interpretation. The rock is relatively unweathered (i.e., characterized by seismic velocities greater than approximately 14,000 ft/sec) below an approximate depth of 60 to 80 feet (vertical). Most of the area appears to be characterized by a relatively thin colluvial cover over granitic bedrock. Some of the narrow bedrock ridges contain remnants of elevated Qt deposits; however, these deposits are likely not extensive. The area between the modern river channel and approximately El. 900 feet is characterized by a thin terrace deposit overlain by coalescing Qf deposits up to 30 feet in depth. The Qf deposits contain abundant granitic gravels and boulders. The buried terrace deposit is probably correlative with the Borrow Area C terrace, but contains approximately one-third to one-half of the Borrow Area C terrace volume.

4.4.6 Borrow Area F

Borrow Area F (designated Quarry B in Bechtel 1992) is located along the steep west-facing slope above Borrow Area C and extends from approximately 100 feet upslope of Nason Road (El. 1,130) downslope to the break in slope representing the eastern margin of Borrow Area C (El. 925 feet) and covers approximately 3.4 acres (Drawing 3-1). This area is characterized by steep slopes, thin colluvial cover over granitic (Kgr) bedrock, and shallow slope failures. Shallow rock fall debris has accumulated at the base of the slope as a talus deposit.

No subsurface exploration was conducted in Borrow Area F; however, bedrock and overlying slopewash materials are well exposed in a 20- to 50-high cutslope along Nason Road and in the rocky slope below Nason Road. The physical characteristics of the granitic rock exposed in the cut slope are variable, from soft, weak and deeply weathered grus, to moderately to very hard, strong, and moderately weathered granite. Fracturing is pervasive in the deeply weathered rock, and closely to moderately fractured (0.1- to 2.0-foot spacing) in the less weathered granite.

Approximately 25 to 30 percent of the rock exposed in the cut slope is metasedimentary. The metasedimentary rock exposures are very closely fractured, highly weathered, weak and friable. Several moderately weathered pegmatite dikes extend through the metasedimentary rock.

4.4.7 Borrow Area G

Borrow Area G is located west and above Borrow Area A, is characterized by moderately steep to steep (20 to 40 percent) slopes representing the surface of an alluvial fan deposit that covers a relatively extensive terrace deposit, and covers approximately 6.9 acres (Drawing 3-1). Conditions in Borrow Area G were explored by boreholes B-5 through B-7, test pits TP-6, TP-13 and TP-14, and seismic refraction lines S-2 and S-4. The terrace-bedrock contact, at approximate El. 1,000 feet, is relatively horizontal. Thus, the thickness of the terrace deposits is about 25 feet across Borrow Area G. Fan deposits overlying the terrace gravels vary in thickness from 50 feet at El. 1,100 feet, to less than 10 feet at the eastern margin of the borrow area (approximate El. 1,000 feet).

The basement rock underlying Borrow Area G is composed primarily of diorite and granodiorite which is likely moderately to highly weathered to a depth of 30 to 70 feet based on borehole data and seismic refraction interpretation. The rock is relatively unweathered (i.e., characterized by seismic velocities greater than approximately 11,000 ft/sec) below an approximate depth of 100 feet. From surface exposures and borehole data, it is estimated that less than 10 to 20 percent of the rock underlying Borrow Area G may be metamorphic.

4.4.8 Borrow Area H

Borrow Area H covers approximately 5.7 acres located immediately south of Borrow Areas A and G, and approximately 0.3 miles upstream of the proposed dam axis (Drawing 3-1). The area slopes moderately from El. 1,040 to the normal reservoir elevation of 1,130 feet. Conditions in Borrow Area G were investigated through geologic mapping, by test pits TP-4 and TP-7, and Bechtel test pits T-7 and T-8 (Drawing 4-1).

The area is comprised of fine-grained alluvial fan deposits overlying highly weathered granitic and metasediments. This area constitutes one of the remnants of the principal sources of impervious material for the 1948-1949 construction of the existing Los Padres Dam.

4.4.9 Borrow Area I

Borrow Area I covers approximately 5.0 acres on the left (southeast) side of Los Padres Reservoir (Drawing 3-1). This area is located approximately 0.1 miles upstream of the existing dam and 0.6 miles upstream of the proposed dam. Investigation of Borrow Area I was by reconnaissance.

The area consists of an approximately 15-foot thick fluvial gravel stream terrace overlying granitic bedrock.

4.4.10 Borrow Area J

Borrow Area J covers approximately 5.1 acres immediately southwest of Borrow Area I (Drawing 3-1). Investigation of the area was by reconnaissance.

Borrow Area J is similar to Borrow Area A and is comprised of approximately 15 feet of fluvial gravel deposits and overlaid by alluvial fan deposits.

4.5 Foundation Conditions

4.5.1 Right Dam Abutment

Ground surface slope gradients in the right abutment area vary from approximately 7 degrees in the western portion above the river canyon to 15 to 35 degrees in the eastern, higher portion. The eastern (right) bank of the river canyon is approximately 100 to 110 feet in height with an average slope gradient of about 25 degrees and locally near-vertical slopes. Granitic rock is exposed along most of the river canyon below approximate El. 975 feet, but is buried by 15 to 60 feet of terrace and fan deposits between Els. 970 and 1,100 feet. Weathered granitic rock is present at or near the ground surface above El. 1,100 feet.

Bedrock exposures along the southern and western margins of the right abutment are mostly diorite and granodiorite, with less than 10 percent metasedimentary rock exposures. These predominantly granitic exposures are characterized by an east-west, near-vertical joint set and a north-south, near-vertical joint set. Rock toppling along these joint sets have yielded bold, near-vertical granitic exposures along the 100-foot high abutment.

Based on seismic refraction lines S-10, S-12, and S-13, hard, unweathered rock (characterized by high seismic velocities 7,930 to 11,100 ft/sec) is present below an approximate depth of 40 to 80 feet. The overlying 40 to 80 feet consists of terrace and fan deposits, and deeply weathered granitic and metamorphic rock.

4.5.2 Center Section of Dam

The center (river) section extends for a width of roughly 200 feet along the proposed dam axis. Bedrock exposures along the channel section consist primarily of slightly weathered, moderately hard to very hard, hornblende-mica granodiorite. The rock mass is characterized by an east-west, near-vertical joint set; a north-south, near-vertical joint set; and a N5°W to N40°W oriented, low-angle, east-dipping joint set. The spacing between the joints varies from 6 inches to 4 feet.

Localized toppling becomes common in the higher portion of the slopes in association with increased weathering and closer spacing of the joints. Toppling occurs primarily along the north-south joint set, and this has resulted in the near-vertical granitic exposures along this portion of the river.

4.5.3 Dam Left Abutment

Slope gradients in the left abutment area vary from approximately 14 degrees in the eastern portion above the river to 40 degrees in the western, higher portion. The western (left) bank of the river canyon is approximately 70 to 80 feet in height with an average slope gradient of about 25 degrees. Granitic rock is exposed along most of the river canyon below El. 935, but is buried by 10 to 35 feet of terrace and fan deposits between approximate Els. 935 and 1,000 feet. Above El. 1,000 feet, the granitic rock is buried by a relatively thin colluvial cover. Below El. 835, granitic rock is at the surface along the banks of the Carmel River.

Bedrock exposures along the lower slopes of the southern and eastern margins of the left abutment are primarily moderately hard to very hard, slightly weathered granodiorite. Several granitic rock outcrops are present above El. 1,000 feet, and these are moderately to very hard, strong, moderately to slightly weathered, and characterized by close to moderate joint spacing (0.1 to 3.0 feet apart). Angular granitic rubble at the base of these outcrops are the result of rock falls which have failed along these joints.

Based on seismic refraction lines S-21 and S-22, hard, unweathered rock (characterized by high seismic velocities of 9,850 to 14,450 ft/sec) is present below an approximate depth of 50 to 80 feet. Above this interval, from an approximate depth of 15 to 35 feet to a depth of 49 to 87 feet, the rock is more weathered and is characterized by moderate velocities (5,190 to 6,850 ft/sec). The overlying 15 to 35 feet of material consists of terrace and fan deposits, and deeply weathered bedrock.

4.5.4 Upstream Migrant Collection Facility

The upstream migrant collection facility is located along the river, approximately 700 feet downstream from the proposed dam axis. This area was investigated by engineering geologic mapping and seismic refraction lines S-16 and S-17.

The upstream migrant collection site is underlain by granitic bedrock (Kgr), which is likely moderately weathered to a depth of 20 to 40 feet based on seismic refraction interpretation. The rock is relatively unweathered (i.e., characterized by seismic velocities greater than approximately 12,900 ft/sec) below an approximate depth of 20 to 40 feet. Within the river channel, granitic rock is locally overlain by a thin (less than approximately 5 feet) layer of stream alluvium. The granitic rock is at or near the surface along the left (west) side of the river, but is overlain by a 10- to 15-foot thick layer of terrace gravels above approximate El. 920 feet. On the right (east) side of the river, the granitic rock is overlain by terrace gravels with an average thickness of approximately 10 to 15 feet above approximate 14 El. 870 feet.

TABLE 4-1: Summary of Subsurface Investigation
Monterey Peninsula Water Management District
New Los Padres Water Supply Project

Source/Borrow Area	Boring/ Test Pit No.*	Ground Surface Elevation (ft)	Fan Thickness (ft)	Top of Terrace Elevation (ft)	Terrace Thickness (ft)	Top of Bedrock Elevation (ft)
A	B-1	965.2	9.0	956.2	16.1	940.1
	B-2	968.4	12.8	955.6	13.2	942.4
	B-3	967.9	7.0	960.9	11.0	949.9
	B-4	961.2	0.0	961.2	13.0	948.2
	T-1	966.0	3.0	963.0	>12.0	unknown
	T-2	963.0	3.0	960.0	>8.0	unknown
	T-3	968.0	3.0	965.0	>9.0	unknown
	T-4	981.0	>15.0	unknown	unknown	unknown
	T-5	991.0	2.0	989.0	>9.0	unknown
	TP-5	998.5	>9.5	unknown	unknown	unknown
	T-6	923.0	3.0	920.0	>7.0	unknown
	T-9	989.0	11.0	978.0	>4.0	unknown
	TP-12	966.6	14.0	952.6	>4.0	unknown
B	TP-1	981.7	2.0	979.7	9.5	970.2
	TP-2	986.7	3.5	983.2	>9.0	unknown
	TP-3	991.9	14.5	977.4	>2.0	unknown
	TP-8	988.2	3.0	985.2	>15.0	unknown
	TP-9	991.5	0.0	991.5	14.5	977.0
	T-10	981.0	3.0	978.0	>10.0	unknown
	TP-10	993.5	0.0	993.5	16.5	977.0
	T-11	979.0	1.5	977.5	>11.5	unknown
	TP-11	992.2	0.0	992.2	12.5	979.7
	TP-16	982.7	3.5	979.2	>14.5	unknown
C	TP-17	906.0	4.0	902.0	>5.0	unknown
	TP-18	895.9	0.0	895.9	9.0	886.9
	TP-19	893.8	0.0	893.8	>18.0	unknown
D	C-1	875 Est.	8	n/e	n/e	867 Est.
	LA-1	986.0	25.0	n/e	n/e	961.0
	RA-1	1006.8	23.0	983.8	14.7	969.1
	T-12	1013.0	>15.0	unknown	unknown	unknown
	T-13	1039.0	>15.0	unknown	unknown	unknown
	RA-2	1048.1	58.8	n/a	n/a	989.3
G	B-5	1086.9	38.0	n/e	n/e	1048.9
	B-6	1066.6	44.5	1022.1	21.0	1001.1
	B-7	992.9	28.4	964.5	18.3	946.2
	TP-6	1093.2	>8.5	n/e	n/e	unknown
H	TP-4	1097.0	>12.0	n/e	n/e	unknown
	T-7	1024.0	>15.0	n/e	n/e	unknown
	T-8	1063.0	6.0	1057.0	n/e	unknown
	TP-7	1064.1	n/a	n/e	n/e	1064.1

* Boreholes and test pits from the current investigation are labeled B-__, and TP-__ respectively. The 1992 Bechtel investigation test pits are labeled T-___. n/e = not encountered.

5.0 DESIGN EARTHQUAKE CHARACTERISTICS

5.1 Overview

Design earthquake properties have been estimated through consideration of active faults located near the damsite. The design earthquake (maximum credible earthquake or MCE) is characterized in terms of its causative fault, magnitude, fault rupture mechanics, and nearest approach to the site. The MCE is the maximum earthquake that appears capable of occurring under the presently known tectonic framework. Parameters considered when deriving the MCE are:

- Local seismic settling;
- Lengths of active faults within a 100-km radius of the site;
- The style or type of faulting;
- Tectonic history; and
- Structural setting.

Site ground motion is specified in this section in terms of peak horizontal ground acceleration (PGA) and pseudo-absolute acceleration (PSAA) at 5% damping and at selected periods of vibration. In addition, the probabilities of exceedance of ground motions have been calculated principally in order to assess the recurrence interval of design ground motions. The resultant of the Cachagua fault investigation conducted during this study (Appendix E) found that the age of last fault movement was at least 85,400 to 213,500 years ago, but may have been 244,000 to 610,000 years ago or more. As such, the Cachagua fault is considered not active.

5.2 Seismogenic Potential of Faults

5.2.1 Tularcitos and Cachagua Faults

In Section 2.0, several sources of information on faults that may produce earthquakes and resulted strong ground shaking at the dam site are reviewed. The nearest faults that are important for the project are the Cachagua and Tularcitos faults. The Tularcitos fault shows evidence of late Quaternary, and probable Holocene, movement. Furthermore, it is probably connected to the Navy fault and MBFZ, which makes the resulting fault zone very significant in terms of activity, length and proximity to the proposed dam site.

As discussed in detail in Appendix E, an extensive regional and site-specific investigation was conducted as part of the current study to evaluate the present level of activity of the Cachagua fault. On the basis of this work, it has been concluded that there is compelling geologic and geomorphic evidence that the Cachagua fault has not experienced movement during the Holocene (i.e., past 11,000 years). This conclusion is based upon evidence that the fault trace does not express geomorphic features that resemble young rift topography commonly found along active faults, and that elevated Quaternary stream terrace deposits along the Carmel River that cover the fault have not been offset. Based upon rates of uplift and river downcutting, it has been estimated that these stream terrace deposits are approximately 85,400 to 213,500 years old. As noted in Appendix E, the last movement along the Cachagua fault may be much older than the terrace deposits; however, the lack of older Quaternary deposits in the area prevents better resolution of the age of faulting. Based upon the findings of this investigation, the Cachagua fault has been shown to be not active and, as such, was not included in the MCE evaluation. Therefore, the Tularcitos fault is considered the controlling fault for the design earthquake.

5.2.2 Model Fault Parameters

This section provides fault parameters for use in computing MCE ground motions, as well as for probabilistic ground motions.

5.2.2.1 Rupture Length and Style of Faulting - "Maximum coseismic fault rupture length" (Lr) is the principal determinant of MCE magnitude, under present engineering seismological practice. Estimation of Lr rests on two approximate procedures: determination of actual total (mapped) fault length (Lt) and the assumption that Lr may not exceed 0.5 Lt.

Lt is usually estimated through interpretation of published fault maps. Named or recognized faults usually appear as a series of segments which are nearly contiguous and which have very similar trends. Post-earthquake field mapping of faults has shown that such segments are often spanned by coseismic ruptures during moderate to large earthquakes (M > 6). A summary of published fault length estimates is shown on Table 2-2.

For the purpose of estimating Lt with the Tularcitos fault, the Navy fault and the MBFZ were included. Where the independent behavior of a fault segment has been established on the basis of historical ruptures, seismicity, and possibly creep, Lr is set equal to the segment length. However, where seismic behavior is poorly defined for multiple, aligned fault segment(s) believed capable of coseismic rupture, then it is standard (i.e.

conservative) practice to use Lr equal to one-half the total length of the aligned segments. Such a procedure has been employed in this analysis. Future research may indicate that this procedure should be altered, either generally or in certain instances.

"Style (type) of faulting" refers to the sense of displacement that occurs during fault rupture, which may be strike-slip, dip-slip (normal or reverse), or a combination of these two (called "oblique"). While many faults in coastal California exhibit essentially pure strike-slip movement, others show other styles. The Tularcitos fault zone appears to have had reverse-oblique displacements (reverse slip with right-lateral strike slip). This information has been utilized to compute MCE accelerations.

Table 5-1 lists total fault lengths (Lt), maximum rupture lengths (Lr), styles, nearest approach to the dam site, and other parameters, described below, for all faults treated in this chapter. This includes distant faults utilized only in the probabilistic analysis.

Although unmapped faults may exist in the project area, their potential seismogenic impact at the site cannot reasonably be greater than that of the Tularcitos fault. This fault has an MCE magnitude of 6.8, passes within 4 km of the site, and produces a site PGA of 0.51 to 0.54 g. Although an unmapped fault could lie closer to the site, the MCE magnitude would be almost certainly be less than 6.5, and therefore the potential site PGA would not exceed 0.50 g.

Reservoir-induced seismicity (RIS) may occur within a few kilometers of the site after the new reservoir is filled. RIS has been documented at many reservoir sites around the world, but there are no recorded instances of RIS magnitudes over 6.4 (this occurred at Koyna Reservoir, India, 1967). Therefore, any RIS would have a site PGA less than the Tularcitos fault.

5.2.2.2 Maximum (MCE) Magnitudes - Maximum coseismic rupture length (Lr) is the principal determinant of MCE magnitude. The term "magnitude" has been used in this analysis to refer to moment-magnitude, which has become the most commonly used magnitude scale in engineering seismology. Once Lr has been estimated, any one of several existing quantitative empirical relationships may be used to predict MCE magnitude.

The type of faulting has also been taken into account in developing some of these relationships. However, results for reverse and oblique fault types are subject to much greater uncertainty than for strike-slip, or for all types combined, because the data base for them is relatively small. Therefore, the results (linear regressions) for all faults combined have been used. The regression chosen for use is that of Wells and Coppersmith (1994), which uses a current data base. Table 5-1 lists the magnitudes predicted by this relationship.

The MCE magnitude (6.8) for the Tularcitos fault was calculated for this study using the predicted rupture length (31 km), and the fault-length vs magnitude regression for all faults (Wells and Coppersmith, 1994). This result is to be compared with MCE magnitudes of 6.4 to 6.75 estimated for this fault in previous investigations of the old and new Los Padres damsites (e.g., Geomatrix, 1985; Bechtel, 1989, 1992).

For the nearest segment of the San Andreas fault, listed in Table 5-1, the MCE magnitude of 7.0 is taken from Wesnousky (1986). This segment of the fault did not experience rupture during the two great historical earthquakes (M about 8) of 1857 and 1906, and historically no earthquakes exceeding M7 have been centered along it. Also, this segment of the San Andreas fault is subject to rapid creep (continuous aseismic slippage). These facts explain the relatively low MCE magnitude, as compared to other segments of the fault. Even if this part of the San Andreas fault was to generate an (unexpected) M 8 earthquake, the site PGA would be only 0.17 g., which is much less than that produced by the Tularcitos fault.

5.2.2.3 Attenuation Relationships - Attenuation is a term used to describe the fall-off of peak ground shaking amplitudes with increasing distance from the earthquake source as a function of magnitude. Although it is possible to compute attenuation using theoretical wave equations, the physical complexity of the earth's crust makes this quite impractical. Numerous empirical formulas (linear regressions using recorded data), have been developed and published over the past 50 years. Over the past decade, many such relationships have been developed by U.S. researchers. These formulas have been based primarily on California earthquake data, which reflects the fact that most U.S. strong-motion data originate in California. Each new large earthquake produces near-field data which appear not to be very well described by previous published relationships.

Recently published studies on attenuation relationships (or "functions") which were considered for use here include those of Idriss (1993a), Geomatrix (1992), and Boore, Joyner, and Fumal, or "BJF" (1993). All of them include results for PGA and PSAA. The Idriss and Geomatrix functions apply only to rock sites, while that of BJF is adjustable to model rock and soil sites. Trial calculations showed that the predictions of BJF were some 25 to 30 percent smaller than those of the other two authors, and in comparison to various older functions. This may be caused by the apparent paucity of near-field data for rock sites in the data base used by BJF. Thus, the BJF relationship was eliminated from further consideration.

The PGA and PSAA attenuation functions of Idriss (1993a) and Geomatrix (1992) include six or seven terms with as many empirically developed coefficients. Distinct sets of coefficients for PGA and for each PSAA period and damping are tabulated; in addition, there are separate sets of coefficients for lower and higher magnitude ranges (below or above M 6 or 6.5). The Idriss function includes a term for fault type, while Geomatrix does not. Both take the source-site distance as the closest approach of the fault rupture surface. In the case of the Tularcitos fault the dip is very steep, and the closest surface expression of the fault as mapped by Dibblee (1973) was used as the distance from the New Los Padres site.

5.3 MCE Peak Ground Motions

PGA and PSAA, at periods of 0.20 and 0.30 second and 5% damping, were calculated for the dam site for the Tularcitos fault. The attenuation relationships of Idriss (1993a), assuming oblique-slip, and that of Geomatrix (1992) were used. Both relationships are for sites on rock outcrop. Values for the median and median-plus-one standard deviation are listed in Table 5-2.

Motions based on the Tularcitos fault are used for input for other portions of this study (Sections 5.5 and 8.0).

It may be noted that the median-plus-one standard deviation PSAA values are much larger (some 50%) than the median values. This reflects the wide variation of PSAA for comparable distances and magnitudes, attributed principally to variable geologic conditions at accelerograph sites yielding data incorporated in the attenuation studies.

5.4 Probabilistic Analysis of Peak Motions

In order to gain insight into the likelihood of the MCE ground motions, probabilistic analyses of peak ground motions (PGA and PSAA for period 0.2 second and damping of 5%) were performed for a presumed exposure interval of 100 years. The method assumes a Poisson distribution of earthquakes in time, so that seismicity "cycles" cannot be considered. The standard deviation of the ground motion parameter is incorporated in the calculations.

In the present analysis, four active and potentially active faults passing within 50 km of the dam site were modeled in terms of their lengths, relative positions, maximum magnitudes, and seismicity recurrence rates. Minimum magnitude for the calculations was set at 5.0, to eliminate the effect of smaller shocks with very short duration and negligible damage potential. The four faults are listed in Table 5-1. The recurrence parameter ('A') was determined from fault slip rate ('S'). Slip rates are rather well documented for the Palo Colorado, King City, and San Andreas faults, but are poorly known for the Tularcitos fault zone, the most critical fault for the dam site. The probabilities of exceedance calculated for PGA and PSAA are shown in Figure 5-1.

5.4.1 PGA

It is seen that a PGA of 0.25g has a 50% probability of being exceeded (in 100 years). At this relatively large probability, the three more distant faults (Palo Colorado, King City, and San Andreas) contribute most of the hazard. For accelerations greater than about 0.4g, the Tularcitos fault is the principal contributor. To the extent that the slip rate estimate of 0.42 mm/yr for this fault is valid, the probability of exceeding 0.50g (nearly the MCE level) in 100 years is about 9%.

5.4.2 PSAA (0.20 sec, 5% damping)

At the 50% probability level, the PSAA is 0.63g, caused primarily by the three more distant faults. At the 10% probability level, PSAA appears to be 1.20g, due essentially to the Tularcitos fault.

The results indicate that the mean return period of ground motions comparable to those of the MCE is approximately 1,100 years, subject to considerable uncertainty on account of the poorly known seismicity of the Tularcitos fault.

5.5 Ground Motion Characteristics

5.5.1 General

Selection of a representative time history during preliminary design was required to perform a 2-D dynamic analysis for a preliminary seismic analysis of the dam and for estimation of tensile stresses. These stresses were then used to estimate the volumes of cement content to meet strength requirements for the RCC dam. The results of the dynamic analysis are presented in Section 8.0.

The fundamental period of vibration of the 2-D model was calculated to range between 0.2 and 0.3 seconds, depending on the loading condition. Therefore, median-plus-one standard deviation PSAA were calculated utilizing relationships by Idriss and Geomatrix (Idriss, 1993a) to bracket the target spectral accelerations of importance to the dam. The Tularcitos Fault was selected as the MCE event, (Table 5-1) with a moment-magnitude of 6.8. The resulting estimated peak ground acceleration and calculated values of PSAA are presented in Table 5-2.

Acceleration time histories for rock sites recorded during the October 17, 1989 Loma Prieta and the January 17, 1994 Northridge Earthquakes were reviewed for use in this preliminary design. The University of California Santa Cruz Lick Laboratory (UCSC) record from the Loma Prieta Earthquake (CSMIP, 1991) and the Kagel Canyon, (CSMIP, 1994c) and Pacoima Dam (CSMIP, 1994a) downstream records from the Northridge Earthquake were selected for further evaluation. All three records have epicentral distances between 10 to 20 km.

5.5.2 Accelerogram Selection/Time History

The UCSC zero degree record and both horizontal components of the Kagel Canyon and Pacoima Dam records were scaled to have a peak ground acceleration of 0.51g and have an approximate predominate period of 3.3 hertz. Husid plots (Idriss, 1993b) were generated from each scaled acceleration time history to evaluate the significant duration of the earthquake. The significant duration is defined as the elapsed time of between 5 percent and 95 percent of the total energy in the accelerogram. The significant duration was then compared to the bracketed duration (Bolt, 1977), defined as the elapsed time between the first and last excursion in the accelerogram over 0.05 g. The scaled time histories and Husid plots are presented in Figures 5-2 through 5-7. The bracketed and significant durations are presented in Table 5-3.

The bracketed duration for a magnitude 6.5 to 7.0 earthquake within 10 km of the site should range between 19 and 26 seconds (Bolt, 1977). Review of Table 5-3 indicates that the UCSC record has a 12 second significant duration and 20 to 22 second bracketed duration. Thus, the UCSC scaled record was selected for use in the dynamic model. The vertical input motion was created by multiplying the scaled horizontal input by a factor of 2/3, as is current practice. However, the approach for selecting vertical input motion should be reviewed further during final design. This generalization may not be appropriate in the near field environment (Green, 1992).

5.5.3 Preliminary Response Spectra

In evaluation of the time histories for use in the dynamic model, a preliminary response spectra was plotted for the 5 scaled time histories (Figure 5-8). Also shown in these plots are the median-plus-one standard deviation PSAA values listed in Table 5-2. From inspection of this figure it can be seen that the scaled UCSC spectra agrees with the calculated PSAA values at the periods of 0.2 and 0.3 seconds while the Northridge scaled spectra peak agree at periods greater than 0.3 seconds.

The equal probability (equal hazard) median-plus-one standard deviation spectra envelope calculated using the Geomatrix relationship and the Seed, Ugas and Lysmer (Seed et. al, 1972) median-plus-standard deviation spectra for rock site curve have also been plotted on Figure 5-8. The spectra from the scaled records are in general agreement with the Geomatrix relationship above the period of 0.2 seconds while the Seed et. al. curve slightly under-predicts the response. The Geomatrix and the Seed envelopes indicate a larger response between 0.1 and 0.2 seconds than is predicted by the scaled records. The significance of this latter observation should be investigated further during final design.

5.6 Conclusions

5.6.1 Cachagua Fault Study

Results from this study indicate that there is compelling geologic and geomorphic evidence that the Cachagua fault has not experienced movement since at least the past 85,400 to 213,500 years. This conclusion is based upon the estimated age of Quaternary stream terrace deposits that cover, but are not offset by, the fault. Thus, the Cachagua fault is not "active" according to the criteria established by the California Division of Mines and Geology. The last movement along the fault may be much older than these stream terrace deposits; however, the lack of older Quaternary deposits in the area prevent better resolution of the age of faulting.

The potential for fault rupture through the proposed dam site as a result of movement along the Cachagua fault zone is considered to be low. No faults were mapped in the vicinity of the proposed dam site and no indications of faulting were observed on seismic refraction profiles in the dam area. The undisturbed nature of Quaternary terrace deposits that have been mapped through the project area indicate a lack of fault activity in the vicinity of the proposed dam site.

5.6.2 Seismic Design Criteria

Use of deterministic methods for seismic modeling in lieu of synthetically generated input time histories appears reasonable at this site. Response spectra from scaled time histories from recently recorded strong motions from Loma Prieta and Northridge earthquakes show good agreement between empirical 84 percentile spectra for periods of vibration at or greater than that of the dam. The Geomatrix (Idriss, 1993a) and Seed, Ugas and Lysmer (1974) empirical relationships calculated greater pseudo-absolute accelerations than those calculated from the scaled records for periods below 2 seconds. The controlling earthquake could occur on the Tularcitos fault with a magnitude of 6.8. This magnitude earthquake could generate a 0.51g peak ground acceleration.

TABLE 5-1 FAULT PARAMETERS FOR SITE GROUND MOTION CALCULATIONS

New Los Padres Water Supply Project
Monterey County, California

Fault	Lt[1]	Lr[2]	D[3]	M[4]	S[5]	A[6]	Style[7]	PGA[13]
Tularcitos	62	31	4	6.8	0.42[8]	2.61	OB	0.51
Palo Colorado[10]	200	100	13	7.8	7	3.76	SS	0.37
King City[11]	206	100	18	7.6	2.4	3.43	SS	0.29
San Andreas[12]	291	40	47	7.0	23	4.92	SS	0.09

[1] total fault length (km), taken from Wesnousky (1986)
[2] maximum rupture length (km)
[3] nearest approach of fault to site (km)
[4] moment magnitude -- Tularcitos is estimated for this study; the rest are from Wesnousky (1986)
[5] slip rate (mm/yr) from Wesnousky (1986)
[6] seismicity rate parameter for probability calculations
[7] OB=oblique slip; SS=strike slip
[8] Bechtel (1988)
[9] minimum slip rate (mm/yr) of Wesnousky -- arbitrarily assigned
[10] using slip rate for San Gregorio-Hosgri fault
[11] considered as extension of the Rinconada fault; using Wesnousky data for the Rinconada fault
[12] considering Wesnousky's Los Altos-San Juan Bautista and San Juan Bautista-Highway 58 (north end of Carizzo Plain) fault segments and averaging the slip rates
[13] peak ground acceleration (g) calculated using Idriss (1991)

TABLE 5-2: MCE GROUND MOTIONS
New Los Padres Water Supply Project
Monterey County, California

Fault	AR[1]	Period (sec)	PGA[2]	PGA+1σ[3]	PSAA[4]	PSAA+1σ[5]
			(g)			
Tularcitos	I	0.00	0.51	0.75		
	G	0.00	0.54	0.79		
	I	0.20			1.26	1.89
	I	0.30			1.19	1.82
	G	0.20			1.23	1.87
	G	0.30			1.11	1.73

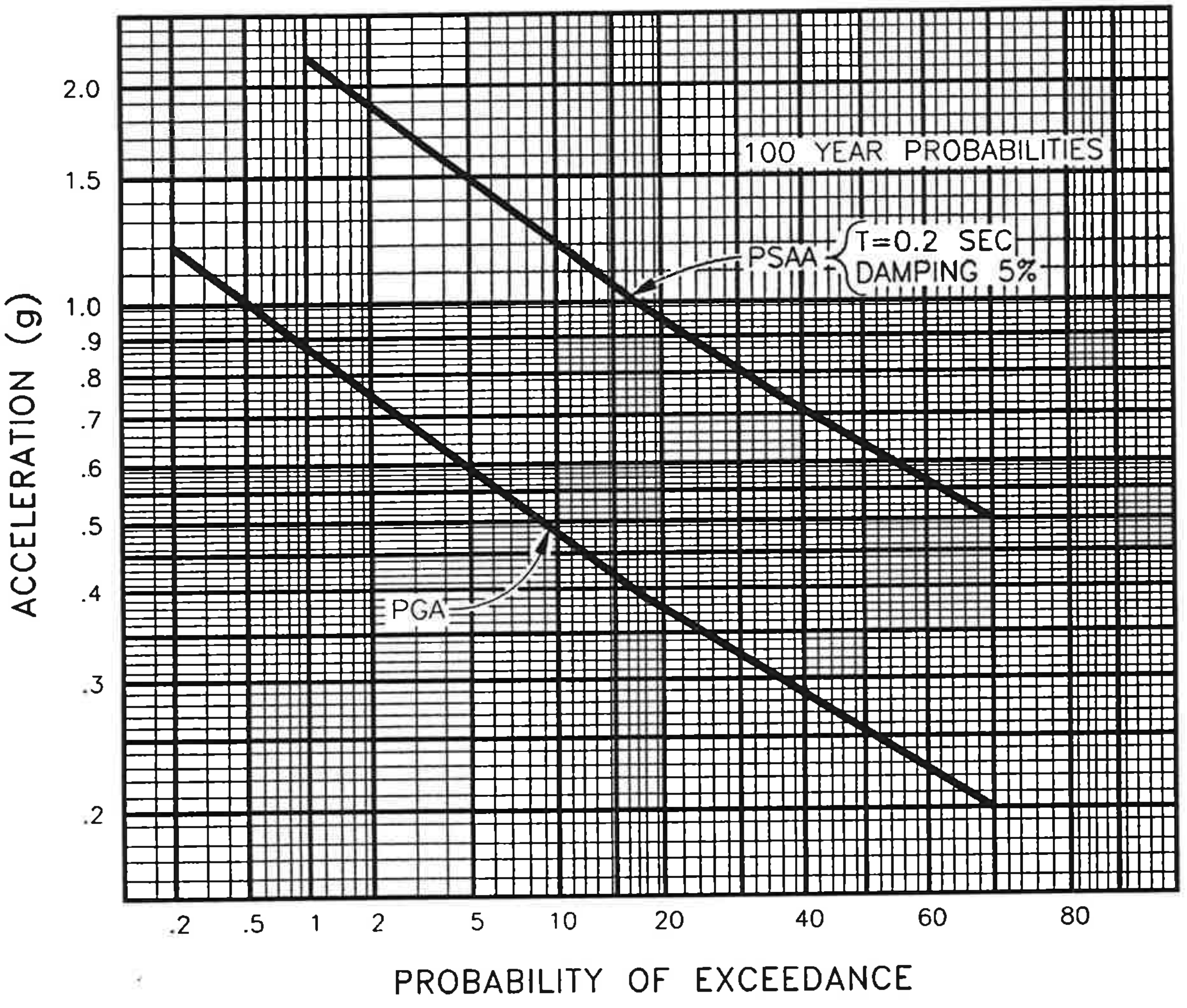
[1] Attenuation relationship: I=Idriss (1991); G=Geomatrix (1992)
[2] median peak horizontal ground acceleration
[3] median PGA + 1 standard deviation
[4] median pseudo-absolute acceleration, 5% damped
[5] median PSAA + 1 standard deviation

TABLE 5-3: SUMMARY OF SELECTED TIME HISTORY CHARACTERISTICS
New Los Padres Water Supply Project
Monterey County, California

Locations	CSMIP[1] S/N	Epicentral Distance (km)	Azimuth (Degree)	Recorded Maximum Acceleration (g)	Duration[2] (Seconds)	
					Bracketed	Significant
UCSC	58135	16	0	0.46	22-25	14
Pacoima Dam	24207	19	265	0.44	16	10
			175	0.42	16	10
Kagel Canyon	24088	13	360	0.44	23	13
			90	0.30	23	13

[1] CSMIP California Strong Motion Instrument Program

[2] Duration taken from scaled accelerograms (0.51 g, 3.3 Hertz)



PREPARED BY: RWG PHD. REVIEWED BY: MHS DATE: 2-9-95

DWG

PROBABILITY OF EXCEEDANCE FOR PGA AND PSAA

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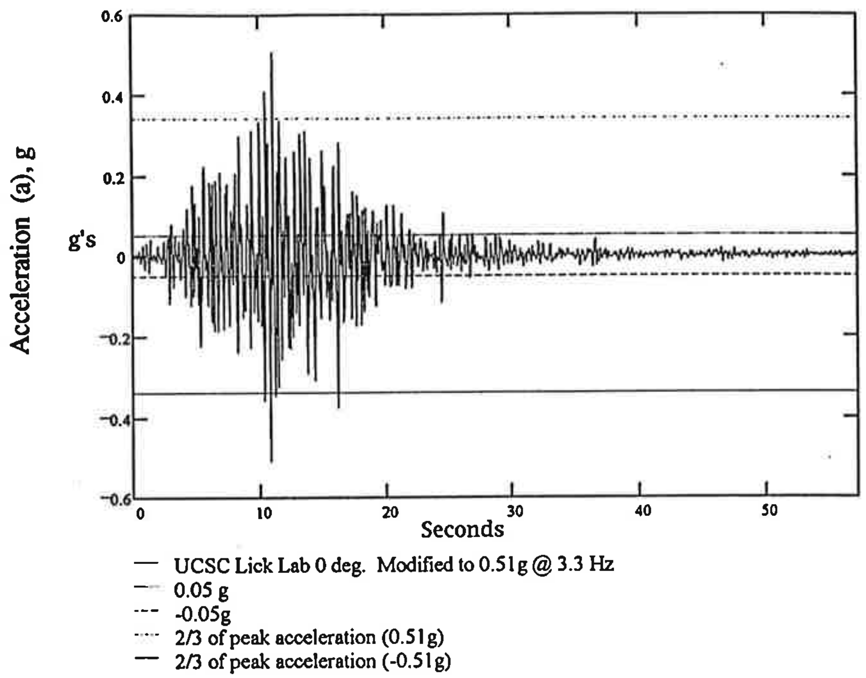
William Cotton and Associates

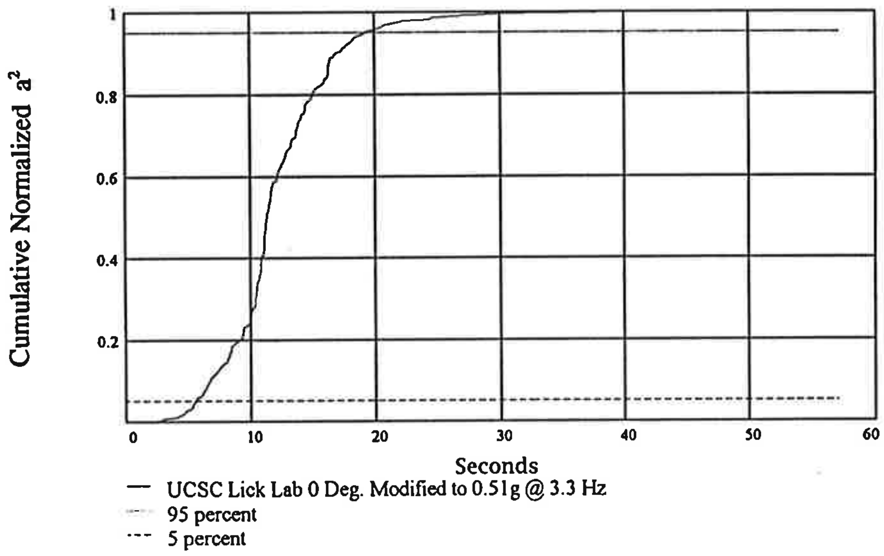
Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
5-1



Normalized Cummulative Acceleration - Husid Plot



NEW LOS PADRES DAM - MODIFIED TIME HISTORY
94-1198801.74

Time History Used in 2-D Finite Element Model :
Loma Prieta - UCSC Lick Lab - 0 Degrees
Horizontal Input Motion

PREPARED BY MHS
REVIEWED BY KBK
DATE 2/14/95

Horizontal Input Motion - UCSC Lick Lab - 0 Degrees

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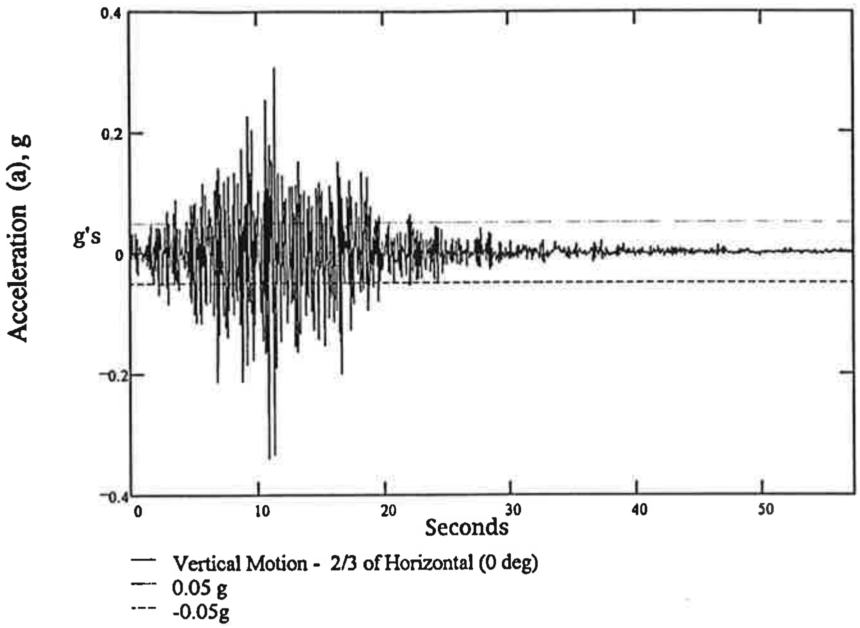
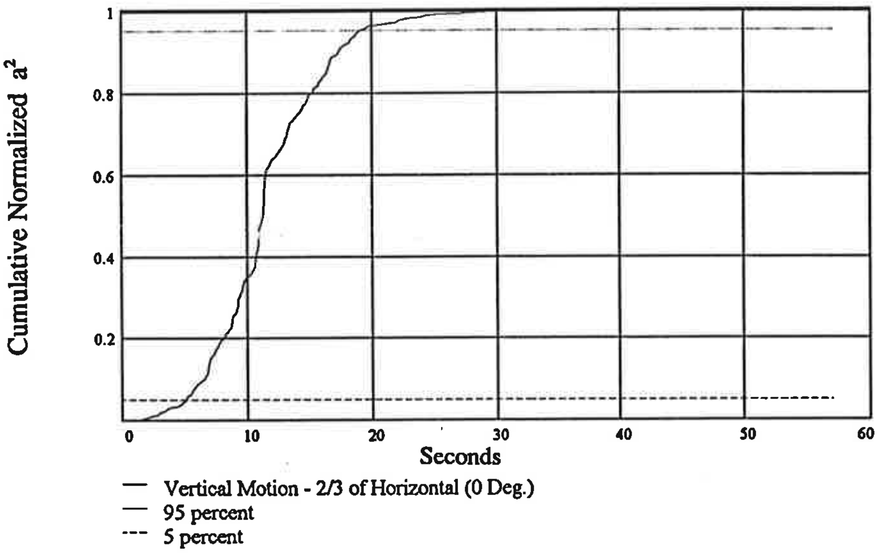
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
5-2



Normalized Cummulative Acceleration - Husid Plot



NEW LOS PADRES DAM - MODIFIED TIME HISTORY
94-1198801.74

Time History Used in 2-D Finite Element Model :
Loma Prieta - UCSC Lick Lab - 0 Degrees
Vertical Motion

PREPARED BY MHS
REVIEWED BY KBK
DATE 2/14/95

Vertical Input Motion - UCSC Lick Lab - 0 Degrees

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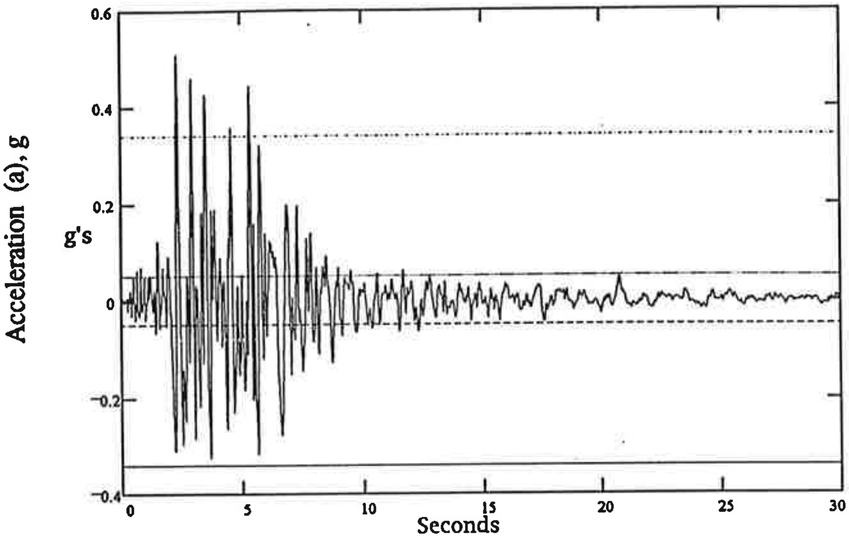
New Los Padres Water Supply Project
Monterey County, California

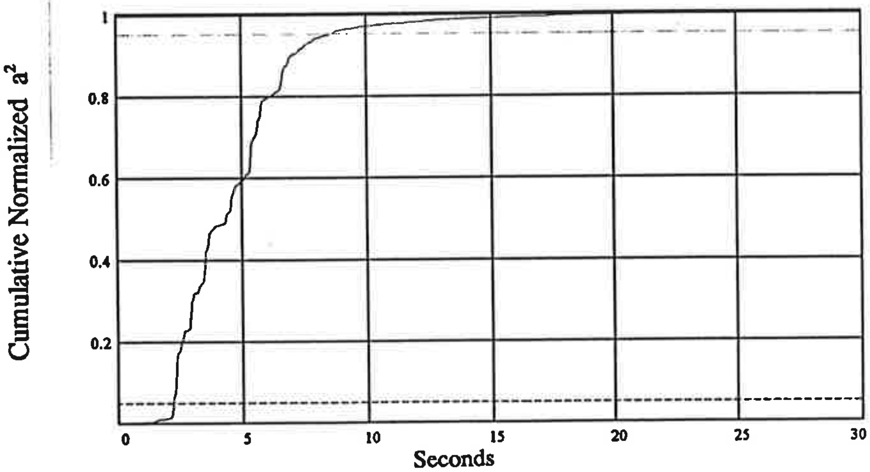
PROJECT NO.
94-1198801.80

FIGURE NO.
5-3



— Kagel Canyon 90 deg. Modified to 0.51g @ 3.3 Hz
-·- 0.05 g
--- -0.05g
····· 2/3 of peak acceleration (0.51g)
— 2/3 of peak acceleration (-0.51g)

Normalized Cummulative Acceleration - Husid Plot



— Kagel Canyon 90 Deg. Modified to 0.51g @ 3.3 Hz
-·- 95 percent
--- 5 percent

NEW LOS PADRES DAM - MODIFIED TIME HISTORY
94-1198801.74

Northridge Earthquake - Kagel Canyon
18 km Epicentral Distance - Sedimentary Rock

PREPARED BY MHS
REVIEWED BY KBK
DATE 2/14/95

Modified Kagel Canyon - 90 Degrees Time History

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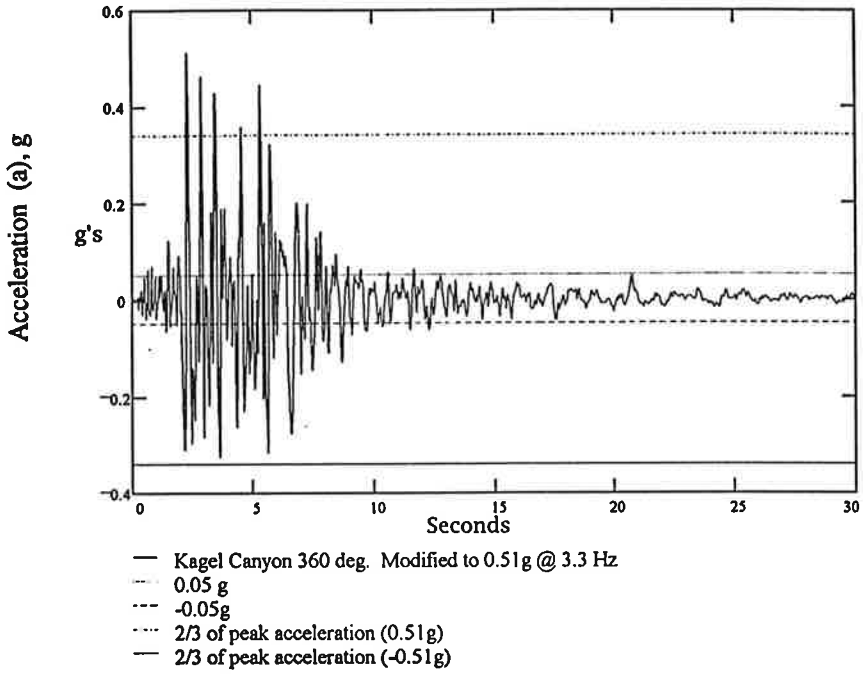
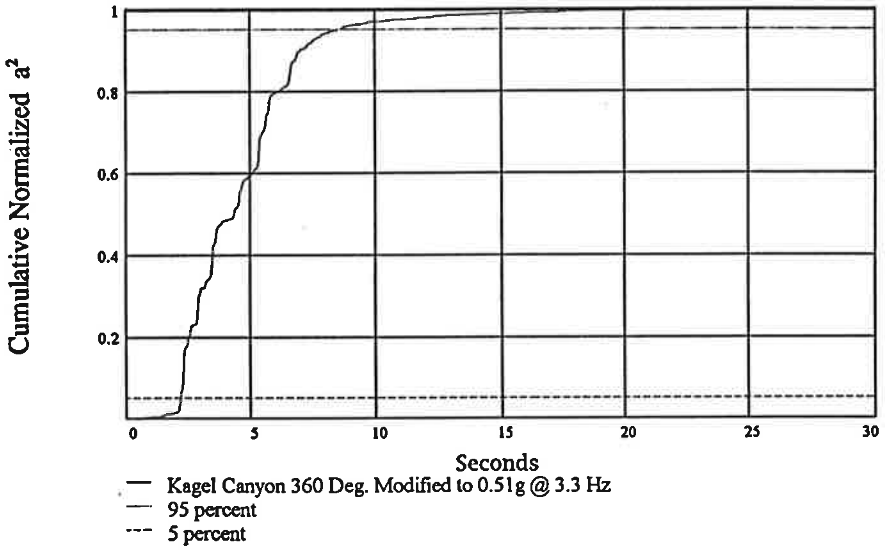
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
5-4



Normalized Cummulative Acceleration - Husid Plot



NEW LOS PADRES DAM - MODIFIED TIME HISTORY
94-1198801.74

Northridge Earthquake - Kagel Canyon
18 km Epicentral Distance - Sedimentary Rock

PREPARED BY MHS
REVIEWED BY KBK
DATE 2/14/95

Modified Kagel Canyon - 360 Degrees Time History

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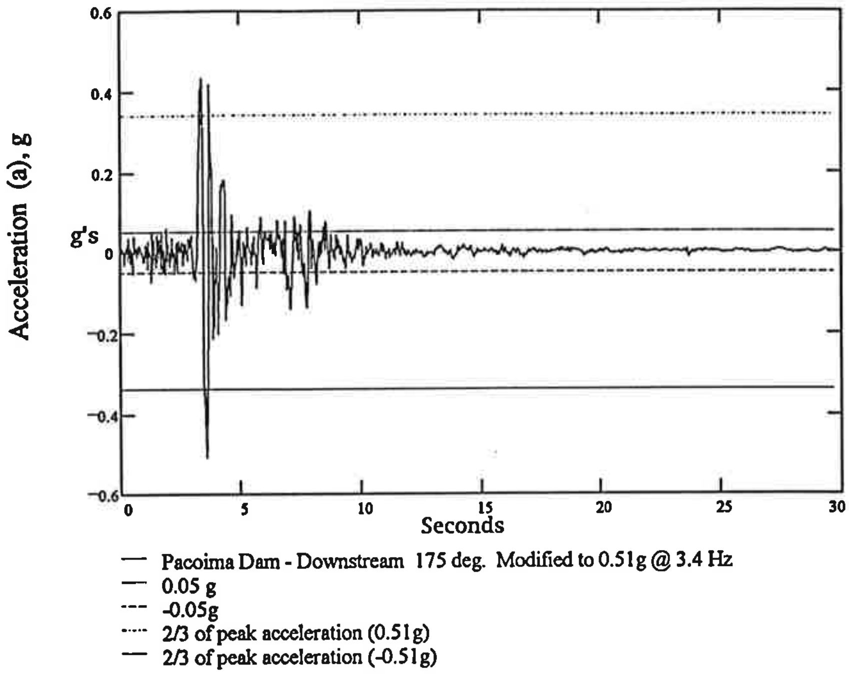
94-1198801.80

FIGURE NO.
5-5

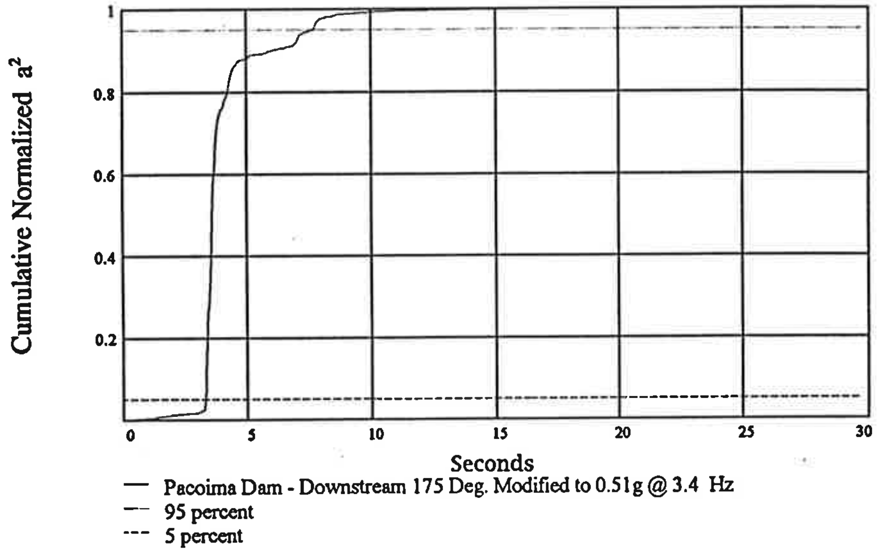
PREPARED BY MHS

REVIEWED BY KBK

DATE 2/14/95



Normalized Cummulative Acceleration - Husid Plot



NEW LOS PADRES DAM - MODIFIED TIME HISTORY
94-1198801.74

Northridge Earthquake : Pacoima Dam - Downstream
19 km Epicentral Distance - Diorite Gneiss

Modified Pacoima Dam Downstream - 175 Degrees Time History

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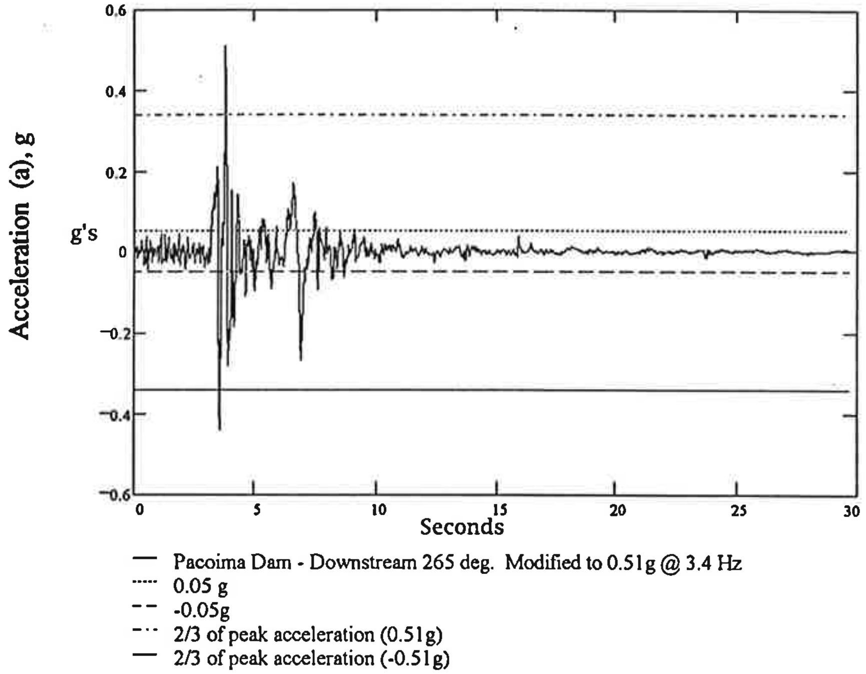
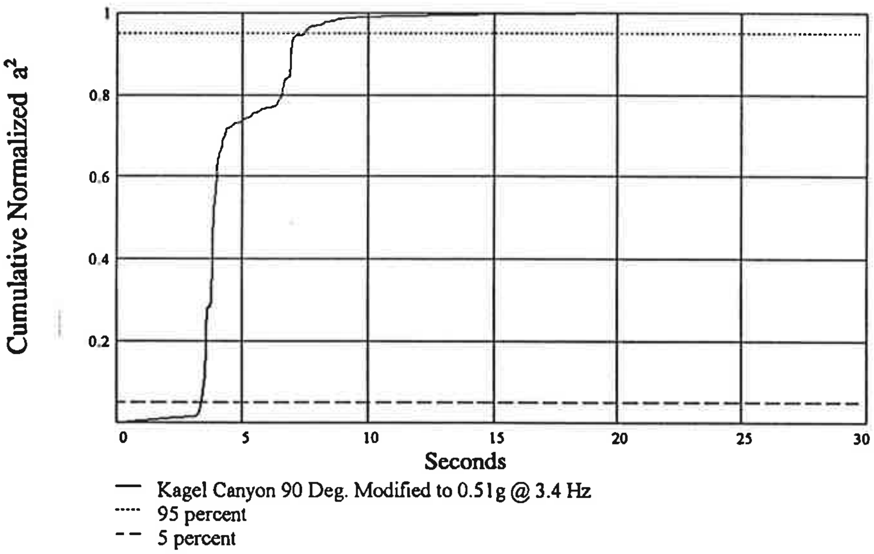
PROJECT NO.
94-1198801.80

FIGURE NO.
5-6



Normalized Cummulative Acceleration - Husid Plot



NEW LOS PADRES DAM - MODIFIED TIME HISTORY
94-1198801.74

Northridge Earthquake: Pacoima Dam -Downstream
19 km Epicentral Distance - Diorite Gneiss

PREPARED BY MHS REVIEWED BY KBK DATE 2/14/95

Modified Pacoima Dam Downstream - 265 Degrees Time History

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FIGURE NO.
5-7

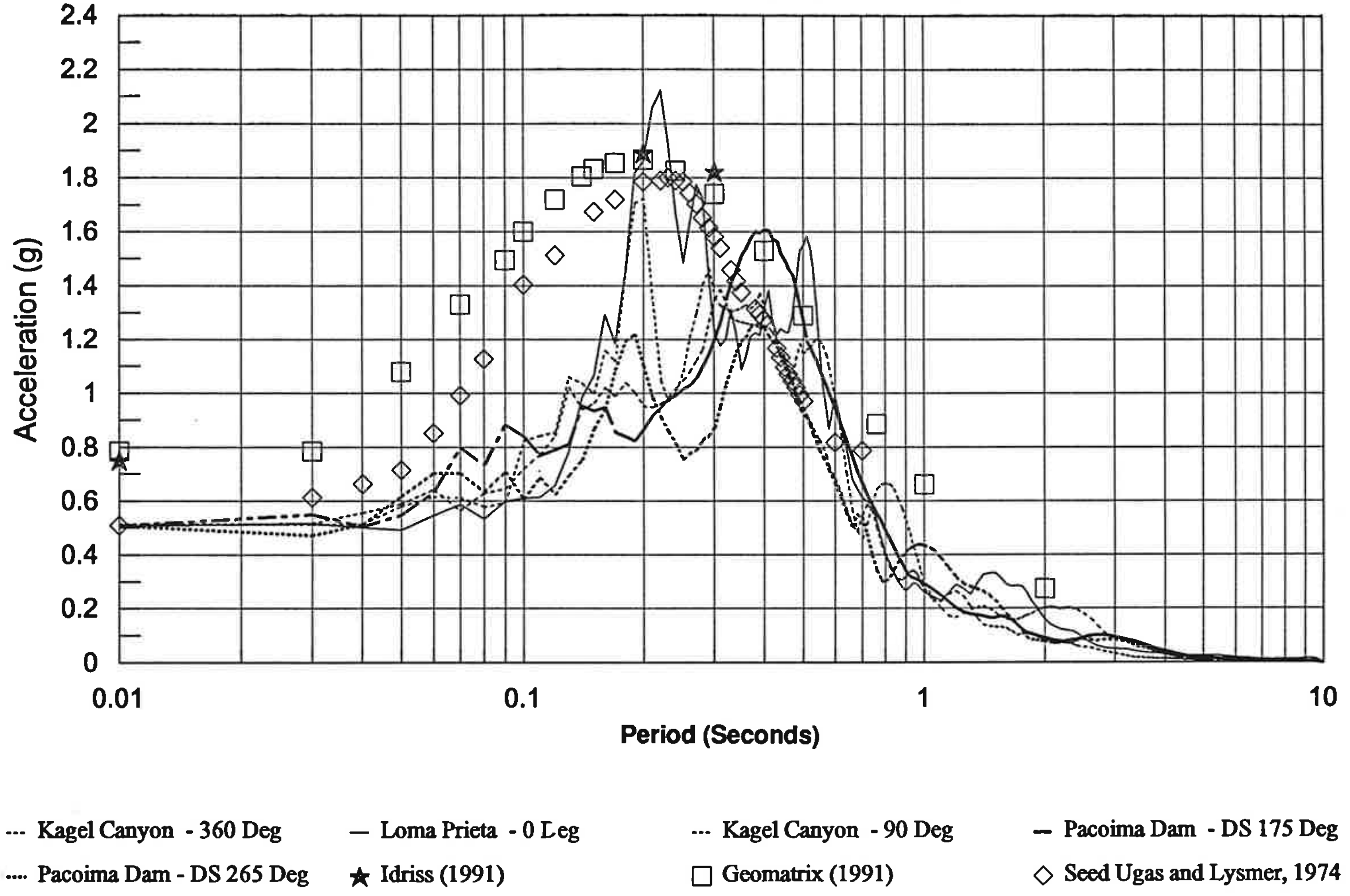
PREPARED BY MHS

REVIEWED BY KBK

DATE 2/14/95

Preliminary Response Spectra

New Los Padres Dam - Damping Ratio - 5 percent



84 Percentile Response Spectra

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New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
5-8

TBLCK (5/28/92)

6.0 RCC MATERIALS TESTING

6.1 Introduction/Previous Studies

Bechtel's 1989 cost estimate assumed the New Los Padres RCC gravity dam would be constructed using aggregate derived from terrace gravel Borrow Areas and rock quarries both on and off site. Previous borrow area studies for New Los Padres Dam were by Bechtel (Bechtel, 1992). Two of the terrace gravel borrow areas upstream of the damsite (Borrow Areas A and B) were investigated with test pits and sampled for laboratory testing. No crushing trials of the aggregate were made nor were RCC trial mixes prepared.

In order to evaluate the quality of the terrace gravel deposits in Borrow Areas A, B, and C as sources for RCC aggregate and to make a better estimate of the cementitious content of the RCC needed to meet the strength requirements of the dam, a field and laboratory testing program was carried out as part of the current study. The program comprised laboratory testing of terrace gravel samples obtained from the test pits described in Section 3.0, crushing a representative sample of the gravel and boulders from the terrace deposits, laboratory testing of the crushed aggregate, and an RCC trial mix program.

This section summarizes the procedures and results of the testing program. Aggregate laboratory test results are presented in Appendix D and the description of the testing procedures are contained in Appendix F. Results of the RCC test program are presented in Appendix G. The 90 day tests for the splitting tensile strengths, compressive strengths and mortar bar tests will be performed after the submitttal of this report. Analysis of the dam for selecting the cement content required for the RCC mixes is included in Section 8.0.

6.2 Terrace Gravel Testing Program

6.2.1 Sampling

Samples of terrace gravel were obtained from test pits excavated in Borrow Areas A, B, and C as described in Section 3.0. Test pits logs are contained in Appendix B. Materials excavated from each test pit were segregated in piles of differing physical characteristics (i.e., silty sand overburden, terrace gravel, and weathered bedrock). The terrace gravel pile was a composite mixture of all the terrace gravel and sand obtained from the pit. A visual estimate (recorded on the log) was made of the gradation of the boulders (maximum size and percent larger than 12-inches in diameter), and cobbles (3-inches to 12-inches in diameter). Approximately 250-pounds of terrace material (minus 3-inch) was collected from various locations in each pile. After digging into the pile to avoid the segregated outer portion, samples were collected in plastic-lined burlap bags using a shovel. The bagged samples were labeled and transported to the laboratory.

6.2.2 Laboratory Testing

Testing of the terrace gravel material was carried out at Testing Engineers Inc. (TEI) laboratories in Oakland and Martinez following ASTM standard testing procedures, as appropriate. Table 6-1 summarizes the type and number of tests performed. Test results are presented in Appendix D.

6.2.3 Test Results

The results of the laboratory testing program are summarized in Tables 6-2 through 6-13. The results are grouped by borrow area and test pit designation for each test. Results from the Bechtel testing program are compiled in the same table for reference. Test pit samples from the current program are labeled TP and those from the Bechtel program T. Shallow samples from test pits T-7 and T-8, and TP-3 and TP-12 are representative of the silty sand alluvial fan deposits that overlie the terrace gravel deposits and should not be considered when analyzing the results of the terrace gravel testing program. The following is a summary of the test results of the current program, a comparison with the Bechtel test results, and analysis of the results.

Gradation (ASTM C-136)

Table 6-2 summarizes the composite gradations of the terrace gravel from Borrow Areas A, B, and C based on the visual field gradations described in the test pit logs, as well as the laboratory sieve analyses from the current and Bechtel investigations. The Bechtel logs do not differentiate between boulders and cobbles, and only estimate the percentage of plus 3-inch size material. Figure 6-1 shows the composite average gradation curve for the minus 4-inch portion of terrace materials from each of the borrow areas.

Generally, the terrace deposits in borrow areas A and B have similar gradations, consisting of approximately 18 percent (%) boulders (12- to 36-inches in diameter), 37%

cobbles (3- to 12-inches), 18% gravel (No. 4 sieve to 3-inches, 23 % sand (No. 4 to No. 200, and 4% silt (smaller than No. 200). Terrace materials in Borrow Area C are much coarser with a gradation of approximately 62% boulders, 22% cobbles, 7% gravel, 9% sand, and a negligible amount of silt.

It should be noted that Bechtel estimated the maximum size of some boulders to be 8-feet in diameter. The maximum size boulder encountered during the current investigation was approximately 5-feet in diameter. The Bechtel field estimates of total cobbles and boulders (greater than 3-inches in diameter) is about 15% higher than that found in the present study. On the other hand the Bechtel sieve analyses of minus 2-inch sand and gravel are finer. This may be due to the fact that the maximum size sampled appears to have been 2-inches. This deficiency in the large gravel sizes may account for the finer gradations indicated by the test results.

Moisture Content (ASTM D-2216)

The moisture content of the terrace gravel samples from borrow areas A and B were about 2.7% and 4.5%, respectively. No moisture samples were collected from the Borrow Area C excavations. Bechtel apparently did not compute the natural moisture content of samples taken during their investigation.

Atterberg Limits

Atterberg Limit tests were not conducted on the silty sand alluvial fan deposits under the current testing program; however, several tests were performed by Bechtel. Their tests showed the material to be non-plastic.

Bulk Specific Gravity and Absorption (ASTM C-127 and C-128)

Table 6-3 summarizes the bulk specific gravity (saturated surface-dry) and absorption results, for the coarse and fine fractions of all samples from both investigations.

Specific gravity and absorption are important properties for designing concrete mixes. High specific gravity generally translates to high strength and durability. Absorption of more than 1% is usually considered a sign of slightly weathered aggregate and therefore of lesser quality.

The average specific gravities of 2.68 and 2.57 for the coarse and fine fractions, respectively, are considered average for the mineral composition of the sands and gravel in borrow areas A, B, and C. Average absorptions of 1.8% and 2.0% are higher than ideal but are well within the range of acceptable values for making quality RCC.

It should be noted that the Bechtel test results generally showed slightly higher specific gravities and lower absorptions than the current testing program. In addition, samples from Borrow Area B showed slightly better results than Borrow Area A.

Sodium Sulfate Soundness (ASTM C-88)

Table 6-4 summarizes the results of sodium sulfate soundness testing from the current program as well as Bechtel's.

The sodium sulfate soundness test is an indicator of structural weakness of aggregate. Compressive strength and freeze/thaw durability are directly related to the percentage loss. Values of 10% or less for coarse material and 8% or less for fine material are considered excellent.

The average percent loss for the coarse and fine fractions from Borrow Areas A and B are 17.5% and 13.4%, respectively, which indicates that the material is slightly weathered and susceptible to breakdown. However, since the project is not located in an area of severe freeze/thaw conditions, the material is considered adequate for RCC aggregate. It appears that the material from Borrow Area B is of slightly higher quality than that from Borrow Area A.

It should be noted that the test results for the current program are significantly better than those from the Bechtel program. This indicates that additional testing should be performed during final design.

Los Angeles Abrasion (ASTM C-131)

Table 6-5 summarizes the results of the Los Angeles Abrasion (L.A. Rattler) testing from the Bechtel and current programs.

The Los Angeles Abrasion test is a measure of hardness and toughness and is an indicator of potential for breakdown during stockpiling, and handling. There is a direct relationship with compressive strength. Generally accepted guidelines for Los Angeles Abrasion tests are that no more than 10% loss should occur after 100 revolutions and no more than 40% loss after 500 revolutions.

The average percentage losses of 16% (100 revolutions) and 50% (500 revolutions) for materials from borrow areas A, B, and C exceed the guidelines for high quality aggregate. The results from samples from the borrow areas are consistent, as are the results between the two testing programs (Bechtel and current). However, the overall performance of the materials during the testing program indicate that the materials are considered acceptable as aggregate for use as mass RCC.

Potential Alkali Reactivity (ASTM C-289)

Table 6-6 summarizes the results of potential alkali reactivity testing. All samples tested for potential alkali reactivity were tested to be innocuous.

Mortar Bar (ASTM C 227-90)

Table 6-7 summarizes the results of mortar bar testing. Samples were measured at 14 and 30 days and all results indicated that expansion was less than or equal to 0.008 percent. Values less than 0.05 percent at 3 months and 0.10 percent at 6 months are not considered expansive. The results shown in Table 6-7 indicate that expansion should not be a problem for aggregate derived from the terrace gravel borrow areas. Additional measurements will be made at 3 and 6 months.

Mineral Count

Table 6-8 summarizes the results of the mineral count of coarse terrace gravel (3/8-inches to 3-inches in diameter). The results are nearly identical for all samples. The presence of 2% to 3% metasedimentary rocks in the terrace gravels probably accounts for some of the losses recorded in the Los Angeles Abrasion tests.

Soil Classification

The terrace gravel deposits from borrow areas A, B, and C can be described as: Silty Sandy Gravel (GP), light brown, moist, gap-graded, arkosic, 20% boulders, 35% cobbles, 20% gravel, 20% sand, 5% silt, sub-round to round, composed of clasts of granite, gabbro, and minor schist.

6.2.4 Conclusions

It can be concluded from the laboratory testing program that the terrace gravel deposits in borrow areas A, B, and C are suitable for processing into aggregate for RCC. The terrace gravels are gap-graded with a preponderance of boulders; however, the gradation can be improved by crushing, screening and blending. Less than optimum test results for absorption, sodium sulfate soundness, and Los Angeles Abrasion indicate that the gravel is slightly weathered and therefore susceptible to breakdown during handling and extreme weather conditions. However, since the RCC aggregate gradation will be adjusted after the materials have been crushed and processed, breakdown from handling should not effect the final product. Since the site is located in an area that is not subject to freeze/thaw conditions, the less than optimum characteristics are not considered serious deficiencies. These aggregate test results indicate that the RCC will probably require a slightly higher cementitious content to achieve the necessary durability and strengths than would RCC using a higher quality aggregate. The results of mortar bar tests indicate that expansion of concrete aggregate should not be a problem.

6.3 Aggregate Crushing and Testing

Aggregate for the RCC trial mix program was obtained from a composite sample of alluvial terrace gravel from test pits TP-12 (Borrow Area A) and TP-8, TP-9, TP-10, and TP-11 (Borrow Area B). The sample was crushed and screened to 1-inch minus size at the Granite Construction Co. plant in San Jose. Figure 6-2 illustrates the gradation of the processed crushed aggregate mix. Although the maximum size aggregate (MSA) is 1 to 1.5-inches smaller than what will probably be used in the dam (2 to 2.5 inch), and the sand content is about 3% higher than what will be allowed in the dam (40%), the material was used as-delivered for the trial mix program. No adjustments were made to the gradation in the laboratory.

Tests were performed in the laboratory on the crushed aggregate sample which are summarized in Table 6-9.

The test results on the crushed sample are consistent with the results of the tests on the discrete terrace gravel samples, although the specific gravity is significantly higher and the absorption lower (i.e., better quality) than the terrace gravel samples. The excellent results from the sodium sulfate soundness tests are in-line with the current test results but much lower than the Bechtel results. In fact, the sodium sulfate soundness results are in-line with generally accepted standards for excellent quality aggregate. The results of the Los Angeles Abrasion tests are very similar to tests run on Borrow Area A and B materials in this and the Bechtel test programs. It is interesting to note that in spite of the slightly higher than desirable results of the Los Angeles Abrasion tests, the terrace gravel did not appear to break down to a great degree during the loading, crushing, screening, and handling processes. Although a total sand content (<No. 4) of 43% resulted, the percentage of fines (<No. 200) remained approximately the same at 4%.

6.4 RCC Trial Mix Program

6.4.1 General

The Bechtel 1989 cost estimate was based on two zones of RCC within the gravity section. A low strength zone comprising about 90% of the section with a cementitious (cement plus fly ash) content of 100 pounds per cubic yard (lbs/cy), and a richer zone at the downstream toe with a cementitious content of 135 lbs/cy. Subsequently, concrete consultant Gary Maas (Maas, 1992), estimated the cementitious content would range from 250 to 400 lbs/cy. Because of the wide spread in the estimated cementitious contents, and the impact cementitious content has on the estimated cost of the dam (up to 30%) it was decided that, even at this early stage of project development, an RCC trial mix program should be performed.

The objective of the program was to prepare five trial RCC mixes over a range of cementitious contents, to determine compressive strength at 7, 28, and 90 days, and splitting tensile strength at 28, and 90 days.

The program was carried out at the TEI laboratory in Oakland on November 16-17, and December 15, 1994. The program was executed in accordance with the procedures described in Appendix F. Test results are contained in Appendix G.

6.4.2 Trial Mix Designs

Initially a trial mix was prepared to determine optimum water content and compaction procedures (i.e., lift thickness, compactive effort, time of compaction, etc.). The mix contained 300 lbs/cy cement, 3,537 lbs/cy aggregate and started out with a free water content of 110 lbs/cy. Figure 6-2 shows the gradation used for the crushed aggregate of the trial mixes. A cylinder was prepared, unit weight measured, and the cylinder stripped and examined. The equivalent of 20 lbs/cy of water was added in increments and the process repeated until the mix had a water content equivalent to 210 lbs/cy. It was then estimated that the optimum water content to achieve maximum density was 190 lbs/cy.

Trial mixes were proportioned by cubic yard (cy) in accordance with Table 6-10. The absolute volume method was used assuming that the aggregate was in a saturated surface-dry (SSD) condition with a specific gravity of 2.69, and an absorption of 1.6% as determined in the laboratory. A theoretical unit weight of 150 pounds per cubic foot (lbs/cf) was calculated assuming an air content of 1.5%.

Water content was maintained approximately constant (adjusted for variations in moisture content of the aggregate for each batch) for all mixes and the water versus cement ratio (W/C) and paste versus mortar ratio (P/M) was allowed to vary.

It became clear after the 7-day breaks that Mix No. 5 did not contain enough water to fully hydrate the cement in the mix or to achieve maximum density. Therefore two additional mixes were proportioned by the same method maintaining a constant W/C ratio of 0.6.

In conventional concrete, the W/C ratio is directly related to concrete strength and durability. A W/C ratio of 0.44 is considered the minimum to achieve satisfactory workability. In RCC the relationship is not as pronounced; however, for RCC of equal density, lower W/C ratios can be expected to produce higher strengths.

The paste (cementitious material, minus No. 200 aggregate fines, water, and air) versus mortar (paste and sand) ratio (P/M ratio) is an important factor in proportioning RCC mixes to ensure workability, minimize segregation, and to facilitate consolidation with a vibratory compactor.

6.4.3 Trial Procedures

Batches of 3 cubic feet (cf) were prepared for each mix. Aggregate and cement were mixed in a 12 cf mixer for five minutes. Water was added and mixing continued for 15 more minutes until the components were thoroughly blended.

Sets of 10 cylinders were made for each trial mix by placing four equal lifts of the material into disposable plastic molds supported by a steel split-frame anchored to the floor, and compacted. Compaction was achieved with a Bosch 1340 Demolition Hammer (Kango Hammer for mix Nos. 7 and 8) fitted with a specially fabricated shoe. Compactive effort included the surcharge of the operator leaning against the hammer. Compaction time for each lift was maintained constant at 20 seconds for all mixes. Following compaction, the cylinders with plastic sleeves, were labeled, weighed, and cured in a fog room until tested.

Two cylinders from each batch were broken in compression at 7, 28, and 90-days. In addition two cylinders each were broken in splitting tension at 28, and 90-days.

The sequence of cylinder breaks was determined at the time they were compacted based on the measured weight of cylinders and observations made during preparation and observations are included in Appendix F. Based on these factors, the poorest cylinders (i.e., those with low density and/or defects) were broken in order of least to most importance (i.e., 7-day compression, 28-day compression, 28-day tension, 90-day compression, and 90-day tension).

6.4.4 Trial Mix Test Results

Compressive and splitting tensile test results for mix Nos. 2, 5, and 7 have been disregarded because of cylinders exhibiting low density. For the remaining mixes low breaks due to defective cylinders have been deleted and pairs of breaks have been averaged. Complete results of the RCC trial mix program are contained in Appendix G.

Unit Weight

The unit weights of all cylinders were measured prior to testing. The average unit weight of cylinders from each mix are shown in Table 6-11.

It can be seen that the average unit weight of cylinders from trial mix Nos. 1, 5, and 7 are well below the design weight of 150.0 lb/cf and the results of strength tests should not be considered.

Compressive Strength

Compressive strength results from the RCC trial mix program are summarized in Table 6-12 and represented graphically in Figures 6-3 and 6-4. The strength gain versus time curves plotted in the figures have been plotted for the 7, 28, and 90 day strengths.

Splitting Tensile Strength

Splitting tensile strength test results of the RCC trial mix program are summarized in Table 6-13 and represented graphically in Figures 6-5 and 6-6. Splitting tensile strengths varied between 12 and 19 percent of compressive strengths.

6.4.5 Conclusions

It can be concluded from the RCC trial mix program that RCC meeting the compressive and tensile strength requirements of the dam can be made utilizing on-site aggregate derived by crushing alluvial terrace gravel from borrow areas A and B.

Additional RCC trial mixes will be required at subsequent levels of design to evaluate the effect of variations in gradation, source of aggregate, water content, cement/pozzolan ratio, and to develop the appropriate cemetitious content as dictated by design strength requirements. Recommendations for additional testing are provided in Section 12.0.

Table 6-1: SUMMARY OF LABORATORY TESTING PROGRAM
New Los Padres Water Supply Project
Monterey County, California

TEST	QUANTITY
Grain Size Analysis (ASTM C 136)	7
Moisture Content (ASTM D 2216)	2
Bulk Specific Gravity and Absorption Coarse and Fine Material (ASTM C 127 and C 128	4
Sodium Sulfate Soundness (ASTM C 88)	3
Los Angeles Abrasion (ASTM C 131)	4
Potential Alkali Reactivity (ASTM C 289)	3
Mortar Bar (ASTM C 227-90)	3
Mineral Count	3

TABLE 6-2: SUMMARY OF COMPOSITE GRADATIONS

New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pit No.	Max. Size Boulder (ft)	% Boulder (>12")	% Cobble (3"-12")	% Gravel (No.4-3")	% Sand (No. 200-No. 4)	% Silt (>No. 200)
A	T-3	3.0	n/a	70	15	12	3
A	T-5	3.0	n/a	50	11	30	9
A	T-6	6.0	n/a	70	10	17	3
A	T-9	2.0	n/a	20	20	49	11
A	TP-12	2.0	20	40	25	14	1
AVERAGE		4.2	4 (20)	50 (40)	16	25	5
B	TP-9	3.0	20	50	14	15	1
B	T-10	n/a	n/a	60	14	23	3
B	T-11	6.0	n/a	75	10	14	1
B	TP-11	4.0	25	15	28	30	3
B	TP-16	3.5	5	30	32	30	3
AVERAGE		4.1	10 (17)	46 (32)	20	22	2
C	TP-18	3.0	70	20	3	7	0
C	TP-19	3.0	55	25	10	10	0
AVERAGE		3.0	62	22	7	9	0

Explanation:

N/A - Not available
(20) - Data from this investigation only

TABLE 6-3: SUMMARY OF BULK SPECIFIC GRAVITY AND ABSORPTION

New Los Padres Water Supply Project
Monterey County, California

BORROW AREA	TEST PIT No.	Coarse		Fine	
		SpG	Absorption (%)	SpG	Absorption (%)
A	T-2	2.74	1.6	2.45	1.7
A	T-3	2.71	1.5	2.55	3.2
A	T-6	2.67	1.6	2.50	2.0
A	TP-12	2.60	2.8	2.61	2.5
	AVERAGE	2.68	1.9	2.53	2.4
B	T-10	2.73	1.0	2.64	2.0
B	T-11	2.72	1.5	2.58	1.0
B	T-12	2.60	2.8	2.61	2.5
	AVERAGE	2.68	1.8	2.61	1.8

TABLE 6-4: SUMMARY OF SODIUM SULFATE SOUNDNESS TESTS

New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pit No.	Weighted Average Loss (%)	
		Coarse	Fine
A	T-2	7.4	11.3
A	T-3	20.8	14.7
A	T-6	42.9	22.5
A	TP-12	7.4	7.6
	AVERAGE	19.6	14.0
B	T-10	25.5	15.6
B	T-11	17.0	12.4
B	TP-11	3.7	5.3
	AVERAGE	15.4	11.1

TABLE 6-5: SUMMARY OF LOS ANGELES ABRASION TESTS
New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pit No.	Percent (%) Loss	
		100 Revolutions	500 Revolutions
A	T-2	n/a	44.2
A	T-3	n/a	49.0
A	T-6	n/a	57.2
A	TP-12	13.4	51.3
AVERAGE		13.4	50.4
B	T-10	n/a	57.1
B	T-11	n/a	48.8
B	TP-11	18.1	47.0
AVERAGE		18.1	51.0
C	TP-19	16.7	47.4
AVERAGE		16	50

TABLE 6-6: SUMMARY OF POTENTIAL ALKALI REACTIVITY TESTS
New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pit No.	R_c (millimoles/liter)	S_c (millimoles SiO2/liter)	Degree of Alkalinity
A	T-2	185.0	19.0	Innocuous
A	T-6	143.0	25.3	Innocuous
A	TP-12	41.0	21.0	Innocuous
AVERAGE		123.0	21.8	Innocuous
B	T-10	153.0	24.3	Innocuous
B	TP-11	50.0	26.0	Innocuous
AVERAGE		101.5	25.4	Innocuous

TABLE 6-7: SUMMARY OF MORTAR BAR TEST RESULTS
New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pit No.	Percent Expansion	
		14 Days	30 Days
A	TP-11	+0.007	+0.008
B	TP-12	+0.004	+0.007
A and B	Crushed Rock	+0.006	+0.008

TABLE 6-8: SUMMARY OF MINERAL COUNTS
New Los Padres Water Supply Project
Monterey County, California

Borrow Area	Test Pit No.	Metasediment (%)	Dark Diorite (%)	Light Diorite (%)
A	TP-9	3	39	59
B	TP-12	2	39	58
A and B	Crushed Rock	3	40	57

TABLE 6-9: SUMMARY OF PHYSICAL CHARACTERISTICS OF CRUSHED AGGREGATE

New Los Padres Water Supply Project
Monterey County, California

TEST	RESULT	
Bulk Specific Gravity (Dry)	2.649	
Bulk Saturated Surface Dry (SSD) Specific Gravity	2.692	
Absorption	1.6%	
Moisture Content	3.0%	
Sodium Sulfate Soundness	Coarse Aggregate	8.8%
	Fine Aggregate	5.4%
L.A. Abrasion	Loss 100 Revolutions	13.6%
	Loss 500 Revolutions	52.2%
Potential Alkali Reactivity	R_c	50
	S_c	22
	Degree of Alkalinity	Innocuous
Mortar Bar	Percent Expansion 14 Days	0.006 %
	Percent Expansion 30 Days	0.008 %
Mineral Count	Metasediments	3%
	Dark Diorite	40%
	Light Diorite	57%

TABLE 6-10: SUMMARY OF RCC TRIAL MIX PROPORTIONS
New Los Padres Water Supply Project
Monterey County, California

Mix No.	Water	Cement	Aggregate	W/C Ratio	P/M Ratio
1	184	100	3,735	1.84	0.35
2*	252	200	3,650	1.26	0.42
3	190	300	3,564	0.63	0.41
4	192	400	3,479	0.48	0.44
5	193	500	3,393	0.38	0.46
6*	189	200	3,650	0.95	0.38
7**	240	400	3,479	0.60	0.47
8	300	500	3,232	0.60	0.53

* Mix No. 6 was prepared to replace Mix No. 2 because of an error in proportioning the original Mix No. 2.

** Mix No. 7 was prepared to check Mix No. 4 with an increased water content but cylinders were not compacted satisfactorily.

TABLE 6-11: SUMMARY OF UNIT WEIGHTS OF RCC TRIAL MIXES
New Los Padres Water Supply Project
Monterey County, California

MIX NO.	AVG. UNIT WEIGHT
1	146.6
2	153.9
3	150.3
4	151.2
5	147.4
6	151.1
7	146.3
8	150.2

TABLE 6-12: SUMMARY OF RCC COMPRESSIVE STRENGTH TESTS
New Los Padres Water Supply Project
Monterey County, California

Trial Mix No.	Cement Content	Compressive Strength (psi)		
		7/11-Day*	28-Day	90-Day
1	100	365	535	645
6	200	1,500	2,035	2,515
3*	300	2,475	2,700	3,340
4*	400	2,530	3,295	4,770
8	500	2,815	3,315	TBP

* Due to Thanksgiving and Christmas Holidays Trial Mix Nos. 3 and 4 were broken at 11 and 29 days rather than 7 and 28 days, respectively.

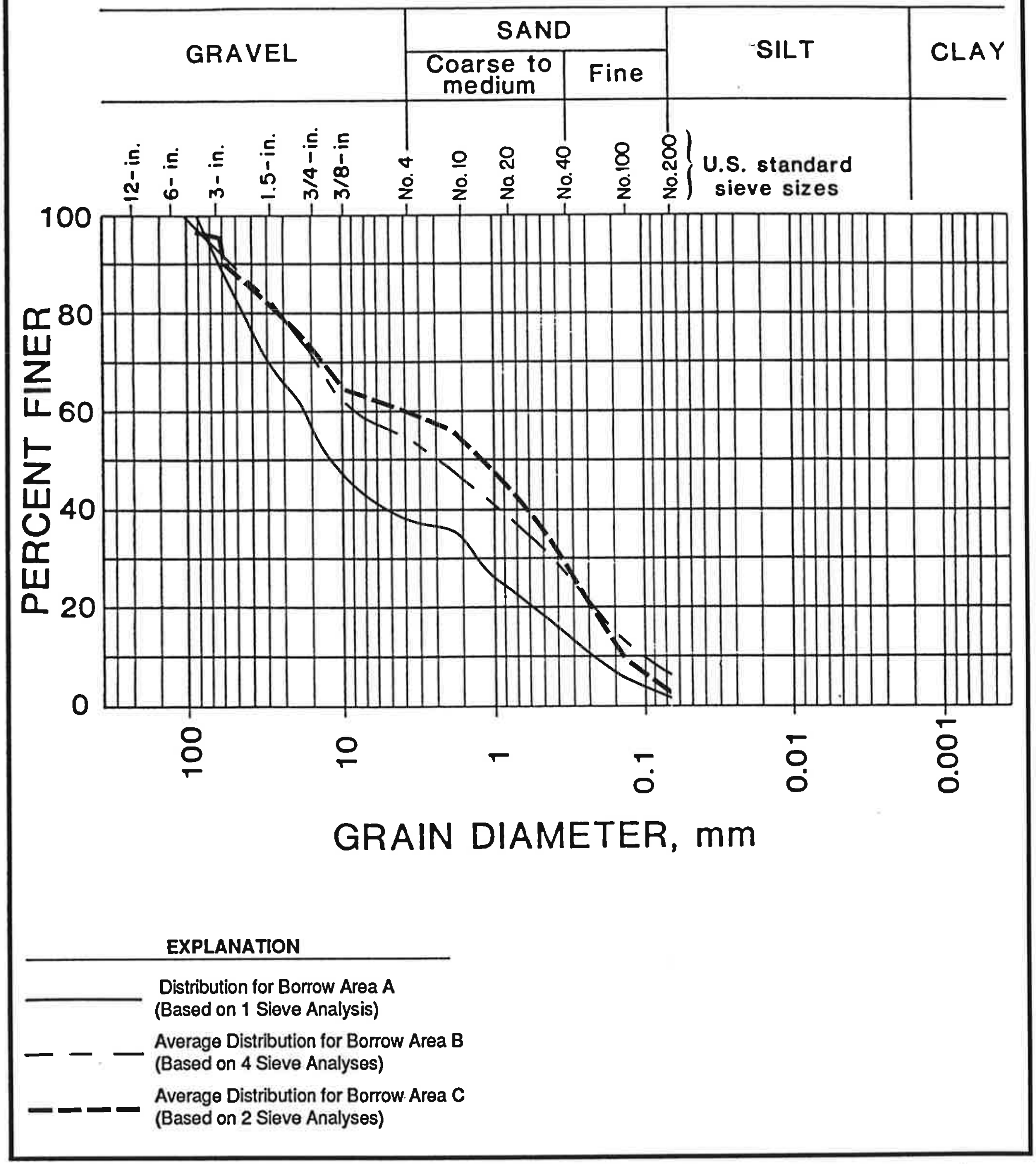
TBP To be performed.

TABLE 6-13: SUMMARY OF RCC SPLITTING TENSILE STRENGTH TESTS
New Los Padres Water Supply Project
Monterey County, California

Trial Mix No.	Cement Content (lbs/cy)	Tensile Strength (psi)	
		28-Day	90-Day
1	100	83	105
6	200	328	390
3*	300	380	485
4*	400	478	595
8	500	445	TBP

* Due to the Christmas Holidays Trial Mix Nos. 3 and 4 were broken at 29 days rather than 28 days.

TBP To be performed.



AVERAGE GRAIN SIZE DISTRIBUTIONS - BORROW AREAS A, B, AND C (TERRACE DEPOSITS)

PREPARED BY DLW/BBE
REVIEWED BY KBK
DATE 2/15/95

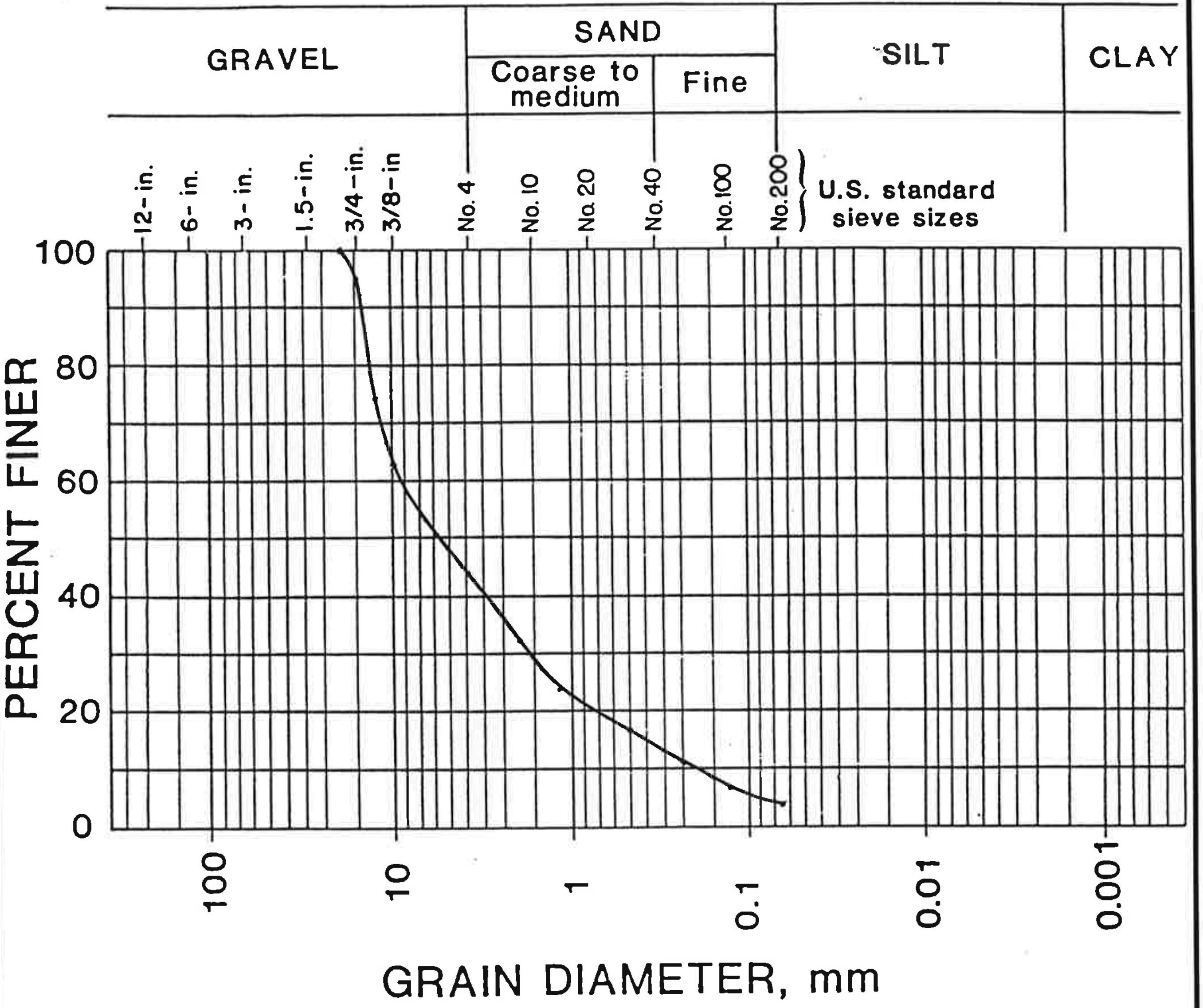
THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
6-1

TBLCK (5/28/92)



PREPARED BY DLW/BBE
REVIEWED BY KBK
DATE 2/15/95

GRAIN SIZE DISTRIBUTION - CRUSHED AGGREGATE

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS

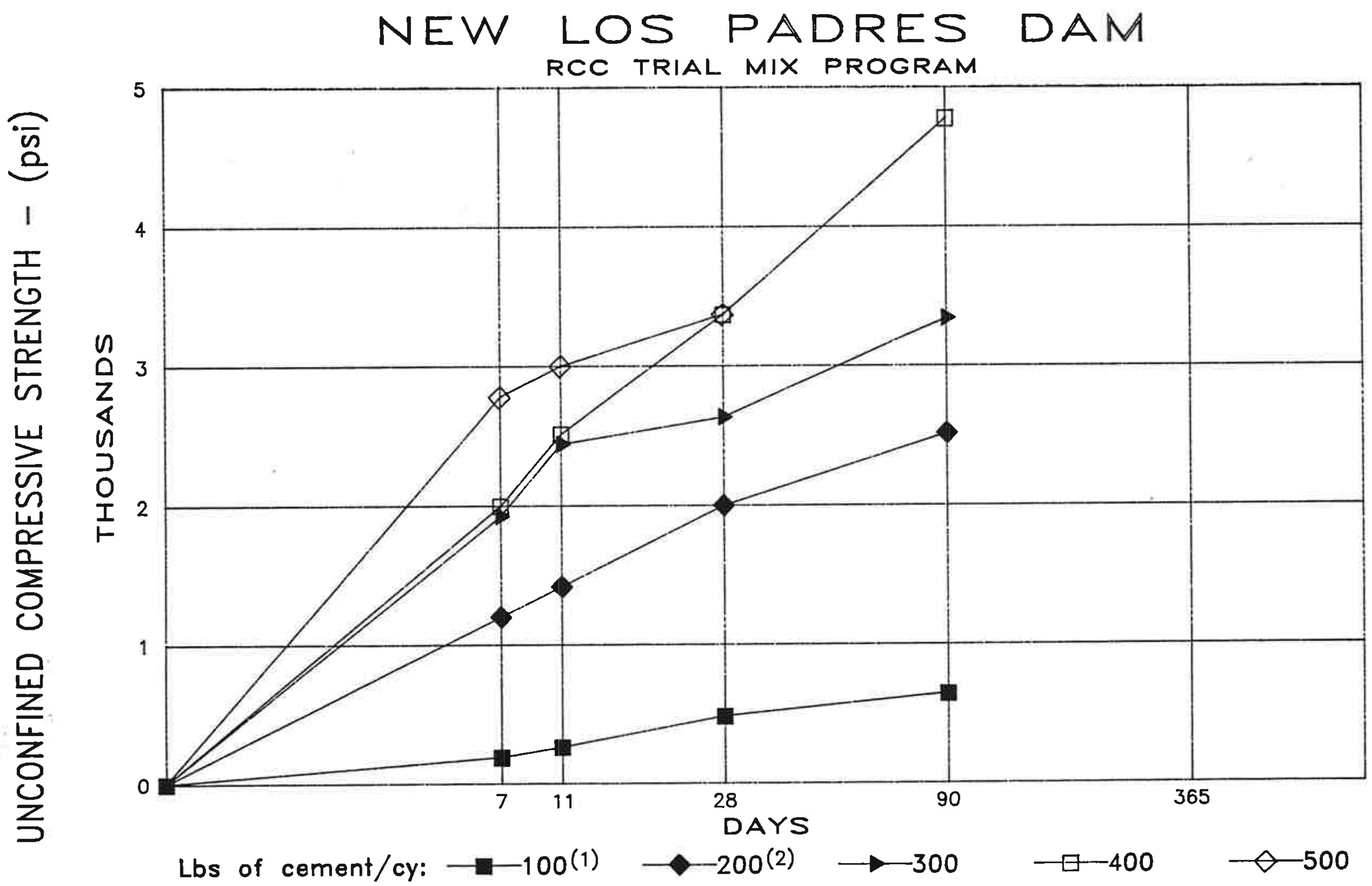
Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
6-2

TBLCK (5/28/92)

PREPARED BY: WAM REVIEWED BY: KBK DATE: 2-9-95



(1) lbs of cement/cy (Typical)

(2) 200 pound/cy cylinder broken at 7 days, others at 11 days. 11-day value for 200 pound/cy mix and 7-day value for others, estimated.

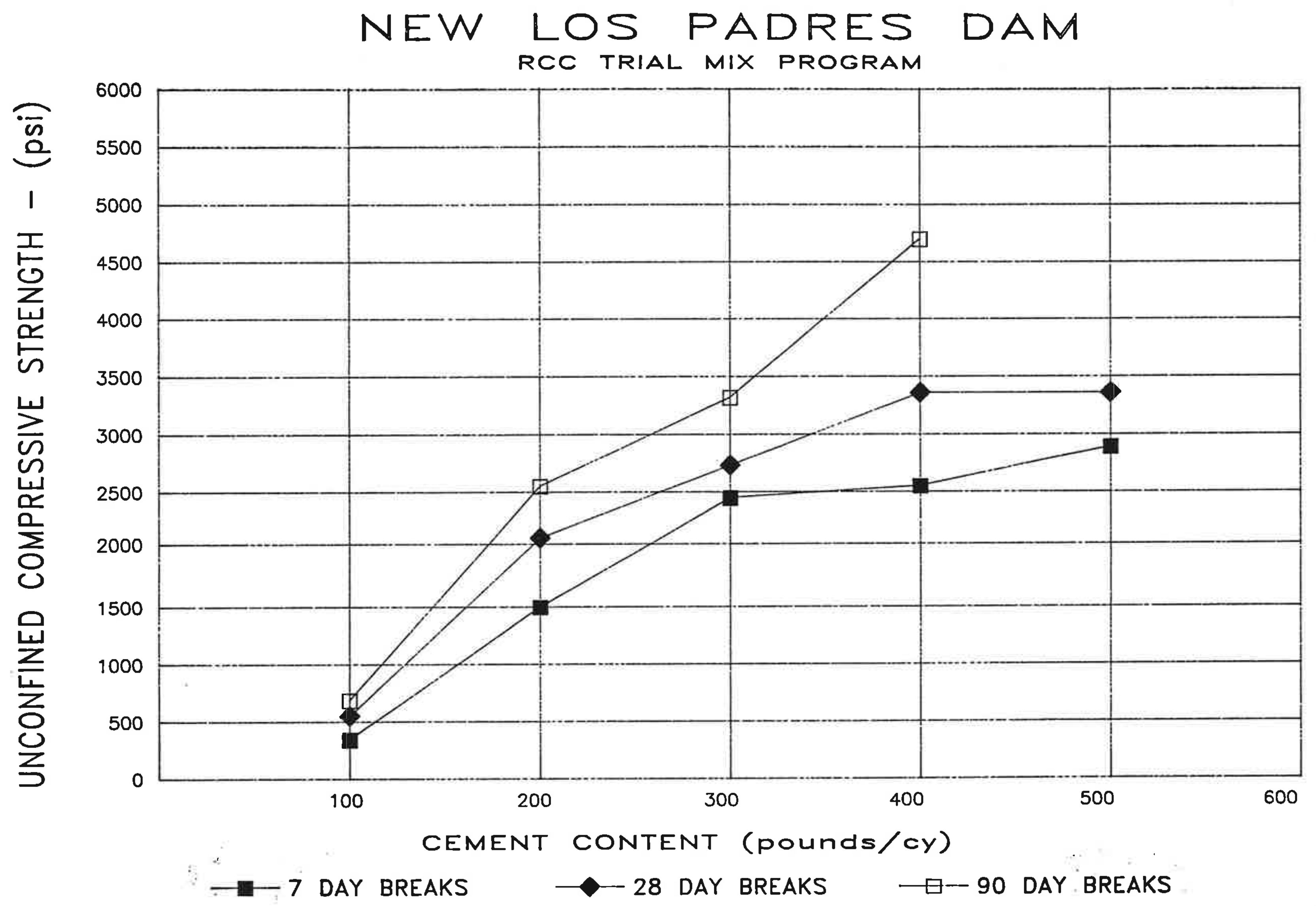
UNCONFINED COMPRESSIVE STRENGTH VERSUS TIME

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS
In Association With
MORRISON KNUDSEN CORPORATION
William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
6-3



PREPARED BY: WAM REVIEWED BY: KBK DATE: 2-9-95

UNCONFINED COMPRESSIVE STRENGTH VERSUS CEMENT CONTENT

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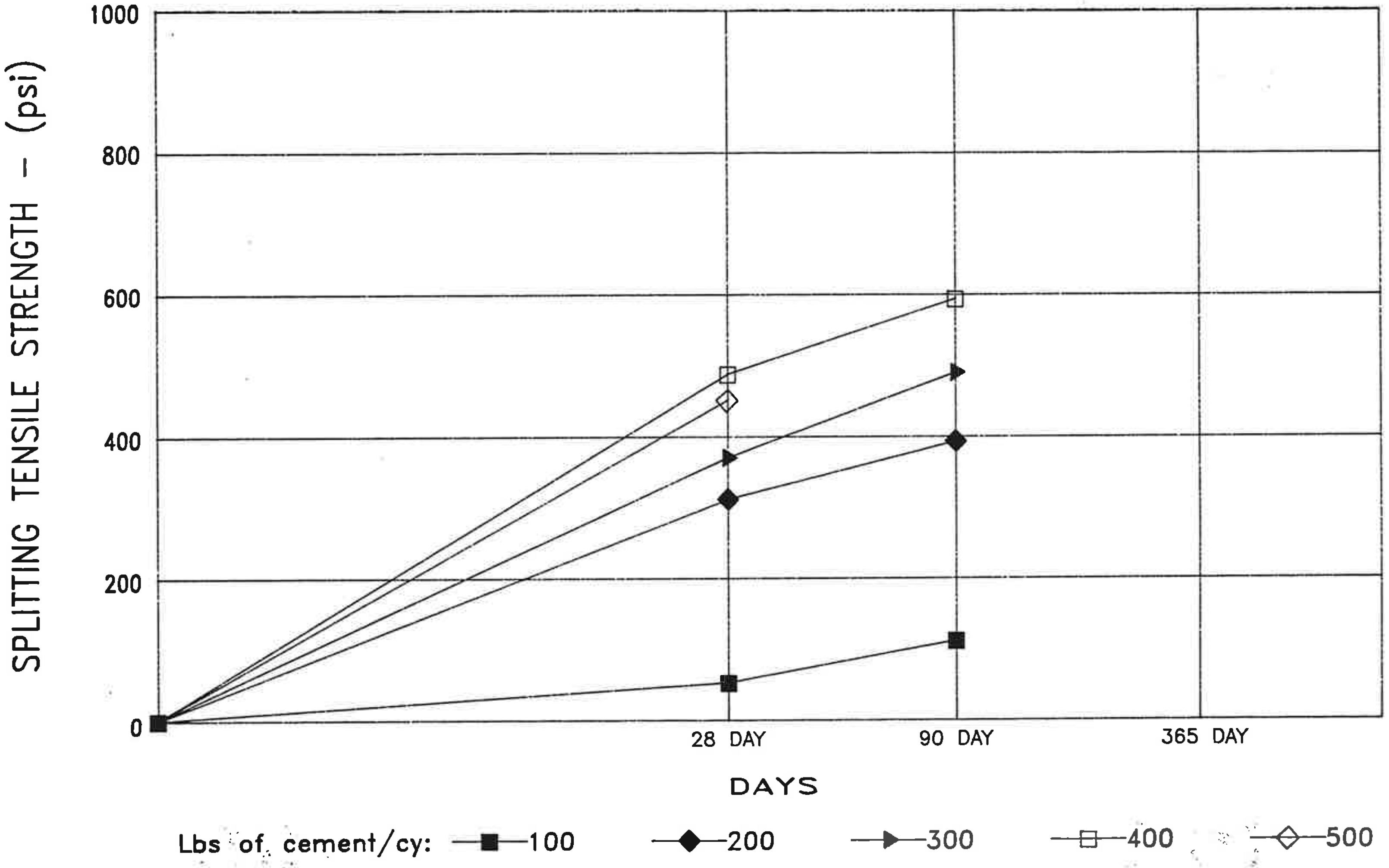
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
6-4

NEW LOS PADRES DAM

RCC TRIAL MIX PROGRAM



UNCONFINED SPLITTING STRENGTH VERSUS TIME

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New Los Padres Water Supply Project
Monterey County, California

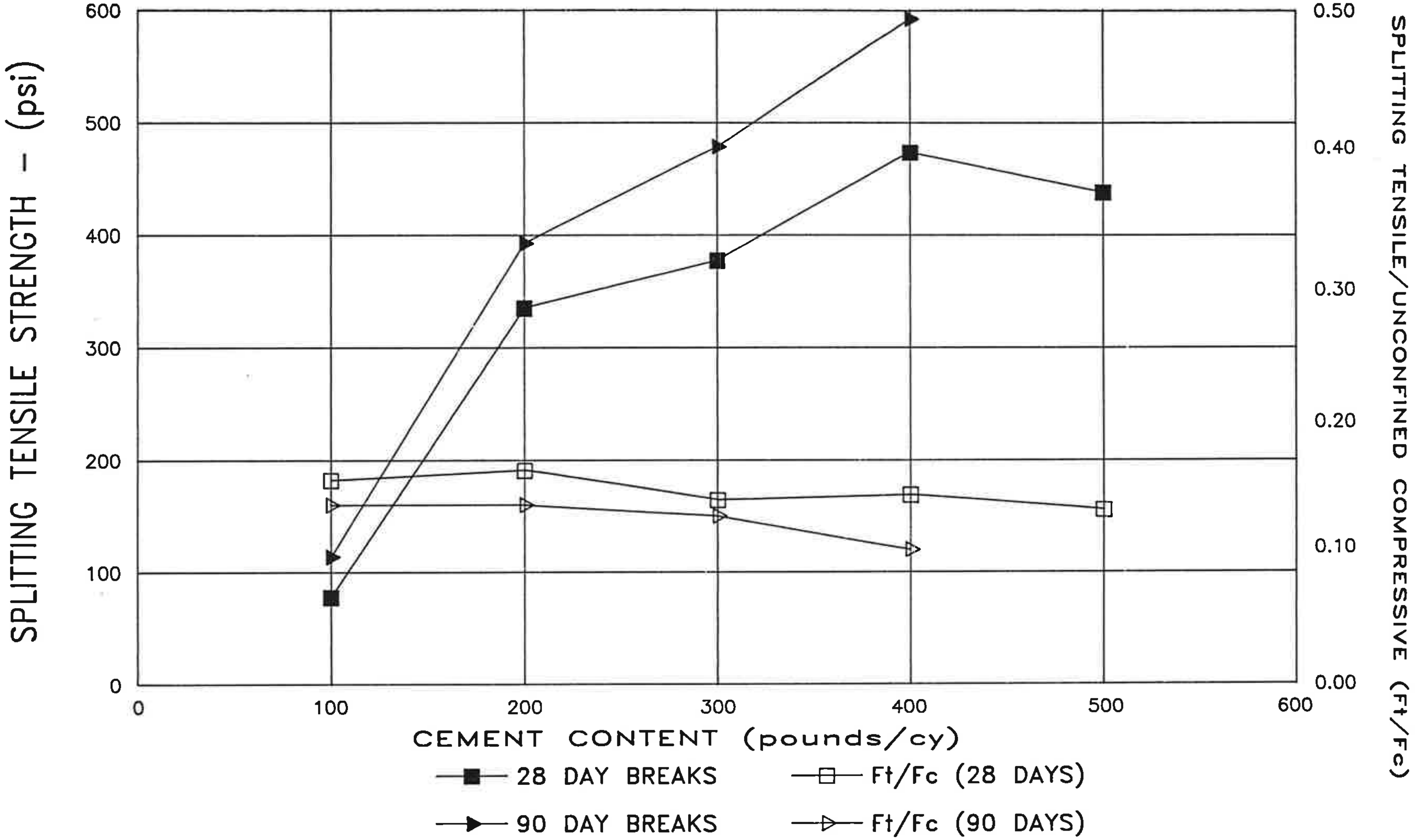
PROJECT NO.
94-1198801.80

FIGURE NO.
6-5

PREPARED BY: WAM REVIEWED BY: KBK DATE: 2-9-95

NEW LOS PADRES DAM

RCC TRIAL MIX PROGRAM



PREPARED BY: WAM REVIEWED BY: KBK DATE: 2-9-95

DWG

UNCONFINED SPLITTING STRENGTH VERSUS CEMENT CONTENT

THE MARK GROUP, INC.
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Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

FIGURE NO.
6-6

7.0 GEOTECHNICAL ENGINEERING EVALUATION

7.1 General

This section of the report first outlines the construction material requirements for the project and describes the potential sources for these materials both from on and off-site sources. It also summarizes relevant information on field investigations carried out by these and previous studies to evaluate these materials. The foundation condition along the dam alignment are then reviewed with respect to anticipated depth of required excavation and construction excavation slopes. Laboratory test results from these studies are discussed in Section 6.0. Assumptions regarding other geotechnical issues including foundation treatment, spacing of consolidation, grout holes, and grout curtain configuration are discussed in Section 9.0.

7.2 Construction Materials

7.2.1 General

Construction materials for a roller compacted concrete (RCC) dam at the New Los Padres site were first evaluated at a reconnaissance level by Bechtel (Bechtel, 1992). Three alluvial terrace gravel borrow areas (borrow areas A, B, C) and two rock quarries (quarries A and B) upstream of the dam (borrow area D), as well as the required excavation for the dam were identified as potential sources of construction material.

The Bechtel subsurface investigation and laboratory testing program were limited to two of the borrow areas (borrow areas A and B). In order to derive the volume of material necessary to construct the dam, the study considered sources within a two-mile radius downstream of the damsite; however, no subsurface investigations were carried out at these locations. Downstream sources included alluvial terraces adjacent to the Carmel River (near Princes Camp) and a rock quarry on the right bank of the river approximately 2.0 miles away. All of these sites are located on private land.

It is apparent that exploration of construction materials from these downstream sites would represent a severe impact on the biophysical and socioeconomic environment of the upper Carmel River Valley and would significantly increase the construction costs of New Los Padres Dam. Therefore, the present study has focused on investigating and evaluating potential construction materials within the future reservoir below El. 1,130 feet.

7.2.2 Material Requirements

RCC is made up of a blend of mineral aggregate meeting a predetermined gradation, that is combined with cement and water. At some sites it is technically and economically advantageous to substitute a portion of the cement content with a pozzolan, such as fly ash. For this project, cement and, if required, pozzolan will be provided from off-site commercial sources. Mineral aggregate, however, will have to be obtained from on-site sources.

Construction of the 282-foot high RCC dam as described in Section 9.0 and the migrant fish collection facilities will require a total of approximately 885,000 cubic yards (cy) of RCC (conventional concrete), which in turn will require an equal amount of suitable processed aggregate. Conventional concrete aggregate for structural walls, slabs, mass concrete, facing concrete, and bedding mortar will have to be processed from the same sources as the RCC aggregate or obtained from off-site. The aggregate will need to be of high enough quality to meet the durability and strength requirements to be established during final design.

Maximum size aggregate (MSA) is expected to be 2- to 3-inches. Although it might be theoretically more economical to use larger MSA, construction experience on other RCC dams has shown that use of large diameter aggregate often leads to excessive segregation which in turn results in rock pockets, poor bonding between lifts, and permeable zones within the dam. Furthermore, sandy mixes with approximately 40% passing the No.4 sieve, have been shown to help reduce segregation. Specifications for RCC aggregate are generally more flexible than those for conventional concrete mixes and can accommodate sizable fractions of non-plastic material finer than the No. 100 and No. 200 sieves as compared to conventional concrete aggregate. Table 7-1 presents a typical well graded, combined RCC aggregate specification.

In all likelihood, naturally occurring construction materials will have to be processed to meet these requirements. Because a certain amount of waste is involved when processing aggregate, it is prudent engineering practice to prove at least 1.5 to 2.0 times the amount anticipated for the project. Therefore, for New Los Padres Dam, a total volume of at least 1.4 to 1.8 million cy of construction material, suitable for processing into concrete aggregate, must be identified for the project to be considered feasible. Once these materials have been identified, it will be up to the contractor to use them in the most cost-effective manner while meeting the technical requirements of the project.

7.2.3 Sources of Construction Material

Construction materials for the dam will be obtained from on-site borrow areas, including the required dam excavation, and from off-site sources. The borrow areas have been labeled A through K and are discussed in the On-site Sources section that follows. Table 7-1 summarizes the gradation (particle size) requirements for processed aggregate for the RCC dam.

7.2.3.1 On-Site Sources - Eleven discrete borrow areas, including rock quarries, and required excavation for the dam and access road, have been identified as potential sources of aggregate for conventional concrete and RCC. The Geologic Map, Drawing 4-1, illustrates the geologic conditions described in detail in Section 4.0 and shows the location of all subsurface investigations to date.

The borrow areas, access road, and dam footprint locations are shown in Drawings 3-1. Sections of the seismic refraction surveys are included in Appendix A. Field investigations are described in Section 3.0 and detailed logs of the boreholes and test pits are included in Appendix B. Laboratory test results are summarized in Section 6.0 and data is included in Appendix C.

Table 7-2 summarizes the availability (gross volume) and product volume of on-site construction materials identified by the current investigation. The following are descriptions of each potential borrow area:

Borrow Area A

Borrow Area A covers about 12 acres on the left bank of the Carmel River approximately 0.1 miles upstream of the proposed dam axis. Borrow Area A, previously known as Left Barnes Flat, was investigated in 1948 as a potential source of impervious fill for construction of the existing Los Padres Dam. Excavation of three test pits revealed alluvial fan deposits ("clayey loam w/ talus fragments") ranging from 3- to 8-ft thick, overlying an unknown thickness of alluvial terrace gravel. The area was not developed for Los Padres Dam.

In 1992, Bechtel investigated Borrow Area A through geologic mapping and excavating seven test pits. These pits showed that the alluvial fan material ranged from 2- to greater than 15-ft thick, and the terrace gravel to be greater than 12-ft thick. Samples of the terrace gravel were obtained and tested in the laboratory. Table 4-1 summarizes the Borrow Area A subsurface investigations. Assuming excavation of the granitic bedrock under the terrace deposits to El. 900 feet, the gross volumes of construction materials available from Borrow Area A are as shown in Table 7-2.

The silty sand alluvial fan material was similar to the material tested in the laboratory by Bechtel. It is unsuitable for use as RCC aggregate and will have to be stripped and wasted to expose the underlying terrace gravel deposit. It is possible that a small percentage of this material could be blended with higher quality aggregate as a non-plastic filler.

The terrace gravel and boulders were sampled and tested, and found to be of acceptable quality for crushing and screening into aggregate for RCC or conventional concrete. Aggregate test results are discussed in Section 6.0. The alluvial terrace gravel can be described as silty sandy gravel (GP), light brown, moist, poorly-graded, composed of approximately 20% boulders, 35% cobbles, 20% gravel, 20% sand, and 5% silt, sub-round to round, and characterized by clasts of granite, gabbro, and schist. These terrace deposits vary from 11 to 16 feet thick.

The bedrock encountered consisted of granodiorite and diorite. It ranged from highly weathered to fresh. Core recovery averaged 74% and RQD averaged 34%. Suitable RCC aggregate can be processed from quarrying the bedrock, but about 25% sand is expected to result, which in turn will require that a significant portion of the material excavated will have to be wasted.

Borrow Area B

Borrow Area B covers approximately 12.6 acres on the right bank of the Carmel River just upstream of Borrow Area A and about 0.4 miles upstream of the proposed dam. Borrow Area B, previously known as Right Barnes Flat, was the primary source of impervious material (primarily silty sand) for construction of Los Padres Dam in 1948-49. This area was investigated in 1948 by excavating six test pits. The pits showed that the alluvial fan material ranged from 5-ft to greater than 25-ft thick. Three pits did not reach the terrace gravel deposit. Bedrock was only reached in one pit where the terrace gravel deposit was 4-ft thick. In the other two pits the terrace gravel exceeded 5-ft in thickness.

In 1992, Bechtel excavated two test pits in Borrow Area B. These pits showed that the thickness of the remaining alluvial fan material ranges from 1.5- to 3-ft thick, with the terrace gravel deposits to be greater than 12-ft thick. Neither of the Bechtel pits reached bedrock. Samples of the gravel were obtained and tested in the laboratory.

Table 4-1 summarizes the results of the Borrow Area B subsurface investigations. The subsurface investigation conducted for the current study showed that the overlying alluvial fan material ranges from 0- to 4-feet thick, except at the northeast end of Borrow Area B, where one pit showed a thickness of 14.5-feet. Terrace gravels were encountered in every pit and bedrock was encountered in all but two test pits. The terrace gravel deposits ranged in thickness from 10- to greater than 15-feet. The gross volumes of construction materials available from Borrow Area B are as shown in Table 7-2.

The silty sand alluvial fan material was similar to the fan material observed in Borrow Area A and is unsuitable for use as RCC aggregate. In addition, the volume is small and therefore will have to be wasted to uncover the underlying terrace gravel deposit.

The terrace deposits were sampled and tested, and found to be similar to those in Borrow Area A. Therefore, these deposits are an acceptable source of material for crushing and screening into aggregate for RCC or conventional concrete. Test results are discussed in Section 6.0.

The bedrock underlying the Borrow Area B terrace gravel deposit primarily consists of highly weathered metasedimentary rock that is unsuitable for use as concrete aggregate.

Borrow Area C

Borrow Area C covers approximately 3.2 acres on the right bank of the river opposite Borrow Area A, about 0.1 miles upstream of the dam axis. Table 4-1 summarizes the results of the Borrow Area B investigations. The investigation showed that the area consists of terrace gravel exposed at the surface with the exception of an approximately 4-foot thick apron of colluvium at the base of the ascending steep rock slope. Terrace gravel deposits were encountered in every pit and ranged from 9-feet to greater than 18-feet thick. The underlying bedrock consists of sound granite. The gross volumes of construction materials available from Borrow Area C are as shown in Table 7-2.

The silty sand alluvial fan/colluvial material was similar to that identified in other locations at the site and is unsuitable for use as RCC aggregate. In addition, the volume is too small to be of any value and, therefore, will have to be wasted to uncover the underlying terrace gravel deposit.

The test pits revealed the alluvial terrace deposits to be somewhat coarser (a high percentage of boulders) than those in Borrow Areas A and B. Borrow Area C is an excellent source of material for crushing and screening into aggregate for RCC or conventional concrete. Additional gravel materials are also available from the river area which could increase the available volume by approximately 25,000 cy.

The bedrock mapped underlying the terrace gravel deposit and encountered in the test pits is fresh granodiorite and diorite; however, because of the low elevation at which it occurs (below the level of the river), it is not considered feasible as a source of quarry rock.

Borrow Area D (Required Excavation)

Borrow Area D covers approximately 11.9 acres and comprises the footprint of the RCC gravity dam. The Bechtel investigation (Bechtel, 1992) of the dam foundation included geologic mapping, seismic refraction surveys, three boreholes, and two test pits. The investigation for the current study consisted of geologic mapping, three seismic refraction survey lines, and one 85-foot borehole in the right abutment area. Table 4-1 summarizes the results of the Borrow Area D investigations. The gross volumes of construction materials available from required foundation excavation are as shown in Table 7-2.

The silty sand alluvial fan material derived from the required dam foundation excavation was tested by Bechtel. It was found to be unsuitable for use as RCC aggregate except for a small percentage which might be used as a blended as fine aggregate filler. The majority of the material will have to be hauled upstream to waste in the reservoir.

The terrace gravels and boulders are expected to be of acceptable quality similar in nature to the alluvial terrace deposits in borrow areas A and B. These terrace deposits are approximately 15 feet thick in the right abutment and approximately 10 feet thick in the left. Terrace gravel from required excavation could be crushed and screened into aggregate for RCC or conventional concrete. If considered economical, the required excavation of terrace gravel and alluvial fan overburden on the right abutment could be expanded to cover all of Borrow Area D.

The weathered upper portion of the bedrock encountered in the boreholes was characterized by a core recovery of 79% and an RQD of 29%. It is estimated that about 50% of this material would be hauled to waste. The remainder could be processed into about 60% coarse aggregate and 40% fine aggregate.

If considered economical, sound granitic rock underlying the weathered rock could be quarried from the rock cliffs above Borrow Area C on the south side of Borrow Area D and adjacent to Borrow Area E (Quarry A).

Borrow Area E (Quarry A)

Borrow Area E covers approximately 4.0 acres just upstream of the left abutment of the dam. Borrow Area E was identified in Bechtel's 1992 report as Quarry A, but was not investigated at that time. The gross volumes of construction materials available from Borrow Area E are as shown in Table 7-2.

Because of its location on the reservoir rim, final post-excavation slopes in Borrow Area E will have to be engineered to be stable. Drawing 7-1 illustrates the anticipated final slopes and Drawing 3-1 shows the estimated area available as useable construction material.

The seismic refraction surveys (Appendix A) indicate that the bedrock is closely fractured and/or highly weathered to a depth from 30 to 50 feet. Excavation of this material, together with the underlying sound rock, is expected to result in about 30% waste. The processed rock is expected to generate approximately 70% coarse aggregate and 30% fine aggregate.

Borrow Area F (Quarry B)

Borrow Area F covers approximately 3.4 acres just upstream of the right abutment of the dam and due east of Borrow Area C. In the Bechtel report (Bechtel, 1992), Borrow Area F was identified as Quarry B, but was not investigated. The gross volumes of construction materials available from Borrow Area F are as shown in Table 7-2.

Because of its location on the reservoir rim, final post-excavation slopes in Borrow Area F will have to be engineered to be stable. The slopes of Borrow Area F would be similar to those shown for Borrow Area E in Figure 7-1.

Approximately 30% of the bedrock in Borrow Area F is expected to be friable metasedimentary rock. This material is considered unsuitable for processing into concrete aggregate and will have to be wasted. Processing the sound granitic rock is expected to result in approximately 70% coarse aggregate and 30% fine aggregate some of which may have to be wasted.

Borrow Area G

Borrow Area G covers approximately 6.9 acres due west of Borrow Area A. The 1992 Bechtel report did not identify or investigate Borrow Area G. Table 7-12 summarizes the results of the Borrow Area G investigations. The gross volumes of construction materials available from Borrow Area G, assuming that sound bedrock is quarried to El. 900 feet, are as shown in Table 7-2.

This investigation showed that the area is underlaid by rocky alluvial fan material ranging to more than 40 feet thick, over approximately 25 feet of alluvial terrace gravel and boulders. The terrace deposits, in turn, overlie sound granitic bedrock. No samples were collected from the test pits in Borrow Area G.

The rocky sand alluvial fan material could probably be processed into suitable fine aggregate; however, there will more than likely be a surplus of fine aggregate from processing materials from other sources. It is expected that only a small percentage of the material in Borrow Area G could be utilized. While the terrace gravels and boulders beneath the alluvial fan deposits were not sampled and tested, they are expected to be similar in quality to those from borrow areas A and B.

The bedrock underlying the terrace gravel deposit is granitic and should produce about 70% coarse aggregate. The remaining 30% fine aggregate would have to be blended with other fine aggregate to meet the desired gradation.

Borrow Area H

Borrow Area H covers approximately 5.7 acres on the left bank of the river immediately upstream of Borrow Area G, about 0.3 miles upstream of the dam axis. The area was one of the principal sources of impervious borrow material for the 1948-1949 construction of the existing Los Padres Dam.

This area was investigated by geologic mapping and two test pits. Table 4-1 summarizes the results of the Borrow Area G investigations. The gross volumes of construction materials available are as shown in Table 7-2.

This investigation showed the area to be partially covered with silty sand alluvial fan material of unknown thickness remaining from the previous construction activity. In other areas this material has been completely removed exposing the top of weathered granitic bedrock. The silty sand from Borrow Area H was tested by Bechtel and is not suitable for use as concrete aggregate.

Borrow Area I

Borrow Area I covers approximately 5.0 acres on the left (west) side of Los Padres Reservoir approximately 0.1 miles upstream of the existing dam and 0.6 miles upstream of the proposed dam. This area (previously known as Martin's Flat) was investigated in 1948 for the existing Los Padres Dam through four test pits. This borrow area was one of the primary sources of impervious material used in construction of the dam. Stripping of the fine grained fan material for use in the dam uncovered the top of the terrace gravel deposit exposed today.

The 1992 Bechtel report did not identify or investigate Borrow Area I. The gross volumes of construction materials available from Borrow Area I are as shown in Table 7-2.

Less than 1-foot of sandy silt reservoir sediment will have to be stripped from the top of the terrace gravel deposit and wasted. The terrace gravels and boulders are expected to be similar in quality to those of Borrow Areas A and B.

Bedrock underlying the terrace gravel deposits appears to be granitic in nature but is located at such a low elevation relative to the Los Padres Reservoir that excavation is not considered practical.

Borrow Area J

Borrow Area J covers an area of approximately 5.1 acres immediately southwest of Borrow Area I.

The 1992 Bechtel report did not identify or investigate Borrow Area I. The gross volumes of construction materials available from Borrow Area I are as shown in Table 7-2.

Topographically the area is similar to Borrow Area A, and it can be expected that the material characteristics are also similar. It has been assumed that the alluvial fan material averages 30-feet thick, the thickness of terrace gravels is equal to Borrow Area A (15-feet), and that bedrock could be excavated to El. 1,010 feet.

Reconnaissance of the reservoir area indicates that additional terrace gravel deposits exist further upstream in the reservoir on the right bank of the river.

Borrow Area K (Required Access Road Excavation)

Construction of the access road from the dam to the downstream migrant fish facility requires excavation of approximately 175,000 cy of closely fractured and/or highly weathered bedrock. It is estimated that about 30% of this volume could be processed into concrete aggregate. The remaining material would be hauled to waste within the reservoir area. The required access road excavation has been designated Borrow Area K.

The gross volumes of construction material available from Borrow Area I are as shown in Table 7-2.

7.2.3.2 Off-Site Sources - Cement, fly ash, and in the early stages of mobilization, select aggregate and ready-mix concrete will have to be obtained from off-site sources. The locations of these sources are described as follows:

Ready-Mix Concrete

Ready-mix concrete will be required before the on-site batching facilities are mobilized. This material is available from a number of commercial plants in Monterey County. These sources include Granite Construction Co. in Greenfield, 35 miles from the site; Granite Rock in Salinas and Seaside, 25 and 22 miles, respectively, from the site; RMC Lonestar in Salinas; and Monterey Concrete in Seaside.

Aggregate

Crushed rock for road base and/or miscellaneous construction use before the on-site crushing and screening facilities are mobilized is available from Granite Construction Co. in Greenfield, and Granite Rock in Salinas and Seaside.

Cement

Type II and II Low Alkali Portland Cement for use in conventional and RCC mixes, respectively, can be supplied by Kaiser Cement of Pleasanton, California. The cement would be transported by truck directly to the damsite from the Kaiser Permanente plant in Cupertino, California, a distance of 120 miles.

Another option is to haul the cement by rail from Cupertino to Seaside or Salinas, and to then haul by truck to the site.

Fly Ash

Type "F" fly ash suitable as a partial substitute for Portland Cement in RCC mixes can be supplied by Western Ash of Sacramento, California from a number of sites in the western United States. The ash would be delivered by rail from ash producers in Nevada and Arizona to Salinas, and then trucked to the site.

There is some question as to the capacity of the ash producing units to meet the demands of the project, therefore, it will be important to notify these sources of tonnage requirements well in advance of construction so that deliveries can be scheduled and delays avoided.

There is an inactive source of natural pozzolan at Hallelujah Junction, Nevada that was used on a dam in northern California, but presently this site is not being operated.

7.2.4 Conclusions

Sufficient amounts of suitable construction materials are available on-site, upstream of the dam, and in the required excavations for the dam and access road, to meet the conventional concrete and RCC aggregate requirements for the dam and appurtenant structures. A gross volume of approximately 1.5 to 2 times that required has been proven.

The most economical materials appear to be the alluvial terrace gravels in borrow areas A, B, C, D (required dam excavation), and I. Assuming a loss factor of 15% during excavation, hauling, stockpiling, re-excavation, and processing, it is estimated that 666,000 cy of suitable concrete aggregate could be processed from these deposits. The remaining required 219,000 cy could be processed from rock excavation for the dam, in Borrow Area A, and access road construction. Exploration of rock sources from other borrow areas is not anticipated, but sources are available if the need arises.

High quality aggregate to supplement on-site sources is available within a reasonable haul distance from the site as is ready-mix concrete. Cement and pozzolan (fly ash) can be supplied from off-site sources at reasonable costs.

7.3 Dam Foundation

7.3.1 Right Abutment

Excavation for dam foundation is estimated to vary from 20 to 80 feet on the right abutment. As shown on Drawing 7-3 the alluvial fan material, terrace gravels, and weathered rock materials will be excavated. Water pressure tests performed on bedrock materials of RA-2 indicated relatively low water loss and are consistent with water loss tests made by Bechtel in Boring RA-1. The largest water losses occurred in the upper ten feet of bedrock materials. A relatively large amount of excavation is planned at the right abutment between El. 950 to El. 1000 (near the river) to remove weathered rock material as shown in Drawing 7-2. The excavated slope of the alluvial fan materials was assumed to be 1.5H:1V and the weathered rock materials 0.1H:1V for this study.

7.3.2 Center Section

The middle (river) section is approximately 200 feet in width along the proposed dam axis. Water pressure tests performed on bedrock material by Bechtel in Boring C-1 generally indicated a relatively low water loss. The largest water loss was 1.4 gallons per minute at a depth of 142 to 152 feet. Excavation in this area is estimated to average approximately 10 feet. River gravels and boulders are to be excavated to the bedrock foundation.

7.3.3 Left Abutment

Excavation for dam the foundations is estimated to vary from 20 to 70 feet on the left abutment. Water pressure tests performed on bedrock materials by Bechtel in Boring LA-1 indicated a relatively low water loss. The largest water logs was 1.0 gallon per minute at a depth of 90 to 100 feet. The depth of excavation is shown on Drawing 7-3. It is anticipated that soil and weathered rock will be excavated to depths of 20 feet below El. 950 and up to 80 feet above El. 1,000. The excavated cut slope through rock and weathered rock materials has been assumed to be 0.1H to 1V (horizontal: vertical) and the excavated slope for soils has been assumed to be 1H:1V for the studies. Stability analyses of the excavated slopes were not performed as part of this investigation.

7.3.4 Conclusions

All alluvial fan material, terrace gravels and weathered rock should be excavated from dam foundations. Additional borings, test trenches and seismic refraction survey lines should be performed to further refine the depth of excavation in the abutments and the river area. During construction of the existing Los Padres Dam, a buried channel was encountered on the right abutment at the spillway. The investigations performed during final design should be in sufficient detail to define the weathered rock surface, depth of weathering, areas of potential water loss for grouting and possible areas of buried channel. No eroded areas (buried stream channels) were noted in the abutments at the New Los Padres Dam site by geologic mapping, borings information or seismic refraction survey data.

TABLE 7-1: TYPICAL RCC AGGREGATE SPECIFICATION

New Los Padres Water Supply Project
Monterey County, California

Sieve Size	Percent Passing
2 1/2"	100
2"	95 - 100
1"	70 - 90
3/4"	60 - 85
3/8"	45 - 60
No. 4	30 - 45
No. 8	25 - 35
No. 16	17 - 27
No. 30	10 - 20
No. 50	8 - 18
No. 100	5 - 15
No. 200	2 - 10

TABLE 7-2: ON-SITE CONSTRUCTION MATERIALS
New Los Padres Water Supply Project
Monterey County, California

VOLUME REQUIRED

Destination	Concrete Aggregate Net Volume (x1000cy)	
	Coarse	Fine
RCC Dam and Appurtenant Structures	535	350

VOLUME AVAILABLE

Source/ Borrow Area	Description*	Gross Volume (x1000 cy)	Product** (cy)		
			Concrete Aggregate (x1000)		Waste (x1000)
			Coarse	Fine	
A	SS	120			120
	TG	311	187	124	
	R	747	448	299	
B	SS	31			31
	TG	353	212	141	
C	SS	8			8
	TG	75	45	30	
D (Dam)	SS	84			84
	TG	45	27	18	
	R	432	130	86	216
E	SS	42			38
	R	296	178	118	
F	SS	28			28
	R	481	202	135	144
G	SS	467			467
	TG	233	140	93	
	R	1,111	667	444	
H	n/a				
I	SS	8			8
	TG	81	49	32	
J	SS	288			288
	TG	123	74	49	
	R	330	198	132	
K (Access Road)	R	175	52	35	88
TOTAL		5,869	2,609	1,706	1,521
SURPLUS OR (DEFICIT)			2,074	1,356	1,521

* SS=Silty Sand, TG=Terrace Gravel, and R=Rock

** No allowance has been made for loss during excavation, hauling, handling, and processing.

8.0 PRELIMINARY ANALYSIS OF RCC DAM

8.1 Purpose of Analysis

The New Los Padres damsite is located in an area of high seismic activity with the potential for earthquakes generating peak horizontal accelerations exceeding 0.5g. Anticipated stress levels induced in the proposed dam under seismic loading have a significant influence on the design configuration and the concrete design tensile strengths. A preliminary seismic analysis was, therefore, performed to estimate the levels of induced tensile stresses that could be expected and their distributions within the dam cross section. This information is necessary to establish the zones where the high tensile strength concrete will be required and for selecting the design mixes for the various zones. The tensile strength requirement has a significant effect on the required cementitious material content of the mixes which in turn affects the cost of the RCC.

8.2 Method and Criteria for Analysis and Evaluation

8.2.1 Method of Analysis

The analysis was performed using the University of California, Berkeley computer program EAGD-84 (UCB,1984). This program performs two-dimensional (2-D) finite element static and dynamic response (seismic) analyses of gravity dams. The analyses include the effects of dam-water-foundation rock interaction and of materials such as alluvium and sediments at the bottom of reservoirs. The hydrodynamic effects of impounded water are modeled by the 2-D wave equation and water compressibility effects are included in the analysis. Uplift forces are not accounted for by the program and are introduced by manual calculation during the stability evaluation. The system is analyzed assuming linear behavior for dam, impounded water, and foundation rock. The program computes static stress distributions for static loading conditions, and dynamic time-history stress distributions for loading consisting of pressure due to normal reservoir level, dead load of concrete, and accelerograms for horizontal and vertical components of the seismic ground motion applied simultaneously.

8.2.2 Geometry for Analysis

The basic geometry of the section analyzed is shown in Drawing 8-1. Using this basic geometry, a 2-D finite element mesh layout was developed for the 2-D FEM analysis. The section was divided into a mesh of quadrilateral and triangular elements. Finite elements representing the foundation rock are not necessary for the EAGD-84 computer program. The effects of dam-foundation interaction are included in the analysis automatically by the program using the input foundation materials properties. The only restriction is that the element faces modeling the concrete-foundation interface must be level and of equal length.

The mesh layout developed for the 2-D FEM analyses of the overflow dam cross section is shown on Drawing 8-2. The model consists of 121 quadrilateral elements and 8 triangular elements connected by 148 nodal points. The triangular elements, which have been shown to be less accurate than rectangular elements, are used in transition areas and are located in regions where stress gradients and stress values are expected to be low.

8.2.3 General Data and Analysis Criteria

The following data and general criteria was used in the analysis:

Basic Elevations

- Crest of Non-overflow Dam Section - El. 1,142
- Base of Non-overflow Dam Section - El. 860
- Ogee Spillway Crest - El. 1,130
- Base of Spillway Section - El. 860
- Normal Full Pool Level - El. 1,130
- PMF Pool Level - El. 1,142
- Tailwater Level at Minimum Discharge - El. 867

Static Material Properties:

The material properties used in this preliminary analysis are based on the limited foundation rock and RCC test data available, and on experience from similar projects and similar materials. These properties are as follows:

> Compressive Strength: Static compressive strengths are assumed to be 3,500 psi for the high strength RCC zone (Zone 1) and 2,000 psi for the remainder of the RCC (Zone 2);

Tensile Strength: The static splitting tensile strength was estimated by the following relationship by Raphael(Raphael, 1984):

$$f_t = 2.3\ f_c^{2/3}$$

Where: f_t = splitting tensile strength

f_c = static compressive strength

using average values of f_c of 3,500 psi for Zone 1 and 2,000 psi for Zone 2 yields corresponding values of 530 psi and 360 psi for the static splitting tensile strength;

Modulus of Elasticity: The modulus of elasticity values used in the analysis for static loading conditions are representative of the sustained modulus of elasticity. These values are assumed to be 2/3 of the average test values of the modulus of elasticity. These average test values are based on the standard ACI Building Code relationship.

$$E_c = 57{,}000 \times f_c^{1/2}$$

Where f_c = The required compressive strength of concrete.

Based upon assumed values of f_c of 3,500 psi for the high strength zone (Zone 1) and 2,000 psi for the remainder of the concrete (Zone 2), the corresponding values of E_c are 3,400,000 psi and 2,550,000 psi, respectively. Accordingly, the corresponding values for sustained modulus of elasticity for Zones 1 and 2 are 2,270,000 psi and 1,700,000 psi, respectively.

Dynamic Material Properties

Dynamic elastic and strength properties of concrete were estimated on the basis of test values under static (normal) loading conditions and on well-established relationships between static and dynamic test values. In recent years, a large number of tests conducted under rapid-rate loading have been performed on concrete cores drilled from several concrete dams. Comparison of dynamic elastic and strength test values with those measured under slow-rate loading tests indicated a very consistent relationship between the static and dynamic test values. Thus, rapid rate load test data have indicated increases of 20 percent in the modulus of elasticity, 30 percent in the compressive strength and 30 percent to 50 percent in the splitting tensile strength observed under slow rate of loading. Dynamic flexural strength was indicated to be 30 percent higher than the dynamic splitting tensile strength. These findings are described and discussed in two papers (Raphael, 1984) appearing in the March-April 1984 ACI Journal entitled "The Nature of Mass Concrete in

Dams", and "Tensile Strength of Concrete." The values of dynamic strength properties for the seismic analysis and evaluation of New Los Padres Dam were, therefore, estimated from assumed static values using the above-mentioned relationships between static and dynamic properties. These values are as follows:

Dynamic Compressive Strength - 30 percent higher than static.

Zone 1 (High Strength Zone) -	4,500 psi
Zone 2 (Remainder of Mass RCC) -	2,600 psi

Dynamic Splitting Tensile Strength - The dynamic splitting tensile strength was estimated by the following relationship by Raphael (Raphael, 1984):

$$f_t = 2.6\ f_c^{2/3}$$

Where:
f_t = dynamic splitting tensile strength
f_c = static compressive strength

Using average values of f_c of 3,500 psi for Zone 1 and 2,000 psi for Zone 2 yields corresponding values of 600 psi and 410 psi for the dynamic splitting tensile strength.

Dynamic Flexural Strength - This value was estimated using the following relationship given (Raphael, 1984).

$$f_t = 3.4\ f_c^{2/3}$$

This relationship yields dynamic flexural tensile strength values of 780 psi and 540 psi for Zones 1 and 2, respectively.

Dynamic Modulus of Elasticity - 20 percent higher than static values:

Zone 1 (High Strength Zone) -	4,100,000 psi
Zone 2 (Remainder of Mass RCC) -	3,100,000 psi

Concrete/Rock Interface Properties

Friction Angle -	45 Degrees
Unit Cohesion -	10 percent of f_c*:

Static - 200 psi
Dynamic - 260 psi

* Conservatively assuming Zone 2 RCC over entire base.

Other Criteria and Assumptions:

- Unit weight of concrete - 150 pcf
- Poisson's ratio of concrete - 0.20
- Mass modulus of elasticity of foundation rock - 3,500,000 psi
- Poisson's ratio of rock - 0.30
- Unit weight of water - 62.5 pcf
- Earthquake loading as represented by the accelerograms for the horizontal and vertical components of ground motion representing the design earthquake discussed in Sections 3.0 and 5.0. The time interval and duration for the time-history response for the dam were the same as the input accelerograms.
- Damping ratio - 5 percent
- Concrete in the dam is homogeneous and uniformly elastic in all directions.
- The dam is elastically bonded at the foundation.
- Uplift - varying linearly from full reservoir pressure at the upstream face to a value equal to tailwater pressure plus 1/3 of the difference between reservoir pressure and tailwater pressure at the drain location and then linearly to tailwater pressure at the downstream face.

8.2.4 Loading Conditions

Usual (Static) Loading Condition:

Reservoir at normal full pool level El. 1,130, dead weight of concrete, foundation interface hydrostatic uplift pressure.

Extreme (Seismic) Loading Condition:

Reservoir at normal full pool level El. 1,130, dead weight of concrete, foundation interface hydrostatic uplift pressure, combined with dynamic earthquake loading corresponding to the design earthquake accelerograms.

8.3 Analysis

8.3.1 General Approach

The EAGD-84 program first performs a static analysis, followed by calculation of the dynamic properties of the dam. The dynamic response analysis consists of a time-history analysis using the computed vibration characteristics of the dam and the design earthquake horizontal and vertical accelerograms. The results of the static analysis consist of deflections computed at each nodal point and normal, shear, and principal stresses computed at the center of each element. The results of the dynamic analysis consist of natural frequencies and mode shapes for the dam and the dynamic response in terms of deflections and stresses (combined static and dynamic) at specified time intervals during the earthquake. A table of the maximum and minimum values of dynamic deflections and principal stresses and their times of occurrence is included in the output.

8.3.2 Natural Periods of Vibration

The first ten modes of vibration were considered adequate for evaluating the dynamic response of New Los Padres Dam.

The computed periods for the first 10 modes are as follows:

Mode No.	Period (seconds)
1	0.28
2	0.16
3	0.12
4	0.07
5	0.05
6	0.04
7	0.04
8	0.03
9	0.03
10	0.03

8.3.3 Stress Distributions

The results of the static analysis are shown on Drawing 8-3 and the results of the dynamic analysis are shown on Drawings 8-4 through 8-6. Static stress distributions are shown in the form of vertical stress contours plotted on the cross section. Dynamic stress contours shown are envelopes of maximum tensile (Drawing 8-4) and compressive (Drawing 8-5) stresses (combined static and dynamic) occurring during the earthquake and the vertical stress distribution occurring at 11.30 seconds, the most critical time instant (Drawing 8-6). These stresses were computed without uplift. The vertical stresses at the concrete-rock interface at the base of the cross section have been adjusted for uplift and linear plots of the resulting stresses are plotted below the appropriate cross sections shown on the drawings.

8.3.4 Concrete-Rock Interface Stability

The shear-friction factor of safety (SSF) at the interface was computed as the ratio of the total stabilizing forces to the total sliding forces in accordance with the following equation:

$$SSF = [C \times A + f \times (N - U)]/V$$

where:

C = Unit cohesion
A = Base area of section considered (uncracked portion as appropriate)
f = Coefficient of internal friction
N = Resultant of normal (vertical) forces
U = Total uplift forces
V = Resultant of horizontal (shearing) forces

The resultant normal (vertical) and horizontal forces were estimated by integrating the base vertical and shearing stresses across the base calculated by the 2-D finite element analysis over the area of the base. No vertical tensile stresses were indicated under static conditions and the shear friction factor was assumed considering the total area of the base.

The critical sliding factor under earthquake conditions occurs at the instant when the resultant horizontal force is maximum. This critical instant was found to be during the 11.30 second time interval. For earthquake conditions, areas of the base where tension is indicated were assumed cracked to the point of zero tension and the shear friction factor was estimated for the uncracked portion of the base. It is not necessary to increase uplift pressure in the crack occurring under dynamic conditions since the loading is instantaneous and full uplift pressure in the crack would not have time to develop.

The computed shear friction factors of safety under static and dynamic conditions are shown on Drawings 8-3 and 8-6 respectively.

8.4 Evaluation of Results

8.4.1 Evaluation Criteria

The structural stability of the dam for the static and dynamic loading conditions was evaluated on the basis of permissible stresses and other criteria in accordance with the current practice and as stipulated in the USBR Monograph "Design Criteria for Concrete Arch and Gravity Dams", Second Revision, 1974, (USBR, 1974). These criteria are as follows:

8.4.1.1 Permissible Stresses:

1. Usual (Static) Loading Conditions
(Reservoir water surface El. 1,130):

Based on static unconfined compressive strength,
f_c = 3,500 psi and 2,000 psi for Zones 1 and 2, respectively.

Maximum compressive stress = $f_c/3$
Zone 1 - 1,150 psi
Zone 2 - 660 psi

Maximum tensile stress = 10 percent of f_c
Zone 1 - 530 psi
Zone 2 - 360 psi

2. Extreme Loading Condition
(Normal reservoir water surface El. 1,130 plus design earthquake:

Based on dynamic compressive strength,
f_{cd} = 4,500 psi and 2,600 psi for Zones 1 and 2, respectively.

Maximum compressive stress = 0.8 f_{cd}
Zone 1 - 3,600 psi
Zone 2 - 2,160 psi

Maximum flexural tensile stress = minimum dynamic flexural strength

Zone 1 - 780 psi
Zone 2 - 540 psi

8.4.1.2 Sliding Stability - The shear friction factor was evaluated against the following minimum criteria values:

Loading Condition	Minimum SSF
Usual Loading (static)	3.0
Extreme Loading (Seismic)	1.1

8.4.2 Evaluation

8.4.2.1 Stresses - Observation of the analytical results indicates that stress levels under static conditions are very low with the maximum principal compressive stress being 231 psi. No principal tensile stresses are indicated. Static stresses, therefore, have no significance on the mix design. Static stress criteria would be satisfied with any reasonably designed concrete mix.

The results of the analysis indicate that the maximum principal stresses under earthquake conditions (static plus dynamic) range from 1,108 psi compression to -756 psi tension. These stress levels can be accommodated by proper mix design and zoning of the RCC in the dam (See Section 8.5). In areas where high vertical tensile stresses are indicated, special treatment will have to be specified for the horizontal lift joints. This treatment would include a mortar bedding mix applied between the lifts and special clean-up requirements for the lift surface when temperature exposure limitation criteria is exceeded. The cement content and stresses for Zone 1 and Zone 2 are discussed in Section 8.5 RCC Cement Content.

8.4.2.2 Sliding Stability at Concrete/Rock Interface - The calculated shear friction factor for usual (static) loading condition is 4.57. This value is higher than the minimum criteria value of 3.00 for the normal operating condition. Calculations indicated that the minimum criteria value would be satisfied even for a cohesion value as low as 90 psi, the criteria cohesion value being 200 psi.

The calculated shear friction factor for extreme (earthquake) loading conditions is 1.25. This value was calculated using only the assumed uncracked area of the base. This value is higher than the minimum criteria value of 1.1 for the extreme loading condition. Calculations indicated that the minimum criteria value would be satisfied even for a cohesion value as low as 195 psi, the criteria cohesion value being 260 psi.

8.5 RCC Cement Content

The envelope of maximum dynamic tensile stresses shown in Drawing 8-4 was used to establish the extent of the zone of high strength RCC (Zone 1) at the dam base and near the upstream and downstream faces. The required dynamic tensile strength of Zone 1, based upon the maximum indicated tensile stress, is 756 psi. Using the relationships between static and dynamic strength discussed in Section 8.2, the corresponding static compressive strength is 3,300 psi. This value is close to the assumed compressive strength value of 3,500 psi which is the basis for the established analysis and evaluation criteria. To provide an extra degree of conservatism at this preliminary stage of design, an average value of 4,000 psi for one-year compressive strength will be assumed for Zone 1. The proposed cross-sectional configuration of the zones is shown in Drawing 8-1.

Total cement contents required to produce RCC that will meet the above requirements were estimated from plots of strength gain versus time developed during the current preliminary trial mix program (Section 6.0). The cement contents are 450 lbs/cy and 180 lbs/cy for Zones 1 and 2, respectively. These estimated cement contents are considered conservative because the use of larger MSA (2 to 3-inch) in construction rather than the small (1-inch) MSA used in the testing program, is expected to yield the same strengths at a lower cement contents.

In order to minimize buildup of heat within the RCC mass due to hydration of cement, Type II (low heat) cement should be specified. To further control heat and to reduce costs, while maintaining long term strength, it is common practice to replace a portion of the cement with pozzolan. The only commercially available pozzolan in the western United States is Type F (low loss on ignition) fly ash which is produced as a by-product at several coal-fired thermal electric plants in Nevada and Arizona. Although the current preliminary trial mix program did not test mixes containing fly ash, based on industry experience it is assumed that approximately 30 percent fly ash can be substituted for portland cement.

Table 8-1 shows the estimated weight per cubic yard of the cementitious materials ingredients of the proposed RCC mixes.

9.0 REVIEW OF PRELIMINARY DESIGN

9.1 General

The preliminary design of the dam and appurtenances by Bechtel (Bechtel, 1989) was reviewed and the design was modified to reflect the results of new investigations described in this report. The capacity of the reservior is 24,000 acre-feet at El. 1130. Major emphasis has been given to those items or features which have the most influence upon project costs. The principal areas of interest were the dam location, layout and configuration (including internal RCC zoning requirements), spillway design, configuration of outlet works, migrant fish collection facilities, and access roads.

9.2 Dam Design Modification

The preliminary Bechtel dam design consisted of a curved gravity RCC structure. The curved configuration was proposed to provide a structure with inherently greater resistance to seismic loading than a straight gravity dam, thereby, increasing the margin of safety. In relatively steep V-shaped canyon sites, the curved concept would significantly enhance the dam stability; however, the New Los Padres site cross-stream profile is relatively wide with a length to height ratio well over five. Review of the site conditions indicate that with this relatively wide canyon profile, the advantage of a curved configuration for improving seismic stability is marginal. Because excavation and RCC quantities for the curved dam would be significantly greater than for a standard straight gravity dam, the preliminary design has been modified to show a straight gravity structure. A number of other changes and refinements to the preliminary design have also been made for the purposes of developing a cost estimate.

The proposed plan for New Los Padres Dam is shown on Drawing 9-1, and typical cross sections and area-capacity curves are shown on Drawing 9-2. The height of dam is estimated to be 282 feet. This height of dam is different from 274 feet stated in the EIS. because of additional assumed foundation excavation and an increase in crest elevation from El. 1,140 to El. 1,142.

9.2.1 Excavation and Foundation Treatment

The dam axis was shifted to take maximum advantage of the newly developed topographic and subsurface information. Geotechnical investigations indicate that excavation depths will be greater than those estimated in the preliminary design. Excavated depths, based upon the latest subsurface investigations, will vary from about 10 feet in the stream bed to up to 50 to 70 feet at several locations in both abutments.

Foundation treatment will consist of thorough cleaning of the excavated surface of all loose and semi-detached pieces of rock by high pressure water jet, vacuum cleaning or other suitable means, dental excavation and concrete backfill (where necessary), and consolidation grouting. For estimating purposes, consolidation grout holes are assumed to be 15 feet deep and drilled in a grid pattern spaced at 10-foot centers each way. A grout curtain will extend from the foundation gallery into the foundation. The grout curtain holes will be spaced at 10-foot centers and will extend for a depth equal to at least 2/3 of the dam height at the hole location with a 50 foot minimum depth at the abutments. A foundation drainage curtain located downstream of the grout curtain consisting of holes drilled at 10-foot centers will extend from the gallery into the foundation for a depth equal to 0.5 of the dam height at the hole location.

9.2.2 Dam Cross Section

The basic cross section of the dam will be trapezoidal with a vertical upstream face and the downstream face sloping at 0.80 horizontal to 1.00 vertical (H:V). The upstream and downstream face planes intersect at the dam crest El. 1,142 as shown on Drawing 8-1. A 16-foot wide rectangular section at the crest of the non-overflow section provides for the access roadway across the crest. An ogee crest spillway as discussed below (Section 9.4) is provided in the central part of the dam.

A drainage system will be provided in the interior of the dam by drilled drain holes extending upward from the gallery to near the top of the dam. The holes will be spaced at 10-foot intervals along the longitudinal dam axis and will be located in the same plane as the foundation drainage curtain.

As discussed in Section 8.0, the interior of the cross section will be zoned to provide higher strength RCC in regions where seismic tensile stresses are expected to be high.

9.2.3 Upstream Face Modifications

The preliminary design called for forming of the upstream face with stay-in-place precast concrete panels. The panels are constructed with a watertight membrane bonded to the inside panel surface against which the RCC is placed. Sealing strips around the perimeter of each panel are installed in the field by a vulcanization process. Unfortunately, this membrane barrier system is patented and the cost is relatively high. Furthermore, the membrane barrier is not approved by DSOD. Therefore, the upstream face forming concept has been revised.

The vertical upstream face will be formed using standard reusable cantilever forms. The forms will be attached to previously-placed RCC with steel anchor bolts and will be provided with strong backs to support the imposed cantilever loading. A form panel normally covers about 8 vertical feet and sufficient panels for three horizontal rows of panels will need to be available. The construction process involves placing the concrete against the form panels to a level just below the top of the top row, relocating the lower row of panels to the top, and alternately placing concrete and "jumping" the lower row of panels to support the concrete placement as it progresses.

Placing and compacting RCC directly against forms does not provide a surface satisfactory for the exposed upstream face. Both water tightness and durability would likely be unsatisfactory because the material in contact with the forms cannot be adequately compacted. A zone of internally-vibrated conventional concrete will therefore be placed in contact with the upstream forms. The zone will have an average width of 18 inches and will be placed in one foot-high lifts immediately prior to placing the adjacent RCC lift.

9.2.4 Downstream Face

The sloping downstream face of the non-overflow section will have an unformed RCC surface. The specified 0.80H:1.00V slope has been found from experience to be about the steepest that an RCC surface will stand without the support of forms. The unformed face is very economical and is entirely satisfactory in mild climates where the surface is not subject to significant freeze-thaw damage. The downstream vertical face at the top of the non-overflow section will be formed in a manner similar to that for the upstream face. The downstream face in the spillway section is described in Section 9.4.

9.2.5 Gallery

A six-foot wide by eight-foot high foundation gallery will be constructed in the dam along the longitudinal profile, 10 to 15 feet above the foundation contact. Adits will be constructed at the lowest level and at intermediate levels as appropriate. The gallery is provided for installation of the foundation grout curtain and drainage system discussed above and for internal inspection of the dam concrete. The gallery construction method will be left up to the contractor. The most common RCC gallery construction methods that have been used are the uncemented (sand) fill method, and conventional removable forms usually constructed using wood. Pre-cast stay-in-place panels have been suggested but are not recommended because they cover the surface of the gallery and prevent locating any cracking in the concrete or evaluation of seepage locations.

9.2.6 Thermal Considerations and Contraction Joints

Because of the inherent temperature rise in newly-placed concrete and the subsequent seasonal temperature drop during operation, a potential for thermal cracking exists. The specifications will require strict temperature control measures during construction. These measures may include:

- Processing and stockpiling of RCC aggregate during cold winter months;
- Shading and/or misting of aggregate piles and conveyor equipment;
- Use of low heat cement with a high percentage pozzolan replacement; or
- Scheduling the placement and prohibiting placement during warm parts of the day if necessary.

Contraction joints will be placed in the dam to provide for thermal contraction. The spacing of the contraction joints along the longitudinal axis of the dam will be specified during the final design, but for estimating purposes the contraction joint spacing is assumed to be 50 feet, based on experience. The joints will be provided with 12 inch wide PVC waterstops and joint drains in the conventional concrete upstream facing. The joints will be formed by installing metal strips in the RCC lifts as they are being placed by means of a hydraulic press.

9.2.7 Bedding Mortar

The foundation rock/concrete contact and the top surfaces of all RCC lifts will be treated with a layer of bedding mortar mix averaging 1/2 inch thick immediately prior to placing the next higher lift. The material will be a broomable high Portland cement/pozzolan content sand mortar. The mortar is provided specifically for achieving bond and seepage mitigation RCC layers and at the foundation contact and eliminating and preventing segregation or voids along boundaries of RCC placement. As discussed in Section 8.0, the bond between lifts is especially critical because of the potential for high induced tensile stresses under seismic conditions.

9.2.8 Instrumentation

The instrumentation system will be similar to that typically provided in concrete dams of the size and function of New Los Padres Dam. The instrumentation will consist of the following:

- Networks of embedded thermometers in the dam at several cross sections to monitor temperature distributions within the dam. This information is especially useful during dam construction, initial filling and the first few years of operation until thermal stability is achieved;
- A network of survey targets at various locations on the crest and downstream face including permanent survey markers at remote locations to monitor dam deflections by triangulation or other surveying techniques;
- Uplift pressure pipes installed at several cross section locations to monitor uplift pressures at the concrete/rock interface. The meters would generally be installed in the foundation gallery or gallery adits; and
- V notch weirs to measure seepage at various locations. A weir would probably be installed at each gallery adit. Provision would be made to install additional weirs where and if necessary as the need develops during initial reservoir filling.

9.2.9 Summary of Elevations and Dimensions

Pertinent elevations and dimensions are summarized in Table 9-1.

9.3 River Diversion

The river diversion will consist of a steel conduit installed in a trench through the rock foundation under the dam. The portion of the trench containing the conduit passing under the dam will be backfilled with conventional concrete up to the level of the dam foundation. As described in Chapter 10.0 (Construction Planning), the diversion conduit will extend for some distance upstream so that the stream bed can be filled around and over the conduit to form a large working/staging area for aggregate stockpiles, batch plant and other equipment. A cofferdam constructed with excavated material to a crest level at El. 950 will be located at the upstream end of the conduit. The conduit will be permanently plugged with concrete upon completion of RCC dam placement.

The diversion conduit has been tentatively sized to be 10-foot diameter for estimating purposes. Assuming a total conduit length of 1,800 feet and the cofferdam crest level at El. 950, the capacity of the diversion would be about 1,650 cubic feet per second (cfs). The diversion conduit would provide for the passing of most floods likely to occur during the construction season (May through October). Winter floods exceeding the diversion conduit capacity would overtop the cofferdam and would flow over the working area provided upstream of the dam and through the dam foundation. The contractor would probably provide a diversion channel to convey the overflow discharge over the working area and provide protection from higher level floods for the facilities including riprap for the stockpile toes.

9.4 Spillway

The spillway will be an ogee crested non-gated overflow section located in the central part of the dam with a stepped chute on the downstream face discharging into a stilling basin at the toe. The spillway crest is set at El. 1,130 with a width of 220 feet. Assuming a discharge coefficient of 3.85, the 33,000 cfs would be passed by the spillway with the reservoir surface at El. 1,141.5, 0.5 feet below the crest of the dam. A 4-foot high concrete parapet wall on the dam crest will provide 4.5 feet of freeboard to protect against wave action at high flood levels.

The stepped type spillway chute concept is presented in the preliminary design. The construction of steps on the face of the dam forming the spillway chute is well adapted to the RCC construction process and provides an economical means of partial energy dissipation.

A number of hydraulic model studies investigating stepped spillways have been performed recently, most notably by the United States Bureau of Reclamation. The results of all of the studies indicate that depending on unit discharge, total fall, and step height, a substantial portion of the energy in the falling water can be dissipated. A site specific hydraulic model study should be performed during the final design phase to evaluate the level of energy loss and the final detailed configuration of the spillway and stilling basin. For estimating purposes, the steps have been sized to be 4 feet high, based on correlation with other model studies.

The spillway training walls will be constructed by flaring out the RCC section adjacent to the walls. The wall surfaces can either be constructed using precast stay-in-place panels or with a conventional concrete facing mix similar to that described above for the upstream face. The curved dam configuration in the preliminary design allowed for convergence of the spillway training walls along radial lines to the 175 feet wide stilling basin. The same objective can be accomplished in the straight gravity dam by converging the walls in plan along straight lines from the crest to the stilling basin as shown on Drawing 9-1.

9.5 Outlet Works

The outlet works intake configuration originally proposed for the preliminary design consisted of a sloping multi-level withdrawal structure located on the left side of the reservoir just upstream of the left abutment of the dam. Such an arrangement is commonly used for fill type dams where it is impractical to construct an intake structure on the sloping dam face and the only other practical alternative is a free standing intake tower located in the reservoir. For concrete gravity dams with a vertical or near vertical upstream face, a much more practical and economical solution is to attach a vertical intake tower structure to the face of the dam. The intake preliminary design has been revised accordingly. The proposed outlet works is shown on Drawing Nos. 9-1 and 9-2 and is described in the following:

- Discharge requirements - DSOD emergency release criteria requires that at maximum discharge the outlet works must be capable of releasing enough water to reduce the head acting on the dam at full pool level to 90% of full pool within seven days. For normal releases the outlet works maximum discharge is 150 cfs with one intake gate open at the selected level in the reservoir.

- Intake - The intake structure is a reinforced concrete tower attached to the upstream face of the dam located to the left of the spillway. A 5 x 5 foot sluice gate is located near the bottom of the tower for emergency reservoir evacuation releases. Six 3 x 3 foot sluice gates are provided at various elevations to allow for selective withdrawal of water from the reservoir as required for water quality and fishery enhancement.

- Conduit - A 48-inch diameter steel pipe will be installed in a concrete-filled trench cut into the foundation rock under the dam. The conduit will convey water from the bottom of the intake tower to the outlet valve.
- Outlet valve - The outlet valve will be a 36-inch diameter Howell-Bunger valve located in a valve house located at the left side of the spillway stilling basin. The valve will discharge into the stilling basin.

9.6 Fish Collection Facilities

A conceptual design of the migrant fish facilities was developed by Bechtel and is described in their report "New Los Padres and New San Clemente Projects - Fish Collection Facilities -Conceptual Designs and Cost Estimate", (Bechtel, January 1991). The drawings are provided in Appendix H. The passage facilities for downstream migration of steelhead are located upstream of the reservoir and are known as the downstream migrant screening facilities. The passage facilities for upstream migration of steelhead are located approximately 700 feet downstream of the crest of the proposed dam and are known as the upstream migrant collection facilities. The Bechtel conceptual design and cost estimate for the fish facilities presented in the report were reviewed and the quantities for the upstream and downstream migrant facilities structures and equipment presented in the conceptual design appear to be reasonable. The general layouts are logical considering site conditions and the structures are appropriately sized based on the criteria provided in the above referenced report.

Based upon recently developed geotechnical information, it is believed that somewhat greater excavated depths than shown on the conceptual drawings will be necessary. However, review of the quantities in the conceptual design cost estimate has indicated that there is probably adequate contingency in the excavation and concrete quantities to provide for the additional excavation. The estimate in this report assumes that any required over-excavation will be filled with conventional fill concrete up to the required structural foundation level. The specifications should allow the Contractor the alternative of substituting RCC fill in these areas.

9.7 Access Roads

The proposed access roads are shown on Drawing 1-2 and consist of the following:

West Side Road

This road extends from Cachagua Road to New Los Padres Dam and to the downstream migrant screening facilities (5 miles). The west side road is approximately 25 feet wide to the crest of the dam and 18 feet wide to the downstream migrant screening facilities. Turnouts are provided at 2,000 foot intervals along the west side of the reservoir. Materials excavated in the reservoir area will be hauled back to stockpile areas designated upstream of the proposed dam. Two bridges will be required. One bridge will be near the Cachagua Road at the Carmel River, and the other bridge will be to cross Danish Creek on the west side of the proposed reservoir.

Road to the Upstream Migrant Collection Facilities (0.5 mile)

The road extends from the west side road to the upstream migrant collection facilities, the outlet works and the downstream toe of the dam. The road will be approximately 25 feet wide.

East Side Road

The road extends from Cachagua Road along Nason Road to the crest of the New Los Padres Dam (0.7 miles). The road will be approximately 25 feet wide.

TABLE 9-1: SUMMARY OF ELEVATIONS AND DIMENSIONS

New Los Padres Water Supply Project
Monterey County, California

Description	Elevation/Dimensions
Dam Crest -	El. 1,142
Spillway Crest -	El. 1,130
Base of Maximum Section -	El. 860
Maximum Dam Height -	282 ft
Crest Length -	1,585 ft
Crest Width -	16 ft
Upstream Face Slope -	Vertical
Downstream Face Slope -	0.80H:1.00V
Assumed RCC Lift Thickness -	12 inches

10.0 CONSTRUCTION PLANNING

10.1 General

This section of the report describes the construction planning concept and summarizes the sequence of activities, schedule, and plant layout assumed for the preparation of the cost estimate in Section 11.0. Each major phase of activity is highlighted along with an engineering balance of on-site construction materials involved.

10.2 Concept

The basic concept of the construction plan is to minimize environmental impacts by obtaining and processing most construction materials on-site within the reservoir inundation area, and restricting construction support facilities (except for the access roads) to the reservoir area upstream of the dam below El. 1,130. This would be accomplished by:

- Excavating and stockpiling all required concrete aggregates in the area to be inundated by the reservoir;
- Diverting the river into a conduit and creating a working area upstream of the dam by constructing a tie-back wall at the upstream face of the dam. The wall would retain waste material derived from required excavation of overburden for the dam, access road and borrow areas. The height of the wall would be governed by the amount of waste generated. The wall would also serve as a form for a portion of the upstream face of the RCC dam and would provide additional protection against seepage through the lower elevations of the dam;
- Setting up all processing and mixing plant facilities on the upstream working area; and,
- Carefully balancing the excavation of suitable construction materials from the dam foundation, access roads, and borrow areas, and processing and stockpiling aggregates in the most efficient sequence within the area and elevation constraints of the site.

The contractor may propose other construction alternatives to the plan presented in this section; however, in order to guarantee compliance with the basic concept of minimizing environmental impacts, the contractor construction plan should conform to the following restrictions:

- All concrete aggregate construction materials, other than ready-mix concrete, are to be obtained from the required excavations for the dam and access road, or from borrow areas below El. 1,130 (except access road);
- The majority of the Contractor construction facilities are to be located upstream of the centerline of the dam. Only facilities required for the construction of the outlet, access roads, and upstream migrant facilities will be allowed downstream; and
- Mixed RCC material is to be transported to the placement location on the dam by conveyor, or combination of conveyor and earthmoving equipment, from upstream of the dam.

The bidding documents should require that the contractor submit a complete construction plan, as well as, Stormwater Pollution Prevention Plan (SWPPP), and an erosion control plan at the time of bid. The feasibility and potential for environmental impact of the plans will be evaluated along with the bid prices when reviewing the bids for award.

10.3 Sequence

The proposed construction plan consists of a sequence of seven phases (I through VII). Each phase is described below along with tables of on-site construction materials balance (Tables 10-1 through 10-7) and Drawings 10-1 through 10-8, and Figure 10-1 (Construction Schedule). Three and one-half months was shown for the advertise, bid and award periods.

Net volumes of construction materials were computed by applying a loss factor of 5% each time the material is handled. An additional 5% loss was applied for the crushing and screening process. Percentages of coarse and fine aggregate resulting from processing were estimated based on the borehole and test pit logs, and trial crushing.

Phase I (2 months, see Drawing 10-1)

- Mobilization;
- Leave river in its present channel;
- Develop necessary construction access roads;
- Construct erosion and sediment control facilities as required;
- Build permanent access road to downstream migrant collection facility and downstream toe of dam;
- Excavate trenches for diversion and outlet works conduits;
- Excavate 31,000 cubic yards (cy) of overburden from Borrow Area B and

haul to designated waste areas;

- Excavate 8,000 cy of overburden from Borrow Area C and haul to designated waste areas; and
- Excavate 75,000 cy of alluvial terrace gravel from Borrow Area C and stockpile on top of Borrow Area B.

Estimated construction material volumes in Phase I are summarized in Table 10-1.

Phase II (2 months, see Drawing 10-2)

- Install 120-inch diameter temporary diversion conduit including gate and operator shaft;
- Divert river into diversion conduit;
- Construct concrete encased, 48-inch diameter outlet works conduit under the dam foundation including inlet tower to at least El. 935;
- Excavate 84,000 cy of overburden from required dam excavation consisting of silty sand fan material and haul to downstream cofferdam and waste area upstream of dam;
- Excavate 120,000 cy of overburden consisting of silty sand fan material from Borrow Area A and haul to upstream cofferdam and waste area upstream of dam;
- Construct upstream cofferdam and spillway to Els. 950 and 920, respectively, utilizing a portion of the 120,000 cy of overburden excavation from Borrow Area A;
- Construct downstream cofferdam to El. 880 using a portion of the 84,000 cy of waste available from dam foundation excavation;
- Construct tie-back retaining wall at upstream face of dam to El. 925 utilizing surplus overburden waste excavation from dam foundation and Borrow Area A not used in cofferdams; and
- Backfill diversion conduit between upstream cofferdam and tie-back retaining embankment to El. 910 utilizing 180,000 cy of surplus overburden excavation from dam and Borrow Area A.

Estimated construction material volumes in Phase II are summarized in Table 10-2. Concrete and aggregate requirements for Phases I and II will be met utilizing off-site sources.

Phase III (2 months, see Drawing 10-3)

- Excavate 45,000 cy of terrace gravel from required dam excavation (Borrow Area D) and haul to stockpile on top of Borrow Area B at El. 990;
- Excavate 432,000 cy of fractured and/or weathered rock from required dam excavation. Haul 50% to waste area upstream of dam, raising the fill to El. 925. Stockpile the remaining rock on top of Borrow Area B;
- Set up aggregate crushing and screening plant in work area at El. 925, upstream of the dam; and
- Construct temporary riprap lined flood channel along left side of work area to carry flood discharges exceeding the capacity of the diversion conduit.

Estimated construction material volumes in Phase III are summarized in Table 10-3.

Phase IV (6 months, see Drawing 10-4)

- Re-excavate 71,000 cy of gravel stockpiled from Borrow Area C, process into 38,000 cy of coarse aggregate, and 26,000 cy of fine aggregate. Stockpile aggregates adjacent to the plant;
- Re-excavate 43,000 cy of terrace gravel and 205,000 cy of rock stockpiled from required dam excavation (Borrow Area D), and process into 134,000 cy of coarse aggregate and 94,000 cy of fine aggregate. Stockpile aggregates adjacent to plant and in Borrow Area B;
- Excavate 353,000 cy of terrace gravel from Borrow Area B to El. 978. Haul material to crushing and screening plant, process into 191,000 cy of coarse aggregate and 127,000 cy of fine aggregate. Stockpile aggregates in stripped Borrow Area B;
- Clean-up and prepare dam foundation;
- Erect RCC and conventional concrete mixing plants upstream of dam at El. 925;
- Install concrete conveyor delivery system; and
- Excavate 311,000 cy of terrace gravel from Borrow Area A to El. 945. Haul gravel to crushing and screening plant, process into 168,000 cy of coarse aggregate and 112,000 cy of fine aggregate, and haul to stockpiles in Borrow Area B.

Estimated construction material volumes in Phase IV are summarized in Table 10-4.

Phase V (3 months, see Drawing 10-5)

- Recover aggregate from stockpiles, mix, and place 420,000 cy of conventional concrete and RCC in dam up to El. 1,000;
- Excavate 175,000 cy of rock from required excavation of access roads (Borrow Area K) to the upstream migrant collection facility. Haul 5,000 cy to crushing and screening plant, and process into 4,000 cy of coarse aggregate and stockpile adjacent to plant. Haul the remaining 170,000 cy to waste in the stripped Borrow Area A; and
- Construct downstream migrant collection facility.

Estimated construction material volumes in Phase V are summarized in Table 10-5.

Phase VI (4 months, see Drawings 10-6 and 10-7)

- Complete dam by recovering aggregate from stockpiles, mixing, and placing the remaining 455,000 cy of conventional concrete and RCC to El. 1,142;
- Complete outlet works inlet tower to El. 1,142;
- Construct outlet works valve chamber and control building;
- Complete construction of permanent access road to upstream migrant collection facility;
- Construct upstream and downstream migrant collection facilities; and
- Decommission/breach existing Los Padres Dam. The existing Los Padres Dam will be breached as shown on Drawing 10-7.

Phase VII (1 month, see Drawing 10-8)

- Demobilize crushing and screening plant, conventional, and RCC mixing plants, and conveyor delivery system; and
- Close temporary diversion gate, construct diversion plug, and begin filling reservoir.

10.4 On-Site Construction Materials Balance

In order to construct the dam it will be unnecessary to explore all the sources of construction material described in Section 7.0. The construction plan presented above takes advantage of the best apparent potential sources in a logical and systematic sequence based on the information available at this time. Table 10-6 summarizes the net volumes of on-site construction materials that can be developed to yield the 535,000 cy of coarse aggregate and 350,000 cy of fine aggregate required to construct the dam.

It may be necessary to adjust the balance of construction materials presented in Table 10-6 depending on the quality of the closely fractured and/or highly weathered rock from required excavation for the dam and access roads. If the rock is of good quality, it may be possible to decrease the volume of terrace gravel taken from the borrow areas. If the rock is poor quality it may be necessary to (1) process more material from the road, (2) explore the terrace gravels in Borrow Area I, or (3) excavate rock from borrow areas A or E.

10.5 Schedule

Figure 10-1 is a schedule of the activities described in the preceding construction plan. The critical path goes through the activities related to excavating, hauling, stockpiling, and re-excavating terrace gravel and rock from the required dam and access road excavations, and borrow areas; processing that material into concrete aggregate and stockpiling; re-excavating and mixing that aggregate into conventional concrete and RCC; delivering the concrete to the dam; and placing and consolidating it. The constraining production rates are related to the aggregate processing plant and the conventional concrete and RCC mixing plant, and conveyor delivery system. Table 10-7 summarizes the rates assumed for these critical activities in the schedule.

The schedule assumes two 10 hour shifts, six days per week during the time of the critical activities. Critical activities are expected to be May through October period in the second year. Equipment and materials deliveries are assumed to be made on a 5 day a week basis (Monday through Friday) to reduce weekend traffic. Since RCC placement is scheduled during the warm, dry months of summer it is anticipated that the two shifts would run from 6 PM to 4 AM, and 4 AM to 2PM. The four hour interval during the hottest part of the day, between 2PM and 6PM, would be used for maintenance and fueling.

10.6 Plant Layout

The plant necessary to achieve the rates described above was laid out especially for the current study of New Los Padres Dam. It consists of a fleet of excavating and hauling equipment, an aggregate processing plant, two concrete mixing plants, a conveyor delivery system, and a spread of placing and compacting equipment. The specialty equipment described below was utilized to develop the cost estimate in Section 11.0. The remaining equipment considered (aggregate processing plant and excavating, hauling, placing and compacting equipment) is standard for RCC projects.

Conventional Concrete Mixing Plant
Aran AR-280 continuous mix pug mill. Rated output 600 T/hr

RCC Mixing Plant
Aran AR-400 continuous mix pug mill. Rated output 1,000 T/hr.

Conveyor Delivery System
The planned system is supplied by ROTEC Industries of Elmhurst, Illinois and consists of:

- One metering conveyor (65' x 48");
- One slope conveyor (500' x 36");
- One tripper conveyor (1,550' x 36");
- One feeder conveyor (90' x 36");
- One crawler placer with a 90' x 36" rotating conveyor;
- One top-out unit (replaces crawler placer for top narrow part of dam);
- Set of jacking posts as required; and
- Complete set of electrical controls.

The 48-inch metering conveyor will be fed directly from the mixing plant, and will provide a continuous flow of material at approximately 250 feet per minute to the 36-inch x 500-foot uphill feeding conveyor, and then to the 36-inch wide, 1,550-foot long tripper conveyor running the longitudinal length of the dam. The tripper conveyor will be positioned on interior jacking towers that will eventually be embedded in the dam, and raised by a jacking device attached to a front end loader. Finally the concrete will be fed to the crawler placer through a 90-foot section of 36-inch conveyor.

TABLE 10-1: PHASE I CONSTRUCTION MATERIAL BALANCE
New Los Water Supply Project
Monterey County, California

Source	Material Type	Volume (1000 x cy)	Handling		Destination	Yield (1000 x cy)
			Activity	% Loss		
Borrow Area B	Silty Sand	31	Haul	5	Waste	29
Borrow Area C	Silty Sand	8	Haul	5	Waste	7
Borrow Area C	Terrace Gravel	75	Haul	5	Stockpile	71

TABLE 10-2: PHASE II CONSTRUCTION MATERIAL BALANCE
New Los Padres Water Supply Project
Monterey County, California

Source	Material Type	Volume (1000 x cy)	Handling		Destination	Yield (1000 x cy)
			Activity	% Loss		
Dam Foundation (Borrow Area D)	Silty Sand	84	Haul	5	Cofferdam,Waste, Backfill	80
Borrow Area A	Silty Sand	120	Haul	5	Cofferdams, Waste, Backfill	114

TABLE 10-3: PHASE III CONSTRUCTION MATERIAL BALANCE
New Los Padres Water Supply Project
Monterey County, California

Source	Material Type	Volume (1000 x cy)	Handling		Destination	Yield (1000 x cy)
			Activity	% Loss		
Dam Foundation (Borrow Area D)	Terrace Gravel	45	Haul	5	Stockpile	43
	Rock	432	Haul	5	Waste	205
					Stockpile	205

TABLE 10-4: PHASE IV CONSTRUCTION MATERIAL BALANCE
New Los Padres Water Supply Project
Monterey County, California

Source	Material Type	Volume (1000 x cy)	Handling		Destination	Yield (1000 x cy)
			Activity	% Loss		
Stockpiled Borrow Area C	Terrace Gravel	71	Haul, Crush, Screen, and Stockpile	10	Coarse Agg.	38
					Fine Agg.	26
Borrow Area B	Terrace Gravel	353	Haul, Crush, Screen, and Stockpile	10	Coarse Agg.	191
					Fine Agg.	127
Stockpiled Dam (Borrow Area D)	Terrace Gravel	43	Haul, Crush, Screen, and Stockpile	10	Coarse Agg.	23
					Fine Agg.	16
	Rock	205			Coarse Agg.	111
					Fine Agg.	74
Borrow Area A	Terrace Gravel	311	Haul, Crush, and Screen	20	Coarse Agg.	168
					Fine Agg.	112

TABLE 10-5: PHASE V CONSTRUCTION MATERIAL BALANCE
New Los Padres Water Supply Project
Monterey County, California

Source	Material Type	Volume (1000 x cy)	Handling		Destination	Yield (1000 x cy)
			Activity	% Loss		
Access Roads (Borrow Area K)	Rock	175	Haul, Crush, and Screen	10	Coarse Agg.	4
					Fine Agg.	0
			Haul	5	Waste	162

TABLE 10-6: SUMMARY OF ON-SITE CONSTRUCTION MATERIAL EXPLORATION

New Los Padres Water Supply Project
Monterey County, California

Source	Material Type	Concrete Aggregate (x 1000 cy)		Waste (1000 x cy)
		Coarse	Fine	
Borrow Area A	Terrace Gravel	168	112	114
Borrow Area B	Terrace Gravel	191	127	29
Borrow Area C	Terrace Gravel	38	26	7
Borrow Area D	Terrace Gravel	23	16	80
Borrow Area D	Rock	111	69	205
Borrow Area K	Rock	4	0	162
	TOTAL	535	350	597

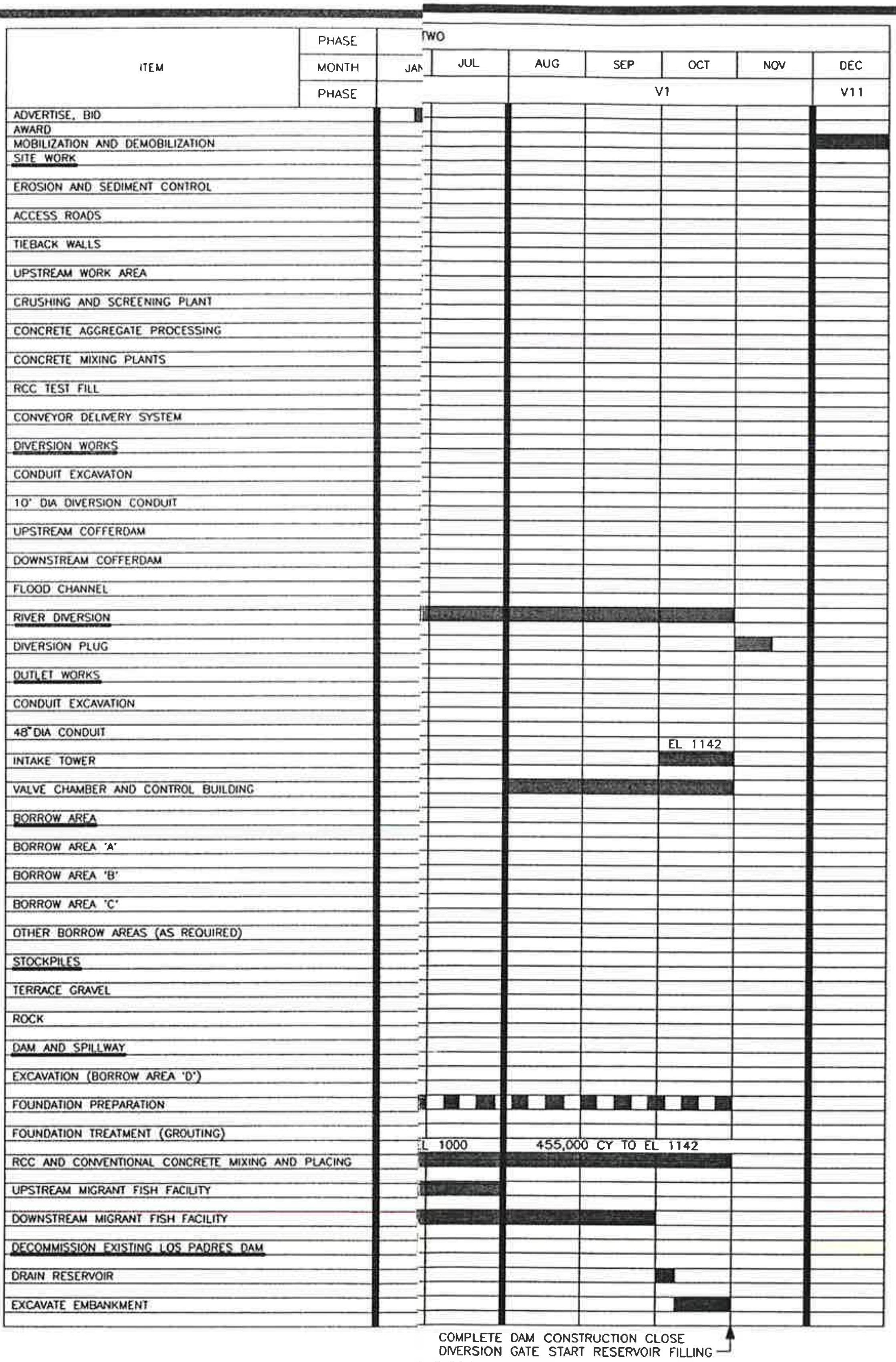
TABLE 10-7: CRITICAL ACTIVITY PRODUCTION RATES

New Los Padres Water Supply Project
Monterey County, California

Activity	Average Production Rate
Excavate and Haul Terrace Gravel and Rock	6,000 cy/day
Process Conventional Concrete and RCC Aggregate	300 cy/hr
Mix Conventional Concrete	200 cy/hr
Mix RCC	430 cy/hr
Deliver Conventional Concrete and RCC	As required in proportion to total delivery of conventional concrete and RCC
Place, and Compact/Consolidate Conventional Concrete and RCC	As required in proportion to total delivery of conventional concrete and RCC

Table 8-1: ESTIMATED RCC MIX PROPORTIONS
New Los Padres Water Supply Project
Monterey County, California

Ingredient	Pounds per Cubic Yard	
	Zone 1	Zone 2
Portland Cement (Type II, Low Alkali)	315	125
Fly Ash, Type F	135	55



SCHEDULE

EXPLANATION

■ ■ ■	INTERMITTENT EFFORT AS REQUIRED
▬	CONTINUING EFFORT
TG	TERRACE GRAVEL
CY	CUBIC YARD

ler Management District
Engineering Studies
ater Supply Project
nty, California

PROJECT NO.
94-1198801.80

FIGURE NO.
10-1

PREPARED BY: HEJ
REVIEWED BY: KBK
DATE: 2/17/95

11.0 COST ESTIMATE

This section of the report presents the estimated cost to construct New Los Padres Dam and appurtenant structures. The estimate was prepared based upon the construction plan and schedule described in Section 10.0. Quantities were measured and calculated from the preliminary drawings included in Section 9.0. Quantities not measurable were estimated from experience on similar projects.

Material prices are based on current quotes adjusted for delivery to the site. Labor costs are based on current Davis Bacon rates in effect in the Monterey area. The cost of operating and maintaining construction equipment was obtained from vendors and other published data which includes depreciated purchase cost, operating cost, cost of fuel, parts, repair, and labor.

Allowances are made in the cost estimate for temporary facilities such as field offices, warehouses, explosive magazines, repair shops, construction access and haul roads, and temporary utilities. Provision has also been made for routine construction support services such as survey and cleanup crews, warehouse staff, electrical power supply, service vehicles, dust abatement and control, small tools and consumables, office expenses, equipment, and furniture.

Engineering costs are estimated at 8 percent, and construction management costs at 6 percent of the construction cost.

A contingency allowance of 20 percent was applied to the construction of the dam to allow for the range of unknowns related to construction materials, concrete aggregate processing, and cementitious content of the RCC and unknown environmental impacts. A 20 precent contingency also was applied to the upstream and downstream migrant fish facilities.

The cost estimate is in terms of present day costs prevailing in January 1995. Cost summaries are presented in Tables 11-1 through 11-3. Figure 11-1 shows the estimate for the dam, Table 11-2 shows the estimate for the fish facilities and Table 11-3 summarizes all costs including final engineering and construction management.

The total cost for the dam is estimated to be $57,369,000 including a 20 percent contingency of about $9,561,000. Downstream migrant fish screening facilities are estimated to cost around $11,500,000 while upstream collection facilities are estimated to cost around

$2,840,000; hence, the fish collection facilities are nearly 20 percent of the total project cost.

Engineering design and construction management costs are typically 8 percent and 6 percent respectively, for projects of this size and nature. Also, engineering design costs do not include any environmental costs. Based on the assumed engineering and construction management costs, the total cost for the New Los Padres Dam is estimated to be around $81,720,000.

Table 11-1: Cost Estimate - Dam
Monterey Peninsula Water Management District
New Los Padres Water Supply Project
January, 1995

No.	Description	Quantity	Unit	Unit Price	Cost
	Bonds & Insurance (0.7%)	1	LS	330,000	$330,000
	Mobilization (4%)	1	LS	1,800,000	1,800,000
101	Clearing & Grubbing	260	AC	4,173	1,084,980
102	Care & Diversion of Water	1	LS	250,000	250,000
201	Excavation - Unclassified	400	CY	7.45	2,980
202	Foundation Preparation	20	SY	4	80
203	Concrete - Backfill	50	CY	125	6,250
204	Concrete - Structural	900	CY	230	207,000
205	Cement	5,200	CWT	3.90	20,280
206	Reinforcing Steel	139,000	lb	0.53	73,670
207	Int. Slide Gates & Operators 3'x 3'	7	EA	7,000	49,000
207	Int. Slide Gates & Operators 5'x 5'	1	EA	10,000	10,000
208	Intake Trashracks	35,000	lb	1.50	52,500
209	Handrail & Grating	1,500	lb	2.50	3,750
301	Excavation - Unclassified	561,000	CY	7.45	4,179,450
304	Excavation - Dental	2,000	CY	15	30,000
305	Mob & Demob for Grouting	1	LS	23,000	23,000
306	Drill Dam Fndn Grout Holes	42,000	LF	14	588,000
307	Drill Consolidation Grout Holes	43,000	LF	13	559,000
308	Drill Drain Holes Above Gallery	14,000	LF	18	252,000
309	Drill Drain Holes Below Gallery	13,000	LF	18	234,000
310	F & I Pipe Dam Grout & Drain Holes	27,000	lb	1.80	48,600
311	Connect to Grout Hoses	2,700	EA	120	324,000
312	Grout Dam Foundation	21,000	CF	17.66	370,860
313	Consolidation Grouting	22,000	CF	17.66	388,520
314	Foundation Preparation	26,000	SY	4	104,000
315	Concrete, Portals Drain Gallery	100	CY	230	23,000
321	Concrete - Dental	2,000	CY	125	250,000
322	Concrete - Bedding Mix Fndn.	800	CY	125	100,000
323	RCC - Test Fill	1,000	CY	23.20	23,200
324	RCC - Interior Mix - Zone 1	175,000	CY	23.20	4,060,000
325	RCC - Interior Mix - Zone 2	637,000	CY	23.20	14,778,400
326	Concrete - Bedding Mix	17,000	CY	125	2,125,000
327	U/S Face Conven. Concrete	15,000	CY	95	1,425,000

Table 11-1: Cost Estimate - Dam Monterey Peninsula Water Management District New Los Padres Water Supply Project January, 1995					
No.	Description	Quantity	Unit	Unit Price	Cost
331	Concrete Spillway Lining	8,600	CY	140	$1,204,000
332	Concrete Spillway Crest	610	CY	140	85,400
334	RCC Spillway Walls	18,000	CY	46.40	835,200
335	Concrete Parapet Walls	410	CY	230	94,300
336	Concrete Roadway Surface, 4"	240	CY	106	25,440
341	Cement	1,539,000	CWT	3.90	6,002,100
342	Pozzolan	660,000	CWT	2.90	1,914,000
343	Reinforcing Steel	1,660,000	lb	0.53	879,800
344	Water Stop	5,300	LF	10	53,000
345	Hand Rail, Dam Crest	11,700	lb	2.50	29,250
351	Drainage System	1	LS	55,000	55,000
361	Instrumentation	1	LS	161,000	161,000
371	Water Quality Control, Reseeding	1	CY	44,000	44,000
372	Decommission Existing Los Padres Dam	1	LS	460,000	460,000
401	Excavation - Dental	200	CY	15.25	3,050
411	Concrete - Backfill	340	CY	125	42,500
412	Concrete - Dental	200	CY	125	25,000
413	Concrete - Walls	520	CY	230	119,600
414	RCC - SLAB	3,600	CY	23.20	83,520
415	RCC - FACING SLAB	1,500	CY	23.20	34,800
416	RCC - Walls	3,000	CY	23.20	69,600
417	Cement	21,000	CWT	3.90	81,900
418	Pozzolan	9,000	CWT	2.90	26,100
421	Drill - Rock Anchors	6,000	LF	7.50	45,000
422	F & I Rock Anchors	27,000	lb	5	135,000
423	Drill 3" Drain Holes - Slab	1,800	LF	20.50	36,900
424	Drill NX Drain Holes - RCC Walls	390	LF	43	16,770
425	Drill NX Drain Holes - Walls & Rock	250	LF	43	10,750
431	Reinforcing Steel	258,000	lb	0.53	136,740
501	Excavation - Rock	700	CY	10.17	7,119
511	Concrete - Outlet Conduit Saddle	600	CY	150	90,000
512	Cement	2,100	CWT	3.90	8,190
521	Reinforcing Steel	24,000	lb	0.53	12,720
531	Steel Liner, 48"	40,000	lb	2.50	100,000

Table 11-1: Cost Estimate - Dam
Monterey Peninsula Water Management District
New Los Padres Water Supply Project
January, 1995

No.	Description	Quantity	Unit	Unit Price	Cost
532	Cone 48" x 36"- 1 ea	1,200	lb	2.50	3,000
533	Spool 36" - 1 ea	1,000	lb	10	$10,000
534	Howell/Bunger Valve 36" diam.	1	EA	200,000	200,000
601	Backfill	6,700	CY	15	100,500
611	Concrete - Outlet Structure	150	CY	230	34,500
612	Cement	710	CWT	3.90	2,769
621	Reinforcing Steel	23,000	lb	0.53	12,190
631	Misc. Metals	1,200	lb	2.50	3,000
632	Handrail	1,700	lb	2.50	4,250
701	Earthwork	200	CY	7.45	1,490
721	Masonry	4,900	SF	15.30	74,970
731	Roofing	250	SF	7.50	1,875
741	Doors, Windows & Louvers	100	SF	50	5,000
751	Finishes	1	LS	5,000	5,000
761	HVAC	1	LS	10,000	10,000
762	Plumbing	1	LS	10,000	10,000
771	Electrical	1	LS	250,000	250,000
801	Access Road/Carmel River Bridge	1	LS	375,000	375,000
			SUBTOTAL		$47,807,813
901	Contingency 20%				9,561,187
			TOTAL		$57,369,000

Explanation:

AC = Acre
CF = Cubic foot
CWT = 100 weight
CY = Cubic yard
EA = Each
F&I = Fabricate and Install
LB = Pound
LF = Lineal feet
LS = Lump Sum
SY = Square yard

Table 11-2: Cost Estimate

Monterey Peninsula Water Management District
Migrant Collection Facilities
New Los Padres Water Supply Project
January 1995

No.	Description	Quantity	Unit	Unit Price	Cost
Downstream Migrant Fish Facilities					
1	Bonds & Insurance	1	LS	50,000	$50,000
2	Mobilization	1	LS	400,000	400,000
3	Clearing	2	AC	4,173	8,346
4	Clearing - Fish Haul Road	39	AC	6,700	261,300
5	Dewatering	1	LS	41,409	41,409
6	Excavation	25,000	CY	8.25	206,250
7	Backfill	3,400	CY	10.75	36,550
8	Excavation-Fish Haul Road	175,000	CY	8.60	1,505,000
9	18" Culverts - Fish Haul Road	1,800	LF	32	57,600
10	36" Culverts - Fish Haul Road	320	LF	80	25,600
11	Fin. Grade/Paving - Fish Haul Road	38,900	SY	15	583,500
12	Bridge - Fish Haul Road	1	LS	550,000	550,000
13	Paving	500	SY	15	7,500
14	Water Piping Systems	2	EA	5,000	10,000
15	Guard Rail & Signs	1,600	LF	20	32,000
16	Fences & Gates	1	LOT	11,840	11,840
17	Foundation Preparation	16,000	SF	4	64,000
18	Concrete - Construction Joints	7,500	SF	.90	6,750
19	9" Waterstops - PVC	560	LF	13.38	7,490
20	Formwork - Shop Fabrication	15,000	SF	4	60,000
21	Formwork - Set & Strip	70,650	SF	5	353,250
22	Reinforcing Steel	404,000	lb	.55	222,200
23	Concrete Placing	4,040	CY	150	606,000
24	Concrete Finishing	83,300	SF	0.70	58,310
25	Misc. Metal Fabrications	39,000	SF	4.50	175,500
26	Gates & Equalizers	1	LOT	1,528,093	1,528,093
27	Racks & Screens	1	LOT	1,177,361	1,177,361
28	Fish Handling Equipment	1	LOT	93,475	93,475
29	Log Boom	1	EA	14,290	14,290
30	Instrumentation	1	EA	16,500	16,500
31	Hoists	2	EA	184,670	369,340
32	Raceway	1,610	LF	19.85	31,959
33	Wire & Cable	11,800	LF	4.01	47,348
34	Grounding System	1	LOT	5,000	5,000
35	Equipment	1	LOT	47,456	47,456
36	Lighting	18	EA	800	14,400
37	Overhead Power Line	7	MI	125,119	875,833
				SUBTOTAL	$9,561,449
				CONTINGENCY 20%	1,912,551
				TOTAL	$11,474,000

Table 11-2: Cost Estimate
Monterey Peninsula Water Management District
Migrant Collection Facilities
New Los Padres Water Supply Project
January 1995

No.	Description	Quantity	Unit	Unit Price	Cost
Upstream Migrant Fish Facility					
50	Bonds & Insurance	1	LS	25,000	$25,000
51	Mobilization	1	LS	110,000	110,000
52	Clearing & Site Preparation	4	AC	4,173	16,692
53	Care & Diversion of Water	1	LS	50,174	50,174
54	Common Excavation	9,000	CY	4.80	43,200
55	Foundation Preparation	3,500	SY	4	14,000
56	Backfill (sand/gravel/rock)	120	CY	25	3,000
57	Common Backfill	1,500	CY	10.75	16,125
58	Riprap	500	CY	120	60,000
59	24" CMP	200	LF	37	7,400
60	Fences & Gates	300	LF	20	6,000
61	Underdrain System	1	LS	27,500	27,500
62	Dam & Gravity Wall Concrete	2,700	CY	240	648,000
63	Stilling Basin Concrete Slab	2,000	CY	240	480,000
64	Training Wall Concrete	1,000	CY	240	240,000
65	Fish Facility Concrete	300	CY	240	72,000
66	Reinforcing Steel	275,000	lb	0.55	151,250
67	Waterstops	700	LF	13.38	9,366
68	Tiedown Anchors	3,000	LF	15	45,000
69	Fish Handling Equipment	1	LOT	55,000	55,000
70	Sluice Gates/Valves/Pumps & Piping	1	LOT	49,500	49,500
71	Electrical Work	1	LOT	46,200	46,200
72	Structural Steel	40,000	lb	1.07	42,800
73	Misc. Metal Work	1	LOT	30,000	30,000
74	Architectural Painting	1	LOT	22,000	22,000
75	Seeding & Mulching	1	LOT	14,300	14,300
76	Access Road	0.5	MI	165,000	82,500
			SUBTOTAL		$2,367,007
			CONTINGENCY 20%		472,993
			TOTAL		$2,840,000

Explanation:

AC = Acre
CY = Cubic yard
EA = Each
lb = Pound
LF = Lineal feet
LS = Lump Sum
SY = Square yard
LOT = Total labor and equipment
MI = Miles
SF = Square feet
SY = Square yard

Table 11-3: Summary of Costs Monterey Peninsula Water Management District New Los Padres Water Supply Project January 1995		
D/S FISH COLLECTION FACILITIES U/S FISH COLLECTION FACILITIES LOS PADRES DAM		$11,474,000 2,840,000 57,369,000
TOTAL		71,683,000
ENGINEERING CONSTRUCTION MANAGEMENT	8% 6%	5,735,000 4,302,000
TOTAL CONSTRUCTION COST		$81,720,000

12.0 CONCLUSIONS AND RECOMMENDATIONS

12.1 Conclusions

12.1.1 Geology and Site Conditions

The primary rock units in the project area are Mesozoic and older crystalline bedrock (granitic and metasedimentary rock), Tertiary sedimentary sandstone and Quaternary surficial deposits, including alluvial fan deposits and stream terrace deposits. Hard, unweathered granitic rock is present below an approximate depth of 20 to 80 feet in the left abutment of the dam, 20 to 70 feet in the right abutment of the dam and 10 feet in the river area.

12.1.2 Cachagua Fault Study

Results from this study indicate that there is compelling geologic and geomorphic evidence that the Cachagua fault has not experienced movement since at least the past 85,400 to 213,500 years. This conclusion is based upon the estimated age of Quaternary stream terrace deposits that cover, but are not offset by, the fault. Thus, the Cachagua fault is not "active" according to the criteria established by the California Division of Mines and Geology. The last movement along the fault may be much older than these stream terrace deposits; however, the lack of older Quaternary deposits in the area prevent better resolution of the age of faulting.

The potential for fault rupture through the proposed dam site as a result of movement along the Cachagua fault zone is considered to be low. No faults were mapped in the vicinity of the proposed dam site and no indications of faulting were observed on seismic refraction profiles in the dam area. The undisturbed nature of Quaternary terrace deposits that have been mapped through the project area indicate a lack of fault activity in the vicinity of the proposed dam site.

12.1.3 Borrow Materials

Sufficient amounts of suitable construction materials are available on-site, upstream of the dam, and in the required excavations for the dam and access roads to meet the conventional concrete and RCC aggregate requirements for the dam and appurtenant structures. A gross volume of approximately 1.5 to 2 times that required has been proven and an estimated surplus of five times that required has been identified.

The most economical materials appear to be the alluvial terrace gravels in Borrow Areas A, B, C, D (required dam excavation), and I. Assuming a loss factor of 15% during excavation, hauling, stockpiling, re-excavation, and processing, it is estimated that 666,000 cy of suitable concrete aggregate could be processed from these deposits. The remaining required 219,000 cy could be processed from rock excavation from the dam, rock excavation in Borrow Area A, and access road construction. Exploration of rock sources from other borrow areas is not anticipated, but sources are available if the need arises.

High quality aggregate to supplement on-site sources is available within a reasonable haul distance from the site as is ready-mix concrete. Cement and pozzolan (fly ash) can be supplied from off-site sources at reasonable costs.

12.1.4 RCC Trial Mix Program

Trial mix program test data indicates that RCC meeting the compressive and tensile strength requirements of the dam can be made utilizing on-site aggregate derived by crushing alluvial terrace gravel from Borrow Areas A and B.

This trial mix program identified the following composition properties that resulted in acceptable strength results:

- Unit weight - Greater than 150 PCF;
- Cement content - 300 to 400 pounds per cubic yard, for Zone 1 and 150 to 200 pounds per cubic yard for Zone 2;
- Water/cement ratio - 0.43-0.63 Zone 1 and 0.95 Zone 2; and
- Paste/mortar ratio - 0.41-0.44 Zone 1 and 0.38 Zone 2.

12.1.5 Seismic Design Criteria

Use of deterministic methods for seismic modeling in lieu of synthetically generated input time histories appears reasonable at this site. Response spectra from scaled time histories from recently recorded strong motions from Loma Prieta and Northridge earthquakes show good agreement between empirical 84 percentile spectra for periods of vibration at or greater than that of the dam. The Geomatrix (Idriss, 1993a) and Seed, Ugas and Lysmer (1974) empirical relationships calculated greater pseudo-absolute accelerations than those calculated from the scaled records for periods below 2 seconds. The controlling earthquake could occur on the Tularcitos fault with a magnitude of 6.8. This magnitude earthquake could generate a 0.51g peak ground acceleration.

12.1.6 Preliminary Design and Cost Estimate

Based on the information obtained in these and previous studies, a 282 foot high RCC dam can be constructed at the site. The estimated cost of the dam and fish facilities with engineering and contingencies is $81,720,000 in January 1995 dollars. The estimated construction cost of the dam alone is $57,369,000. This cost includes a 20 percent contingency factor to provide for uncertainties inherent in the preliminary level of design. Major costs items are dam foundation excavation which is about 9 percent of the total construction cost and RCC construction, including cement and pozzolan, which constitutes about 56 percent of the estimated cost of the dam.

12.2 Recommendations

12.2.1 Additional Geotechnical Investigations

Additional geotechnical investigations are necessary for final design. These investigations should include the following:

- A more detailed structural geologic evaluation of the proposed dam site, including analysis of site-specific data such as joint patterns (orientation, inclination, spacing, strength, etc.) and rock fabric studies such as foliation and lineation patterns.

- For the dam foundations and fish collection facilities, additional seismic refraction surveys, geologic mapping, borings, water pressure tests, laboratory testing and test trenches should be performed to define the depth of foundation excavation and foundation conditions.

- Although the potential for slope instability and seepage in the reservoir may not be significant, the area should be evaluated by a geotechnical investigation of reservoir hillslopes. The focus of the investigation should be the identification of potentially unstable slopes and characterization of geologic conditions associated with slope instability and seepage.

- A detailed engineering geologic investigation of the proposed access road alignment should be performed in order to identify areas of potential instability and to provide the MPWMD with cut slope design criteria and road construction recommendations. Geologic mapping and seismic refraction profiling to determine the extent and estimated thickness of surficial deposits and weathered bedrock should be completed as part of this investigation.

- Additional borrow area investigations should be performed to further evaluate the available volumes of borrow materials using borings, test trenches, and seismic refraction analysis.

- Excavations should be performed to obtain samples of the rock underlying the borrow areas for use in RCC trial mixes.

12.2.2 Additional RCC Trial Mixes

Additional RCC trial mixes in two phases should be performed.

Phase 1

- Vary aggregate gradation with regard to percentage of sand, and percentage finer than No. 200 sieve by blending silty sand fines from overburden material.

- Prepare mixes using crushed rock from a trial excavation of rock from the dam foundation, access roads, and/or borrow areas.

Phase 2

Using the most favorable gradation, and both crushed terrace gravel and rock, prepare test mixes to obtain the best apparent:

- Water content
- Cement/Pozzolan ratio
- Paste/mortar ration
- Cementitious content

12.2.3 Hydraulic Model Studies

Hydraulic model studies should be performed to establish the final configuration of the spillway and stilling basin. The objectives of the studies would be to establish the height of the steps on the face of the spillway to evaluate the level of energy loss and to evaluate the required dimensions and elevations for the stilling basin.

12.2.4 Dynamic Analysis

Since high dynamic tensile stresses were indicated in the dam by the results of the preliminary linear elastic seismic analysis, the final design should include a two-dimensional dynamic analysis to better define the seismic stress distribution. Further studies should be performed to select appropriate time histories for input into the final design model(s).

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A1S-18809-ABG-267-4-11, frames 112 through 118, Scale 1:20,000, black and white, June 6, 1939.

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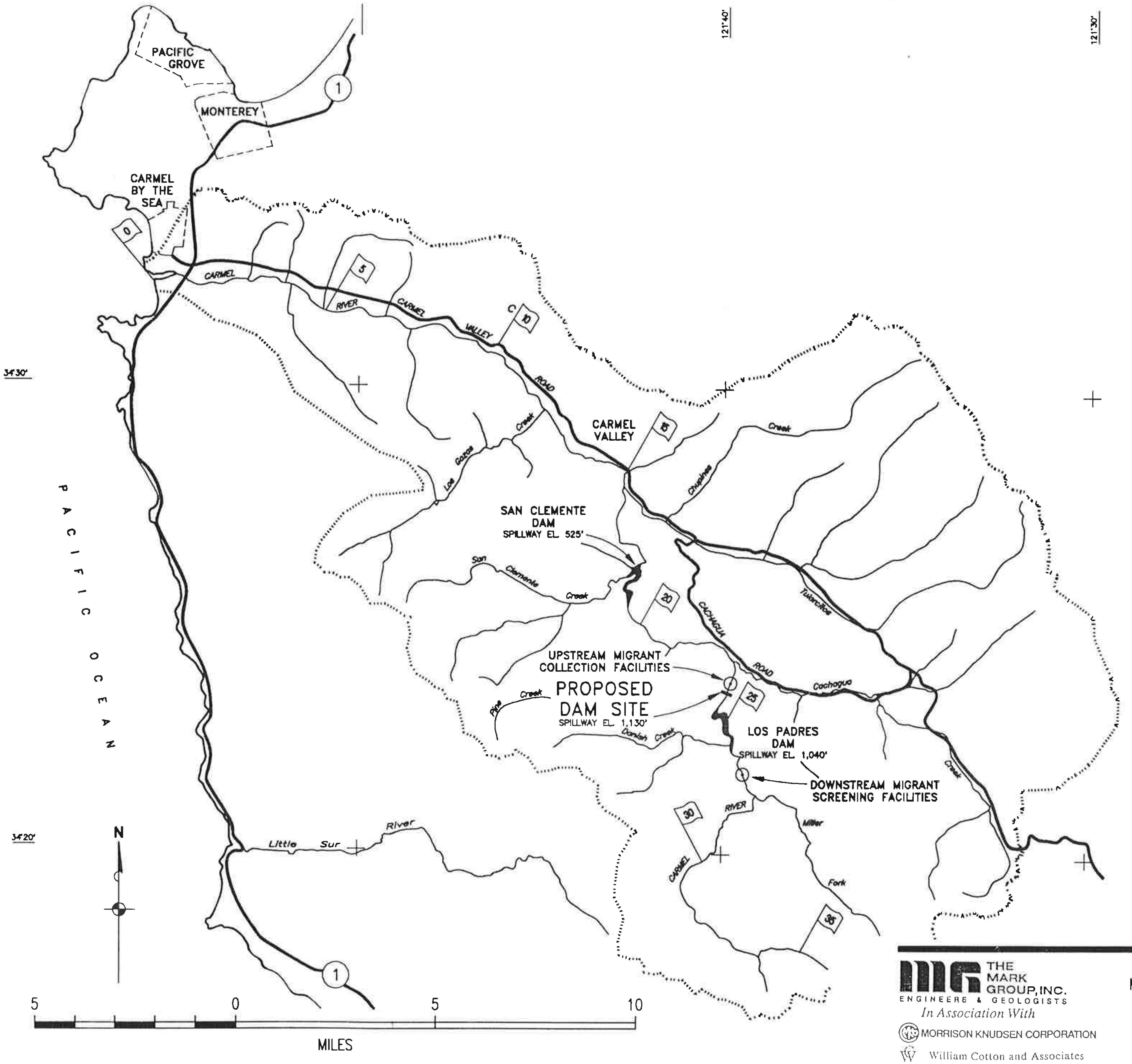
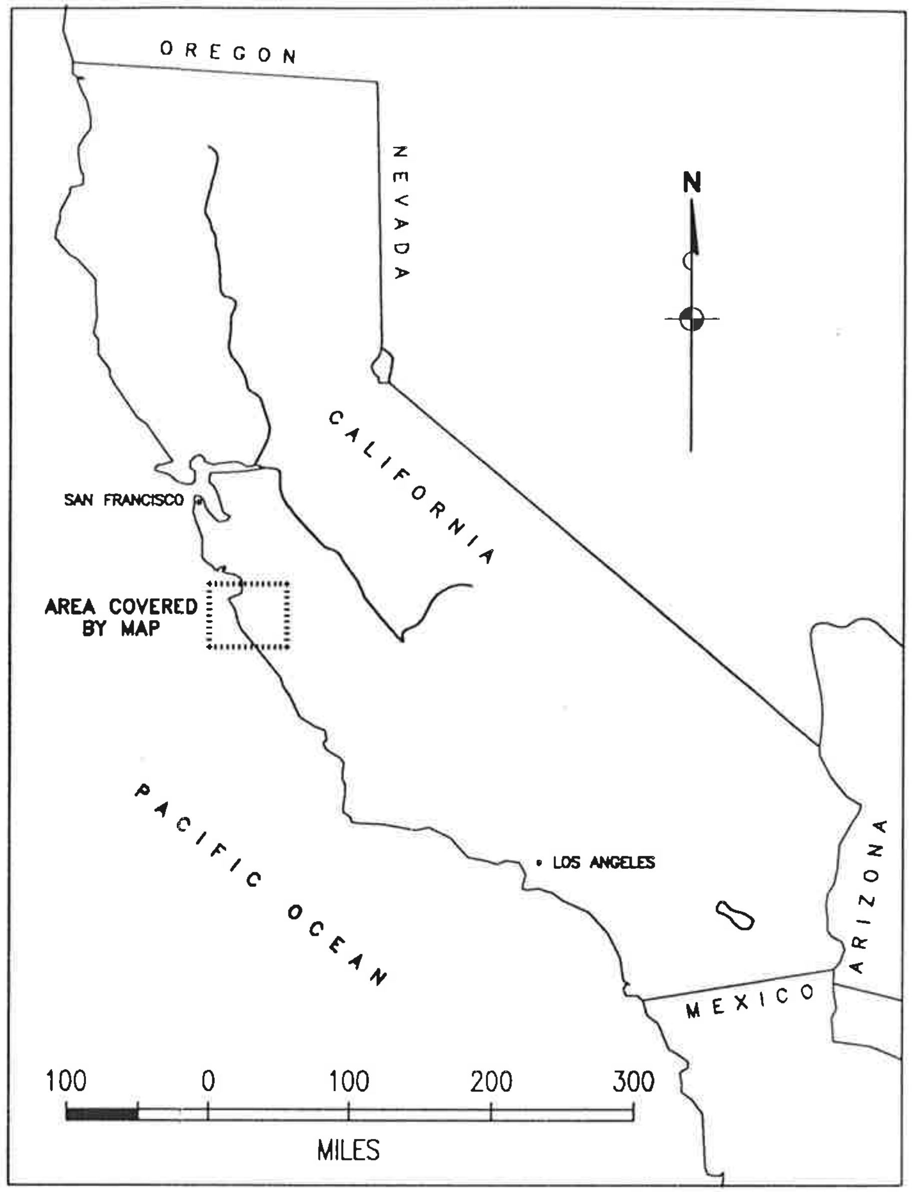
Drawings

PREPARED BY: HEJ REVIEWED BY: KBK DATE: 2/17/95





EXPLANATION

||||||||| DRAINAGE BASIN BOUNDARY

10 RIVER MILE

1 STATE HIGHWAY

EXISTING DAM

LOCATION MAP

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS
In Association With
MORRISON KNUDSEN CORPORATION
William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

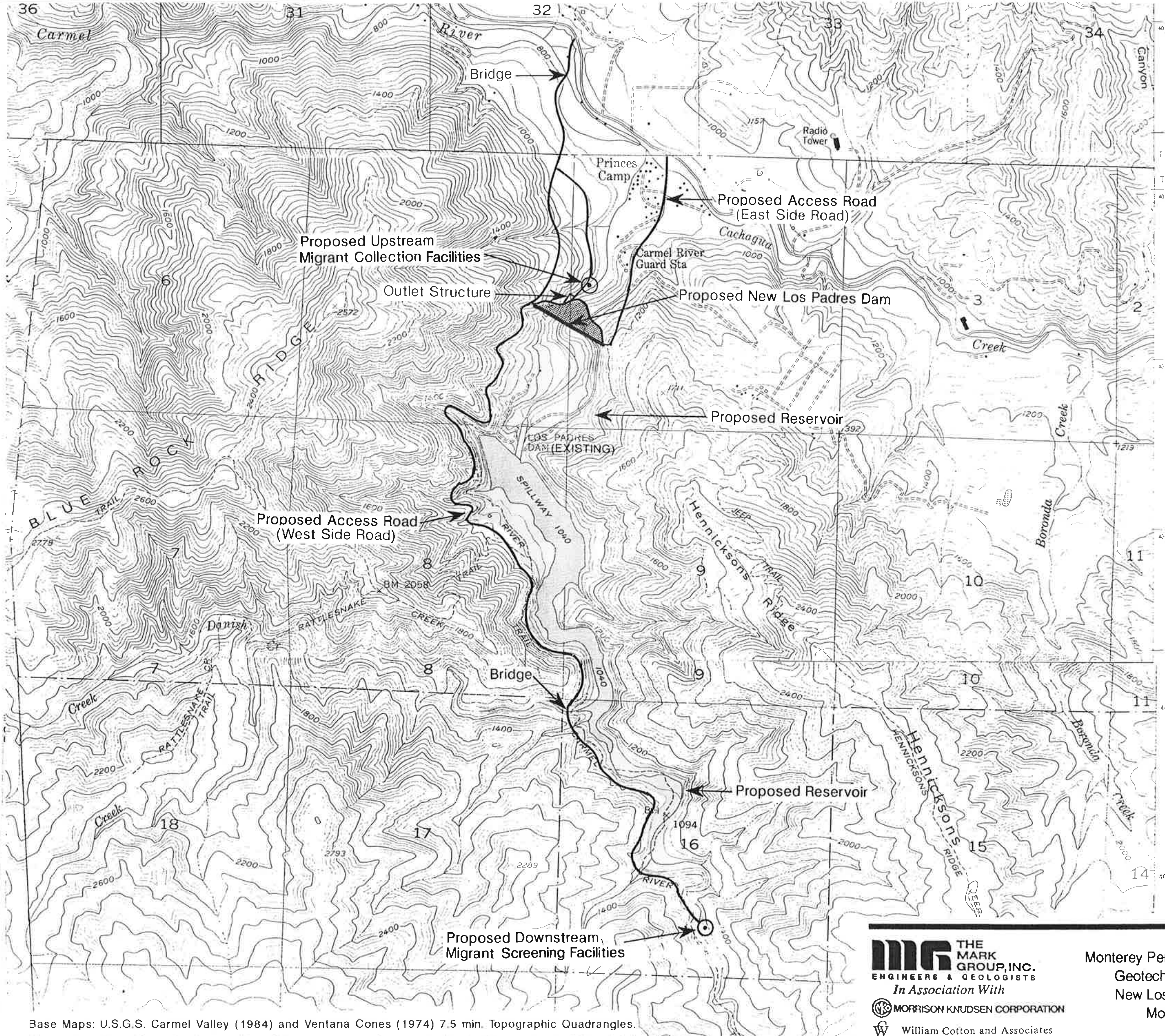
PROJECT NO.
94-1198801.80

DRAWING NO.
1-1

Prepared By JMH

Approved By KBK

Date 2/2/95



Base Maps: U.S.G.S. Carmel Valley (1984) and Ventana Cones (1974) 7.5 min. Topographic Quadrangles.

EXPLANATION

▩	Approximate Location - New Los Padres Dam
░	Approximate Limit of New Los Padres Reservoir
∼	Approximate Alignment of Access Roads
⊙	Approximate Location of Migrant Collection Facilities

N

SCALE

0 2000

1"=2000'

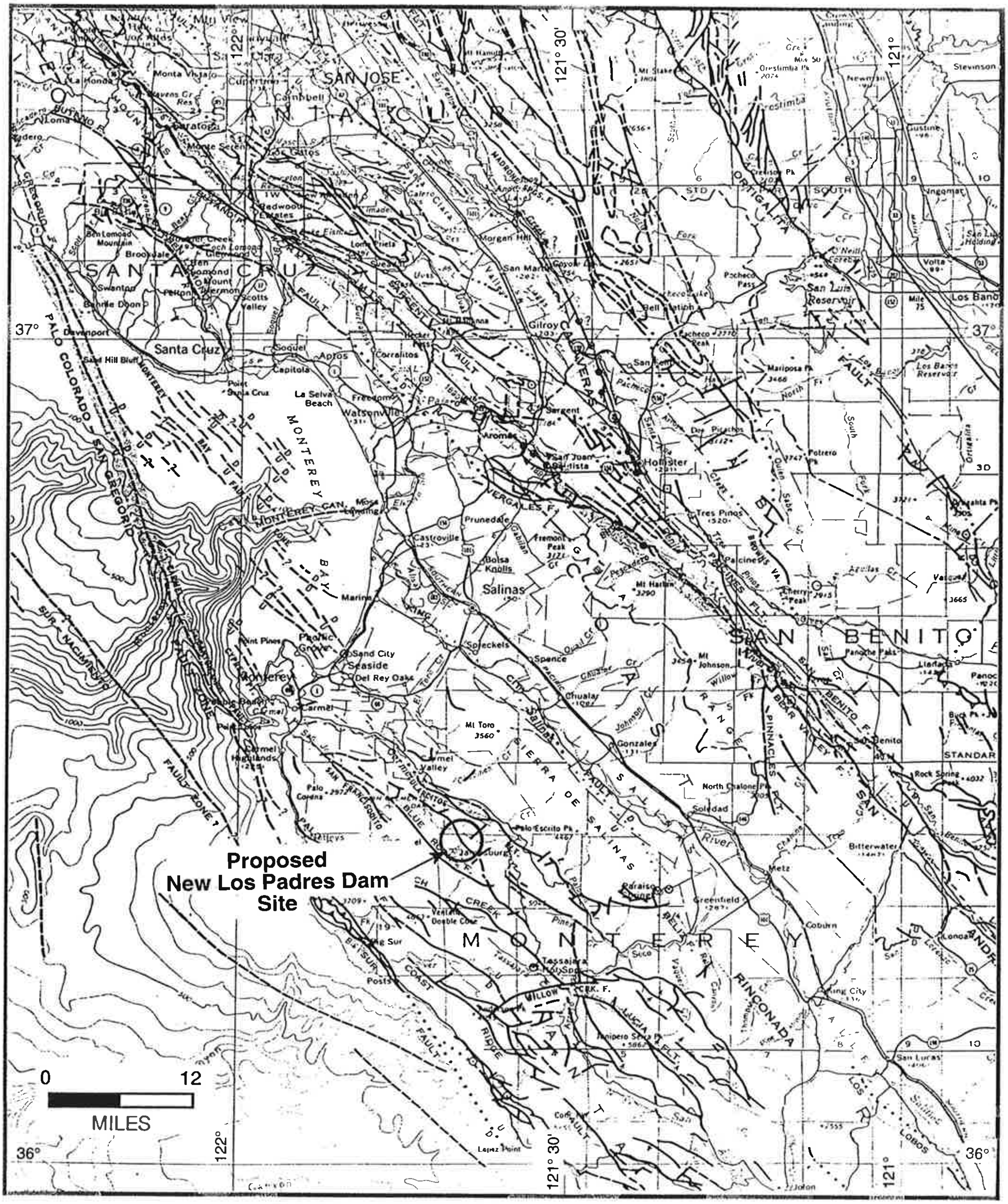
PROJECT SITE MAP

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS
In Association With
MORRISON KNUDSEN CORPORATION
William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

DRAWING NO.
1-2



REGIONAL FAULT MAP (FROM JENNINGS, 1975).

PREPARED BY WFC

REVIEWED BY WRC

DATE 3/8/95

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS
In Association With
MORRISON KNUDSEN CORPORATION
William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

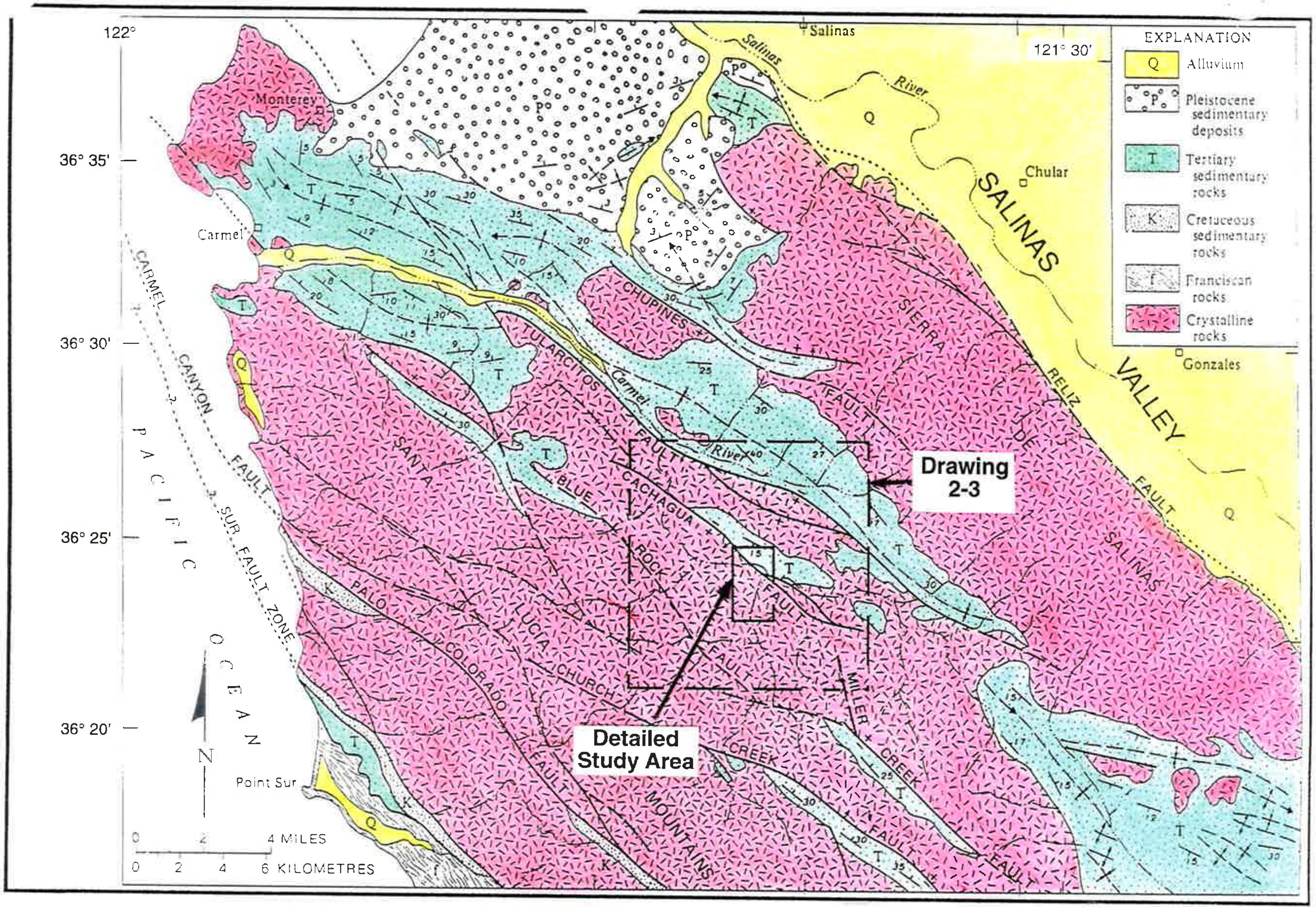
PROJECT NO.
94-1198801.80

DRAWING NO.
2-1

PREPARED BY WFC

REVIEWED BY WRC

DATE 3/8/95



GEOLOGY OF NORTHERN SANTA LUCIA RANGE (MODIFIED FROM DIBBLEE, 1976).

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2-2

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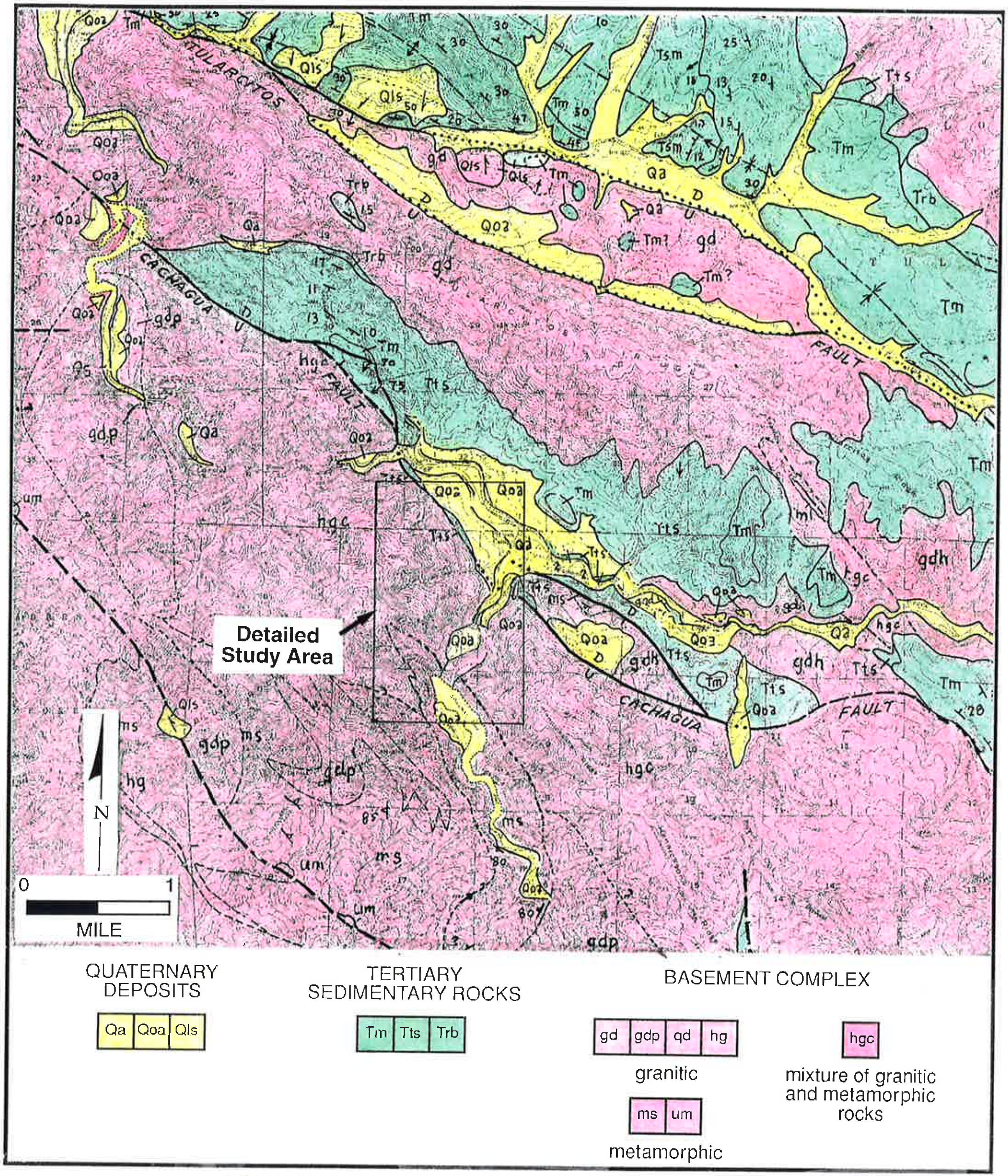

REGIONAL GEOLOGIC MAP (MODIFIED FROM DIBBLEE, 1972).

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2-3

Prepared By WAM
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Date 2/9/95

EXPLANATION

Approximate Limits of Borrow Areas

BORROW AREA LOCATIONS

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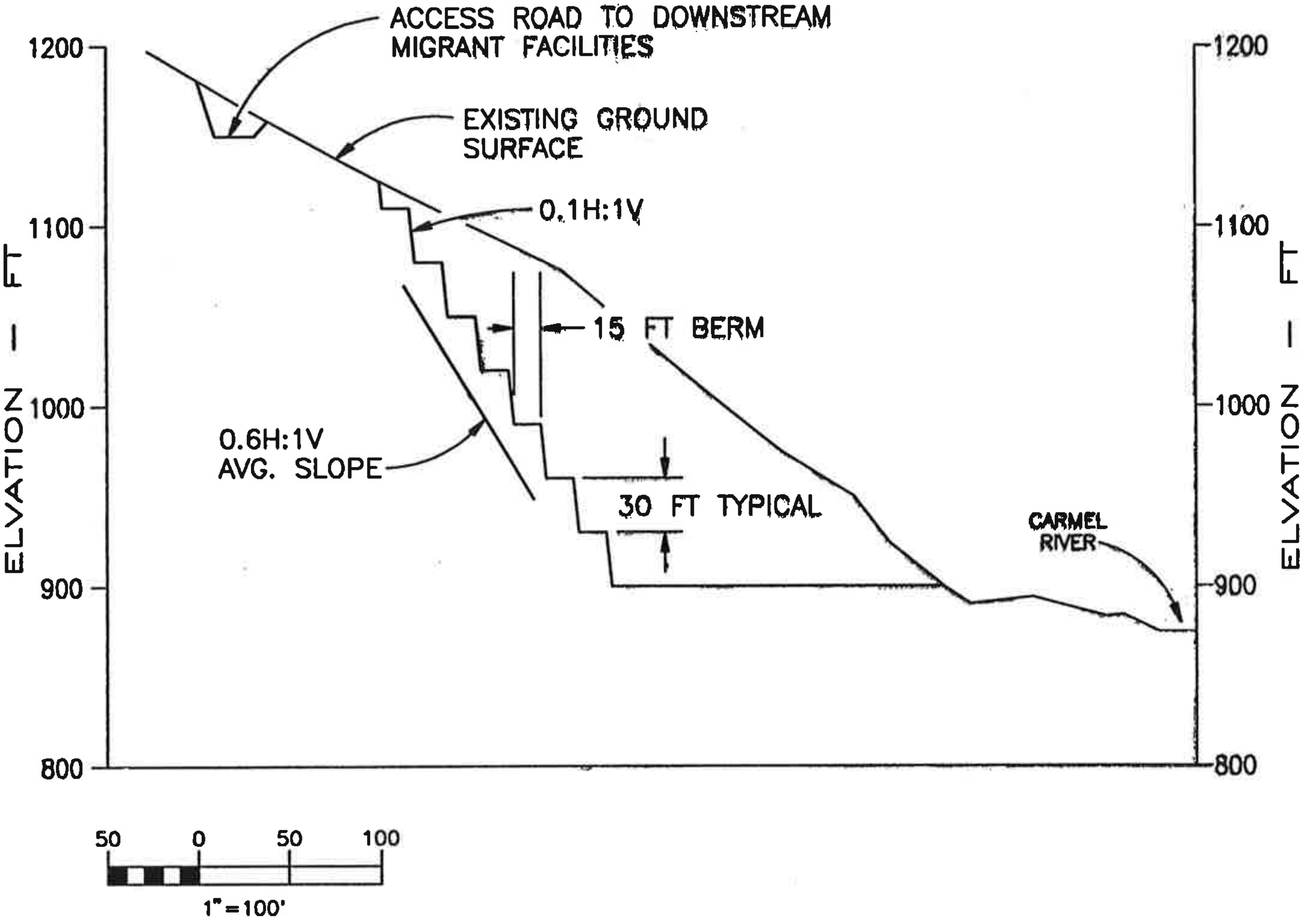

TYPICAL EXCAVATED SECTION AT BORROW AREA E

PREPARED BY: VEH REVIEWED BY: KBK DATE: 2-17-95

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7-1

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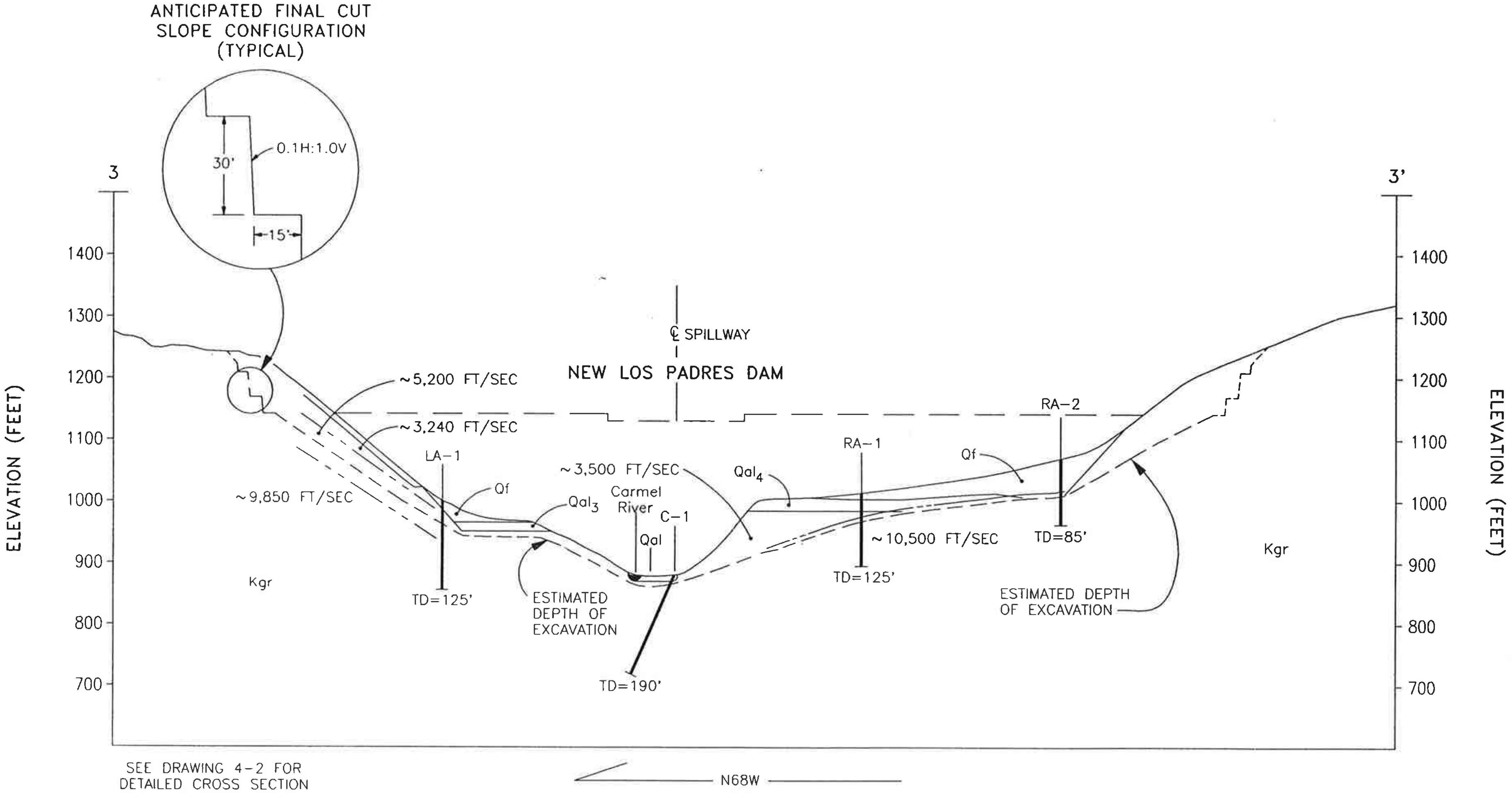

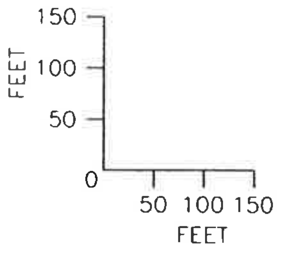

EXPLANATION

—— - - —— APPROXIMATE SEISMIC REFRACTION VELOCITY BOUNDARY

— — — — ESTIMATED DEPTH OF EXCAVATION

ESTIMATED DEPTH OF EXCAVATION FOR DAM FOUNDATION

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DRAWING NO.
7–2

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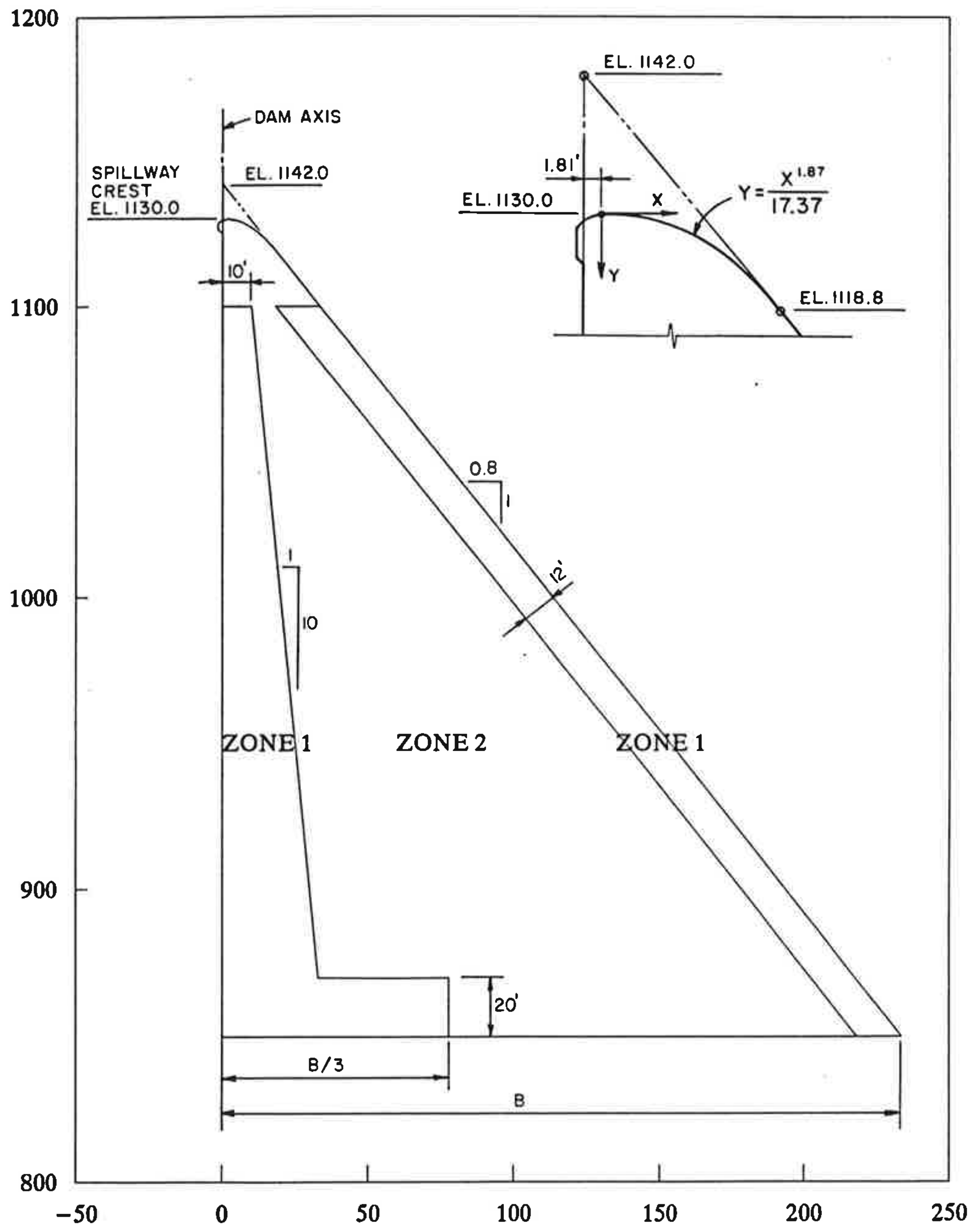

SPILLWAY SECTION OUTLINE

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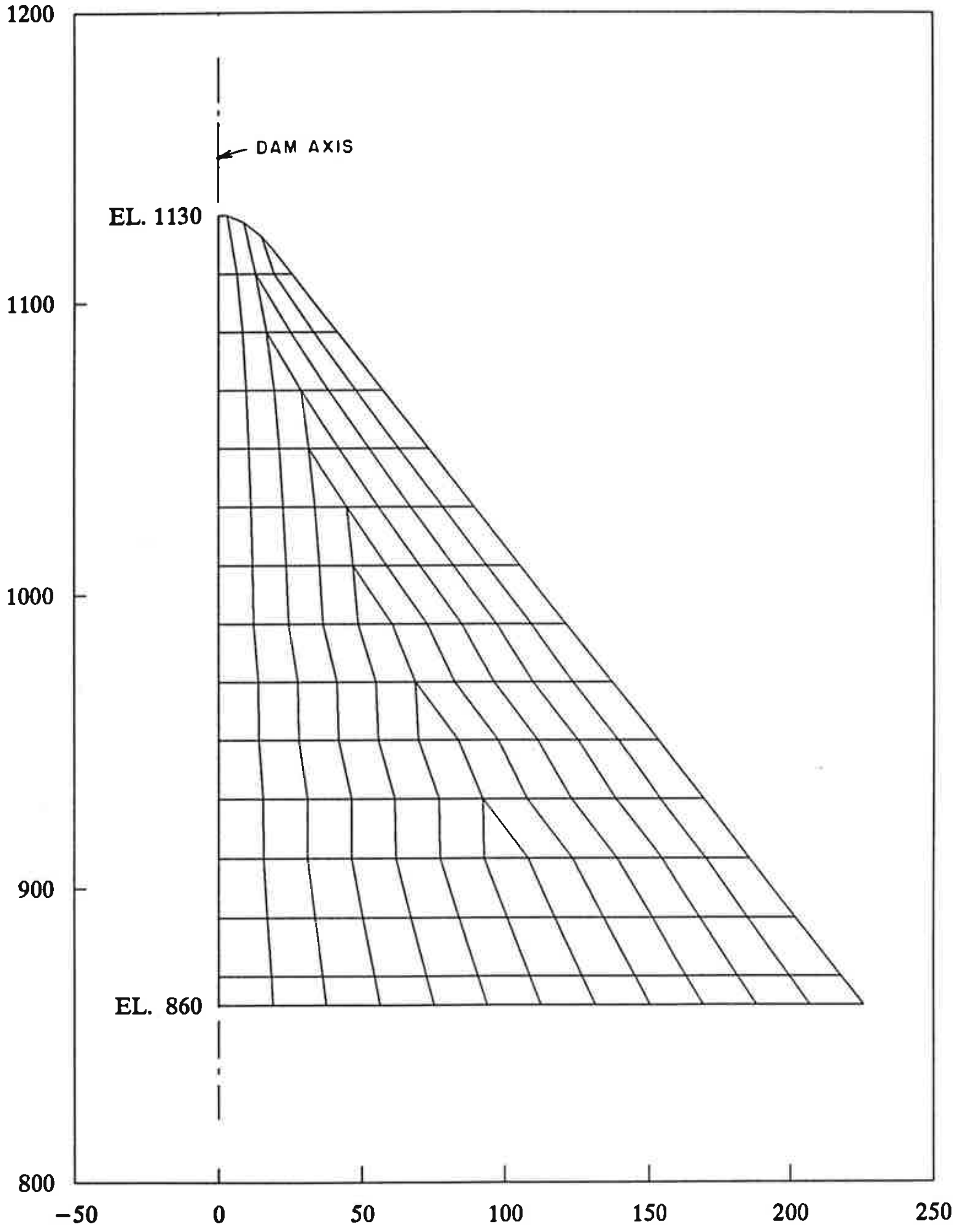
DRAWING NO.
8-1



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FEM MODEL

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8-2

STATIC LOADING CONDITIONS
- RESERVOIR WS EL. 1130.0
- DEAD LOAD
- UPLIFT PRESSURE AT BASE

SLIDING STABILITY

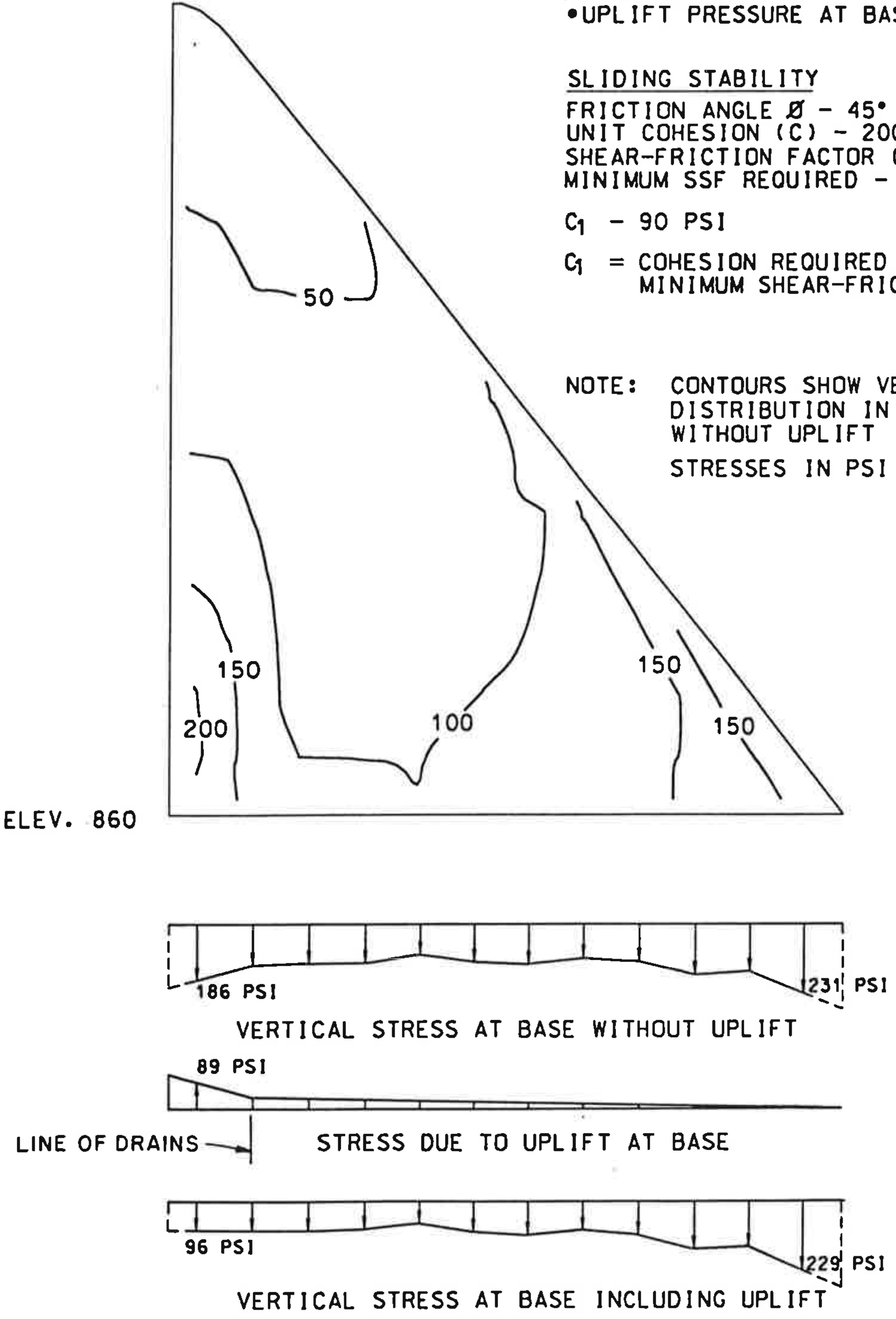
FRICTION ANGLE Ø - 45°
UNIT COHESION (C) - 200 PSI
SHEAR-FRICTION FACTOR (SSF) - 4.6
MINIMUM SSF REQUIRED - 3.0

C_1 - 90 PSI

C_1 = COHESION REQUIRED TO OBTAIN MINIMUM SHEAR-FRICTION FACTOR

NOTE: CONTOURS SHOW VERTICAL STRESS DISTRIBUTION IN PSI COMPUTED WITHOUT UPLIFT

STRESSES IN PSI



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STATIC VERTICAL STRESS DISTRIBUTION

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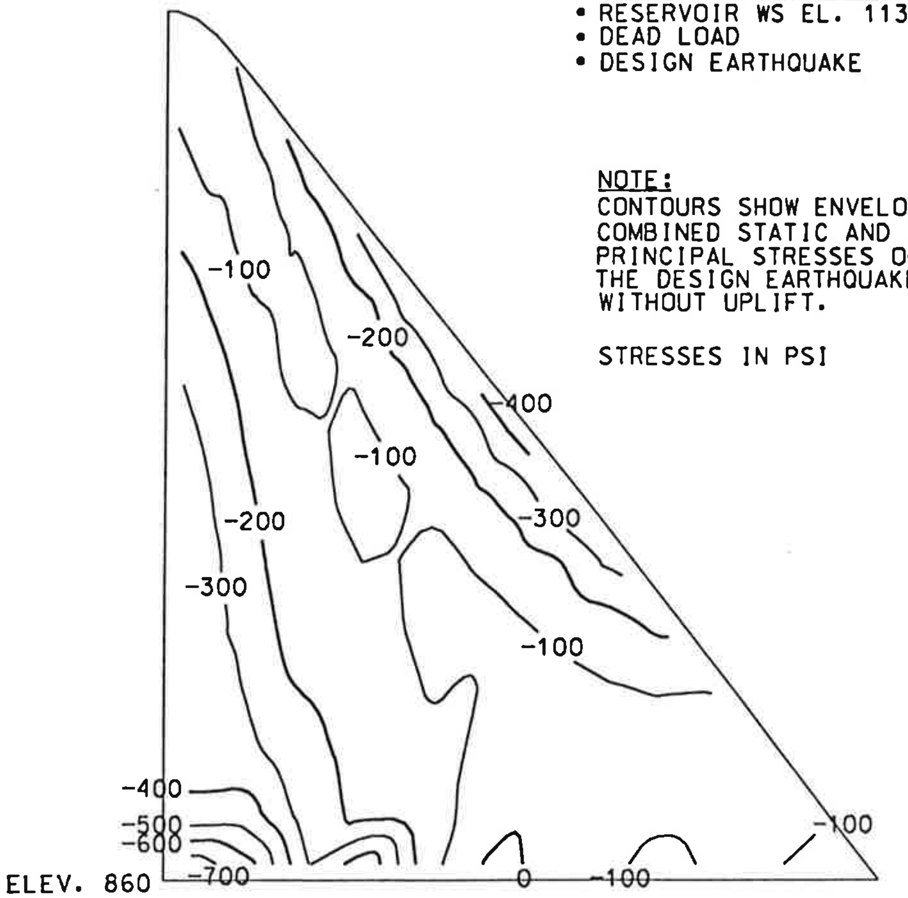
DRAWING NO.
8-3



LOADING CONDITIONS

- RESERVOIR WS EL. 1130.0
- DEAD LOAD
- DESIGN EARTHQUAKE

NOTE:
CONTOURS SHOW ENVELOPES OF MAXIMUM COMBINED STATIC AND DYNAMIC TENSILE PRINCIPAL STRESSES OCCURING DURING THE DESIGN EARTHQUAKE COMPUTED WITHOUT UPLIFT.

STRESSES IN PSI

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MAX. DYNAMIC PRINCIPAL TENSILE STRESS DISTRIBUTION

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PROJECT NO.
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DRAWING NO.
8-4

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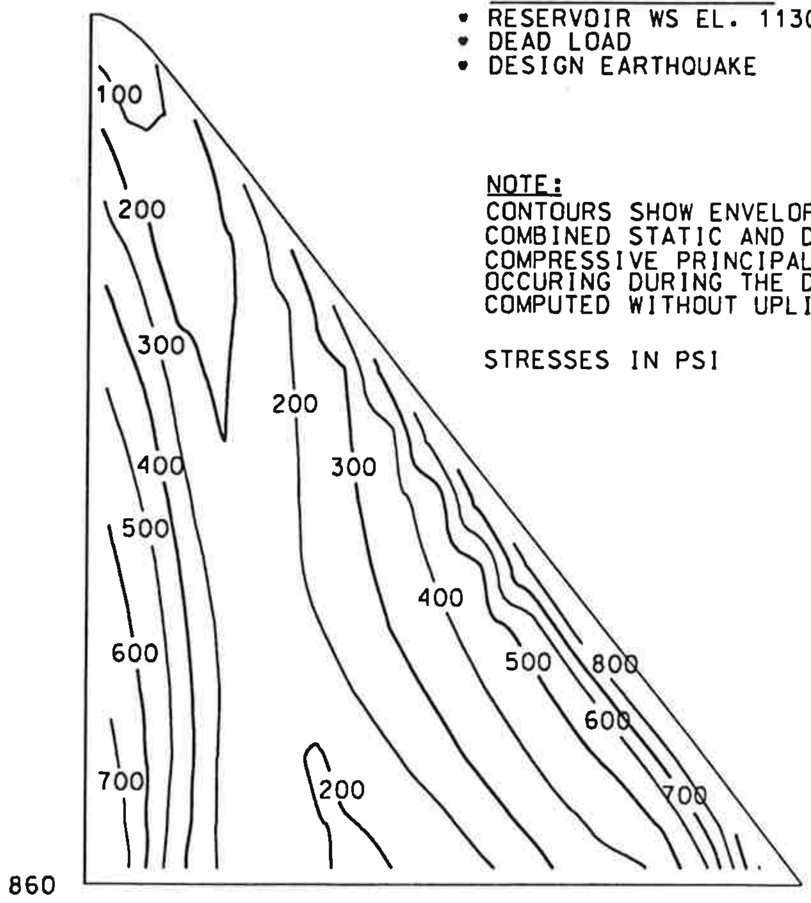

LOADING CONDITIONS

- RESERVOIR WS EL. 1130.0
- DEAD LOAD
- DESIGN EARTHQUAKE

NOTE:
CONTOURS SHOW ENVELOPES OF MAXIMUM COMBINED STATIC AND DYNAMIC COMPRESSIVE PRINCIPAL STRESSES OCCURING DURING THE DESIGN EARTHQUAKE COMPUTED WITHOUT UPLIFT

STRESSES IN PSI

MAX. DYNAMIC PRINCIPAL COMPRESSIVE STRESS DISTRIBUTION

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DRAWING NO.
8-5

STATIC LOADING CONDITIONS
- RESERVOIR WS EL. 1130.0
- DEAD LOAD
- UPLIFT PRESSURE AT BASE
- DESIGN EARTHQUAKE

SLIDING STABILITY AT 11.30 SECONDS

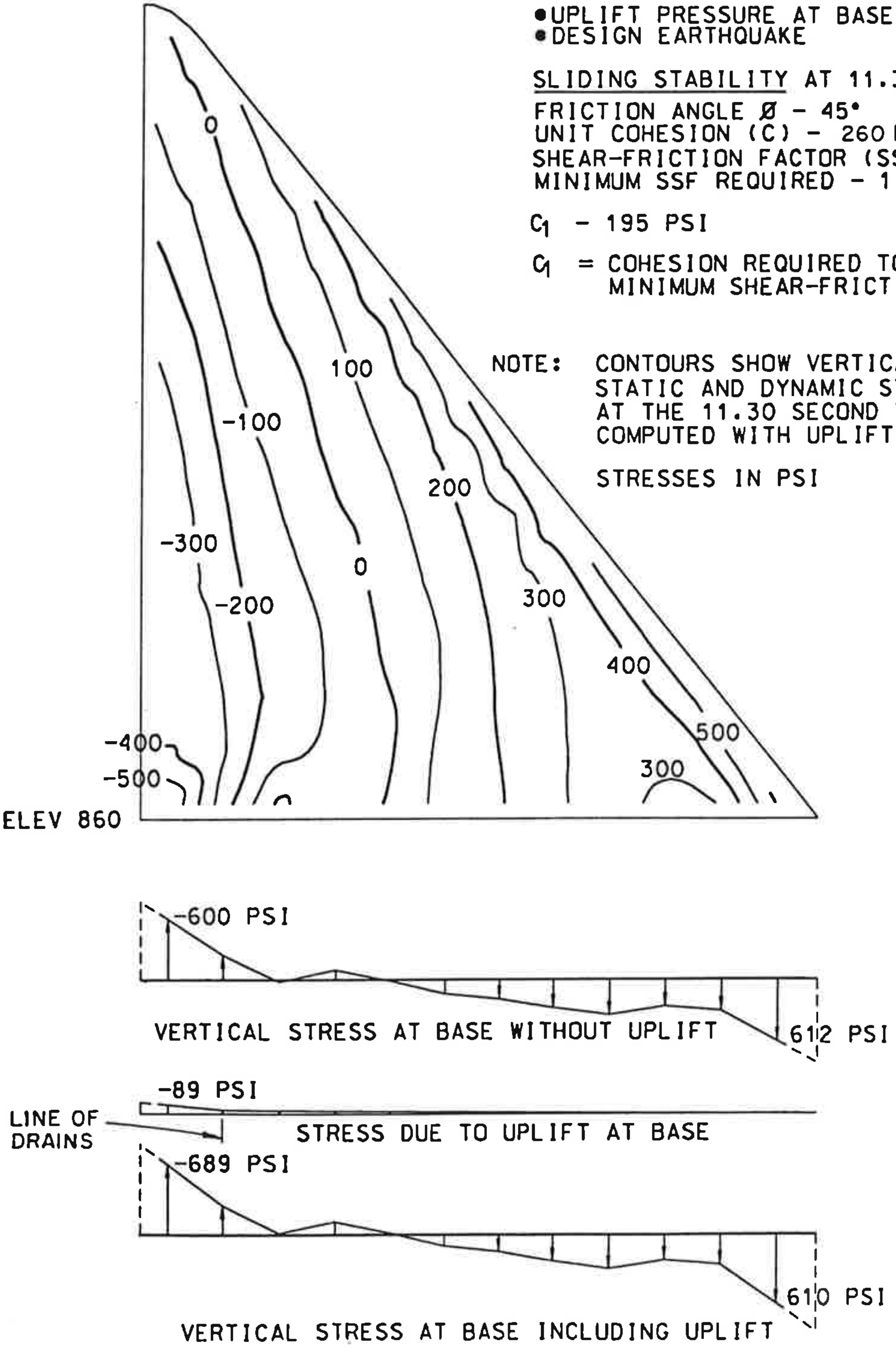
FRICTION ANGLE Ø - 45°
UNIT COHESION (C) - 260 PSI
SHEAR-FRICTION FACTOR (SSF) - 1.25
MINIMUM SSF REQUIRED - 1.1

C_1 - 195 PSI

C_1 = COHESION REQUIRED TO OBTAIN MINIMUM SHEAR-FRICTION FACTOR

NOTE: CONTOURS SHOW VERTICAL COMBINED STATIC AND DYNAMIC STRESSES OCCURING AT THE 11.30 SECOND TIME INTERVAL COMPUTED WITH UPLIFT.

STRESSES IN PSI



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DATE 2/9/95

VERTICAL STRESSES AT T=11.30 SECONDS

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DRAWING NO.
8-6

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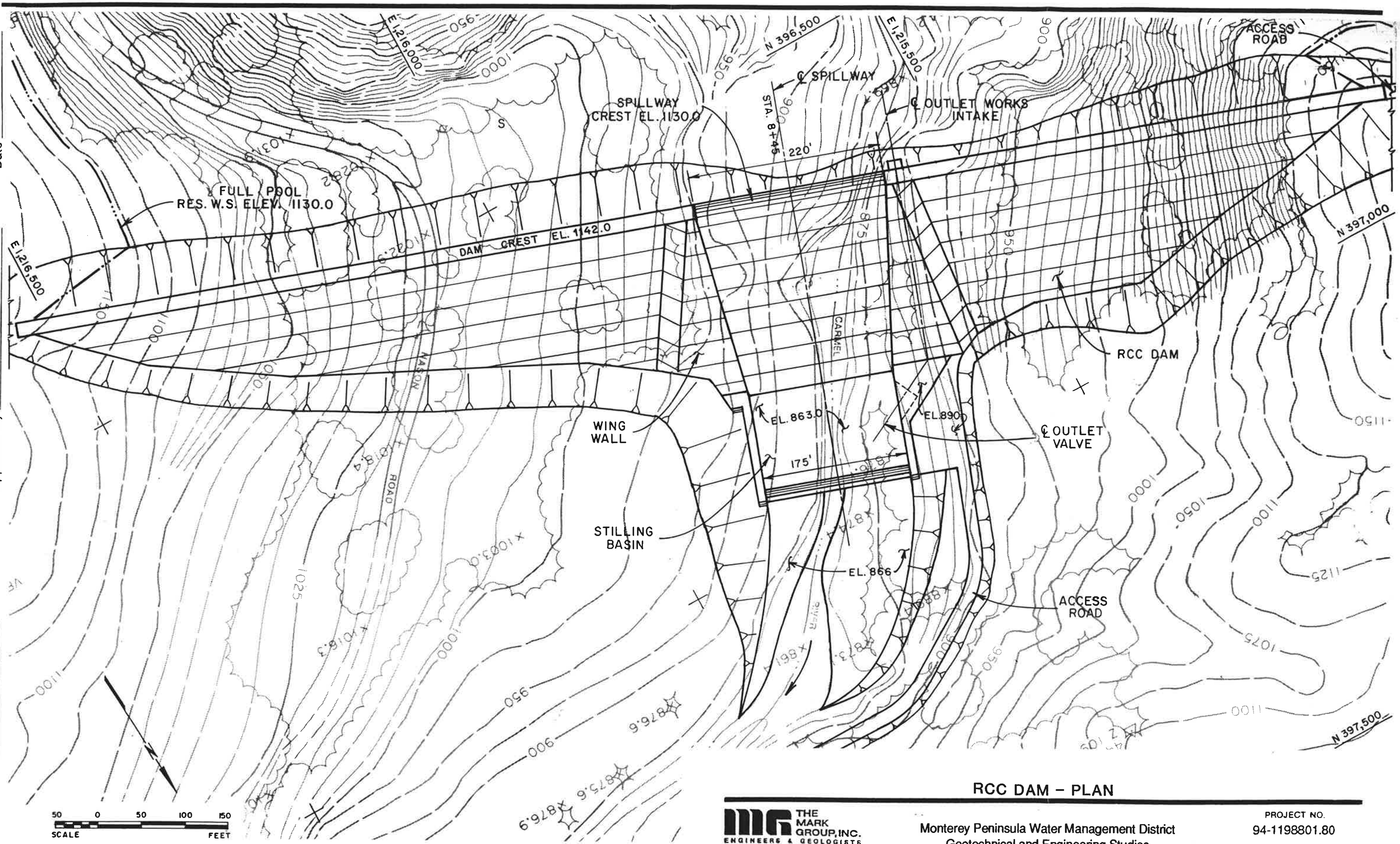
Approved By KBK

Date 2/9/95



RCC DAM - PLAN

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DRAWING NO
9-1

Date 2/9/95

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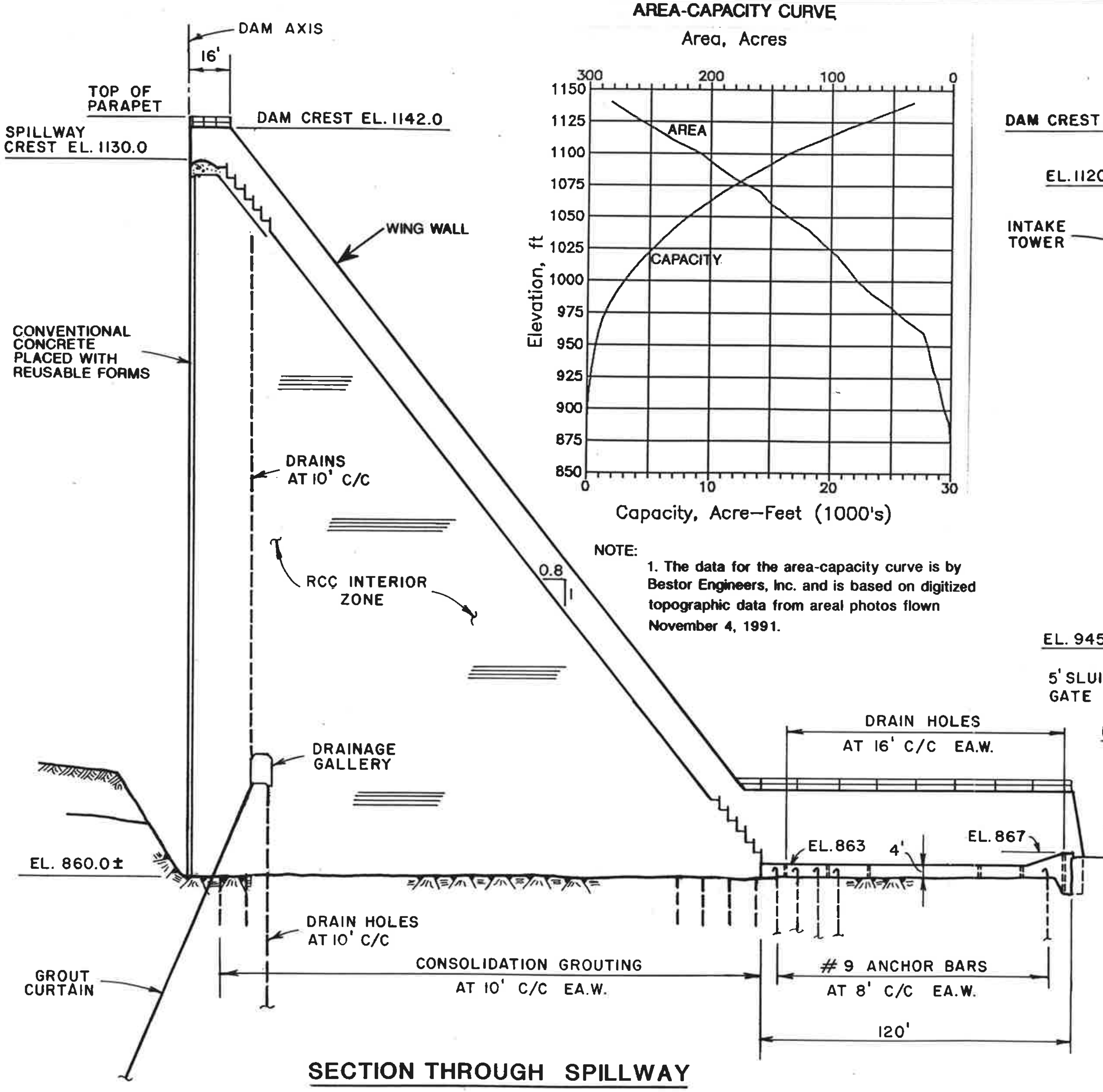
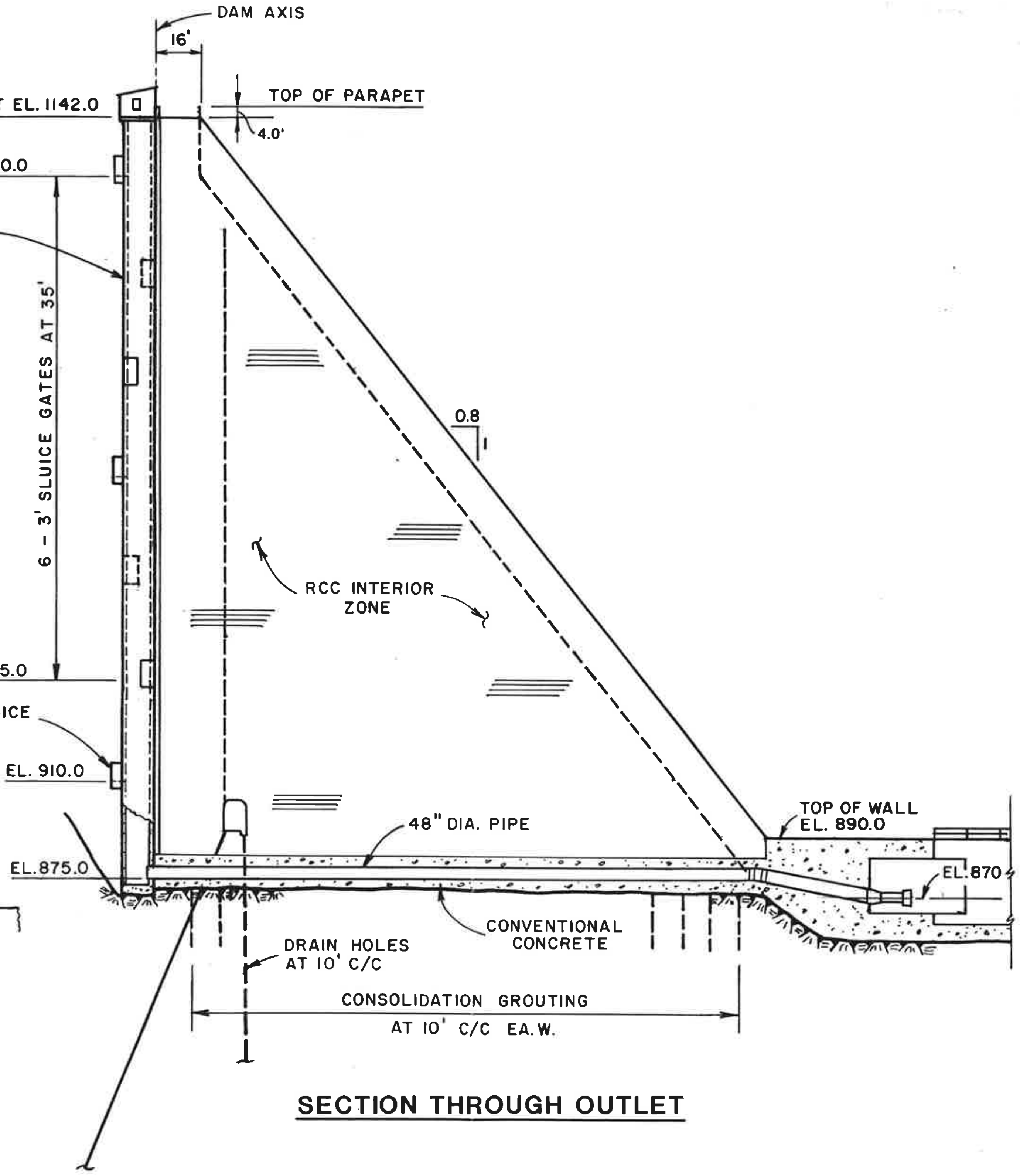


AREA-CAPACITY CURVE

NOTE:

1. The data for the area-capacity curve is by Bestor Engineers, Inc. and is based on digitized topographic data from areal photos flown November 4, 1991.

SECTION THROUGH SPILLWAY



SECTION THROUGH OUTLET

RCC DAM - SECTIONS

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PROJECT NO.
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DRAWING NO
9-2

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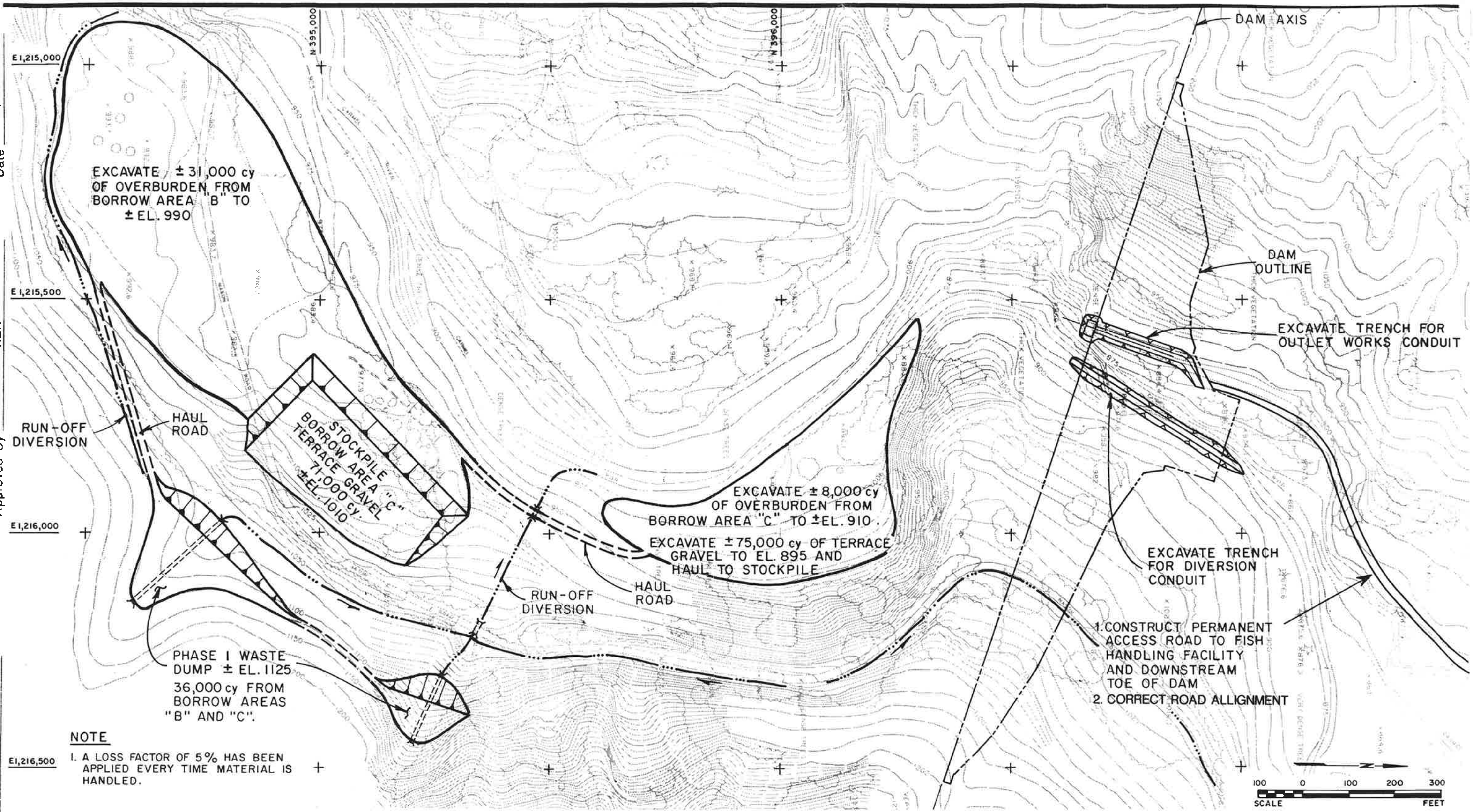
Approved By KBK

Date 2/9/95



NOTE

1. A LOSS FACTOR OF 5% HAS BEEN APPLIED EVERY TIME MATERIAL IS HANDLED.

CONSTRUCTION SEQUENCE - PHASE I

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Monterey County, California

PROJECT NO
94-1198801.80

DRAWING NO
10-1

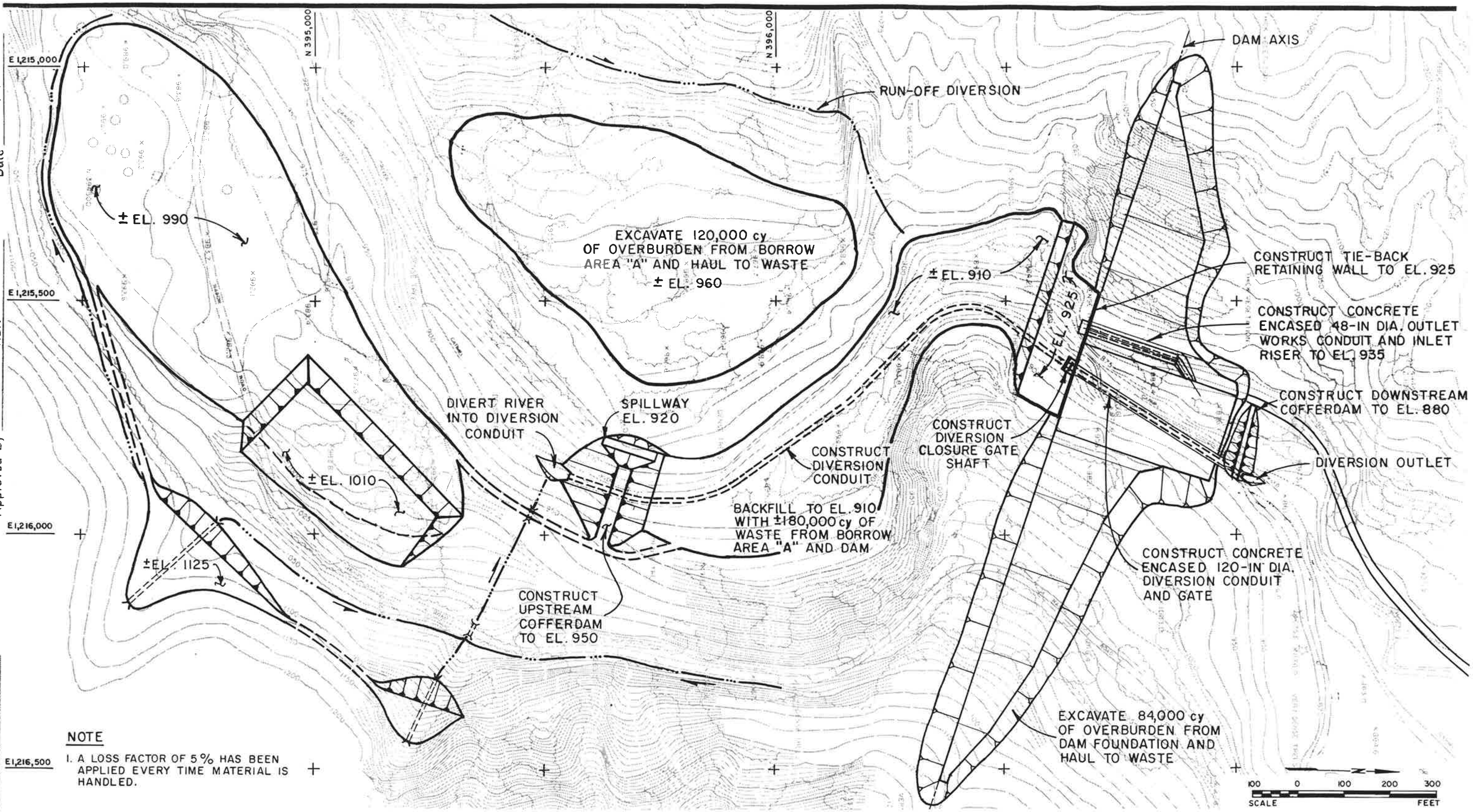
Date 2/9/95

KBK

Approved By

WAM

Prepared By



NOTE

1. A LOSS FACTOR OF 5% HAS BEEN APPLIED EVERY TIME MATERIAL IS HANDLED.

CONSTRUCTION SEQUENCE – PHASE II

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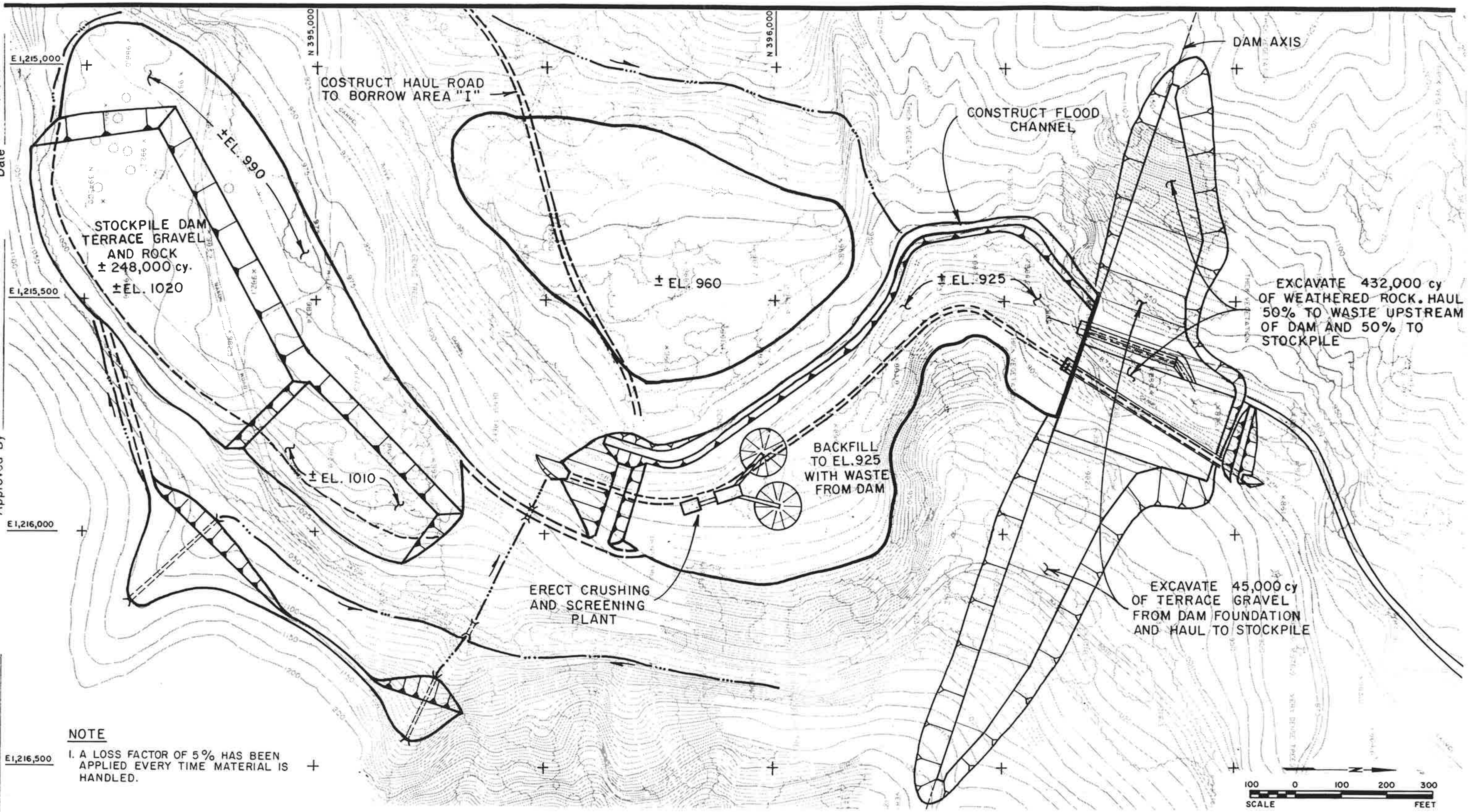
DRAWING NO
10-2



NOTE

1. A LOSS FACTOR OF 5% HAS BEEN APPLIED EVERY TIME MATERIAL IS HANDLED.

CONSTRUCTION SEQUENCE - PHASE III

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DRAWING NO
10-3

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Approved By KBK
Prepared By WAM

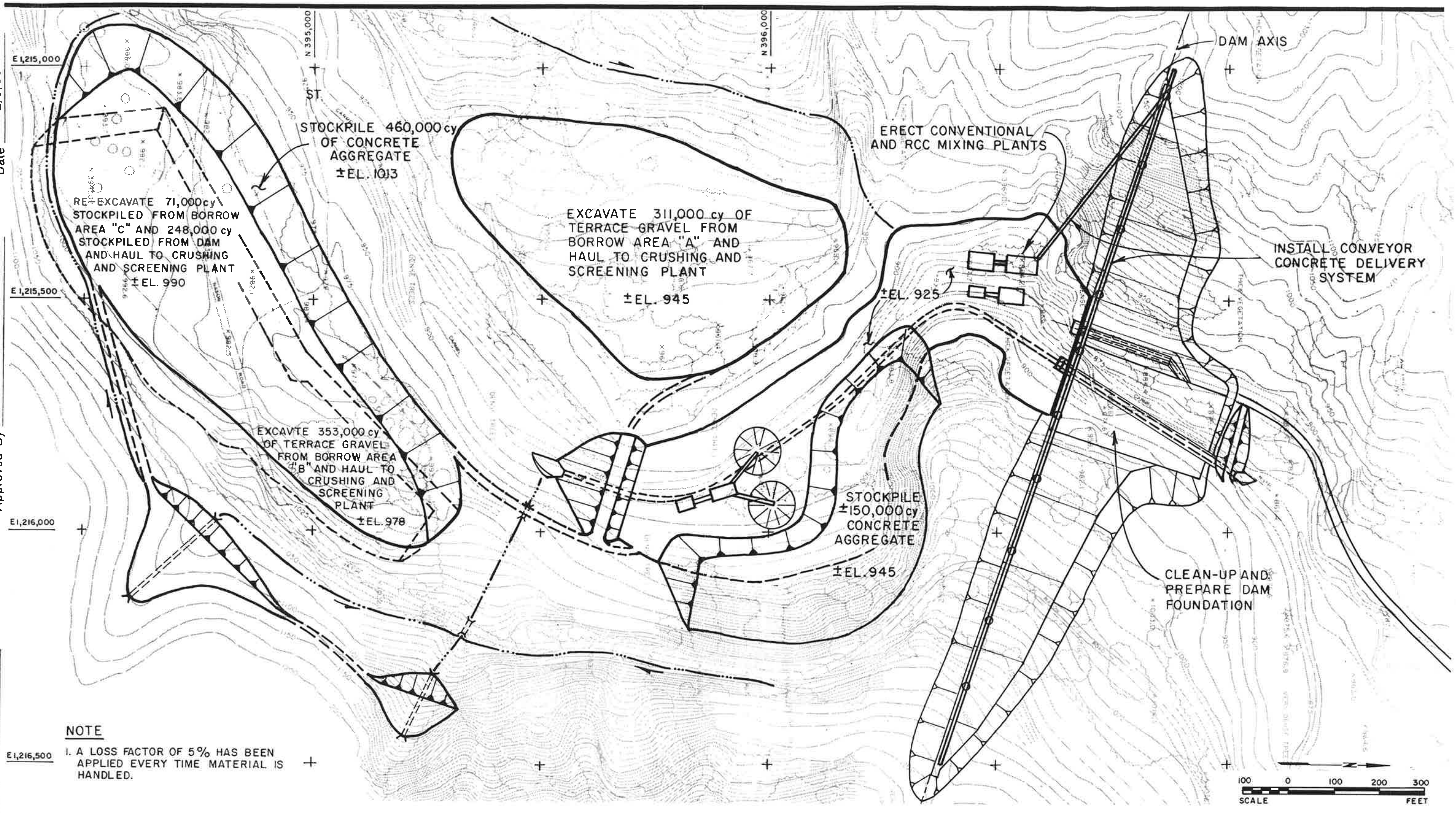
Prepared By

WAM

Approved By

KBK

Date 2/9/95



NOTE

1. A LOSS FACTOR OF 5% HAS BEEN APPLIED EVERY TIME MATERIAL IS HANDLED.

CONSTRUCTION SEQUENCE – PHASE IV

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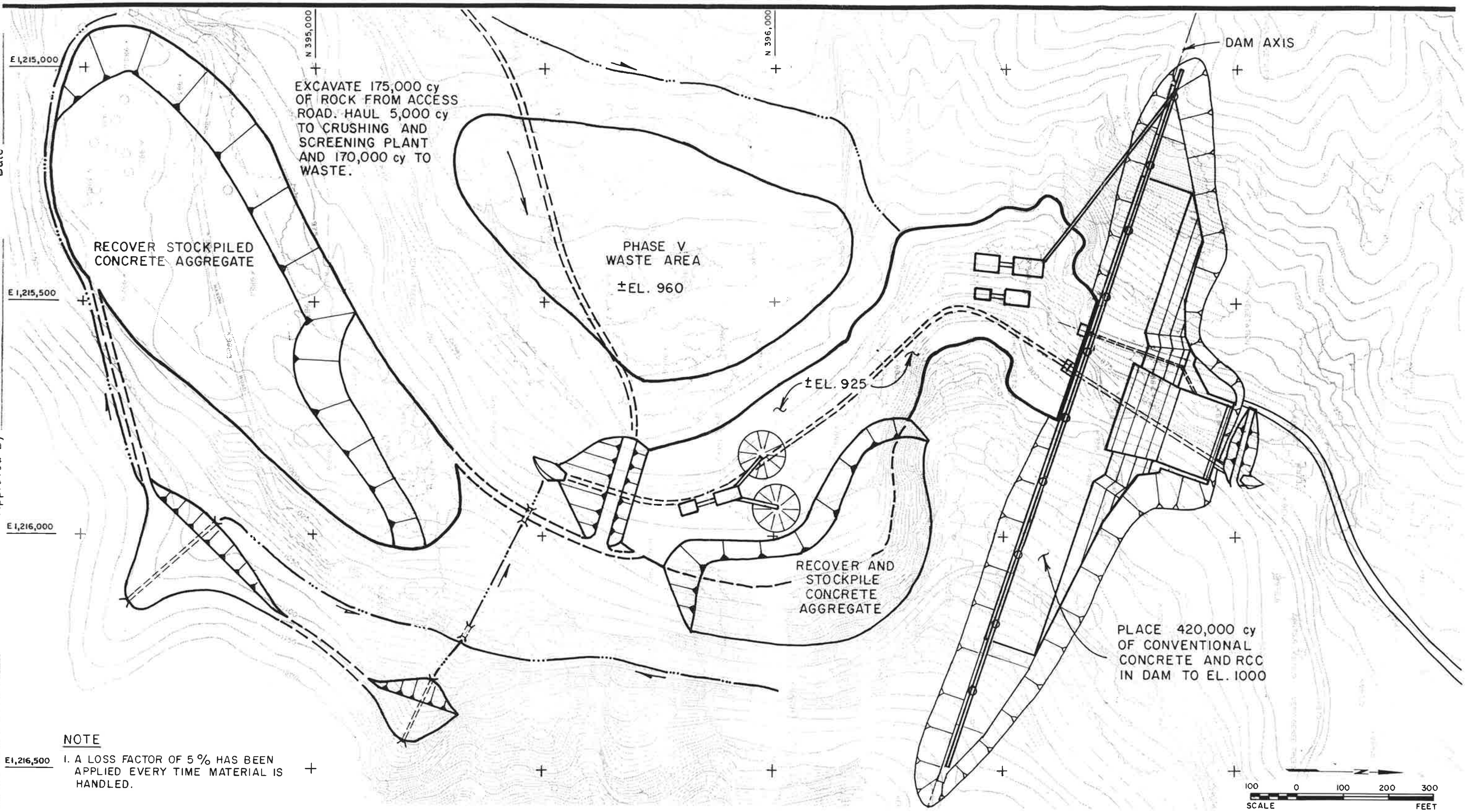

NOTE

1. A LOSS FACTOR OF 5 % HAS BEEN APPLIED EVERY TIME MATERIAL IS HANDLED.

CONSTRUCTION SEQUENCE – PHASE V

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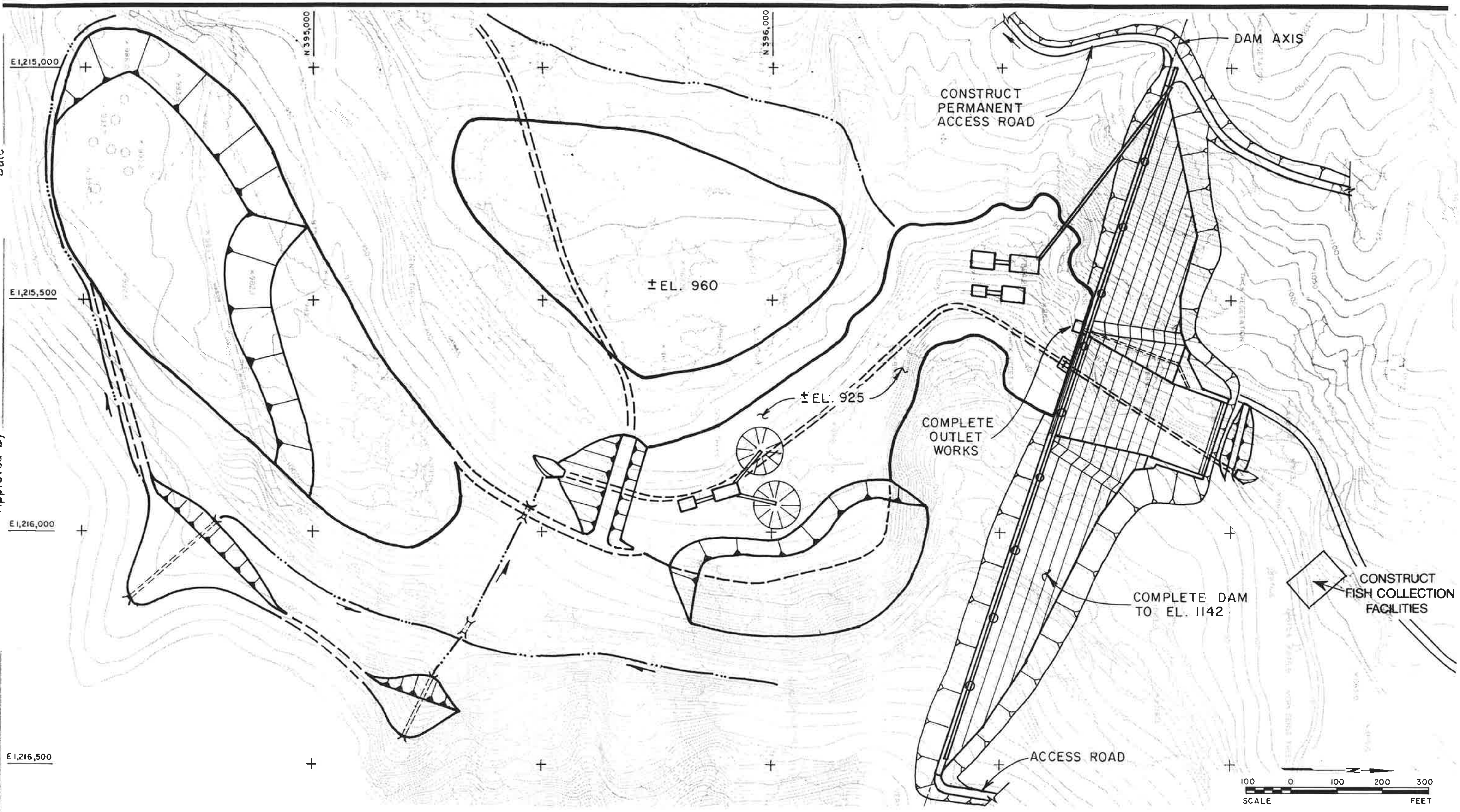

Construction Sequence - Phase VI (Sheet 1 of 2)

Date 2/9/95

KBK

Approved By

WAM

Prepared By

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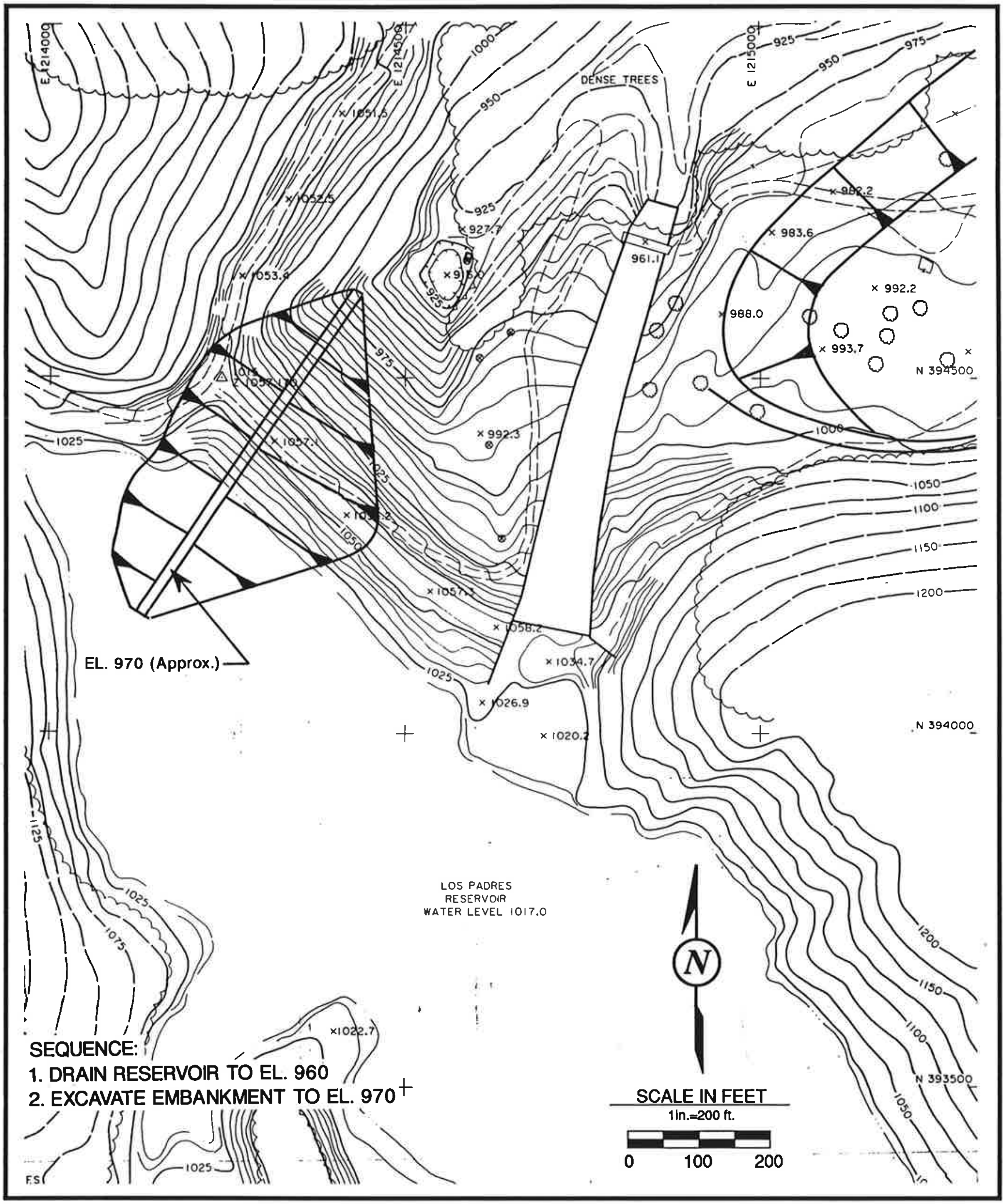
10-6



SEQUENCE:
1. DRAIN RESERVOIR TO EL. 960
2. EXCAVATE EMBANKMENT TO EL. 970

SCALE IN FEET
1in.=200 ft.
0 100 200

Construction Sequence - Phase VI (Sheet 2 of 2)

PREPARED BY HEJ REVIEWED BY KBK DATE 3/10/95

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DRAWING NO.
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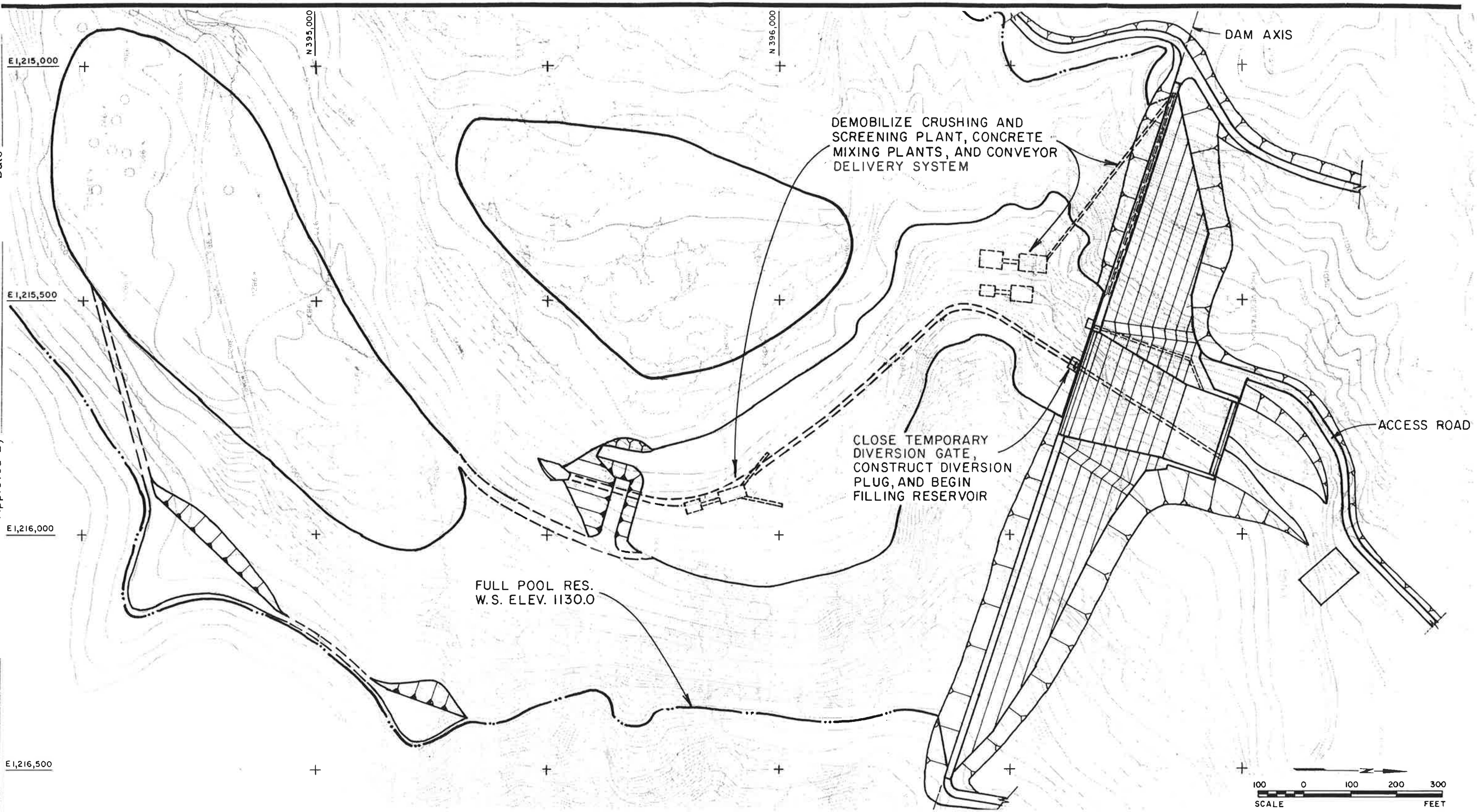
Date 2/9/95

Approved By KBK

Prepared By WAM



CONSTRUCTION SEQUENCE - PHASE VII

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DRAWING NO
10-8

Appendix A

APPENDIX A

SEISMIC REFRACTION SURVEY

New Los Padres Water Supply Project

Monterey County, California

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  1.2 Purpose and Scope of Investigation A-1
  1.3 Geologic Setting A-1

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  2.1 Method and Equipment A-2
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A-2 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-2
A-3 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-3
A-4 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-4
A-5 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-5

FIGURES (Cont.)

A-6 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-6
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A-12 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-12
A-13 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-13
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A-16 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-16
A-17 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-17
A-18 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line No. S-18
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1.0 INTRODUCTION

This appendix describes the methods and equipment used, interpreted results, limitations and conclusions of a seismic refraction geophysical survey of the New Los Padres Dam site in Monterey County, California. This survey was conducted in accordance with our proposal and contract.

1.1 Project Description

The project consists of the proposed construction of a new water storage dam (New Los Padres Dam) for the Monterey Peninsula Water Management District below the existing Los Padres Dam in Monterey County, California. In addition to the foundation area of the dam and fish handling facilities, areas needed for borrow and quarrying were also surveyed.

1.2 Purpose and Scope of Survey

The purpose of our investigation was to: 1) investigate subsurface conditions in the vicinity of the proposed dam site using the seismic refraction geophysical method, 2) evaluate the depth to bedrock (and indications of depth of weathering and/or fracturing) as well as the rippability of materials in the vicinity of the proposed dam, borrow and quarry areas, and fish handling facilities, and 3) evaluate an area of suspected faulting to help constrain the location of the Cachagua fault.

The scope of work included a field seismic refraction survey followed by geophysical analysis of the acquired data, preliminary consultation with the project team, and preparation of this report.

1.3 Geologic Setting

The project area is generally characterized by steeply sloping hillside terrain underlain by colluvial soil materials over either Mesozoic crystalline basement or Tertiary sedimentary rocks, and more moderately to gently sloping terrain underlain by alluvial fan and river terrace deposits over bedrock.

2.0 SEISMIC REFRACTION SURVEY

A total of twenty-two (22) individual seismic refraction lines* with a combined spread length of 9,100 feet were recorded in the general area of the proposed dam. These seismic refraction lines were recorded from September 28, to November 18, 1994 in the locations shown on Drawing 4-1. The interpreted results are presented on figures A-1 through A-20, with rippability information presented on Figure A-21. The method and equipment used, the interpreted results, limitations and a summary of pertinent conclusions are discussed in the following sections of this report.

---

* For the purposes of this report, a seismic refraction line is defined as twelve to twenty-four geophones spaced at equal intervals of 10 to 25 feet along a straight line and monitored simultaneously while a sledge hammer is repeatedly impacted off each end and at the center of the line.

2.1 Method and Equipment Used

The seismic refraction survey procedure for this project consisted of placing twelve (12) to twenty-four (24) geophones in as straight a line as practical (in plan) spaced at 10- to 25-foot intervals along as constant a slope as practical (in profile). A large sledge hammer was impacted at 10 to 12.5 feet off both ends of each line and at the center of each line. The hammer impacts generated seismic compression waves which were refracted through subsurface materials and received by the deployed geophones. The signal from the energy source initiation (time break) started the instrument sweep as signals from the geophones were monitored (amplified, filtered and stacked) simultaneously by a digital seismograph with an on-board computer and displayed graphically in analog form on the built-in computer monitor. Digital records stored in the computer were field checked, stored on magnetic disk and returned to our office for printing, data reduction and interpretation.

Seismic refraction lines were surveyed for location and elevation using hand level, Brunton compass and measuring tape methods. Lines were marked with stakes in the field and located on the base map. Locations and relative elevations should be considered approximate.

The data reduction and interpretation procedure consisted of the following sequence of tasks:

- computerized selection of first breaks of compression waves (P-waves) from the digital records of the seismic system computer,
- visual adjustment of first break picks by observing the analog record,
- plotting of time-distance graphs utilizing raw data,
- preliminary determination of apparent velocities,
- plotting of elevation data along the profiles,
- measurement of differences between actual geophone elevations and a constant slope profile,
- computer analysis of preliminary apparent velocities and elevation differences to determine travel-time corrections,
- adjustment of the time-distance graphs and refinement of apparent velocity determinations satisfying reciprocity,
- comparison of time-distance and velocity data with a catalog of subsurface structures to interpret an appropriate seismic refraction model,
- computer analysis, using computer program developed by Shires (1983), involving principles published by Mooney (1977), satisfying the condition of reciprocity, travel-time = distance/velocity, and Snell's Law of Refraction of apparent velocity and intercept time data to determine depths of refractors, true velocities, dips of refractors, and angles of wave incidence (seismic ray paths),

- measurement of time deviations from "best fit" apparent velocity slopes on the time-distance graphs,

- computer analysis of apparent velocities and time deviations to determine refractor profile corrections,

- adjustment of refractor depths to reflect time deviations,

- correlation of results with known geologic factors (from mapping and/or borehole logs), with adjacent or overlapping seismic refraction data, and

- final preparation of interpreted subsurface velocity profiles.

The equipment used for the seismic refraction survey consisted of twelve (12) to twenty-four (24) geophones at one time of 10 Hz natural frequency. The geophones were connected to 10- to 25-foot take-out spacing cables using Mueller clips. The combination seismograph/oscillograph used was a 24-channel ABEM™ Terraloc Mark 3 Seismic System mounted on a pack frame for portability.

The energy source consisted of a 16-pound sledge hammer equipped with a seismograph triggering mechanism. The sledge hammer was repeatedly impacted on a steel plate placed in a cleared area on the ground surface. Repeated impact signals were stacked for each seismic record.

2.2 Interpreted Results

2.2.1 Borrow Areas A and G

Seismic refraction lines S-1 through S-5 were recorded in proposed Borrow Areas A and G in the locations shown on Drawing 4-1. Lines S-1 and S-2 were recorded in an east-west direction in the northerly portion of the borrow area, lines S-3 and S-4 were recorded in an east-west direction in the southerly portion of the borrow area, and Line S-5 was recorded in a north-south direction in the easterly portion of the borrow area. Results are presented on figures A-1 through A-5.

Lines S-1 through S-5 are interpreted to be underlain by four (4) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1030 to 1330 ft/sec) materials to a thickness of 3 to 12 feet beneath the ground surface. This upper zone corresponds to surficial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (2780 to 3730 ft/sec) materials from 3 to 12 feet to 23 to 68 feet beneath the ground surface. This zone corresponds to alluvial fan materials, terrace deposits and/or deeply weathered granitic bedrock. The underlying zone is characterized by medium to high velocity (4800 to 8020 ft/sec) materials from 23 to 68 feet to 56 to 132 feet beneath the ground surface. This zone probably corresponds to less weathered or more saturated granitic bedrock materials. The deepest zone encountered is characterized by high velocity (11,440 to 17,970 ft/sec) materials from 56 to 132 feet on down to the depth limit of information obtained (about 96 to 196 feet). This zone probably corresponds to relatively unweathered granitic bedrock materials.

2.2.2 Borrow Area B

Seismic refraction lines S-6 through S-9 were recorded in proposed Borrow Area B in the locations shown on Drawing 4-1. Lines S-6 and S-7 were recorded in a northeast-southwest direction in the southerly portion of the borrow area, Line S-8 was recorded in a northwest-southeast direction in the central portion of the borrow area, and Line S-9 was recorded in a north-south direction in the northeastern portion of the borrow area. Results are presented on figures A-6 through A-9.

Lines S-6 through S-9 are interpreted to be underlain by four (4) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1340 to 2090 ft/sec) materials to a thickness of 2 to 13 feet beneath the ground surface. This upper zone corresponds to surficial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (3150 to 3990 ft/sec) materials from 2 to 13 feet to 11 to 32 feet beneath the ground surface. This zone corresponds to terrace deposits and/or deeply weathered granitic and metamorphic bedrock. The underlying zone is characterized by medium velocity (4310 to 6540 ft/sec) materials from 11 to 32 feet to 36 to 74 feet beneath the ground surface. This zone probably corresponds to less weathered or more saturated granitic and metamorphic bedrock materials. The deepest zone encountered is characterized by high velocity (10,300 to 20,110 ft/sec) materials from 36 to 74 feet on down to the depth limit of information obtained (about 140 to 196 feet). This zone probably corresponds to relatively unweathered granitic and metamorphic bedrock materials.

2.2.3 Borrow Area C

Seismic refraction lines S-14, S-15 and S-18 were recorded in proposed Borrow Area C in the locations shown on Drawing 4-1. Lines S-14 and S-15 were recorded in a northwest-southeast direction and Line S-15 was recorded in an east-west direction in the central portion of the borrow area. Results are presented on figures A-14, A-15 and A-18.

Lines S-14, S-15 and S-18 are interpreted to be underlain by four (4) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1330 to 2400 ft/sec) materials to a thickness of 2 to 7 feet beneath the ground surface. This upper zone corresponds to surficial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (3820 to 5680 ft/sec) materials from 2 to 7 feet to 12 to 20 feet beneath the ground surface. This zone corresponds to alluvial fan materials, terrace deposits and/or deeply weathered granitic bedrock. The underlying zone is characterized by high velocity (8320 to 10,280 ft/sec) materials from 12 to 20 feet to 35 to 57 feet beneath the ground surface. This zone probably corresponds to less weathered or more saturated granitic bedrock materials. The deepest zone encountered is characterized by higher velocity (13,170 to 18,180 ft/sec) materials from 35 to 57 feet on down to the depth limit of information obtained (about 83 to 140 feet). This zone probably corresponds to relatively unweathered granitic bedrock materials.

2.2.4 Borrow Area D (Right Dam Abutment)

Seismic refraction lines S-10, S-12 and S-13 were recorded in proposed Borrow Area D in the locations shown on Drawing 4-1. Lines S-10 and S-13 were crossed in a general easterly-westerly direction across the upper plateau of the right abutment borrow area, and Line S-12 was recorded in a north-south direction along the western rim of the upper plateau. Results are presented on figures A-10, A-12 and A-13.

Lines S-10, S-12 and S-13 are interpreted to be underlain by three (3) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1090 to 1420 ft/sec) materials to a thickness of 2 to 10 feet beneath the ground surface. This upper zone corresponds to surficial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (3290 to 3640 ft/sec) materials from 2 to 10 feet to 42 to 81 feet beneath the ground surface. This zone corresponds to alluvial fan materials, terrace deposits and/or deeply weathered granitic and metamorphic bedrock. The deepest zone encountered is characterized by high velocity (7930 to 11,100 ft/sec) materials from 42 to 81 feet on down to the depth limit of information obtained (about 96 to 140 feet). This zone probably corresponds to relatively unweathered granitic and metamorphic bedrock materials.

2.2.5 Borrow Area D (Left Dam Abutment)

Seismic refraction lines S-21 and S-22 were recorded in proposed Borrow Area D at the left abutment of the proposed dam in the locations shown on Drawing 4-1. Lines S-21 and S-22 were crossed with Line S-21 running roughly east-west perpendicular to contour, and Line S-22 running roughly north-south along contour in the central, left abutment portion of the borrow area. Results are presented on Figure A-20.

Lines S-21 and S-22 are interpreted to be underlain by four (4) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1160 to 1500 ft/sec) materials to a thickness of 3 to 7 feet beneath the ground surface. This upper zone corresponds to residual soil or colluvial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (3030 to 3350 ft/sec) materials from 3 to 7 feet to 18 to 33 feet beneath the ground surface. This zone corresponds to alluvial fan materials, regolith and/or deeply weathered granitic bedrock. The underlying zone is characterized by medium velocity (5190 to 6850 ft/sec) materials from 18 to 33 feet to 49 to 87 feet beneath the ground surface. This zone probably corresponds to less weathered or more saturated granitic bedrock materials. The deepest zone encountered is characterized by high velocity (9850 to 14,450 ft/sec) materials from 49 to 87 feet on down to the depth limit of information obtained (about 96 feet). This zone probably corresponds to relatively unweathered granitic bedrock materials.

2.2.6 Borrow Area E

Seismic refraction lines S-19 and S-20 were recorded in proposed Borrow Area E in the locations shown on Drawing 4-1. Lines S-19 and S-20 were crossed with Line S-19 running roughly north-south along contour and Line S-20 running roughly east-west perpendicular to contour in the central, lower portion of the borrow area. Results are presented on Figure A-19.

Lines S-19 and S-20 are interpreted to be underlain by four (4) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1020 to 1120 ft/sec) materials to a thickness of 2 to 7 feet beneath the ground surface. This upper zone corresponds to residual soil or colluvial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (2090 to 2860 ft/sec) materials from 2 to 7 feet to 14 to 34 feet beneath the ground surface. This zone corresponds to deeply weathered granitic bedrock. The underlying zone is characterized by medium velocity (3940 to 6740 ft/sec) materials from 14 to 34 feet to 59 to 88 feet beneath the ground surface. This zone probably corresponds to less weathered or more saturated granitic bedrock materials. The deepest zone encountered is characterized by high velocity (14,850 to 17,700 ft/sec) materials from 59 to 88 feet on down to the depth limit of information obtained (about 96 feet). This zone probably corresponds to relatively unweathered granitic bedrock materials.

2.2.7 Fish Handling Facilities Area

Seismic refraction lines S-16 and S-17 were crossed in the location shown on Drawing 4-1. Results are presented on figures A-16 and A-17.

Lines S-16 and S-17 are interpreted to be underlain by three (3) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1900 to 2140 ft/sec) materials to a thickness of 4 to 13 feet beneath the ground surface. This upper zone corresponds to surficial soil materials that are relatively dry. The underlying zone consists of medium to high velocity (6320 to 8810 ft/sec) materials from 4 to 13 feet to 22 to 42 feet beneath the ground surface. This zone corresponds to weathered granitic bedrock. The deepest zone encountered is characterized by high velocity (12,920 to 15,550 ft/sec) materials from 22 to 42 feet on down to the depth limit of information obtained (about 83 to 96 feet). This zone probably corresponds to relatively unweathered granitic bedrock materials.

2.2.8 Cachagua Fault

Seismic refraction Line S-11 was recorded in a north-south direction in a suspected area traversed by the Cachagua fault in the location shown on Drawing 4-1. Results are presented on Figure A-11.

Line S-11 is interpreted to be underlain by three (3) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1770 to 2000 ft/sec) materials to a thickness of 7 to 20 feet beneath the ground surface. This upper zone corresponds to surficial soil materials that are relatively dry. The underlying zone consists of low to medium velocity (3380 to 3670 ft/sec) materials from 7 to 20 feet to 50 to 75 feet beneath the ground surface. This zone corresponds to alluvial fan materials, terrace deposits and/or deeply weathered granitic or sandstone bedrock. The deepest zone encountered is characterized by medium to high velocity (7250 to 7330 ft/sec) materials from 50 to 75 feet on down to the depth limit of information obtained (about 140 feet). This zone probably corresponds to less weathered granitic or sandstone bedrock materials.

A step-type anomaly was noted beneath the northern end of the line. This anomaly could be indicative of faulting or could represent effects from a canyon fill prism located in this area.

2.3 Rippability

Rippability is strongly influenced by the physical condition of the rock masses to be ripped. Structural features in rock such as bedding planes, cleavage planes, joints, fractures and shear zones influence rippability. Rock masses tend to be rippable if they have closely-spaced fractures, joints, or other planes of weakness. Massive rock materials lacking discontinuities, even where partially weathered, may exhibit marginal rippability, requiring blasting for removal.

Seismic compression wave velocities can be related to both rock hardness and fracture density. Seismic refraction velocities have been related to rippability by Caterpillar Inc. (1990) as displayed on graphs relating seismic velocity for various rock types to rippability with various types of equipment (combinations of dozers and rippers). Two examples of these graphs are presented on Figure A-21 for both D8L and D9N dozer/ripper combinations. In general, rocks such as the granitic and metamorphic rocks present at this site become marginally rippable above velocities of 9500 to 9600 feet/second using either a D8 Dozer with a Multi or Single Shank No. 8 Ripper, or a D9 Dozer with a Multi or Single Shank No. 9 Ripper.

The charts of ripper performance should be considered as being only one indicator of rippability. The following precautions should be observed when evaluating the rippability of a given rock formation:

- Ripper tooth penetration is usually the key to successful ripping, regardless of seismic velocity. This is particularly true in finer-grained homogeneous materials and in tightly cemented formations.
- Although low seismic velocities in sedimentary rocks indicate probable rippability, if the fractures and bedding joints do not allow tooth penetration, the material may not be ripped effectively.
- Pre-blasting or "popping" may be required to induce sufficient fracturing to allow tooth penetration, but the economics of this should be checked carefully in the higher grades of sandstones, limestones and granites.
- Impact ripping may be used in marginal situations because significant boosts in production may be possible relative to conventional ripping by using an impact ripper mounted on a D10N or D11N dozer.
- Ripping success may well depend on the operator finding the proper combination of number of shanks used, length and depth of shank, tooth angle and direction and throttle position.

Based on the seismic velocities measured at this site, it appears that the surficial materials (with velocities of 1020 to 2400 ft/sec) and deeply weathered bedrock or terrace/alluvial fan materials (with velocities of 2090 to 6850 ft/sec) should be easily rippable, but the weathered granitic and metamorphic bedrock materials (with velocities of 7250 to 8810 ft/sec) will likely be difficult, but still rippable with either the D8 dozer with No. 8 ripper combination, or a D9 dozer with No. 9 ripper. The less weathered granitic and metamorphic bedrock materials (with velocities in excess of 10,000 ft/sec) will likely not be rippable and will require drilling and blasting for excavation.

3.0 INVESTIGATION LIMITATIONS

The subsurface profiles presented in this report represent the most reasonable interpretation of geophysical survey data based on our limited knowledge of the existing geologic conditions at the site. The results are presented for design information only and are not intended to serve as information for determining construction procedures. Interpretations were made in accordance with generally accepted geophysical methods and practices. This warranty is in lieu of all other warranties, express or implied.

The quality of seismic refraction data for this survey was good, but in some cases affected by background noise, irregular terrain, wind, and lateral inhomogeneity. These factors produced noise signals and/or scatter in the recorded data, limiting the accuracy of first break compression wave picks and interpretation. The seismic refraction method used has some inherent limitations such as the possibility for undetectable hidden layers, blind zones, and velocity inversions. The maximum depth of reliable seismic information obtained during this survey can be assumed to be approximately one-third of the length of the individual lines, with information at a maximum depth underlying the middle one-third of the lines. For example, a seismic refraction line 300 feet in length will typically yield reliable data on subsurface materials to a depth of about 100 feet beneath the middle 100 feet of the line.

4.0 SUMMARY OF CONCLUSIONS

The site area surveyed was generally underlain by three to four velocity zones to the depth limit surveyed (about 83 to 196 feet beneath the ground surface). These zones were generally characterized by lower velocities where surficial soils were present, low to medium velocities where alluvial fan materials or terrace deposits were present, medium to high velocities where weathered bedrock materials were present and high velocities where less weathered or relatively unweathered bedrock materials were present. Depending on planned depths of excavation, bedrock materials could present difficult excavation characteristics for conventional excavation equipment should proposed excavation depths intersect the less weathered or relatively unweathered bedrock.

5.0 REFERENCES

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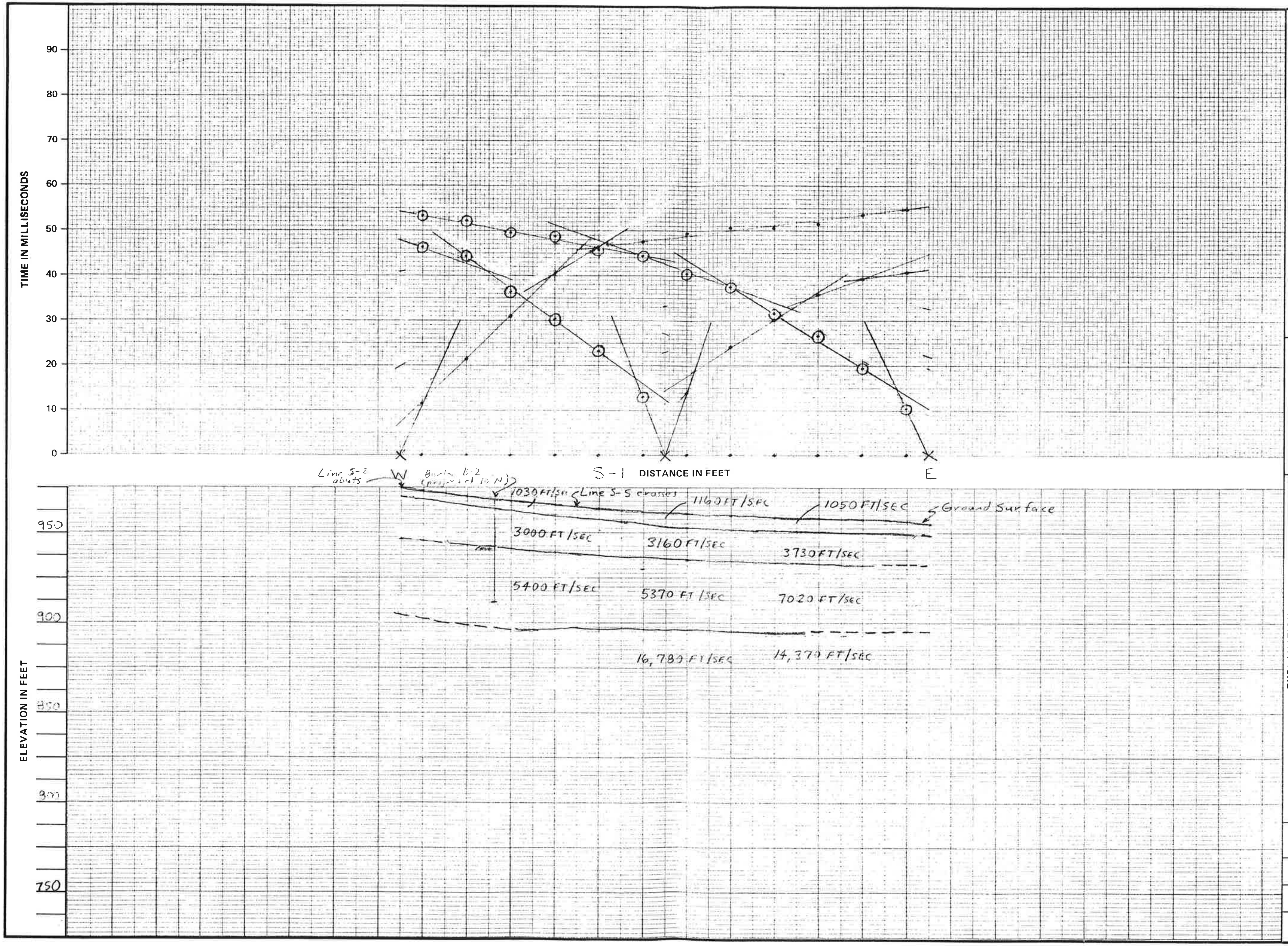
Caterpillar, Inc., 1990, *Caterpillar™ Performance Handbook*, a CAT™ Publication by Caterpillar, Inc., Peoria, Illinois, U.S.A.

Mooney, H. M., 1977, *Handbook of Engineering Geophysics*, Bison Instruments Inc., Minneapolis, Minnesota, U.S.A..

Shires, P. O., 1983, *A Seismic Refraction Interpretation Program for Multi-Dipping Layers*, unpublished computer program.



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SEISMIC REFRACTION SURVEY
DATA AND INTERPRETED SUBSURFACE VELOCITY PROFILES

NEW LOS PADRES WATER SUPPLY PROJECT
MONTEREY COUNTY, CALIFORNIA

PATRICK O. SHIRES
No. 879

EXPLANATION

1. TIME-DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.

   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.

   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

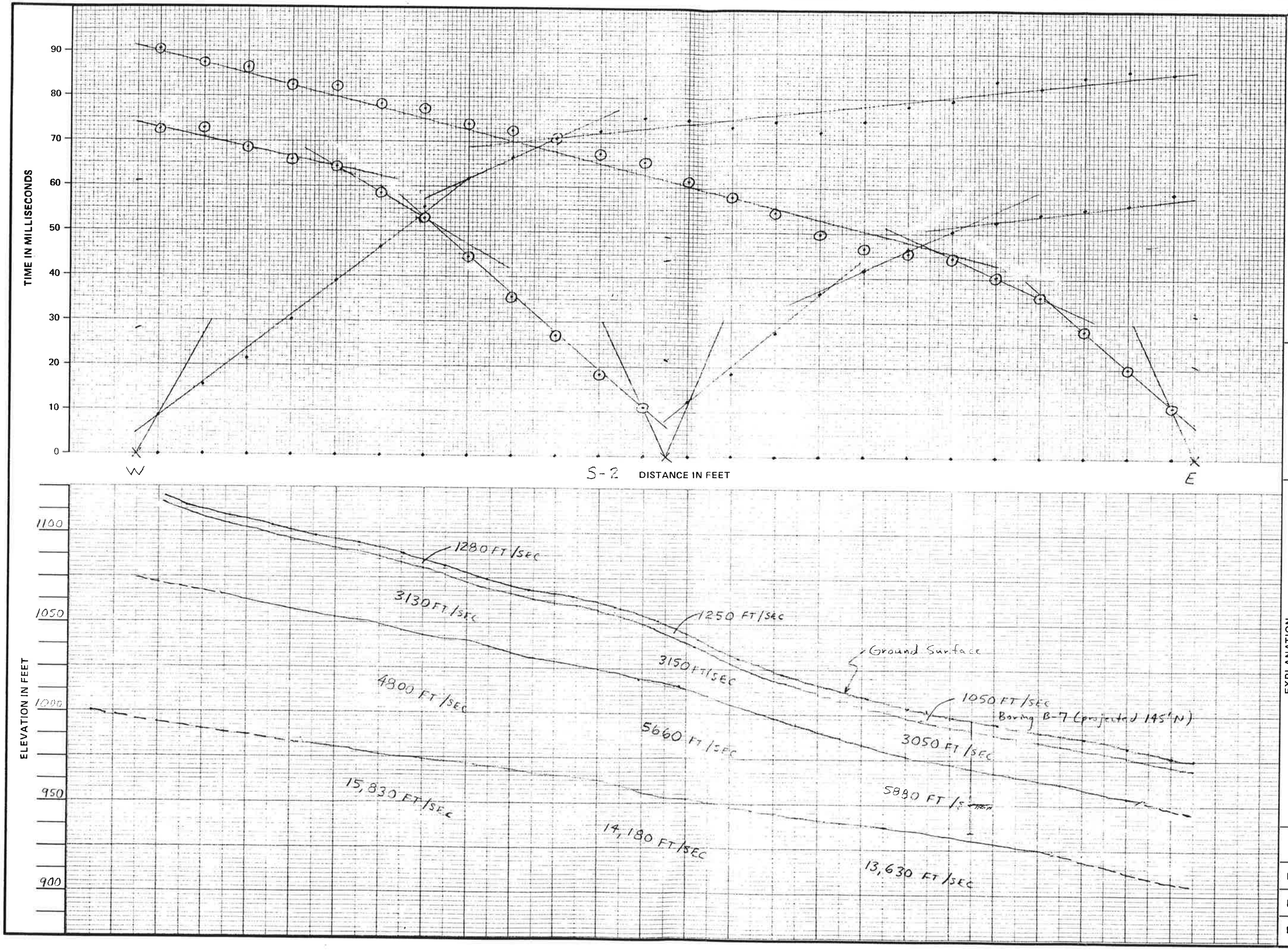
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-1

DATE: 9/28/94

PROJECT NO.: G3214C

FIGURE NO.: A-1



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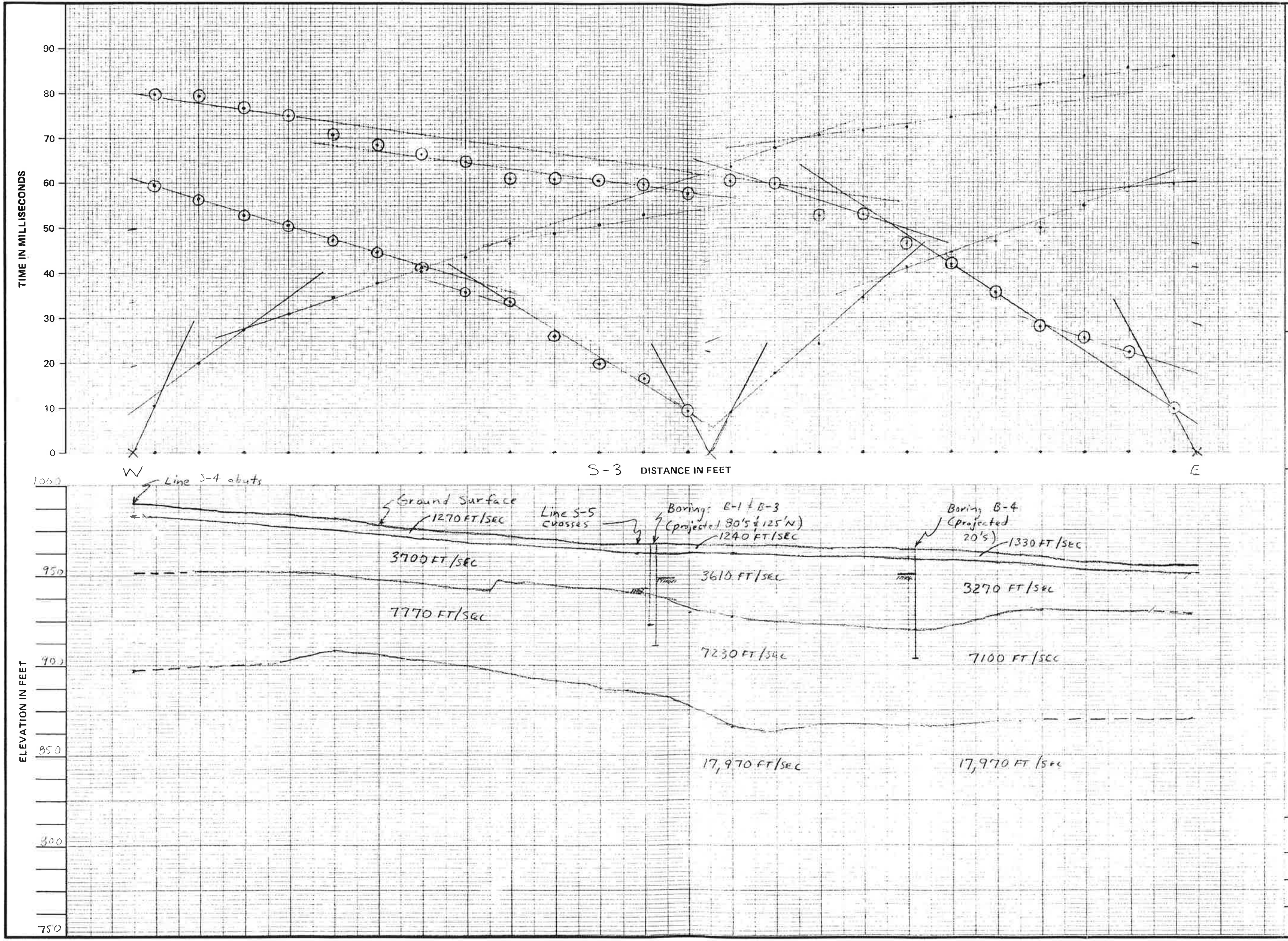
SEISMIC LINE NO.(S):
S-2

DATE: 9/28/94

PROJECT NO.: G3214C

FIGURE NO.: A-2



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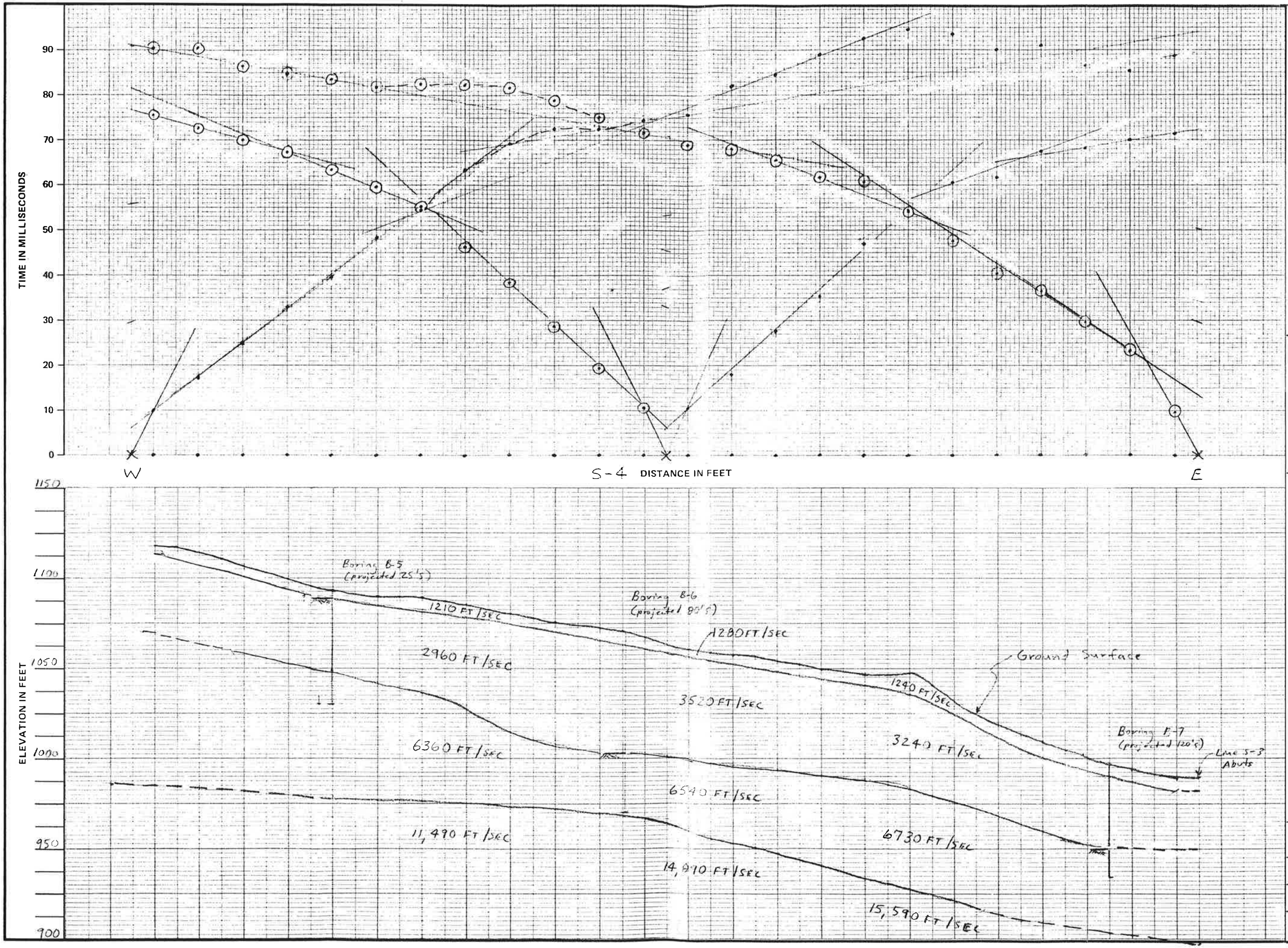
SEISMIC LINE NO.(S):
S-3

DATE: 9/28/94

PROJECT NO.: G3214C

FIGURE NO.: A-3



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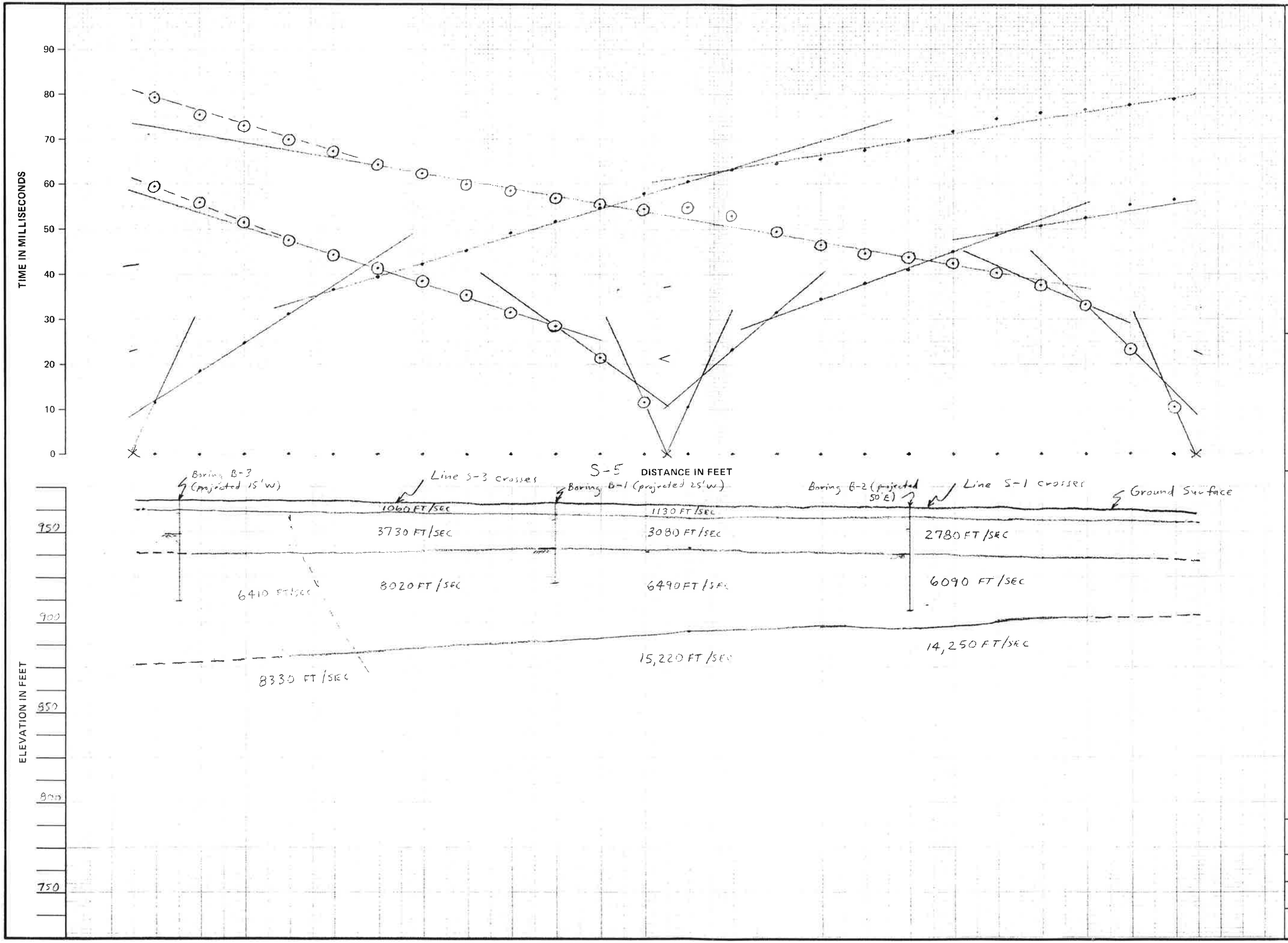
SEISMIC LINE NO.(S): S-4

DATE: 9/28/94

PROJECT NO.: G3214C

FIGURE NO.: A-4



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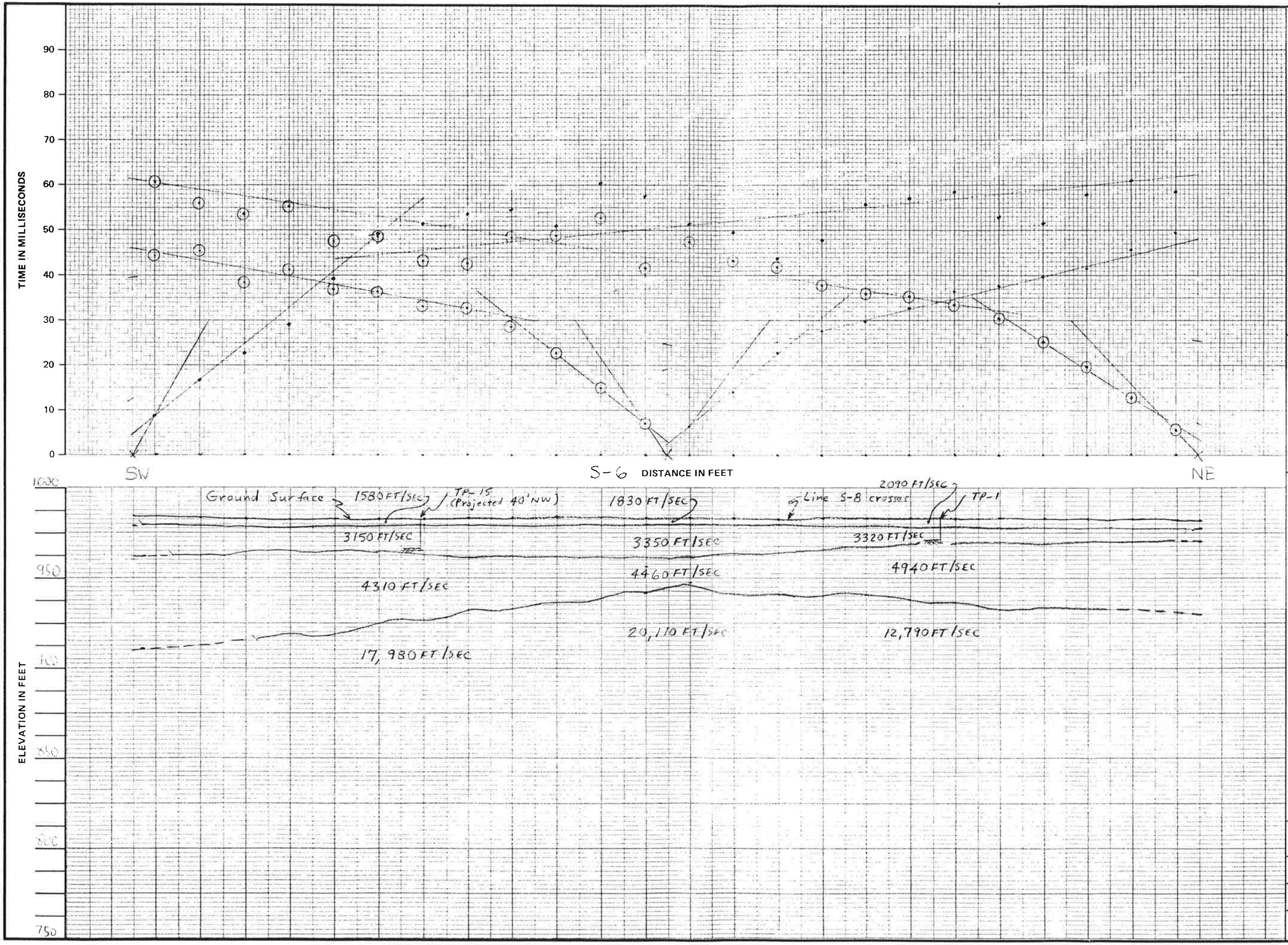
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-5

DATE: 9/28/94

PROJECT NO.: G3214C

FIGURE NO.: A-5



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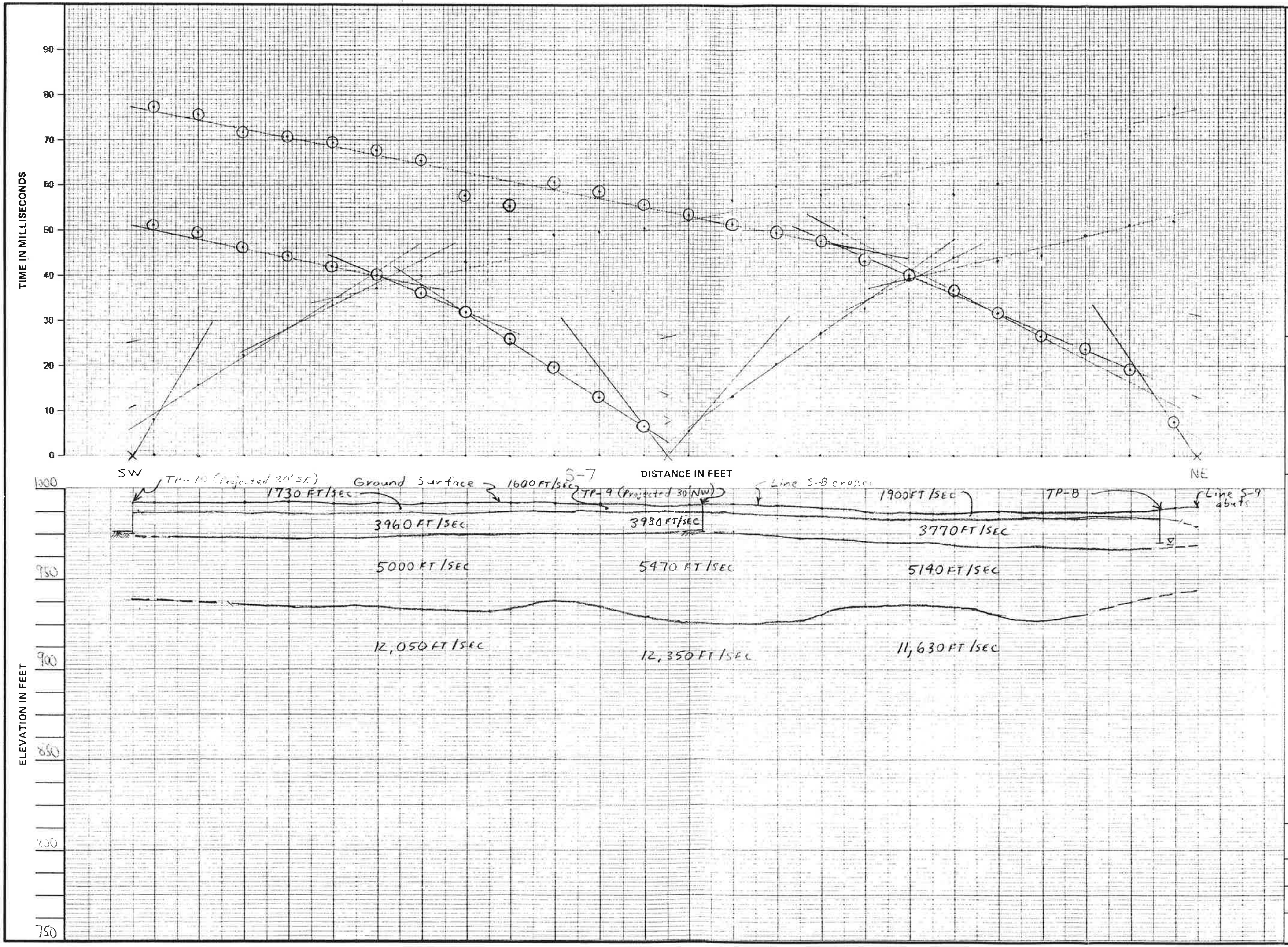
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S):
S-6

DATE: 9/29/94

PROJECT NO.: G3214C

FIGURE NO.: A-6



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   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

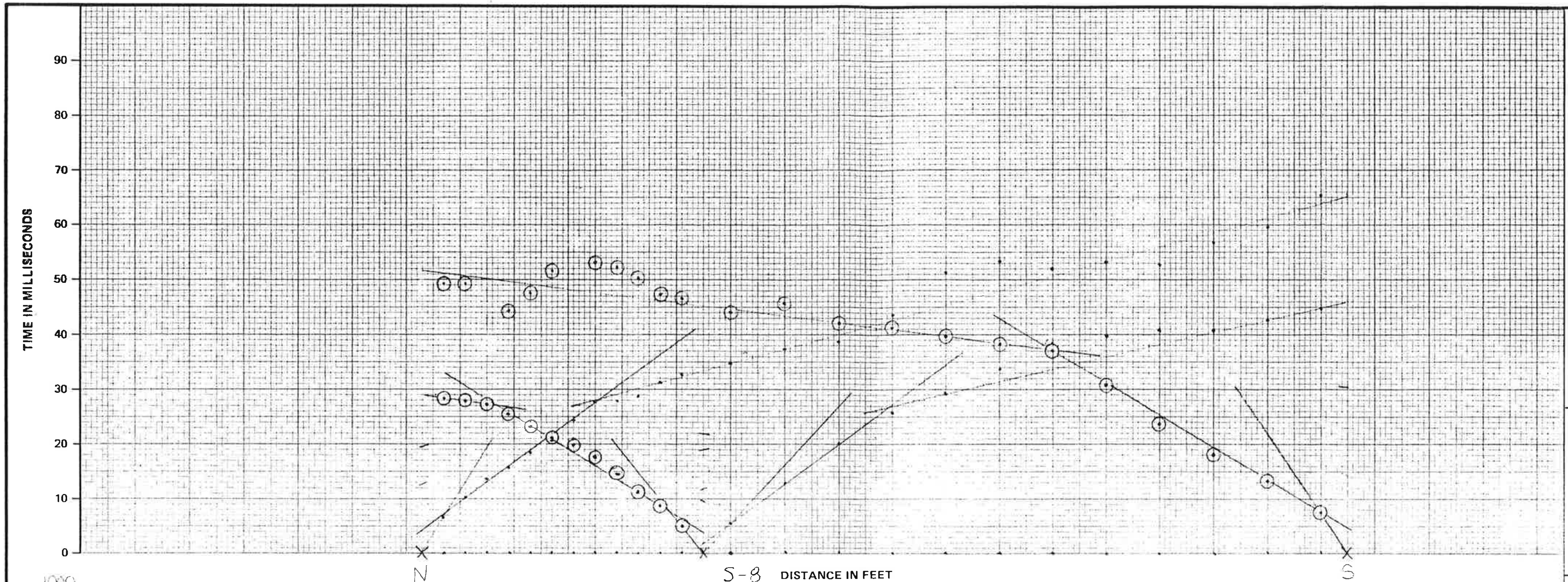
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

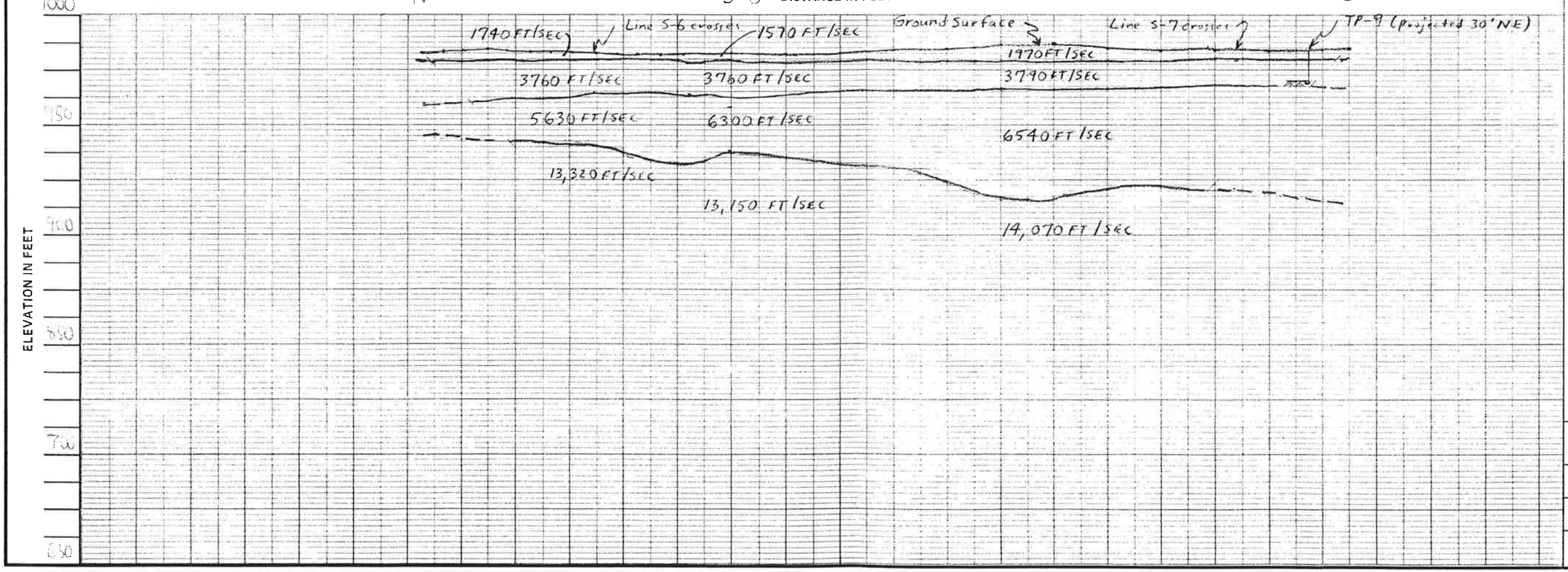
SEISMIC LINE NO.(S):
S-7

DATE: 9/29/94

PROJECT NO.: G3214C

FIGURE NO.: A-7





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   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

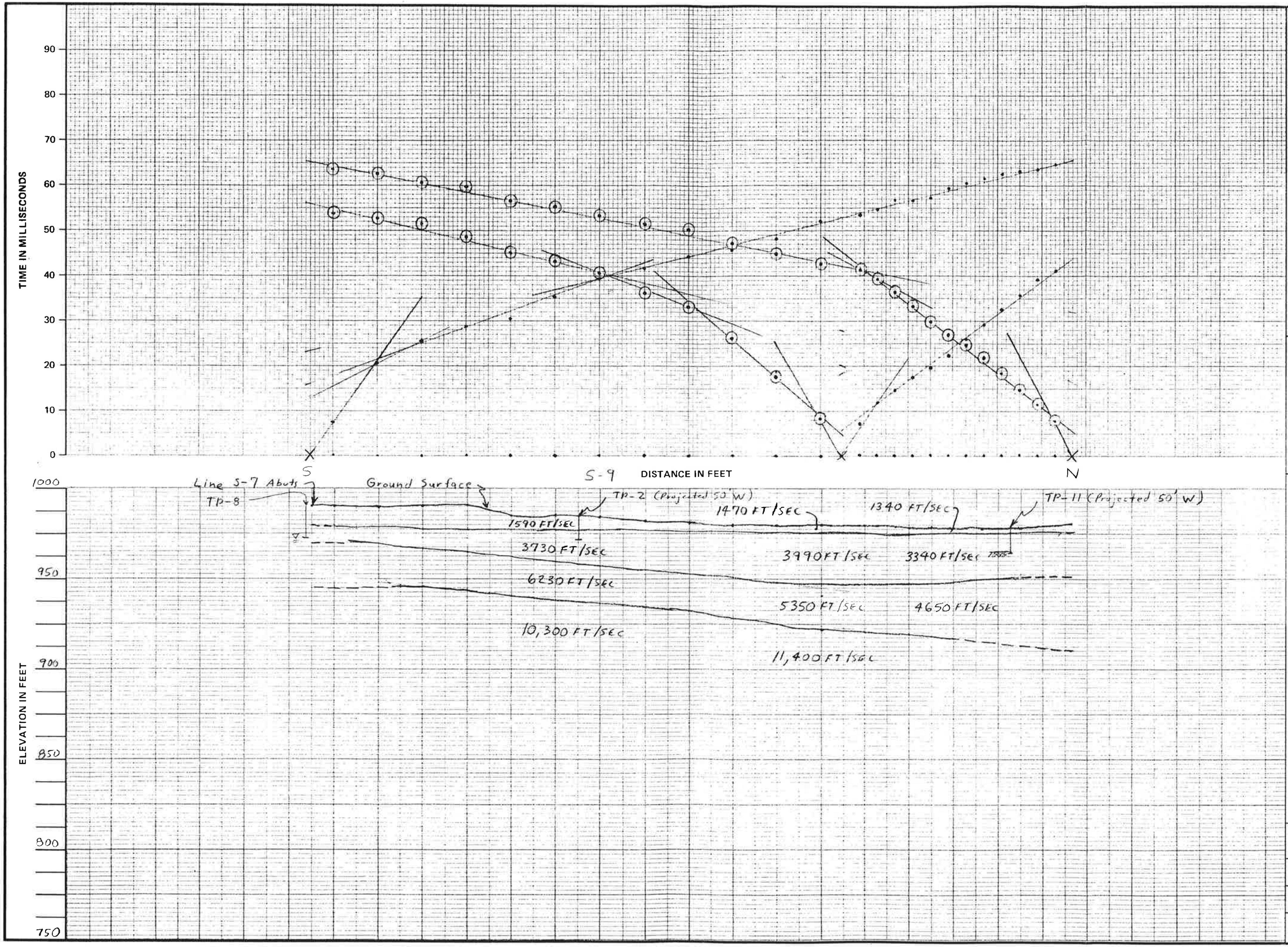
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-8

DATE: 9/29/94

PROJECT NO.: G3214C

FIGURE NO.: A-8



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SEISMIC REFRACTION SURVEY
DATA AND INTERPRETED SUBSURFACE VELOCITY PROFILES

NEW LOS PADRES WATER SUPPLY PROJECT
MONTEREY COUNTY, CALIFORNIA

REGISTERED GEOPHYSICIST
PATRICK O. SHIRES
No. 879
STATE OF CALIFORNIA

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   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET

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   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET

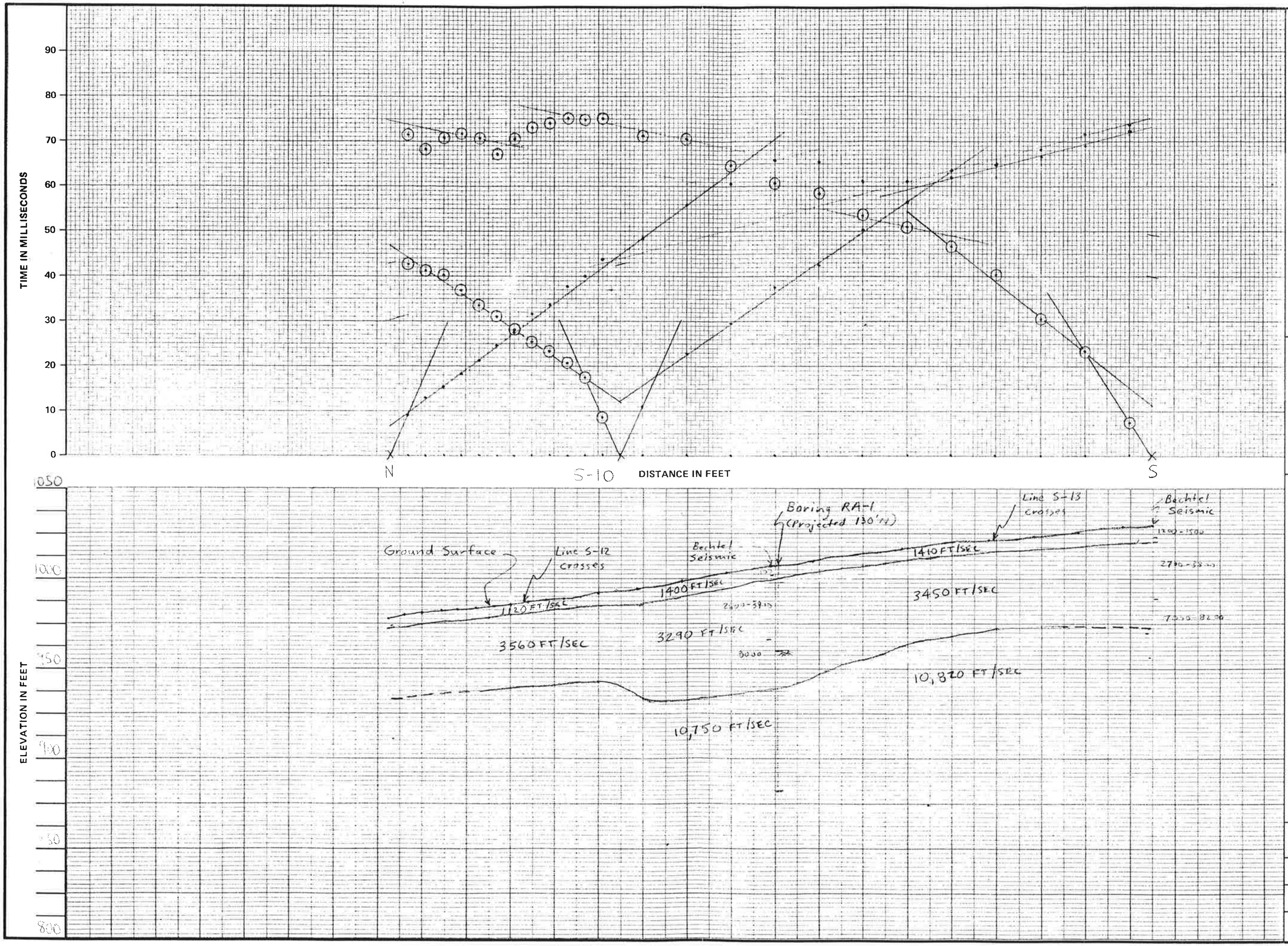
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-9

DATE: 9/29/94

PROJECT NO.: G3214C

FIGURE NO.: A-9



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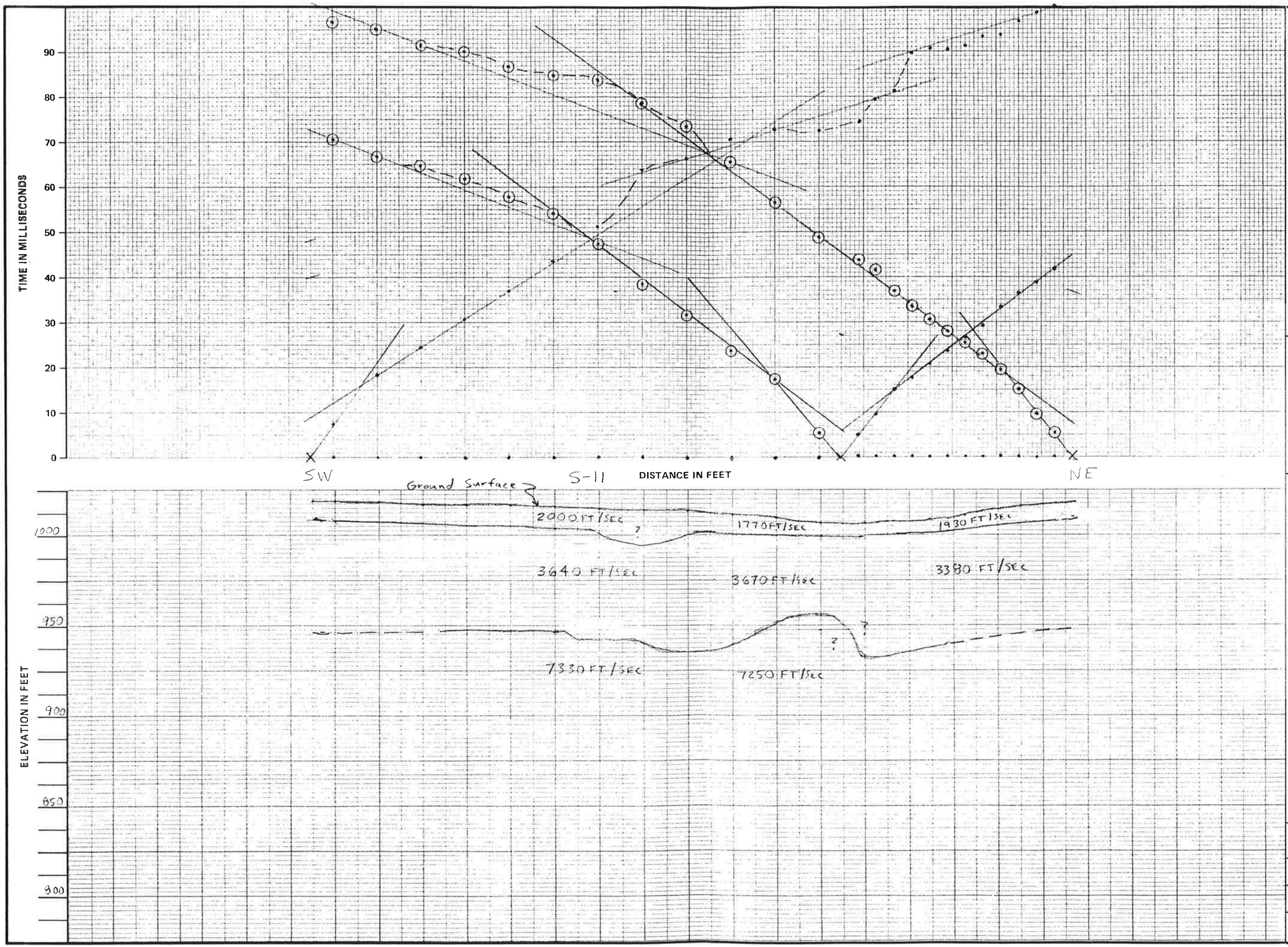
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S):
S-10

DATE: 9/29/94

PROJECT NO.: G3214C

FIGURE NO.: A-10



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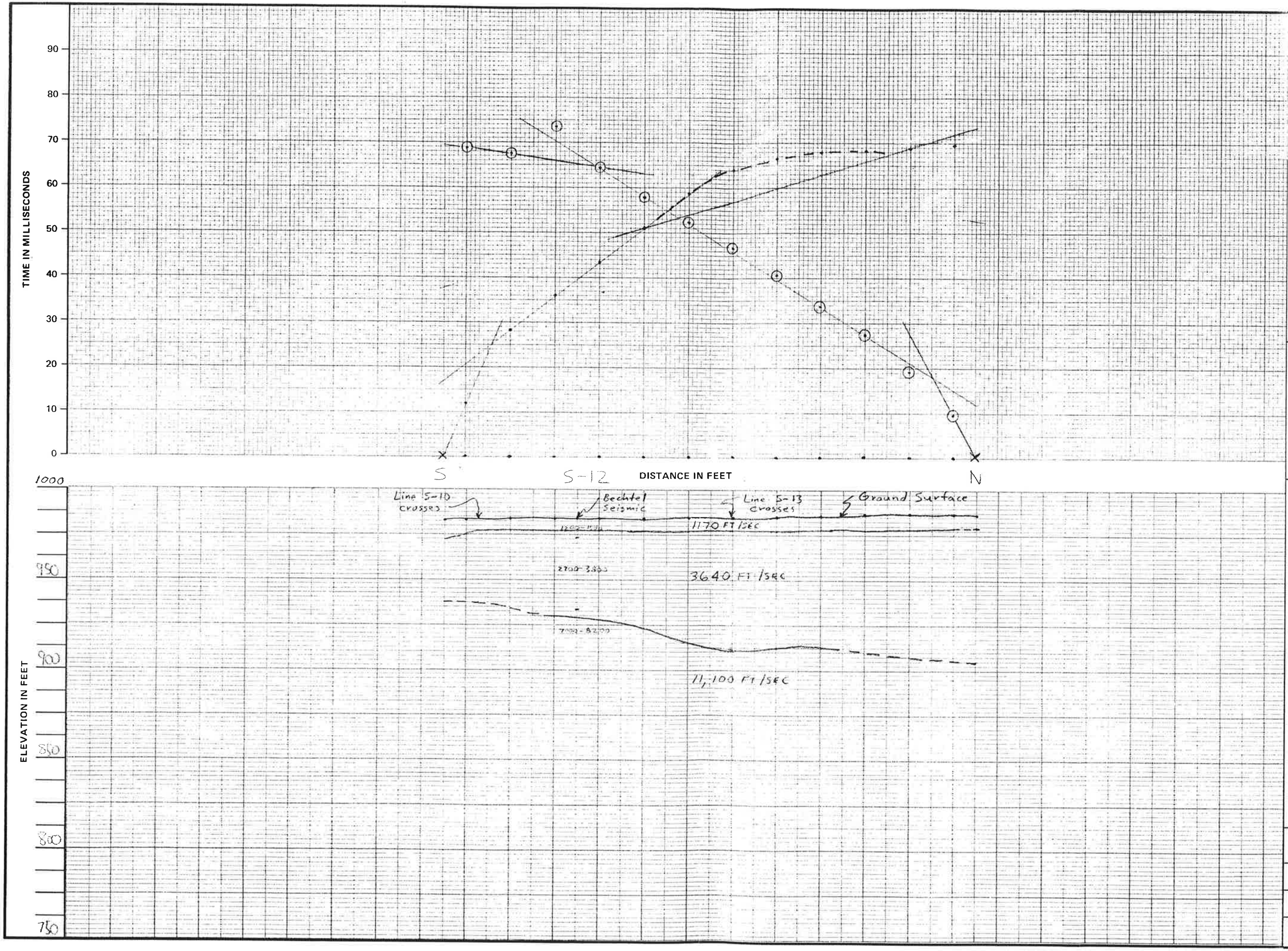
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S):
S-11

DATE: 10/20/94

PROJECT NO.: G3214C

FIGURE NO.: A-11



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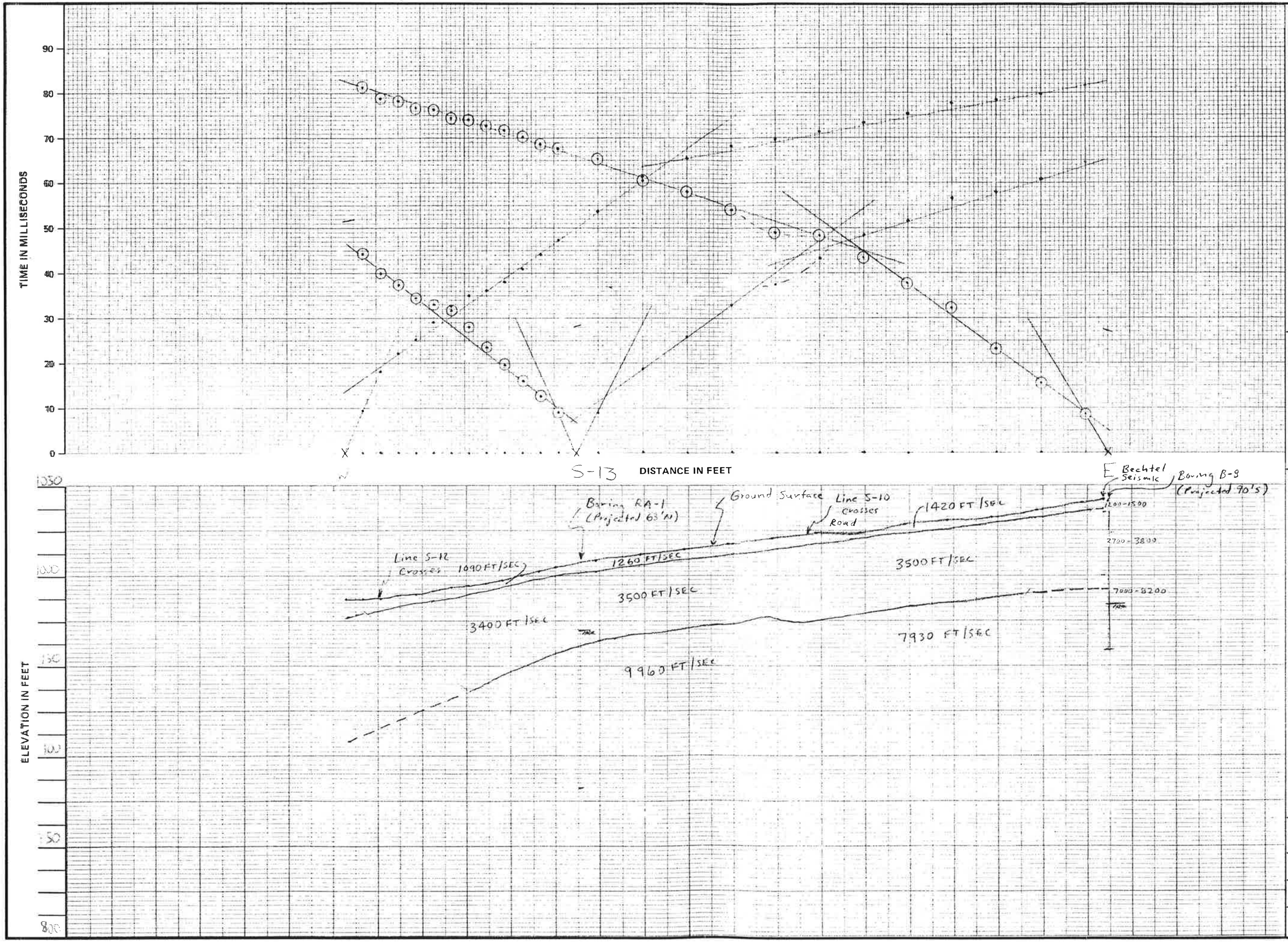
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S):
S-12

DATE: 10/21/94

PROJECT NO.: G3214C

FIGURE NO.: A-12



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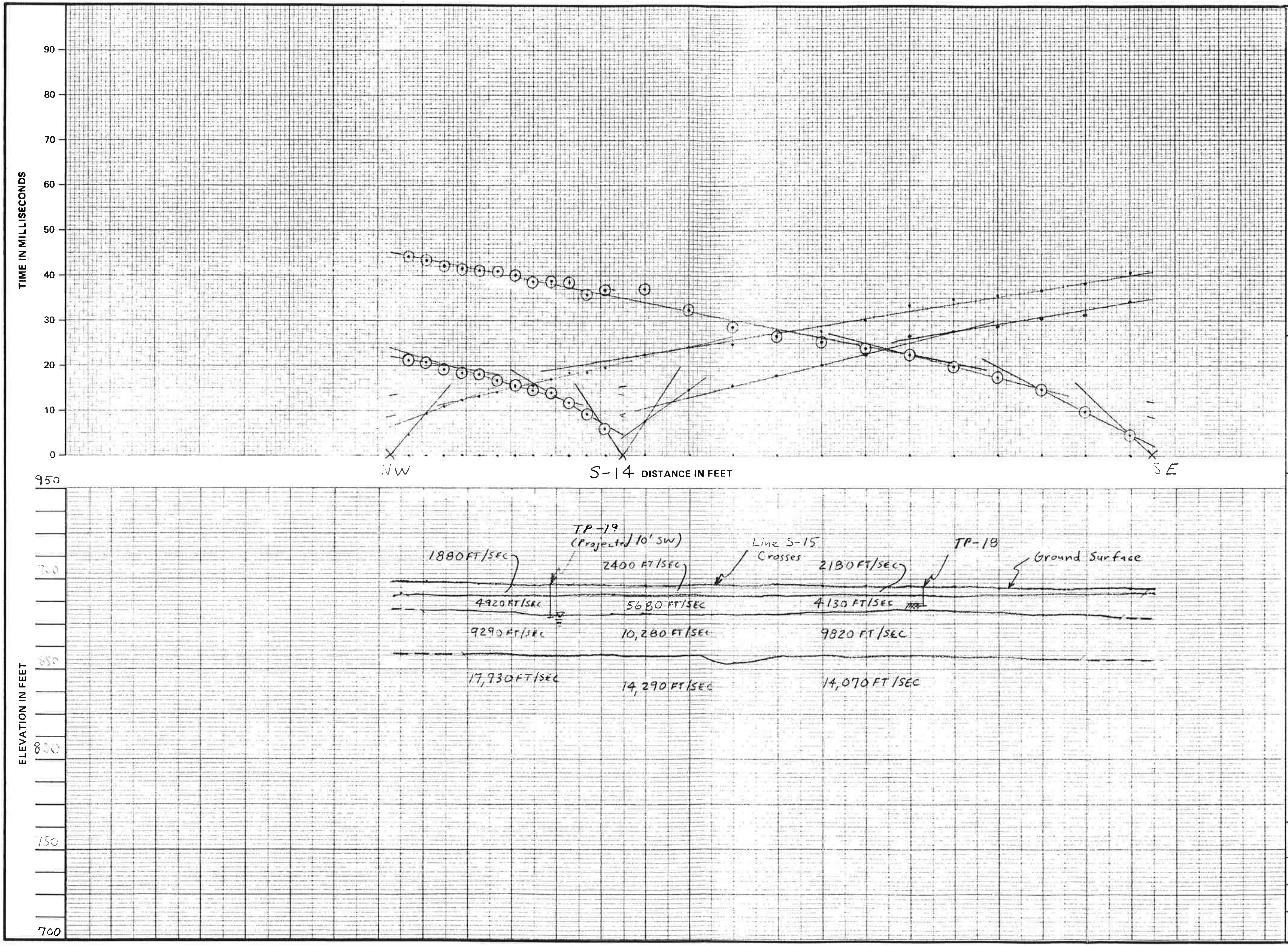
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-13

DATE: 10/24/94

PROJECT NO.: G3214C

FIGURE NO.: A-13



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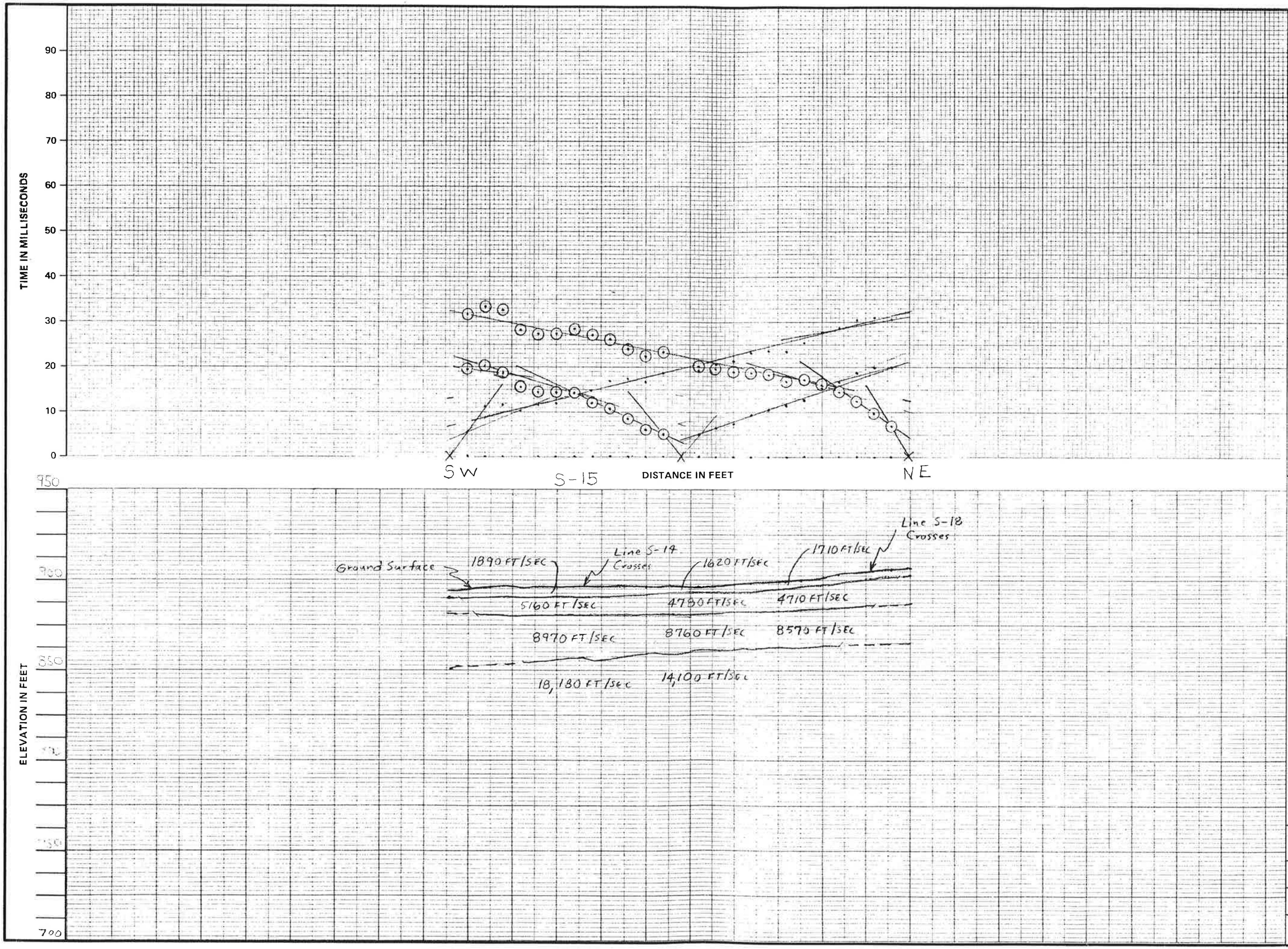
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S):
S-14

DATE: 10/21/94

PROJECT NO.: G3214C

FIGURE NO.: A-14



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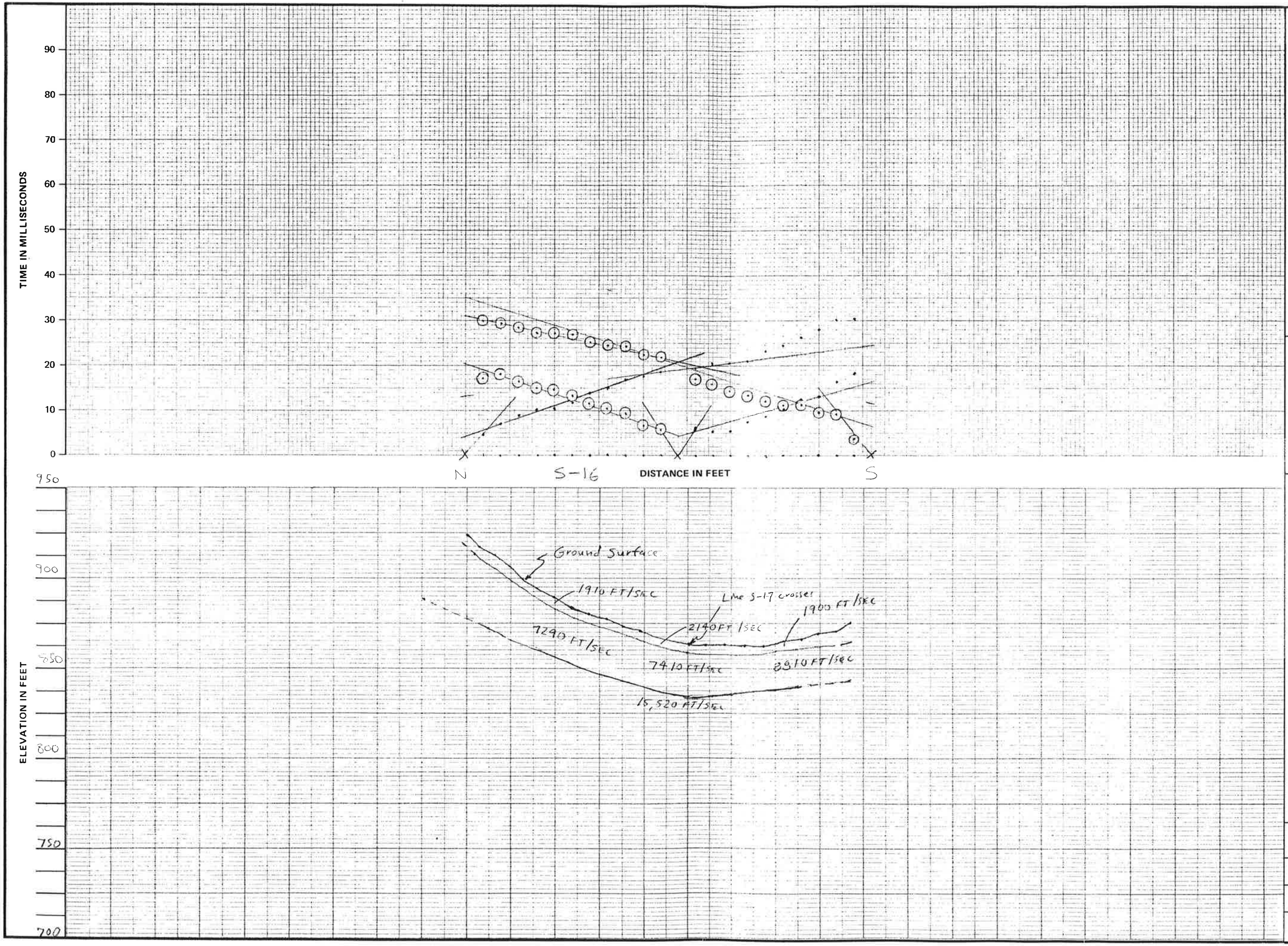
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-15

DATE: 10/21/94

PROJECT NO.: G3214C

FIGURE NO.: A-15



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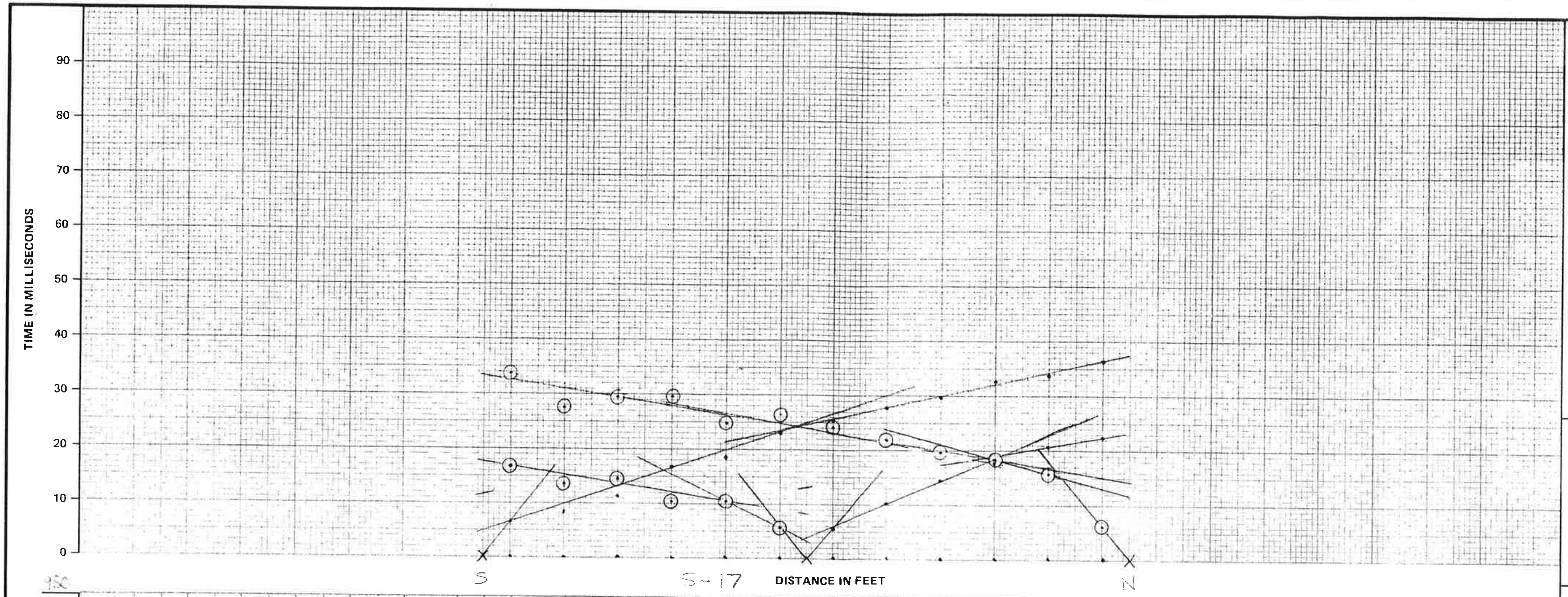
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

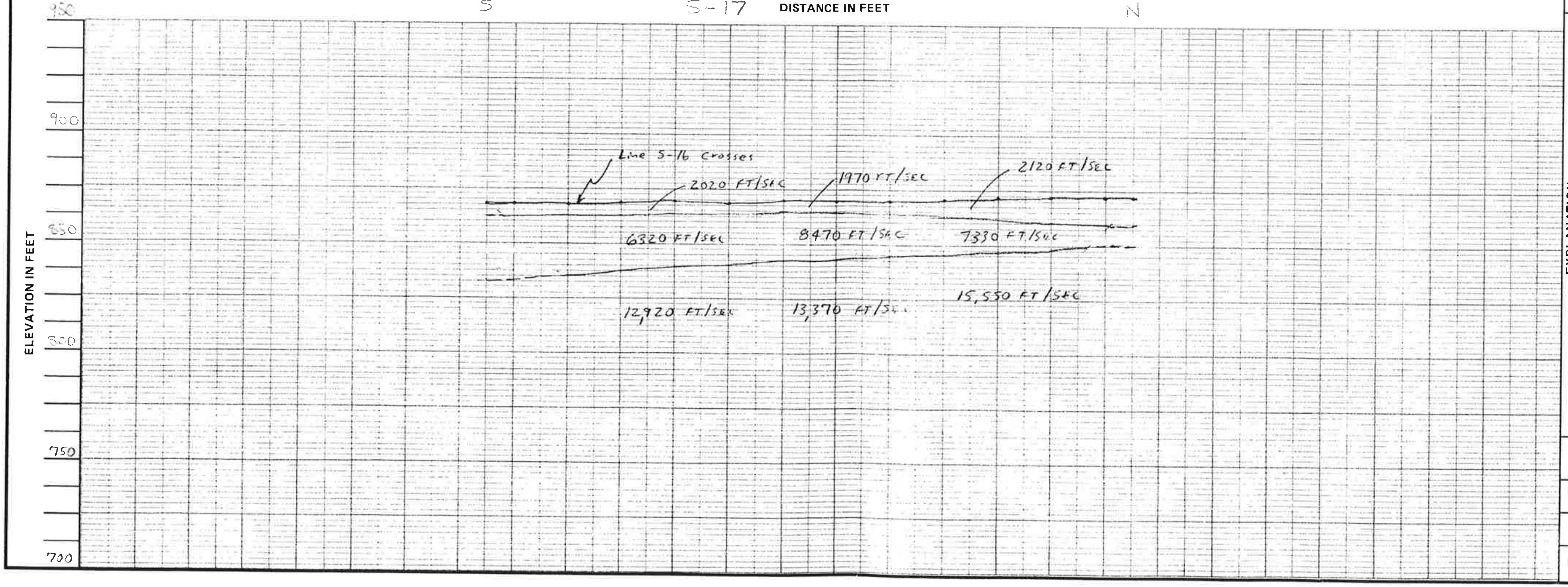
SEISMIC LINE NO.(S): S-16

DATE: 10/21/94

PROJECT NO.: G3214C

FIGURE NO.: A-16





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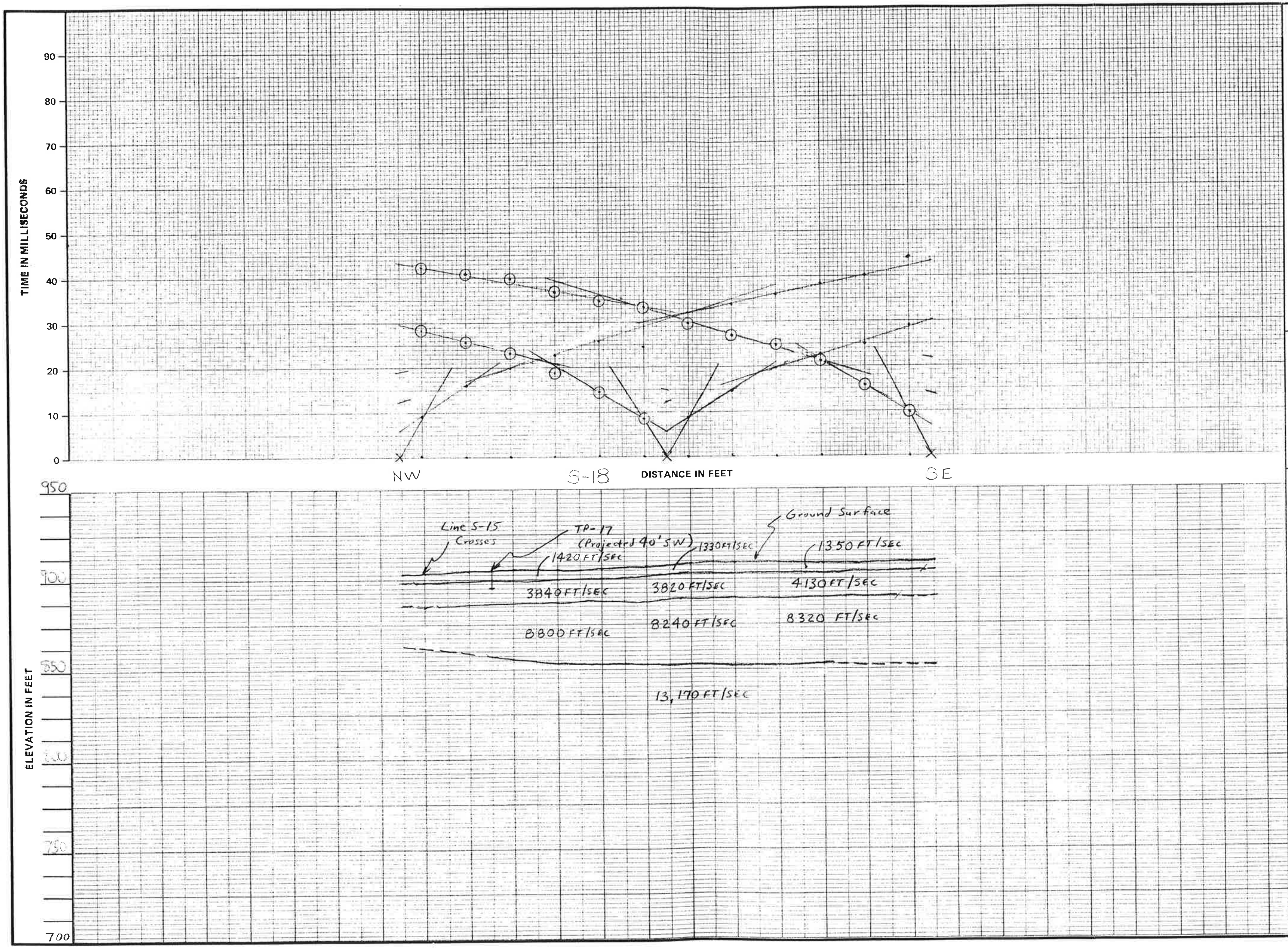
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-17

DATE: 10/21/94

PROJECT NO.: G3214C

FIGURE NO.: A-17



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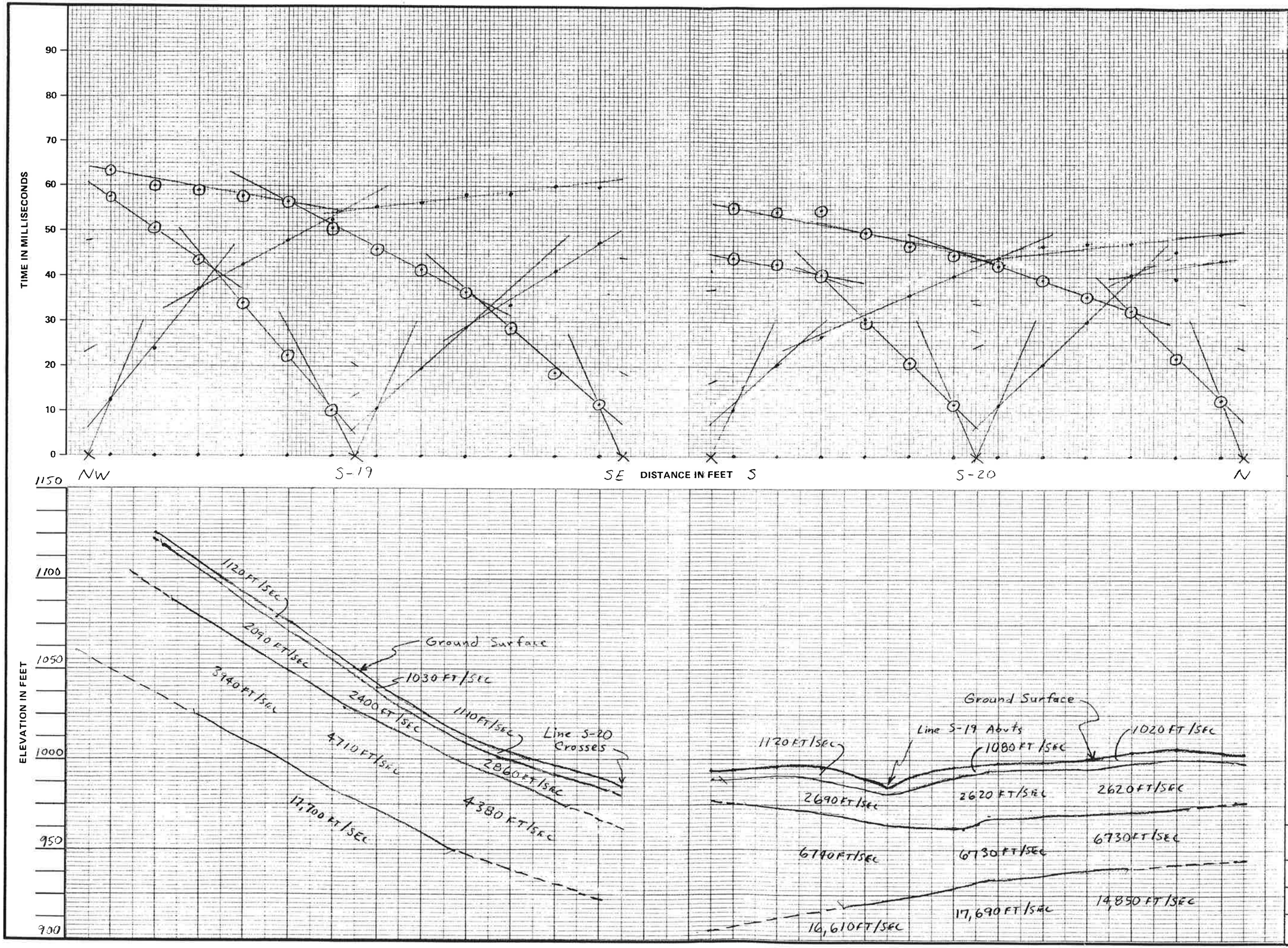
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-18

DATE: 10/21/94

PROJECT NO.: G3214C

FIGURE NO.: A-18



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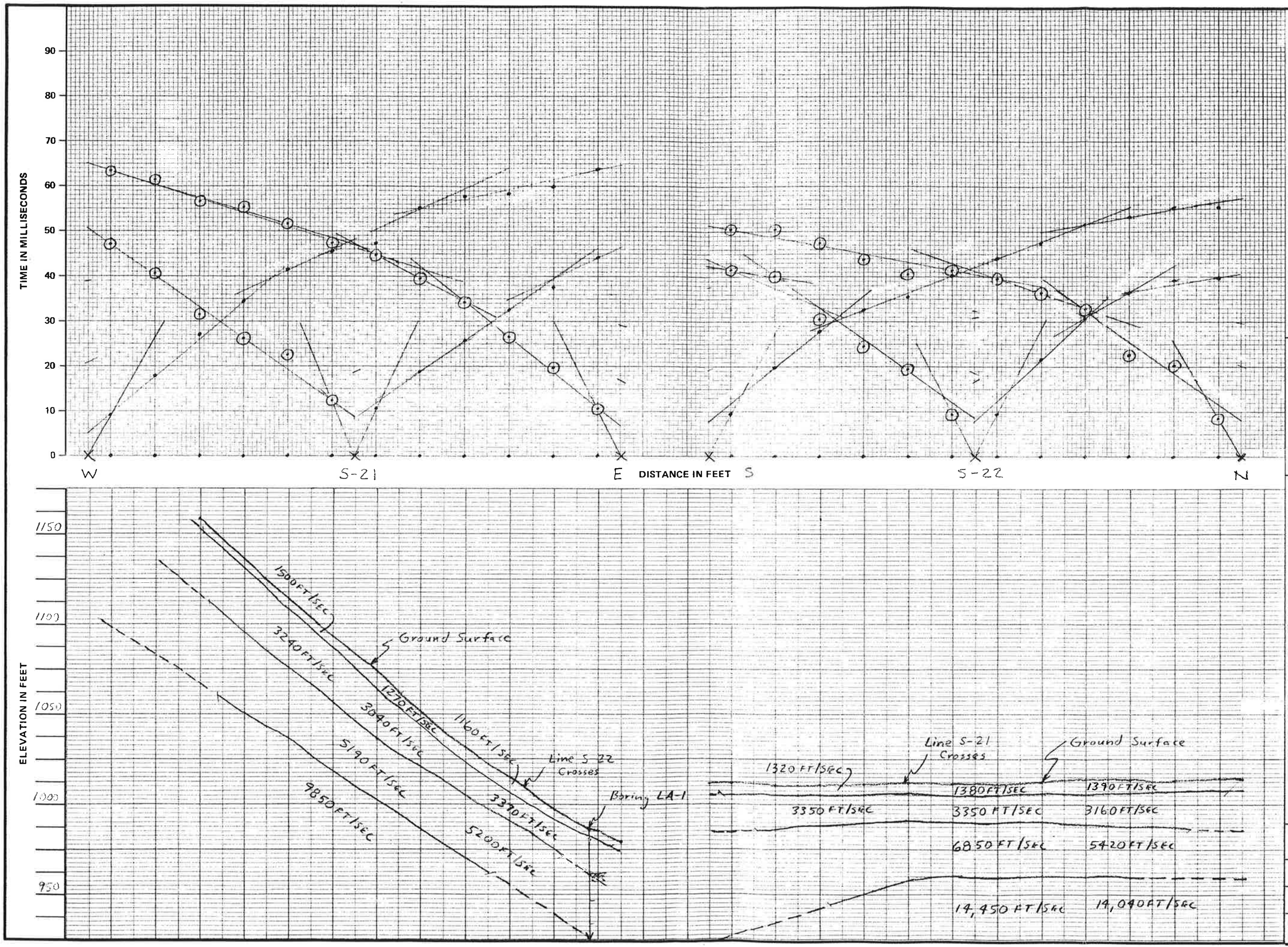
3. LOCATION OF SEISMIC LINES SHOWN ON DRWG. NO. 4-1

SEISMIC LINE NO.(S): S-19 & S-20

DATE: 11/18/94

PROJECT NO.: G3214C

FIGURE NO.: A-19



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SEISMIC LINE NO.(S):
S-21 S-22

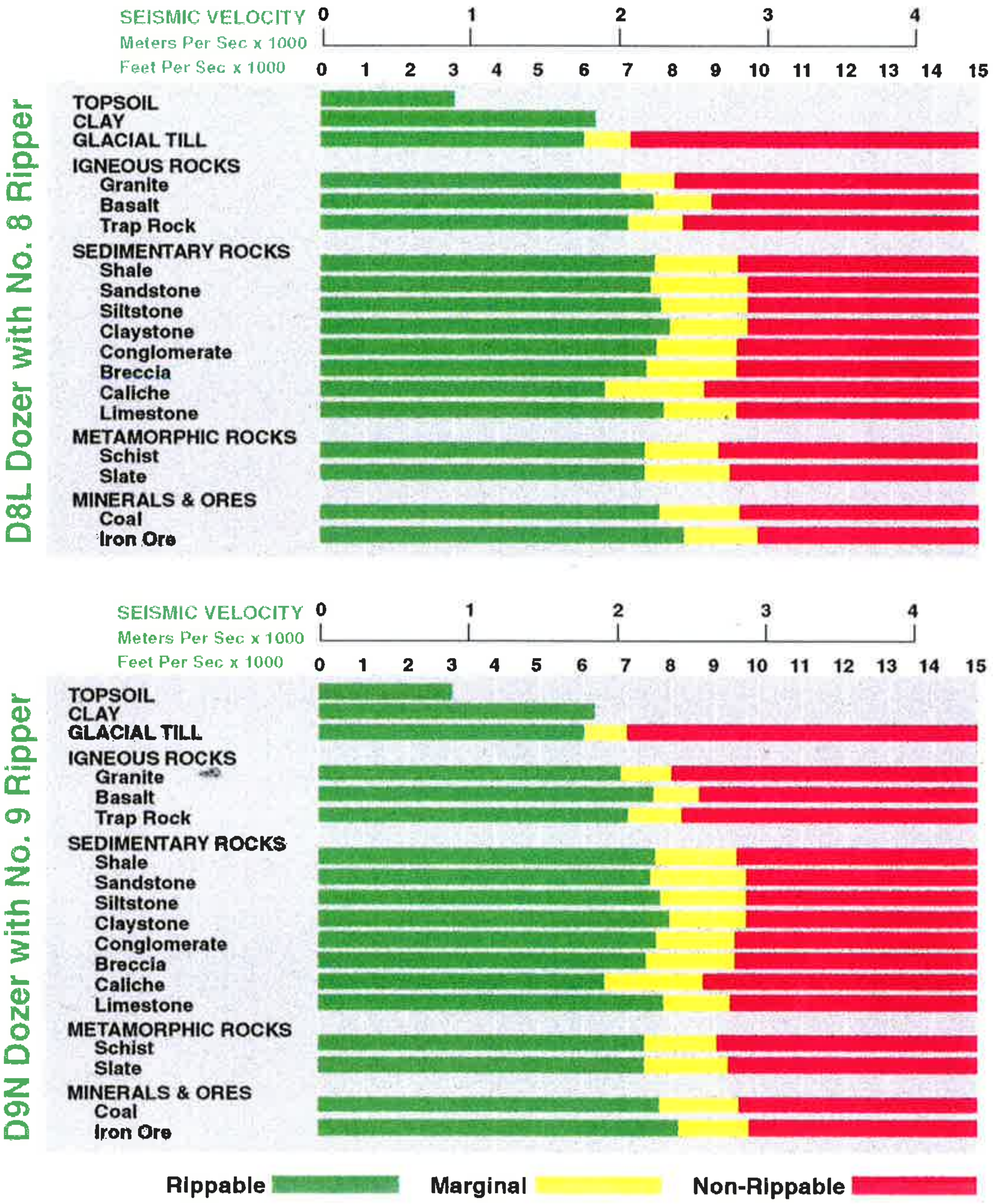
DATE: 11/18/94

PROJECT NO.: G3214C

FIGURE NO.: A-20

RIPPER PERFORMANCE

Multi or Single Shank Rippers
Estimated by Seismic Wave Velocities



Source: Caterpillar^{TM}, 1990

PREPARED BY Pos

REVIEWED BY JFC

DATE 3/8/95

RIPPABILITY CHARTS (CATERPILLAR, INC., 1990).

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS
In Association With
MORRISON KNUDSEN CORPORATION
William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

DRAWING NO.
A-21

Appendix B

APPENDIX B

BORING, TRENCH, AND WELL CONSTRUCTION LOGS

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS

Descriptive Terms of Rock

DEGREE OF WEATHERING

DESCRIPTIVE TERM	DISCOLORATION EXTENT	FRACTURE CONDITION	SURFACE CHARACTERISTICS	ORIGINAL TEXTURE	GRAIN BOUNDARY CONDITION
Unweathered	None (F) Fresh	Closed or Discolored	Unchanged	Preserved	Very Tight
Slightly Weathered	Less than 20% of fracture spacing on both sides of fracture. (SW)	Discolored. May contain thin filling	Partial discoloration	Mainly Preserved	Tight
Moderately Weathered	Greater than 20% of fracture spacing on both sides of fracture. (MW)	Discolored. May contain thick filling	Partial to complete discoloration, not friable except poorly cemented rocks.	Slightly Preserved	Partial Opening
Highly Weathered	50% of rock mass (HW)	Faint fractures	Friable and possibly pitted	Faint	Partial Separation
Completely Weathered	100% of rock mass (D) Decomposed	Obscure fractures	Resembles a soil	Obscure	Complete Separation

HARDNESS

DEGREE OF HARDNESS	FIELD TEST
Hard	Difficult to scratch with knife point. Cannot break hand held specimen.
Moderately Hard	Cannot be scraped or peeled with a knife, but can be scratched with knife point. Hand held specimen breaks with firm blows of the pick.
Soft	Can just be scraped with a knife. Indentations of 2 to 4 mm with firm blows of the pick point.
Very Soft	Can be peeled with a knife. Material crumbles under firm blows with the sharp end of a geologic pick.
Extremely Soft	Fist size piece can be crushed with hand.

DISCONTINUITY SPACING

DESCRIPTION FOR FORMATIONAL FEATURES: BEDDING, FOLIATION, OR FLOW BANDING	SPACING		DESCRIPTION FOR DEFECTS: JOINTS, FAULTS OR OTHER FRACTURES
	(METRIC)	(ENGLISH)	
Very thickly spaced	More than 2 meters	More than 6 feet	Very widely spaced
Thickly spaced	60 cm-2 meters	2-6 feet	Widely spaced
Medium spaced	200 cm-60 cm	8-24 inches	Medium spaced
Thinly spaced	60-200 mm	2½-8 inches	Closely spaced
Very thinly spaced	20-60 mm	¾-2½ inches	Very closely spaced
Extremely close spaced	6-20 mm	¼-¾ inch	Extremely close spaced
Laminated	Less than 6 mm	Less than ¼ inch	Laminated

NATURE OF FRACTURES

SEPARATION OF FRACTURE WALLS

Description	Separation of Walls	
	(Metric)	(English)
Closed	0	0
Very Narrow	0-0.1	Hairline
Narrow	0.1-1	1/16 inches
Wide	1-5.0	1/16-¼ inches
Very Wide	5-25 +	¼-1 + inches

FRACTURE FILLING

Description	Definition
Clean	No fracture filling material
Stained	Discoloration of rock only. No recognizable filling material
Filled	Fracture filled with recognizable filling material
Cemented	Fracture cemented with filling material

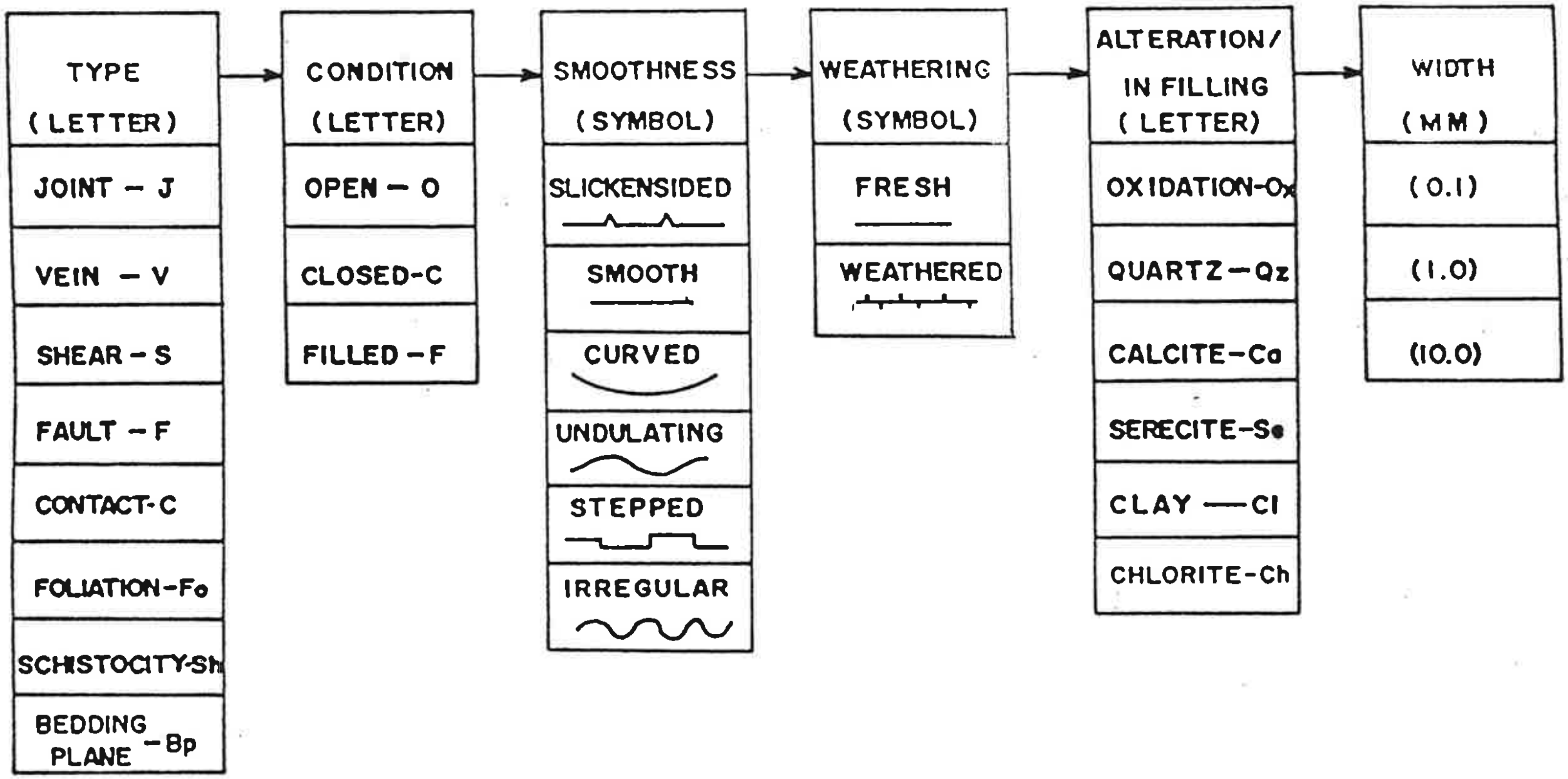
SURFACE ROUGHNESS

Classification	Field Test
Smooth	Appears smooth and is essentially smooth to the touch. May be slickensided.*
Slightly Rough	Asperities on the fracture surfaces are visible and can be distinctly felt
Moderately Rough	Asperities are clearly visible and fracture surface feels abrasive to touch
Rough	Large angular asperities can be seen. Some ridge and high side angle steps evident.
Very Rough	Near vertical steps and ridges occur on the fracture surface.

* Where slickensides are observed, the direction of the slickensides should be recorded after the standard discontinuity surface description.

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS

Discontinuity Classification Chart



EXAMPLE CLASSIFICATION:

JOINT + OPEN + IRREGULAR + WEATHERED + OXIDIZED + 1 mm WIDE JO ∿ Ox (1)

Major Divisions		Letter	Symbol		Name
			Hatching	Color	
Coarse-grained Soils	Gravel and Gravelly Soils	GW	▨	Red	Well-grained gravels or gravel-sand mixtures, little or no fines
		GP	▨		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	▨	Yellow	Silty gravels, gravel-sand silt mixtures
		GC	▨		Clayey gravels, gravel-sand-clay mixtures
	Sand and Sandy Soils	SW	▨	Red	Well-graded sands or gravelly sands, little or no fines
		SP	▨		Poorly graded sands or gravelly sands, little or no fines
		SM	▨	Yellow	Silty sands, sand-silt mixtures
		SC	▨		Clayey sands, sand-silt mixture
Fine-grained Soils	Silts and Clays (LL<50)	ML	▨	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
		CL	▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays silty clays, lean clays
		OL	▨		Organic silts and organic silt-clays of low plasticity
	Silts and Clays (LL>50)	MH	▨	Blue	Inorganic silts and organic silt-clays of low plasticity
		CH	▨		Inorganic clays of medium to high plasticity, organic silts
		OH	▨		Peat and other highly organic silts
Highly Organic Soils		Pt	▨	Orange	Peat and other highly organic soils

Rock	Diorite	▨
	Metasediment	▨
	Gneiss	▨
	Pegmatite	▨
	Anorthosite	▨

SAMPLE TYPE

"Standard Penetration Test": split spoon sampler, 2.0" OD/ 1 3/8" ID, driven with 140 lb. weight, 30" drop	T
Drive Sample: 2.5" ID split barrel sampler driven with 200 lb. weight, 24" drop	S
Ring Sample: 2.5" ID ring sampler driven	R
Continuous Sampler	C

TEST TYPE

Classification:	
Grain Size Analysis	ma
Plasticity	pi
Specific Gravity	sg
Shrink/Swell	s/s
Strength:	
Direct Shear	ds
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Torvane Shear	TVS
Consolidaton	c
Dynamic Tests:	
Cyclic Triaxial Compression	ctx
Chemical:	
EPA Method 8240	8240
EPA Method 8270	8270
EPA Method 8010	8010
EPA Method 8020	8020
Metals Scan	ICP
Photo Ionization Detector	PID

Water level at time of drilling: ═

Stabilized water level on date noted:

PREPARED BY JMH

REVIEWED BY

DATE

Unified Soil Classification System and Boring Log Explanation

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.41

DRAWING NO.
B-1

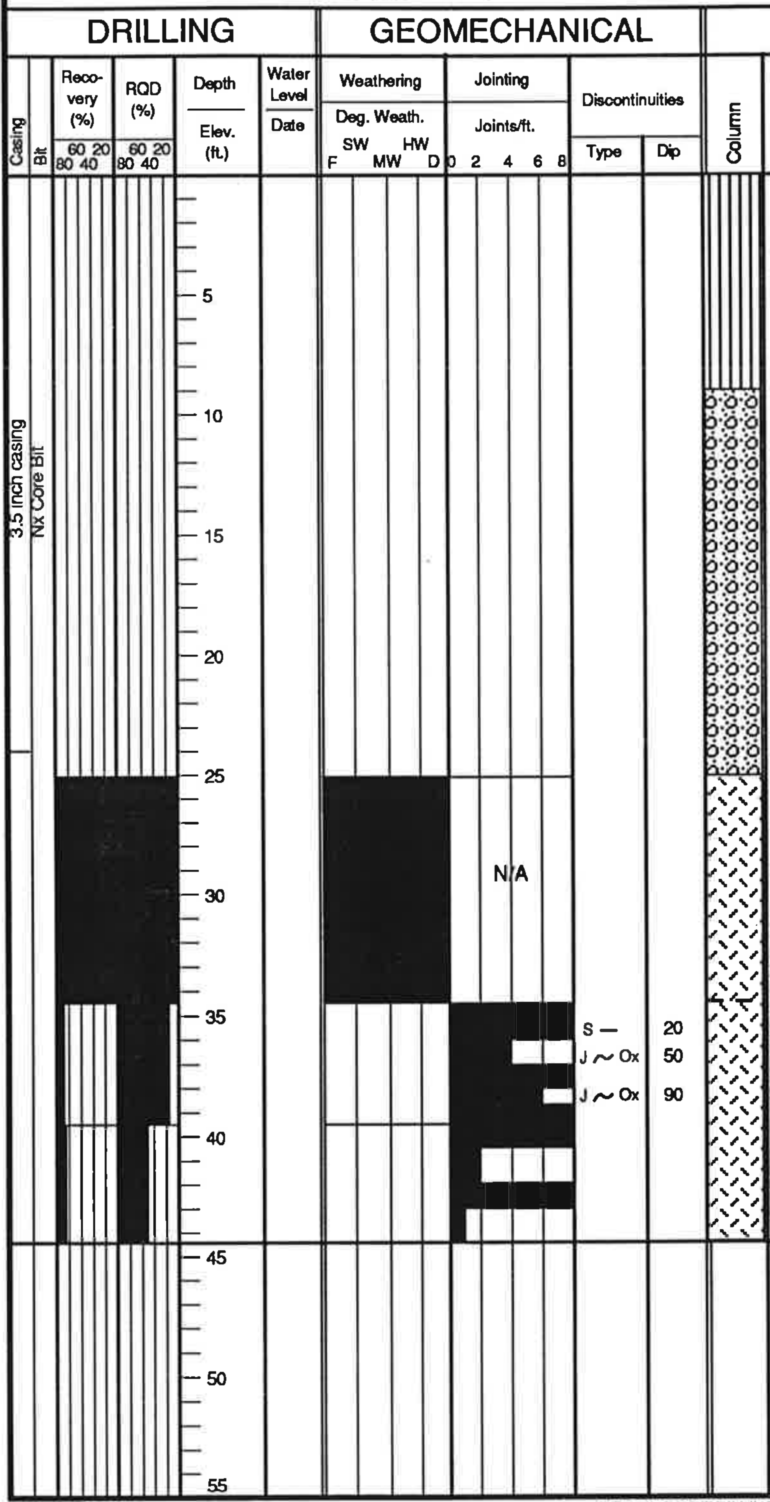
REPT-1 (01/93)

BOREHOLE LOG B-1

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 395800.14
E 1215456.73

ELEVATION 965.19
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH

DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 10/25/94 1000
FINISH 10/25/94 1600



GEOLOGICAL

Description

0.0 - 9.0 ft. Fan Deposits
Sandy SILT (ML) - dry to damp, pale yellowish brown (10YR 6/2), fine angular sand, trace angular gravel

9.0 - 25.1 ft. Terrace Deposits
Gravelly SAND / Sandy GRAVEL (SW/GW) - clasts slightly to moderately weathered, hard to moderately hard

25.1 - 44.4 ft. Bedrock
25.1 - 34.4 decomposed

34.4 - 44.4 ft.
Diorite - medium to medium dark gray (N5/N4), fine to medium grained, hard, 10% plagioclase, 5% mica, 80% hornblende

TD Boring @ 44.4 ft.
Piezometer Installed
No Water Encountered

TESTING

Sampling
Laboratory
Standard
Penetration
Water Pressure

PREPARED BY JMH
REVIEWED BY ADG
DATE 2/17/95

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

THE MARK GROUP, INC.
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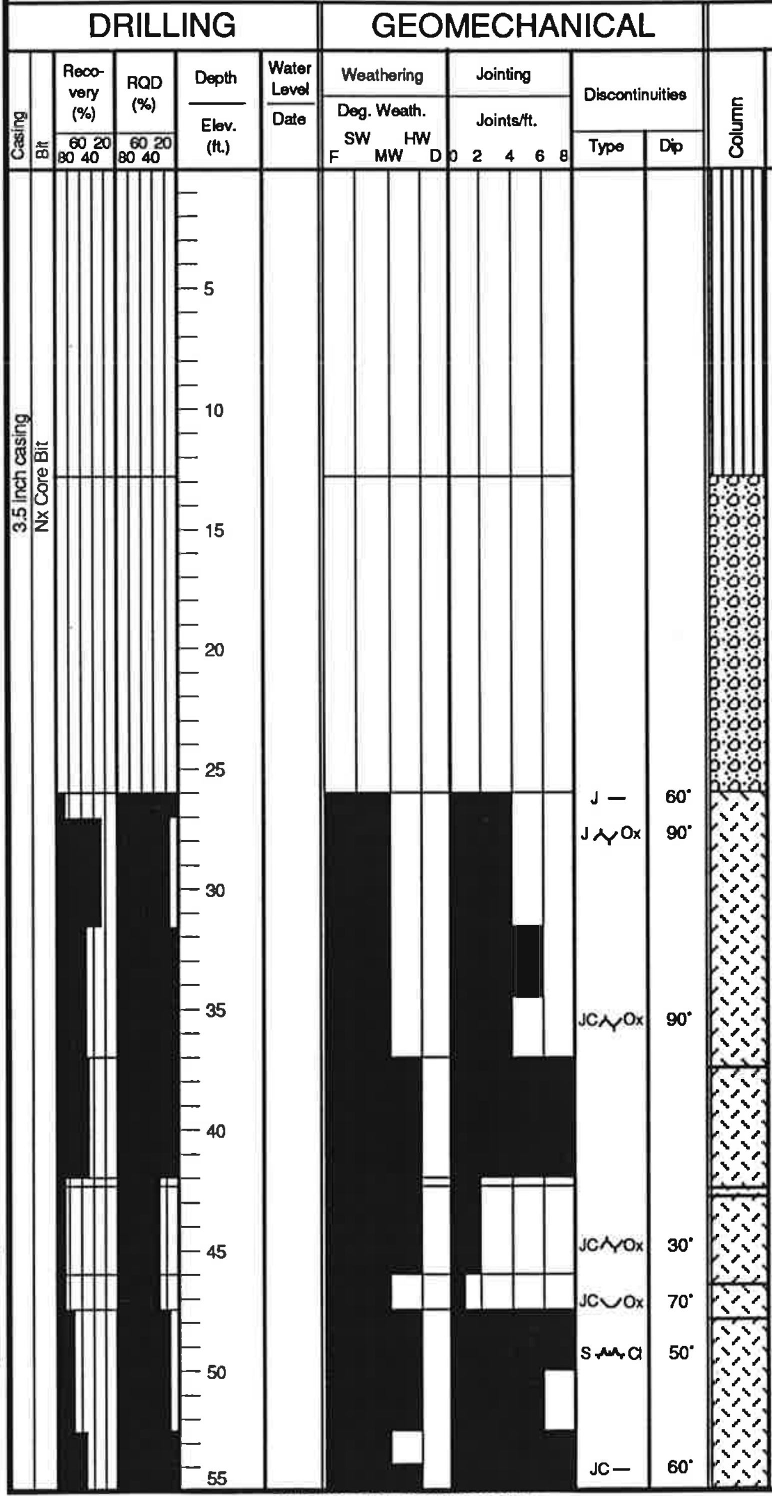
(11/5/94)

BOREHOLE LOG B-2

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 396019.62
E 1215377.34

ELEVATION 968.42
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH

DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 10/26/94 1100
FINISH 10/26/94 1530



Type	Dip
J —	60°
J ⋏ Ox	90°
JC ⋏ Ox	90°
JC ⋏ Ox	30°
JC ◡ Ox	70°
S ⋏⋏ Cl	50°
JC —	60°

GEOLOGICAL

Description

0.0 - 12.8 ft. Fan Deposits
Sandy SILT (ML) - dry to damp, pale yellowish brown (10YR 6/2), fine angular sand, trace subangular gravel to cobbles

12.8 - 26.0 ft. Terrace Deposits
Gravelly SAND / Sandy GRAVEL (SW/GW) - dioritic to granodioritic cobbles to boulders, clasts slightly to moderately weathered, hard

26.0 - 57.0 ft. Bedrock
26.0 to 37.0 Diorite - light gray (N9), medium to coarse grained, 70% plagioclase, 10% mica, 15% hornblende, 5% quartz

37.0 to 42.0 ft. Diorite - dark yellowish brown (10YR 5/2), fine to medium grained, medium hard, 10% plagioclase, 10% mica, 75% quartz
42.0 to 42.2 ft. Pegmatite - quartz
42.2 to 46.0 ft. Diorite - dark yellowish brown (10YR 5/2), fine grained, medium hard
46.0 to 47.5 ft. Diorite - dark gray (N3), fine to locally coarse grained, 30% plagioclase, 40% mica, 20% hornblende

TESTING

Sampling
Laboratory
Standard
Penetration
Water Pressure

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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BOREHOLE LOG B-2

DRILLING						GEOMECHANICAL				GEOLOGICAL		TESTING
Casing	Bit	Reco-very (%) 80 60 40 20	RQD (%) 80 60 40 20	Depth / Elev. (ft.)	Water Level / Date	Weathering Deg. Weath. F SW MW HW D	Jointing Joints/ft. 0 2 4 6 8	Discontinuities Type	Discontinuities Dip	Column	Description	Sampling Laboratory Standard Penetration Water Pressure
		■	■	56		■	■			▨		
											TD Boring @57.0 ft. Boring grouted with cement/bentonite No Water Encountered	
				60								
				65								
				70								
				75								
				80								
				85								
				90								
				95								
				100								
				105								
				110								
				115								

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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(11/16/94)

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BOREHOLE LOG B-3

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 395575.09
E 1215437.80

ELEVATION 967.96
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH

DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 10/27/94 1100
FINISH 10/27/94 1410

DRILLING

Casing | Bit | Recovery (%) 80 60 40 20 | RQD (%) 80 60 40 20 | Depth / Elev. (ft.) | Water Level / Date

GEOMECHANICAL

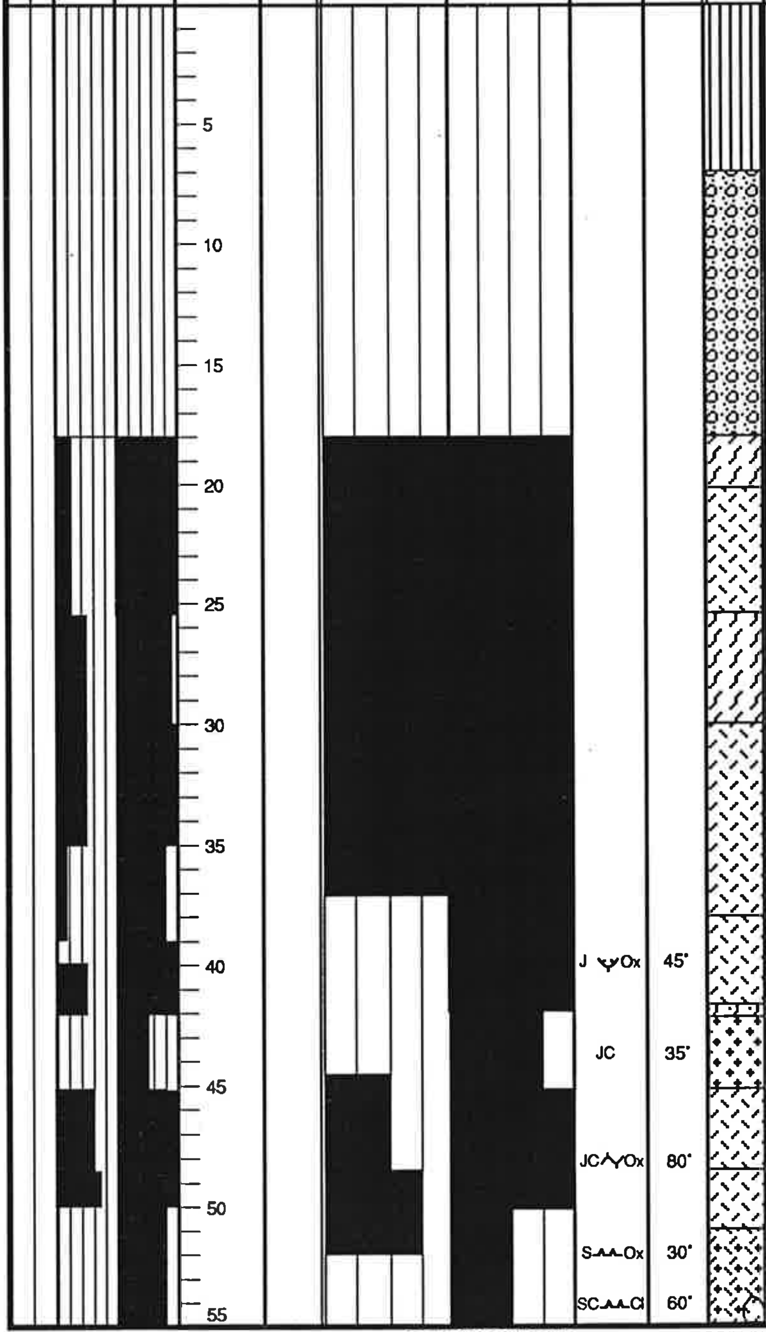
Weathering: Deg. Weath. F SW MW HW D | Jointing: Joints/ft. 0 2 4 6 8 | Discontinuities: Type, Dip

GEOLOGICAL

Column | Description

TESTING

Sampling
Laboratory
Standard
Penetration
Water Pressure



0.0 - 7.0 ft.Fan Deposits
Sandy SILT (ML) - dry to damp, pale yellowish brown (10YR 6/2), fine angular sand, trace subangular gravel

7.0 - 18.0 ft. Terrace Deposits
Gravelly SAND / Sandy GRAVEL (SW/GW) - moderate yellowish brown (10YR 5/4), subangular to subround

18.0 - 55.2 ft. Bedrock
18.0 to 20.2 Metasediment - moderate brown (5YR 4/4)
20.2 to 25.4 Diorite - yellowish gray (5Y 7/2), fine to medium grained, very soft

25.4 to 30.0 ft. Metasediment - dark yellowish orange (10YR 6/6), very soft

30.0 to 38.0 ft. Diorite - yellowish gray (5Y 7/2 to light gray (N7), fine grained, soft to very soft

38.0 to 41.7 ft. Diorite - very light gray (N8), medium to coarse grained, hard
41.7 to 42.2 ft. Gneiss - plagioclase and hornblende/mica
42.2 to 44.5 ft. Pegmatite - plagioclase/mica
44.5 to 48.6 ft. Diorite - very light gray (N8), medium to coarse grained, 75% plagioclase, 25% mica/hornblende
48.6 to 51.0 ft. Hornblendite - plagioclase-bearing, medium hard
51.0 to 55.2 ft. Anorthosite - white (N9) to dusky yellowish green (5GY 5/2), coarse grained, 5% mica

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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BOREHOLE LOG B-3

DRILLING						GEOMECHANICAL				GEOLOGICAL		TESTING
Casing	Bit	Reco-very (%) 80 60 40 20	RQD (%) 80 60 40 20	Depth / Elev. (ft.)	Water Level / Date	Weathering: Deg. Weath. F SW MW HW D	Jointing: Joints/ft. 0 2 4 6 8	Discontinuities: Type	Discontinuities: Dip	Column	Description	Sampling Laboratory Standard Penetration Water Pressure
				56						+ +	51.0 to 55.2 ft. Anorthosite - white (N9) to dusky yellowish green (5GY 5/2), coarse grained, 5% mica	
				60							TD Boring @55.2 ft. Boring grouted with cement/bentonite No Water Encountered	
				65								
				70								
				75								
				80								
				85								
				90								
				95								
				100								
				105								
				110								
				115								

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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Monterey County, California

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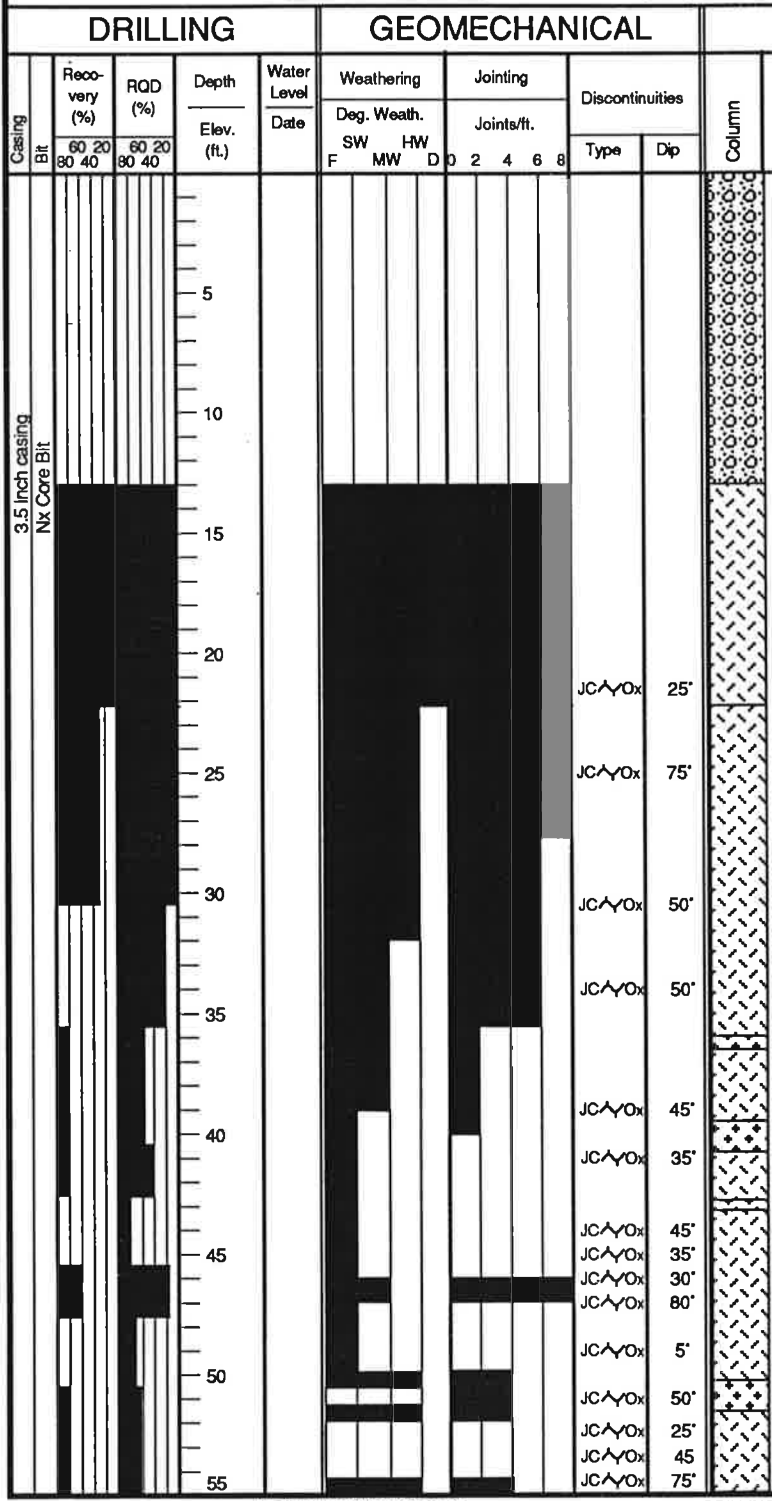
(11/15/94)

BOREHOLE LOG B-4

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 395741.93
E 1215625.53
ELEVATION 961.21
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH
DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 10/28/94 1100
FINISH 10/28/94 1800



GEOLOGICAL

Description

0.0 - 13.0 ft. Terrace Deposits
Sandy GRAVEL (GW)

13.0 - 60.1 ft. Bedrock
13.0 to 22.2 ft. Decomposed Bedrock - no recovery

22.2 to 36.0 ft. Diorite - pale brown (5YR 5/2), fine to medium grained, 8% plagioclase

36.0 to 36.6 ft. Pegmatite - plagioclase

36.6 to 39.5 ft. Diorite - medium dark gray (N4), fine to medium grained, hard

39.5 to 40.8 ft. Pegmatite - plagioclase, 5% mica, coarse grained

40.8 to 42.8 ft. Diorite - fine to coarse grained, hard

42.8 to 43.2 ft. Pegmatite - plagioclase

43.2 to 50.3 ft. Diorite - grayish black (N2), fine to medium grained, hard

50.3 to 51.6 ft. Pegmatite - very light gray (N8), plagioclase, 10% mica, very hard

51.6 to 60.1 ft. Diorite - grayish black (N2) to moderate yellowish brown (10YR 5/4), fine to medium grained, hard to moderately soft

TESTING

Sampling
Laboratory
Standard
Penetration
Water Pressure

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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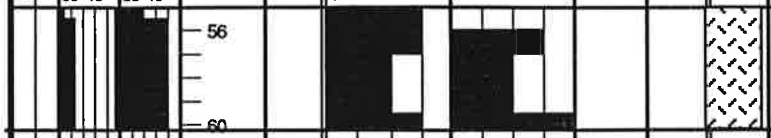
BOREHOLE LOG B-4

DRILLING	GEOMECHANICAL	GEOLOGICAL	TESTING
Casing Bit Recovery (%) 80 60 40 20 RQD (%) 80 60 40 20 Depth / Elev. (ft.) Water Level / Date	Weathering: Deg. Weath. F SW MW HW D Jointing: Joints/ft. 0 2 4 6 8 Discontinuities: Type, Dip	Column Description	Sampling Laboratory Standard Penetration Water Pressure



51.6 to 60.1 FT. Diorite - grayish black (N2) to moderate yellowish brown (10YR 5/4), fine to medium grained, hard to moderately soft

TD Boring @60.1 ft.

Boring grouted with cement/bentonite
No Water Encountered

Depth (ft.): 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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(11/16/94)

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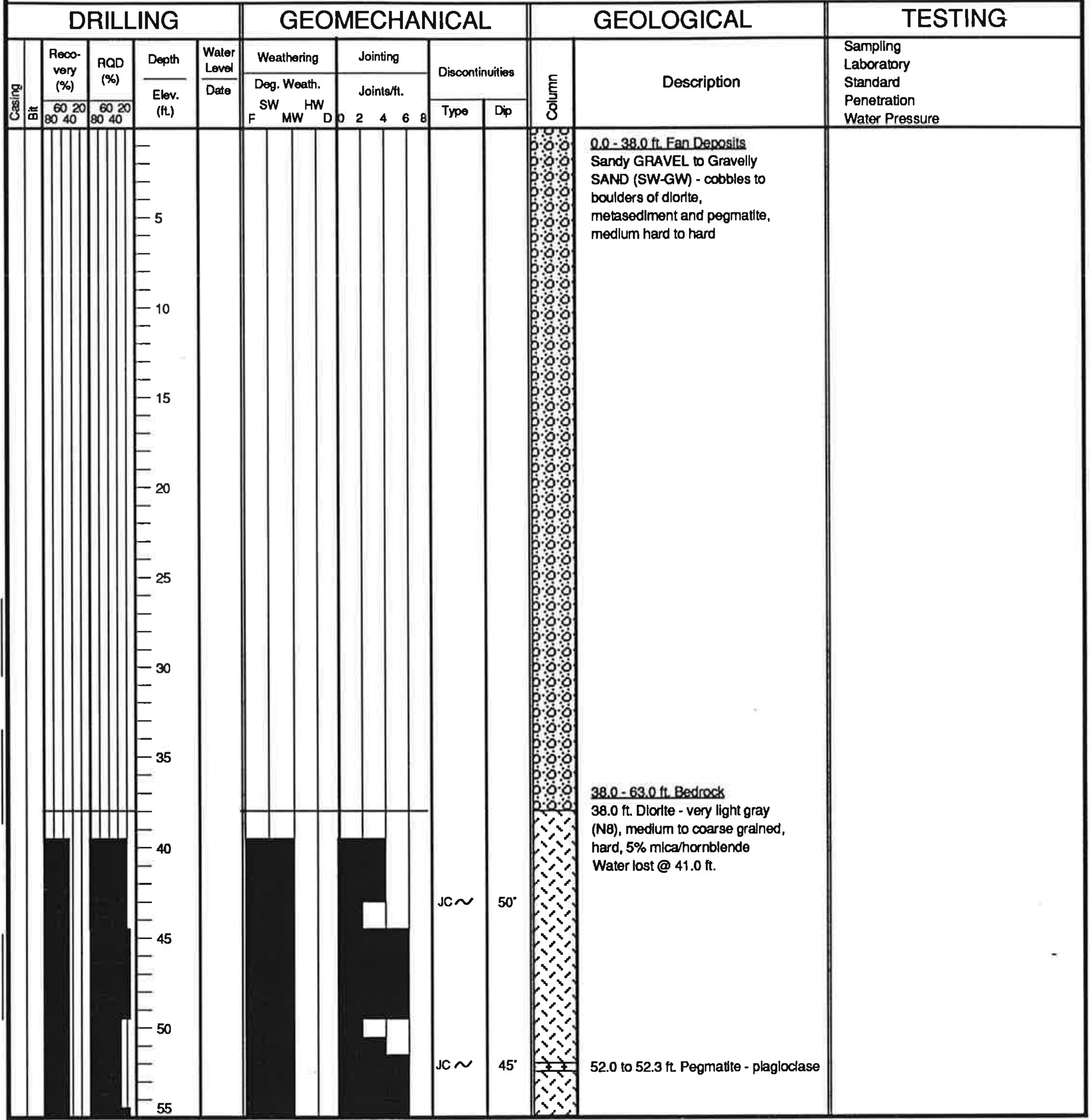
BOREHOLE LOG B-5

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 395749.74
E 1214721.86

ELEVATION 1086.94
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH

DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 10/31/94 1130
FINISH 11/01/94 1030



0.0 - 38.0 ft. Fan Deposits
Sandy GRAVEL to Gravelly SAND (SW-GW) - cobbles to boulders of diorite, metasediment and pegmatite, medium hard to hard

38.0 - 63.0 ft. Bedrock
38.0 ft. Diorite - very light gray (N8), medium to coarse grained, hard, 5% mica/hornblende
Water lost @ 41.0 ft.

52.0 to 52.3 ft. Pegmatite - plagioclase

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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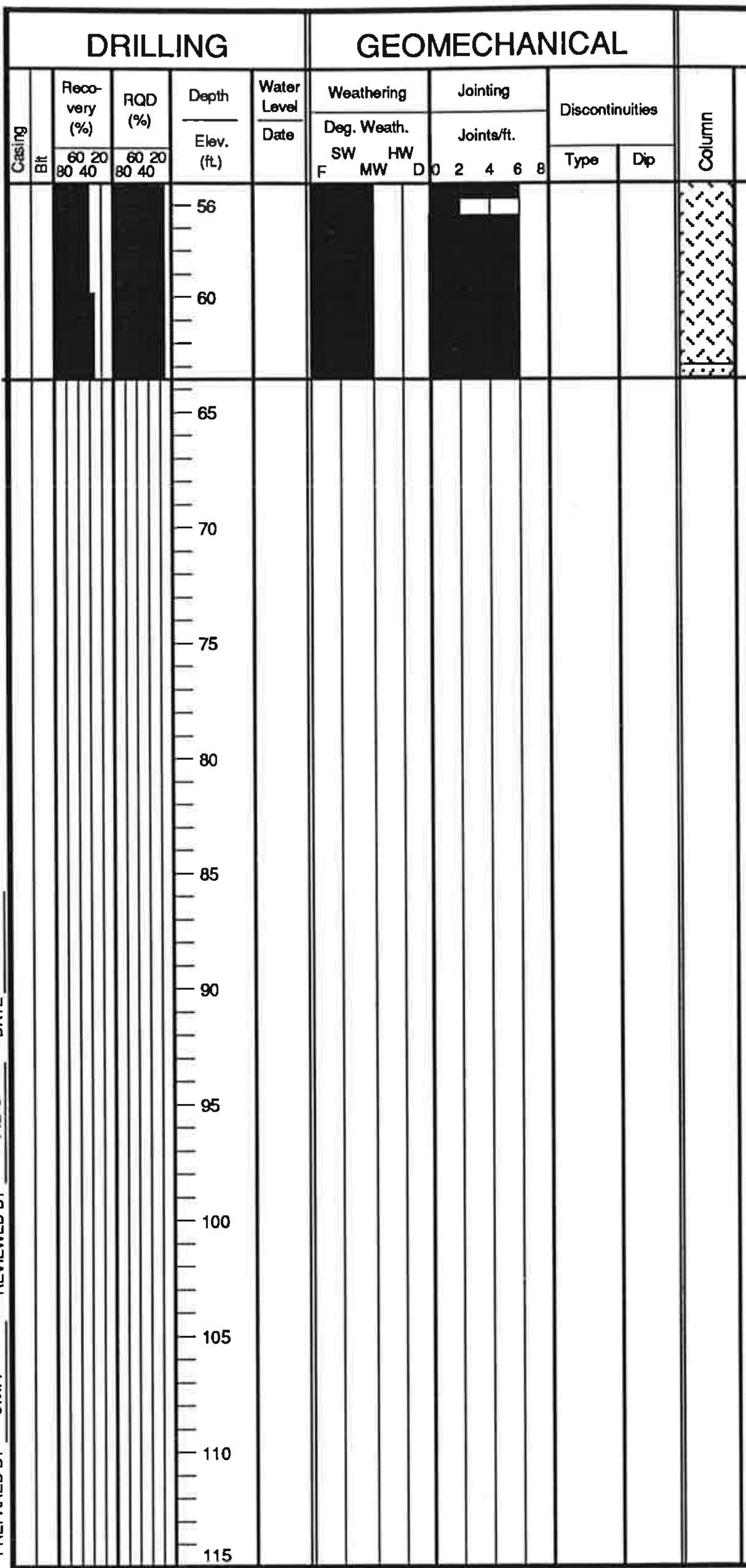
BOREHOLE LOG B-5

DRILLING | GEOMECHANICAL | GEOLOGICAL | TESTING



Description

63.0 to 66.5 ft. Gneiss - greenish black (5G 2/1) with dark yellowish orange (10YR 6/6), plagioclase, hard

TD Boring @63.5 ft.

Boring grouted with cement/bentonite
No Water Encountered

Testing:
Sampling
Laboratory
Standard
Penetration
Water Pressure

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

BOREHOLE LOG B-6

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 395805.12
E 1214902.71

ELEVATION 1066.57
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH

DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 11/01/94 1130
FINISH 11/02/94 1500

DRILLING						GEOMECHANICAL				GEOLOGICAL		TESTING
Casing	Bit	Recovery (%) 60 20 80 40	RQD (%) 60 20 80 40	Depth Elev. (ft.)	Water Level Date	Weathering Deg. Weath. SW HW F MW D	Jointing Joints/ft. 0 2 4 6 8	Discontinuities Type	Dip	Column	Description	Sampling Laboratory Standard Penetration Water Pressure
											<u>0.0 - 44.5 ft. Fan Deposits</u> Gravelly SAND to Sandy GRAVEL (SP-GP) - angular to subangular cobbles to boulders of diorite and pegmatite, clasts slightly weathered to decomposed	
				5								
				10								
				15								
				20								
				25								
				30								
				35								
				40								
				45							<u>44.5 - 65.5 ft. Terrace Deposits</u> Gravelly SAND / Sandy GRAVEL (SW/GW) - subround to round gravel to boulders, clasts moderately to highly weathered	
				50								
				55								

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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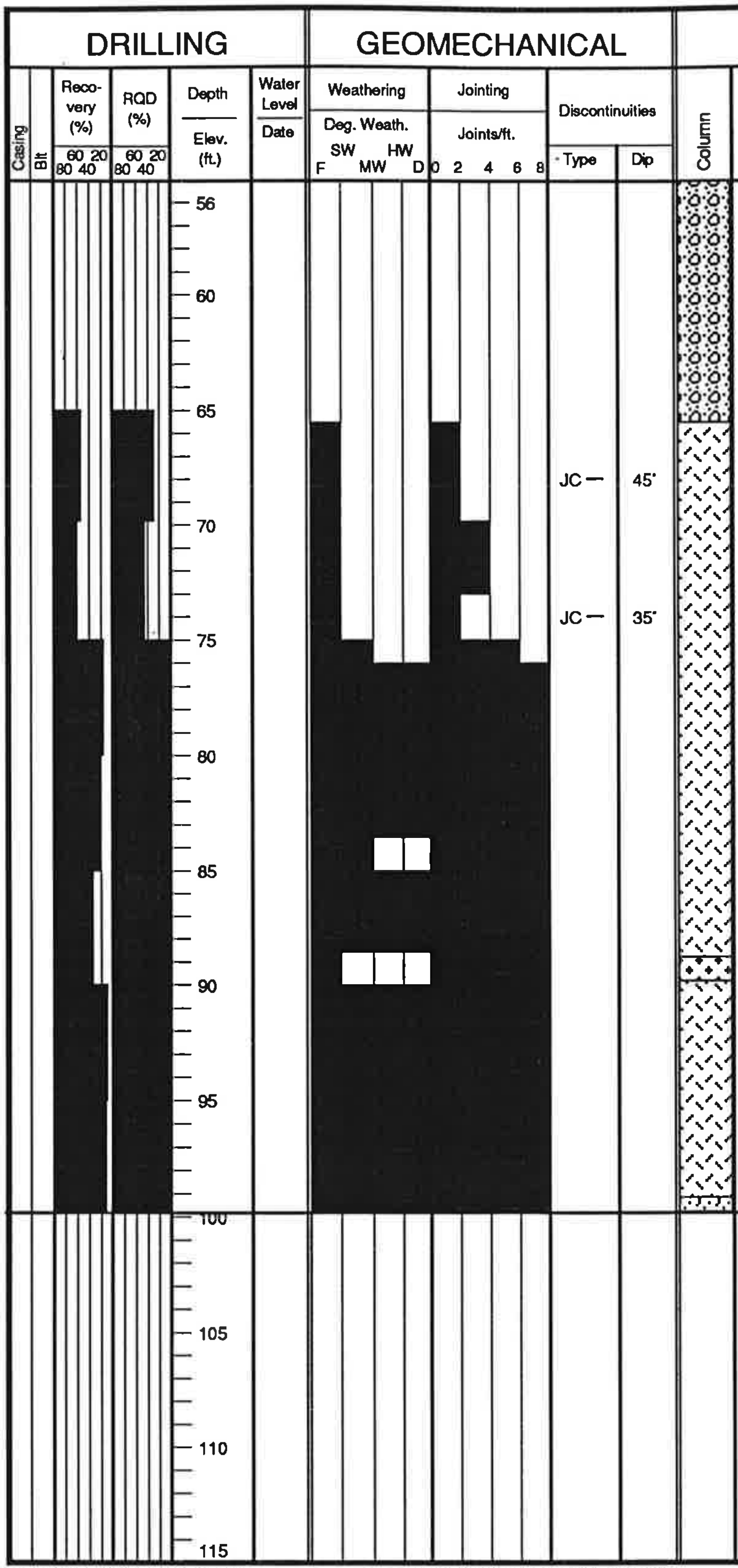
BOREHOLE LOG B-6



GEOLOGICAL

Description

TESTING

Sampling
Laboratory
Standard
Penetration
Water Pressure

<u>65.5 - 99.9 ft. Bedrock</u>

65.5 to 88.8 ft. Diorite - very light gray (N8), medium to coarse grained, 5-10% mica/horneblende, 90% plagioclase, hard

88.8 to 89.9 ft. Pegmatite - plagioclase, 2% mica

89.9 to 99.3 ft. Diorite - very light gray (N3) to greenish gray (5G 6/1), 15% mica, hard

99.3 to 99.9 ft. Gneiss - greenish black (5GY 2/1) with very light gray (N8), plagioclase bands, moderately hard

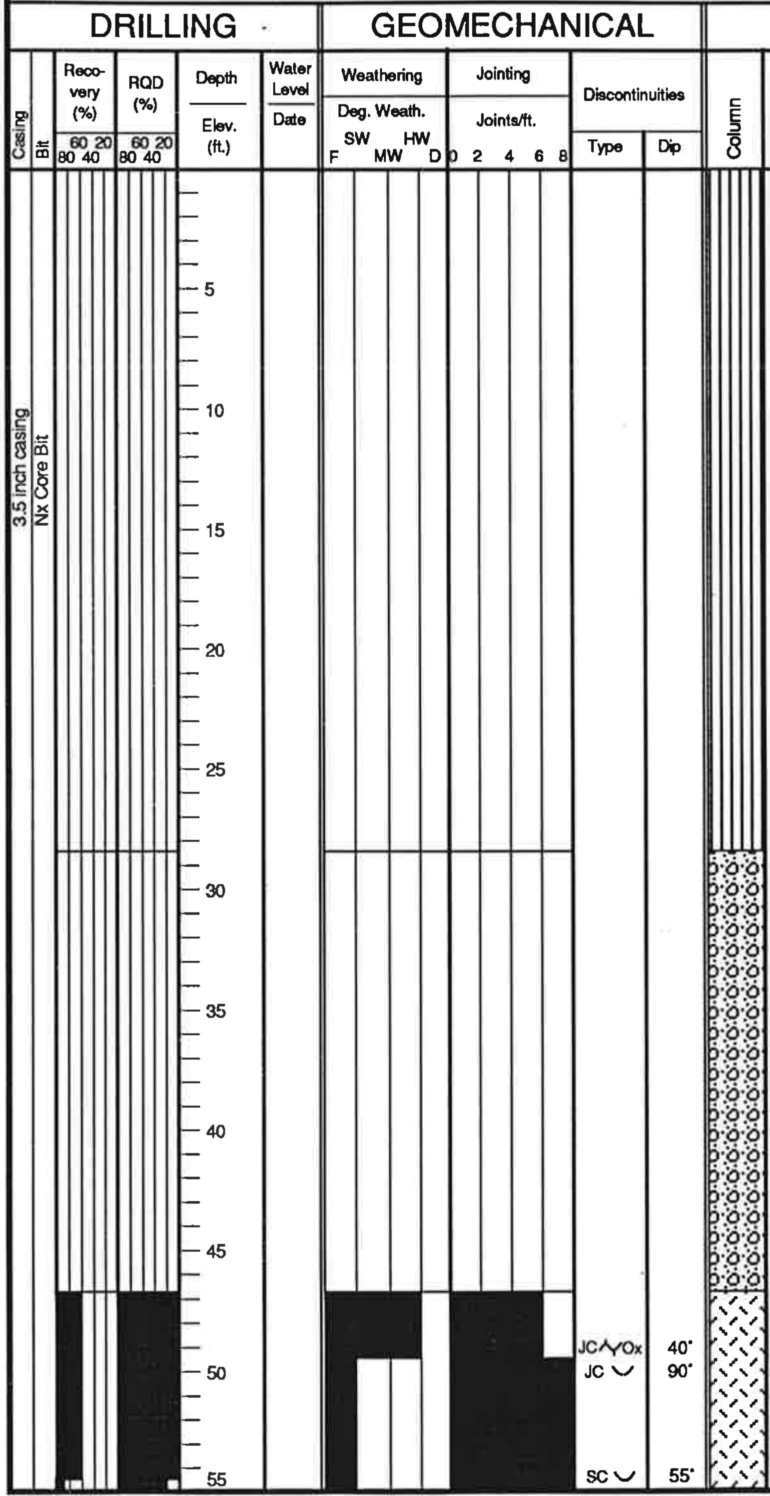
TD Boring @99.9 ft.

Boring grouted with cement/bentonite
No Water Encountered

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

BOREHOLE LOG B-7

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 395828.92
E 1215140.28
ELEVATION 992.88
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH
DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 11/03/94 0800
FINISH 11/04/94 1615



GEOLOGICAL

Description

0.0 - 28.4 ft. Fan Deposits
Sandy SILT (ML) - dry to damp, pale yellowish brown (10YR 6/2), fine angular sand, trace to some subangular gravel, cobbles, boulders, diorite boulders, moderately weathered

28.4 - 46.7 ft. Terrace Deposits
Gravelly SAND /Sandy GRAVEL (SW/GW) - gravel, cobbles to small boulders, clasts moderately weathered

46.7 - 62.5 ft. Bedrock
46.7 to 49.4 ft. Diorite - light gray (N7), medium grained, some pitting, 60% plagioclase, 30% mica/hornblende
49.4 to 62.5 ft. Diorite - grayish black (N2), medium grained, 40% plagioclase, 60% mica/hornblende

TESTING

Sampling
Laboratory
Standard
Penetration
Water Pressure

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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THE MARK GROUP, INC.
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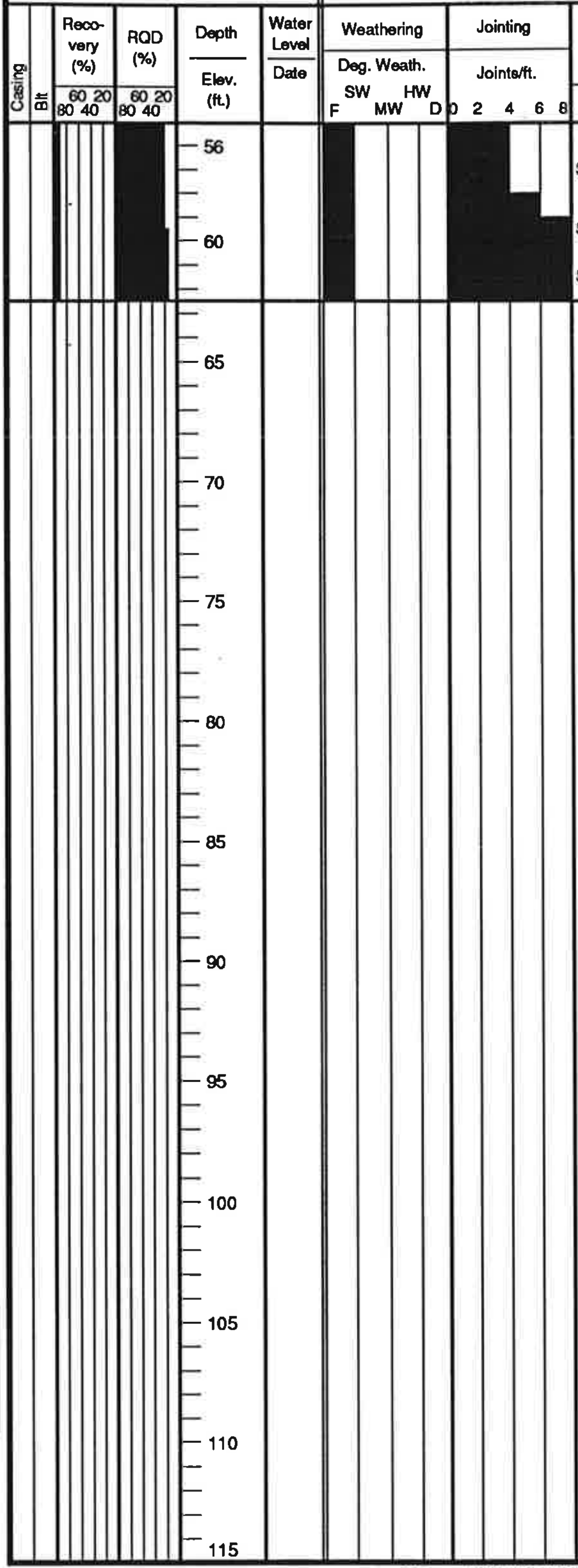
BOREHOLE LOG B-7

DRILLING	GEOMECHANICAL	GEOLOGICAL	TESTING



Discontinuities Type	Dip	Column	Description	Sampling Laboratory Standard Penetration Water Pressure
JC∿	50°	⋰	49.4 to 62.5 ft. Diorite - grayish black (N2), medium grained, 40% plagioclase, 60% mica/hornblende, locally sheared to bottom of boring, some chlorite	
S.AA.Ch	55°			
S.AA.Ch	40°			
S.AA.Ch	40°			
			TD Boring @62.5 ft. Piezometer Installed No Water Encountered	

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PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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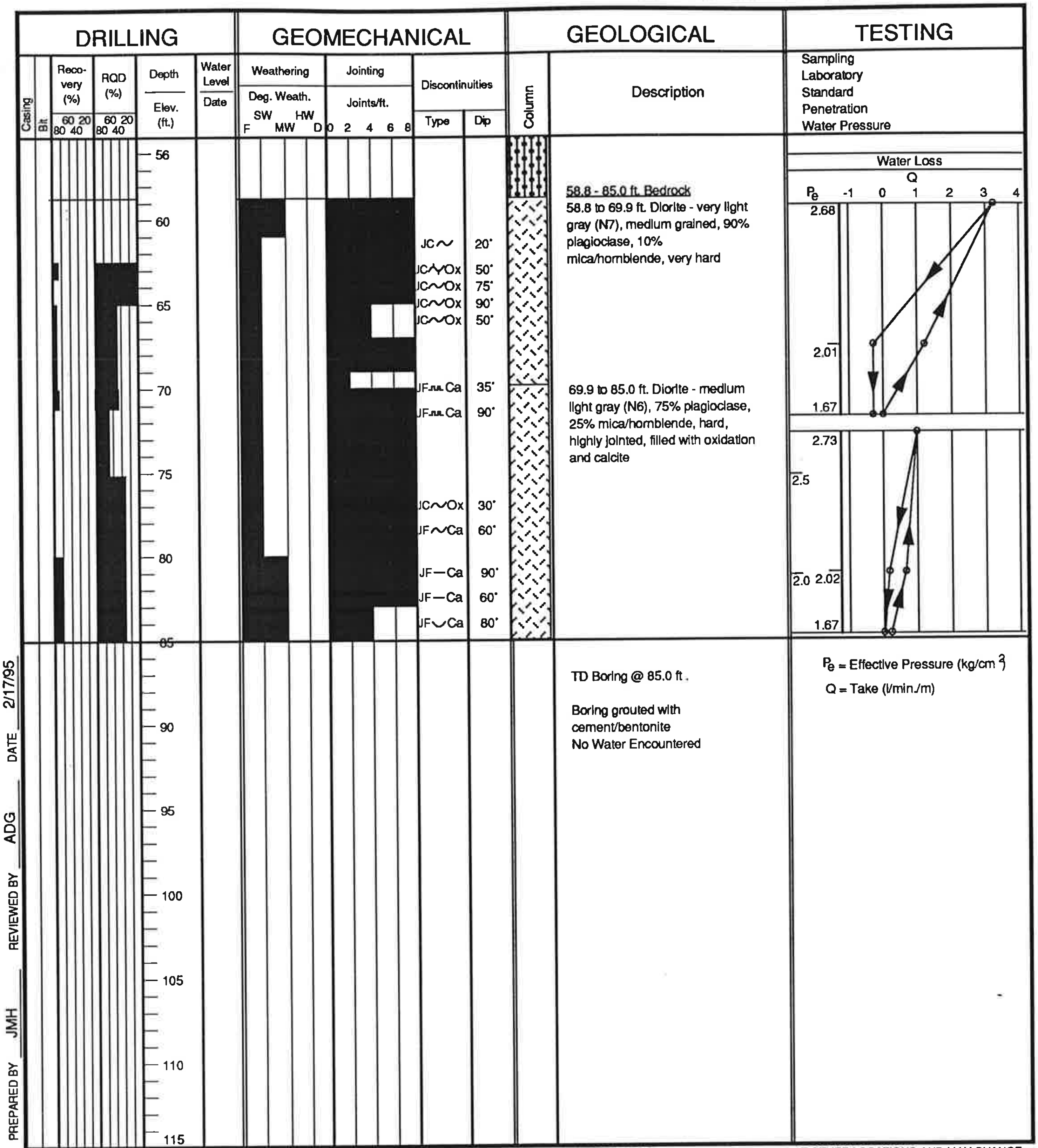
BOREHOLE LOG RA-2

PROJECT MPWMD - New Los Padres Dam
LOCATION Borrow Area A
COORDINATES N 396451.8
E 1216248.4
ELEVATION 1048.1
INCLINATION Vertical
DIRECTION ---
LOGGED BY JMH
DRILLING CONTRACTOR PC Exploration
RIG Mobile B-53
START 11/07/94 0900
FINISH 11/08/94 1700

DRILLING						GEOMECHANICAL				GEOLOGICAL		TESTING
Casing	Bit	Recovery (%) 80 60 40 20	RQD (%) 80 60 40 20	Depth / Elev. (ft.)	Water Level / Date	Weathering / Deg. Weath. F SW MW HW D	Jointing / Joints/ft. 0 2 4 6 8	Discontinuities Type	Discontinuities Dip	Column	Description	Sampling Laboratory Standard Penetration Water Pressure
				0							<u>0.0 - 61.0 ft. Fan Deposits</u> Sandy SILT/Silty SAND (ML/SM) - damp to moist, pale yellowish brown (10YR 6/2), cobbles to boulders, clasts moderately to highly weathered	
				5								
3.5 inch casing	Nx Core Bit			10								
				15								
				20								
				25								
				30								
				35								
				40								
				45								
				50								
				55								

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

BOREHOLE LOG RA-2



58.8 - 85.0 ft. Bedrock

58.8 to 69.9 ft. Diorite - very light gray (N7), medium grained, 90% plagioclase, 10% mica/hornblende, very hard

69.9 to 85.0 ft. Diorite - medium light gray (N6), 75% plagioclase, 25% mica/hornblende, hard, highly jointed, filled with oxidation and calcite

TD Boring @ 85.0 ft.

Boring grouted with cement/bentonite
No Water Encountered

P_e = Effective Pressure (kg/cm^2)
Q = Take (l/min./m)

PREPARED BY JMH REVIEWED BY ADG DATE 2/17/95

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(11/16/94)

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CONSTRUCTION LOG OF WELL No. B-1

Well Type: Groundwater Level Monitoring

Casing Type: Schedule 40 PVC

Casing Dia.: 1 inch

Casing Lengths: 10 foot

Screen Size: 0.020 inch

Screen Length: 5 feet

Tailpipe Length: None

Locking Cover Type: N/A

Locking Cover Stickup (+) / Depression (-) : +18"

Filter Material: #3 Lonestar Sand

Filter Volume: 0.31 cf

Seal Material 1/4-inch bentonite chips

Seal Volume: 0.09 cf

Grout Material: cement/bentonite grout

Grout Volume: 1.92 cf

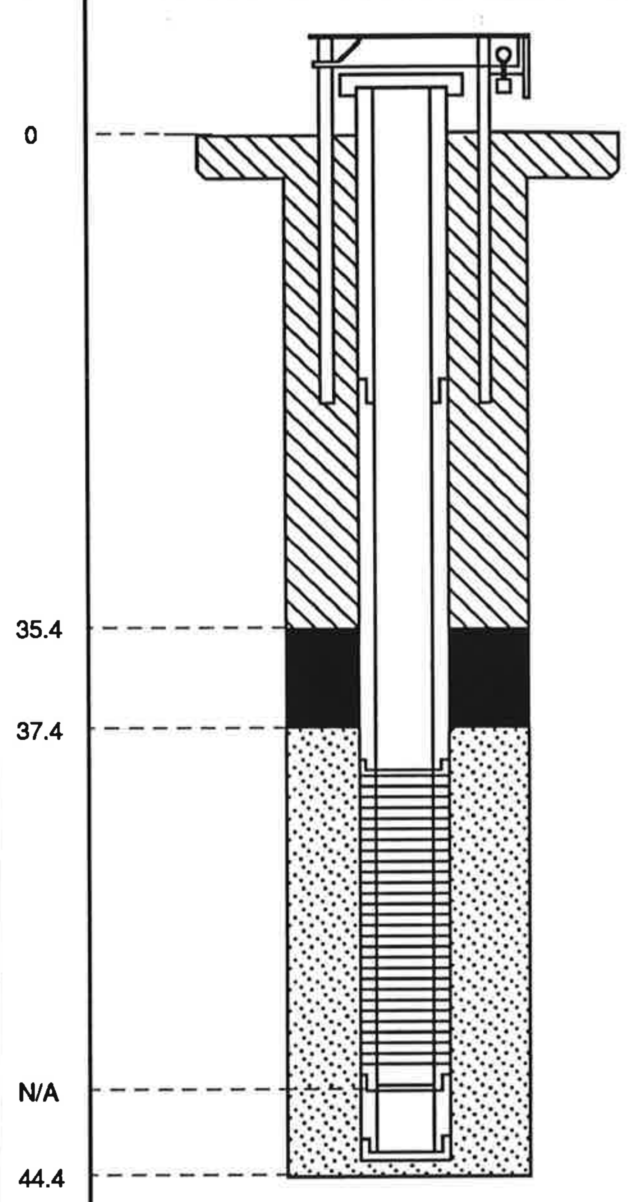
Bore Dia.: 3 to 3-1/2 inches

Total Depth: 44.4'

Comments: Water from drilling process (rotary wash) standing in well.

DEPTH IN FEET

Date Completed: 10/26/94



PREPARED BY JMH

REVIEWED BY KBK

DATE 2/9/95

THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS. THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS WELL AT THE TIME OF CONSTRUCTION. CONDITIONS SHOWN MAY CHANGE WITH THE PASSAGE OF TIME.

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Monterey Peninsula Water Management District
Geotechnical and Engineering Services
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.41

SHEET 1 OF 1

WELL - (6/30/92)

CONSTRUCTION LOG OF WELL No. B-7

Well Type: Groundwater Level Monitoring

Casing Type: Schedule 40 PVC

Casing Dia.: 1 inch

Casing Lengths: 10 foot

Screen Size: 0.020 inch

Screen Length: 5 feet

Tailpipe Length: None

Locking Cover Type: N/A

Locking Cover Stickup (+) / Depression (-) : +18"

Filter Material: #3 Lonestar Sand

Filter Volume: 0.31 cf

Seal Material 1/4-inch bentonite chips

Seal Volume: 0.09 cf

Grout Material: cement/bentonite grout

Grout Volume: 3.43 cf

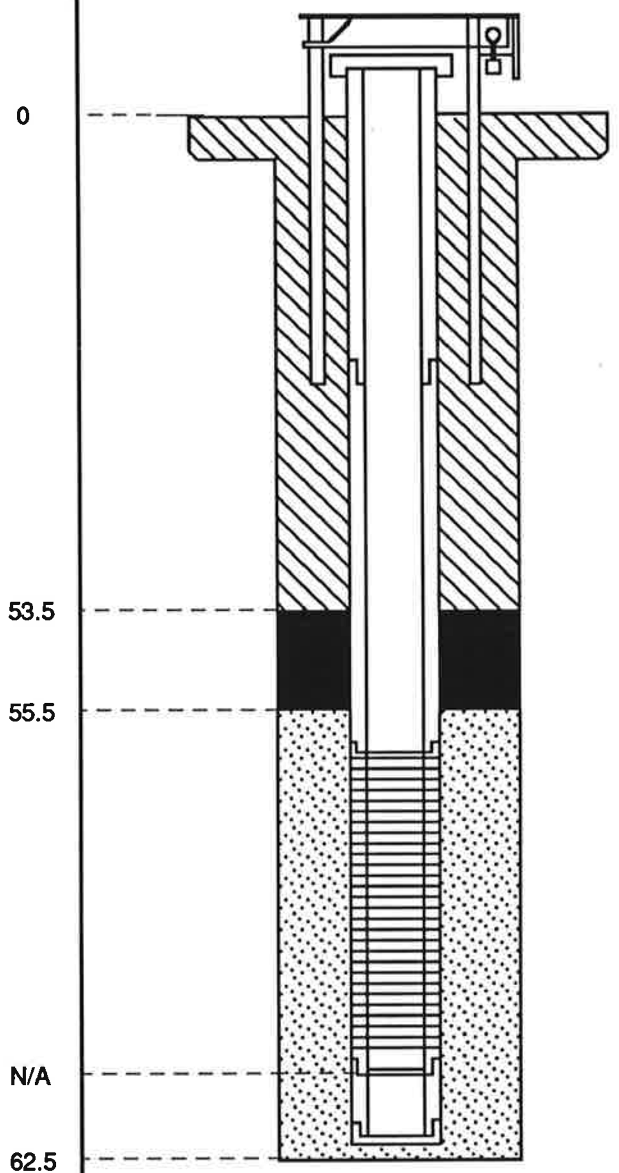
Bore Dia.: 3 to 3-1/2 inches

Total Depth: 62.5"

Comments: Water from drilling process (rotary wash) standing in well.

DEPTH IN FEET

Date Completed: 11/6/94



PREPARED BY JMH

REVIEWED BY KBK

DATE 2/9/95

THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS. THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS WELL AT THE TIME OF CONSTRUCTION. CONDITIONS SHOWN MAY CHANGE WITH THE PASSAGE OF TIME.

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Monterey County, California

PROJECT NO.
94-1198801.41

SHEET 1 OF 1

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

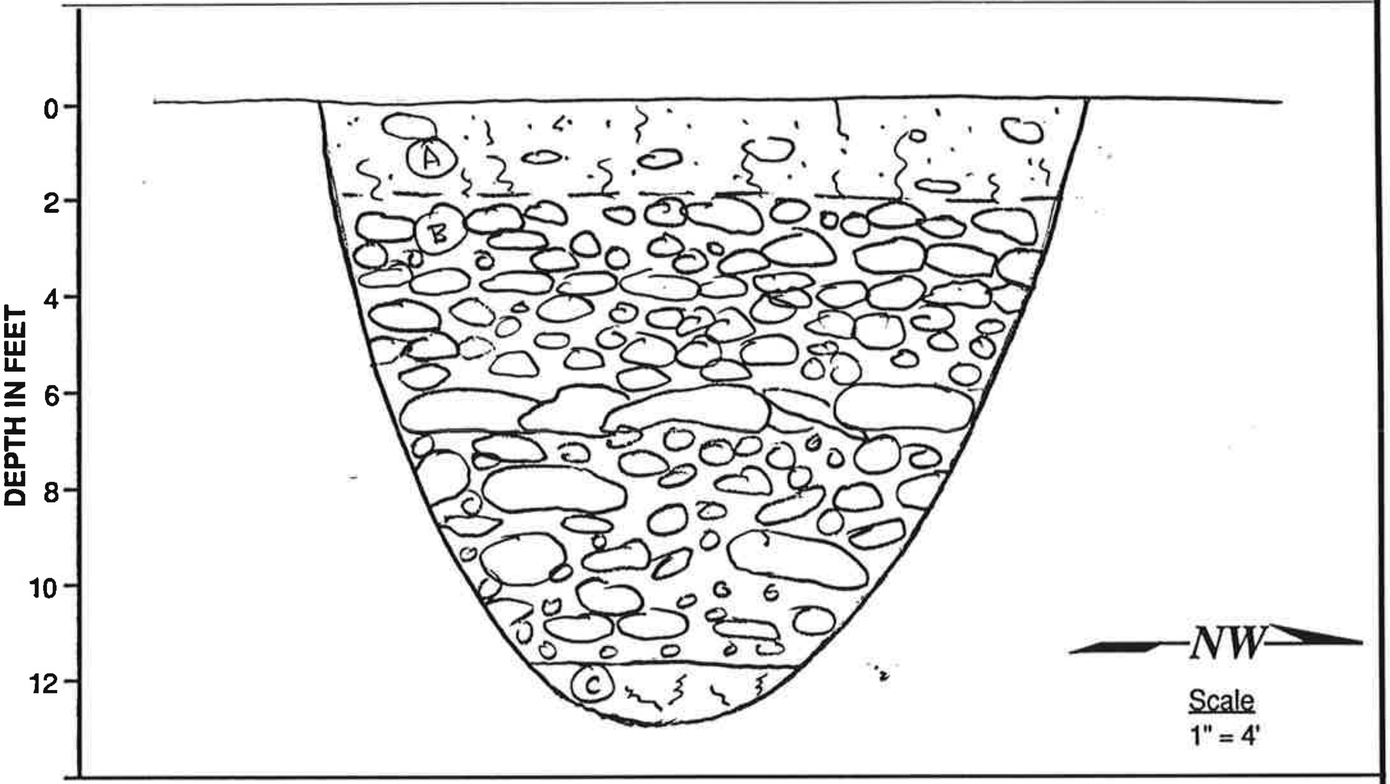
Log of Test Pit No. TP-1

ELEVATION: 981.7 FT.

Date 2/17/95

Approved By ADG

Logged By JMH



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Terrace Deposits
		0-2		Ⓐ SAND (SW) - dry to damp, moderate yellowish brown (5YR 5/4), some boulders, subangular to subround, rootlets
		2-11.5		Ⓑ Sandy GRAVEL (GP/GW) - moist, moderate yellowish brown (5YR 5/4), clasts subround to subangular, clast supported -3" = 30% +3" = 50% +12" = 20% Maximum clast diameter = 3'
				Bedrock
		11.5-13		Ⓒ Metasediments, granitic veins, moderately to highly weathered, hard below 12.5 feet. TD @13 feet, no water

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS TEST PIT AND AT THE TIME OF EXCAVATION. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

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Monterey County, California

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SHEET 1 OF 1

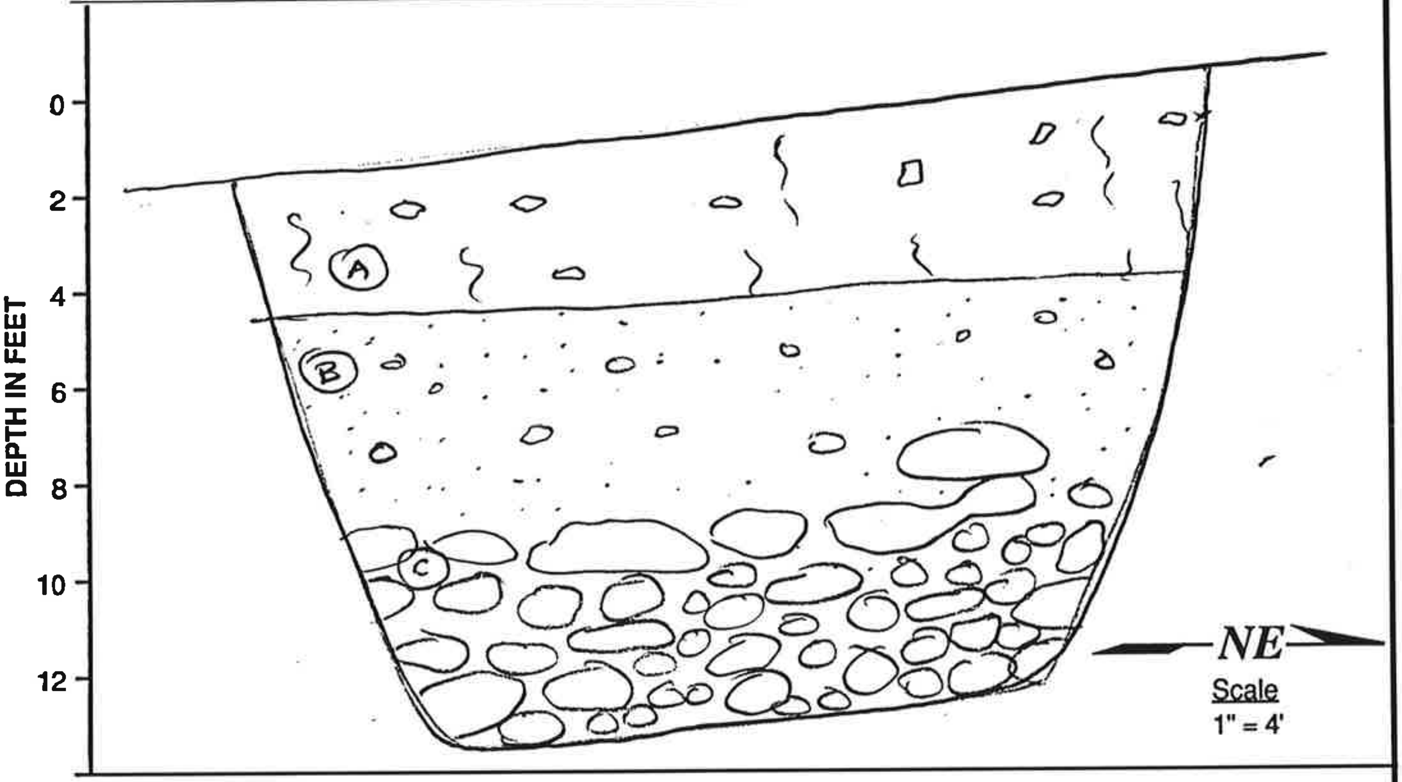
FLD-6 11/9/87

Log of Test Pit No. TP-2

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

ELEVATION: 986.7 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
		0-4		**Fan Deposits** Ⓐ Gravelly SAND (SW) - dry to damp, pale yellowish brown (10YR 6/2), fine sand to coarse gravel, granitic and metamorphic, angular to subrounded, rootlets -3" = 70% +3" = 20% +12" = 10% Maximum clast diameter = 3'
TP-2	Sack	4-8.5		**Terrace Deposits** Ⓑ Gravelly SAND (SW) - moist, moderate yellowish brown (10YR 5/4), fine to coarse sand with some fine gravel, subrounded
		8.5-12.5		Ⓒ Sandy GRAVEL (GP/GW) - moist, moderate yellowish brown (10YR 5/4), coarse gravel to boulders, subangular to subround -3" = 40% +3" = 55% +12" = 5% Maximum clast diameter = 2' TD @12.5 feet, no water

Logged By JMH

Approved By ADG

Date 2/17/95

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SHEET 1 OF 1

FLD-6 11/9/87

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

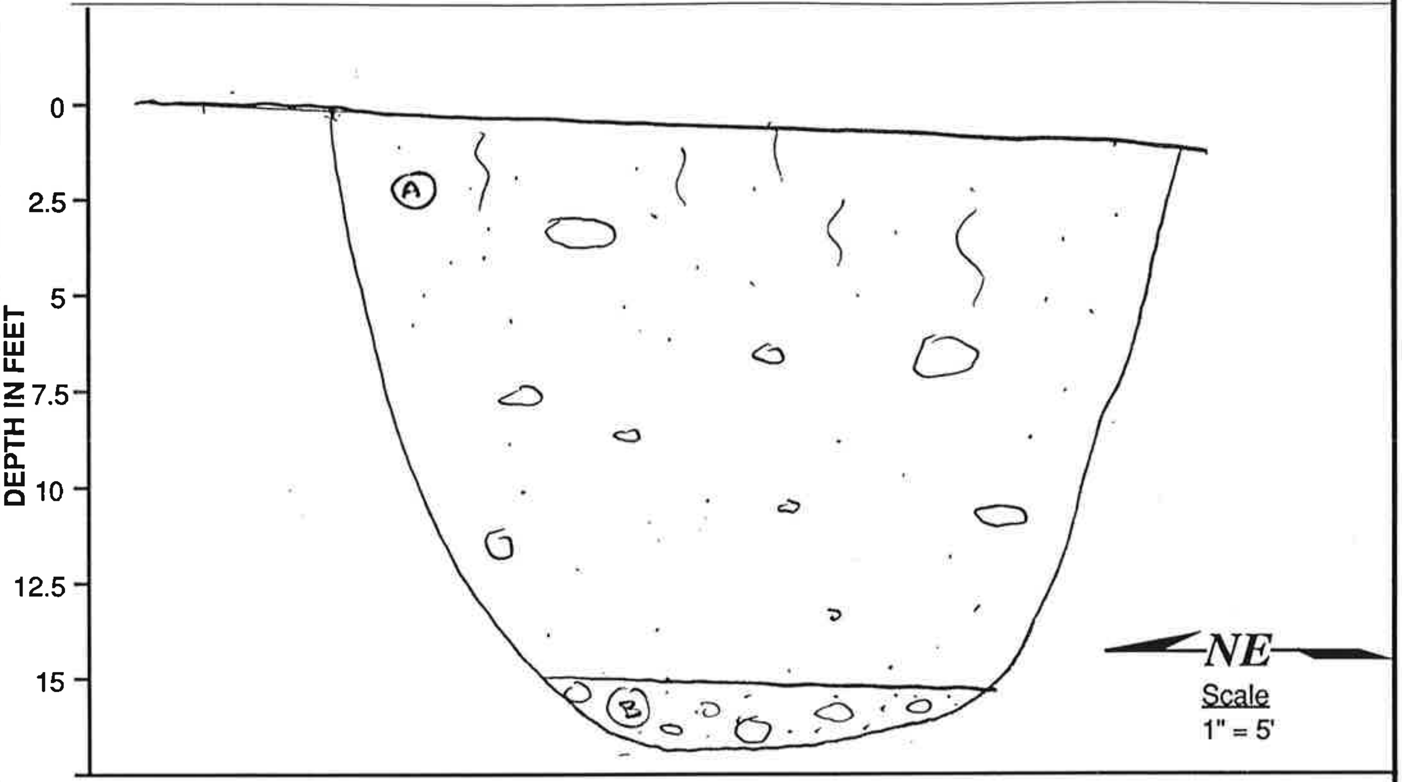
Log of Test Pit No. TP-3

ELEVATION: 991.9 ft.

Logged By JMH

Approved By ADG

Date 2/17/95



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-14.5		Ⓐ Silty SAND (SW) - dry with damp over depth, pale yellowish brown (10YR 6/2), fine sand to coarse gravel to small boulders, angular to subangular, rootlets
				Terrace Deposits
TP-3	Sack	14.5-16.5		Ⓑ SAND (SW) - moist, moderate yellowish brown (10YR 5/4), fine to coarse sand with a trace fine gravel, subangular -3" = 70% +3" = 30% +12" = 0% Maximum clast diameter = 8 inches
				TD @16.5 feet, no water

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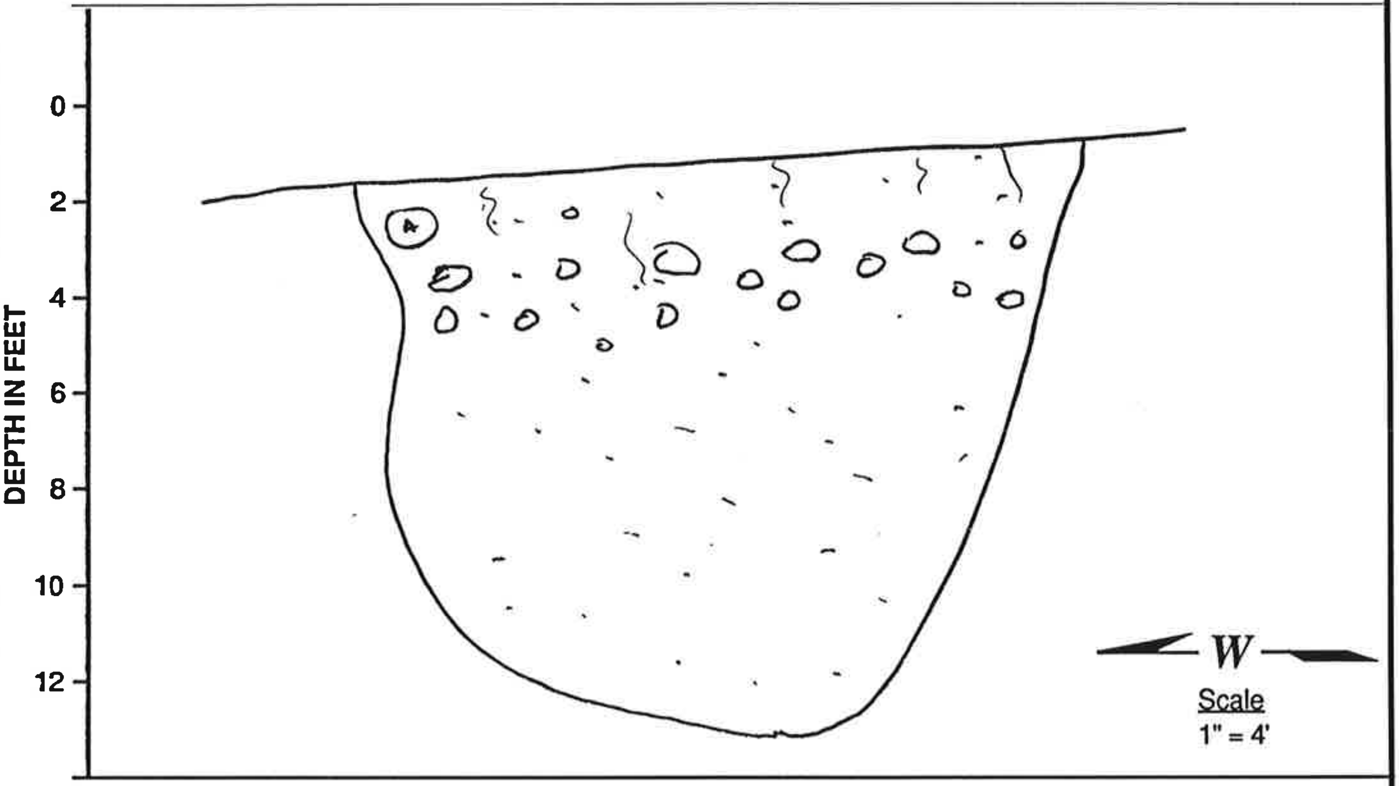
FLD-6 11/9/87

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

Log of Test Pit No. TP-4

ELEVATION: 1097.0 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-12		Ⓐ Silty SAND (SM) - damp to moist, moderate yellowish brown (10YR 5/4), trace clay, coarse gravel, boulders in upper 3-4 feet, subround, rootlets -3" = 75% +3" = 20% +12" = 5% Maximum clast diameter = 12 inches TD @12 feet, no water

Logged By JMH

Approved By ADG

Date 2/17/95

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New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
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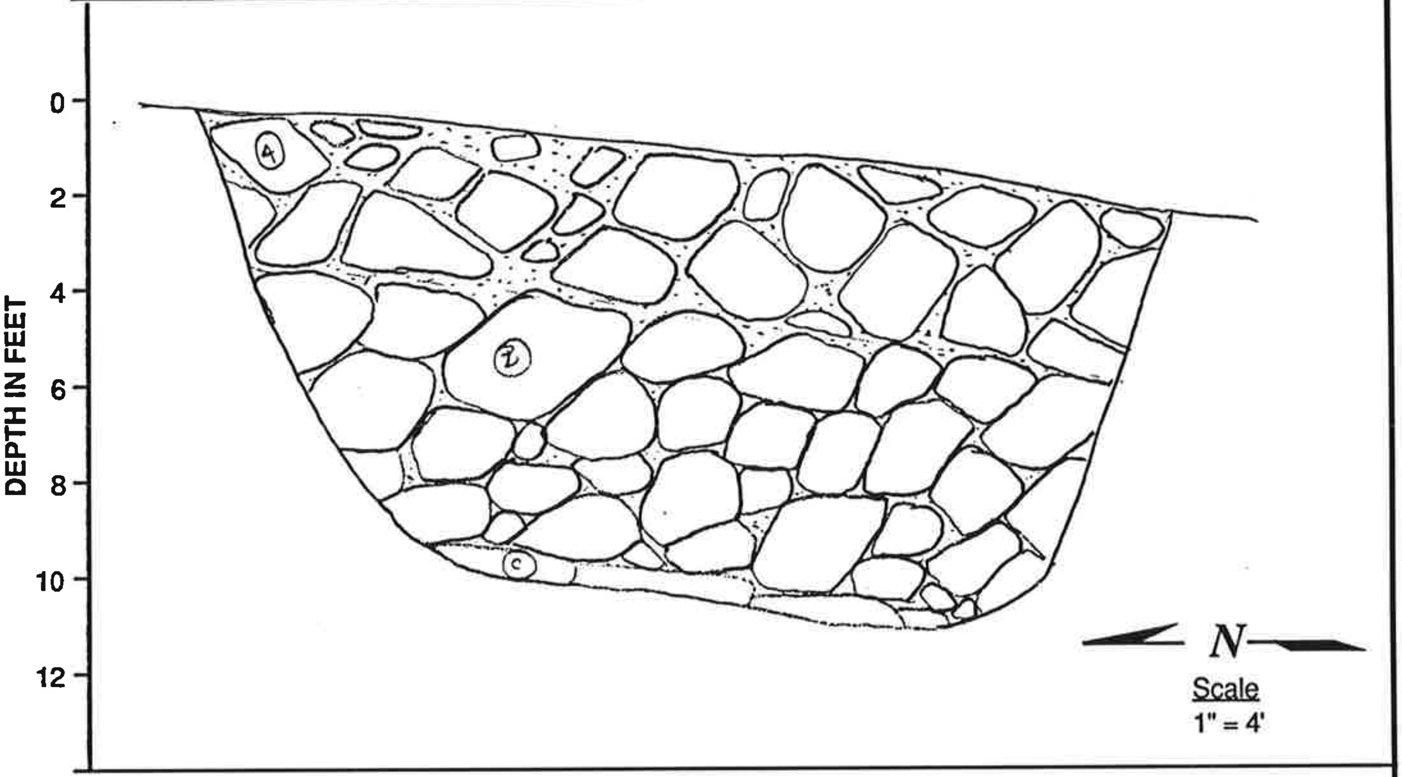
SHEET 1 OF 1

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

Log of Test Pit No. TP-5

ELEVATION: 998.5 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-3.5		Ⓐ Sandy GRAVEL (GP) - dry, pale yellowish brown (10YR 6/2), cobbles to boulders, angular to subangular, silty fine sand matrix, abundant roots to 3" diameter, clasts slightly to moderately weathered
		3.5-9		Ⓑ As in Ⓐ but much less matrix
		9-9.5		Ⓒ Large boulders, highly weathered rind, moderately weathered cores
				Refusal at 9.5 feet, no water

Logged By JMH

Approved By ADG

Date 2/17/95

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SHEET 1 OF 1

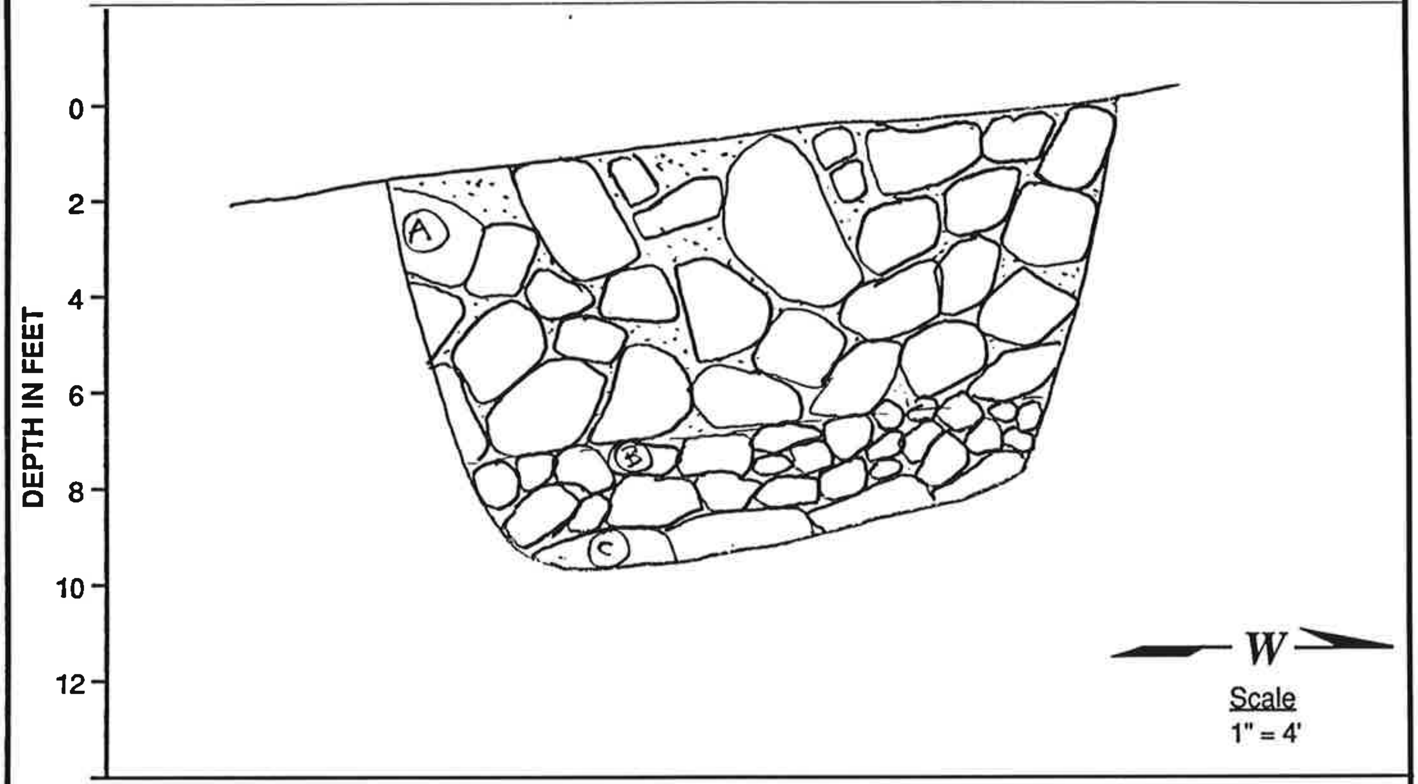
FLD-6 11/9/87

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

Log of Test Pit No. TP-6

ELEVATION: 1093.2 ft.



Scale
1" = 4'

SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-6		Ⓐ Sandy GRAVEL (GP) - dry, pale yellowish brown (10YR 6/2), cobbles to boulders, angular to subangular, silty fine sand matrix, abundant roots to 1" diameter, clasts slightly to moderately weathered
		6-8		Ⓑ As in Ⓐ but much less matrix, clasts moderately to highly weathered
		8-8.5		Ⓒ Large boulders, highly weathered rind, moderately weathered cores
				Refusal at 8.5 feet, no water

Logged By JMH

Approved By ADG

Date 2/17/95

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FLD-6 11/9/87

DATE EXCAVATED: 10/20/94

EQUIPMENT: Deere 710B

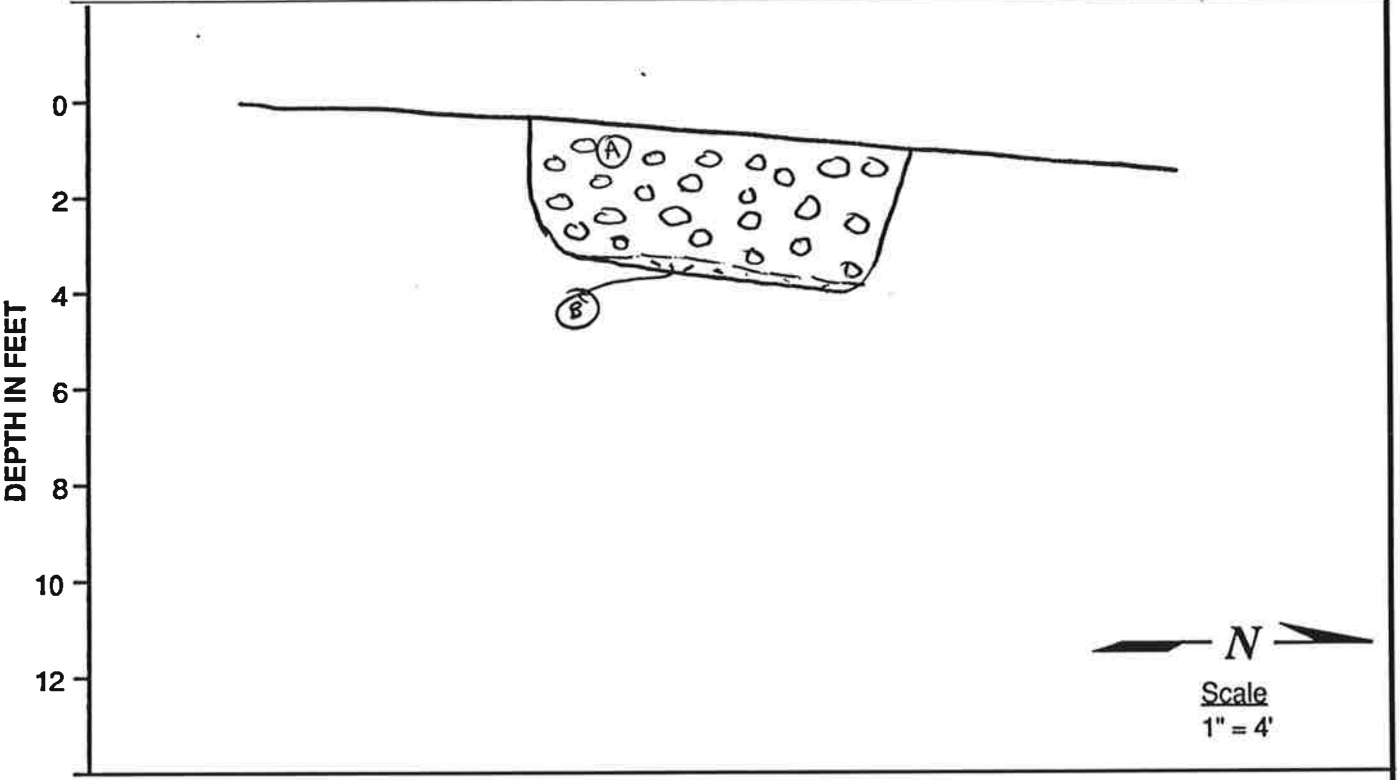
Log of Test Pit No. TP-7

ELEVATION: 1064.1 ft.

Date 2/17/95

Approved By ADG

Logged By JMH



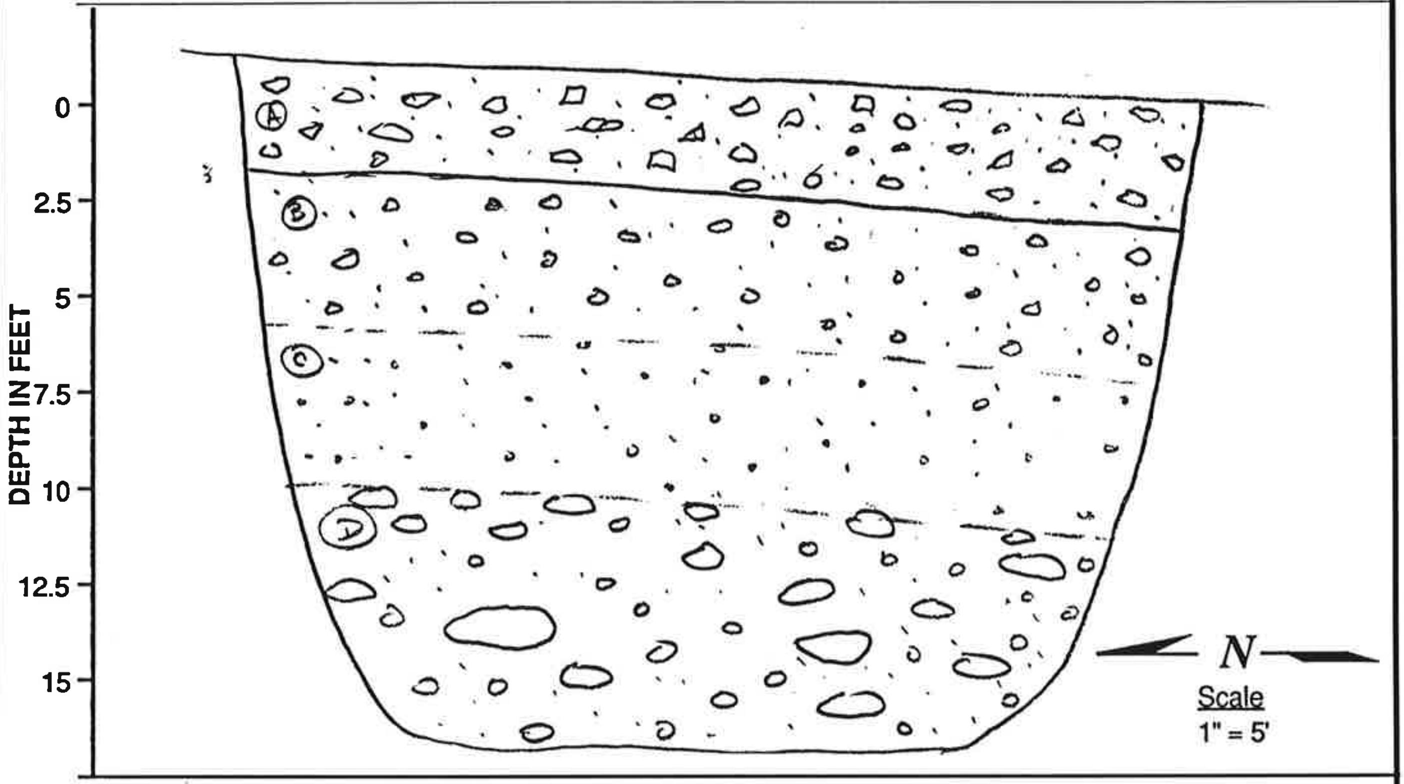
SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Bedrock
		0-2.5		Ⓐ Decomposed Granite - damp, moderate yellowish brown (10YR 5/4), very soft, round remnants of granite bedrock
		2.5-3		Ⓑ Granite - highly weathered
				TD @ 3.5 feet, no water

DATE EXCAVATED: 10/21/94

EQUIPMENT: Cat EL 200 B

Log of Test Pit No. TP-8

ELEVATION: 988.2 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-3		Ⓐ Silty SAND (SM) - damp, pale yellowish brown (10YR 6/2), some subangular to subround fine to coarse gravel -3" = 60% +3" = 40% (most <6") +12" = <1% Maximum clast diameter = 12"
				Terrace Deposits
		3-7		Ⓑ SAND (SW) - moist, moderate yellowish brown (10YR 5/4), some fine to medium gravel, subround
		7-11		Ⓒ As in Ⓑ, fine gravel, subround
		11-18		Ⓓ Gravelly SAND (SW) - fine gravel to boulders -3" = 70% +3" = 25% +12" = 5% Maximum clast diameter = 3' TD @ 18 feet, water @ 18 feet

Logged By JMH

Approved By ADG

Date 2/17/95

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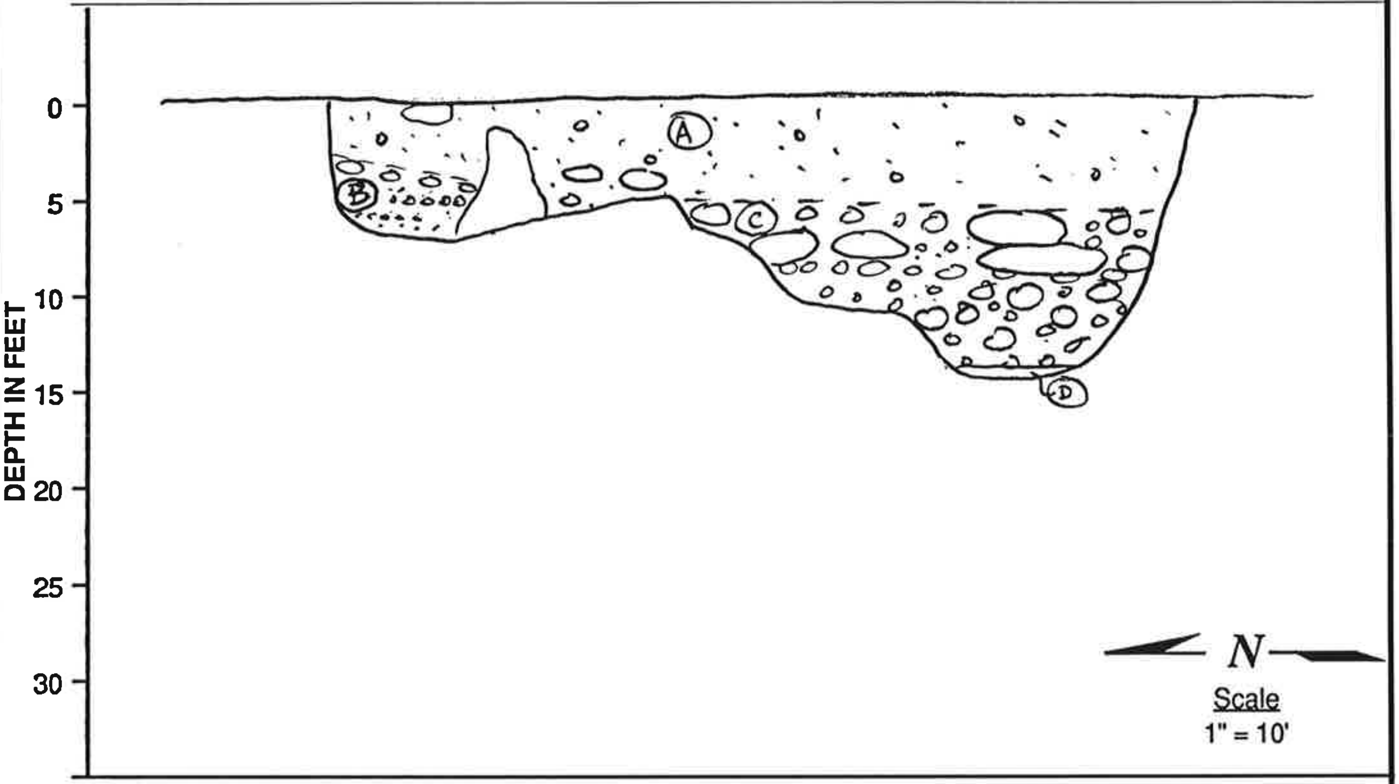
DATE EXCAVATED: 10/21/94

EQUIPMENT: Cat EL 200 B

Log of Test Pit No. TP-9

ELEVATION: 991.5 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Terrace Deposits
		0-6		Ⓐ SAND (SP) - damp, moderate yellowish brown (10YR 5/4), trace silt, trace coarse sand to fine gravel
		3-6		Ⓑ SAND (SW) - damp, light brownish gray (5YR 6/1), fine to coarse sand, imbricate cobbles on upper surface, subround, laminated
		6-14		Ⓒ Sandy GRAVEL (GP/GW) - crudely bedded, subround -3" = 30% +3" = 50% +12" = 20% Maximum clast diameter = 3'
				Bedrock
		14-14.5		Ⓓ Metasediments, light brown (5YR 6/4), medium hard
				TD @ 14.5 feet, no water

Date 2/17/95

Approved By ADG

Logged By JMH

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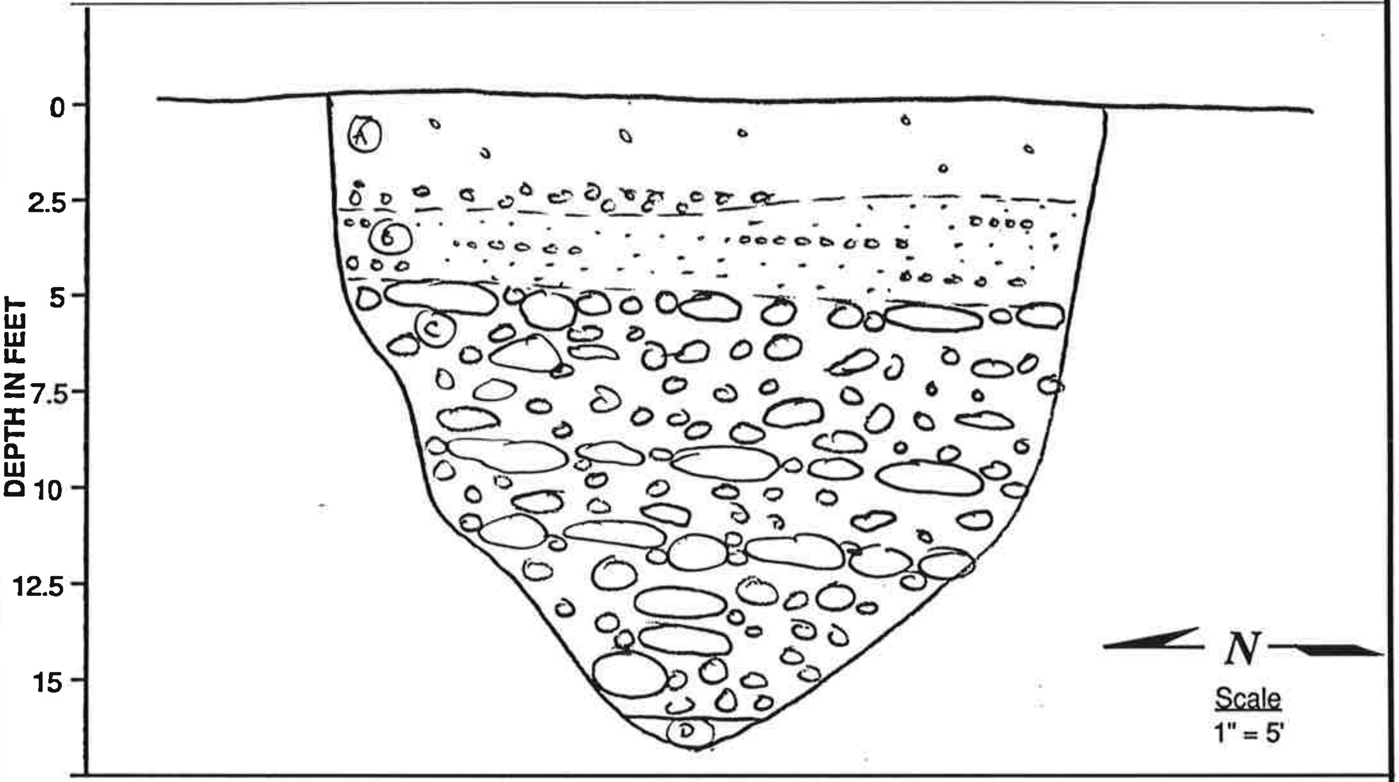
FLD-6 11/9/87

DATE EXCAVATED: 10/21/94

Log of Test Pit No. TP-10

EQUIPMENT: Cat EL 200 B

ELEVATION: 993.5 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Terrace Deposits
		0-3		Ⓐ Silty SAND (SM) - moderate yellowish brown (10YR 5/4), fine sand, <10% fine gravel in upper portion, 25% fine to coarse gravel near base, subround
		3-6		Ⓑ SAND (SW) - moist, light brownish gray (5YR 6/1), fine to medium gravel (25 %), subround, bedded
TP-10	SACKS (4)	6-16		Ⓒ Sandy GRAVEL (GP/GW) - fine to coarse gravel, boulders, subround, crudely bedded -3" = 25% +3" = 60% +12" = 15% Maximum clast diameter = 3'
				Bedrock
		16-17		Ⓓ Metasediments, light brown (5YR 6/4), medium hard
				TD @ 17 feet, no water

Date 2/17/95

Approved By ADG

Logged By JMH

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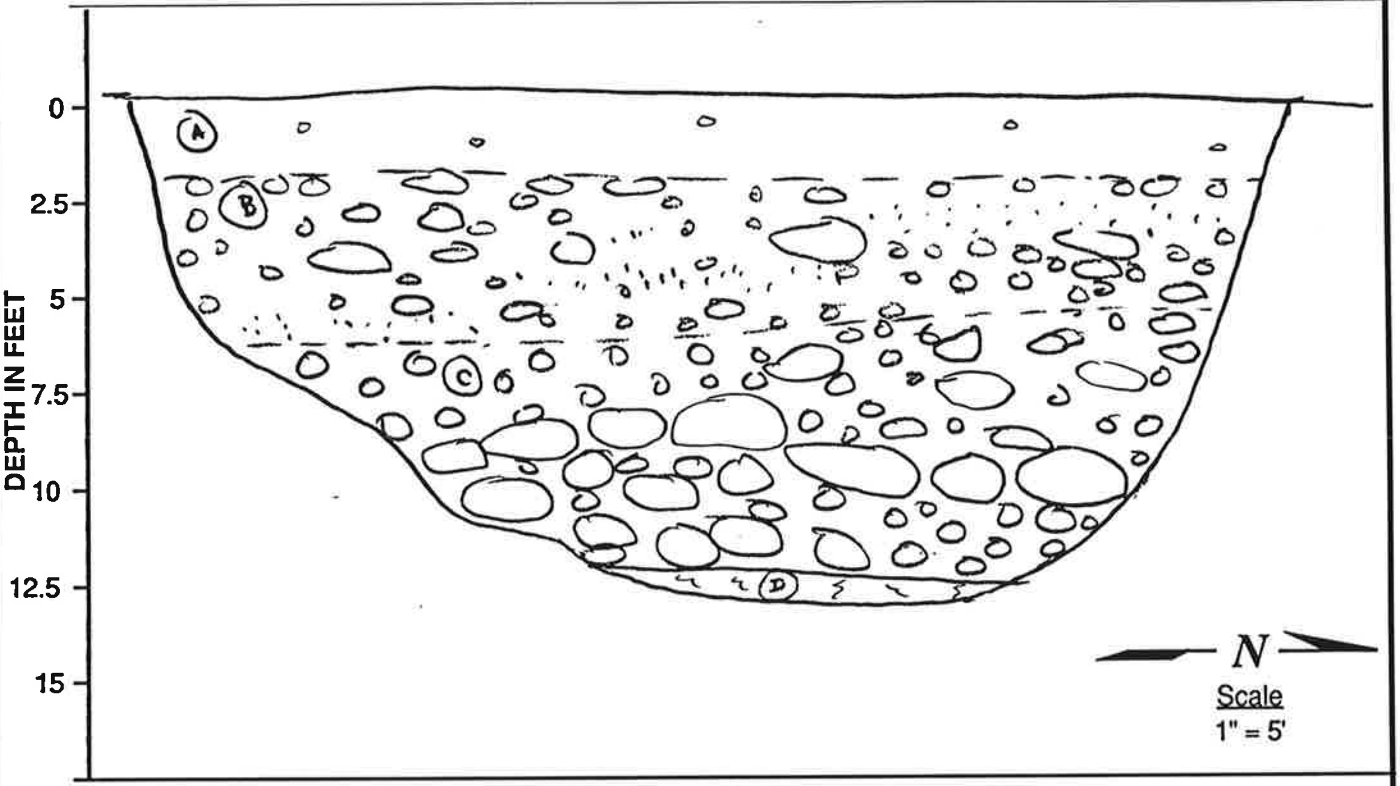
FLD-6 11/9/87

DATE EXCAVATED: 10/21/94

Log of Test Pit No. TP-11

EQUIPMENT: Cat EL 200 B

ELEVATION: 992.2 ft.



Logged By JMH

Approved By ADG

Date 2/17/95

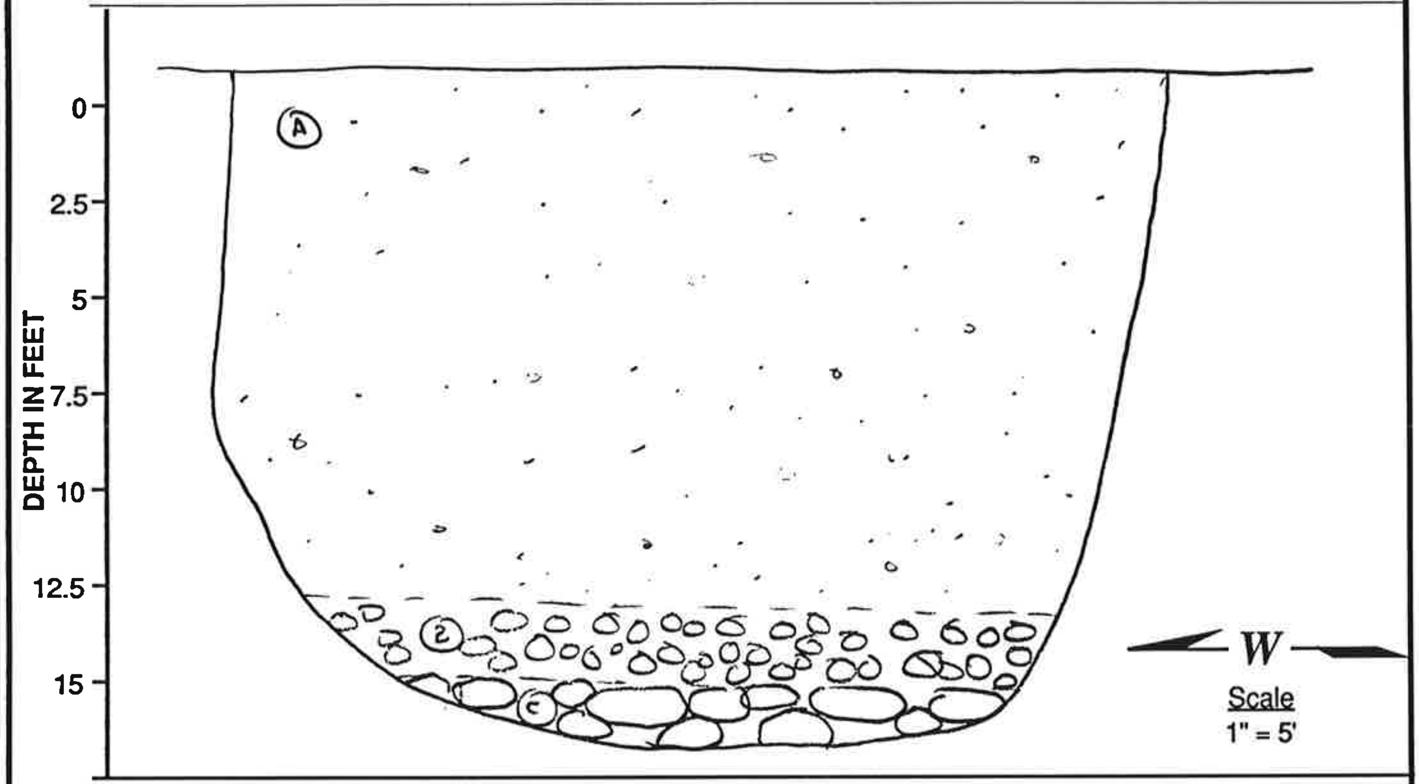
SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Terrace Deposits
TP-11	Sack (4)	0-2		Ⓐ Silty SAND (SM) - moderate yellowish brown (10YR 5/4), trace clay, fine and coarse sand, trace medium gravel
		2-6.5		Ⓑ Gravelly SAND (SW) - damp to moist,pale yellowish brown (10YR 6/2), crudely bedded, lenses of fine to medium sand (SP) -3" = 55% +3" = 40% +12" = 5% Maximum clast diameter = 2'
		6.5-12.5		Ⓒ Sandy GRAVEL (GP/GW) - crudely bedded -3" = 20% +3" = 30% +12" = 50% Maximum clast diameter = 4'
				Bedrock
		12.5-13		Ⓓ Metasediments, light brown (5YR 6/4), medium hard TD @ 13 feet, no water

DATE EXCAVATED: 10/21/94

EQUIPMENT: Cat EL 200 B

Log of Test Pit No. TP-12

ELEVATION: 966.6 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
TP-12	Sack (1)	0-14		Ⓐ SAND (SP) - damp, pale yellowish brown (10YR 6/2), trace silt, trace fine gravel, angular
				Terrace Deposits
TP-12	Sacks (4)	14-16		Ⓑ Sandy GRAVEL (GW) - moist, moderate yellowish brown (10YR 5/4), fine to coarse gravel, subround -3" = 45% +3" = 50% +12" = 5% Maximum clast diameter = 18"
		16-18		Ⓒ Sandy GRAVEL (GP/GW) - moist, moderate yellowish brown (10YR 5/4), boulders, subround -3" = 40% +3" = 40% +12" = 20% Maximum clast diameter = 24" TD @ 18 feet, no water

Date 2/17/95

Approved By ADG

Logged By JMH

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS TEST PIT AND AT THE TIME OF EXCAVATION. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

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Monterey County, California

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SHEET 1 OF 1

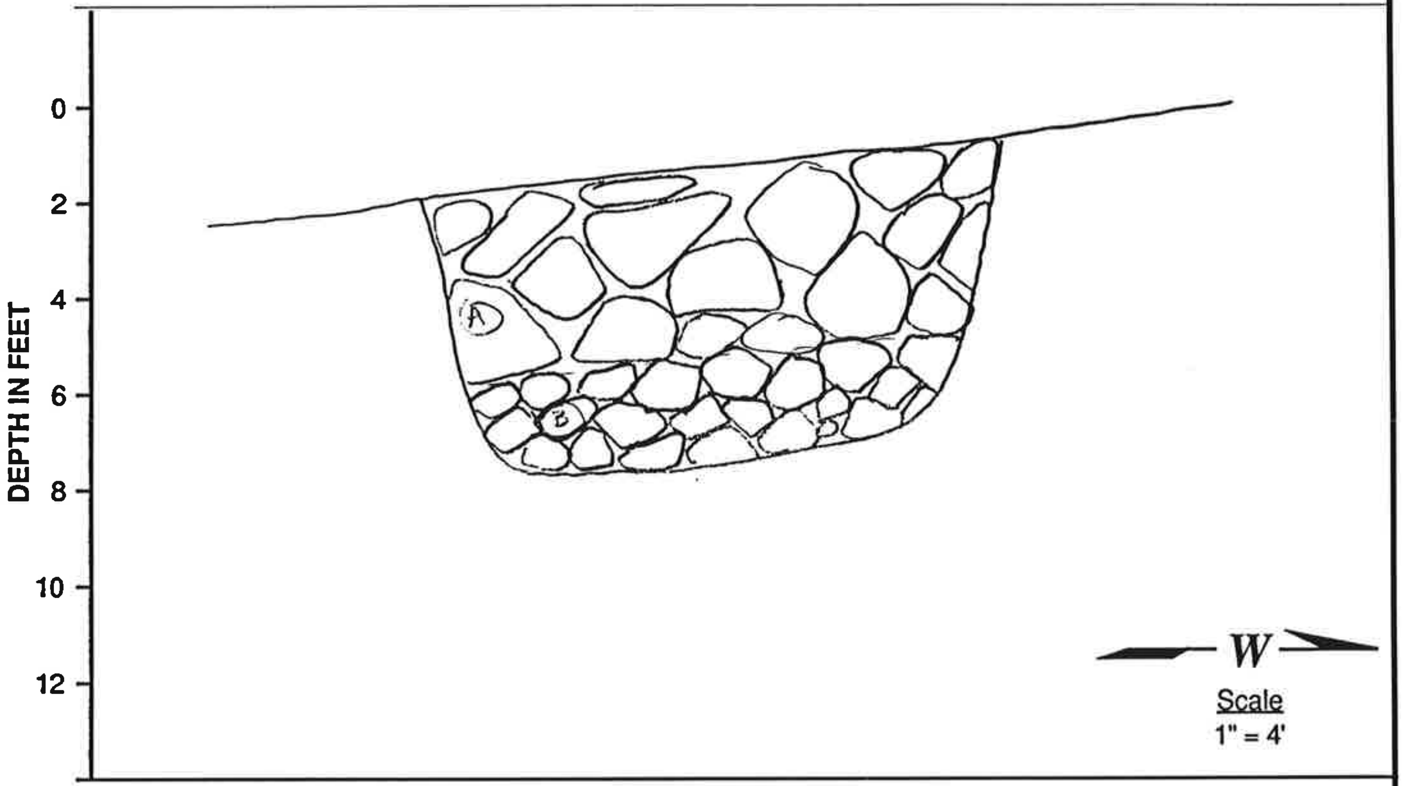
FLD-6 11/9/87

DATE EXCAVATED: 10/21/94

EQUIPMENT: Cat EL 200 B

Log of Test Pit No. TP-13

ELEVATION: 1091.5 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-4		Ⓐ Sandy GRAVEL (GP) - dry, pale yellowish brown (10YR 6/2), cobbles to boulders, angular to subangular, silty fine sand matrix, roots to 2" diameter, clasts slightly to moderately weathered
		4-6		Ⓑ As in Ⓐ but much less matrix, clasts highly weathered
				Refusal @ 6 feet, no water

Logged By JMH

Approved By ADG

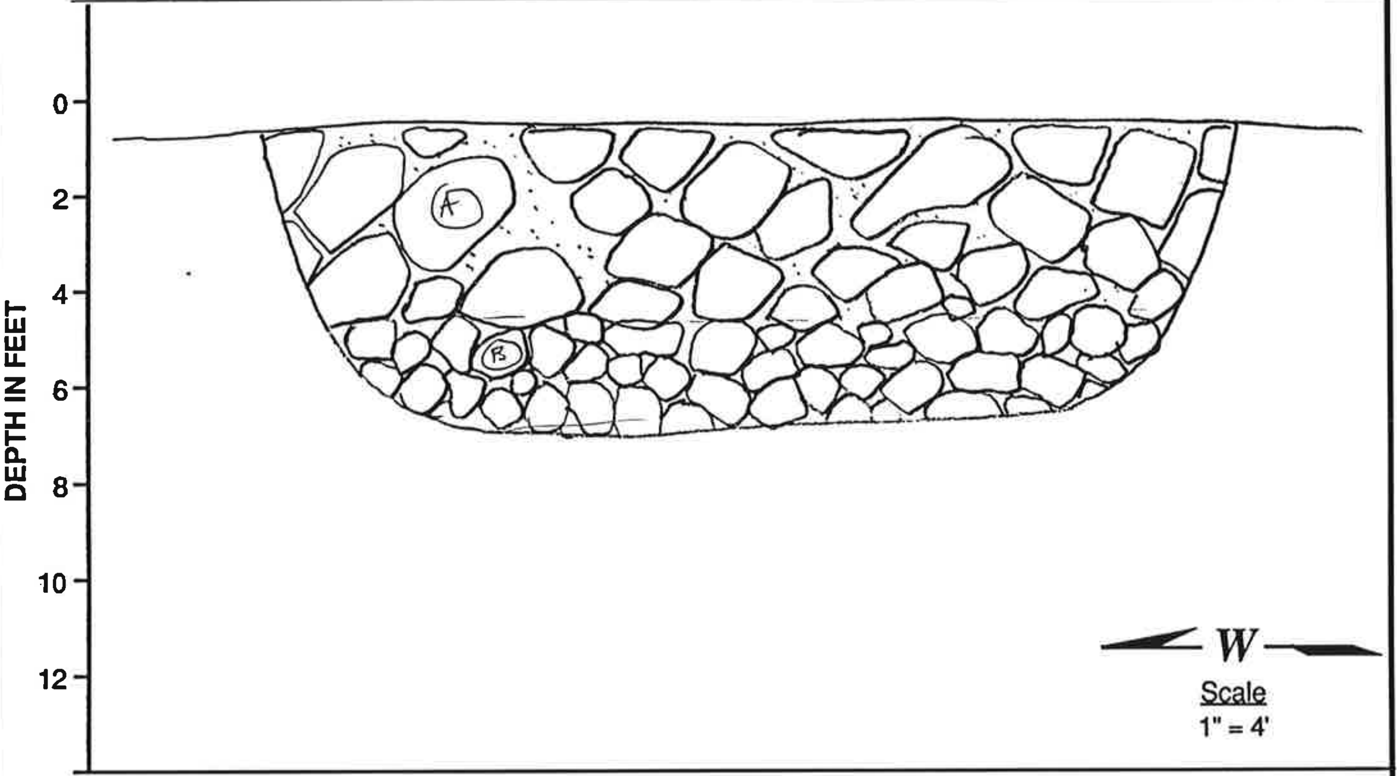
Date 2/17/95

DATE EXCAVATED: 10/21/94

EQUIPMENT: Cat EL 200 B

Log of Test Pit No. TP-14

ELEVATION: 1057.4 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
			0-4	Ⓐ Sandy GRAVEL (GP) - dry, pale yellowish brown (10YR 6/2), cobbles to boulders, angular to subangular, silty fine sand matrix
			4-6	Ⓑ As in Ⓐ but much less matrix, clasts cobble to small boulder-sized, clasts highly weathered
				Refusal @ 6 feet, no water

Date 2/17/95

Approved By ADG

Logged By JMH

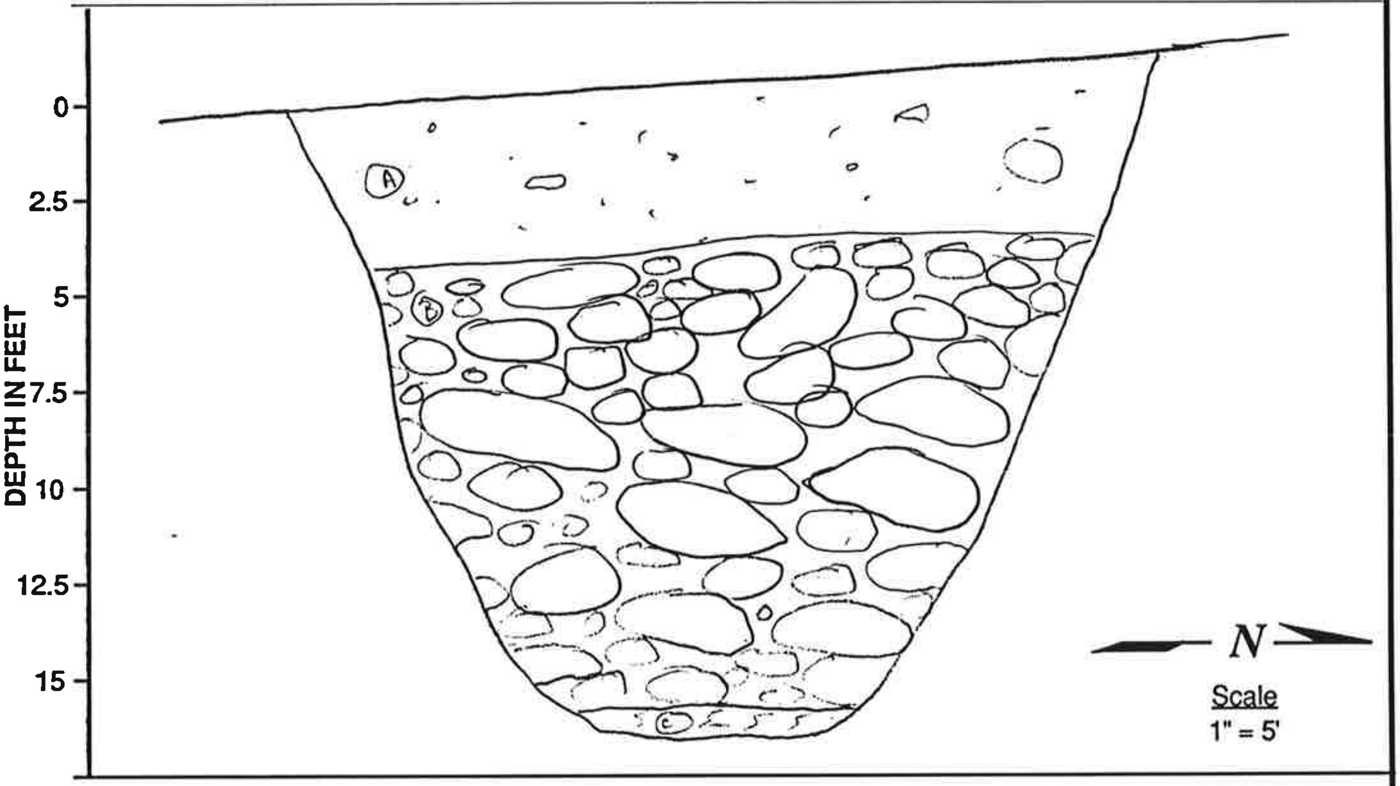
THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS TEST PIT AND AT THE TIME OF EXCAVATION. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DATE EXCAVATED: 11/18/94

EQUIPMENT: Cat 225 Excavator

Log of Test Pit No. TP-15

ELEVATION: 986.9 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
		0-4		(A) Silty SAND (SM) - damp, pale yellowish brown (10YR 6/2), trace to some gravel to small boulders, subangular to subround
				Terrace Deposits
4	Sacks	4-16		(B) Sandy GRAVEL (GP/GW) - damp to moist, sand to boulders, subround to round -3" = 10% +3" = 50% +12" = 35% +24" = 5% Maximum clast diameter = 3.5'
				Bedrock
		16-17		(C) Metasediments TD @ 17 feet, no water

Logged By JMH

Approved By ADG

Date 2/17/95

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Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.43

SHEET 1 OF 1

FLD-6 11/9/87

DATE EXCAVATED: 11/18/94

EQUIPMENT: Cat 225 Excavator

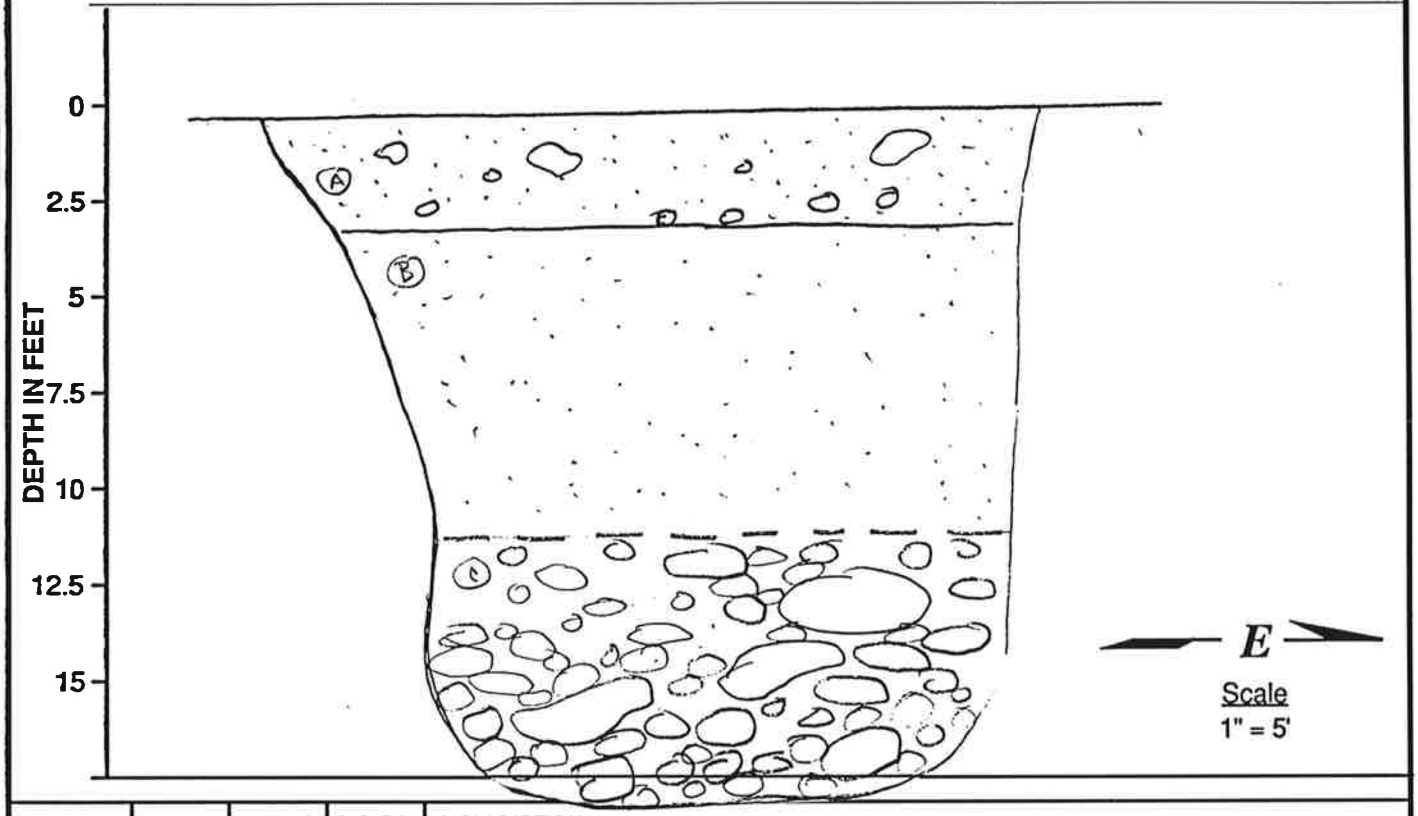
Log of Test Pit No. TP-16

ELEVATION: 982.7 ft.

Logged By JMH

Approved By ADG

Date 2/17/95



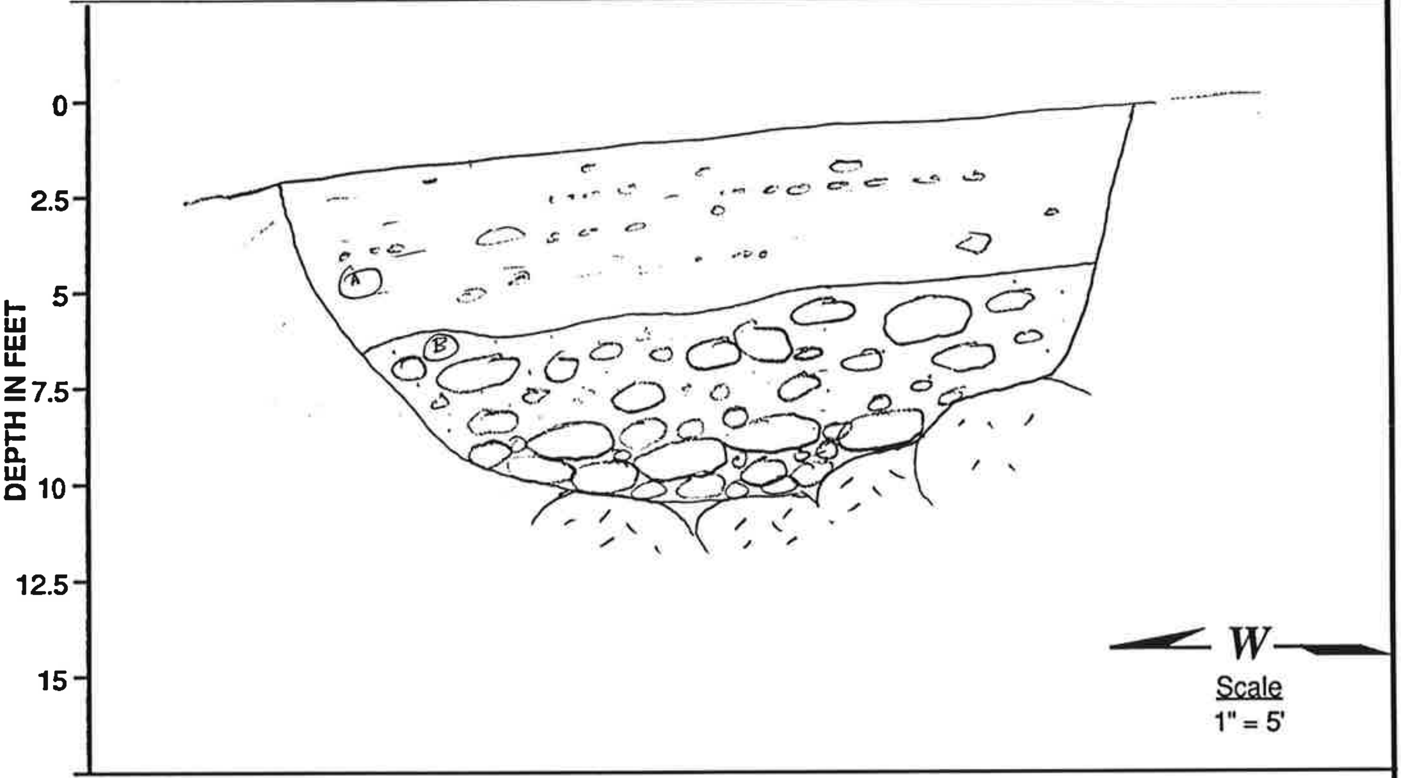
SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Fan Deposits
1	Sack	0-3		Ⓐ Silty SAND/Sandy SILT (SM/ML) - damp, pale yellowish brown (10YR 6/2), some subround to subangular cobbles to boulders
				Terrace Deposits
4	Sacks	3-11		Ⓑ SAND (SP) - damp to moist, moderate yellowish brown (10YR 5/4), fine to medium grained, trace silt
		11-18		Ⓒ Gravelly SAND/Sandy GRAVEL (GW/SW) - moist, pale yellowish brown (10YR 6/2), subround to round, clast-supported -3" = 30% +3" = 60% +12" = 10% Maximum clast diameter = 3.5' TD @ 18 feet, no water

DATE EXCAVATED: 11/19/94

EQUIPMENT: Cat 225 Excavator

Log of Test Pit No. TP-17

ELEVATION: 906.0 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Colluvium
		0-4		Ⓐ Silty SAND (SM) - damp, pale yellowish brown (10YR 6/2), fine to medium sand, some gravel to small cobble-sized clasts, angular to subangular, thinly bedded, 5% gravel/cobbles
				Terrace Deposits
4	Sacks	4-9		Ⓑ Gravelly SAND/Sandy GRAVEL (SW/GW) - trace to some silt, subangular to subround clasts, cobble-sized clasts from -4' to -6', boulders to TD -3" = 5% +3" = 50% +12" = 45% Maximum clast diameter = 4 to 5'
				Refusal on abundant large boulders @ 9 feet, no water

Logged By JMH

Approved By ADG

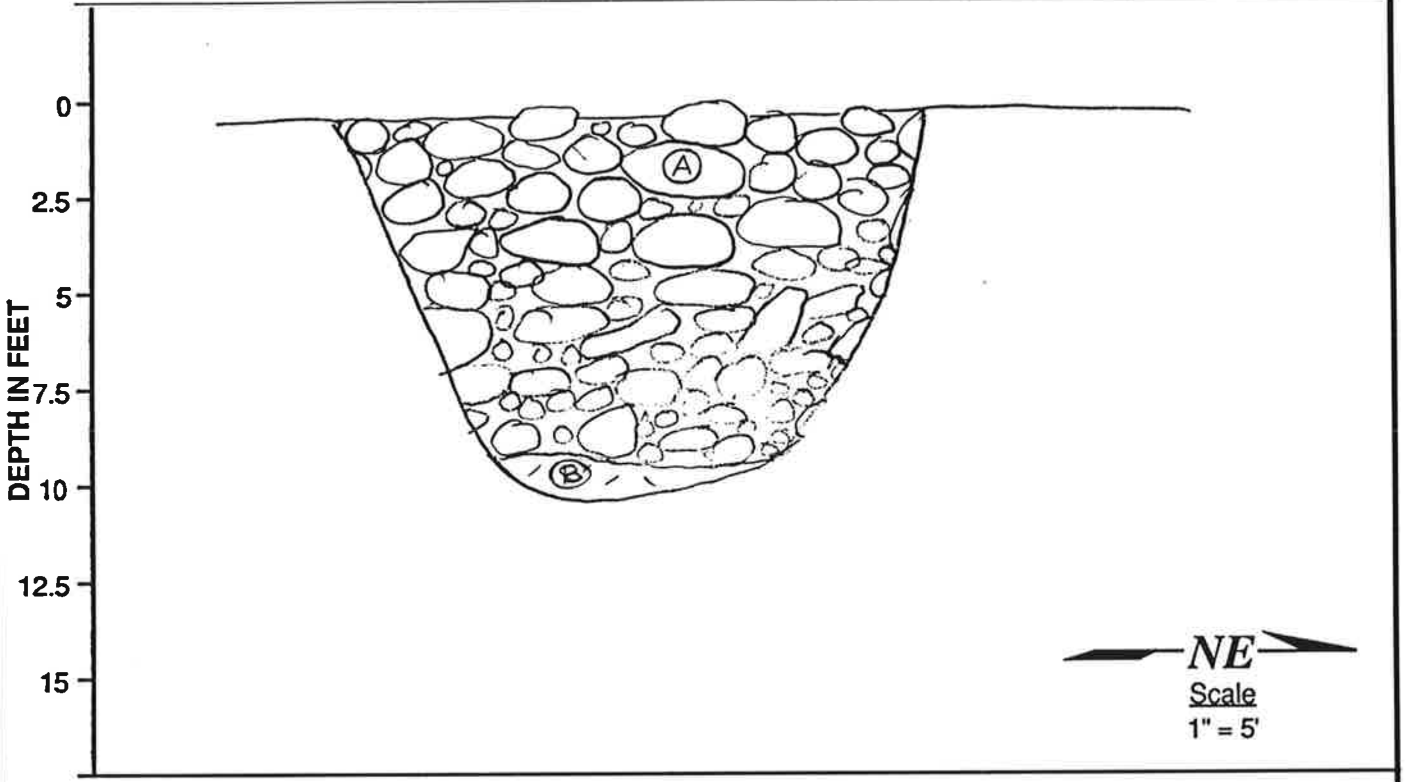
Date 2/17/95

DATE EXCAVATED: 11/19/94

EQUIPMENT: Cat 225 Excavator

Log of Test Pit No. TP-18

ELEVATION: 895.9 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Terrace Deposits
4	Sacks	0-9		Ⓐ Sandy GRAVEL (GP) - moist, pale yellowish brown (10YR 6/2), fine to medium sand, trace gravel, some cobbles, abundant small to large boulders, subround to round, clast supported -3" = 10% +3" = 20% +12" = 70% Maximum clast diameter = 3'
				Bedrock
		9-10		Ⓑ Diorite - medium dark gray (N4), fresh, very hard, 80% plagioclase, 20% mica/hornblende
				TD @ 10 feet, no water

Logged By JMH

Approved By ADG

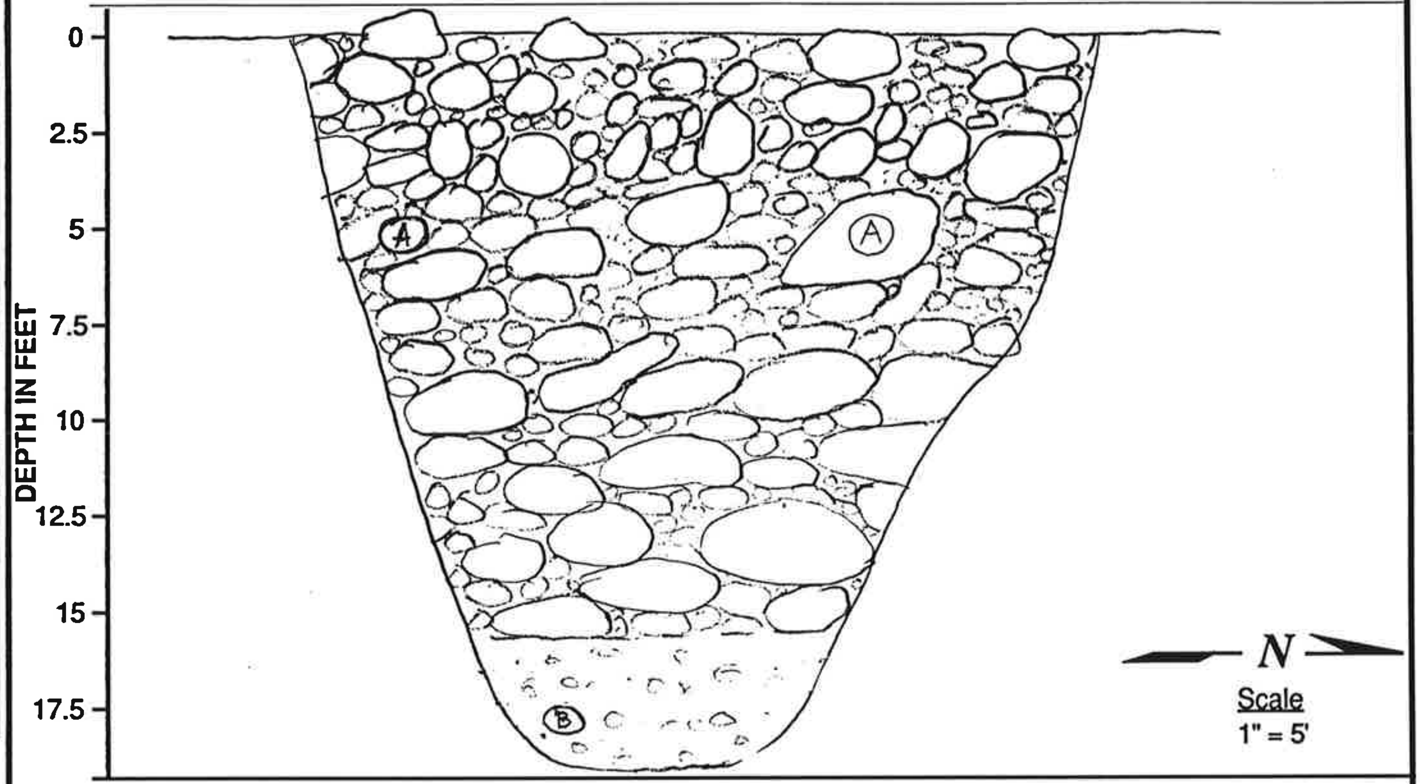
Date 2/17/95

DATE EXCAVATED: 11/19/94

EQUIPMENT: Cat 225 Excavator

Log of Test Pit No. TP-19

ELEVATION: 893.8 ft.



SAMPLE NO.	TYPE	DEPTH (feet)	TESTS	DESCRIPTION
				Terrace Deposits
		0-14		Ⓐ Sandy GRAVEL (GP) - moist, pale yellowish brown (10YR 6/2), fine to medium sand, trace gravel, some cobbles, abundant boulders, subround to round, clast supported -3" = 10% +3" = 20% +12" = 70% Maximum clast diameter = 3'
4	Sacks	14-18		Ⓑ Gravelly SAND /Sandy GRAVEL (SW/GW) - very moist, moderate yellowish brown (10YR 5/4), trace boulders -3" = 50% +3" = 45% +12" = 5% Maximum clast diameter = 12" Bedrock not encountered TD @ 18 feet, water at 18 feet

Logged By JMH

Approved By ADG

Date 2/17/95

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Geotechnical and Engineering Studies
New Los Padres Water Supply Project
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PROJECT NO.
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SHEET 1 OF 1

Appendix C

APPENDIX C

WATER PRESSURE TEST RESULTS

WATER LOSS TEST
CALCULATION SHEET

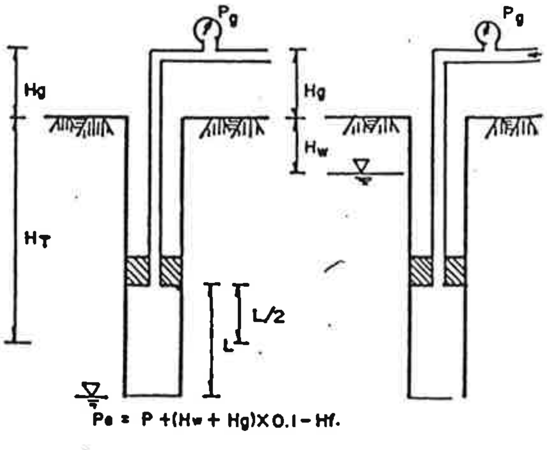
DATE: 11/9/94

HOLE NO: RA-2
TEST NO: 1
FROM 62.8' TO 73.8' FT.
PACKER TYPE: Push
HOLE DIAMETER: 3"
TEST LENGTH 11' FT.
INCLINATION: 90°
DIRECTION: —
WATER TABLE: 55'
GAUGE HEIGHT: 2.6'

1 elbow => 3.97 psi loss

STAGE	FLOW LIT./MIN.	TEST LENGTH (m)	WATER ABS. LIT/MIN/M (Q)	PRESSURE			EFFECTIVE PRESS, Pe $-H_f$ $=P_g+(H_w-H_g)X0.1$	$LU=\frac{10XQ}{P_e}$	
				GAUGE PRESS (P_g)	HYDROSTATIC $(H_w-H_g)x0.1$	FRICTION HEADLOSS(H_f)			
I	0	3.35	0	0.35	1.60	0.28	1.67	0	
II	4.2	3.35	1.25	0.70	1.60	0.29	2.01	6.22	
III	11.4	3.35	3.40	1.41	1.60	0.35	2.66	12.78	
IV	-.8	3.35	-0.24	0.7	1.60	0.28	2.02	-1.19	
V	-.8	3.35	-0.24	0.35	1.60	0.28	1.67	-1.44	

SEE BORING LOG RA-2 (SHEET 2) FOR WATER PRESSURE TEST TEST PLOT OF Pe Vs Q.



$P_e = P + (H_w + H_g) X 0.1 - H_f.$

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New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

SHEET 2 OF 4

WATER LOSS TEST
FIELD DATA SHEET

DATE: 11/9/94
HOLE NO: RA-2
TEST NO: 1
FROM 62.8' TO 73.8'
PACKER TYPE: push
HOLE DIAMETER: 3"
TEST LENGTH 11' + 2' slough
INCLINATION: vertical
GAUGE HEIGHT: 2.6
WATER TABLE: 55' bgs
HOSE Ø (mm): 1" (25-30' long)
ELBOWS No: 1 @ 1¼" ϕ

STAGE	GAUGE PRESSURE	TIME			DISCHARGE(Lit)			REMARK
		FROM	TO	minutes TOTAL	INITIAL	FINAL	TOTAL	
I	5	8:56	9:58	2	24.2	24.2	0	
	5	8:58	9:00	4	24.2	24.2	0	
	5	9:00	9:02	6	24.2	24.2	0	
	5	9:02	9:04	8	24.2	24.2	0	
	5	9:04	9:06	10	24.2	24.2	0	0
II	10	9:06	9:08	12	24.2	33.2	9.0	
	10	9:08	9:10	14	33.2	33.6	0.4	
	10	9:10	9:12	16	33.6	33.8	0.2	
	10	9:12	9:14	18	33.8	34.4	0.6	
	10	9:14	9:16	20	34.4	35.1	0.7	10.9 gal ⇒ 1.1 gpm (4.2 l/min)
III	20	9:16	9:18	22	35.1	41.5	6.4	
	20	9:18	9:20	24	41.5	43.5	2.0	
	20	9:20	9:22	26	43.5	45.0	1.5	
	20	9:22	9:24	28	45.0	46.4	1.4	
	20	9:24	9:26	30	46.4	48.0	1.6	12.9 ⇒ 1.3 gpm (4.92 l/min)
IV	10	9:26	9:28	32	48.0	46.6	-1.4	
	10	9:28	9:30	34	46.6	46.6	0	
	10	9:30	9:32	36	46.6	46.7	0.1	
	10	9:32	9:34	38	46.7	47.3	0.6	
	10	9:34	9:36	40	47.3	47.8	0.5	-0.2 gal ⇒ -0.02 gpm (-.76 l/min)
V	5	9:36	9:38	42	47.8	47.6	-0.2	
	5	9:38	9:40	44	47.6	47.6	0	
	5	9:40	9:42	46	47.6	47.6	0	
	5	9:42	9:44	48	47.6	47.6	0	
	5	9:44	9:46	50	47.6	47.6	0	-0.2 ⇒ -0.02 gpm (-0.76 l/min)

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DATE

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Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

SHEET 1 OF 4

WATER LOSS TEST
FIELD DATA SHEET

DATE: 11/9/94

HOLE NO: RA-2
TEST NO: 2
FROM 74' TO 85'
PACKER TYPE: Push
HOLE DIAMETER: 3"
TEST LENGTH 11'
INCLINATION: vertical
GAUGE HEIGHT: 2.6
WATER TABLE: 55bgs
HOSE Ø (mm): 1"
ELBOWS No: 1 @1¼" Ø

STAGE	GAUGE PRESSURE	TIME			DISCHARGE(Lit)			REMARK
		FROM	TO	TOTAL	INITIAL	FINAL	TOTAL	
I	5	3:30	3:32	2	48.3	48.6	0.3	
	5	3:32	3:34	4	48.6	49.5	0.9	
	5	3:34	3:36	6	49.5	49.9	0.4	
	5	3:36	3:38	8	49.9	50.4	0.5	
	5	3:38	3:40	10	50.4	50.6	0.2	2.3gal ⇒ 0.23gpm (0.87ℓ/min
II	10	3:40	3:42	12	50.6	53.9	3.30	
	10	3:42	3:44	14	53.9	54.4	0.5	
	10	3:44	3:46	16	54.4	55.0	0.6	
	10	3:46	3:48	18	55.0	55.5	0.5	
	10	3:48	3:50	20	55.5	56.0	0.5	5.4g ⇒ 0.54gpm (2.04ℓ/min)
III	20	3:50	3:52	22	56.0	58.9	2.9	
	20	3:52	3:54	24	58.9	60.6	1.7	
	20	3:54	3:56	26	61.6	62.5	1.9	
	20	3:56	3:58	28	62.5	62.5	0	
	20	3:58	4:00	30	62.5	63.7	1.2	7.7gal ⇒ 0.77gpm (2.9ℓ/min)
IV	10	4:00	4:02	32	63.7	61.8	-1.9	
	10	4:02	4:04	34	61.8	62.8	1.0	
	10	4:04	4:06	36	62.8	63.1	0.3	
	10	4:06	4:08	38	63.1	64.3	1.2	
	10	4:08	4:10	40	64.3	65.3	1.0	1.6 ⇒ 0.16gpm (0.61ℓ/min)
V	5	4:10	4:12	42	65.3	65.2	-0.1	
	5	4:12	4:14	44	65.2	65.3	0.1	
	5	4:14	4:16	46	65.3	65.5	0.2	
	5	4:16	4:18	48	65.5	65.8	0.3	
	5	4:18	4:20	50	65.8	66.1	0.3	0.8gal ⇒ 0.08gpm (0.3ℓ/min)

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Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

WATER LOSS TEST
CALCULATION SHEET

DATE : 11/9/94

HOLE NO: RA-2 TEST NO: 2 FROM 74' TO 85' ft.

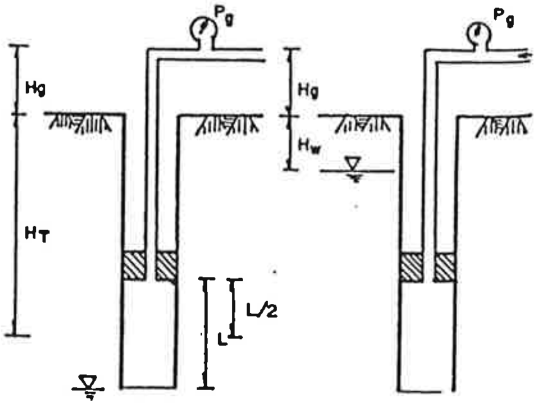
PACKER TYPE: Push HOLE DIAMETER: 3" TEST LENGTH 11' ft.

INCLINATION: Vert DIRECTION : —

WATER TABLE: 55 GAUGE HEIGHT: 2.6'

	FLOW LIT./MIN.	TEST LENGTH	WATER ABS. LIT/MIN/M (Q)	PRESSURE			EFFECTIVE PRESS, $P_e - H_f$ $= P_g + (H_w - H_g) \times 0.1$	$LU = \frac{10 \times Q}{P_e}$	
				GAUGE PRESS (P_g)	HYDROSTATIC $(H_w - H_g) \times 0.1$	FRICTION HEADLOSS(H_f)			
I	0.87	3.35	0.26	0.35	1.60	0.28	1.67	1.56	
II	2.04	3.35	0.61	0.70	1.60	0.28	2.02	3.02	
III	2.91	3.35	0.87	1.41	1.60	0.28	2.73	3.19	
IV	0.61	3.35	0.18	0.70	1.60	0.28	2.02	0.89	
V	0.3	3.35	0.09	0.35	1.60	0.28	1.67	0.54	

SEE BORING LOG RA-2 (SHEET 2) FOR WATER PRESSURE TEST TEST PLOT OF Pe Vs Q.



$P_e = P + (H_w + H_g) \times 0.1 - H_f$.

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New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

SHEET 4 OF 4

Appendix D

APPENDIX D

LABORATORY TEST DATA

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-2 Report date : 11/10/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :
Mat'l Descr :brown gravelly silty sand
Location :TP-3
Sampled by :others
Date received :11/8/94

===

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
5	100
4	100
3"	100
2 1/2"	92
2	91
1 1/2"	87
1"	79
3/4"	75
1/2"	69
3/8"	65
#4	57
#8	50
#16	44
#30	37
#50	28
#100	18
#200	11

Moisture Content (D-2216) 4.4%

Warren Benson

2cc: The Mark Group

Reviewed by Warren Benson

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Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0318-1 Report date : 12/5/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project
Date sampled :
Mat'l Descr :brown gravelly sand
Location :TP-9
Sampled by :client
Date received :11/21/94

===

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4"	100
3 1/2"	96
3"	92
2 1/2"	88
2	86
1 1/2"	78
1"	72
3/4"	67
1/2"	63
3/8"	59
#4	53
#8	46
#16	38
#30	28
#50	16
#100	9
#200	5

Warren Benson

2cc: The Mark Group

Reviewed by Warren Benson

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No.	:L0305-3	Report date : 11/18/94
Project No.	:35787	
Client	:The Mark Group	
Project	:New Los Padres Dam Project-Task 61 & 63	
Date sampled	:not reported	
Mat'l Descr	:	
Location	:TP-11 3/4	
Sampled by	:others	
Date received	:11/8/94	

===

Specific Gravity - Coarse Aggregate (C-127)

	Gr/CC	PCF
Bk Sp Gr (OD)	2.506	156.4
Bk Sp Gr (SSD)	2.571	160.5
Bk Sp Gr (App)	2.681	167.3

Absorption % 2.6

Specific Gravity - Fine Aggregate (C-128)

	Gr/CC	PCF
Bk Sp Gr (OD)	2.430	151.7
Bk Sp Gr (SSD)	2.515	157.0
Bk Sp Gr (App)	2.657	165.9

Absorption % 3.5

Los Angeles Rattler (C-131) (insufficient large agg for C-535)

% loss @ 100 rev	18.1%	
% loss @ 500 rev	47.0%	('A' Grading)

Moisture Content (D-2216) 4.3%

Warren Benson

2cc: The Mark Group

Reviewed by Warren Benson

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-5 Report date : 12/5/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :
Mat'l Descr :brown gravelly silty sand
Location :TP-11 3/4
Sampled by :others
Date received :11/8/94

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4	100
3 1/2"	93
3"	89
2 1/2"	83
2	79
1 1/2"	75
1"	72
3/4"	68
1/2"	64
3/8"	61
#4	53
#8	48
#16	39
#30	27
#50	16
#100	8
#200	4

Sodium Sulfate Soundness (C-88)

Weighted Avg Loss of Coarse Aggregate = 3.7%

Weighted Avg Loss of Fine Aggregate = 5.3%

Moisture Content (D-2216) = 4.5%

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REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-7 Report date : 11/18/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :
Mat'l Descr :brown gravelly silty sand
Location :TP-12 Fan 1/1
Sampled by :others
Date received :11/8/94

==

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4	100
3 1/2"	100
3"	100
2 1/2"	100
2	100
1 1/2"	100
1"	99
3/4"	99
1/2"	99
3/8"	98
#4	94
#8	85
#16	71
#30	56
#50	42
#100	29
#200	20

Moisture Content (D-2216) 2.7%

Warren Benson
Reviewed by Warren Benson

2cc: The Mark Group

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-8 Report date : 11/10/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :
Mat'l Descr :brown sandy gravel
Location :TP-12 Terrace 1/4
Sampled by :others
Date received :11/8/94

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4	100
3 1/2"	100
3"	94
2 1/2"	86
2	75
1 1/2"	69
1"	62
3/4"	56
1/2"	51
3/8"	47
#4	38
#8	37
#16	28
#30	18
#50	10
#100	4
#200	2

Moisture Content (D-2216) 2.6%

Warren Benson

2cc: The Mark Group

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Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-8 Report date : 12/16/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :
Mat'l Descr :brown sandy gravel
Location :TP-12 Terrace 1/4
Sampled by :others
Date received :11/8/94

Specific Gravity - Coarse Aggregate (C-127)

	Gr/CC	PCF
Bk Sp Gr (OD)	2.527	157.7
Bk Sp Gr (SSD)	2.597	162.1
Bk Sp Gr (App)	2.717	169.6

Absorption % 2.8

Specific Gravity - Fine Aggregate (C-128)

	Gr/CC	PCF
Bk Sp Gr (OD)	2.543	158.8
Bk Sp Gr (SSD)	2.606	162.7
Bk Sp Gr (App)	2.713	169.3

Absorption % 2.5

Sodium Sulfate Soundness (C-88)

Weighted Avg Loss of Coarse Aggregate = 7.4%

Weighted Avg Loss of Fine Aggregate = 7.6%

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Reviewed by Warren Benson

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REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-9 Report date : 11/18/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :not reported
Mat'l Descr :
Location :TP-12 Terrace 2/4
Sampled by :others
Date received :11/8/94

Los Angeles Rattler (C-131) (insufficient large agg for C-535)

% loss @ 100 rev	13.4%	
% loss @ 500 rev	51.3%	('A' Grading)

Warren Benson
Reviewed by Warren Benson

2cc: The Mark Group

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0318-2 Report date : 12/5/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project
Date sampled :
Mat'l Descr :brown gravelly sand
Location :TP-16 QT
Sampled by :client
Date received :11/21/94

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4"	100
3 1/2"	96
3"	96
2 1/2"	95
2	92
1 1/2"	86
1"	77
3/4"	71
1/2"	65
3/8"	60
#4	51
#8	47
#16	41
#30	32
#50	19
#100	9
#200	4

Warren Benson
Reviewed by Warren Benson

2cc: The Mark Group

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0318-3 Report date : 12/5/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project
Date sampled :
Mat'l Descr :brown gravelly sand
Location :TP-18
Sampled by :client
Date received :11/21/94

===

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4"	100
3 1/2"	100
3"	98
2 1/2"	93
2	88
1 1/2"	86
1"	81
3/4"	77
1/2"	75
3/8"	73
#4	71
#8	69
#16	64
#30	49
#50	27
#100	10
#200	3

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REPORT OF AGGREGATE TESTS

Laboratory No. :L0318-4 Report date : 12/5/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project
Date sampled :
Mat'l Descr :brown gravelly sand
Location :TP-19
Sampled by :client
Date received :11/21/94

===

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
4"	100
3 1/2"	93
3"	93
2 1/2"	85
2	82
1 1/2"	78
1"	72
3/4"	66
1/2"	61
3/8"	57
#4	49
#8	43
#16	33
#30	22
#50	12
#100	5
#200	2

Warren Benson
Reviewed by Warren Benson

2cc: The Mark Group

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Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0373 Report date : 2/10/95
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :not reported
Mat'l Descr :brown gravelly sand
Location :TP-19 Qt
Sampled by :others
Date received :11/9/94

Los Angeles Rattler (C-131) (insufficient large agg for C-535)

% loss @ 100 rev	16.7%	
% loss @ 500 rev	47.4%	('A' Grading)

Warren Benson

Reviewed by Warren Benson

2cc: The Mark Group

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-12 Report date : 11/10/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :not reported
Mat'l Descr :crushed granite rock and fines
Location :crushed rock, box samples
Sampled by :others
Date received :11/8/94

Grading Analysis (C-136)

Sieve Size	Pct. Pass.
5	100
4	100
3"	100
2 1/2"	100
2	100
1 1/2"	100
1"	100
3/4"	95
1/2"	74
3/8"	62
#4	43
#8	32
#16	24
#30	17
#50	11
#100	7
#200	4

Los Angeles Rattler (C-131) (insufficient large agg for C-535)

% loss @ 100 rev	13.6%
% loss @ 500 rev	52.2%

Moisture Content (D-2216) 3.0%

Warren Benson

2cc: The Mark Group Reviewed by Warren Benson

Quality Assurance Services
Materials Consulting

Testing Engineers, Inc.

REPORT OF AGGREGATE TESTS

Laboratory No. :L0305-12 Report date : 11/18/94
Project No. :35787
Client :The Mark Group
Project :New Los Padres Dam Project-Task 61 & 63
Date sampled :not reported
Mat'l Descr :crushed granite rock and fines
Location :crushed rock, box samples
Sampled by :others
Date received :11/8/94

Specific Gravity - Coarse Aggregate (C-127)

	Gr/CC	PCF
Bk Sp Gr (OD)	2.649	165.4
Bk Sp Gr (SSD)	2.692	168.0
Bk Sp Gr (App)	2.767	172.7
Absorption %	1.6	

Specific Gravity - Fine Aggregate (C-128)

	Gr/CC	PCF
Bk Sp Gr (OD)	2.459	153.5
Bk Sp Gr (SSD)	2.546	158.9
Bk Sp Gr (App)	2.692	168.1
Absorption %	3.5	

Sodium Sulfate Soundness (C-88)

Weighted Avg Loss of Coarse Aggregate = 8.8%

Weighted Avg Loss of Fine Aggregate = 5.4%

Warren Benson

2cc: The Mark Group

Reviewed by Warren Benson

827 Arnold Drive, Bay 4 • Martinez, California 94553 • (510) 370-7002
SANTA CLARA • DIABLO VALLEY

Testing Engineers, Inc.

Quality Assurance Services
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RECEIVED
FEB 22 1995
THE MARK GROUP

February 14, 1995

Project No. 35787
Laboratory No. K0341

The Mark Group, Inc.
Engineers & Geologists
3480 Burkirk Avenue, Suite 120
Pleasant Hill, CA 94523

Attn: Mr. Kenneth King

Project: NEW LOS PADRES DAM - TASK 61 AND 63,
MONTEREY PENINSULA WATER - MANAGEMENT DISTRICT, CA

Subject: Report of LA Rattler Test per ASTM C-131
Sample ID No.: TP-19-BRN SANDY GRAVEL

<u>RESULTS:</u>

Percent Loss at 100 Revolutions	16.7
Percent Loss at 500 Revolutions	47.4

Respectfully submitted,

TESTING ENGINEERS, INCORPORATED

Young Davis

Young Davis, Supervisor
Oakland Structural Department

2811 Adeline Street, P.O. Box 24075 • Oakland California 94623 • (510) 835-3142

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Testing Engineers, Inc.

Quality Assurance Services
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RECEIVED
FEB 22 1995
THE MARK GROUP

February 9, 1995

Project No. 35787
Laboratory No. K0340

The Mark Group, Inc.
Engineers & Geologists
3480 Burkirk Avenue, Suite 120
Pleasant Hill, CA 94523

Attn: Mr. Kenneth King

Project: NEW LOS PADRES DAM - TASK 61 AND 63, INTERIM REPORT

Subject: Report of Potential Alkali Reactivity Test of Cement - Aggregate Combinations as per ASTM C-227, Mortar-Bar Method.

RESULTS:

Sample Identification Number	Percentage (%) Expansion At:	
	14 Days	30 Days (1 Month)
TP - 11 BRN. Gravelly Silty Sand	+0.007	+0.008
TP - 12 (2) Terrace Gravel	+0.004	+0.007
Crushed Granite Rock	+0.006	+0.008

Note: The next readings are scheduled for (3 months) April 6, 1995.

Should you have any further questions, please contact the undersigned at (51)0 419-1103.

Respectfully submitted,

TESTING ENGINEERS, INCORPORATED

Young Davis, Supervisor
Oakland Structural Department

Enclosure - Copy of ASTM C-33 (FYI)

2811 Adeline Street, P.O. Box 24075 • Oakland California 94623 • (510) 835-3142

SANTA CLARA • DIABLO VALLEY

File No. 35787

Lab. No. ______

TEI

Mix Design ______

Date Cast 1-9-95

Slump ______

Beam Size 1X1X11

Period of Moist Cure ______

Initial Rod Reading ______

Temperature °F ______

Relative Humidity ______

LABORATORY COMPRESSION TESTS

Specimen			
Age, days			
Strength, p.s.i.			

SAMPLE:
TP-11 (2 OF 2)

LINEAL SHRINKAGE

ays of rying	Date	Specimen Number						Ave. Diff	Shri
		A		B					
	1-10	0519		0537					
15	1-24	0527	+0008	0544	+0007				+0.008
1-Month	2-8	0528	+0009	0544	+0007				+0.008
3-Month	4-11								

File No. 35787

Lab. No. ______

Mix Design ______

Date Cast 1-11-95

Slump ______

Beam Size [illegible]

Period of Moist Cure ______

Initial Rod Reading ______

Temperature °F ______

Relative Humidity ______

LABORATORY COMPRESSION TESTS

Specimen			
Age, days			
Strength, p.s.i.			

Sample:
TP-11 (1 of 2)

LINEAL SHRINKAGE

~~205-4~~

ys of ying	Date	Specimen Number						Ave. Diff	Shrin
		A		b					
0	1-12	0541		0575					
14	1-23	0548	+0007	0580	+0005				+0.006
1-Month	2-10	0549	+0008	0584	+0009				+0.009
3 "	4-11								

File No. 35787

Lab. No.

Mix Design

Date Cast 1-11-95

Slump

Beam Size

Period of Moist Cure

Initial Rod Reading

Temperature °F

Relative Humidity

LABORATORY COMPRESSION TESTS

Specimen			
Age, days			
Strength, p.s.i.			

SAMPLE

TP-12(2) TERRACE GRAVEL
(2 OF 2)

LINEAL SHRINKAGE

~~305-8~~

ys of ying	Date	Specimen Number A		b				Ave. Diff	Shri
0	1-12	0555		0530					
14	1-25	0560	+0005	0535	+0005				+005
1-MONTH	2-10	0563	+0008	0538	+0008				+008
1-14	4-1								

File No. 35787

Lab. No.

Mix Design

Date Cast 1-9-95

Slump

Beam Size

Period of Moist Cure

Initial Rod Reading

Temperature °F

Relative Humidity

LABORATORY COMPRESSION TESTS

Specimen			
Age, days			
Strength, p.s.i.			

LINEAL SHRINKAGE

SAMPLE

TP-12 (2) TERRACE GRAVEL

(1 of 2)

ays of rying	Date	Specimen Number A		B				Ave. Diff	Shri
0	1-10	0555		0543					
19	1-28	0557	+0002	0548	+0005				+0004
1-Month	2-8	0558	+0003	0551	+0008				+0006
3 - 11	4-11								

File No. 35787

Lab. No. ________

Mix Design ________

Date Cast 1-9-95

Slump ________

Beam Size ________

Period of Moist Cure ________

Initial Rod Reading ________

Temperature °F ________

Relative Humidity ________

LABORATORY COMPRESSION TESTS

Specimen			
Age, days			
Strength, p.s.i.			

SAMPLE - CRUSHED GRANITE ROCK
(1 OF 2)

LINEAL SHRINKAGE

ays of rying	Date	Specimen Number						Ave. Diff	Shri
		A		B					
0	1-10	0632		0560					
14	1-23	0637	+0005	0568	+0008				+007
1-Month	2-8	0643	+0011	0567	+0007				+009
3-M	4-11								

File No. 35787

Lab. No. ______

Mix Design ______

Date Cast 1-11-95

Slump ______

Beam Size ______

Period of Moist Cure ______

Initial Rod Reading ______

Temperature °F ______

Relative Humidity ______

LABORATORY COMPRESSION TESTS

Specimen			
Age, days			
Strength, p.s.i.			

SAMPLE- CRUSHED GRANITE ROCK
(2 OF 2)

LINEAL SHRINKAGE

~~305-75~~

ys of ying	Date	Specimen Number						Ave. Diff	Shri
		A		b					
0	1-12	0614		0543					
14	1-25	0620	+0006	0549	+0006				+006
1-Month	2-9	0622	+0008	0550	+0007				+008
3 "	4-11								

TESTING ENGINEERS, INC.

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Oakland • *Santa Clara* • *Diablo Valley* • *Monterey*

NEW LOS PADRES PROJECT
MONTEREY PENINSULA
WATER MANAGEMENT DISTRICT

90 DAY SPLITTING TENSILE TEST/ UNIT WEIGHT TEST

PROJECT NO. 35787
TASK NO. 63 RCC

MIX NO.	SAMPLE NO.	DIAMETER (IN.)	LENGTH (IN.)	LOAD (LBS.)	PSI	AVERAGE PSI	UNIT WT. LBS/CU. FT.
1	2	5.97	12.0	13,486	120	-	151.2
1	5	5.97	12.0	10,263	90	105	147.6
2	7	5.97	12.0	30,236	270	270	154.8
3	5	5.97	12.0	55,608	495	-	154.2
3	8	5.97	12.0	53,433	475	485	150.7
4	1	5.97	12.0	76,728	680	-	153.8
4	4	5.97	12.3	57,546	510	595	154.3
5	1	5.97	11.9	43,842	390	-	147.1
5	5	5.97	12.0	53,670	475	435	147.1
6	3	5.97	12.0	44,020	390	-	153.3
6	7	5.97	12.0	43,743	390	390	151.7

Appendix E

APPENDIX E

CACHAGUA FAULT INVESTIGATION

APPENDIX E
CACHAGUA FAULT INVESTIGATION

An extensive regional and site-specific investigation was conducted to evaluate the present level of activity of the Cachagua fault. On the basis of this work, it has been concluded that there is compelling geologic and geomorphic evidence that the Cachagua fault has not experienced fault movement since the late Pleistocene. This conclusion is based upon evidence that, elevated Quaternary stream terrace deposits along the Carmel River that cover the fault have not been offset. Furthermore, the fault trace does not exhibit geomorphic features that resemble young rift topography commonly found along active faults. Based upon rates of uplift and river incision (downcutting), it has been estimated that these stream terrace deposits are approximately 85,400 to 213,500 years old, thus placing the age of faulting prior to the Holocene, and as such, not active.

E.1 Technical Approach to Fault Evaluation

Assessment of the seismogenic potential of the Cachagua fault is based on the level of activity of the fault, the capability of the fault to generate a significant earthquake, and the size of that earthquake. The results of such an assessment are critical elements in the formulation of the project seismic design criteria. Essential to this evaluation is the careful identification of the location of the fault, the relation of fault to the geologic formations found along its trace, the age of those formations, and the regional seismotectonic setting.

The key to judging the timing of the most significant period of movement along the Cachagua fault is defining the late Tertiary and Quaternary history of the fault. The Cachagua fault juxtaposes Mesozoic and older crystalline basement rock against Tertiary sedimentary rock estimated to be as young as 3 million years old (RJA, 1984; Dibblee, 1972). Therefore, the Cachagua fault is known to have been an active seismogenic structure during the time interval that postdates the youngest Tertiary deposits.

The portion of the geologic record that has the most bearing on the assessment of the recent seismic activity of the fault is the brief historical record and, more importantly, the Quaternary record as defined by the structural relationship between the fault and the Quaternary deposits found along the Carmel River. The Quaternary period includes the Holocene Epoch (last 11,000 years) and the Pleistocene Epoch (11,000 to 2.5 million years).

As defined by the California Division of Mines and Geology, "active faults" are those that show evidence of movement since the end of the Pleistocene, while "potentially active faults" are those showing evidence that they offset Quaternary deposits.

The historic record of fault activity covers the past approximately 150 years, and normally includes the location of earthquake epicenters recorded along the mapped trace of the fault or over the down dip-projection of the fault plane, and any record of surface rupture that has taken place along the fault. Although a low level of microseismicity has been recorded in the region, no clear historical seismic activity has been associated with the Cachagua fault (Cockerham and others, 1990).

As discussed in Section 4.0 of the main body of this report, Quaternary deposits in the area are primarily of two distinct types, including stream terrace materials (Qt) deposited by the Carmel River and isolated alluvial fan deposits (Qf) formed by smaller tributaries to the Carmel River (Drawing E-1). These two types of deposits are characterized by distinct land forms and depositional histories that can be effectively used to define the Quaternary depositional history along the section of the Carmel River that traverses the project area.

To evaluate the Quaternary paleoseismic history of the Cachagua fault, stereoscopic aerial photographic interpretation, detailed engineering geologic mapping and subsurface exploration was performed to constrain the location of the fault and to focus on delineating the relationship of the fault to a set of well-defined Quaternary time horizons represented by the base of the stream terrace deposits. The Quaternary terrace deposits rest on irregular, but nearly horizontal, surfaces beveled into the underlying bedrock. These surfaces represent very significant geologic time horizons , and thus they represent stratigraphic and structural markers that hold the key to defining the late Quaternary history of the Cachagua fault. During field work conducted for this investigation, special attention was paid to defining the location and areal distribution of the stream terrace deposits and their basal horizons. Although many levels of terraces were observed, efforts were focused on the two most widespread terrace deposits. These deposits were mapped in detail along the canyon wall, downstream from the existing dam site, and across the Cachagua fault without interruption. Surveying of the elevations of the associated terrace-bedrock unconformities in a longitudinal transect across the Cachagua fault demonstrates that the horizons have not been offset by faulting. In addition, an excavation across the fault exposes the unfaulted terrace strath surface (Drawing E-2).

The following sections provide detailed discussions of regional aspects of the Cachagua fault (Section E.2); an analysis of Quaternary stream deposits (Section E.3); the location of the Cachagua fault in the study area (Section E.4); and an assessment of the late Quaternary history of the fault (Section E.5). These sections refer to the Engineering Geologic Map and Engineering Geologic Cross Sections (Drawing 4-1 and 4-2, respectively) that are included with the main body of the report, and are not included in this appendix.

E.2 Regional Overview

E.2.1 Geologic Setting

The geomorphology of region was examined by interpretation of stereoscopic aerial photographs taken in 1939, 1971, and 1987, and through geologic reconnaissance. Regionally, the Cachagua fault is clearly defined by the distribution of contrasting rock types (i.e., crystalline basement rocks juxtaposed against Tertiary sedimentary rocks), Drawing E-1. Throughout the northern Santa Lucia Range, the oldest Tertiary deposits that nonconformably rest on the crystalline basement rock include a sequence of very coarse, poorly sorted, and massive continental sandstone (i.e., "Unnamed Redbeds" (Trb) of Dibblee, 1972). This unit was likely formed as streams flowed across a broad, low relief erosional surface above the ancestral Santa Lucia Range during the late Oligocene and early Miocene (approximately 30 to 36 million years ago). These redbeds are found today as isolated pockets throughout the area.

The redbed deposits are conformably overlaid by a thick sequence of fine- to coarse-grained, marine sandstone (Tts), reflecting a transition from a continental environment to a shallow-water marine environment. This transition occurred over a large area of the range during the middle Miocene (approximately 16 to 30 million years ago). These deposits are, in turn, overlain by the thin bedded, siliceous marine shale of the Monterey Formation (Tm). These shale deposits are much more widespread throughout the northern Santa Lucia Range and represent the deposits of a deep-water marine environment that covered a vast area of the Coast Ranges during the upper Miocene (approximately 10 to 16 million years ago).

This late Tertiary stratigraphic record is reflected in the bedrock that underlies Cachagua Valley which has been juxtaposed against the Cretaceous basement rock by the

Cachagua fault. The Cachagua fault cuts across all of these formations with a minimum vertical separation of approximately 2,000 feet to 3,000 feet (RJA, 1984). As such, the Tertiary bedrock record in the Cachagua Valley area indicates that the fault is younger than the upper Miocene Monterey Formation (Tm).

In the Carmel Valley area, further to the north, the Monterey Formation is overlain by a younger group of marine, near-shore, shallow-water sandstones known as the Santa Margarita Formation (Tsm). While the Cachagua fault does not come in contact with the Santa Margarita Formation, this deposit represents the initial stages of the tectonic emergence (uplift) of the Santa Lucia region following the subsidence reflected in the deep-water environment of the Monterey Formation. The Santa Margarita Formation represents a period of time during the late Miocene to early Pliocene (approximately 3 to 10 million years ago) when continental drainage systems, like the Carmel River, became the primary geologic processes that characterized the Santa Lucia region during late Pliocene to Quaternary time. The Cachagua fault probably became an active seismogenic structure during, or shortly after, this time period.

E.2.2 Geomorphology

Along most of its trace, the Cachagua fault is expressed geomorphically as a series of subdued linear features such as short northwest-trending stream segments and steep, east-facing mountain fronts. These geomorphic features that mark the fault trace all appear to be the product of erosional processes that reflect, for the most part, the transition from hard crystalline basement rock (south of the fault) to soft Tertiary sandstone bedrock (north of the fault). These landscape features are clearly the result of subsequent erosion along the fault trace and are not the products of young fault activity of Holocene to late Pleistocene age. Young geomorphic features that would reflect late Quaternary faulting, such as linear scarps, sag ponds, ridges, are not found along the Cachagua fault. Previous work conducted for the New San Clemente Dam located north of the site reached similar conclusions about the origin of the topographic features found along the Cachagua fault (e.g., RJA, 1984). The steep east-facing mountain fronts found along the fault trace have been considered to be erosional "fault line" scarps, and not the product of fault displacement (RJA, 1985).

In the Cachagua Valley, the fault is found near the base of a significant, although somewhat indistinct, geomorphic break between steep, resistant mountainous terrain and gentle slopes underlain by soft Tertiary rock and Quaternary stream deposits. The Cachagua fault generally defines the southwestern edge of the valley. The Cachagua Valley has developed by erosional downcutting of the Carmel River and lateral erosion into the soft Tertiary bedrock. The general landscape features of Cachagua Valley are structurally controlled by the character and northwest-trending strike of the bedrock that underlies the valley.

On the southwest side of the fault, the high, rugged mountainous terrain is underlain by hard crystalline rocks. Within this terrain, the Carmel River flows toward the northeast in a narrow, steep-walled canyon from upstream of the existing dam for a distance of approximately one mile beyond the dam, crossing the Cachagua fault at the southwestern edge of the valley (Drawing E-1). After joining Cachagua Creek near the center of the Cachagua Valley (near Princes Camp), the river turns to the northwest. The river flows down the center of the valley for approximately one mile before it turns toward the west, and again crosses the fault. At this point, the river re-enters a narrow, v-shaped, river gorge in the high mountains terrain. This same pattern is repeated further downstream, where the river once again flows to the north and crosses the fault near San Clemente Reservoir.

The relationship between Carmel River drainage system and the Cachagua fault can be used to define the Quaternary history of fault movement. The river drainage pattern strongly indicates that movement on the Cachagua fault postdates the time of development of the drainage system. The present drainage system likely started during the Pliocene or early Pleistocene by downcutting into the thick, Tertiary-age marine basin fill sediments and at the end of the regressive sedimentary cycle that culminated with the deposition of the shallow-water marine sandstone of the Santa Margarita Formation (Tsm).

At the northwest end of the Cachagua Valley, the Carmel River turns to the west and flows into a 1,200-foot high mountain front through a narrow v-shaped canyon (i.e., water gap) on the upthrown side of the Cachagua fault. If movement on the Cachagua fault had been greater than the incision rate, the Carmel River would have been "defeated" and the course of the river would have likely been deflected to the northwest along the trace of the fault. The lack of any geomorphic evidence for such deflection indicates that the Carmel River drainage system was well established prior to vertical movement of the fault and that the erosional downcutting of the river was able to keep pace with the uplift once it was initiated.

As concluded in previous studies, it appears that the course of the Carmel River is essentially "blind" to the presence of the Cachagua fault, and is therefore, judged to be a predecessor to fault movement (RJA, 1984, 1985). The movement on the Cachagua fault must postdate the late Pliocene to early Pleistocene time when broad tectonic fault displacement resulted in the region emerging from a marine depositional environment, and the establishment of a terrestrial drainage system now represented by the main stream channel of the Carmel River. This geomorphic analysis restricts the initiation of significant fault movement on the Cachagua fault to the late Pliocene to early Pleistocene.

E.3 Analysis of Quaternary Terrace Deposits

Quaternary stream terrace deposits are very well developed along the Carmel River both upstream and downstream of the proposed New Los Padres Dam site. The stream deposits are defined by near horizontal, terrace surfaces (i.e., terrace-bedrock unconformity) cut into hard bedrock and unconformably overlain by a very coarse stream gravel deposits. The geomorphic shape of the stream terrace deposits resembles an elevated stair tread that parallels the Carmel River and commonly has a steeply sloping edge much like a stair riser (Drawing E-3). The thin terrace deposits and the strath surfaces represent former elevated positions of the ancestral Carmel River. During this investigation, several levels of remnant terraces that represent formerly continuous levels of the Carmel River were mapped. Of these, two well-defined flights of older, paired terrace deposits were identified as the most prominent stream terraces that can be correlated from the existing dam, downstream through the proposed dam axis, and across the Cachagua fault to the Princes Camp area. These terrace levels have been designated as the third Quaternary steam terrace (Qt_3) and fourth Quaternary stream terrace (Qt_4), respectively.

Terrace strath surfaces resulting from stream downcutting were formed as the Carmel River incised its channel and widened the river canyon by lateral erosion during a period of fluvial static equilibrium. The result of this erosion is a nearly horizontal bedrock surface that is commonly covered by a thin layer of very coarse and bouldery stream debris. Stream deposits initially formed during periods of aggradation and are the result of the accumulation of coarse stream alluvium on the strath surfaces. Subsequent channel downcutting through the accumulated alluvium left remnants of the former valley floors as the tread surface of the terraces (Drawing E-3).

The Quaternary fluvial history of the Carmel River is clearly defined and constrained by the Quaternary stream terrace deposits that flank the modern Carmel River canyon. The use of these deposits to define the paleoseismic history of the Cachagua fault is dependent on the recognition and careful geologic mapping of two distinct elements of the deposits. The first element is the geomorphic form of the terrace deposit which is marked by the nearly flat surface of the tread and the sloping bank of the riser (Drawing E-3). The geomorphic form is a mappable landscape element that was formed during a discrete geologic time period during the Quaternary. The Engineering Geologic Map (Drawing 4-1) and cross-sections (Drawing 4-2) exhibit these geomorphic elements at various heights above the active Carmel River channel. The youngest stream terrace deposits are found at the lowest elevations and immediately adjacent to the modern stream channel. Older stream terrace deposits (Qt_3, Qt_4 and older) are found much higher on the slope. These older terrace deposits have locally been deeply dissected and in places are partially or wholly covered by younger alluvial fan deposits (Qf). As a result of alluvial fan deposition, the original stream terrace treads in many places have been buried. This is especially true for the older, higher terrace surfaces (i.e., Qt_4 and higher) that have been exposed to erosion and alluvial fan deposition for a longer period of time. The degree of preservation of the stream terrace surfaces and their alluvial fan cover was used as a general guide to correlate flights of paired terrace deposits along the Carmel River and across the Cachagua fault.

The second and most critical element of the Quaternary stream terraces are the buried strath surfaces that have been beveled into hard bedrock. These surfaces are defined by the unconformity between bedrock and the overlying stream terrace gravels (Drawing E-3). Strath surfaces are clearly exposed in many outcrops along the steep canyon walls downstream of the existing dam. They are commonly marked by sharp erosional bedrock surfaces that are overlain by very coarse, granitic bouldery stream deposits. The eroded edges of strath surfaces are best exposed in the canyon walls that have been oversteepened by the Carmel River. These surfaces have been mapped with considerable accuracy during this study from downstream of the existing dam and across the Cachagua fault into Cachagua Valley at Princes Camp. These strath surfaces are essentially planar with a northeast gradient approximating that of the modern Carmel River. Two key strath surfaces were the focus of the study; one at the base of the third terrace deposit (Qt_3); and a higher strath surface at the base of the fourth terrace deposit (Qt_4). Both of these straths were successfully mapped across the Cachagua fault without interruption. The Qt_3 and the Qt_4 straths are elevated approximately 75 feet and 155 feet above the modern Carmel River channel, respectively.

Based upon geologic mapping, twenty-six (26) exposures of the Qt_3 and Qt_4 strath surfaces were identified (Drawing 4-1). These exposures were subsequently surveyed by Bestor Engineers using Global Positioning Satellite (GPS) techniques. The surveyed strath exposure locations extend from the existing dam downstream to Cachagua Valley. The GPS data and the geologic mapping clearly established the longitudinal profiles of the Qt_3 and Qt_4 strath surfaces. These profiles define the elevated positions and gradients the ancient Carmel River and the lack of tectonic deformation across the Cachagua fault.

In summary, the identified strath surfaces represent significant time intervals during the Quaternary when the Carmel River was actively engaged in lateral erosion of the valley floor. These periods of valley widening were followed by significant stages of channel downcutting that resulted in abandonment of the stream channels (terraces) now elevated high above the present Carmel River. As described in the following sections, the Qt_3 and Qt_4 stream profiles cross the Cachagua fault without showing any signs of tectonic disturbance, demonstrating that movement on the Cachagua fault has not taken place at least since the time that the Qt_3 and Qt_4 strath surfaces were being formed.

E.4 Constraints on Fault Location

Regional geologic maps place the Cachagua fault approximately 1,400 feet downstream of the New Los Padres Dam site (e.g., Dibblee, 1972). At this location, the Carmel River flows northeast from the proposed dam site, crossing the northwest-trending Cachagua fault at nearly a right angle. This relationship between a steeply inclined, northwest-trending fault and a deep, steep-walled, northeast-trending stream canyon provides a unique opportunity to accurately locate the fault and to explore its structural relationship with the bedrock formations and the overlying Quaternary deposits.

For the this investigation, the fault was initially constrained to a narrow, 200-foot wide zone by mapping closely spaced bedrock outcrops exposed along the canyon walls at a scale of 1:1,200 (1 inch = 100 feet). On the northwest side of the river, the fault zone was mapped across a small, east-west-trending tributary canyon that crosses the fault at right angles. Because of the excellent geologic exposures in this canyon, and the cross sectional relationships with the fault, more detailed geologic mapping of this area was performed at a scale of 1:240 (1 inch = 20 feet). Detailed mapping revealed that the fault crosses the canyon approximately 100 feet from the mouth of the canyon. At this location, the fault juxtaposes crystalline granitic and metamorphic rocks on the west side of the fault against Tertiary sandstone to the east. A well-exposed Quaternary stream deposit which, in turn, is partially overlain by alluvial fan debris covers the faulted bedrock. An elevated strath surface beveled into the bedrock by the ancient Carmel River is exposed at the base of the Quaternary deposits. This strath surface, which represents the base of the third major terrace deposit above the modern river channel (Qt_3), was explored by a series of small backhoe excavations over a distance of 200 feet along both sides of the canyon. Geologic observation of the backhoe pits allowed further constraint of the location of the Cachagua fault to a 60-foot wide zone. A large, vertical excavation was then excavated into the northwest side of the canyon to expose the fault. This excavation revealed a faulted interval between highly fractured granitic and metamorphic bedrock and steeply dipping, coarse grained sandstone (Drawing E-2).

The geologic structure and physical condition of geologic units exposed in the 40-foot wide excavation indicate that faulting has influenced the bedrock units, but not the overlying Quaternary terrace deposits. The crystalline bedrock on the southwest side of the fault possesses a shear fabric that approximates the fault orientation (approximately N10°W). The sandstone strata dips steeply (70°) to the northeast, away from the fault at angles and directions that are inconsistent with the regional southwest-dipping geologic structure of this sedimentary bedrock unit. The overlying Quaternary stream terrace deposits can be clearly traced across the fault without offset or interruption. The strath surface can be traced from a point located upstream from the excavation over a distance of approximately 125 feet, to a point downstream (past the excavation) a horizontal distance of nearly 300 feet. This same strath surface also can be correlated with similar straths located to the north and south of the canyon for at least 4,000 feet and 2,000 feet, respectively.

E.5 Probable Age of Latest Fault Movement

Strath surfaces found at the base of the third and fourth Quaternary stream terrace deposits (Qt_3 and Qt_4) in the area of the proposed New Los Padres Dam clearly overlie the trace of the Cachagua fault and are not offset. As a result, the latest movement on the fault predates the time of erosion of the strath surfaces and the age of the overlying terrace deposits. Thus, the timing of the last movement on the Cachagua fault can be constrained by estimating the likely age of the unfaulted strath surfaces at the base of the Quaternary stream terrace deposits. Radiometric dating of the terrace deposits could not be performed due to the lack of charcoal. Even if charcoal had been found, however, the age of the terraces is likely greater than can be dated radiometrically. For this study, absolute ages were estimated by relating the length of time required for the Carmel River to downcut from the Qt_3 and Qt_4 elevations to its present level.

In the area where the fault crosses the Carmel River, the elevation of the Qt_3 strath is approximately 900 feet, and the elevation of the Qt_4 strath is about 980 feet. The present elevation of the Carmel River in the area where the fault traverses the river is approximately 840 feet. This places the ancient Carmel River channel above the present river level a vertical height of 60 feet for the Qt_3 deposits and 140 feet for the Qt_4 deposits. To estimate the age of the unfaulted terrace materials, the times needed for the Carmel River to downcut from these former river levels to the present position of the modern channel has been calculated. Two methods of calculations were performed.

The first method depends upon the assumption that the river incision rate is equal to the uplift rate. Uplift rates are not well constrained for the northern Santa Lucia Range; however, the mean Quaternary uplift rate for an area of relatively rapid uplift in the southern Santa Cruz Mountains adjacent to the San Andreas fault has been estimated to be 0.8 mm/year (Burgmann and others, 1994). Using these uplift rates, and the assumption that erosion was constant during the downcutting process, the incision time represented by Qt_3 and Qt_4 would be approximately 22,900 years and 53,350 years, respectively. These time ranges are considered unrealistically short given that the very youthful, rugged mountainous landscape clearly suggests that the Quaternary uplift has well exceeded the rate of river incision. As such, the Qt_3 and Qt_4 deposits must be considerably older than that indicated by these uplift rate calculations.

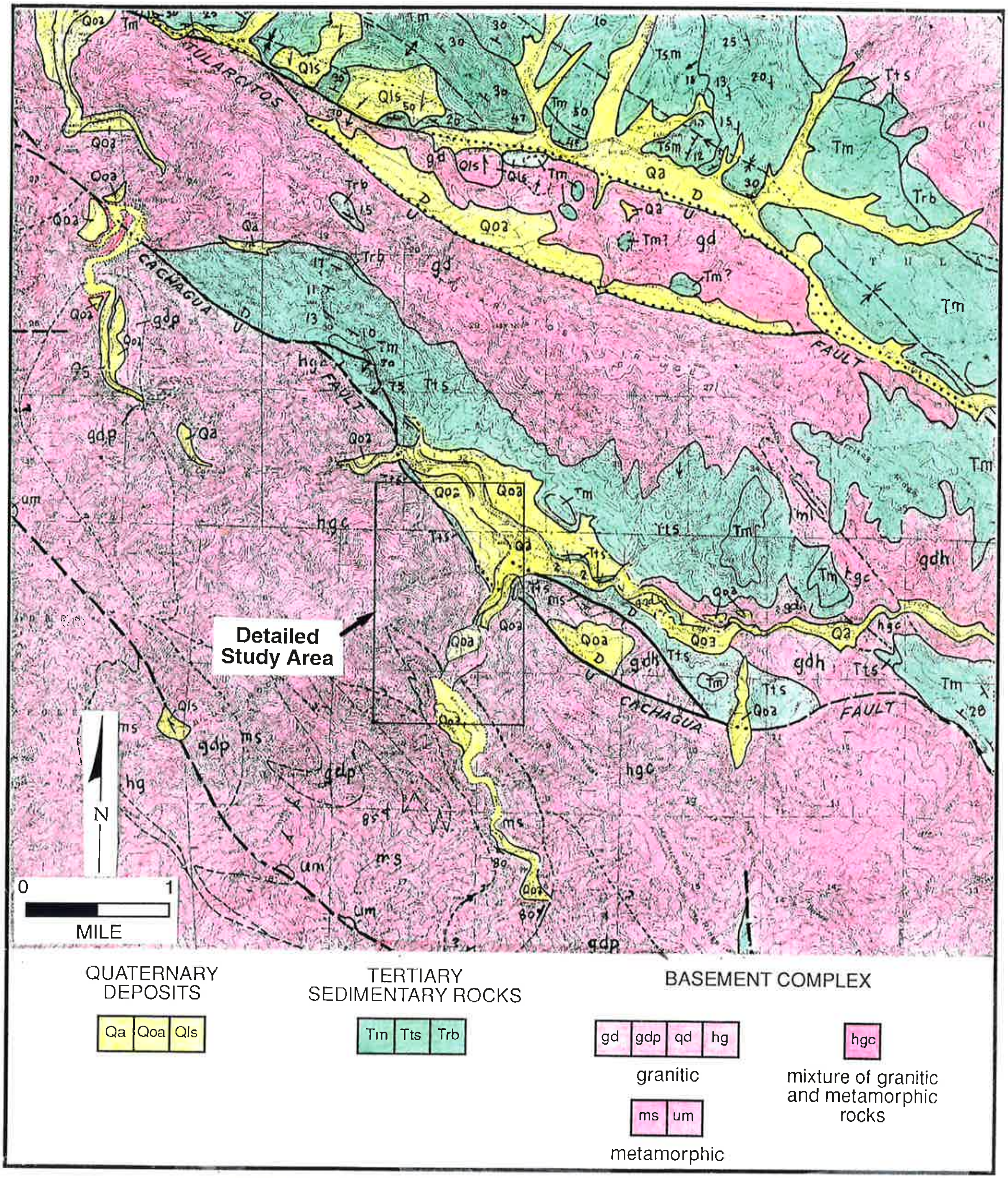
The second calculation involves the use of more reasonable river incision rates.

While published incision rates for the northern Santa Lucia Range are not available, long-term incision rates for an area in California with somewhat similar geomorphic and geologic settings as the northern Santa Lucia Range were selected. Very well constrained incision rates of 0.5 to 0.2 mm/year have been estimated in the Feather River drainage of the northern Sierra Nevada, where bedrock materials and the fluvial stage are similar to the project area (Wakabayashi and others, 1994). Using these estimates, the approximate calculated age ranges are 36,600 to 41,500 years for the Qt_3 stream terrace, and 85,400 to 213,500 years for the Qt_4 stream terrace.

Given the above descriptions and calculations, the Qt_4 terrace deposit represents the oldest widespread Quaternary deposit in the project area and it has not been offset by the Cachagua fault. The estimated age of the Qt_4 terrace deposit is between 85,400 years and 213,500 years before present. Therefore, based on the continuity of terrace deposits, the behavior of the Cachagua fault is one of inactivity over at least the past approximately 85,400 to 213,500 years.

In addition to the Qt_3 and Qt_4 terrace deposits in the area of the site, an even older Quaternary alluvial terrace deposit has been mapped along the fault approximately 4,000 feet south of the Carmel River (Drawing E-1). This deposit is situated on an elevated plateau approximately 400 to 500 feet above the modern Carmel River channel. It represents the highest major Quaternary stream terrace material within the Cachagua Valley region. Although this deposit is shown by Dibblee (1972) to be in contact with the crystalline basement rocks along the Cachagua fault, no field evidence could be found during this study to confirm this relationship, nor could geomorphic features be found to indicate that movement on the fault had taken place since the time that the alluvial materials were deposited. This lack of tectonic geomorphic features has also been documented at this locality by Bryant (1985) during the Alquist-Priolo field evaluation of the level of activity of the Cachagua fault. If the Cachagua fault had a moderate to high slip rate during the late Quaternary time, noticeable geomorphic features that are characteristic of young rift topography would be present.

While the geologic age of this deposit is unknown, its elevated position suggests that it is considerably older than the Qt_4 strath. The base of the terrace deposit could not be observed due to poor exposure. However, an approximate height of the strath above the Carmel River was estimated to be 400 feet. Using the same incision rates from which the geologic age of the Qt_4 strath was estimated at approximately 244,000 years to 610,000 years. These ages, although somewhat less constrained by the lack of precision for the height of the strath, appear to be reasonable estimates. As such, the last movement event on the Cachagua fault would predate these age estimates and the fault would be considered not active. The above age estimates represent the minimum age of the last faulting event, and the actual last movement on the Cachagua fault may be much older. The lack of older Quaternary deposits in the area, however, prevent better determination of the age of the last fault movement.



REGIONAL GEOLOGIC MAP (MODIFIED FROM DIBBLEE, 1972).

PREPARED BY WFC

REVIEWED BY WRC

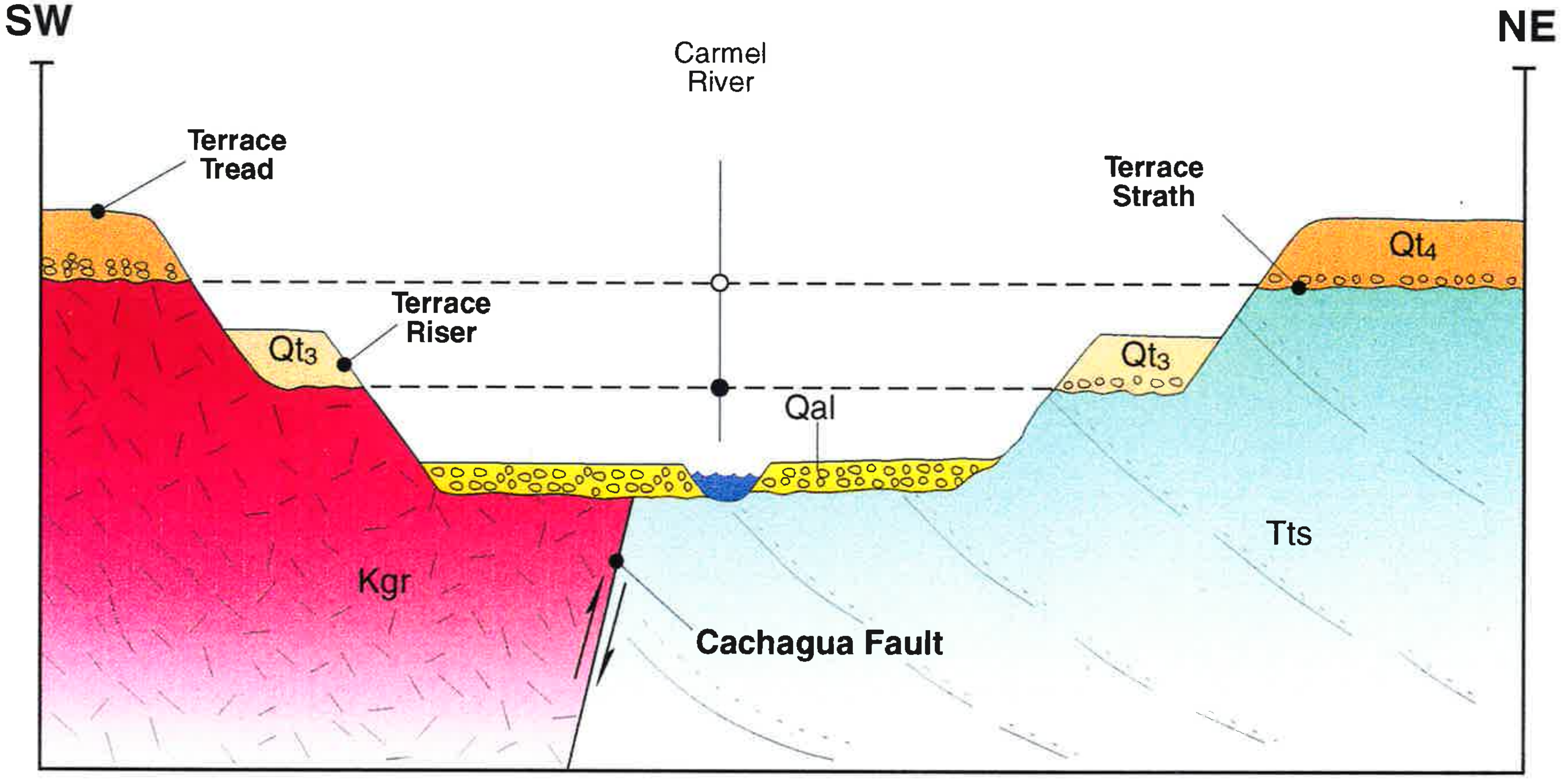
DATE 3/8/95

THE MARK GROUP, INC.
ENGINEERS & GEOLOGISTS
In Association With
MORRISON KNUDSEN CORPORATION
William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

DRAWING NO.
E-1



SCHEMATIC DIAGRAM OF STREAM TERRACE-FAULT RELATIONSHIP.

PREPARED BY WRC

REVIEWED BY WFC

DATE 3/8/95

THE
MARK
GROUP, INC.
ENGINEERS & GEOLOGISTS
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William Cotton and Associates

Monterey Peninsula Water Management District
Geotechnical and Engineering Studies
New Los Padres Water Supply Project
Monterey County, California

PROJECT NO.
94-1198801.80

DRAWING NO.
E-3

Appendix F

APPENDIX F

PROCEDURES OF RCC TESTING

NEW LOS PADRES DAM

RCC TRIAL MIX PROGRAM

PROCEDURES

This procedure has been prepared to familiarize Testing Engineers Inc. (TEI) laboratory personnel with the material and equipment required, and the procedure to be followed in carrying out the RCC trial mix program for the New Los Padres Dam.

Purpose

The purpose of the RCC trial mix program is to develop strength vs time curves for RCC mixes over a range of cement contents, utilizing processed aggregate obtained from the alluvial terrace gravel borrow areas upstream of the dam.

Materials

A composite sample was obtained from test pits in Borrow Area A (TP-12) and B (TP-8,9,10,and 11). The material, characterized as sandy gravel with cobbles and boulders to 36-inch diameter, was crushed and screened to 1-inch minus, at Granite Construction Company's plant in San Jose.

Cement for the trial mix program is to be furnished by TEI, and should be Type II Portland Cement.

Equipment

Equipment to be supplied by Morrison Knudsen (MK):

- Cylinder mold support
- Impact Hammer w/shoe

Equipment to be supplied by TEI:

- Concrete mixer (9 cf capacity)
- 60 6x12" disposable plastic cylinder molds
- Platform scales
- 1000cc beaker
- Scoop
- Sheltered area with concrete floor

Compaction Trial

Prior to preparing the trial mixes a compaction trial will be carried out following the procedure described below in order to determine the optimum water content for the RCC mixes. The compaction trial will consist of preparing a mix with 300 lbs/cy cement and 3,672 lbs/cy of aggregate with a varying water content. The mixer will be charged initially with 170 lbs/cy water content. Two cylinders will be prepared and weighed. Water content will be added to the mixer in 20 lbs/cy (aprroximately 0.5%) increments, and one cylinder compacted and weighed at each increment, until the maximum density is reached. Density will be evaluated not only by weighing the cylinders but also by visual observation during mixing and compacting.

Once the optimum water content is determined, optimum compaction method will be determined by varying the number (thickness) of lifts placed in the mold, the surcharge applied to the hammer, and the time of compaction.

Mix Proportions

Five trial mixes will be prepared. 10 cylinders will be cast for each mix. Water content, as determined by the compaction trial described above, will be kept constant (with minor adjustments based on visual observation) for all mixes. Batch aggregate weight will be determined SSD, by the absolute volume method (Bulk SSD SpG of the aggregate is 2.692; air content is assumed to be 1.5%). Cement content is predetermined as shown in the following table (SpG of cement is assumed to be 3.15). Water/Cement (W/C) ratio will be determined from the proportions. Mixing water will be determined by subtracting 1.4% of the aggregate weight from the total water content (moisture content of the aggregate is 3.0% with an absorption of 1.6%).

After optimum water content has been determined the following table should be completed:

Mix No.	Water(lbs/cy)*	Cement(lbs/cy)	Aggregate(lbs/cy)	W/C Ratio
1		100		
2		200		
3		300		
4		400		
5		500		

10 cylinders (approximately 2.0 cf) will require mixing a 3.0 cf batch for each trial mix. Therefore actual batch weights can be determined by dividing the weights in the table by 9.

Procedure

1. Anchor the cylinder mold support to the concrete floor.

2. Place a plastic cylinder mold in the support, close and cap.

3. Charge the mixer with the predetermined weight of aggregate for each mix.

4. Add the predetermined weight of cement and mix thoroughly.

5. Add the predetermined weight of water and mix thoroughly.

6. Dump the batch onto a moist concrete floor or tarp. Keep the mix from drying out by spraying with water as necessary.

7. With a scoop, fill the cylinder to approximately 1/3.

8. Compact the mix with the the impact hammer using the full weight of the operator as surcharge. Time of compaction will be determined during the compaction trial but is expected to range from 10-20 seconds or until paste begins to appear around the compation shoe. It is important to work quickly, as compressive strength of RCC mixes decreases from time of mixing.

9. Repeat this process in three increments until the cylinder is full.

10. Remove the cylinder from the mold support and determine unit weight. Label and store in fog room.

11. Prepare 10 cylinders of each mix.

12. Break two cylinders of each mix in compression at 7, 28, and 90-days. Break two cylinders of each mix in splitting tension at 28, and 90-days. Measure unit weight prior to breaking each cylinder.

Appendix G

APPENDIX G

RESULTS OF RCC TEST PROGRAM

APPENDIX G
RESULTS OF RCC TEST PROGRAM

NEW LOS PADRES DAM RCC TRIAL MIX										
Mix No. (1)	COMPRESSION STRENGTH						SPLITTING TENSILE STRENGTH			
	7-Day		28-Day		90-Day		28-Day		90-Day	
	f_c	γ	f_c	γ	f_c	γ	f_t	γ	f_t	γ
1 100 /cy	450 280	144.0 142.5	540 530	146.1 143.0	680 610	150.2 148.1	80 85	146.1 147.1	120 90	147.6 151.2
2 200 /cy	820	153.2	1,410	152.2	1680	153.8	225	152.7	270	154.8
3 300 /cy	2,510 2,440	149.6 149.1	2,860 2,540	148.6 150.2	3790 2890	150.2 150.7	350 410	150.2 149.7	495 475	154.2 150.7
4 400/cy	2,530 2,000	150.1 146.0	4,230 2,360	149.1 148.6	5040 4500	154.3 152.7	495 460	153.2 150.2	680 510	153.8 154.3
5 500 /cy	2,770 2,270	148.0 147.6	3,290 2,830	146.1 146.6	3360 3540	146.6 147.1	500 445	148.6 147.6	390 475	147.1 148.6
6 200 /cy	1,530 1,470	149.6 147.6	2,090 1,980	150.2 150.7	2390 2640	151.7 152.2	320 335	151.2 152.7	390 390	153.3 151.7
7 400 /cy	1,650 1,890	146.6 146.6	2,230 2,360	147.1 147.1	TBP	144.0 147.1	330 270	145.5 144.5	TBP	149.7 145.0
8 500 /cy	2,860 2,770	149.7 150.7	3,240 3,390	149.7 150.2	TBP	147.6 148.6	450 440	152.2 149.7	TBP	152.2 151.2

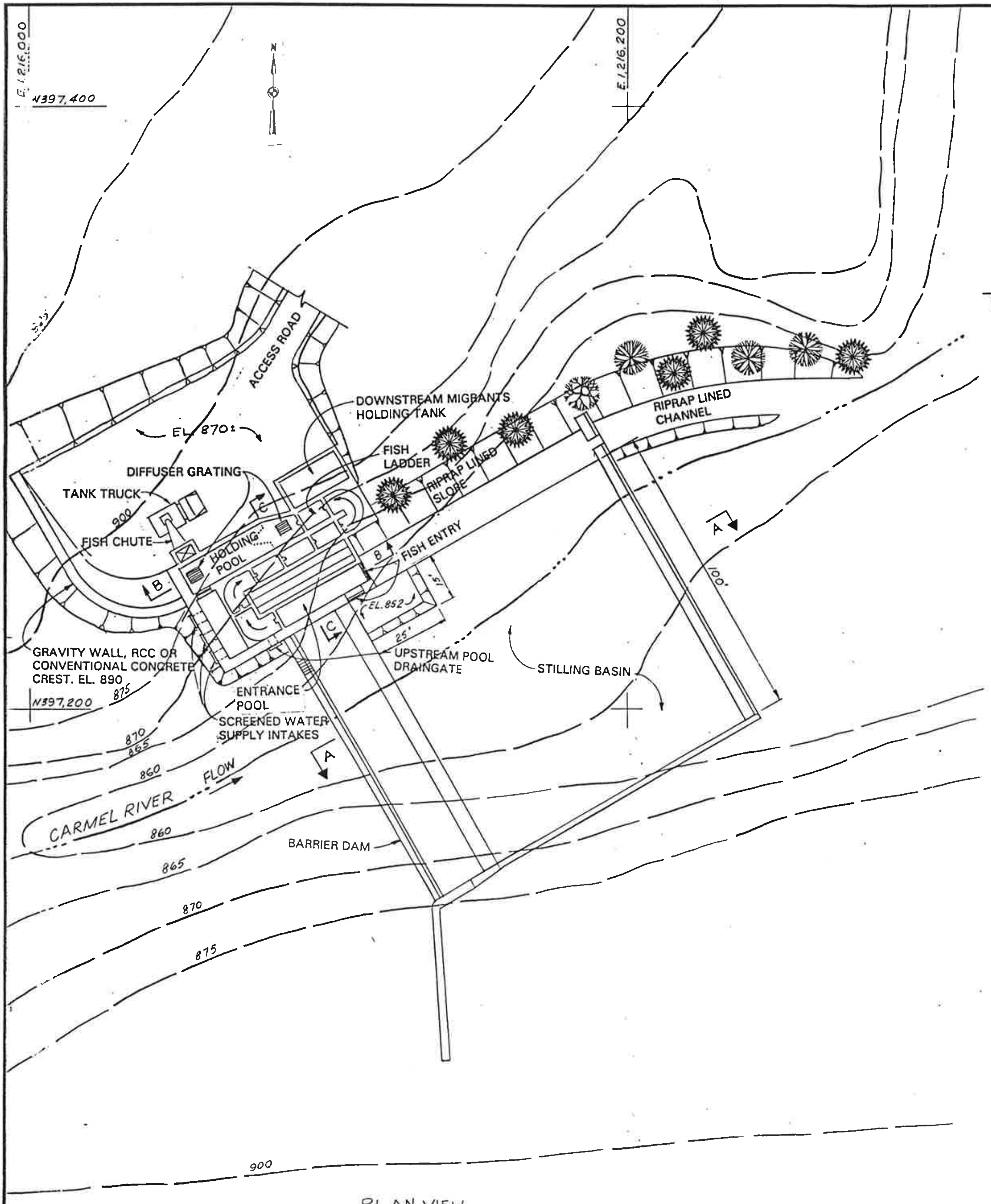
* 11-Day and 29-Day
TBP = To be performed
f_c = compression strength - psi
γ = density of cylinder - lb/ft^3
f_t = splitting tensile strength - psi
(1) = mix no. and pounds of cement/cy

Appendix H

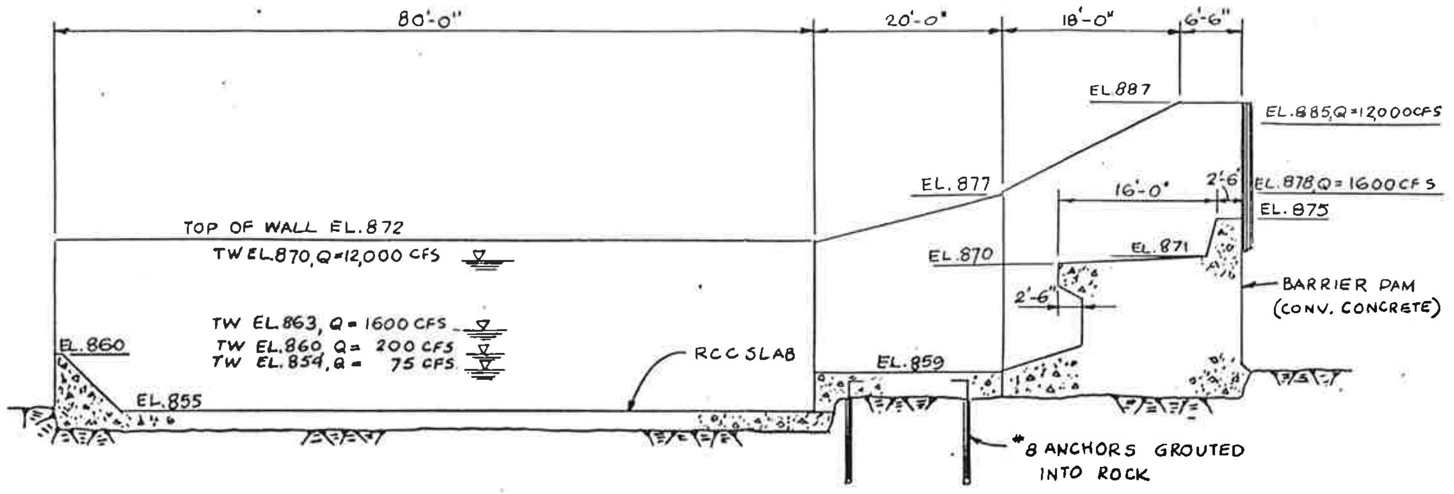
APPENDIX H

FISH FACILITIES

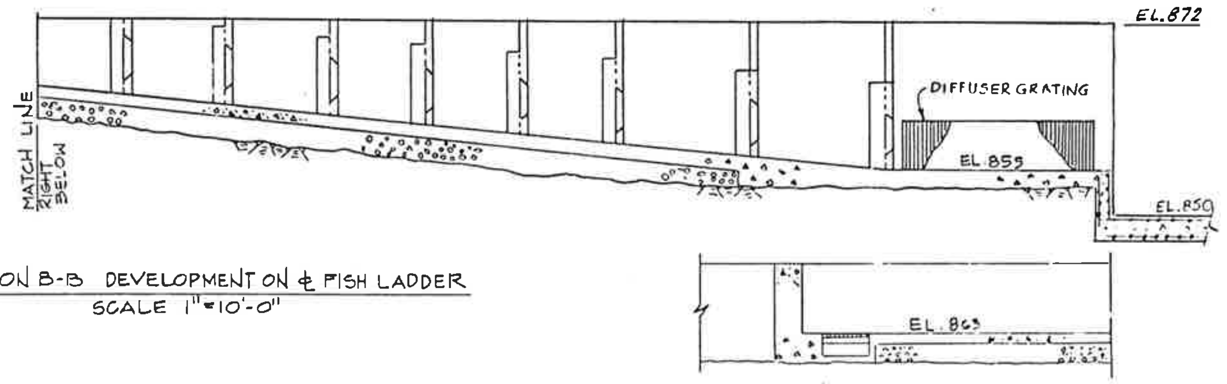
SIZE D 22 x 34



PLAN VIEW
SCALE 1"=20'-0"

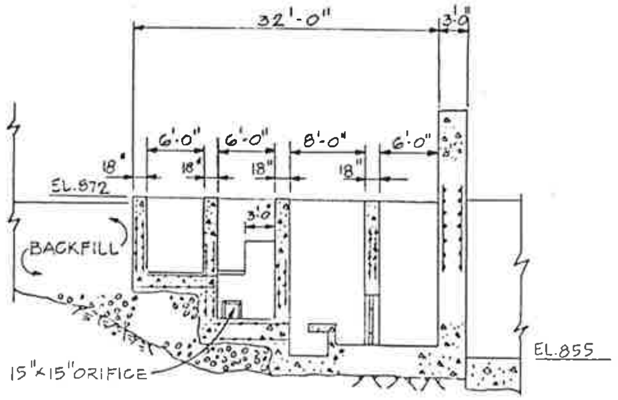


SECTION A-A
SCALE 1"=10'-0"



SECTION B-B DEVELOPMENT ON ℄ FISH LADDER
SCALE 1"=10'-0"

NOTE:
1. ELEVATIONS SHOWN ARE APPROXIMATE.



SECTION C-C
SCALE 1"=10'-0"

MONTEREY PENINSULA WATER MANAGEMENT DISTRICT

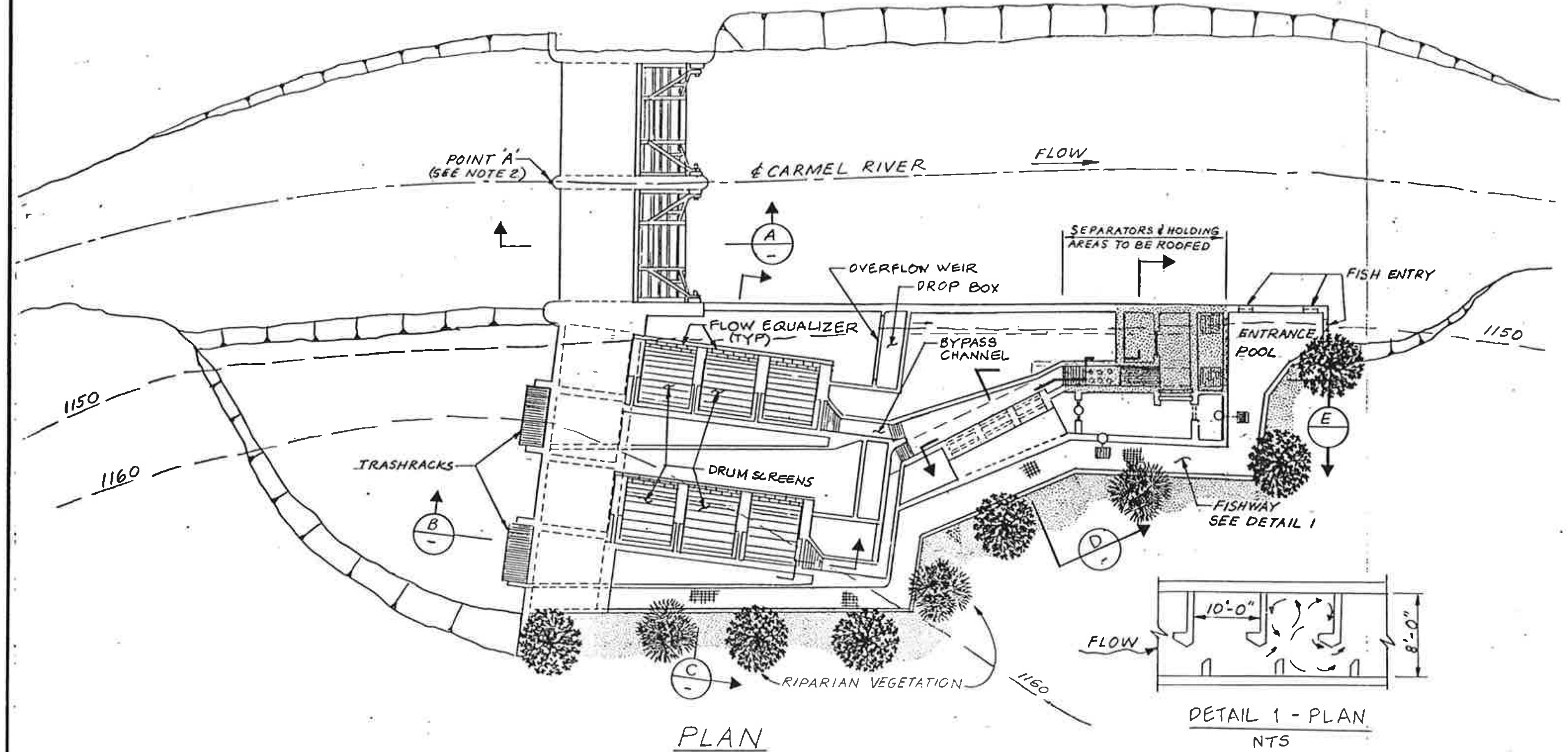
MONTEREY PENINSULA WATER SUPPLY PROJECT
NEW LOS PADRES DAM
UPSTREAM MIGRANT COLLECTION FACILITIES
PLAN AND SECTIONS

FEBRUARY 1993

Bechtel Corporation
San Francisco, California

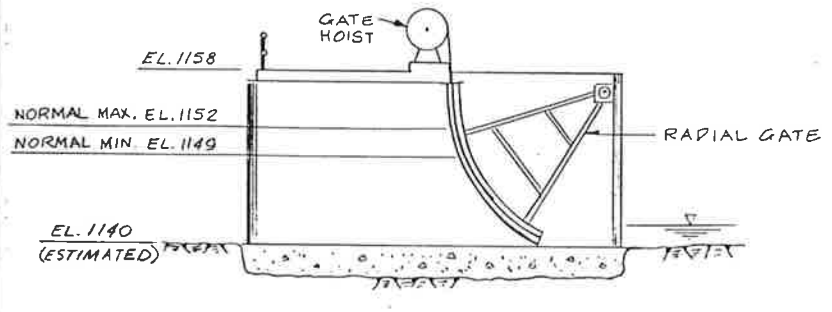
NOTES

1. ELEVATIONS SHOWN ARE APPROXIMATE.
2. ELEVATIONS ARE BASED ON THE RIVERBED GRADE AT THE HEAD OF THE CENTER PIER BETWEEN THE RADIAL GATES (POINT 'A') BEING EL.1140 WHICH IS ESTIMATED FROM USGS 1:24,000 VENTANA CONES QUAD SHEET.

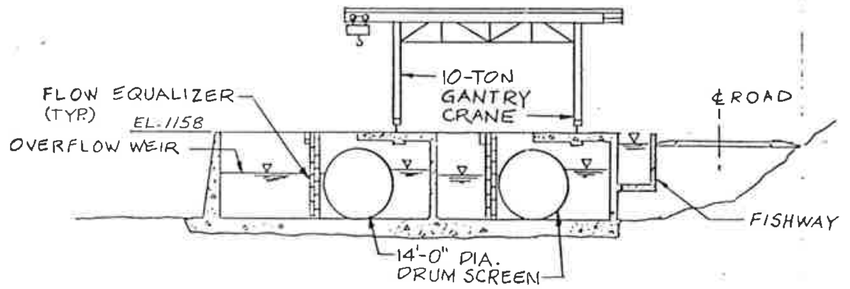


PLAN
SCALE 1"=20'

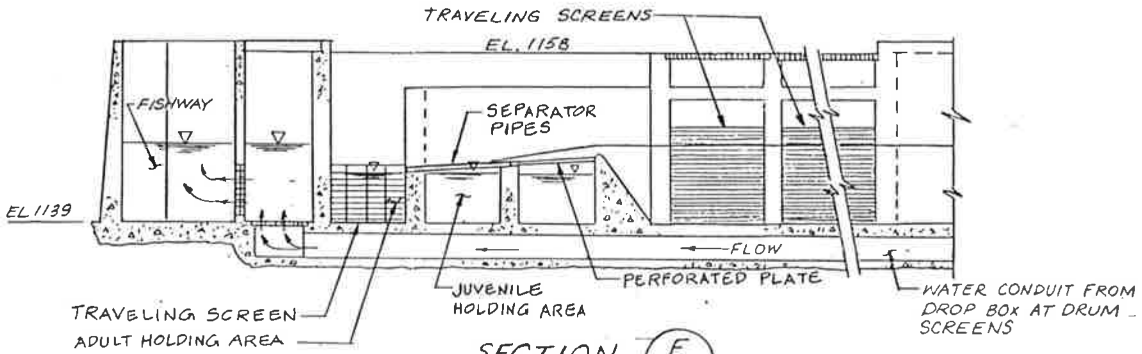
DETAIL 1 - PLAN
NTS



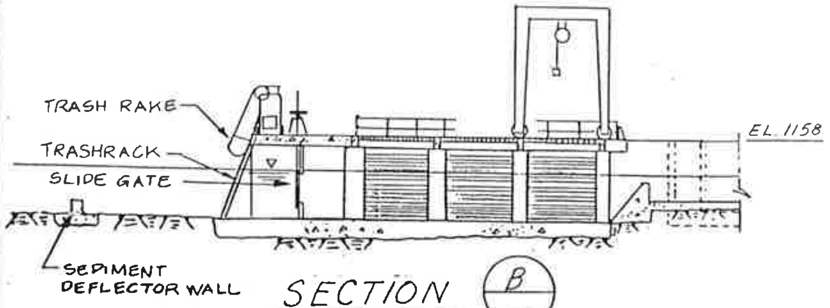
SECTION A
SCALE 1"=10'



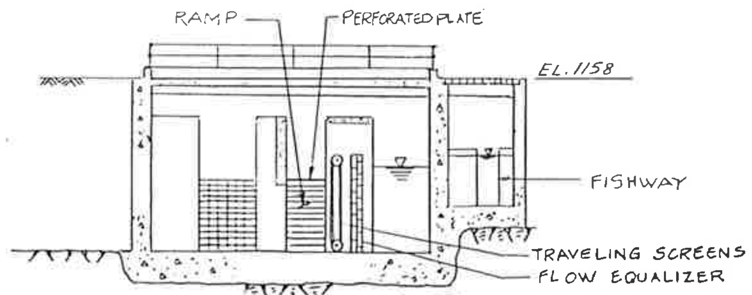
SECTION C
SCALE 1"=20'



SECTION E
SCALE 1"=10'



SECTION B
SCALE 1"=20'



SECTION D
SCALE 1"=10'

MONTEREY PENINSULA WATER MANAGEMENT DISTRICT

MONTEREY PENINSULA WATER SUPPLY PROJECT
NEW LOS PADRES DAM
DOWNSTREAM MIGRANT SCREENING FACILITIES
PLAN AND SECTIONS

JANUARY 1991

Bechtel Corporation
San Francisco, California

FIGURE 3-2

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