US Army Corps
of Engineers
New York District

DRAFT

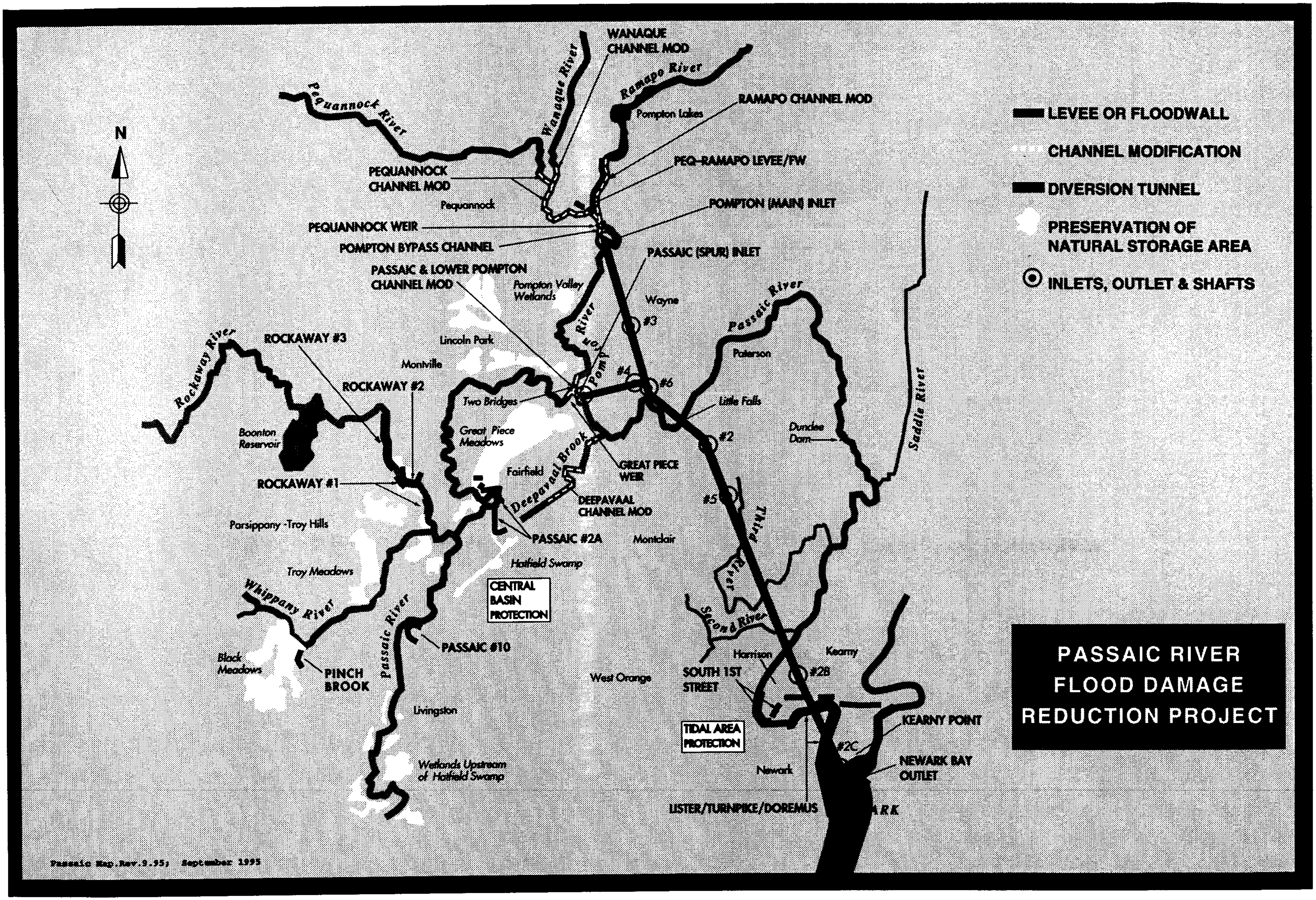
General Design Memorandum

Passaic River
Flood Damage Reduction Project

Appendix E - Geotechnical Design
Tunnel and Shafts

September 1995

Vol. I of III


PASSAIC RIVER
FLOOD DAMAGE
REDUCTION PROJECT
LEVEE OR FLOODWALL
CHANNEL MODIFICATION
DIVERSION TUNNEL
PRESERVATION OF NATURAL STORAGE AREA
INLETS, OUTLET & SHAFTS
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WANAQUE CHANNEL MOD
Wanaque River
Ramapo River
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RAMAPO CHANNEL MOD
PEQ-RAMAPO LEVEE/FW
PEQUANNOCK CHANNEL MOD
Pequannock
POMPTON (MAIN) INLET
PEQUANNOCK WEIR
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SOUTH 1ST STREET
#2B
Livingston
TIDAL AREA PROTECTION
KEARNY POINT
#2C
Wetlands Upstream of Hatfield Swamp
Newark
NEWARK BAY OUTLET
LISTER/TURNPIKE/DOREMUS
Passaic Map.Rev.9.95; September 1995

APPENDIX E

GEOTECHNICAL DESIGN

TUNNELS AND SHAFTS

VOLUME I

APPENDIX E

SECTION 1

GEOLOGY

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

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APPENDIX E

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APPENDIX E

SECTION 1

GEOLOGY

1.1 INTRODUCTION

It is the intent of this section to present the geology for the entire project area encompassing the tunnel and all other project features. Included are discussions of physiography, geology, seismicity, and subsurface investigation rationale.

1.2. REGIONAL SITE DESCRIPTION

1.2.1 <u>Physiography and Land Use</u> The Passaic River Basin occupies a 935 square mile area in northeast New Jersey and southeast New York. Parts of eight New Jersey and two New York counties, containing 132 municipalities, are present in the basin. The basin is located completely within the Appalachian Province, which covers the northern one third of New Jersey. This province is further divided into six lesser provinces, two of which, the Highlands and Piedmont, contain the Passaic River watershed. In some of the literature the Piedmont is divided into two sub-provinces, the Central Basin and the Lower Valley. These three physiographic regions the Highlands, The Central Basin, and The Lower Valley, shown on Figure E.1.1, are distinguishable by widely differing topographic, hydrologic, and land use characteristics.

1.2.1.1 <u>The Highlands Province</u> The Highlands Province is a heavily wooded mountainous region of about 500 square miles in the western and northern half of the basin. The countryside is rural in character and there is much undeveloped land. The headwaters of the major tributaries of the Passaic River are located in this area. Except for the headwaters of the Wanaque and Ramapo Rivers, which are located in New York, most of the Highland area of the Passaic Basin is in New Jersey.

1.2.1.2 <u>The Central Basin</u> The Central Basin, which is located entirely in New Jersey, is an oval 262-square-mile depression consisting of low, rolling hills, flat meadowlands, and freshwater swamps. Extensive, and expanding residential and commercial development is typical of this area although small tracts of undeveloped land still remain. The area experiences frequent flooding.

1.2.1.3 <u>The Lower Valley</u> The Lower Valley, a relatively flat area covering 173 square miles, is located in the southeastern part of the basin near New York City, and is susceptible to extensive flooding. Except for a small portion of the Saddle River drainage area, this area is in New Jersey. The Lower Valley, from Newark upstream to Paterson, is the most densely populated and heavily industrialized part of the basin. Very few areas of natural vegetation remain in the valley and wetlands, once common, are almost completely gone. Remaining open areas are used to satisfy recreation needs.

1.2.2 Geomorphology and Topography

1.2.2.1 The Highlands Province The Highlands Province is a dissected, high relief, mountainous region higher in elevation than the adjacent Piedmont Province. Altitudes range from 600 to 1500 feet above mean sea level (msl) with some slopes having grades in excess of 20 percent. Valley depths range from 300 to 600 feet below the ridge crests. Glacially formed lakes are another characteristic of this scenic area.

1.2.2.2 The Piedmont Province The Piedmont Province, which encompasses the Central Basin and the Lower Valley, is topographically low and smooth in relief except for the three generally northeasterly-southwesterly trending ridges known as the Watchung Mountains. The undulating plain of the Piedmont attains its highest elevation along the border fault (Ramapo Fault) at the western margin of the province and generally slopes southeastward. The rolling and undulating topography of the plains is developed on glacial material which covers the area. The Watchung Mountains, which result from differential erosion around resistant beds of basalt, are two to three hundred feet higher in elevation than the surrounding plain and reach elevations which range from 450 to 870 feet msl at High Mountain north of Paterson. A further discussion of the nature of this region is given in paragraph 1.2.3.

1.2.3 Regional Geology

1.2.3.1 Stratigraphy

1.2.3.1.1 Soils Almost the entire Passaic River Basin was subjected to glacial erosion and deposition, producing lasting effects on its topography and drainage. The last stage of the Wisconsinan glacier created the present landscape of the Passaic River Basin as it swept away deposits from previous glaciations. The southernmost limit of the glacial advancement is marked by a terminal moraine ridge from Dover to Summit. Also, as the glacier retreated, a blanket of ground moraine was deposited on bedrock. The moraine material is generally comprised of an unsorted, heterogenous mixture of material, ranging in size from clay to boulders, and can be 100 feet or more in thickness. Glacial till and stratified drift deposits are present in the Highlands Province and the Piedmont Province. These deposits generally consist of silt, sand, and gravel and can be up to 10 feet or more in thickness. West of Watchung Mountains, glacial Lake Passaic deposited considerable quantities of impermeable silts and clays. These clay deposits underlie the vast meadowlands of the Central Basin.

1.2.3.1.2 Bed Rock

In New Jersey the Highlands Province is underlain mostly by Precambrian rocks which extend northeastward into the Hudson Highlands of New York and southwestward toward Reading Pennsylvania. The term Reading Prong has been used by many authors to name this region. The geology is typified by a complex of metamorphic and granitic rocks. Paleozoic rocks are present in long narrow valleys which trend northeast-southwest. No project features are to be developed in the Highlands Province.

The Piedmont Province is known geologically as the Newark Basin. Sandstones, shales, limy shales, and conglomerates were deposited in the basin during Triassic and Jurassic times. Around the margins of the basin the sediments are typically much coarser being closer to the upland source areas. Adjacent to the northwest limit of the basin this conglomeritic material is named the Hammer Creek Formation. The basal unit is argylitic and known as the Lockatong Formation. Above this the rocks belong to the Brunswick Formation. They grade from red sandstone, with minor arkosic, conglomeritic beds, upward to interbedded shale and siltstone, culminating with limy shale. Separating these sedimentary deposits are three basalt beds composed of several flows each. Generally the oldest sediments of the Brunswick Formation are the most uniform and massive. The sequence of younger sedimentary beds are characterized as cyclic with each cycle becoming progressively finer toward the top. This is characteristic of a geosynclinal basin which has undergone periods of rapid subsidence followed by quiescent periods. Coarser sediments filled the basin at the start of the active periods and as the basin filled the stream gradients decreased and thus the texture of the sediments they carried decreased. The continental origin of these rocks is supported by their texture, composition, and by the scant fossil evidence they contain. Dinosaur foot prints are preserved in some of these sediments. The tunnels are to be driven in the Brunswick

Formation.

The basalt layers, collectively known as the Watchung Flows, were extruded as sheet lava flows also known as flood basalt. Each elongated ridge of the Watchung Mountains is made up of several flows. Extended periods between volcanic activity allowed paleosoils to develop on some of the flows. In some places flows encountered standing water and developed a characteristic pillow structure. Only minor heat alteration, usually limited to less than 2 ft of the underlying sedimentary rock, is present at the conformable lower contacts. The physical characteristics of the basalt are dependent upon where in the lava flow it cooled. The top of the flows cooled most rapidly resulting in an aphanitic crust which was occasionally broken through by molten lava creating a characteristic Pahoehoe structure that is preserved in some locations. The bottom portion of the flow, being in contact with the underlying rock, also cooled more rapidly than the interior of the flow. These zones of rapid cooling are usually vesicular to amygdaloidal. The interior portions of the lava flows cooled more slowly resulting in the basalt being more phaneritic or coarse crystalline. In some of the thicker flows this rock is sufficiently well crystallized to be classified as a gabbro. Another characteristic of the flood basalt is the secondary mineralization which they contain. This area is well known for the unusual assemblage of zeolite minerals which is found here. Prenite, pectolite, analcite, and natrolite are just a few of the mineral species which are present. Associated with these are calcite, pyrite and quartz. Occasional diabase intrusives in the Triassic sediments are associated with the basalt.

The Newark Basin structure results in progressively younger rocks being exposed toward the northwest (Fig. E.1.2). To the east of the lowermost basalt formation, the rocks are late Triassic age with the Triassic-Jurassic contact at the base of this basalt. A geologic column of the section involved in the tunneling is shown on Figure E.1.3 and a geologic map is shown on Figure E.1.4.

1.2.3.2 <u>Structural Geology</u> The Newark Basin is the largest of six major (and numerous lesser) Triassic rift basins which stretch in a sinuous belt for more than 1,000 miles along the east coast of North America. These features are associated with an episode of tensional forces that accompanied the widening of the Atlantic Ocean Basin. The Newark Basin is described as a post-orogenic half graben which formed during the Palisade Disturbance near the end of the Appalachian orogenic cycle. The half graben developed as the crustal block underlying it dropped along its northwestern boundary on a prominent structural feature known as the Ramapo or Border Fault while hinging on its southeastern border. As subsidence continued the basin was filled with a wedge of continental sediments derived from erosion of the highlands which surrounded it. Judging from the nature of the sediments the downward movement was not uniform over time and coarse and fine sediments inter-finger in great complexity. The strike of the formations within the Newark Basin are to the north-northeast, parallel with the axis of the basin. Both the sedimentary and basalt beds along the tunnel alignment predominantly dip between 7 degrees and 15 degrees to the northwest. This regional pattern was imposed by the filling and continued down-dropping of each successive layer of sediment. The Ramapo Fault is a northeast trending right oblique normal fault and dips 50 deg. to 60 deg. to the southeast. It is estimated that a total of at least 18,000 feet of displacement has occurred on this fault. Numerous other faults are present in the Highlands Province where they parallel the Ramapo border fault and dip mostly southeastward. In the Piedmont Province the known faults are mainly steeply dipping normal faults which also parallel the trend of the Newark Basin. Parallel to the trend of the Newark Basin and close to its northwestern margin is the Watchung Syncline plunging to the northeast. The Main Tunnel alignment crosses the axis of the syncline and, therefore encounters a reversal of the bedding dips. Most of the rocks in the Passaic River Basin are part of the eastern limb of the syncline. Local, perpendicular trending anticlines and synclines plunging to the northwest are present in addition to the regional syncline. This folding is responsible for the sinuous outcrop pattern of the Hook Mountain Basalt.

1.2.3.3 Glacial Geology

Continental glaciation occurred in the Passaic River Basin during the Kansan, Illinoisan, and Wisconsin stages. The present landscape of the basin was created by the last stage of the Wisconsin glacier. All evidence of earlier glaciation was removed along with any weathered bedrock during this final period. The terminal moraine deposits delimiting the southern extent of Wisconsin ice are located south of the project area (Fig. E.1.1). A blanket of ground moraine was deposited directly over the bedrock as the glacier retreated. Both terminal and ground moraines consist of unsorted, heterogenous material ranging in size from clay to boulders. These overlying deposits take on the petrographic character of the underlying bedrock. In the Piedmont area along the tunnel route, the clastic material comprising the moraine is predominantly red sandstone with lesser amounts of metamorphic and igneous rocks entrained by ice and brought in from outside the province.

As the glaciers retreated meltwater was dammed by the Watchung and Highland Mountains creating glacial Lake Passaic. Deltas were built where streams flowed into the lake and varved silt and clay were deposited in the lake proper. These low permeability lake deposits restrict percolation which has led to the formation of wetlands in the middle and upper Passaic River Basin. The melt-water streams deposited well sorted, stratified silt, sand and gravel along the length of their valleys which became deeply incised into the weaker rocks of the basin and are up to 300 feet below the current stream bed elevations.

1.2.3.4 Commercial Geolouv The primary mineral industry production in the basin involves construction materials. Commercial sand and gravel operations producing from the glacial outwash are located generally south of the project limits. Several trap rock quarries are still actively producing from the Watchung Mountain basalts while several abandoned quarries are also present along the project alignment. Clay pits supporting brick manufacture have been operated in historic times. One such pit reportedly operated near the Spur Tunnel in the vicinity of Hole C-118. Commercial deposits of copper ore were once worked in the area and one of the first mines to be operated by Europeans in the US, the Schuyler mine circa 1700, is adjacent to the tunnel alignment. It is interesting to note that the first use of steam powered machinery in the North America was associated with the development of this mine. These deposits are exhausted and no longer commercially viable.

1.3. SITE INVESTIGATIONS

1.3.1 Subsurface Investigations, Rationale and Scope A project of this magnitude requires a substantial amount of exploration if one is to have a reasonable prospect of characterizing the conditions to be encountered along its 20.5 mile length. The U.S. Committee on Tunneling Technology published a study in September, 1985 that included, as one of its main findings, the level of exploration that was warranted for tunneling projects. This study included 67 tunnel projects in non-mountainous terrain where the overburden depth was not abnormal. The parameters which were used in this study to determine the impact of the level of exploration were linear feet of boring per running foot of tunnel versus some measure of cost growth. The original engineers' estimate and the bid price were used as the base line against which cost growth was measured. The study concluded that at the level of 1.5 ft of boring per running ft of tunnel, the risk of increased construction expenditures due to cost overruns reach a minimum. As a practical matter it may not be possible or even necessary to obtain this level of exploration, however, this study has been kept in mind while establishing the drilling program for the Passaic River Basin Tunnel Project. At present there are over 110,800 ft of exploratory borings planned before preparation of plans and specifications. Drilling footage will ultimately be based on whatever drilling is necessary to perform geologic correlation. There are several benefits of an adequate exploration program. Sufficient coverage provides a more "level footing" for bidders so less risk is taken and more competitive bids are tendered. In this situation the spread on the bids should be lessened, the interest in competing for the bid should be increased, and the risk to the sponsor should be lessened. This latter issue is particularly important since the Corps is entering into a cost sharing agreement with the local sponsor. Overruns on the agreed on price will be a major item of contention should they occur. Another benefit of adequate exploration is the ability to defend the original design and estimate against unwarranted change of condition claims. Table 1.3.1 presents a summary of the factors which lead to cost overruns in the USCTT study.

At the heart of the exploration program is the location and definition of all buried valleys that may exist along the alignment. The interception of a buried valley with a TBM is totally unacceptable relative to safety and impact on costs and schedule.

The holes drilled to date have been located as much as possible directly over the proposed alignment of the tunnels and at the work shaft and structure locations. However, availability of open land is very limited in many areas and numerous holes had to be offset from the ideal location. Additionally, in much of the lower reach of the project, rights-of-entry were unavailable.

Downhole video camera studies have been conducted on a selected number of borings to determine the orientation of discontinuities. Along with this, water pressure testing has been accomplished on most borings in an effort to establish the quantity of water inflow that might be expected during construction.

TABLE 1.3.1
PROBLEMS AND CLAIMS* REPORTED FOR MINED TUNNELS

	Problems (% of tunnels)	Claims (%of tunnels)
Blocky/slabby rock, overbreak, cave-ins	38	16
Running ground	27	9
Flowing ground	5	4
Squeezing ground	19	8
Spalling, rock bursts	6	4
Groundwater inflow	33	6
Noxious fluids	6	4
Methane gas	7	2
Existing utilities	1	0
Soft bottom in rock	2	2
Soft zones in rock	4	2
Hard, abrasive rock (TBM's)	5	2
Face instability, rock	5	1
Roof slabbing	4	1
Pressure binding (equipment)	4	4
Mucking	5	2
Surface subsidence	9	2
Face instability, soil	11	5
Obstructions (boulders, piles, high rock in invert, cememnted sand)	12	11
Steering problems	4	0
Air slaking	1	0

*The word "claim" encompasses <u>all</u> requests for extras as a result of an unexpected subsurface condition.

1.3.2 <u>Laboratory Rock Mechanics Testing Rationale</u> The rock mechanics testing for the Passaic Tunnels began during the Feasibility Study. This and subsequent testing has involved a wide range of both field and laboratory techniques. The majority of the testing has been performed on 2½ inch diameter HQ core.

In general, for tunneling jobs, the International Society for Rock Mechanics suggests that 8 characteristics of a rock mass be investigated as part of the evaluation. The characteristics and some of the available tests are as follows:

Characteristic	Suggested Tests
a. Hardness	Schor Scleroscope, Schmidt Hammer
b. Strength	Unconfined Compressive Strength, Brazilian Test, Punch Shear, Point load, Fracture Toughness
c. Texture	Quartz Content, Texture Coefficint, Grain size and Shape
d. Drillability	
(Translational)	Goodrich Drillability, Seviers "J" number, VOEST-Alpine Rock Cuttability Index, Taber Abradability
(Penetrative)	NCB Cone Indenter Index, Morris Drillability, Handewith Test
(Percussive)	Rock Impact Hardness Number, Protodyakonov Test
e. Abrasiveness	Goodrich Abrasivity, Surchar Abrasivity, LCPC Abrasivity, Taber Abrasivity
f. Geologic Structure	CSIR or NGI Rock Mass Classification Systems
g. Seismic Properties	P&S Laboratory Velocities, Field Seismic Velocities
h. Swelling Properties	Slake Durability, Free Swell Test, Swell Pressure Test

The results of the testing program are presented in detail in section E-3 TUNNELS.

1.4 SEISMIC STUDY

The Passaic project is located in a moderately seismic area that is subject to strong shaking from infrequent earthquakes. Experience with more seismic areas of the world has shown that underground structures are very resistant to damage caused by earthquakes, however, the surface works are a vital element of the project and will have to be designed to resist an appropriate level of shaking. At the current stage of design key project features have been analyzed including an earthquake load case using the pseudo-static method and an acceleration of 0.10 g. A more detailed, dynamic analysis using peak motions will be required in the next stage of design. The seismological evaluation presented in Appendix E.3.1 is the first step in this process. Several similar studies for nuclear power plants and Corps of Engineers Dams in the Northeast have been prepared and were consulted during the preparation of Appendix E.3.1.

PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

SECTION 1 GEOLOGY

FIGURES

PASSAIC RIVER BASIN TUNNEL, NEW JERSEY

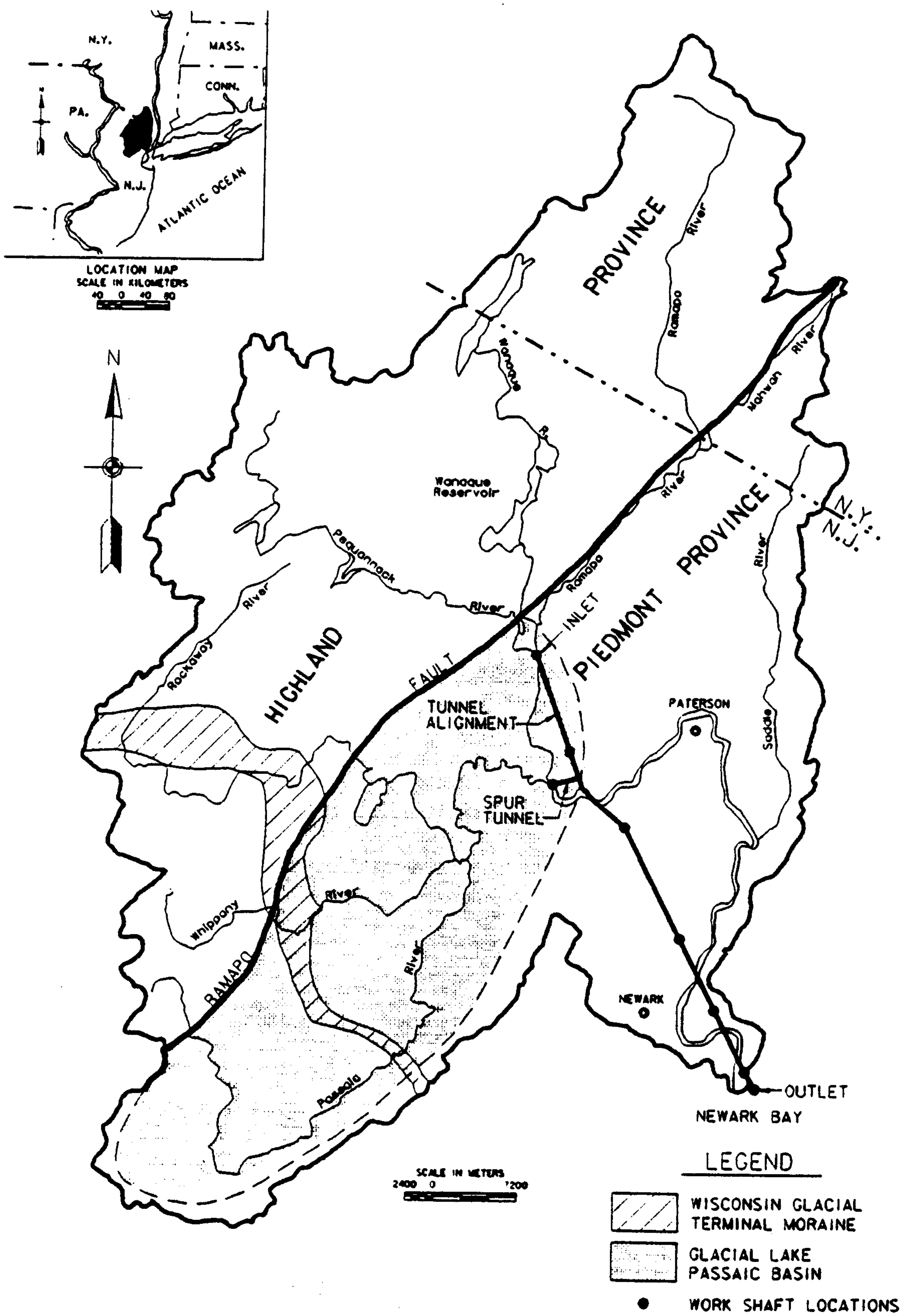

Figure E.1.1 :Physiography of Passaic River Basin and location map.

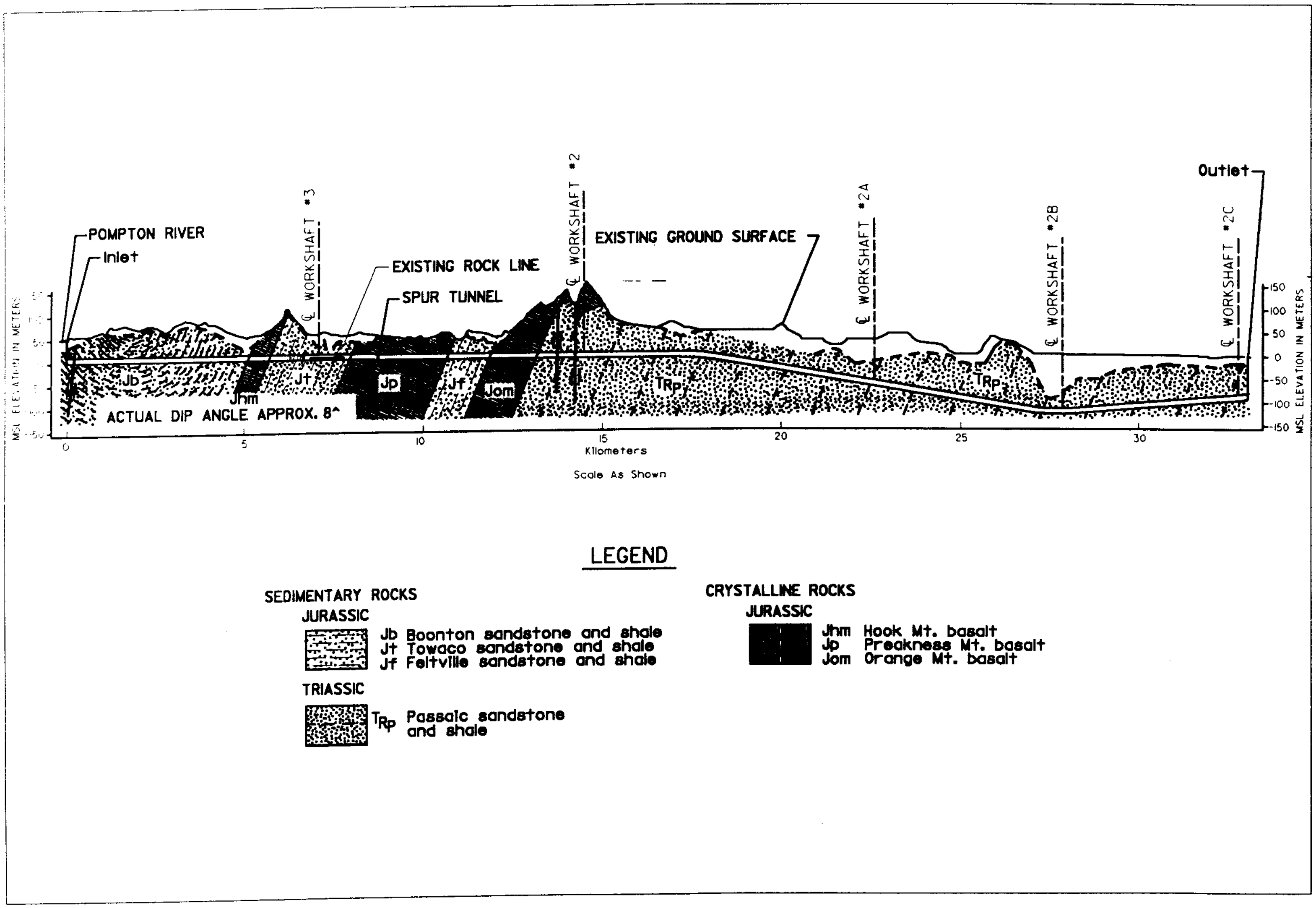

Figure E.I.2 : Geologic profile along Passaic River Basin Tunnel alignment.

AGE	THICKNESS (FEET)	DESCRIPTION
Recent	0-40	Alluvium: Reworked silts, sands, and clays.
Pleisto-cene	0-300	Glacial Deposits: Basal gravel till in buried valleys, varved silts, upper deposits are fluvial outwash sands, & silts w/ some gravel.
JURASSIC	740	Boonton Formation: Interbedded sandstone, siltstone, & shale. Cyclic deposits with fining upwards sequences going from sandstone and shale red beds through grey shale.
	280	Hook Mt. Basalt: Blocky unit consisting of at least four flows.
	1,125	Towaco Formation: Interbedded sandstone, siltstone, & shale. Cyclic deposits with fining upwards sequences going from sandstone and shale red beds through grey shale. Cycles range from 80 to 330 feet in thickness. Basal contact contains basalt breccia.
	1,080	Preakness Mt. Basalt: Consists of 2 units divided by a 9 foot thick layer of sandstone. The upper unit is made up of at least 2 flows with vesicular basalt at the top. Slow cooling of the thick flows produces a gabbro like texture in places. Lower unit contains at least two flows.
	435	Feltville Formation: Mainly fine sandstone with trace amounts of coarse sandstone. 38% red shale and 2% black shale.
	515	Orange Mt. Basalt: Basalt layer consisting of two flows. Each flow is massive and blocky at teh base and vesicular at the top. There is a three foot thick pillow basalt at the base of the unit.
TRIASSIC	6,750	Passaic Formation: Red beds, predominantly sandstone and siltstone, with occasional arkosic layers near the top of the formation and shale beds. Shale beds are generally less than 5 feet thick but may be much thicker near the base of the unit.

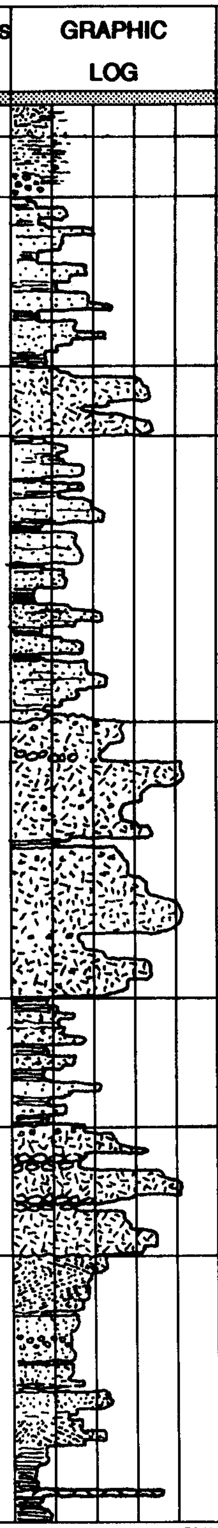

Figure E.1.3. Generalized Geologic Column for the formations of the Newark Basin which are involved in the Passaic River Tunnel

PASSAIC RIVER BASIN TUNNEL, NEW JERSEY

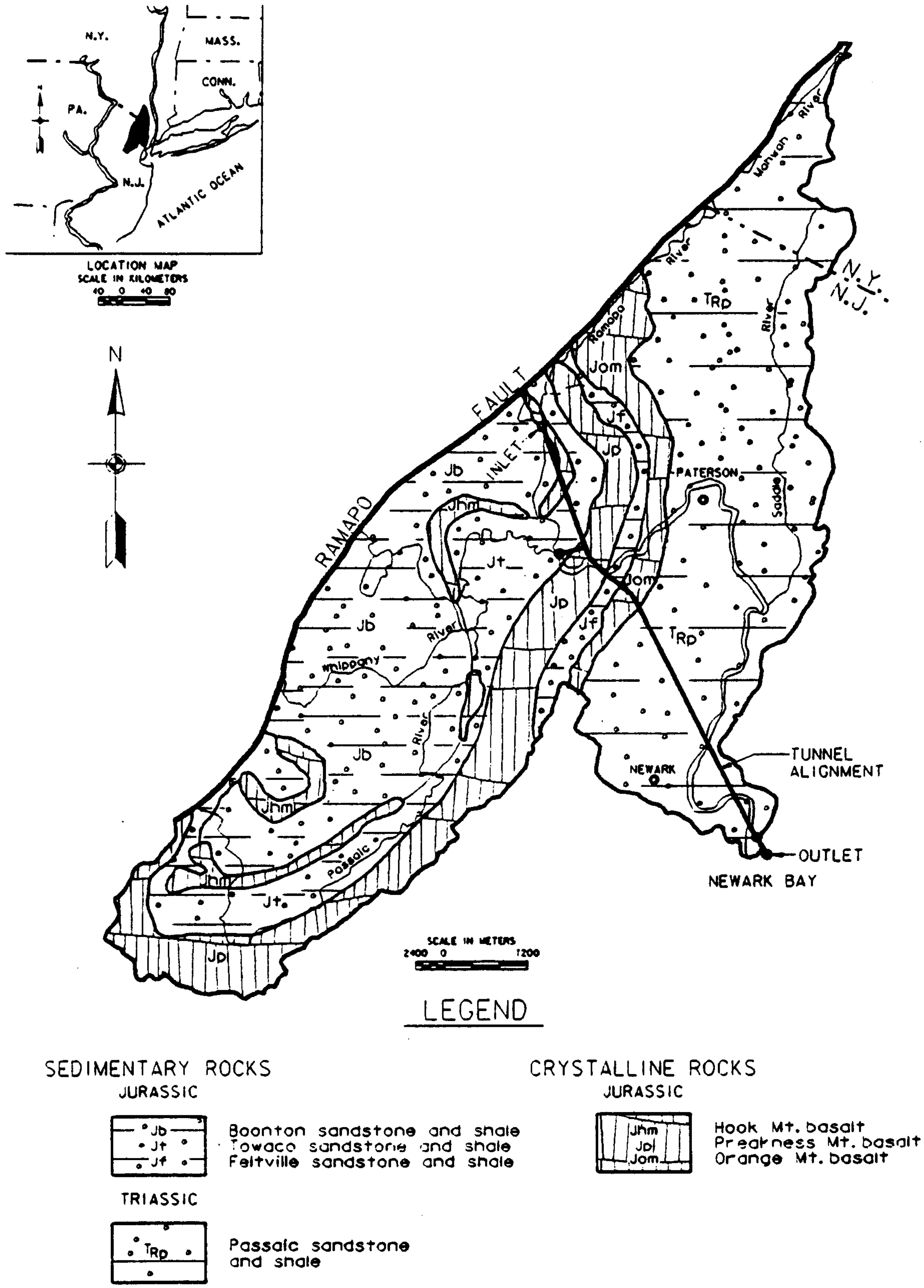

Figure E.I.4 :Geologic Map

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.1.1

SEISMIC STUDY

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ATTACHMENT E.1.1
SEISMIC STUDY

I. PURPOSE. The Passaic River Flood Damage Reduction Project is located in a moderately seismic area that is subject to strong shaking from infrequent earthquakes. Experience with more seismic areas of the world has shown that underground structures are very resistant to damage caused by earthquakes, however, the surface works are vital to the operation of the project, and will have to be designed to resist an appropriate level of shaking. A dynamic structural analysis using response spectra may be required in the next stage of design. The seismological evaluation presented here is the first step in this process. The major source zones which are likely to cause shaking of the project area are described and recommended peak motions from each of these determined. Several similar studies for nuclear power plants and Corps of Engineers Dams in the Northeast have been performed and were consulted during the preparation of this study.

II. SCOPE. This attachment to the geotechnical appendix was prepared according to the general guidelines presented in the most recent draft of ER 1110-2-1806, dated 30 May 1995, Earthquake Design and Analysis for Corps of Engineers Projects. Paragraph 4. *Policy*, of this ER states, "The seismic design for new projects.....should be accomplished in accordance with this regulation. This regulation applies to all projects which have the potential to malfunction or fail during major seismic events and cause hazardous conditions related to loss of human life, appreciable property damage, disruption of lifeline services, or unacceptable environmental consequences." Further, paragraph 5.c. *Evaluation* requires, "Detailed site explorations, site specific ground motion studies and structural analysis...only for projects in zones 3 and 4, or for zone 2A and 2B project when seismic loads control the design." The Passaic project is in zone 2A (see Figure 1) so the requirements are not clear cut. For non-impoundment flood control projects a detailed seismic analysis is not usually specified because of the improbability of a major earthquake occurring at the same time as a major flood. In spite of this, the consequences of failure of some of the structures for the Passaic Tunnel do warrant a more detailed analysis. Because of the nature of this project and the consequences of failure, it is probably appropriate to use an operating basis earthquake (OBE) for future analysis. The OBE should be based on the design life of the project. The site parameters are determined by the deterministic method using the USGS maps given in the new ER. As a check of this method a probablistic analysis is also presented for an OBE. A 100 year design life is consistent with the economic studies and has been selected as appropriate for this study.

At this stage of study only the peak site motions are to be determined for the OBE. The detailed structural response analysis utilizing response spectra for these values for individual structures is deferred until the next stage of design. In the structural appendix, critical structures have already been analyzed using a pseudo-static analysis for an OBE with a 50 year return period and a non-flood stage loading case. This was consistent with the seismic design criteria required by the previous version of ER 1110-2-1806. Figure 2 is the seismic zone map from the old ER that was used to select the pseudo-static design value of 0.10 g acceleration. This was not the critical load case for any of the structures analyzed so greater seismic resistance for all of these structures is evident.

III. MODES AND CONSEQUENCES OF FAILURE. The tunnel shafts that are excavated through deep soil cover are likely to be most vulnerable to earthquake damage. Differential movement between the soil and rock at their contact may result in liner failure at this location. A liner system which is separate for the soil and rock is likely to be most successful in resisting this condition. In general the proposed designs for the shafts satisfy this requirement in that the shafts through the soil are larger in diameter than the shafts in rock. By inspection the differences in diameter (1 foot or more) would accommodate the range of motion that would be expected in the OBE earthquake. A more rigorous analysis based on the OBE is warranted in the next phase of design. The inlets and outlet structures are a special case of shaft. The control structures at the surface change the dynamic response of these structures making the analysis more complex. Failure could be regarded as unacceptable and the structures may have to be designed to the MCE. This decision should be made after further consideration of the costs and consequences. Surface structures such as the Pequannock and Great Piece Weirs and operational buildings will also be prone to damage and should be designed to appropriate OBE levels.

IV. GEOLOGY

A. CRUSTAL INVESTIGATIONS. Knowledge of the structure of the crust of the earth is used by seismologists to determine earthquake hypocenter locations and parameters from seismographic records. Many of these crustal studies are conducted by analyzing seismic returns from known sources, for example, large blasts. Many authors have studied the crustal structure of the eastern United States (References 21, 26, 29 and 53) with a fair degree of consistency between their models. Layers are classified on the basis of seismic wave velocities which in turn are correlated with gross lithologies. Lateral variations in structure naturally exist, however, seismic refraction and other geophysical studies demonstrate that in the eastern United States these variations are quite small. Table 1 is currently used by the Northeastern U.S. Seismic Network.

B. GEOLOGIC HISTORY. The geology of the region around the Passaic Project is linked to and controlled by the tectonics of the Appalachian system. Table 2 (Reference 35) provides a summary of this tectonic history. The main text of Appendix E provides detailed information on the geologic history of the project which will only be summarized here.

The project is located in Mesozoic Aged rift valleys. According to many authors, this activity represents the point at which the Atlantic Ocean changed from a closing system to an expanding ocean basin which eventually lead to a more quiescent period which has continued to the present.

West of the rift valleys is extensive and intense folding and faulting in the orogenic belt of western New Jersey and Eastern Pennsylvania. This area has probably received the greatest accumulation of Paleozoic sediments. Periodic uplift and erosion of this area has provided much of the clastic filling of the Allegheny Synclinorium.

Adjacent to and west of the Paleozoic Orogenic Belt lies what Eardley (Reference 14) labels the Central Stable Region. The Central Stable Region is underlain by a Precambrian basement of high grade metamorphic and plutonic rocks, which are clearly related to the Grenville province of the Canadian Shield. This correlation is supported by exposures in the Adirondack Mountains, along the Frontenac Axis, and in the St. Francois Mountains. A profound unconformity marks the boundary between these ancient Precambrian rocks and the overlying late Precambrian Caloctin greenstone, Ocoee series, and Mt. Rodgers Volcanics. It is these rocks that contain the first evidence that the Appalachian region was systematically different from the rest of North America. Figure 3 shows the major geologic features in the region surrounding the project. Glaciation of the region during the Pleistocene was the last major event which is reflected in the geology of the region.

C. PLATE TECTONIC THEORY. The advancement of the plate tectonic theory has provided a broad framework by which the dynamics of the earth may be explained. The continuing expansion of the Atlantic Ocean Basin, which is estimated to be ≥2cm/year in the North Atlantic (see Reference 33), induces a horizontal compressive force on the Eastern United States. This west-north-westerly trending stress field is probably one of the primary engines which drive the seismicity currently observed in the region. This is evidenced by fault plane focal mechanism studies (see Paragraph IV.G.), which are predominantly compressional for the eastern United States, and in situ stress measurements which are discussed in Paragraph IV.D. Local variations in this pattern occur as a result of differing geologic conditions, such as pre-existing fault orientations, localized uplift or subsidence, residual stress fields and anisotropic lithology.

However, these variations are consistent with Plate Tectonic Theory when these localized conditions are taken into account (see Paragraph IV.E. on seismogenic mechanisms).

A discussion of Plate Tectonic Theory and how it relates to mid-continent diastrophism (epeirogenic tectonics) would not be complete without mentioning the work of Burke and Dewey (1973). They brought forth a "cause and effect" mechanism for rifting where crust located within an intraplate region fails due to a mantle-derived plume or "hot spot", in the form of a triple-junction. If rifting continues long enough, an ocean basin is produced. According to Burke and Dewey (1973), the Grenville Province represents reactivated basement which developed following continental collision (a classic Wilson cycle) approximately one billion years ago. The destroyed triple junction is presumably marked by a suture lying beneath cover rocks to the east.

Many proposed triple junctions are believed to have become inactive before an ocean basin was created, possibly as a result of changes in deep-seated convective processes. One such example of a failed triple junction is centered in the Great Lakes region near the eastern shore of Lake Superior (Burke and Dewy, 1973). In this model, the mid-continent gravity high corresponds with one failed rift arm, another runs through to the Michigan Basin (Hinge et al., 1972, 1975), and a third extends out toward Hudson Bay.

Aulacogens are long-lived, deeply subsiding troughs which evolve from narrow grabens to broad downwarps, some of which form during initiation of the triple junction. These can ultimately undergo a final compressional stage of folding and faulting, along with periodic fanglomerate and alkalic basalt extrusion (Burke and Dewey, 1974). This concept has been applied in explaining many features including the Raritan Embayment as failed arms of a triple junction.

It appears that under the proper tectonic conditions, reactivation of these paleo-rift zones (or paleo-sutures) could occur and be reflected in terms of higher than average seismicity. Figure 4 shows a Plate Tectonic Map of the world.

D. IN-SITU STRESS MEASUREMENTS. Measurements of in situ stress can be acquired in a number of ways. The stress field orientation can be inferred from focal mechanism studies (see Paragraph IV.G.), or it can be measured directly by a hydrofracture technique or a strain relief overcoring technique. The magnitude of the maximum and minimum principal stresses can be determined using the direct measurement technique. Figure 3 and Table 3, which were prepared using Reference 36, summarizes several in situ stress studies. These studies indicate a good deal of complexity east of the Appalachian Front, becoming more uniform west of this area. The test results are highly influenced by the site conditions. Tests on intrusive bodies may reveal a residual stress domain which differs greatly from that of the host rock. Surface measurements may not be representative of the stresses at depth. Fault plane solution data are probably more indicative of the tectonic stress regime than the shallow test data shown on Figure 5. The magnitude of the stresses which are indicated by these tests are quite high in many instances, indicating the potential for earthquakes.

E. SEISMOGENIC MECHANISMS. Perhaps the main problem in evaluating the seismicity of the eastern United States (that is east of the Rocky Mountains) is the absence of surface expression of the features which are producing the seismic activity. To date, causative faults which are so readily observable in the west (e.g., San Andreas), are, at best, poorly exposed in the east. For this reason, the nature and location of the active faulting must be evaluated through fault plane and focal mechanism studies (see Paragraph IV.G.) and by evaluating earthquake distribution. The mechanisms producing this faulting are also shrouded in mystery with the result that several hypotheses exist. It is no doubt true that no single hypothesis explains all the seismic activity of the eastern United States and adjacent parts of Canada, and for this reason, several of these hypotheses are presented herein. This topic is interrelated to the Plate Tectonics and in situ stress measurements discussed in Paragraphs IV.C. and IV.D. A further discussion of where these seismogenic mechanisms are thought to be active is contained in Paragraph IV.F.

1. One of the most widely cited seismogenic mechanisms for intraplate or epeirogenic tectonics involves the reactivation of pre-existing zones of weakness. Some of these ruptures originated as long ago as the late Precambrian (>600 million years before present). Many of them are associated with continental rifting or aulacogens as discussed in Paragraph IV.C. If these zones are suitably oriented to the present day regional stress field and the shear stress on the fault plane is large enough, reactivation will occur.

Some seismicity, notably the Cape Ann, Massachusetts area and the Grand Banks area, is located near the end of major oceanic transform faults. Other zones are associated with Triassic basins. The Ramapo fault in New Jersey and New York is a fine example of a favorably oriented Mesozoic rupture which is associated by some authors with present day seismicity. This association is not conclusive in the minds of many because of inaccuracies in location of hypocenters and a relative scarcity of data. A study by Dames and Moore tried to resolve the question of whether the Ramapo was a capable fault by trenching and study of the actual shear plane of the fault near the surface. This study found that the secondary mineralization along the fault had been deposited at least 2 million years ago and that it had not been subjected to any stress. This has been interpreted as sufficient evidence to say that the Ramapo is not capable. Whether movement might occur at depth and not be reflected in these near surface minerals could be argued but it has been the official Corps interpretation in several other similar studies that the Ramapo is, at most, only mildly active.

2. Some seismicity in this region has been linked to stress amplification around intrusive bodies. This hypothesis is supported by a correlation of seismic activity with near circular, positive gravity and magnetic anomalies. This hypothesis is based on marked differences between the elastic properties of the host rock and the intrusives. Under the regional stress field, the two rock types deform to different degrees. The more elastic rocks compress differentially, resulting in a stress concentration in the less elastic rocks. This situation has been compared to the way a stress field is modified around a hole in a plate. Some authors feel that sufficient stress concentration can be generated in this way to rupture the less elastic rocks, while the more elastic rocks deform plastically. On a large scale, this condition can be observed around the Adirondack Dome in the observed stress field. The major principal stresses appear to vary in response to this large, isolated, relatively inelastic mass. The extent to which this type of regional anisotropy affects the occurrence of earthquakes is conjectural, but it is very likely that it does play some part. A study by Boston Edison Company of New England earthquakes using field velocity measurements and rock mechanics studies found that near the Ossipee Pluton in New Hampshire, a 20 percent increase in the maximum principal stress could be produced.

3. Post glacial rebound (isostatic uplift) is somewhat controversial as a seismogenic mechanism. Some authors, especially prior to the advent of Plate Tectonic theory, viewed unloading by continental ice sheets as a major stress source and earthquake driving mechanism. More recently, it has been recognized that the seismicity does not correlate completely with the limits of glaciation. It is also felt that sufficient time has elapsed since the last continental glaciation in the area (Wisconsin age 75,000 to 11,000 years before present) to have accommodated much of the rebound. This is not to say that post-glacial rebound is not contributory to present day seismicity. Post glacial pop-ups (vertical rock bursts) and other rock squeeze features are well documented in New York state (Chippewa, Rochester, Buffalo) and several authors propose a superimposed stress field due to glaciation, which contributes to the seismicity by producing stress concentrations. The Buffalo, New York/Hamilton, Ontario seismicity may be related to this mechanism.

4. Human activities have been connected with some isolated seismicity. Several reservoirs in the east have been monitored with microseismic networks during and after filling. Clark Hill, Jocassee, Montecello and North Anna reservoirs were all associated with increased microseismic activity. An interesting occurrence is described in Reference 34. Surface quarrying at Wappingers Falls, New York, is credited with triggering numerous low magnitude, shallow focal depth (0 to 1 1/2 km) earthquakes. It is felt that the quarrying unloaded the area, reducing the minimum principal stress. This resulted in increasing the deviator stress sufficiently to cause faulting of previously intact rock to occur. While significant from a seismological standpoint, none of these mechanisms are thought to be capable of producing damaging earthquakes. Other reported earthquakes are probably the result of blasting, especially around quarry operations.

5. Many areas in the Northeast exhibit significant subsidence and uplift. Areas of subsidence include the Salisbury Embayment in the Chesapeake Bay area, Sandy Hook, New Jersey, the coastal area of the Connecticut River Valley, Southern Maine, Passamaquoddy Bay, Maine, and the LaMalbaie area of the St. Lawrence River. Uplift is still occurring in the Adirondack Mountains. Subsidence along the coastal areas may be caused by loading from thickening of the sea floor adjacent to the continental margins and deposition of sediments derived from the contents. The uplift of the Adirondack Dome is not so easily explained. It may be more correct to label these vertical movements as products of seismic activity, rather than seismogenic mechanisms, however, seismic activity is observed at locations where subsidence or uplift are occurring.

6. The May 31, 1908 earthquake near Allentown, Pennsylvania was felt over a very small area, (80.5 square km), but was quite strong (maximum intensity VI). It is speculated that it was caused by a large roof collapse in a limestone cavern. This event is not significant from a tectonic standpoint, and is only explained to eliminate it from further consideration.

7. An informal suggestion by some researchers attributes some low level seismicity within the area of the Salines Formation to the development of salt domes and subsequent rupture of the surrounding rock. This has not been substantiated by any field investigations and is not regarded as a likely source of strong shaking.

F. DISTRIBUTION OF FAULTS. In eastern North America, faults have been located by surface mapping, geophysical methods, and seismicity. Figure 6 shows a map of the larger faults in the region around the project. The majority of surface faulting parallels the geologic grain of the Appalachian system.

Northeasterly strikes predominate with variable dips. Much of the area to the east of Eardley's Central Stable Region (see Paragraph IV.B) is typified by thrust faulting. These regional faults are believed to flatten at depth and extend under the Valley and Ridge province as a regional decollement separating younger Paleozoic strata from sedimentary and crystalline basement rocks (see Reference 35). Further east in the Piedmont, the Triassic basins are typically bounded and cut by normal faulting. Geophysical investigations have demonstrated that many of these features extend to great depths. Basement faults which are not reflected at the surface have also been detected by geophysical methods which include profiling and gravity studies. The Clanendon-Linden Fault in western New York is a notable example of this type of fault.

To date, no surface rupture has been linked to seismic activity in the eastern United States. Hypocenters are normally in the mid to upper crustal depth range (5 to 10 km) and offsets of Cenozoic age have only been demonstrated by seismic profiling. The Ramapo Fault is located within 2 miles of the Pompton Lakes Inlet for the tunnel.

G. FAULT PLANE AND FOCAL MECHANISM STUDIES. Here, also, there is limited data available for the eastern United States. Since active faulting in this area is not well defined by surface features, the type and orientation of causative faults must be deduced by analyzing seismic records of earthquakes. In order to construct a focal mechanism solution for a particular earthquake, suitable records must be analyzed from a number of seismographs which must be well distributed around the epicenter. The construction is based on P wave first arrival times, and whether the first arrival is dilational or compressional in nature. The angle i_h, at which the ray arriving at a particular station leaves the earthquake focus is given by the relationship:

$$\text{Sine } i_h = V_h \text{ [dT/d Delta]}$$

where V_h is the wave velocity and [dT/d Delta] is the slope of the time travel curve. The resulting angle i_h is strongly influenced by the focal depth and the velocity model used (see Paragraph IV.A). The data is plotted as a stereographic projection on the lower hemisphere of what is known as the focal sphere. The resulting pattern of dilational and compressional segments is characteristic for a particular fault type and orientation. The planes which form the boundaries between dilational and compressional segments are called nodal planes.

Each focal mechanism solution will have two nodal planes, one of which corresponds to the fault plane orientation. Determining which nodal plane is the most likely fault plane solution is often difficult and must be based on the regional structure. The nature of the faulting is determined by the distribution of dilational and compressional components. If the center of the focal mechanism plot is in a compressional domain, the faulting is reverse; if it is dilational, the faulting is normal. An angle between the strike lines of the nodal planes indicates that there is a strike slip component of the fault plane. The number of fault plane focal mechanism studies available for the eastern United States is still quite limited. Some of this information, gathered from various sources, is presented in Table 4 and on Figures 7 and 8. This data indicates that a majority of earthquakes in the eastern United States are the result of reverse fault movements. Many of these faults are high angle and exhibit some amount of strike slip. Rare instances of normal faulting are indicated. In some cases, this is explained as tear faulting resulting from major strike slip movement (New Madrid). An interesting hypothesis for the normal faulting observed at Lake Hapatcong, New Jersey proposes that differential movement on unhealed, oceanic transform faults may serve to localize tensional stresses where they intersect the continental margin. This local effect is not observed farther inland. In many cases, where a focal mechanism study is not possible, epicenter locations and focal depths can be computed. This is also very useful information for defining the seismicity of an area.

H. SURFACE EFFECTS ON THE TRANSMISSION OF SEISMIC WAVES. For purposes of illustration, assume a seismic wave train with a given energy originating in rock. When this wave grouping enters a soil foundation, (since seismic velocities for soils are typically low) the incoming wave lengths are shortened. In order to maintain the energy in the system, the waves, in effect pile up on one another, increasing their amplitude. In passing through the soil, the waves experience high friction losses, most profoundly in the shear wave components and at the higher frequencies. The same wave train passing through a rock foundation would lose energy through particle friction at a slower rate than in soil, and for both cases the frictional losses are lower in the saturated condition. Records obtained from the Pacoima Dam site during the San Fernando earthquake on 9 February 1971, indicate the effects of topography on earthquake motions. In mountainous terrain, some amplification takes place at the higher elevations. This would be expected for surface works around work shaft 2.

V. SEISMOLOGY

A. HISTORY OF SEISMOLOGY IN THE NORTHEAST. Reports of earthquakes in the Northeast go back in the St. Lawrence area to 1534. Early accounts of earthquakes are limited to personal journals, diaries, and newspaper reports from which only intensity data can be interpreted. Table 5 presents a chronological listing of earthquakes around the Passaic project. The first seismic station in North America to use a seismograph with continuous recording was at Toronto, Ontario in 1897. By 1901, there were similar stations at Baltimore, MD. and Philadelphia, PA., followed in 1904 by stations at Washington, D.C. and Cheltenham, MD. These instruments typically had a low magnification, permitting the detection of only the stronger earthquakes. This type of development continued during the 20th Century with installation of a limited network of seismographs of greater sensitivity in the 1930's. These provided valuable information on seismic wave velocities and improved epicenter location accuracy. In the 1960's, the World-Wide Standard Seismographic Network was established with the installation of calibrated, high-gain instruments with excellent time control. Most recently, the use of micro-earthquake detection devices, with the ability to detect low magnitude events, has aided in defining seismic boundaries, determining focal mechanisms and extending the magnitude-recurrence curve. These instruments have been used in permanent installations and as portable stations to study specific sites.

Since October 1975, the Northeastern U.S. Seismic Network has been publishing quarterly bulletins on the seismicity of the northeastern United States. The network, which is coordinated by the U.S. Geological Survey, represents the combined efforts of numerous universities, state, and federal agencies with input from the Earth Physics Branch, Department of Energy, Mines and Resources, Ottawa, Canada. In 1984, a total of 162 seismic stations were in operation gathering data for inclusion in the bulletins. This project is being funded through state and federal programs concerned largely with the siting of nuclear power producing facilities.

B. SEISMIC ZONE MAPS. The recommended method for design of Corps of Engineers projects with regard to earthquake loads involves a deterministic approach and the use of seismic hazard maps. These maps are still used in most national building codes, largely because of availability, uniformity, and relative ease of application. Generally, these maps are subdivided into zones based on some seismic parameter, typically peak particle velocity or a seismic coefficient, which is expressed as a decimal fraction of the gravitational acceleration. For a project within a certain zone, the specified seismic coefficient for that zone is used in computing expected earthquake loads. These values are normally based on the probability of non-exceedance over some time period and do not necessarily represent maximum credible earthquake values.

Figure 2 is the seismic zone map used under the old version of ER 1110-2-1806. The design of the project features to date has included an earthquake load case using an acceleration value of 0.10 g from this map. The new, draft ER has updated versions of Figure 2 which are included as Figures 9 and 10. These present pseudo-acceleration maps for 50 year and 250 year return period earthquakes. Based on Figure 9 the pseudo-acceleration that should be used for the OBE at Passaic is 0.15 g.

C. ISOSEISMAL MAPS. Isoseismal maps present contours of intensity. Intensity is a subjective value dependant upon human judgment and observation. The value of intensity and the accuracy of the resulting map is controlled by population densities, building construction and other human factors which have not held constant through recorded time. Attempts have been made by researchers to minimize these effects, and the intensity scale itself is designed to reduce errors, especially at higher values. In any case, for earthquakes which occurred in the study area prior to the first decade of the 20th Century, intensity data is the only record which exists. This means that most of the violent earthquakes of the past were not instrumentally recorded and must be evaluated on the basis of contemporary descriptions, which may be presented as isoseismal maps.

Since isoseismal maps are drawn directly from observed surface effects, an isoseismal map is a picture of how earthquake energy is dissipated from its source. This unique feature has a major advantage over empirical formulae in depicting attenuation. Formulae have been derived using both theoretical relationships and observed data. They are, therefore, limited by fixed observation points and imply a radial attenuation pattern which is not true to life, although they have the advantage of being more generally applicable in determining peak motions at a site.

Shock waves travel outward from an earthquake's hypocenter in all directions. In a homogeneous, isotropic medium, they spread like ripples on a pond; however, each geologic contact that the wave front hits modifies the signal and distorts the radial propagation. The complexity of the isoseismal is controlled by many parameters: magnitude of the earthquake, focal mechanism, focal depth, source location dimensions and configurations, regional geology, regional stress patterns, topography and surface deposits. It is reasonable to assume that the maximum earthquake will occur in the same seismic zone as the maximum historic earthquakes, provided the period of observation is long enough. This assumption is verified by seismicity in more active areas of the world and microseismic data in the study area. This means that most of the controlling factors mentioned above will have the same effects on the maximum credible earthquake as they had on the maximum historic earthquakes. Only focal depth and source location remain variable within the limits outlined in Paragraphs IV.F. and IV.G.

Isoseismal maps from some of the largest historical earthquakes that have effected the region have been collected from various sources and are presented on Figures 11, 12, and 13. From these and various other studies, the maximum intensity experienced any where in the Passaic project area in historic time was between VI and VII. Quoting from the Intensity Scale, this is described as follows:

> VI. Felt by all, many people are frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII Rossi-Forel Scale).

> VII. Everybody runs outdoors. Damage is negligible in building of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars. (VIII Rossi-Forel Scale)

While this does not necessarily represent the maximum credible value, it is probably representative of an operating basis earthquake.

Another type of isoseismal map is presented in Reference 7 and included as Figure 14 in this report. This map represents cumulative maximum intensity over a fixed time period (1928 through 1973). This type of map gives a good idea of intensities which might be experienced at a particular site over a comparable time period. Its shortcoming lies in the length of time it represents and the variability of seismicity. The effects of a maximum credible earthquake are not well established for the eastern United States. The project area was subjected to an Intensity of VI during this time period.

D. ATTENUATION. A seismic impulse, while traveling from its source to a site, loses energy through geometric spreading and anelastic attenuation. Energy loss through geometric spreading is a function of distance from the epicenter. Herrmann has demonstrated that the coefficient of geometric spreading changes at a horizontal distance approximately equal to the focal depth of the earthquake (see Paragraph IV.G.). This distance varies, but is generally taken to be 15 kilometers for the eastern United States. At this distance from a Seismic Source Zone (see Reference 15) the type of analysis changes from a near-field to far-field. The near-field in the eastern United States is typified by high accelerations, high frequencies, and short durations.

Anelastic attenuation occurs when energy is absorbed by the transmitting medium. The amount of energy absorbed is dependent upon the elastic properties of the transmitting medium and the frequency of the seismic wave. The higher the frequency, the higher the absorption. As a result, at increasing distance from the hypocenter, the average frequency of the seismic wave train decreases.

As can be seen from the above discussion, attenuation is dependent upon the region of interest. The values of attenuation coefficients are not interchangeable from one region to another. A prime example of this can be seen in eastern versus western United States earthquakes. Typically, an eastern earthquake will have a felt area ten or more times larger than a western earthquake of the same magnitude.

Numerous methods for determining the attenuation have been developed. For the Passaic project the seismic parameters were estimated using two methods: Cornell and Mertz using Herrmann's attenuation equation (assuming point source at the closest point on the source zone) and Krinitzsky and Marcusson (intensity vs. acceleration curves and Intensity attenuation curves from Chandra). Both of these methods give roughly comparable results as shown on Table 6.

With the Cornell and Mertz method the general form of the attenuation equation is:

$$\log P = a_0 + a_1 s - a_2 \log R - a_3 R$$

where P is the strong ground motion parameter to be estimated, s is a measure of the earthquake size, i.e., magnitude, and R is the distance from the source. The "a" coefficients are specific to the source zone and region and apply to only one frequency. The coefficient of geometrical spreading is a_2 and the coefficient of anelastic attenuation is a_3. These coefficients must be empirically derived by fitting observed data to the general equation. Here again, there is a limited amount of data for the eastern United States and within this data is significant variation.

For the central and eastern United States, Herrmann presented the following equation for peak horizontal acceleration:

For $R \geq$ 15km

$$\log a_h = 0.54 + 0.5 m_b - 0.83 \log R - 0.0019R$$

Where: a_h is the horizontal acceleration
m_b is the body wave magnitude of the earthquake and
R is the distance from the site to the source zone boundary

Krintizsky and Marcusson evaluate attenuation based on an empirical study of Intensity and its equivalent values of the seismic parameters. Type curves for various regions of the country relating loss of Intensity to epicentral distance have been developed by several authors. Figure 15 presents one of these. Using the epicentral Intensity for the earthquake of interest, for example the OBE, the distance from the source zone to the project and the appropriate type curve, a site Intensity can be estimated. The next step is to use this site intensity with the appropriate curves shown on Figures 16, 17, and 18 to arrive at the site seismic parameters.

In a dynamic analysis, the next step is to generate a motion time history which fits the peak values that were calculated. This can be done by selecting an earthquake record from a site with foundation characteristics which are similar to those at the study site. This record is scaled if necessary by fitting the peak value on the record to the calculated peak value. If a suitable record is not available, a synthetic earthquake record is generated. By either method, the above procedure provides a vibration time history which is used to analyze the dynamic response of the project feature being analyzed (i.e., concrete portion of dams, foundation soils, etc.). This procedure is beyond the scope of this report and should be done in the next stage of design.

E. PROBABILITY STUDIES. A full understanding of the seismic risk within a region is not possible without a knowledge of the distribution of earthquakes in time; in other words, the probability of occurrence for each earthquake. Probabilities are determined from the historical record and based on the relationship between the strength of an earthquake and the frequency of occurrence with larger earthquakes being less frequent. Based on studies of numerous seismic areas, this relationship is generally defined by the equation:

$$\log N = a - bM$$

(Gutenberg and Richter)

where N is the number of earthquakes of a given strength per year, M is the strength of the earthquake (e.g., magnitude or intensity) for which N is to be computed, and "a" and "b" are constants which are particular to the seismic zone being studied. This relationship is typically presented in graphical form on a semi-logarithmic plot of N versus M. This is known as a recurrence curve. Gumbel (1958) emphasized the importance of a complete data set in this type of study. In order to compensate for the random nature of occurrence, he recommends that the observation period be no less than 5 times the return period of the earthquake magnitude being studied.

Some researchers contend that this curve is linear throughout the magnitude range, which enables them to compute representative values of "a" and "b" by microseismic monitoring of very small earthquakes over a shorter period of time. Reference 20 describes a study of this kind conducted in a deep mine in northern New Jersey. The resulting values of "a" and "b" were 2.6 and 0.9 respectively. A similar study of an earthquake swarm which occurred in the Blue Lake area of the Adirondack

Mountains resulted in a "b" value of 1.5. The value of "a" could not be readily computed from the data. There is sufficient disagreement on the linearity of the magnitude/recurrence curve to raise a question on the validity of these short term studies. Many authors propose that the curve is either quadratic or bi-linear. Some authors suggest that the curve asymptotically approaches a maximum magnitude. The higher magnitude events are of greater interest in a stability analysis. The magnitude/recurrence curve for these events should be based upon an analysis over the long term or historic data.

The results of this type of study are also strongly influenced by the size and boundaries of the area which is analyzed. In other words, a magnitude/recurrence curve for the entire eastern United States would be markedly different from one for the Passaic River basin. This can be accounted for in a variety of ways. The curve can be normalized to area, for example, cumulative number of earthquakes per year per square kilometer. In addition, the region can be subdivided into seismic source zones (see Paragraph V.F.) which are discrete with regard to geologic setting, tectonics, and seismicity. Magnitude/recurrence curves can then be developed from the seismic history of each zone. The shortcoming of this approach in the eastern United States is the scarcity of data especially at higher magnitudes. The maximum credible earthquake is not represented in the historic record. Nuttli and others have studied the zones in which the maximum credible earthquake is thought to have occurred (New Madrid, Charleston). In both these cases, the maximum earthquake corresponds to a return period of about 1,000 years on an extension of the magnitude/recurrence curve. More involved methods of analysis have been suggested, but for the present level of study, the 1,000 year return period for an MCE is assumed to be representative for all seismic zones in the East. The slopes used on the magnitude/recurrence curves in this study are those suggested by Nuttli although other authors have proposed slightly different coefficients. The operating basis earthquake was selected by reading the magnitude of the 100 year or 0.01 recurrence interval earthquake from the magnitude-recurrence curves for the various source zones. The earthquakes derived in this way were used with the attenuation equation or the site Intensity curves (see Paragraph V.D.) to yield the predicted site motions for the Passaic project given on Table 6.

F. SEISMIC SOURCE ZONES. The distribution of earthquakes is not uniform. Events cluster in some areas, are scattered diffusely in other areas, and are absent in others. This variability is to be expected since earthquakes occur on fixed geologic structures of differing characteristics under a non-uniform stress distribution. In terms of human time frames, these conditions can be expected to remain fairly constant. It is possible, therefore, through analysis of earthquake distribution in light of seismogenic mechanisms (see Paragraph IV.E.), geologic history (see Paragraph IV.B), and tectonics (see Paragraphs IV.C. and IV.G.), to subdivide the region into seismic source zones. This has been done for the eastern United States by several authors. Reference 4 was prepared for the Nuclear Regulatory Commission to facilitate site investigations for nuclear power plants. It is equally applicable for other large civil works projects and is used as a basis for this paragraph. Figure 19, which is taken directly from this report, shows the source zone boundaries for the region. A refinement of this zonation taken from reference 62, is presented as Figure 20.

The Passaic project is located within the boundaries of the Raritan Bay zone but would be subjected to shaking from the other zones in the area. The problem, therefore, becomes one of determining which zone will have the greatest effect on the project area. The following are the zones most likely to effect the Passaic project. Table 6 gives computed site values for each of these zones.

1. NIAGARA-ATTICA. It has been associated with the Clarendon-Linden fault, the only major fault in the area. One solution to focal mechanism studies is consistent with the north-northwesterly strike of this fault, however, the seismicity forms a somewhat diffuse east-west trending ellipse, which aligns with another focal mechanism solution, but is not consistent with the Clarendon-Linden strike. This question of source has not yet been resolved.

The first earthquake in this zone which is included in the historical record was in February 1796. The largest earthquake in the zone to date was an Intensity VIII event on August 12, 1929. Instrumental data and felt reports suggest that these earthquakes were shallow focus (see Table 4). As such, they exhibit high epicentral intensities and frequency content (loud sharp noises were common to most descriptions). Isoseismal maps indicate that the attenuation of these earthquakes is somewhat higher than other eastern North American earthquakes. A magnitude/recurrence curve for Niagara-Attica is shown on Figure 21. The maximum credible earthquake derived from this is Mb = 6.4. The 100 year return period earthquake (OBE) is Mb = 4.9.

2. ANNA-CLEVELAND. The concentration of seismicity in western and northern Ohio is in line with the northeasterly trending belt of seismicity which extends from New Madrid to the St. Lawrence. The cause of this concentration of activity is not well understood, but some believe it is part of a major rift initiated during the Mesozoic and still active today. The first recorded event in the Anna-Cleveland zone was of Intensity VII and occurred on June 18, 1875. Subsequent activity has been regular and unusual for the percentage of higher intensity events. A magnitude/recurrence curve for this zone is shown on

Figure 22. The maximum credible earthquake for this zone is Mb = 6.5. The 100 year return period earthquake (OBE) is Mb = 5.4.

3. NEW ENGLAND-RARITAN BAY. A long, well documented history of seismic activity in New England begins with an account of an Intensity VII event in 1568, which affected what is now Connecticut and Rhode Island. The entire zone, which for this report includes southern New York and northern New Jersey, is undoubtedly very complex, involving numerous faults of varying character. They are grouped here based on related tectonics and the relative uniformity of activity throughout the outlined area. Greater resolution of the activity is undoubtedly possible with analysis of focal mechanism and microseismic studies, but does not serve the purpose of this paper. Much of the activity is thought to be occurring in reactivated fault zones and rifts of late Mesozoic or older age. As described in paragraph IV.E., some feel that stress concentrations may occur where major oceanic transform faults truncate on the continental margin. It is interesting to note that the northern portion of New England seismicity aligns with the projection of the trend of the New England seamount chain. The Adirondack-Western Quebec seismic zone is located further west on this same general bearing. Whether these features are genetically related is still very much in question. The Passaic project is within the limits of this zone. Because of the diffuse nature of the earthquakes and lack of clear evidence on causative features (See Paragraph IV.E.1.) selection of site parameters have been done using Far Field curves for attenuation. A magnitude/recurrence curve for this zone is shown on Figure 23. Based on this, the maximum credible earthquake for the zone is Mb = 6.4. The 100 year return period earthquake (OBE) is Mb = 5.3.

4. ADIRONDACK-WESTERN QUEBEC. The earliest event listed for this zone is an Intensity VII earthquake which occurred on February 10, 1661. The historical seismicity for this zone defines a roughly elliptical area extending from Lake Champlain in the southeast to Timiskaming, Quebec in the northwest. Correlations between the seismicity and geological and topographical features have been observed. As mentioned in Paragraph V.F.3., this zone generally aligns with a small circle describing the movement of the North American continent with relation to Africa during the opening of the Atlantic and passing through the New England seamount chain. The seismicity is also largely confined within the Grenville supergroup to the central metasedimentary belt. This zone of marbles, quartzites, and paragneisses containing complex north and northeast striking structural trends is distinct from the adjacent older gneissic terrain to the west, and the younger area of granulitic terrain to the east. The southern boundary of the seismicity appears to be a west northwesterly trending fracture zone represented by Proterozoic dikes and Paleozoic faults, some of which form the Ottawa-Bonnechere graben. Some seismicity included within the

zone falls outside the geologic boundaries described above, notably the Blue Lake swarm in the Adirondacks and Timiskaming in Quebec. This activity is included because of its proximity, although it is probably genetically unrelated.

The zone is conterminous to a large degree with a topographically low area known as the "Gatineau Triangle". The significance of this relationship is not clear. A magnitude/recurrence curve for this zone is shown on Figure 24. Based on this, the maximum credible earthquake for this zone id Mb = 6.5. The 100 year return period earthquake (OBE) is Mb = 5.4.

5. RICHMOND. One of the first accounts of an earthquake from this zone is for an Intensity IV event on March 22, 1758. The first strong shock in the historical record was an Intensity VII earthquake on February 21, 1774. The seismicity is quite localized and has been associated by some authors with a general seismic trend along the fault line. Here again, this is believed to be related to resurgent tectonics or reactivated zones of weakness. Numerous Cretaceous and Cenozoic faults are located in the area. Seismic profiling demonstrates offsets of 50 - 60 meters in early Cenozoic sediments with some indication of progressive offset up through surficial sediments in isolated locations. Notable faults in the area of this seismic zone are the Brandywine and the Stafford. The exact nature and extent of these faults is not yet confirmed, but at present, they are thought to be reverse northeastwardly striking zones, dipping steeply either northwest or southeast, with no apparent predominance. Strike slip movement is not normally noted. It is interesting to note that where offsets can be measured over a complete section of the Cenozoic strata, they indicate a general decrease in the rate of movement. A magnitude/recurrence curve for the zone is shown on Figure 25. Based on this, the maximum credible earthquake for the zone is Mb = 6.1. The 100 year return period earthquake (OBE) is Mb = 5.0.

6. CHARLEVOIX. This zone, located about midway on the St. Lawrence River has produced one of the largest historical earthquakes in eastern North America. The first recorded quake in the zone was an Intensity IX earthquake on June 11, 1638. An Intensity X event was reported on February 5, 1663 from the same region. The persistent, strong seismicity of Charlevoix-LaMalbaie is associated with two major geologic features: Logan's Line, which separates Precambrian shield rock on the north shore of the St. Lawrence River from Paleozoic sediments on the south shore, and the Charlevoix impact structure. Almost all the seismicity appears to be confined to the southeastern half of the crater, where it intersects Logan's Line. The impact structure is interpreted as a deep-seated zone of weakness, which serves to localize stress. Logan's Line is interpreted as a graben, which may currently be influenced by a compressional

stress field. Thrust faulting is indicated in some focal mechanism studies and by evidence of seismic velocity changes prior to earthquakes. This velocity phenomenon has been observed in more active regions and is always associated with thrusting. A magnitude/recurrence curve for this zone is shown on Figure 26. Based on this, the maximum credible earthquake for this zone is Mb = 6.8. The 100 year return period earthquake (OBE) is Mb = 5.75.

VI. CONCLUSIONS. The Passaic project is located in a moderately seismic area. Earthquakes have occurred periodically causing minimal damage. Because of the consequences of failure it is appropriate to use an operating basis earthquake for design. This earthquake is based on a 100 year return period. Other seismic source areas may contribute to shaking experienced within the project boundaries but their effect is negligible for a 100 year event. The greatest effect will likely be felt from the New England-Raritan Bay seismic source zone. Based on restraining the design earthquake for the Passaic project to a 100 year operating basis earthquake as determined in this report, the following peak site parameters are recommended:

Peak Horizontal Acceleration = 153 cm/sec/sec = 0.16 g
Peak Horizontal Velocity = 14 cm/sec
Peak Horizontal Duration with acceleration > 0.05g = 8 seconds

These values are in keeping with values from the draft ER and similar engineering reports prepared for other structures in the surrounding area. It was decided that, while the project is spread over a large area, a single set of design parameters would suffice. This was assumed since the project lies within the New England-Raritan Bay Seismic Zone and therefore is equally prone to shaking within that zone.

REFERENCES

1.The American Association of Petroleum Geologists, Geological Highway Map, Northeastern Region, 1976.

2.The American Association of Petroleum Geologists, Geological Highway Map, Great Lakes Region, 1978.

3.Anderson, John G. "On the Attenuation of Modified Mercalli Intensity with Distance in the United States," Bulletin of the Seismological Society of America; Vol. 68, Number 4, August 1978.

4.Barstow, Brill, Nuttli, Pomeroy. "An Approach to Seismic Zonation for Siting nuclear Electric Facilities in the Eastern United States," Reference No. NUREG/CR-1577, May 1981.

5.Barstow, N.L. and Miller E. Schlesinger. "Seismicity in New York and Adjacent Areas, 1981-1982," Earthquake Notes 54, April-June 1983, 67-68, 320/E17/V54(2).

6.Basham, P.W., D.H. Weichert, and M.J. Berry, 1979, "Regional Assessment of Seismic Risk in Eastern Canada," Bulletin of the Seismological Society of America, Vol. 69, 1567-1602.

7.Beavers, J.E., editor. "Earthquakes and Earthquake Engineering--Eastern United States," Ann Arbor Scientific Publishers, Inc., 2 Vols., 1191 pages, 1981.

8.Brazee, Rutlage J. "An Analysis of Earthquake Intensities with Respect to Attenuation, Magnitude and Rate of Recurrence - Revised Edition," NOAA Technical Memorandum EDS NGSDC, 2 August 1976.

9.Burke, K. and J.F. Dewey. "Plume-Generated Triple Junction: Key Indicators in Applying Plate Tectonics to Old Rocks," Journal of Geology, Vol. 81, 1973, pp 406-433.

10.Chandra, U. "Attenuation of Intensities in the Untied States,"Bulletin of the Seismological Society of America, Vol. 69, No. 6, December 1969.

11.Chang, Frank K. "State of the Art for Assessing Earthquake Hazards in the United States; Duration, Spectral Content, and Predominant Period of Strong Motion Earthquake Records from Western United States," Miscellaneous Paper S-73-1, Report 8, December 1977, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

12.Chinnery, M.A. "A Comparison of the Seismicity of Three Regions of the Eastern United States," Bulletin of the Seismological Society of America, Vol. 69, No. 3, June 1979.

13.Coffman, J.L. and C.A. von Hake. "Earthquake History of the United States," U. S. Department of Commerce Publication

41-1, Revised Edition (through 1970), UDC,550.34(73)(091).

14.Drysdale, J.A. and R.J. Wetmiller. "Eastern Canadian Earthquakes 1981-82, "Earthquake Notes, 54, April-June 1983, 320/E17/V54(2).

15.Eardley, A.J., "Structural Geology of North America," Harper & Brothers, 1951.

16.Fletcher J.P. and J.G. Anderson. "First Strong Motion Records From a Central or Eastern United States Earthquake," Bulletin of the Seismological Society of America, Vol. 64, No. 5, October 1974, pp 1455-1465.

17.Fox, F.L. 1970, "Seismic Geology of the Eastern United States," Association of Engineering Geologists, Bull. 7:21-43.

18.Hamel, J.V. "Rock Foundation Re-evaluation Mahoning Dam and Kinzua Dam," Pittsburgh District Corps of Engineers, May 1976.

19.Horner, R.B., A.E. Stevens, H.S. Hasegawa and G. LeBlanc. "Focal Parameters of July 12, 1975 Maniwaki, Quebec Earthquake, and Example of Intraplate Seismicity in Eastern Canada," Bulletin of the Seismological Society of America, Vol. 68. No. 3, June 1978.

20.Herrmann, R.B. "A Seismological Study of Two Attica, New York Earthquakes," Bulletin of the Seismological Society of America, Vol. 68, No. 3, June 1978, pp 641-651.

21.Isacks, B., Oliver J. "Seismic Waves with Frequencies from 1 to 100 Cycles per Second Recorded in a Deep Mine in Northern New Jersey," Bulletin of the Seismological Society of America, Vol. 54, No. 6, Part A, December 1964.

22.Iurita, Tuneto. "Regional Variations in the Structure of the Crust in the Central United States from P Wave Spectra," Bulletin of the Seismological Society of America, Vol. 60, No. 3, June 1970.

23.Jones, F.B., L.T. Long and J.H. McKee. "Study of the Attenuation and Azimuthal Dependance of Seismic Wave Propagation in the Southeastern United States," Bulletin of the Seismological Society of America, Vol. 67, No. 6, pp 1503-1513, December 1977.

24.Krinitzsky, Ellis and Frank K. Chang. "State of the Art for Assessing Earthquake Hazards in the United States; Specifying Peak Motions for Design Earthquakes," Miscellaneous Paper S-73-1, Report 7, December 1977, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

25.Lytle, W.S. and J.H. Gath. "Oil and Gas Geology of the Kinzua Quadrangle, Warren and McKean Counties, Pennsylvania," Commonwealth of Pennsylvania State Planning Board, Topographic and Geologic Survey, Mineral Resources Report M 62, 1970.

26.Makanic, B. "On the Frequency Destribution of Earthquake Magnitude and Intensity," Bulletin of the Seismological Society of America, Vol. 70, No. 6, pp 2253-2260, December 1980.

27.Masse, R.P. "Compressional Velocity Distribution Beneath Central and Eastern North America," Bulletin of the Seismological Society of America, Vol. 63, No. 3, pp 911-935, June 1973.

28.McGuire, R.K. "Effects of Uncertainty in Seismicity on Estimates of Seismic Hazard for the East Coast of the United States," Bulletin of the Seismological Society of America, Vol. 67, No. 3, pp 827-848, June 1977.

29.McKeown, F.A. 1978, "Hypothesis: Many Earthquakes in Central and Southeastern U. S. are Causally Related to Mafic Intrusive Bodies," Journal Res. U.S.G.S., 6:41-50.

30.Mitchell, Brian J. and Robert B. Herrmann. "Shear Velocity Structure in the Eastern United States from Inversion of Surface-Wave Group and Phase Velocities," Bulletin of the Seismological Society of America, Vol. 69, No. 4, August 1979.

31.Nuttli, Otto W. "State of the Art for Assessing Earthquake Hazards in the United States; Attenuation of High Frequency Seismic Waves in the Central Mississippi Valley," Miscellaneous Paper S-73-1, Report 10, July 1978, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

32.Nuttli, Otto W. "State of the Art for Assessing Earthquake Hazards in the United States; Credible Earthquakes for the Central United States," Report 12, December 1978, U. S. Engineer Waterways Experiment Station, CE, Vicksburg, MS.

33.Ocala, L.C. and R.P. Meyer. 1973, "Central North America Rift System, Structure of the Axial Zone from Seismic and Gravimetric Data," J. Geophys. Res., 78:5173-5194.

34.Pitman, W.D. and M. Talwani. 1972, "Sea-floor Spreading in the North Atlantic," Bull. Geol. Soc. Am., 83:619-649.

35.Pomeroy, P.W., D.W. Simpson and M.L. Sbar. "Earthquakes Triggered by Surface Quarrying--The Wappingers Falls, New York Sequence of June 1975," Bulletin of the Seismological Society of America, Vol. 66, No. 3, pp 685-700.

36.Rogers, J. "The Tectonics of the Appalachians," John

Wiley & Sons, Inc., 1970, Library of Congress Catalogue Card No. 72-116771.

37.Sbar, M.L. and L.R. Sykes. 1973, "Contemporary Compressive Stress and Seismicity in Eastern North America: An Example of Intra-plate Tectonics," Geol. Soc. Am. Bull., 84:

38.Sbar, M.L., J.M. Rynn, F.J. Gumper and J.C. Lahr. "An Earthquake Sequence and Focal Mechanism Solution, Lake Hopatcong, Northern New Jersey," Bulletin of the Seismological Society of America, Vol. 60, No. 4, August 1970, pp 1231-1243.

39.Sbar, M.L., R.R. Jordan, C.D. Stephens, T.E. Pickett, K.D. Woodruff and C.G. Sammis. "The Delaware-New Jersey Earthquake of February 28, 1973," Bulletin of the Seismological Society of America, Vol. 65, No. 1, February 1975.

40.Sbar, M.L., J. Armbruster and Y.A. Aggarwal. "The Adirondack, New York Earthquake Swarm of 1971 and Tectonic Implications," Bulletin of the Seismological Society of America, Vol. 62, No. 5, October 1972.

41.Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, Riley M. 1984. "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluation," Journal of Geotechnical Engineering ASCE, Vol. III, No. GT12, December 1985, pp 1425-1445.

42.Shepps, V.C., G.W. White, J.B. Droste and R.F. Sitler. "Glacial Geology of Northwestern Pennslyvania," Commonwealth of Pennsylvania Department of Internal Affairs, Topographic and Geologic Survey, Bulletin G 32, 1959.

43.Simmons, G. 1976, "Field Velocity Measurements and Rock Mechanics Studies," Vol. BE-SG7606, in Geologic and Seismologic Investigation, prepared by Weston Geophysical Res., Inc. for Boston Edison Co.

44.Slemmons, David B. "State of the Art for Assessing Earthquake Hazards in the United States; Faults and Earthquake Magnitude," Miscellaneous Paper S-73-1, Report 6, May 1977, U. S. Army Engineer Waterways Experiment Station CE, Vicksburg, MS.

45.Slemmons, David B. "State of the Art for Assessing Earthquake Hazards in the United States; Definition of Active Fault," Miscellaneous Paper S-73-1, Final Report, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

46.Spall, Henry. "Understanding Seismicity Within the Continents," Earthquake Information Bulletin, Vol. 11, No. 3, May-June 1975.

47.Stevens, A. "Reexamininination of Some Larger LaMalbaie, Quebec Earthquakes (1924-1978)," Bulletin of the Seismological Society of America, Vol. 70, No. 2, April 1980, pp 529-557.

48.Street, R.L. "Scaling Northeastern United States/Southeastern Canadian Earthquakes by their Lg Waves," Bulletin of the Seismological Society of America, Vol. 66, No. 5, October 1976.

49.Street, R.L. and F.T. Turcotte. "A Study of Northeastern North American Spectral Moments, Magnitudes, and Intensities," Bulletin of the Seismological Society of America, Vol. 67, No. 3, June 1977.

50.Walper, Jack L. "State of the Art for Assessing Earthquake Hazards in the United States; Plate Tectonics and Earthquake Assessment," Miscellaneous Paper S-73-1, Report 5, March 1976, U. S. Army Engineer Waterway Experiment Station, CE, Vicksburg, MS.

51.Weichert, D.H. and W.G. Milne. "On Canadian Methodologies of Probablistic Seismic Risk Estimation," Bulletin of the Seismological Society of America, Vol. 69, No. 5, October 1979, pp 1549-1566.

52.Weigle, Robert L. "Earthquake Engineering," Prentice Hall, 1970, 3rd Printing.

53.Winkler, L. "Catalog of U. S. Earthquakes Before the Year 1850," Bulletin of the Seismological Society of America, Vol. 69, No. 2, April 1979.

54.Wylie, P.J. "The Dynamic Earth, Textbook in Geosciences," John Wiley & Sons, Inc., New York, 1971.

55.Federal Emergency Management Agency. "Federal Guidelines for Earthquake Analysis and Design of Dams," FEMA 65/March 1985.

56.Northeastern U. S. Seismic Network Bulletins. "Seismicity of the Northeastern United States," NUREG, WES-238066.

57.Department of the Army Corps of Engineers. "Earthquake Analysis for Corps of Engineers Project," ER 1110-2-1806, 16 May 1983.

58.Department of the Army Corps of Engineers, St. Louis District. "Earthquake Potential of the St. Louis District," February 1981.

59.Department of the Army Corps of Engineers, Omaha District. "Evaluation of Embankment and Foundation Liquefaction Potential, Fort Randall Dam," February 1985.

60.Department of the Army Corps of Engineers, Huntington District. "Seismological Study of the Huntington District," October 1, 1980.

61.Department of the Army Corps of Engineers, Waterways Experiment Station. "Geological-Seismological Evaluation of Earthquake Hazards at Franklin Falls Damsite, New Hampshire." September, 1986

62.Department of the Army Corps of Engineers, Waterways Experiment Station. "Geological-Seismological Evaluation of Earthquake Hazards at Prompton and Francis E. Walter Damsites, Pennsylvania." September, 1986

63.Department of the Army Corps of Engineers, Mobile District. "Seismic Design at Pompton Lake Dam, New Jersey." June, 1991

64.Aggarwal, Y.P., and Sykes, L.R. "Earthquakes, Faults, and Nuclear Power Plants in Southern New York and Northern New Jersey.", Science Vol. 200, pp 425-429, 28 April, 1978. American Association for the Advancement of Science

FIGURES

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.1.1

SEISMIC STUDY

FIGURES

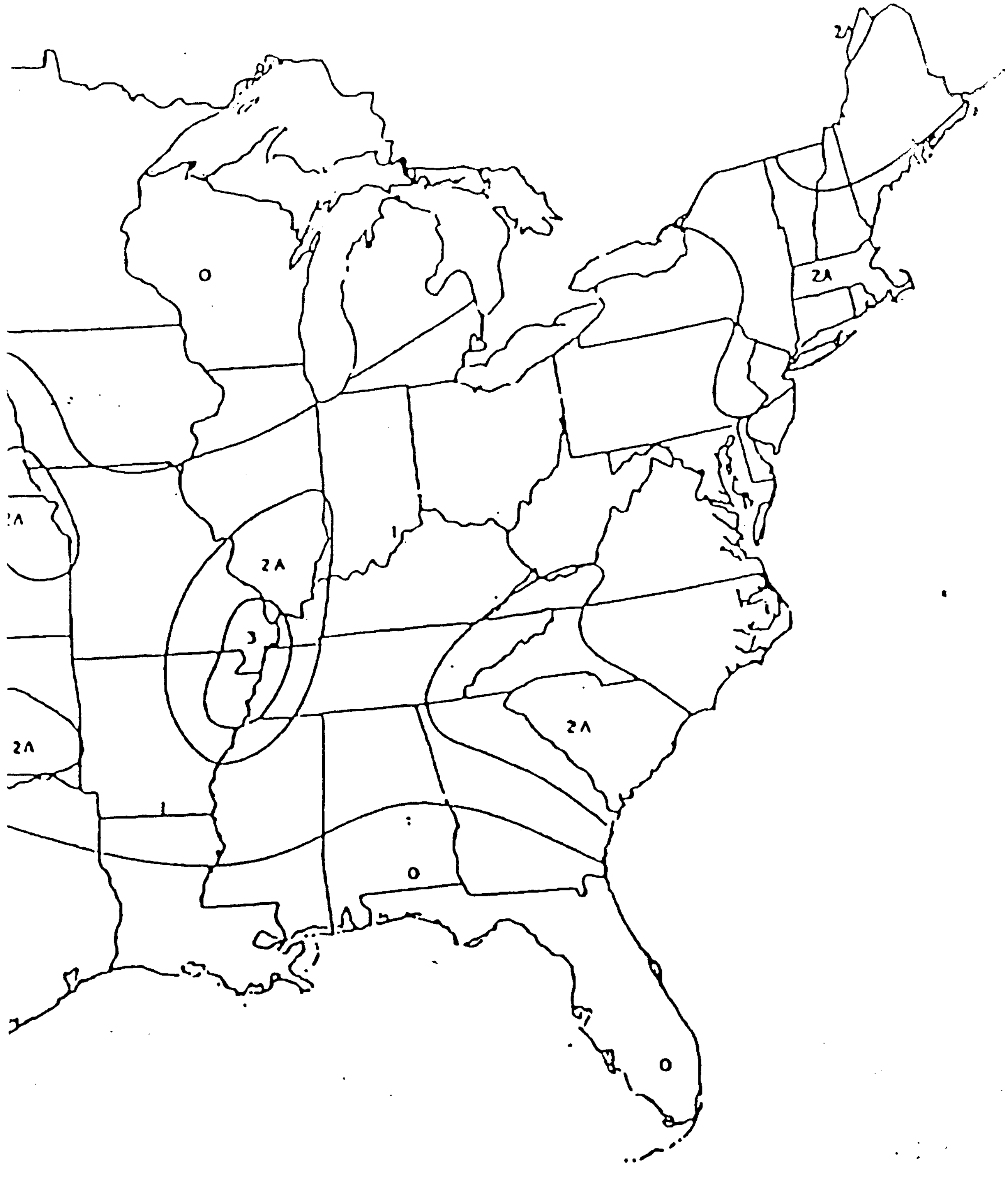
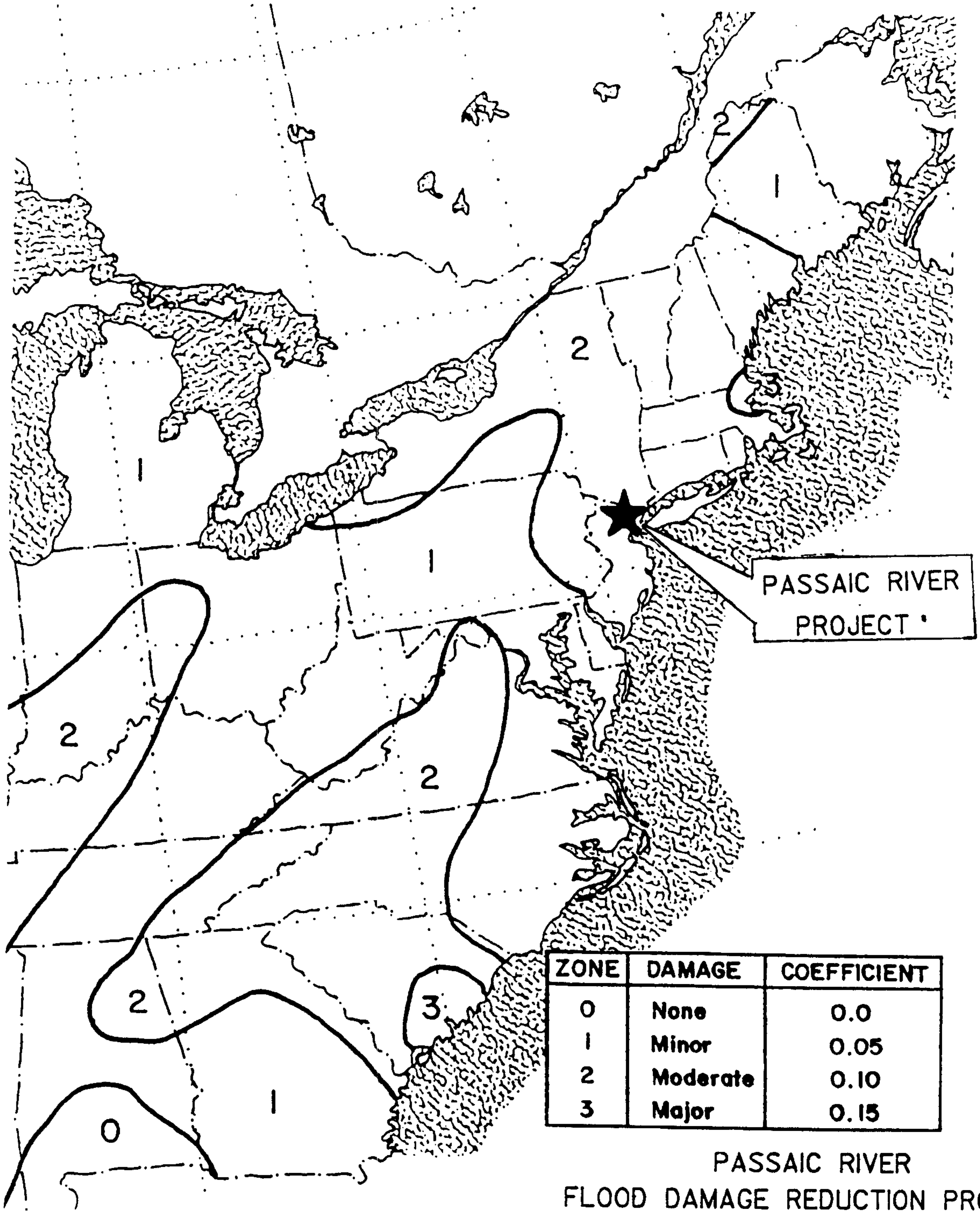


Figure 1. SEISMIC ZONE MAP OF THE UNITED STATES
From ER-1110-2-1806 DRAFT, 30 May, 1995



ZONE	DAMAGE	COEFFICIENT
0	None	0.0
1	Minor	0.05
2	Moderate	0.10
3	Major	0.15

From Corps of Engineers ER 1110-2-1806, 16 May, 1983
"EARTHQUAKE DESIGN AND ANALYSIS FOR CORPS OF ENGINEERS PROJECTS"

PASSAIC RIVER
FLOOD DAMAGE REDUCTION PROJECT
SEISMIC STUDY
US ARMY CORPS OF ENGINEERS
PASSAIC RIVER DIVISION

SEISMIC ZONE MAP

FIGURE: 2.

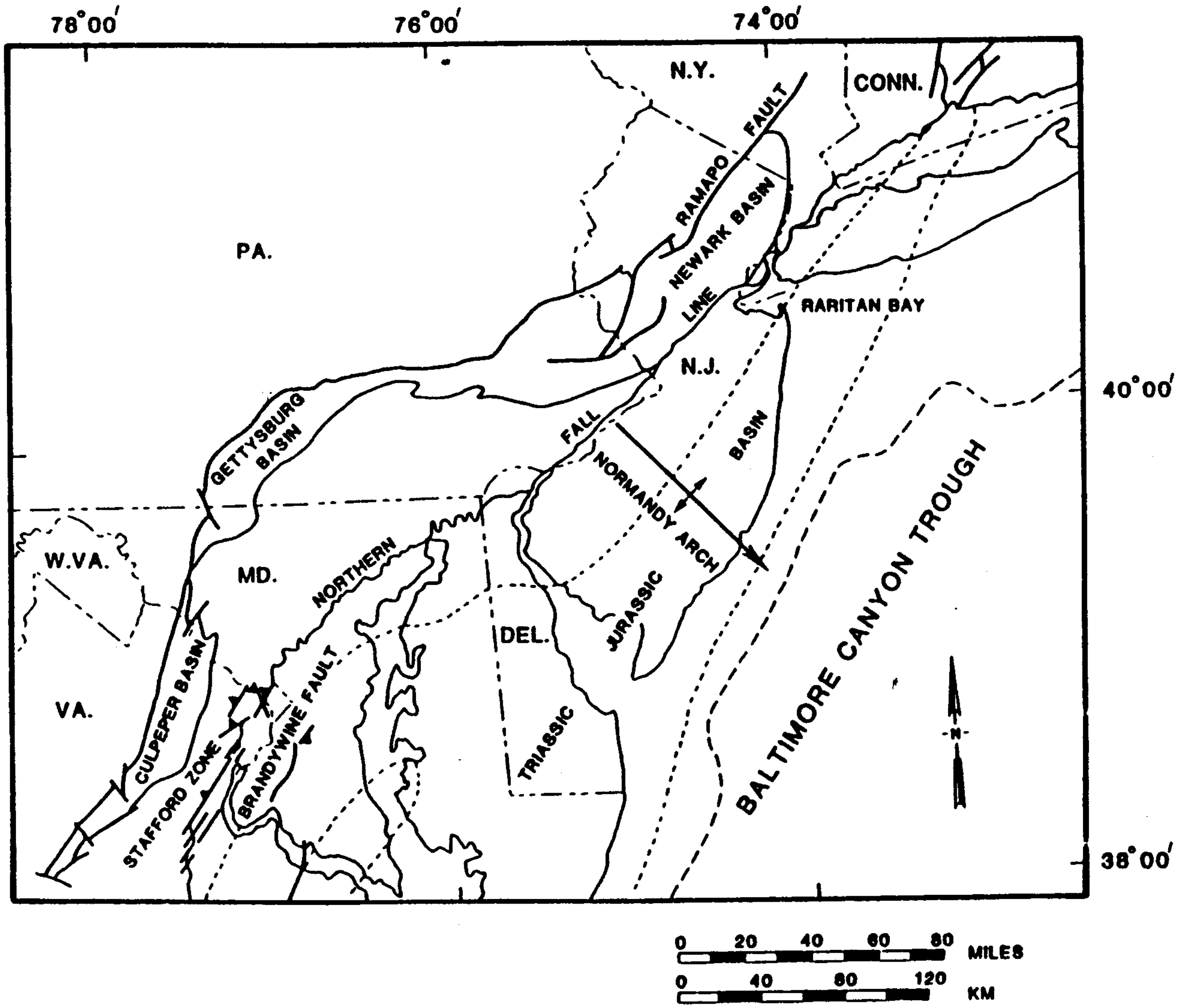
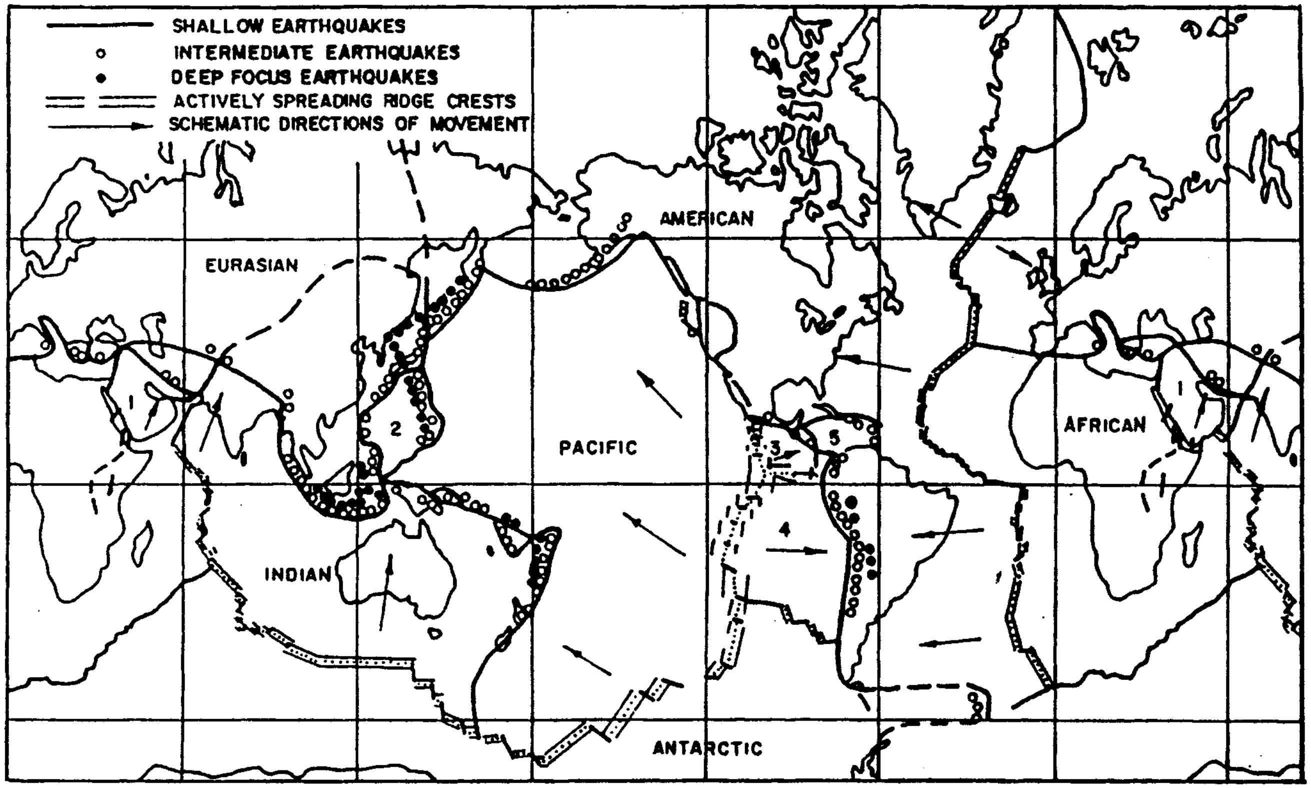


Figure 3. Map of Raritan Embayment and surrounding region showing location of early Mesozoic basins, oceanic fracture zones, and select geologic features (from Wentworth and Mergner-Keefer, 1983)



From: Gass, Smith, and Wilson

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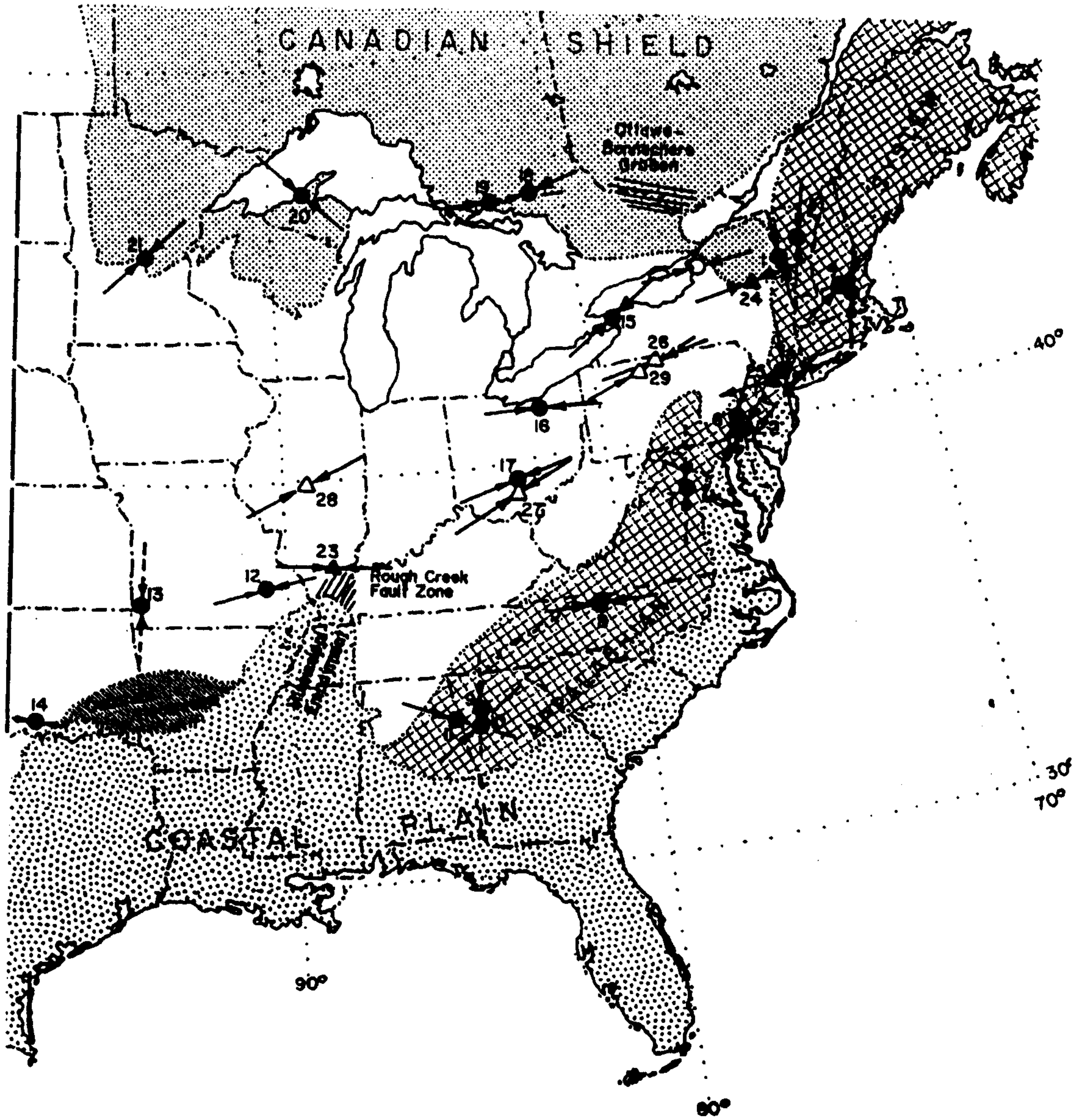
PLATE TECTONICS

FIGURE: 4.



Map of part of North America with selected tectonic features showing fault plane solutions of earthquakes (solid triangles), strain relief-in situ stress measurements (solid circles), hydrofracture in situ stress measurements (open triangles), and a pop up near Chippewa Bay, New York (open circle) Strike of horizontal component of maximum or minimum compressive stress is shown at each locality Arrows denoted by a dotted line are less reliable (that is ratio of σ_1 to σ_3 1 5 or less)

Numbers refer to Table 3

This figure taken from "Contemporary Compressive Stress and Seismicity in Eastern North America: An Example of Intra-Plate Tectonics", Sbar, Marc L., Sykes, Lynn R Bulletin of the Seismological Society of America: Volume 84, No 6 June 1973, p.p. 1861-1882.

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IN-SITU STRESS MEASUREMENTS

FIGURE: 5.

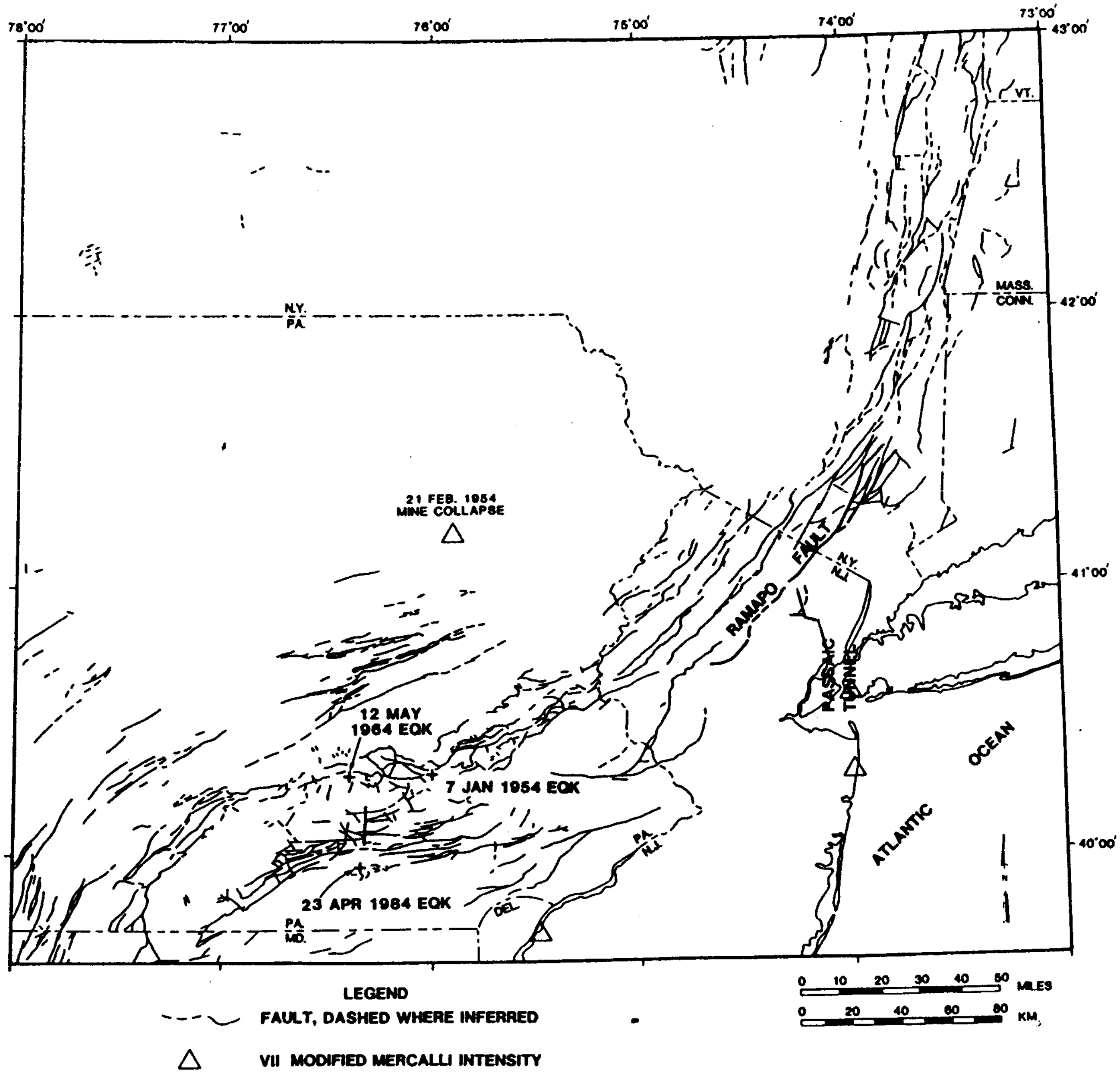
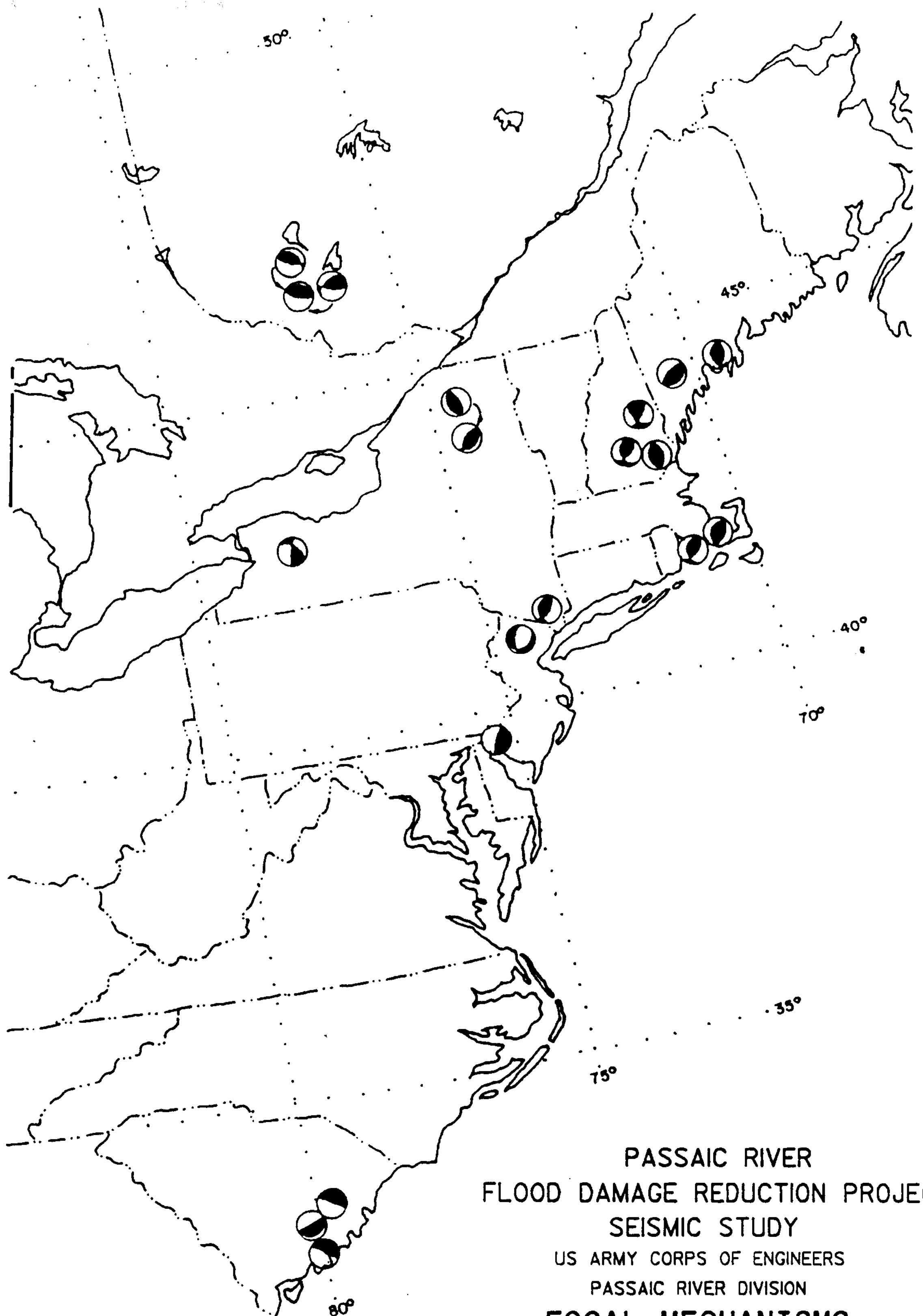


Figure 6. Patterns of faulting in the study



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FOCAL MECHANISMS

FIGURE: 7.

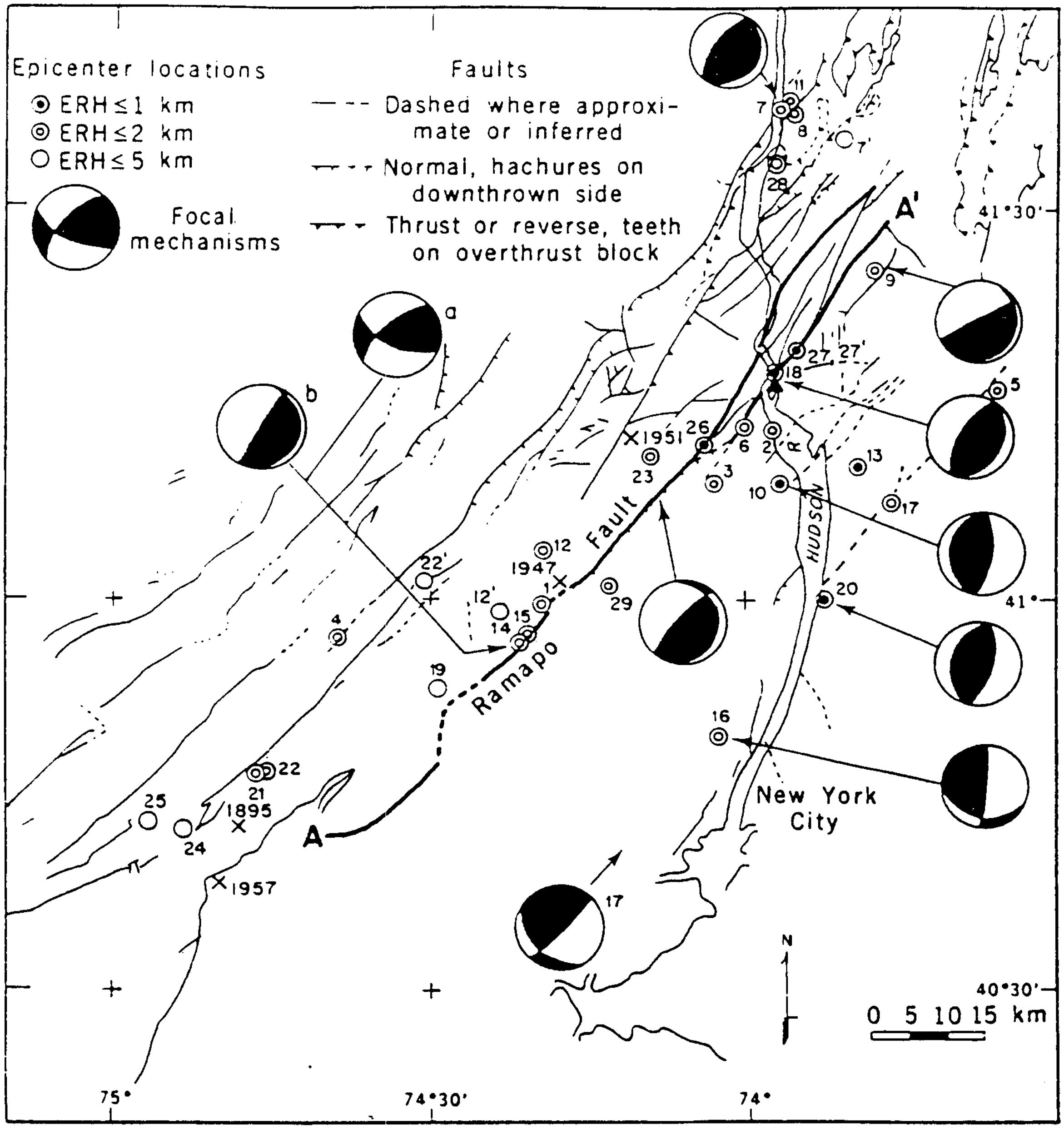


Fig.8 . Fault map (*4, 5, 29*) of southeastern New York and northern New Jersey showing epicenters (circles) of instrumentally located earthquakes from 1962 through 1977. Indicated uncertainties (*ERH*) in epicentral locations represent approximately two standard deviations. Focal mechanism solutions are upper-hemisphere plots; the dark area represents the compressional quadrant. For event 14 there are two possible focal mechanism solutions; the data, however, are more consistent with solution *b* than *a*. The Ramapo fault and two of its major branches (*A*-*A'*) are shown by the heavy lines; ×'s denote locations for other events discussed in the text. The solid triangle shows the location of the Indian Point nuclear power reactors.

From "Earthquakes, Faults, and Nuclear Power Plants in Southern New York and Northern New Jersey." by Aggarwal and Sykes

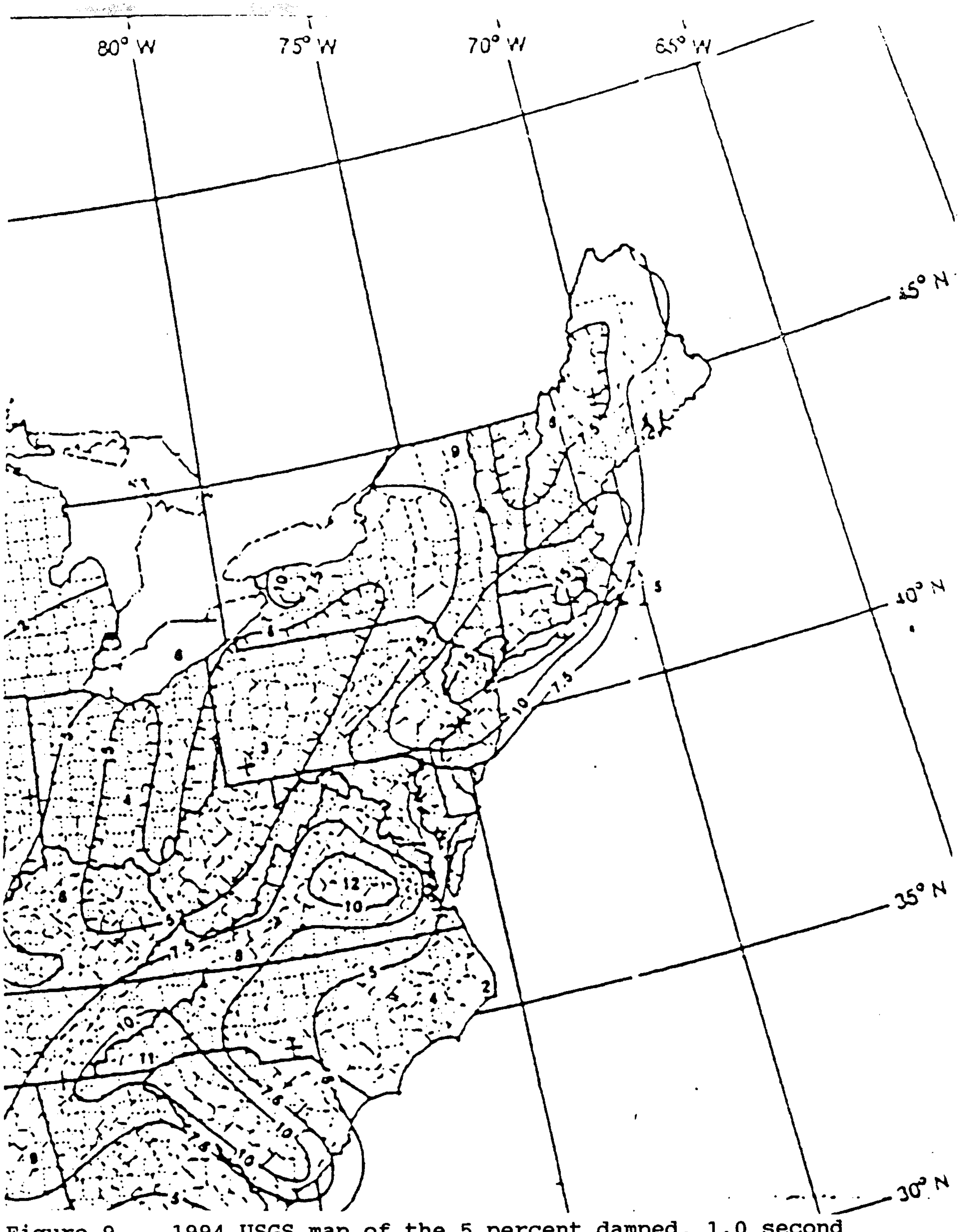


Figure 9. 1994 USGS map of the 5 percent damped, 1.0 second pseudo-acceleration spectral responce, expressed in percent of the acceleration of gravity, with a 10 percent probability of exceedance in 50 years.
From ER 1110-2-1806 DRAFT 30 May,1995

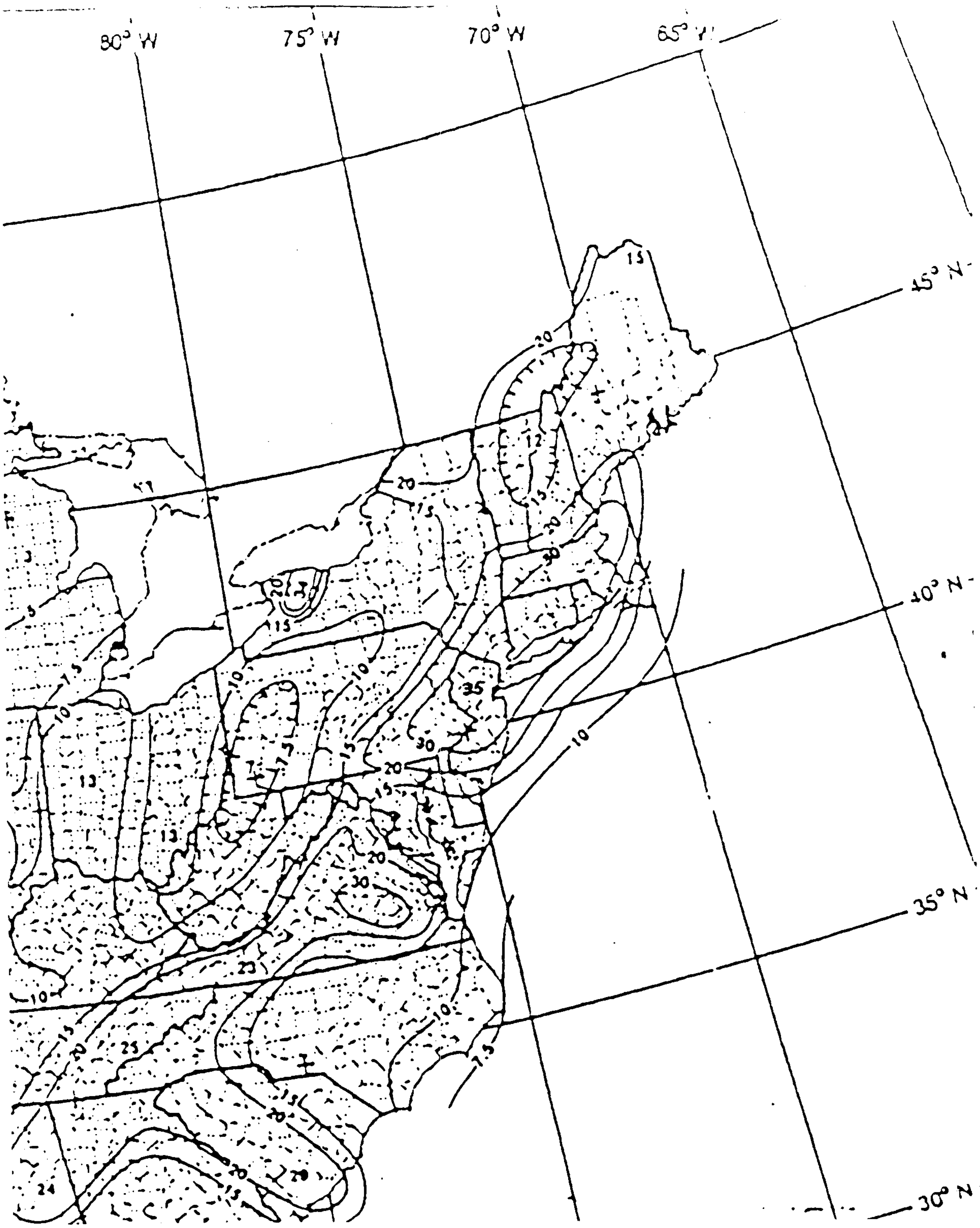
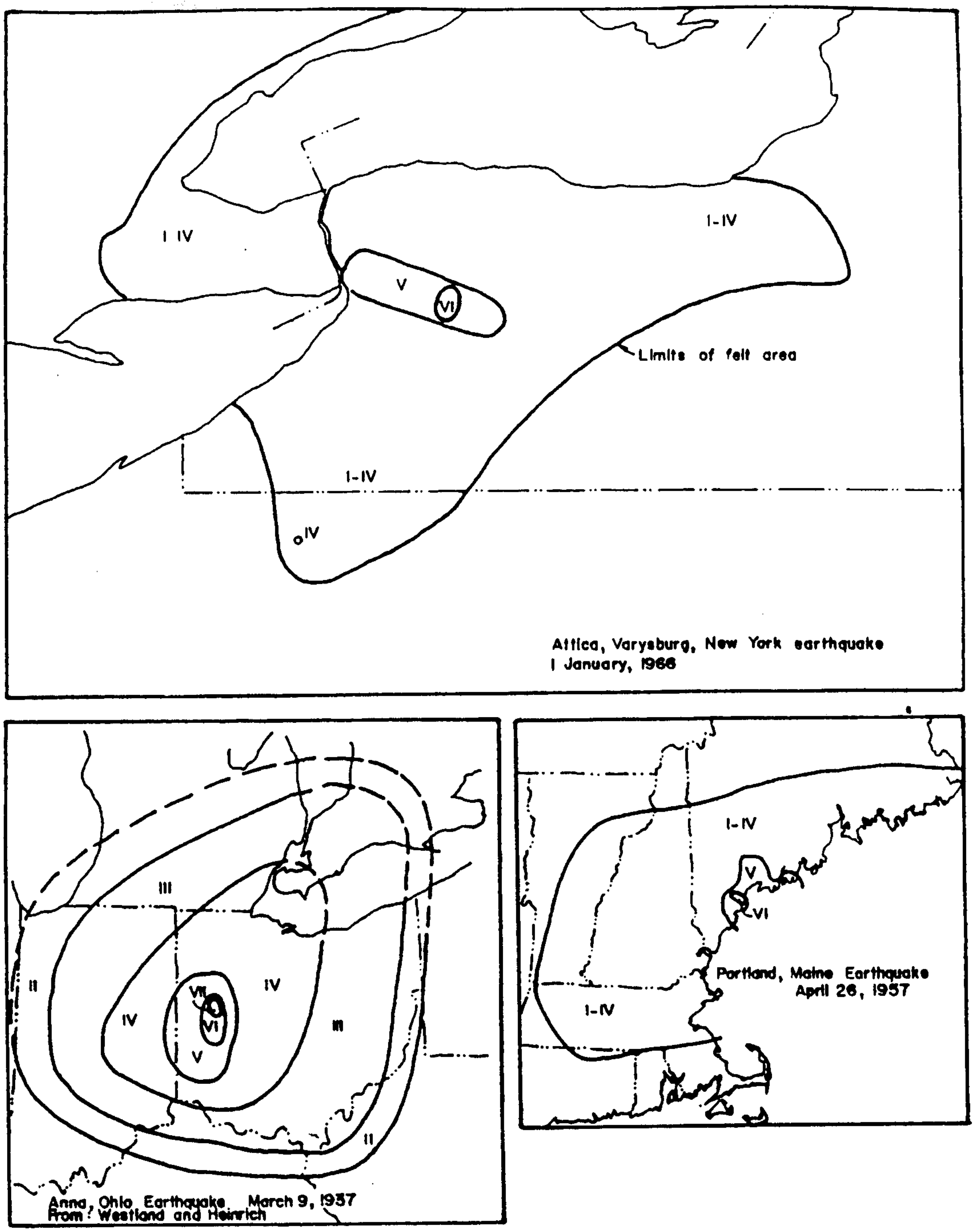


Figure 10. 1994 USGS map of the 5 percent damped, 1.0 second pseudo-acceleration spectral responce, expressed in percent of the acceleration of gravity, with a 10 percent probability of exceedance in 250 years.
From ER 1110-2-1806 DRAFT 30 May,1995



PASSAIC RIVER
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US ARMY CORPS OF ENGINEERS
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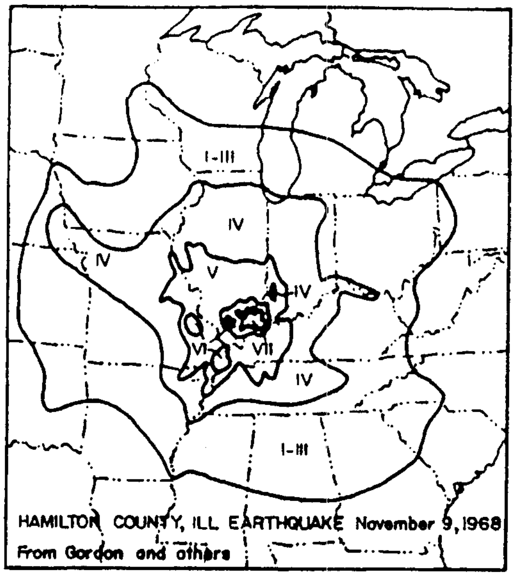
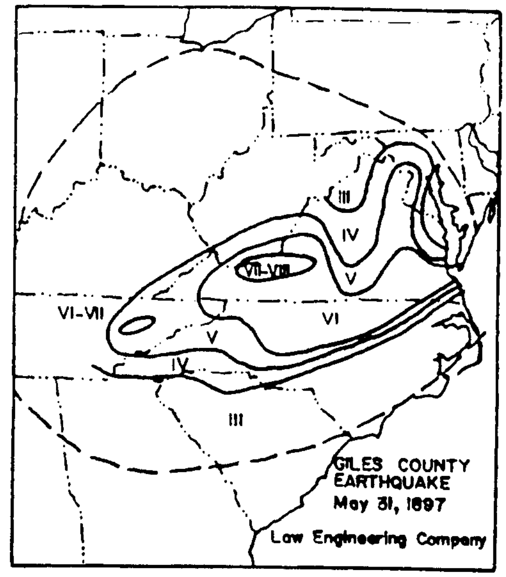
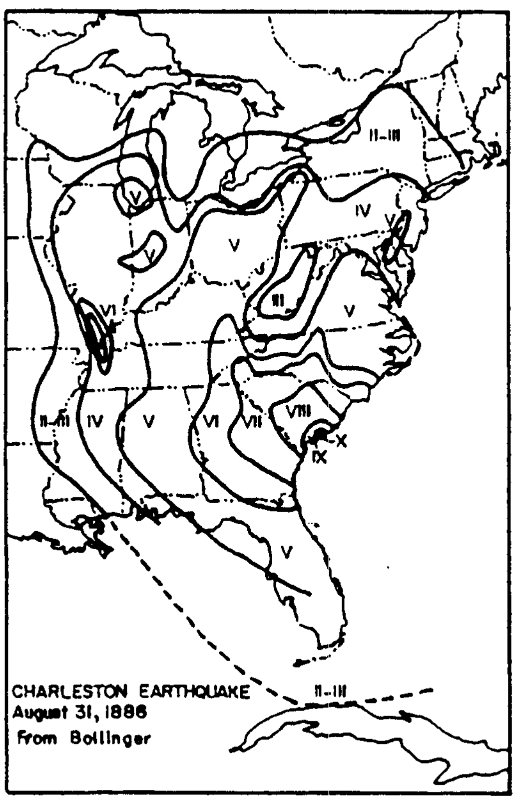
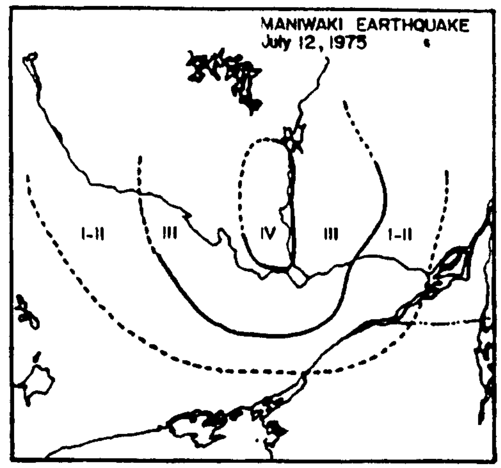
ISOSEISMAL MAPS

FIGURE: 11.









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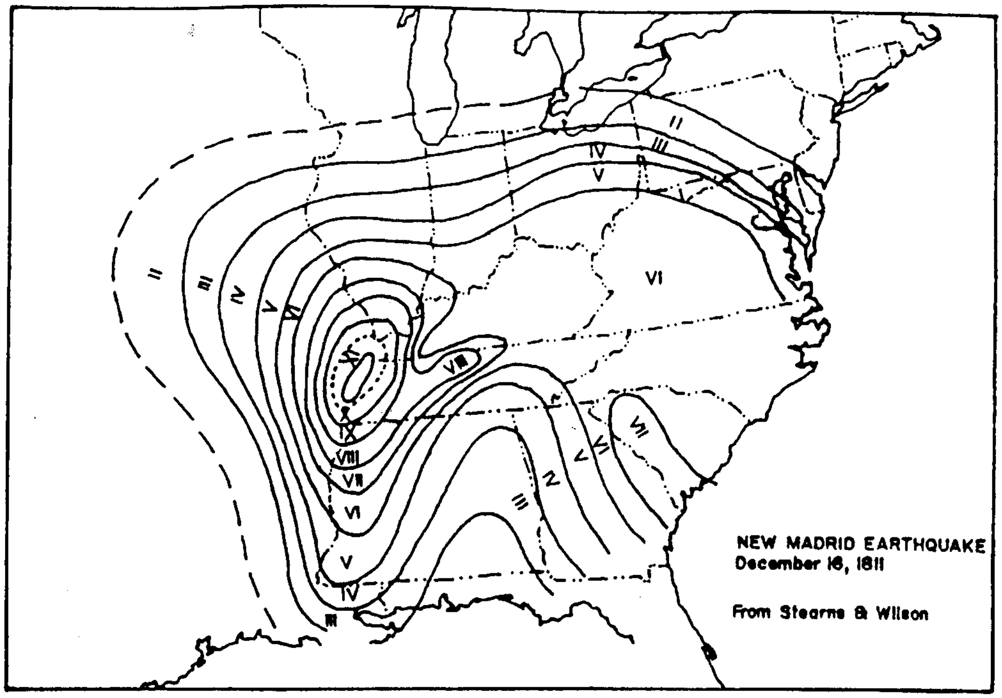
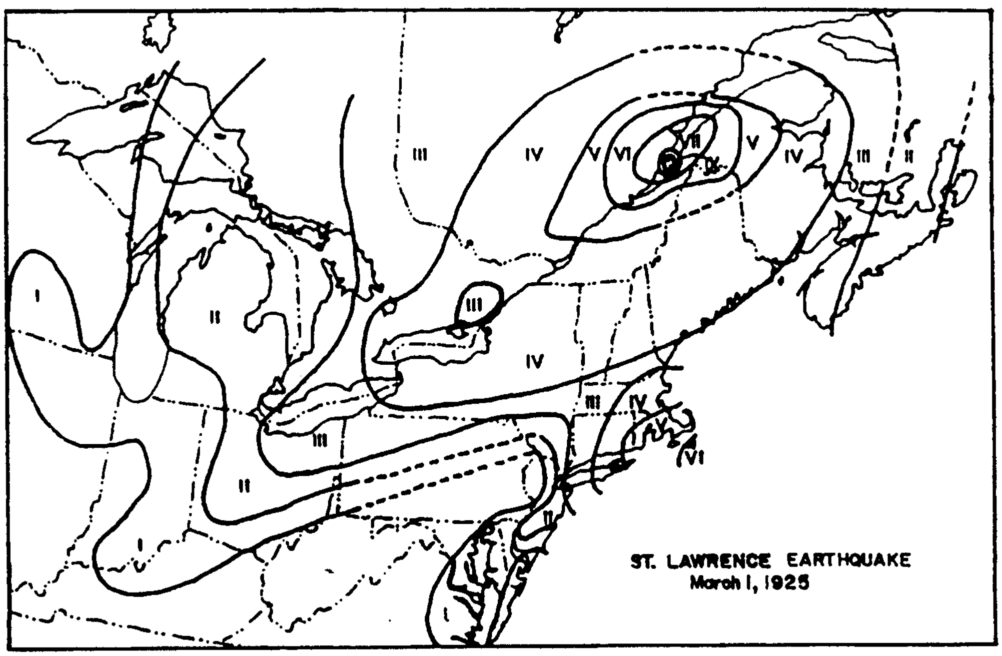
ISOSEISMAL MAPS

FIGURE: 12.





PASSAIC RIVER
FLOOD DAMAGE REDUCTION PROJECT
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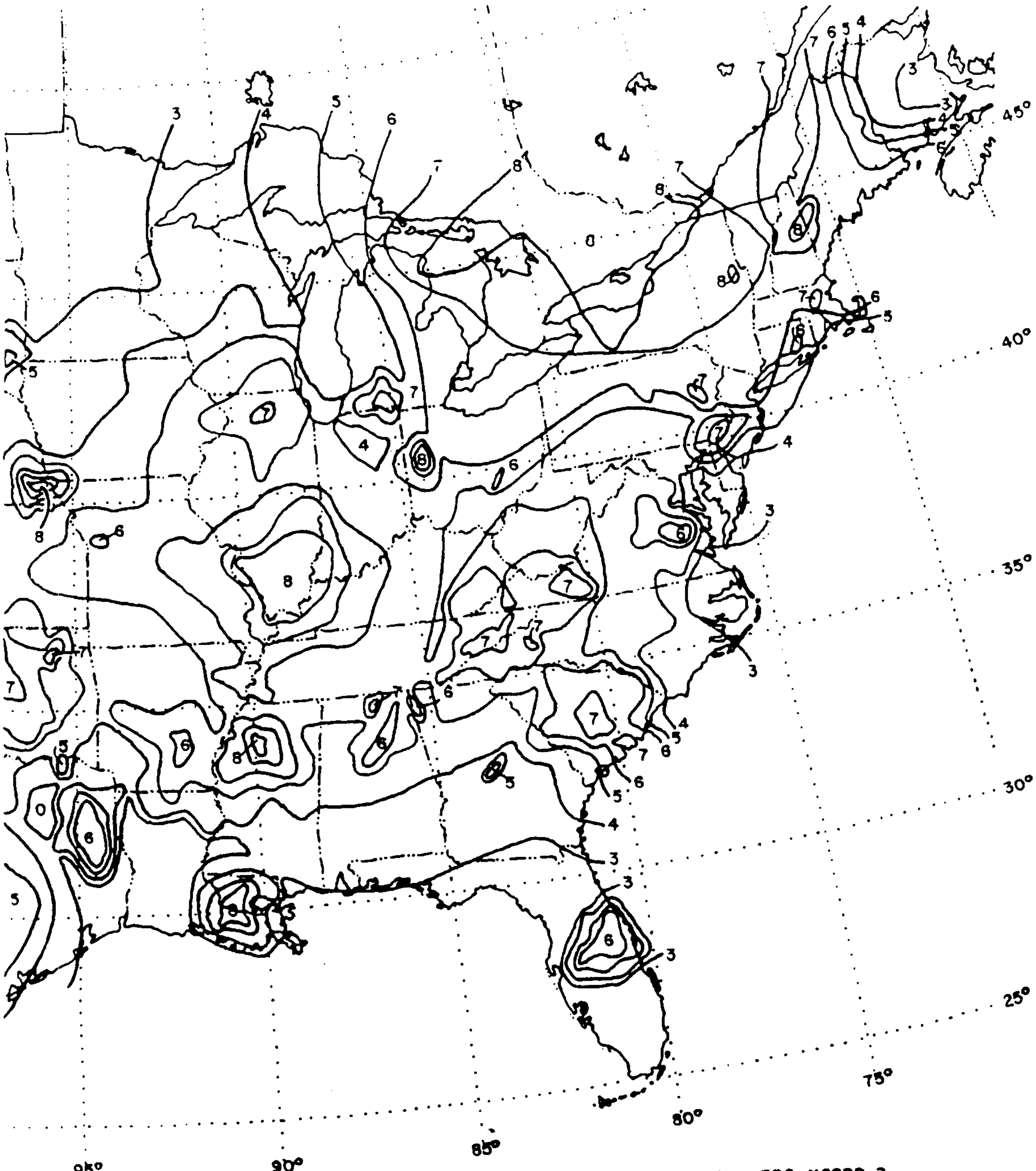
ISOSEISMAL MAP

FIGURE: 13.



From NOAA Technical Memorandum EDS NGSDC-2
August 1976

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MAXIMUM INTENSITIES EXPERIENCED FROM 1928 THROUGH 1973

FIGURE: 14.

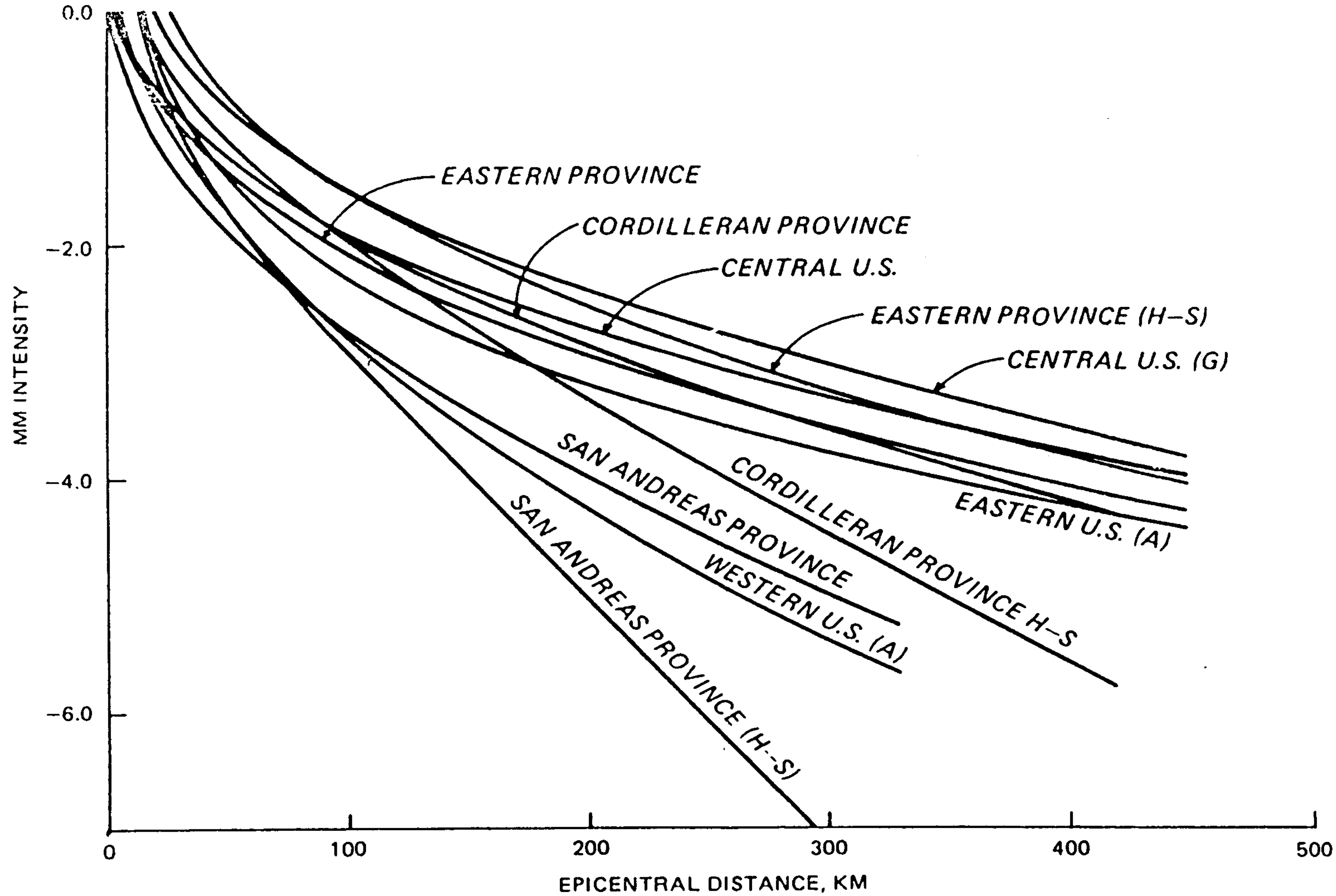


Figure 15. Attenuation of MM Intensities with distance (A = Anderson; G = Gupta; H-S = Howell-Schultz) (from Chandra, 1979)

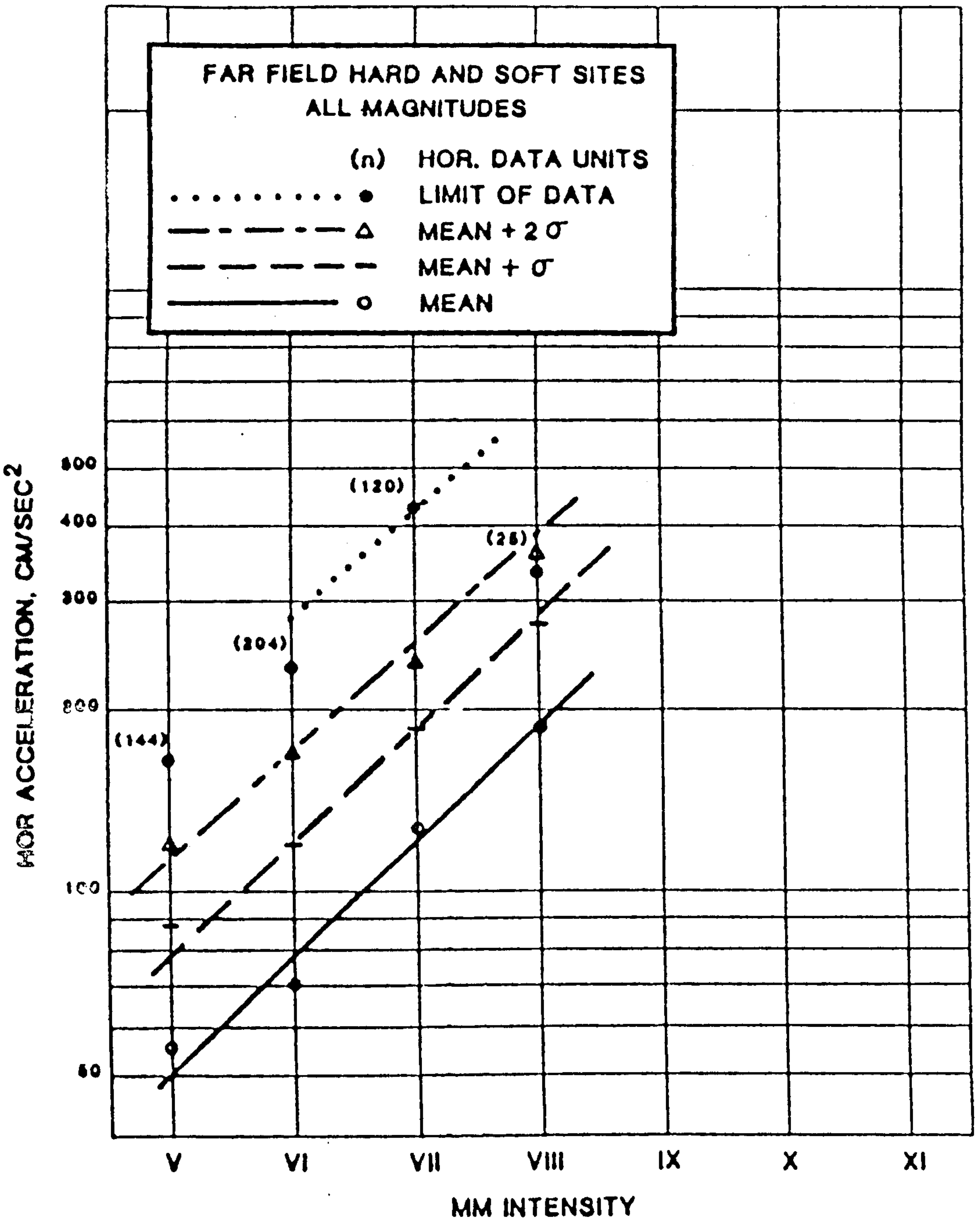


Figure 16. MM Intensity versus Horizontal Acceleration Far Field all sites from Krintzsky and Chang "Intensity-Related Earthquake Ground Motions."

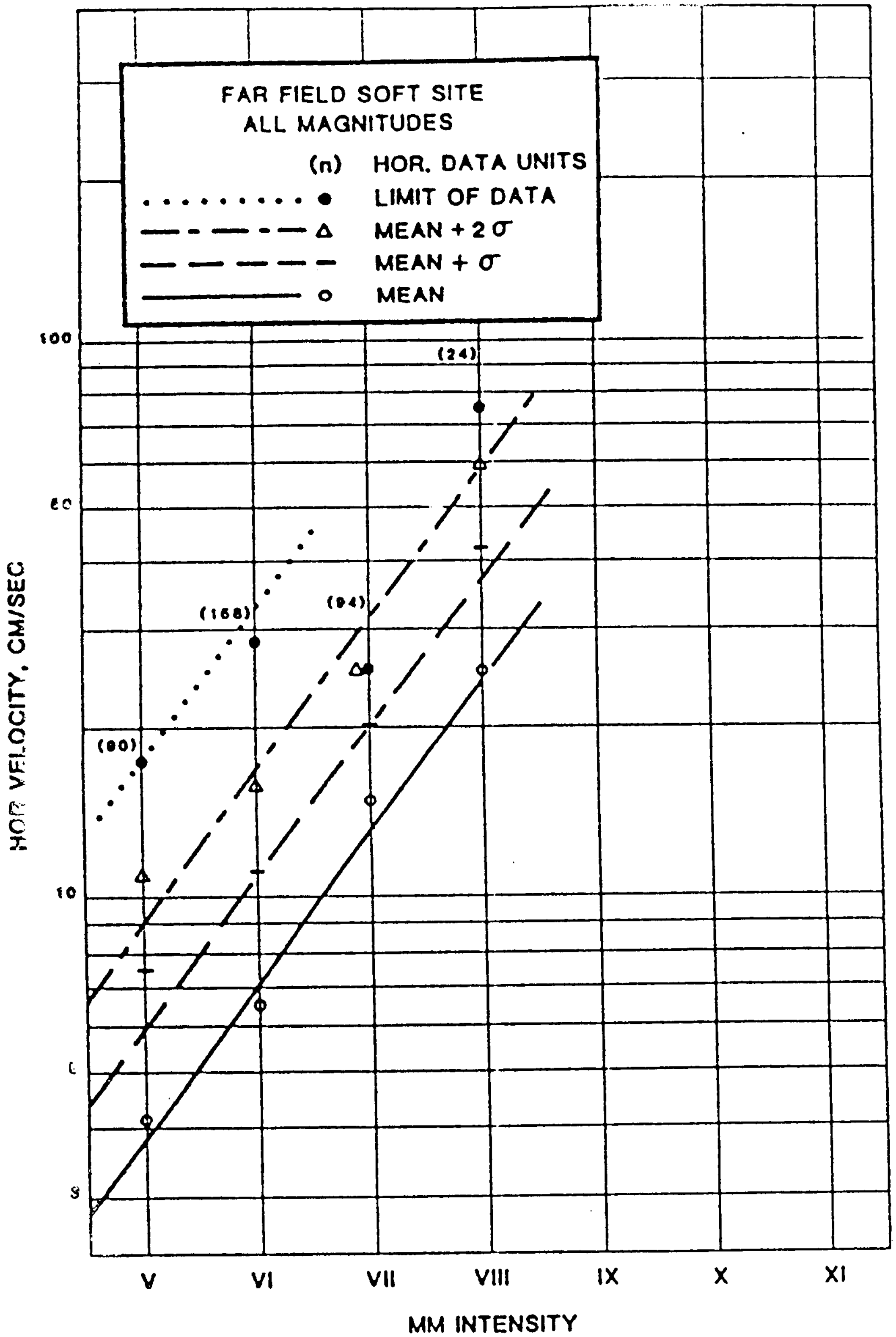


Figure 17. MM Intensity versus Horizontal Velocity Far Field soft sites from Krintzsky and Chang "Intensity-Related Earthquake Ground Motions."

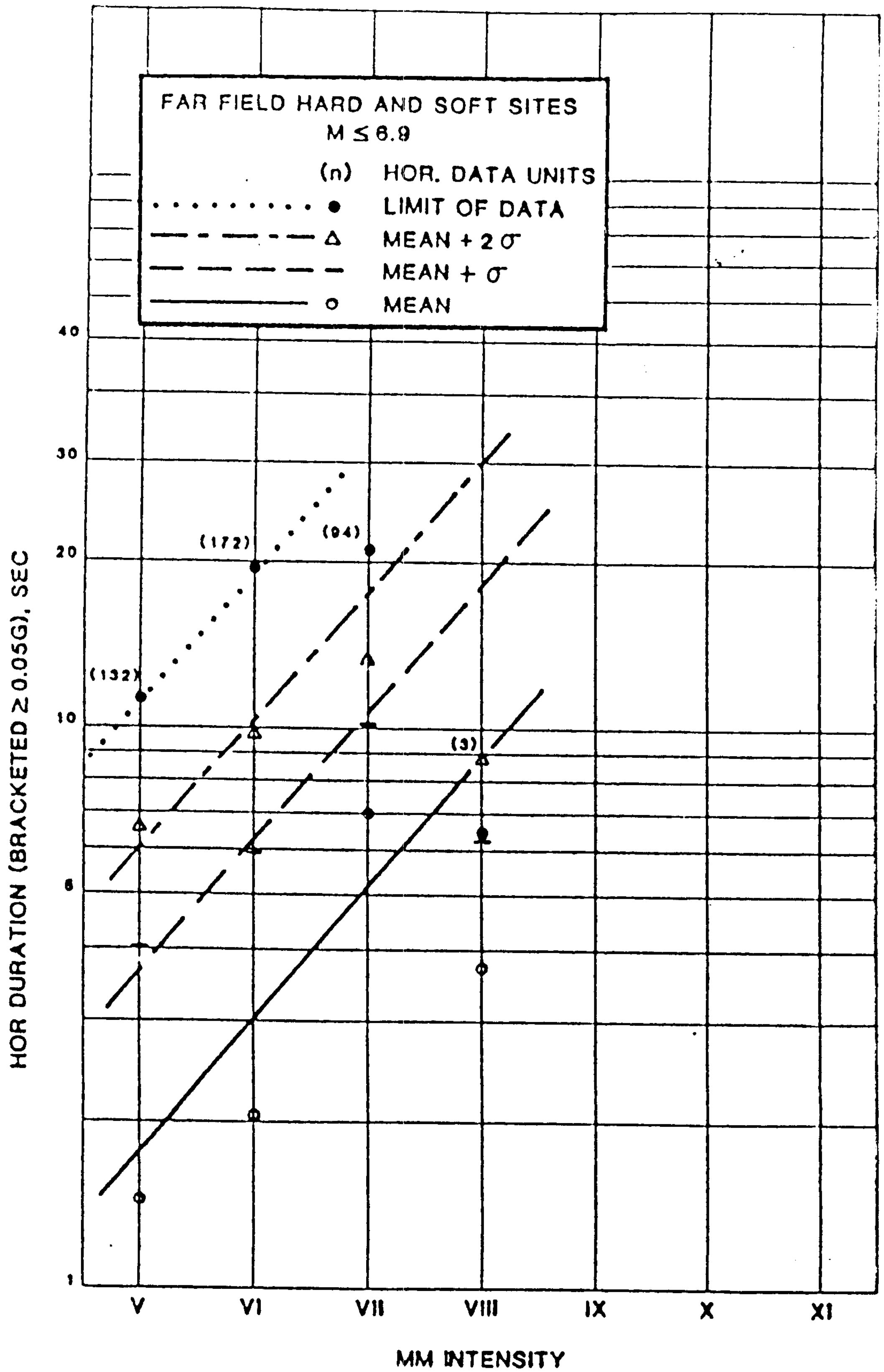
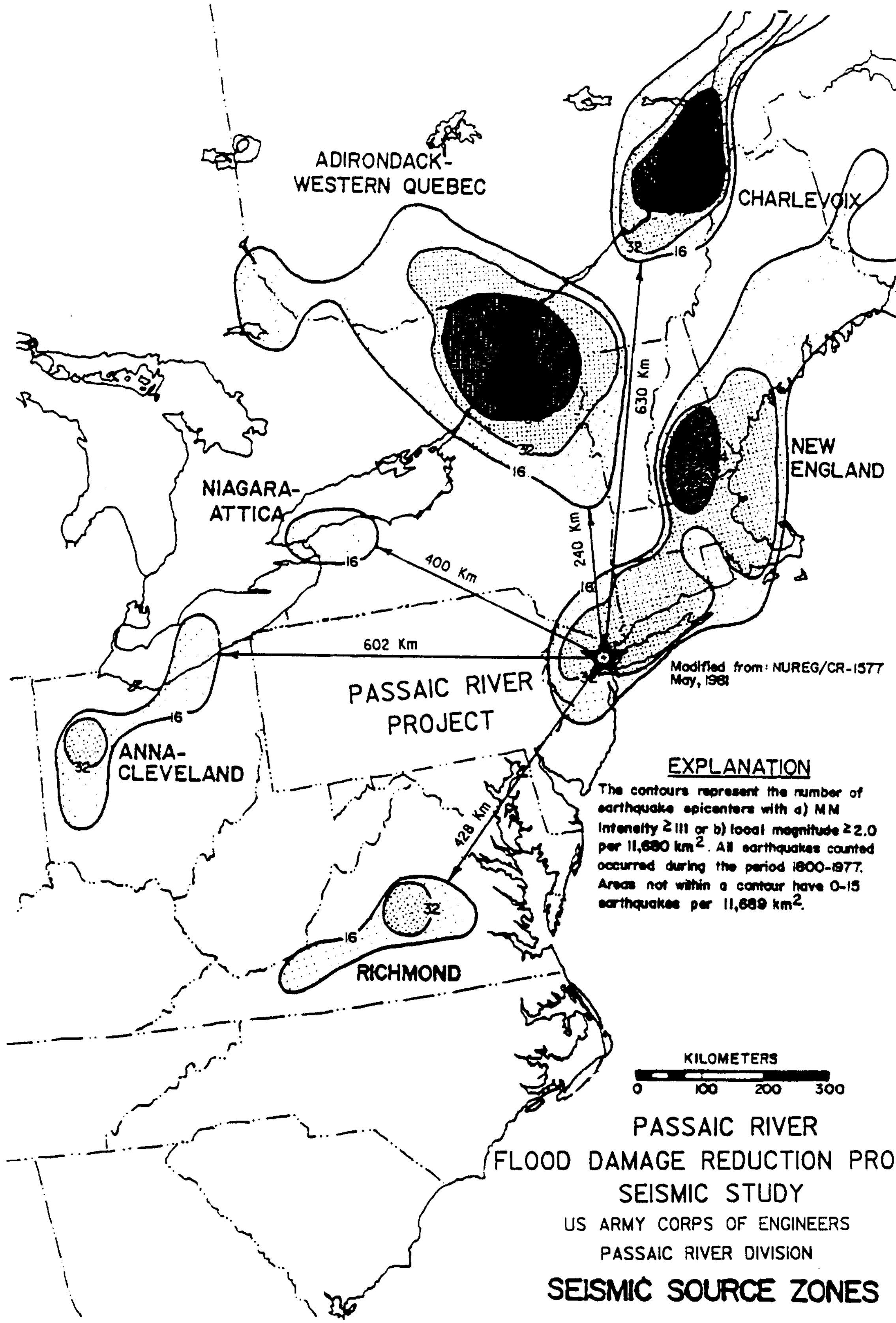


Figure 18. MM Intensity versus Horizontal Duration Far Field all sites from Krintzsky and Chang "Intensity-Related Earthquake Ground Motions."



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SEISMIC SOURCE ZONES

FIGURE: 19.

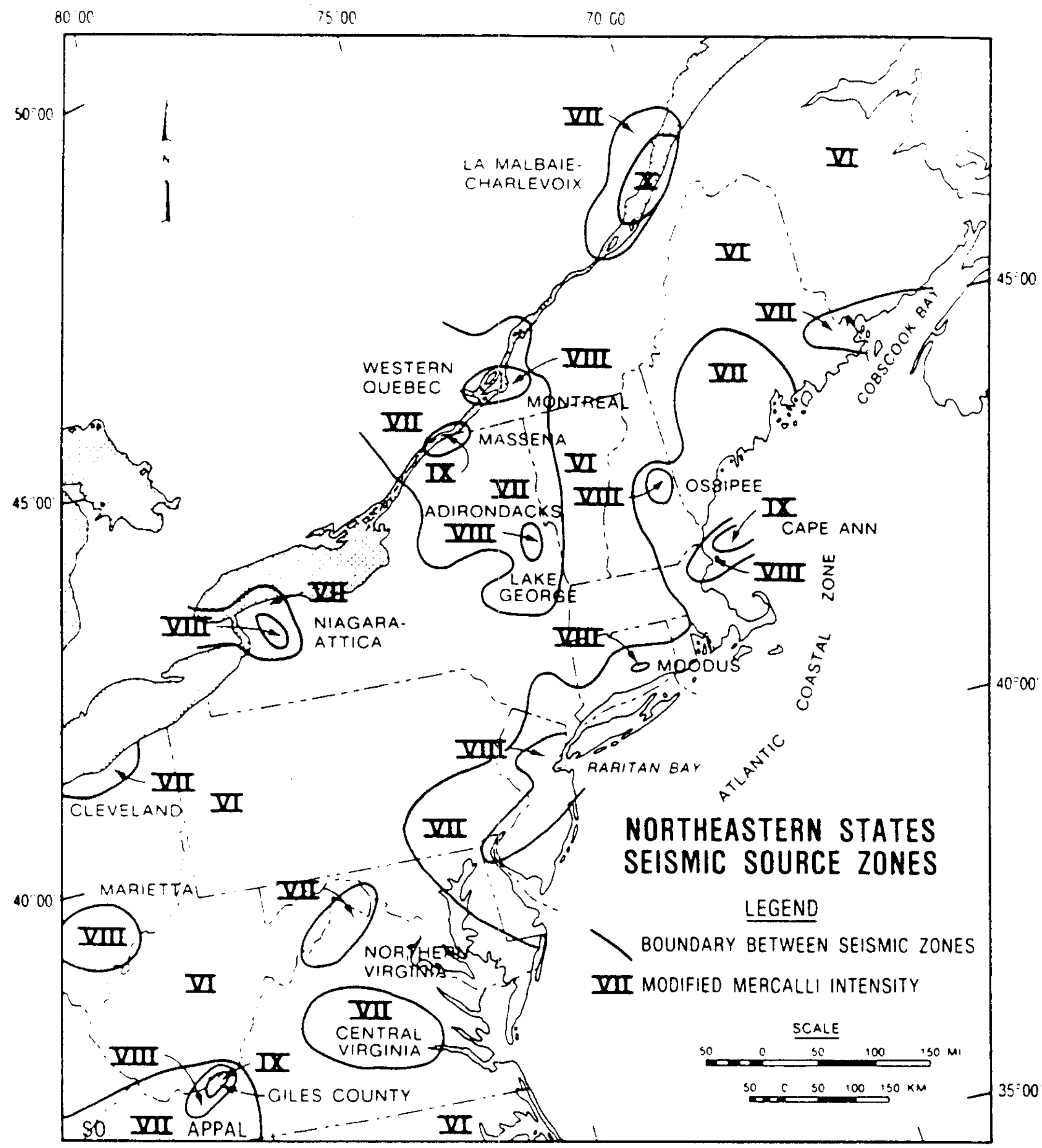
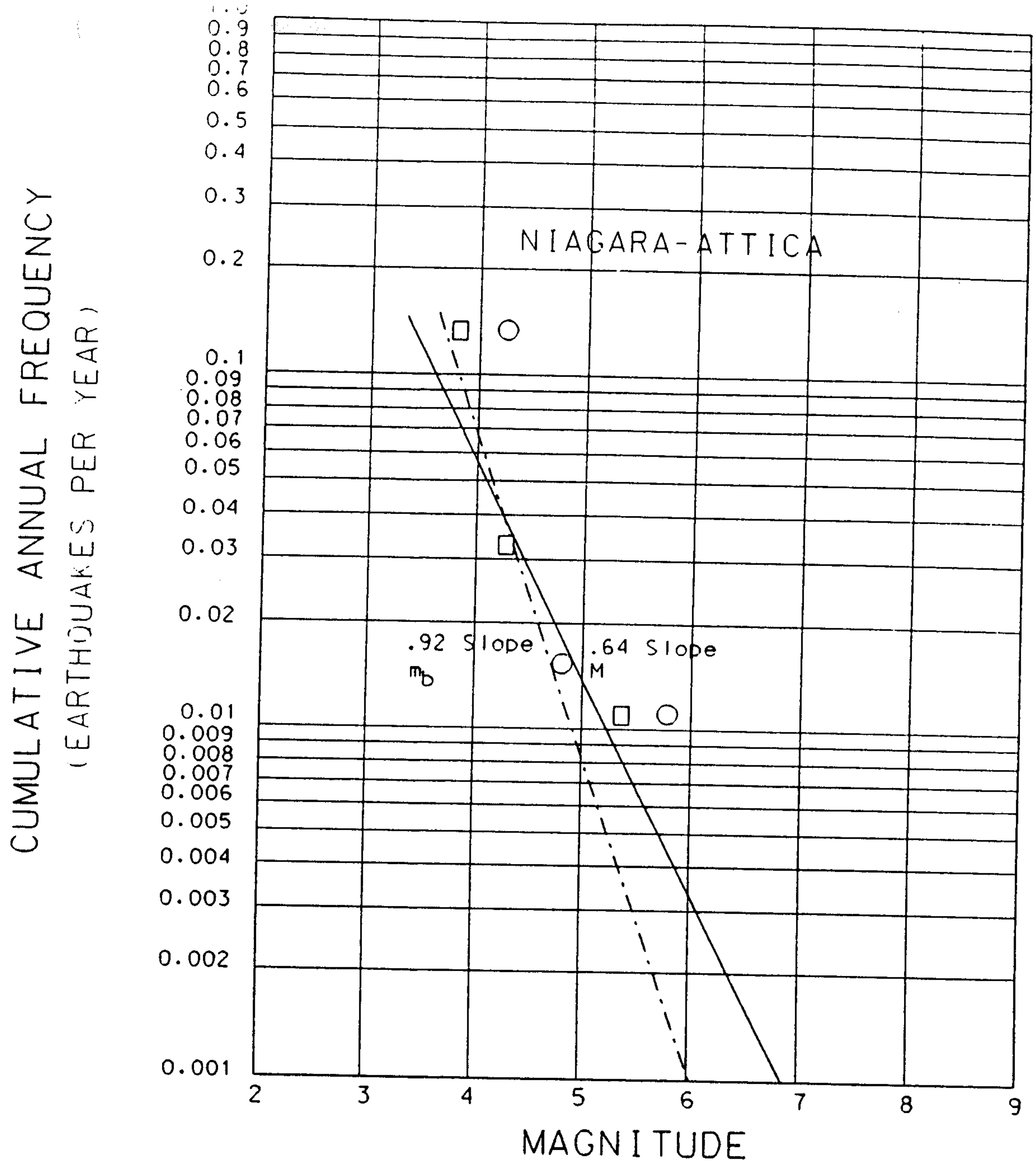


Figure 20. Seismic source zones interpreted for the Northeastern US



m_b = body wave magnitude

M = Richter magnitude

NUREG formula M=0.5I+1.75

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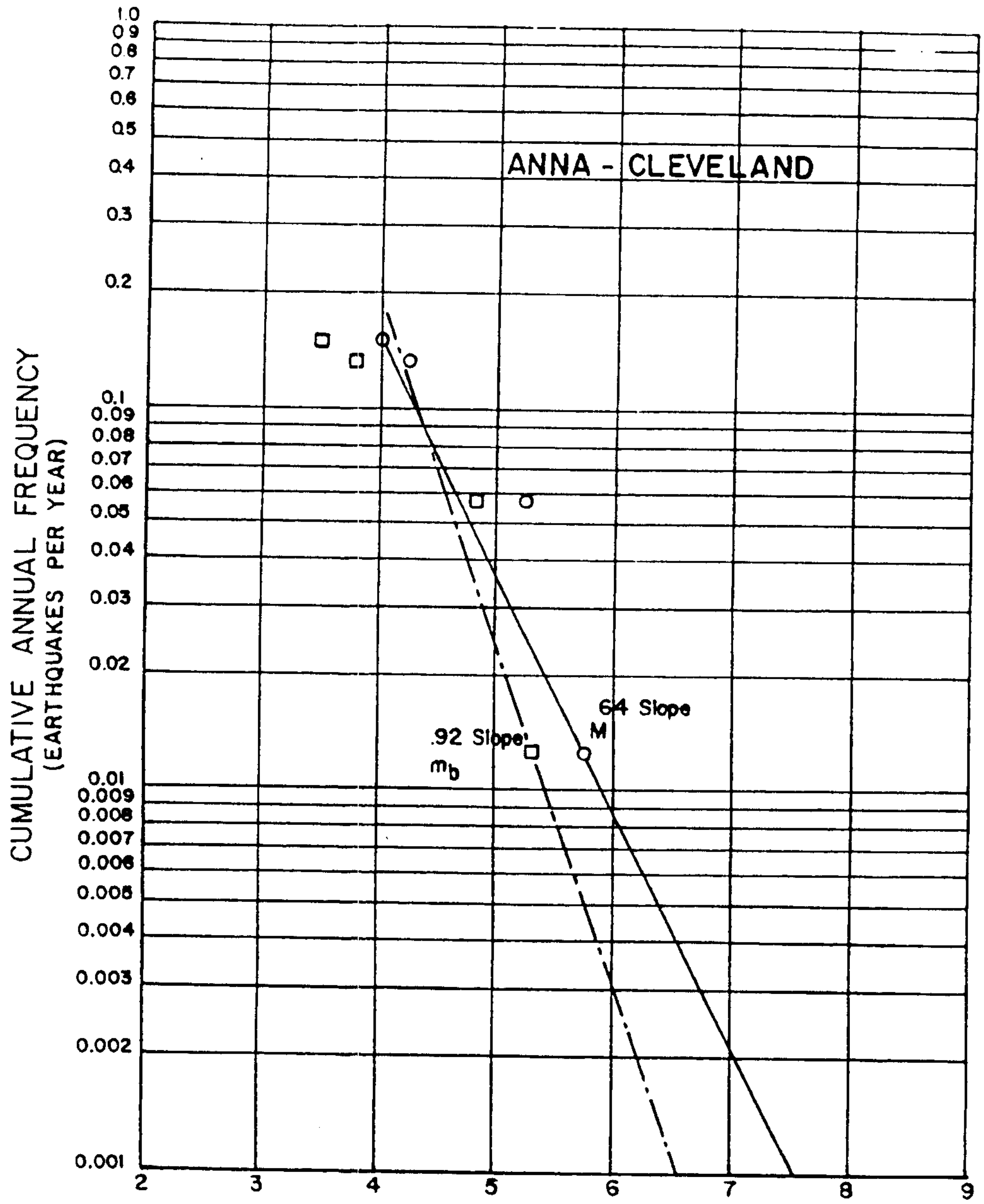
MAGNITUDE-RECURRENCE

FIGURE: 21.



m_b = body wave magnitude

M = Richter magnitude

NUREG formula M=0.5I+1.75

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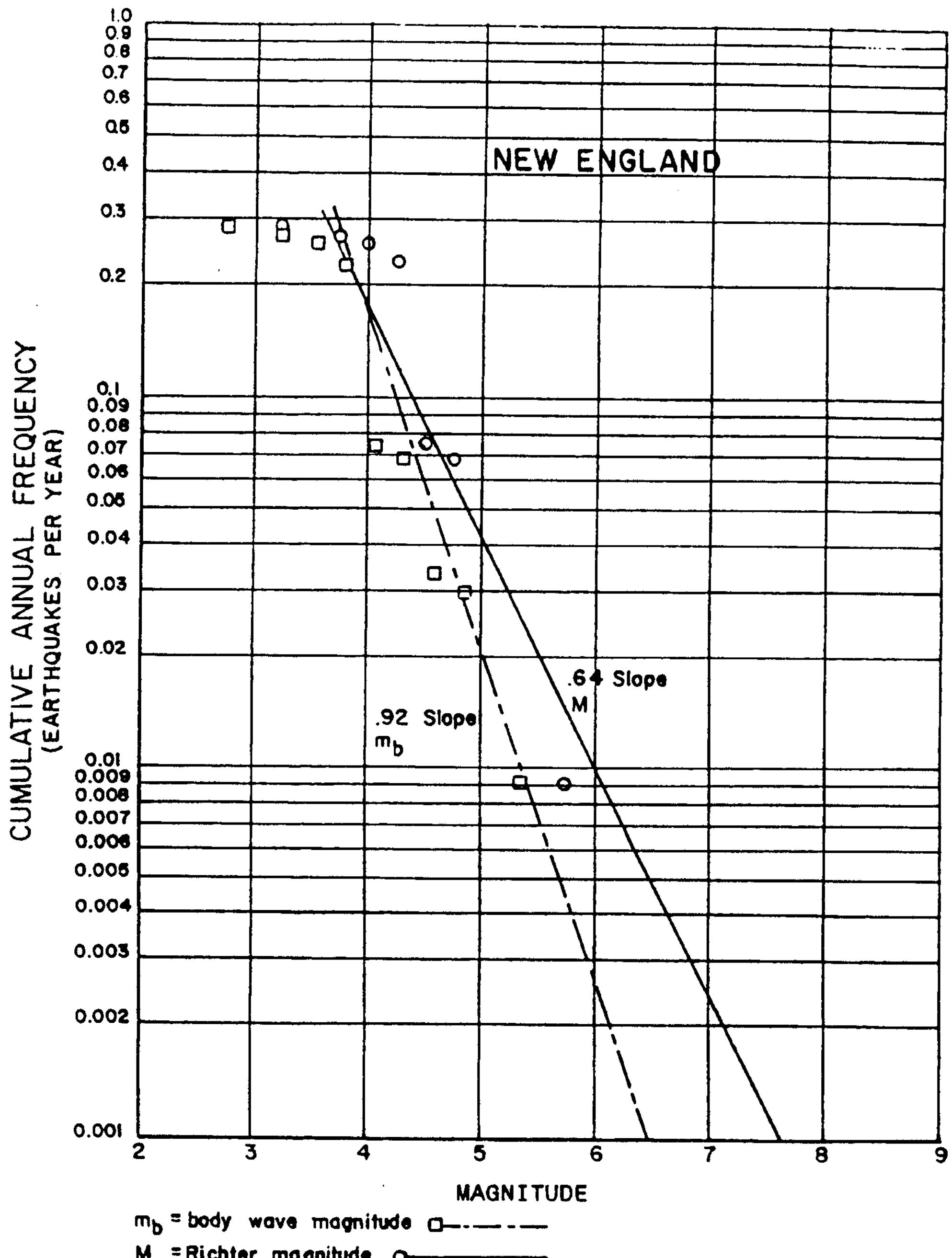
MAGNITUDE RECURRENCE

FIGURE: 22.



NUREG formula $M = 0.5\,I + 1.75$

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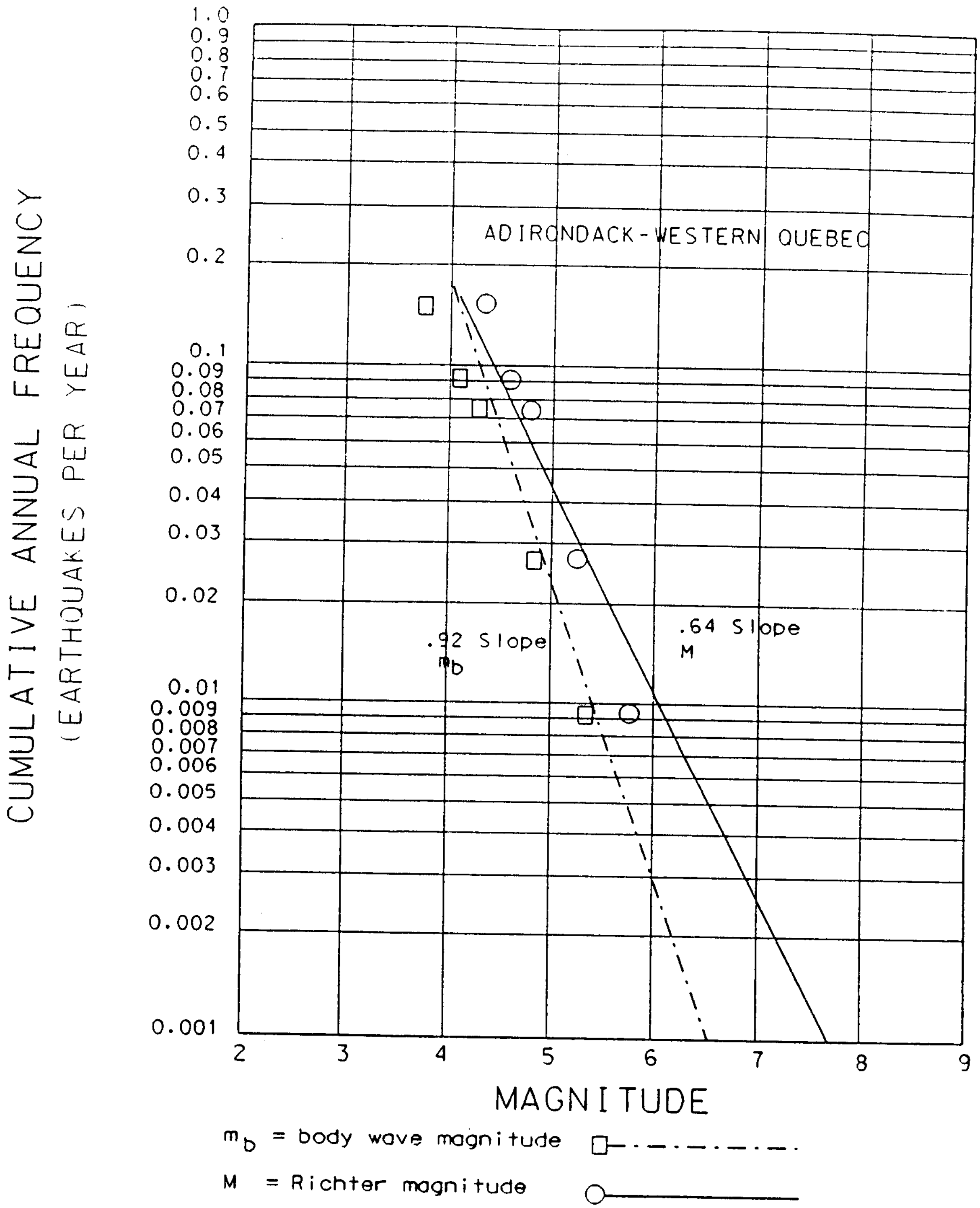
MAGNITUDE-RECURRENCE

FIGURE: 23.



NUREG formula M=0.5I+1.75

PASSAIC RIVER

FLOOD DAMAGE REDUCTION PROJECT

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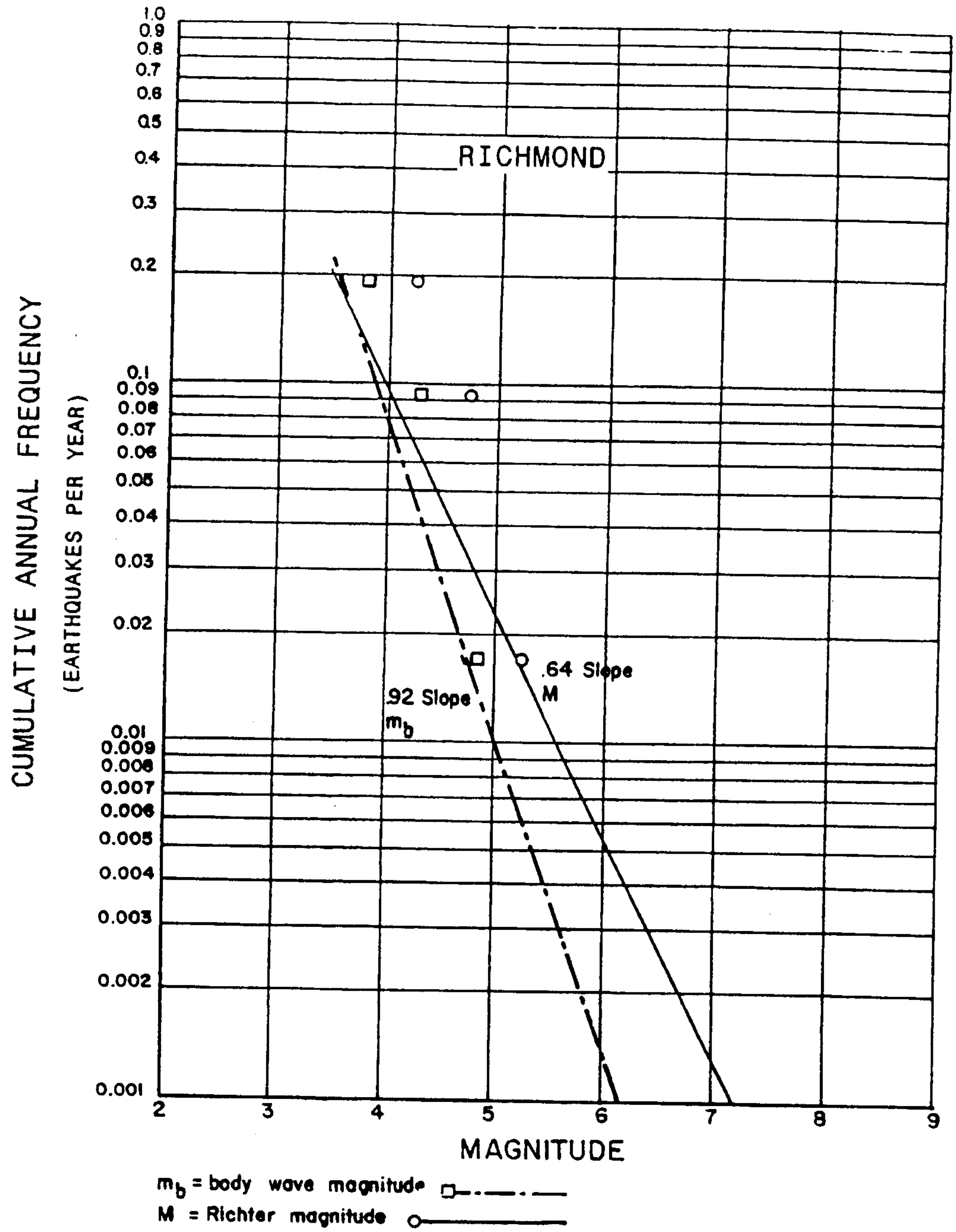
MAGNITUDE-RECURRENCE

FIGURE: 24.



NUREG formula $M=0.5I+1.75$

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FLOOD DAMAGE REDUCTION PROJECT
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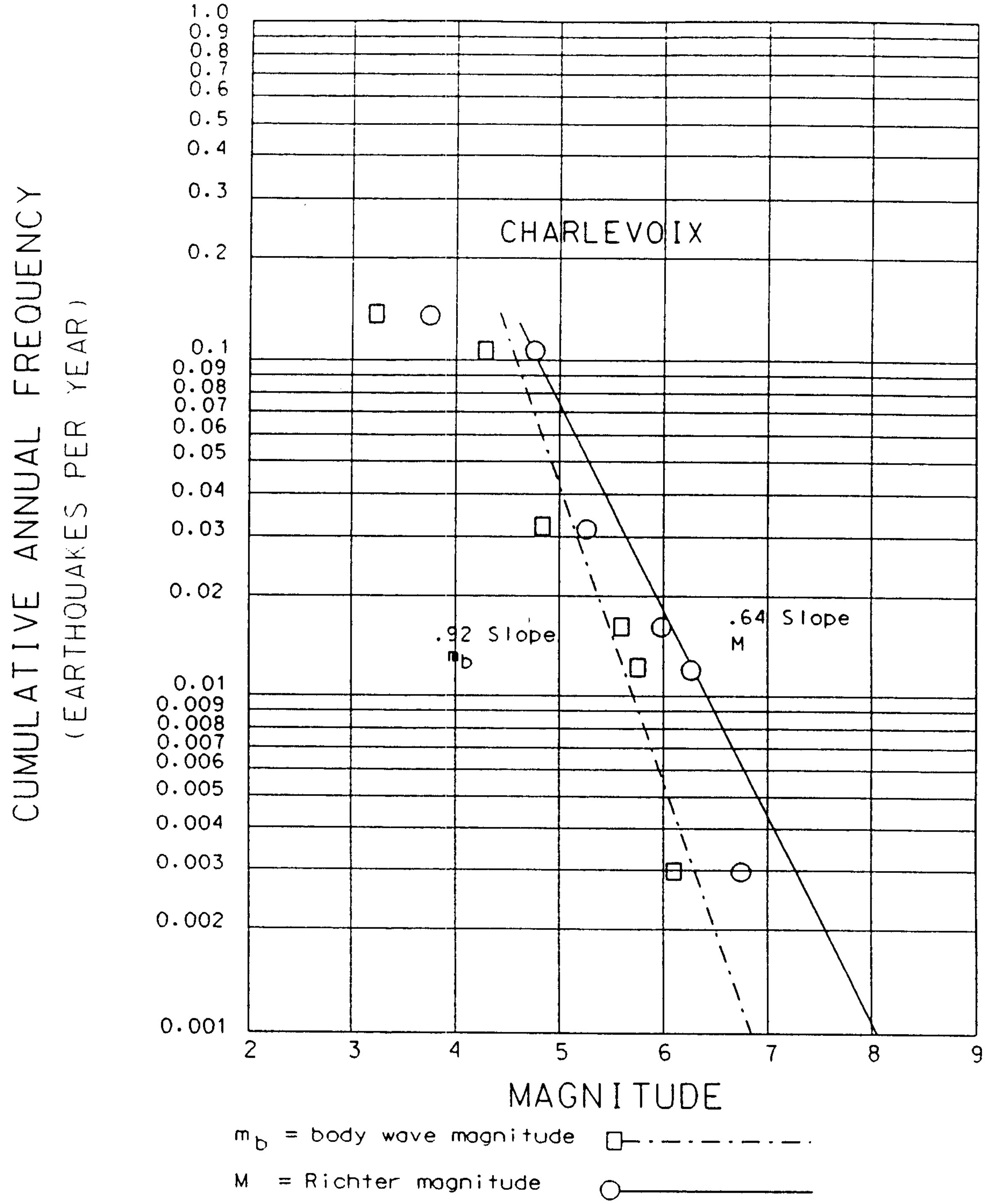
MAGNITUDE - RECURRENCE

FIGURE: 25.



NUREG formula M=0.5I+1.75

PASSAIC RIVER
FLOOD DAMAGE REDUCTION PROJECT
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MAGNITUDE-RECURRENCE

FIGURE: 26.

TABLES

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.1.1

SEISMIC STUDY

TABLES

TABLE 1

VELOCITY MODELS USED FOR EPICENTER LOCATIONS IN THE NORTHEASTERN UNITED STATES

REGION	VELOCITY km/sec	To sec	DEPTH TO TOP km	GENERALIZED LITHOLOGY*
Northern New York and Adirondacks	6.1	0.0	0.0	Granodiorite
	6.6	0.5	4.0	Gabbro
	8.1	6.3	35.0	Peridotite
Attica, NY	4.5	0.0	0.0	Sedimentary
	5.0	0.2	1.0	Metamorphics
	6.0	1.4	6.0	Granodiorite
Blue Mountain Lake, NY	5.9	0.0	0.0	Granodiorite
Southeastern New York and northern New Jersey	5.98	0.0	0.0	Volcanics
	6.62	1.0	7.0	Gabbro
	8.1	6.5	35.0	Peridotite
New England	5.31	0.0	0.0	Sedimentary
	6.06	0.16	0.88	Granodiorite
	6.59	1.78	13.09	Gabbro
	8.10	6.72	34.60	Peridotite

*Speculative Lithology for illustration only

TABLE 2
OROGENIC MOVEMENTS IN THE
APPALACHIAN REGION

Orogenic Episode and Approximate Date	Known Area of Influence	Maximum Manifestation
Appalachian movements Palisades Late Triassic (Carnian-Norian) 190 - 200 m.y.	Belt along central axis of already completed mountain chain.	Fault troughs, broad warping, basaltic lava, dike swarms.
Allegheny Pennsylvanian and/or Permian (Westphalian and later) 230 - 260 m.y.	West side of central and southern Appala-chians, south-east side of northern Appalachians,perhaps also in Carolina Piedmont.	Strong folding, also middle-grade metamorphic and granite intrusion at least in southern New England.
Acadian Devonian, mainly Middle but epi-sodic into Miss-issippian (Emsian-Eifelian) 360 - 400 m.y.	Whole of northern Appalachians, exceptnorthwest edge; as far southwest as Pennsylvania	Medium to high grade metamorphism, granite intrusion
Salinic Late Silurian (Ludlow)	Local on northwest side of northern Appalachians.	Mild angular uncon-formity, minor clastic wedge.
Taconic Middle (and Late) Ordovician (Caradoc, locally probably older) 450 - 500 m.y.	General on north-westside of northern Appalachians, local elsewhere; an early phase in Carolinas and Virginia, perhaps general in Piedmont province.	Strong angular uncon-formity, gravity slides(?), at least low grade metamorphism, granodioritic and ultramafic intrusion.
Avalonian Latest Precambrian	Southeastern New-foundland, Cape Breton Island, southern New Brunswick; probably also central and southern Appalachians (Florida?)	Probably some deformation, uplift of sources of coarse arkosic debris, gravity slides (?)

TABLE 2 (continued)
OROGENIC MOVEMENTS IN THE
APPALACHIAN REGION

Orogenic Episode and Approximate Date	Known Area of Influence	Maximum Manifestation
Late Precambrian About 580 m.y	Southeastern Newfoundland, Cape Breton Island, southern New Brunswick; perhaps eastern Massachusetts	Mostly low grade metamorphism, granitic and other intrusion
Grenville (pre-Appalachian movements) Late Precambrian 800-1100 m.y.	Eastern North America including western part of Appalachian region.	High grade metamorphism, granitic and other intrusion.

PASSAIC FLOOD DAMAGE REDUCTION STUDY
ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 3

TAKEN FROM REFERENCE 37. (SEE FIGURE 2)

SOME STRAIN-RELIEF IN-SITU STRESS MEASUREMENTS

NUMBER FROM FIGURE 2	LOCATION	PRINCIPAL STRESS (BARS)		DEPTH (METERS)	STRESS RATIO	TREND OF MAXIMUM STRESS	ROCK TYPE
		MAXIMUM	MINIMUM				
1	Barre, VT	118	54	-----	2.19	N 14 E	Granite
2	Proctor, VT	19	35	-----	2.57	N 4 W	Dolomite
3	Tewksbury, MA	81	45	-----	1.80	N 2 W	Paragneiss
4	W. Chelmsford, MA	145	76	-----	1.91	N 56 E	Granite
5	Nyack, NY	12	5	-----	2.40	N 2 E	Diabase
6	St. Peters, PA	56	23	-----	2.43	N 14 E	Norite
7	Rapadan, VA	114	94	-----	1.21	N 6 E	Diabase
8	Mt. Airy, NC	168	81	-----	2.07	N 87 E	Granite
9	Lithonia, GA	102	68	-----	1.50	N 8 E	Granite
10	Lithonia, GA	111	64	-----	1.73	N 49 E	Gneiss
11	Douglasville, GA	35	19	-----	1.84	N 64 W	Gneiss
12	Carthage, MO	217	95	-----	2.28	N 67 E	Limestone
13	Graniteville, MO	73	53	-----	1.38	N 2 E	Granite
14	Troy, OK	73	35	-----	2.09	N 84 W	Granite
15	Niagra Falls, NY	68	-0.7	-----	----	N 55 E	Dolomite
16	Barbertown, OH	440	230	850	1.91	N 90 W	Limestone
17	Gibsonville, OH	-----	-----	-----	----	N 78 E	Sandstone
18	Sudbury, ONT	510	-440	-----	----	ENE	-------
19	Eliot Lake, ONT	210	180	300-400	1.17	E	Sandstone
20	White Pine, MI	170	-----	-----	----	NW	-------
21	St. Cloud, MN	-----	-----	-----	----	N 50 E	Granite
22	Morgantown, PA	510	40	700	12.75	N 27 E	Diabase

SOME FAULT PLANE SOLUTIONS

NUMBER FROM FIGURE 2	LOCATION	FAULT PLANE SOLUTION			ORIENTATION OF P OR T AXIS		ORIENTATION OF PRINCIPAL STRESS	
		ECHANIS	STRIKE	DIP	TREND	PLUNGE	TREND	PLUNGE
23	Southern, IL	Thrust	N 15 E	45 W	83 W	1 E	85 W	15 W
23	Southern, IL	Thrust	N 1 W	47 E	----	-----	----	-----
24	Blue Mt. Lake, NY	Thrust	N 12 W	25 E	80 E	20 W	80 E	5 W
25	Lake Hopatcong, NJ	Normal	N 12 E	60 SE	58 W	11 SE	88 E	4 W

SOME HYDROFRACTURE IN-SITU MEASUREMENTS

NUMBER FROM FIGURE 2	LOCATION	PRINCIPAL STRESS (BARS)		DEPTH (METERS)	STRESS RATIO	TREND OF MAXIMUM STRESS
		MAXIMUM	MINIMUM			
26	Alma Township, NY	223	147	512	1.52	N 77 E
27	Falls Township, OH	280	150	815	1.87	N 64 E
28	Illinois	75	46	95	1.63	N 62 E
29	Bradford, PA	---	---	---	---	N 70 E

PASSAIC FLOOD DAMAGE REDUCTION STUDY
ATTACHMENT E.1.1 SEISMIC STUDY
TABLE 4
FOCAL MECHANISM STUDIES

LOCATION	STATE	LATITUDE	LONGITUDE	MAGNITUDE	INTENSITY	DEPTH (Km)	FOCAL PLANE SOLUTIONS			DATE
							STRIKE	DIP	MECHANISM	
Attica	NY	42.80	78.20	4.60	—	2.0	N14E	70SE	Reverse	01-01-1966
Attica	NY	42.90	78.20	4.20	—	3.0	N20E	72SE	Reverse	06-13-1967
Blue Lake	NY	43.90	74.50	2.2-3.6	—	<2.0	N12W	25NE	Thrust	1971
Blue Lake	NY	43.90	74.50	2.0	—	>2.0	N31E	59SE	Thrust	1971
Dover	DW	39.70	75.44	3.80	—	5-8.4	N28E	NW	Reverse	02-28-1973
Goodnow	NY	43.96	74.28	2.1	—	5.34	—	—	—	04-13-1984
Hatfield	PA	40.32	75.30	2.2	—	7.46	—	—	—	05-10-1984
Ira	VT	43.52	73.12	1.2	—	20.2	—	—	—	04-05-1984
Keene	NH	43.02	72.39	2.30	—	14.67	—	—	—	06-16-1984
Kinnelon	NJ	41.00	74.41	1.30	—	0.20	—	—	—	06-03-1984
Lake Hopatcong	NJ	—	—	1.25	—	0.5-3.3	N25E	60SE	Normal	10-06-1969
Mahopac	NY	41.35	73.83	0.7	—	7.2	—	—	—	06-06-1984
Maniwki	QB	46.46	76.21	4.2	—	17.0	N64W	65SW	Thrust	07-12-1975
Marticville	PA	40.13	76.04	2.9	—	7.5	—	—	—	04-19-1984
Marticville	PA	39.95	76.32	2.5	—	10.0	—	—	—	04-23-1984
Marticville	PA	39.95	76.32	2.3	—	10.0	—	—	—	05-17-1984
Morristown	NJ	40.78	74.48	1.70	—	7.0	—	—	—	06-06-1984
Mt. Hope	NJ	40.92	74.54	2.10	—	5.63	—	—	—	05-13-1984
Portland	ME	43.23	70.21	2.20	—	10.87	—	—	—	06-08-1984
Quabbin	MA	42.59	23.40	2.40	—	10.47	—	—	—	06-14-1984
Rotterdam	NY	42.89	74.16	2.1	—	3.98	—	—	—	05-05-1984
Utica	NY	43.19	75.17	2.5	—	4.07	—	—	—	06-01-1984
Wappingers Falls	NY	41.63	73.94	—	V	0-1.5	N40W	30SW	Thrust	06-1974

PASSAIC FLOOD REDUCTION STUDY ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

CHRONOLOGIC LISTING OF EARTHQUAKES AROUND THE PASSAIC PROJECT BY REGION

The distances are approximate and calculated based on the Latitude and Longitude.

The Inlet is located at Lat. 40.9716, Long. 74.2808. The Outlet is located at Lat. 40.7153, Long. 74.1159.

LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
-------	CT	06-28-1875	41.8	73.2	5.0	129	143
Southeastern Mass.	MA	09-21-1876	42.8	70.9	4.5	349	356
Northern New York	NY	11-04-1877	44.5	74	7.0	391	419
Hudson River	NY	10-04-1878	41.5	74	5.0	63	87
Canada, West of Buffalo	CAN	08-21-1879	43.2	79.2	5.0	482	508
Northeastern Mass.	MA	05-12-1880	42.8	70.9	4.5	349	356
Bath	ME	01-20-1881	44	70	4.5	492	502
-------	NH	12-19-1882	43.2	71.4	5.0	346	358
-------	ME	12-31-1882	45	67	5.0	757	764
-------	RI	02-27-1883	41.5	71.5	5.0	241	237
Contoocook	NH	01-18-1884	43.2	71.7	4.0	329	342
New York	NY	08-10-1884	40.6	74	7.0	47	16
Southern NH	NH	11-23-1884	43.2	71.7	5.5	329	342
Southern NH	NH	05-01-1891	43.2	71.6	5.0	334	347
New York	NY	03-09-1893	40.6	74	5.0	47	16
-------	ME	03-22-1896	45.2	67.2	4.5	757	765
Northeastern New York	NY	05-27-1897	44.5	74.5	6.0	391	420
Belfast	ME	09-17-1898	44.3	69.1	4.5	571	579
Eastern Mass.	MA	01-27-1903	42.1	70.9	5.0	311	311
Northeastern Mass.	MA	04-24-1903	42.7	71	5.0	336	342
Madrid	NY	12-25-1903	44.7	75.5	5.0	425	456
Southeastern Maine	ME	03-21-1904	45	67.2	7.0	744	751
-------	ME	07-15-1905	44.3	69.8	5.0	527	538
Rockingham County	NH	08-30-1905	43	71	4.5	356	364
Northern VT	VT	10-22-1905	44.9	72.2	4.5	469	490
Schenectady	NY	01-24-1907	42.8	74	4.5	204	231
Northeastern Mass.	MA	10-15-1907	42.8	71	5.0	342	349
Cumberland County	ME	01-22-1910	43.8	70.4	5.0	452	463
Calais	ME	12-11-1912	45	68	5.5	691	700
Potsdam	NY	04-28-1913	44.8	75.3	6.0	432	463
Lake Placid	NY	08-10-1913	44	74	5.0	336	364
-------	ME	01-13-1914	45.1	67.2	5.0	751	758
-------	CAN	02-10-1914	45	76.9	7.0	497	529
Western Maine	ME	02-21-1914	45	70.5	5.0	548	563
Lake George	NY	01-05-1916	43.7	73.7	5.0	306	332
Mohawk Valley	NY	02-02-1916	43	74	5.0	226	253
New York	NY	06-08-1916	41	73.8	4.5	41	41
Glenns Falls	NY	11-01-1916	43.3	73.7	5.0	262	288

PASSAIC FLOOD REDUCTION STUDY ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

CHRONOLOGIC LISTING OF EARTHQUAKES AROUND THE PASSAIC PROJECT BY REGION

The distances are approximate and calculated based on the Latitude and Longitude.

The Inlet is located at Lat. 40.9716, Long. 74.2808. The Outlet is located at Lat. 40.7153, Long. 74.1159.

LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
NORTHEASTERN REGION							
St. Lawrence Valley	CAN	06-13-1638	46.5	72.5	9.0	630	655
Newbury	MA	06-11-1643	42.8	70.8	3.0	356	362
St.Lawrence Valley	CAN	02-10-1661	45.5	73	7.0	513	538
St.Lawrence Valley	CAN	02-05-1663	47.6	70.1	10.0	814	834
Newbury	MA	11-09-1727	42.8	70.8	8.0	356	362
St.Lawrence Valley	CAN	09-16-1732	45.5	73.6	9.0	505	531
Boston	MA	02-17-1737	42.4	71	3.0	318	322
New York	NY	12-18-1737	40.8	74	7.0	30	14
Eastern Mass.	MA	06-24-1741	42.2	71.2	7.0	293	295
East of Cape Ann	MA	11-18-1755	42.5	70	8.0	398	399
East of Cape Ann	MA	11-22-1755	42.5	70	5.0	398	399
East Haddam	CT	05-18-1791	41.5	72.5	8.0	161	161
Exeter	NH	11-09-1810	43	70.9	6.0	362	370
Central Maine	ME	05-12-1817	46	69	6.0	712	726
Woburn	MA	10-05-1817	42.5	71.2	6.5	309	315
New London	CT	08-23-1827	41.4	72.7	4.5	141	141
Hartford	CT	04-12-1837	41.7	72.7	5.0	155	161
Southern Conn.	CT	08-09-1840	41.5	72.9	5.0	130	134
Northeastern Mass.	MA	11-27-1852	42.8	71	5.0	342	349
Northern New York	NY	03-12-1853	43.7	75.5	6.0	319	350
Newburyport	MA	12-10-1854	42.8	70.8	5.0	356	362
Canada	CAN	11-08-1855	46	64.5	6.0	993	998
Western New York	NY	10-23-1857	43.2	78.6	6.0	439	467
New Haven	CT	06-30-1858	41.8	73	5.0	141	152
Canada	CAN	10-17-1860	47.5	70	8.5	807	827
Canada	CAN	07-12-1861	45.4	75.4	7.0	499	530
-------	VT	12-18-1867	44	73	5.0	352	376
Bay of Fundy	CAN	10-22-1869	45	66.2	8.0	813	817
Canada	CAN	10-20-1870	47.4	70.5	9.0	779	800
Canada	CAN	01-09-1872	47.5	70.5	7.0	790	810
Westchester County	NY	07-11-1872	40.9	73.8	5.0	41	34
Concord	NH	11-18-1872	43.2	71.6	4.5	334	347
Ontario	CAN	06-06-1873	43	79.5	5.0	493	519
Southeastern Maine	ME	11-27-1874	44.8	68.7	5.0	632	642
Westchester	NY	12-10-1874	40.9	73.8	6.0	41	34

PASSAIC FLOOD REDUCTION STUDY ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

CHRONOLOGIC LISTING OF EARTHQUAKES AROUND THE PASSAIC PROJECT BY REGION

The distances are approximate and calculated based on the Latitude and Longitude.

The Inlet is located at Lat. 40.9716, Long. 74.2808. The Outlet is located at Lat. 40.7153, Long. 74.1159.

LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
St. Lawrence Valley	CAN	05-22-1917	45	75	4.5	450	480
Southern Maine	ME	08-20-1918	44.2	70.6	7.0	473	486
St. Lawrence Valley	CAN	09-30-1924	47.6	69.7	5.0	829	848
Eastern Mass.	MA	01-07-1925	42.6	70.6	5.0	358	362
St. Lawrence River	CAN	02-28-1925	47.7	70.5	8.0	810	831
Southeastern Mass.	MA	04-24-1925	41.8	70.8	5.0	307	304
Southeastern NH	NH	10-09-1925	43.7	70.7	6.0	427	438
Hartford	CT	11-14-1925	41.5	72.5	6.0	161	161
Manchester	NH	03-18-1926	42.9	71.4	6.0	323	333
New Rochelle	NY	05-11-1926	40.9	73.9	5.0	33	27
Western Maine	ME	08-28-1926	44.7	70	5.0	548	561
Concord	NH	03-08-1927	43.3	71.4	5.0	354	366
Milo	ME	02-08-1928	45.5	69	6.0	670	682
Saranac Lake	NY	03-18-1928	44.5	74.3	5.5	391	419
Berlin	NH	04-25-1928	44.5	71.2	5.0	469	485
Attica	NY	08-12-1929	42.9	78.3	8.0	400	427
Grand Banks	CAN	11-18-1929	44	56	10.0	1574	1566
Attica	NY	12-02-1929	42.8	78.3	5.0	394	421
-------	CAN	01-07-1931	47.4	70.5	4.0	779	800
Lake George	NY	04-20-1931	43.4	73.7	7.0	273	299
St. Johnsville	NY	10-29-1933	43	74.7	4.0	227	258
Adirondack Mountains	NY	04-14-1934	44.5	73.9	5.5	392	419
Cape Cod	MA	04-23-1935	42.2	70.2	4.0	369	368
Timiskaming	CAN	11-01-1935	46.8	79.1	6.0	762	793
Bangor	ME	08-22-1938	44.7	68.8	5.0	619	628
-------	CAN	10-19-1939	47.8	70	5.0	837	857
Buzzards Bay	MA	01-28-1940	41.6	70.8	5.0	301	296
Lake Ossipee	NH	12-20-1940	43.8	71.3	7.0	401	416
Lake Ossipee	NH	12-24-1940	43.8	71.3	7.0	401	416
Dover-Foxcroft	ME	01-14-1943	45.3	69.6	5.0	620	634
Massena	NY	09-04-1944	44.9	74.8	8.0	437	467
Dover-Foxcroft	ME	12-28-1947	45.2	69.2	5.0	634	646
Southwestern Maine	ME	10-04-1949	44.8	70.5	5.0	530	545
Rockland County	NY	09-03-1951	41.2	74.1	5.0	30	54
Burlington	VT	01-29-1952	44.5	73.2	6.0	401	426
Mohawk Valley	NY	08-24-1952	43	74.5	5.0	225	255
Poughkeepsie	NY	10-08-1952	41.7	74	5.0	84	109
South-central Quebec	CAN	10-14-1952	48	69.8	5.0	864	884

PASSAIC FLOOD REDUCTION STUDY ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

CHRONOLOGIC LISTING OF EARTHQUAKES AROUND THE PASSAIC PROJECT BY REGION

The distances are approximate and calculated based on the Latitude and Longitude.

The Inlet is located at Lat. 40.9716, Long. 74.2808. The Outlet is located at Lat. 40.7153, Long. 74.1159.

LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
Stamford	CT	03-27-1953	41.1	73.5	5.0	67	67
West-central VT	VT	03-31-1953	43.7	73	5.0	321	343
Burlington	VT	02-02-1955	44.5	73.2	5.0	401	426
Attica	NY	08-16-1955	42.9	78.3	5.0	400	427
St. Johnsbury	VT	04-23-1957	44.4	72	5.0	425	445
-------	ME	04-26-1957	43.6	69.8	6.0	476	483
Cape Elizabeth	ME	09-19-1958	43.5	70.2	5.0	443	451
Massena	NY	04-22-1961	44.9	74.9	5.0	438	468
Niagra Falls	NY	03-27-1962	43.1	79.1	5.0	469	495
-------	VT	04-10-1962	44.1	73.4	5.0	354	379
Southern Quebec	CAN	06-20-1962	45.4	72.7	5.0	508	532
Milford	NH	12-29-1962	42.8	71.6	5.0	303	313
-------	MA	10-16-1963	42.5	70.8	6.0	338	342
Peabody	MA	10-30-1963	42.7	70.8	6.0	350	355
Tilton-Laconia	NH	12-04-1963	43.6	71.6	5.0	368	383
Massena	NY	03-29-1964	44.9	74.9	5.0	438	468
Warner	NH	06-26-1964	43.3	71.9	6.0	326	341
Westchester County	NY	11-17-1964	41.2	73.7	5.0	55	64
Nantucket	MA	10-24-1965	41.3	70.1	5.0	353	344
Narraganset Bay	RI	12-07-1965	41.7	71.4	5.0	255	253
Attica-Varisburg	NY	01-01-1966	42.8	78.2	6.0	387	414
Jonesport	ME	07-23-1966	44.5	67.6	5.0	684	690
Manchester	NH	10-23-1966	43	71.8	5.0	307	319
Narraganset Bay	RI	02-02-1967	41.4	71.4	5.0	247	241
Attica-Alabama	NY	06-13-1967	42.9	78.2	6.0	393	420
Kennebec County	ME	07-01-1967	44.4	69.9	5.0	529	540
Westchester County	NY	11-22-1967	41	73.7	5.0	49	47
Southern Ontario	CAN	10-19-1968	45.4	74	5.0	491	519
Moultonboro	NH	08-06-1969	43.8	71.4	5.0	396	411

PASSAIC FLOOD REDUCTION STUDY   ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

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LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
EASTERN REGION							
Annapolis	MD	04-24-1758	38.9	76.5	3.0	296	284
Philadelphia	PA	03-17-1800	39.8	75.2	3.0	151	136
Philadelphia	PA	11-11-1840	39.8	75.2	3.0	151	136
Charlotte Court House	VA	02-02-1855	37	75.5	5.0	451	427
Wilmington	DE	10-09-1871	39.7	75.5	7.0	174	162
Arvonia	VA	12-22-1875	37.6	78.5	7.0	515	505
Delaware Valley	DE	09-10-1877	40.3	74.9	4.5	91	80
Delaware Valley	DE	03-25-1879	39.2	75.5	4.5	221	204
Hartford County	MD	03-11-1883	39.5	76.4	4.5	241	235
Hartford County	MD	03-12-1883	39.5	76.4	4.5	241	235
Allentown	PA	05-31-1884	40.6	75.5	5.0	110	117
-------	MD	01-02-1885	39.2	77.5	5.0	334	330
-------	VA	10-09-1885	37.7	78.8	6.0	525	516
-------	PA	03-08-1889	40	76	5.0	180	177
Newark	NJ	09-01-1895	40.7	74.8	6.0	53	58
Pulaski	VA	05-03-1897	31.7	80.7	6.0	1160	1141
Giles County	VA	05-31-1897	37.3	80.7	7.0	676	670
Southwestern VA	VA	10-21-1897	37	81	5.0	716	710
Ashland	VA	12-18-1897	37.7	77.5	5.0	452	439
Pulaski	VA	02-05-1898	37	80.7	6.0	696	690
Southwestern VA	VA	02-13-1899	37	81	5.0	716	710
Seaford	DE	05-08-1906	38.7	75.7	5.0	278	260
Arvonia	VA	02-11-1907	37.7	78.4	6.0	501	491
Allentown	PA	05-31-1908	40.6	75.5	6.0	110	117
Powhatan	VA	08-23-1908	37.5	77.9	5.0	490	477
Martinsburg	WV	04-02-1909	39.4	78	5.5	358	358
Arvonia	VA	05-08-191-	37.7	78.4	5.0	501	491
Luray	VA	04-09-1918	38.7	78.4	6.0	428	424
Front Royal	VA	09-05-1919	38.8	78.2	6.0	408	404
-------	NJ	01-26-1921	40	75	5.0	123	109
Mendota	VA	07-15-1921	36.6	82.3	6.0	830	825
New Canton	VA	08-07-1921	37.8	78.4	5.0	493	484
Roanoke	VA	12-25-1924	37.3	79.9	5.0	623	616
Asbury Park	NJ	06-01-1927	40.3	74	7.0	78	47
Charlottesville	VA	06-10-1927	38	79	5.0	515	509

PASSAIC FLOOD REDUCTION STUDY ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

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LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
Central VA	VA	12-26-1929	38.1	78.5	6.0	476	469
Trenton	NJ	01-24-1933	40.2	74.7	5.0	92	75
Erie	PA	10-29-1934	42	80.2	5.0	511	531
South Blair County	PA	07-15-1938	40.4	78.2	6.0	336	345
Central NJ	NJ	08-22-1938	40.1	74.5	5.0	98	75
Salem County	NJ	11-14-1939	39.6	75.2	5.0	170	153
Sinking Spring	PA	01-07-1954	40.3	76	6.0	163	165
Wilkes-Barre	PA	02-21-1954	41.2	75.9	7.0	139	159
Wilkes-Barre	PA	02-23-1954	41.2	75.9	6.0	139	159
West-central NJ	NJ	03-23-1957	40.75	74.75	6.0	46	53
-------	VA	04-23-1959	37.5	80.5	6.0	649	644
Lehigh Valley	PA	09-14-1961	40.75	75.5	5.0	105	116
-------	PA	12-27-1961	40.5	74.75	5.0	65	58
Galax	VA	10-28-1963	36.7	81	5.0	737	730
Cornwall	PA	05-12-1964	40.2	76.5	6.0	205	208
Richmond	VA	05-31-1966	37.6	78	5.0	487	475
Southern NJ	NJ	12-10-1968	39.7	74.6	5.0	143	120
Louisville	KY	12-11-1968	38.7	85.7	5.0	993	999
Southern WV	WV	11-19-1969	37.4	81	6.0	690	685
Richmond	VA	12-11-1969	37.8	77.4	5.0	438	425

PASSAIC FLOOD REDUCTION STUDY ATTACHMENT E.1.1 SEISMIC STUDY

TABLE 5

CHRONOLOGIC LISTING OF EARTHQUAKES AROUND THE PASSAIC PROJECT BY REGION

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CENTRAL REGION

LOCATION	STATE	DATE	LAT.	LONG.	INTENSITY MERCALLI	DISTANCE FROM INLET(km)	DISTANCE FROM OUTLET(km)
Western	OH	06-18-1875	40.2	84	7.0	822	833
Columbus	OH	09-19-1884	40.7	84.1	6.0	826	840
-------	OH	05-17-1901	39.3	82.5	5.0	716	722
Ohio Valley	OH	09-22-1909	38.7	86.5	5.0	1058	1065
Southeastern OH	OH	11-05-1926	39.1	82.1	6.5	689	695
Cleveland	OH	09-09-1928	41.5	82	5.0	652	669
Bellefontaine	OH	03-08-1929	40.4	84.2	5.0	837	849
-------	OH	09-30-1930	40.3	84.3	7.0	846	858
Anna	OH	09-20-1931	40.4	84.2	7.0	837	849
Western OH	OH	03-02-1937	40.4	84.2	7.0	837	849
Western OH	OH	03-08-1937	40.4	84.2	7.0	837	849
Lake Erie area	OH	03-08-1943	41.6	81.3	4.5	594	612
Southeastern OH	OH	06-20-1952	39.7	82.2	6.0	681	689
Cleveland	OH	05-26-1955	41.5	81.7	5.0	627	644
Cleveland	OH	06-28-1955	41.5	81.7	5.0	627	644
Cleveland	OH	05-01-1958	41.5	81.7	5.0	627	644
Northwestern OH	OH	02-22-1961	41.2	83.4	5.0	767	783
Columbus	OH	04-08-1967	39.6	82.5	5.0	708	716
-------	OH	04-27-1967	39.6	82.5	5.0	708	716

PASSAIC RIVER FLOOD REDUCTION STUDY

ATTACHMENT E.1. SEISMIC STUDY

TABLE 6. SEISMIC PARAMETERS OBE WITH 100 YEAR RETURN PERIOD

METHOD 1 CORNELL AND MERTZ (POINT SOURCE)

SEISMIC ZONE	DISTANCE (KILOMETERS)	Mb BODY WAVE MAGNITUDE		PEAK HORIZONTAL ACCELERATION (cm/sec/sec)	PEAK HORIZONTAL ACCELERATION % of g		
Niagra-Attica	400	4.9		1.17	0.12		
Cleveland-Anna	602	5.4		0.61	0.06		
New England-Raritan Bay	15	5.3		153.2	15.7		
Adirondack-Western Quebec	240	5.4		6.52	0.67		
Richmond	428	5		1.11	0.11		
Charlevoix	630	5.75		0.79	0.08		

METHOD 2 KRINITZSKY AND MARCUSSON (INTENSITY CURVES)

SEISMIC ZONE	DISTANCE (KILOMETERS)	EPICENTRAL INTENSITY	SITE INTENSITY (Chandra)	* PEAK ACCELERATION (MEAN+SIGMA) (cm/sec/sec)	PEAK HORIZONTAL ACCELERATION % of g	PEAK HORIZONTAL VELOCITY	PEAK HORIZONTAL DURATION (Seconds > .05g)
Niagra-Attica	400	VI	2.2	Negligible	Negligible	Negligible	Negligible
Cleveland-Anna	602	VII	2.4	Negligible	Negligible	Negligible	Negligible
New England-Raritan Bay	15	VII	6.4	134	13.7	14	8
Adirondack-Western Quebec	240	VII	4.1	Negligible	Negligible	Negligible	Negligible
Richmond	428	VI	2.3	Negligible	Negligible	Negligible	Negligible
Charlevoix	630	VIII	3	Negligible	Negligible	Negligible	Negligible

* Assumed soil site because of deep cover of glacial material

APPENDIX E

SECTION 2

GROUNDWATER

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

GEOTECHNICAL DESIGN
APPENDIX E

SECTION 2
GROUNDWATER

FIGURES

TABLES

APPENDIX E

SECTION 2

GROUNDWATER STUDY

2.1 INTRODUCTION

A comprehensive groundwater study was performed for this GDM because of the importance of the potential impacts of tunnel construction and operation on groundwater resources. It is anticipated that the groundwater modeling performed as a part of this investigation will be adequate for feature design studies. Quantitative studies were performed for the tunnel and Great Piece Meadow and qualitative evaluations for other project features.

The proposed tunnel system consists of a main 20.1-mile long, 40-foot diameter diversion tunnel (Main Tunnel) along with a 1.2-mile long, 20-foot diameter spur (Spur Tunnel). See Fig. E.2.1 for tunnel location map. The Main Tunnel will convey flood waters from the upper reach of the Pompton River to an outlet in Newark Bay located in the vicinity of Kearny Point. The Spur Tunnel will convey flood waters from the Passaic River through an inlet located just south of the confluence of the Passaic and Pompton Rivers to an underground junction with the Main Tunnel.

The specific environmental concerns in the proposed tunnel area include (1)reduction of hydraulic head in local aquifer systems and interference with local water users; (2)seepage of potentially contaminated groundwater into the tunnel during construction activities and worker exposure; (3)seepage of potentially contaminated groundwater into the tunnel during operation, and (4)mobilization of contaminants at Hazardous, Toxic, and Radioactive Waste (HTRW) sites near the tunnel or shafts which may possibly affect the local groundwater use. Groundwater inflow into the tunnel and shafts during construction is also an engineering concern.

The objectives of the groundwater investigation were to characterize the hydrogeologic environment and to obtain estimated aquifer parameters for groundwater flow modeling. Data were also used to develop a regional hydrogeologic framework. Data collected during the groundwater investigation were used during the modeling study to evaluate the interconnection of the shallow subsurface with the deep bedrock aquifers, and the potential for tunnel construction and operation activities to mobilize contaminants. Other objectives include estimation of inflow quantities of groundwater into the tunnel and shaft excavations during and following construction, localized interference with water supply wells, and dewatering-induced settlements.

The groundwater investigation included intrusive investigations ranging from straddle packer testing in pilot boreholes to multi-well, multi-zone pumping tests at several shaft locations. The groundwater investigation was conducted in conjunction with the HTRW field investigation to minimize the number of boreholes, samples, and field tests required.

2.1.1 Regional Environmental Setting

2.1.1.1 Physiography

New Jersey has been divided into four general physiographic province, which have distinctive rock types, landforms, and drainage patterns (New Jersey Geological Survey [NJGS], 1994). From northwest to southeast, these regions are: Valley and Ridge, Highlands, Piedmont, and Coastal Plain. The upper portion of the Passaic River basin lies in the Highlands, while the majority of the river basin lies in the Piedmont Province. The Passaic River Tunnel project lies entirely in the Piedmont Province. See Fig. E.2.1 for project location map.

2.1.1.2 Regional Geology

The Piedmont province is the result of sedimentation and igneous activity in a Mesozoic aged geologic feature known as the Newark Basin. This rift basin developed as one of a series along the eastern seaboard of North America, from Florida to Nova Scotia in which large, elongate crustal blocks were dropped downward during the initial stages of the opening of the Atlantic Ocean.

The rocks of the Newark Basin include Triassic and Jurassic interbedded sandstone, siltstone, shale, conglomerate, basalt, and diabase. These rocks form a broad lowland area interrupted by long northeast-southwest trending ridges which are formed by the erosion-resistant diabase and basalt formations. Bedrock within the Newark Basin include, from oldest to youngest; the Passaic Formation, the Orange Mountain Basalt, the Feltville Formation, the Preakness Basalt, the Towaco Formation, the Hook Mountain Basalt and, the Boonton Formation.

The region traversed by the tunnel project is mantled by deposits of unconsolidated sediments. These deposits are Quaternary in age, and most were formed during several Pleistocene glaciations of the region. These glacial deposits may be broadly grouped into three categories: continuous or discontinuous sheets of glacial till; lacustrine (i.e., lakebed) deposits of silt, clay, and fine sand; and coarser-grained outwash and kame deposits.

2.1.1.3 Regional Hydrogeology

Three types of stratigraphic units can generally be defined in the project area. These include sedimentary rocks of the Newark Group, basalt flows of the Newark Group, and unconsolidated sediments (Hoffman, 1989a; Gill and Vecchioli, 1965; Nichols, 1968; and Hoffman and Quinlan, 1994). These groupings are exceptionally broad as the hydrogeologic properties and hydraulic interconnection of these units are very heterogeneous.

The sedimentary rocks of the Newark Group contain both confined and unconfined aquifers. Unconfined conditions generally occur in upland areas where overlying unconsolidated deposits are thin or absent. Confined and semi-confined conditions exist in lowland areas, especially where clay beds in the unconsolidated Quaternary deposits mantle the underlying rock units. Similarly, the unconsolidated Pleistocene deposits have varied hydrogeologic characteristics and may comprise both unconfined and confined aquifers.

Groundwater is used for municipal, commercial, industrial and individual domestic water supplies along the tunnel alignment. The degree of usage varies depending on the availability of surface water and the hydrogeologic and economic factors that would favor groundwater usage.

Groundwater is derived from both the unconsolidated glacial and alluvial materials as well as the fractured bedrock. The fractured bedrock produces small to moderate and sometimes large water supplies. Where the unconsolidated materials consist of thick stratified sand and gravel deposits in buried glacial valleys, high capacity wells, capable of pumping more than 1,000 gallons per minute (gpm), are not uncommon, especially in the southern part of the Central Passaic River Basin (Hoffman and Quinlan, 1994). In general, the most productive surficial wells yield more groundwater than the most productive bedrock wells.

2.2 INVESTIGATIVE APPROACH

Hydrogeologic field investigations were conducted to obtain data at several proposed workshaft and inlet locations along the planned Passaic Tunnel alignment. The data from the field investigation and information from available literature were used to develop groundwater models for seven areas along the alignment.

2.2.1 Field Investigations Activities The field investigation activities included soil borings, soil and rock sampling, well installation, borehole geophysics, and hydraulic testing of the aquifers for the Passaic River Flood Protection Project area. The results from these field activities were used to characterize the hydrogeologic environments and to provide information used to construct groundwater models of the project area.

2.2.2 Numerical Simulation Numerical groundwater models were constructed, calibrated, and run for seven different areas. These areas were chosen because they have different geologic, hydrologic, and topographic characteristics or specific engineering concerns. In general, the hydrogeologic conditions for these models covered the spectrum of conditions that were anticipated in the vicinity of the tunnel alignment. Results of these models were used to predict potential hydrological impacts resulting from tunnel construction in these and other areas that were not specifically modeled.

2.3 GROUNDWATER INVESTIGATION

Three separate hydrogeologic/modeling studies were conducted as part of the investigation; a major hydrogeologic study for the entire tunnel length, a hydrogeologic study of the Pompton River Inlet area and, a hydrogeologic study of the Spur Tunnel Inlet area. The results of these studies are summarized below. The complete studies are available in the Passaic River Division office.

2.3.1 Tunnel Groundwater Investigation The purpose of this investigation was to estimate the potential impact of groundwater on the design of the tunnel as well as estimate the potential impact of tunnel construction and operation on the local groundwater resources. Field investigations were conducted at five locations and groundwater simulations were performed for these areas. See Figure E.2.2 for workshaft locations.

2.3.1.1 Borings and Wells As part of the geotechnical boring program, groundwater measurements were made when initially encountering the water table, at the beginning of each day, and at hole completion. Refer to the geologic profile drawings in Section 3 for approximate groundwater levels. In addition, pressure tests were made in the rock at 10 ft intervals to determine in-situ rock permeability. Some borings were converted to observation wells on which monthly measurements have been made since completion of the borings. The purpose of these measurements was to provide a data base for seasonal groundwater fluctuations.

2.3.1.2 Site Investigations

Field investigation results for Workshafts 3, 2, 2C, 2BF and 2BK were obtained using a variety of investigative techniques including geotechnical, geophysical, and hydraulic analyses. It should be noted that Workshaft designations 2BF and 2BK refer to the site investigations performed at the Fiore site and the

Keegan Landfill site, respectively. Ultimately, the Keegan Landfill site was selected as the Workshaft 2B location. The intrusive field investigations at the Workshaft sites included soil sampling, rock coring, soil and rock characterization, and the installation of a combination of bedrock boreholes, overburden wells, multiport wells, and pumping-test wells. In addition, extensive geophysical and hydraulic testing programs were completed at each workshaft. The results from these field investigations provided the information needed to characterize the hydrogeologic environment, estimate aquifer parameters for groundwater modeling, and were incorporated with data from other locations to develop a regional hydrogeologic framework of the Passaic River Flood Protection Project area. The workshaft groundwater investigation sites are shown in Figure E.2.2

The stratigraphy encountered in the pumping-test borehole at Workshaft 3, from ground surface to the top of competent bedrock at 101 feet, consisted of silty gravel fill to a depth of 2 feet; brown-gray clay with little sand to 5 feet; brown-green, coarse sand and fine gravel with some silt to 11 feet; brown, silty clay to 19 feet; brown clay to 39 feet; red, varved clay to 61.5 feet; sandy clay to 74 feet; glacial till consisting of red, sandy gravel and silty coarse sand to approximately 97 feet; and weathered gray shale fragments mixed with clay to 101 feet. The stratigraphy in the overburden borehole was similar, with the upper sand and gravel deposit at 3.5 to 11.5 feet, underlain by brownish-gray, silty slay.

The bedrock at the Workshaft 3 location is indicative of the Towaco Formation. It consists of dusky-red, micaceous shale with thin laminations from 101 to 165 feet below ground surface; medium-light-gray siltstone with 1/2-inch bedding planes and calcite veins from 165 to 185 feet; and weathered, black shale with fine laminations and a hydrocarbon odor from 185 to 190 feet. The medium-light gray siltstone reoccurred form 190 to 194 feet and dusky-red shale reoccurred from 194 feet to the bottom of the borehole at 355 feet.

The bedrock surface at Workshaft 2 was encountered at approximately 20 feet below existing grade, and was overlain by weathering products consisting of silt, clay, and sand, and sandstone rock fragments. The grayish-red, fine-grained sandstone containing quartz and calcite veins encountered through the length of the borehole is indicative of the middle unit of the Passaic Formation. The bedrock is medium-hard, except for soft zones at 450 to 465 feet and 500 to 525 feet. Water-bearing zones were encountered in the borehole at depths of 50 feet, 75 to 80 feet, 155 feet, 330 feet, 420 to 434 feet, 450 to 465 feet, and 500 to 510 feet.

The stratigraphy at Workshaft 2BK, from ground surface to the top of the competent bedrock at 155 feet, consists of refuse and soil fill material to a depth of 9 feet; underlain by interbedded sand, silt, gravel and varved clay. The overburden stratigraphy at Workshaft 2BF, from ground surface to the top of competent bedrock at 285 feet, consists of fill material to 14 feet, underlain by interbedded silt, sand, gravel and varved clay to 132 feet and till to 285 feet. The bedrock at the Workshaft 2B location consists of the lowermost unit of the Passaic Formation and is represented by interbedded, reddish-brown shale and siltstone.

At Workshaft 2C, the stratigraphy encountered in the pilot borehole, from ground surface to the top of competent bedrock at 81 feet, consisted of silty and sandy gravel fill to a depth of 7.5 feet; organic clay and clayey silt with a thin, basal layer of peat to 19 feet; medium to coarse sand to 27.5 feet; clayey silt and silty clay to 39 feet; fine to medium sand to 46.5 feet; varved silt and clay to 76 feet; and weathered rock fragments mixed with silt and sand to 81 feet. The underlying bedrock at the Workshaft 2C location is indicative of the lowermost unit of the Passaic formation which consists of interbedded, moderate-reddish-brown shale and siltstone with a few beds of sandstone and conglomerate.

2.3.1.3 Groundwater Modeling

The complexity and wide range of heterogeneities in the groundwater system along the proposed tunnel alignment makes the quantitative assessment of these potential problems difficult. Therefore, a groundwater modeling effort was designed and performed in order to simulate the range of hydrogeological conditions that might be encountered before, during, and after tunnel construction. The use of computer modeling as a predictive tool in groundwater investigations has increased over the years because large amounts of complex data can be manipulated quickly and sensitivity analyses can be performed to evaluate the reliability of prediction.

The goal of the groundwater modeling investigations was to evaluate the short-and long-term environmental impacts that could potentially arise as a result of tunnel construction, operation, and maintenance. Because the main tunnel length is relatively long (20.1 miles) and transects a variety of geological, hydrogeological, and physiographic conditions, the groundwater modeling studies for the Tunnel Groundwater Investigation were performed in five different smaller subregions. Each of the five models were intended to evaluate small areas in greater detail. Each model area has specific geologic or hydrogeologic conditions that are different from the other model areas. In this way, the five models span the full range of conditions that are anticipated along the tunnel alignment and results can be extrapolated to areas that were not modeled. Additional groundwater modeling was performed as part of the Pompton Inlet and Spur Inlet groundwater investigations.

2.3.1.3.1 Packanack Lake Model

The Packanack model area is located approximately 6,700 feet south of the Pompton Inlet of the main tunnel. See Figure E.2.2 for model area location and Figure E.2.3 for a geologic section. The local bedrock consists of Towaco Formation, the Hook Mountain Basalt, and the Boonton Formations, which are concealed by surficial glacial deposits lies primarily in the township of Wayne, in Passaic County. The model area is 10,000 feet long by 15,000 feet wide. Packanack Lake overlies the southeast quadrant of the model area and the Pompton River flows from north to south through the northwest portion of the model area. The main tunnel runs northwest-southeast, bisecting the model area into two equal halves.

The Packanack Lake model was used to simulate groundwater conditions in the Boonton Formation near the north end of the tunnel alignment. Results of the transient and steady-state tunnel simulations using the Packanack Lake model indicate that maximum drawdowns in the unfractured rock directly adjacent to the tunnel will be about 90 to 100 feet. At 1,000 feet distance, the expected drawdowns are roughly 10 to 15 feet in the unfractured rock. Little or no drawdown (i.e., less than 3 feet) is expected in fractured rock layers and none is predicted for the glacial overburden.

During the period of tunnel construction, the model predicted a maximum flow rate into the tunnel of 300 gpm, (158 gpm/mile of tunnel). Following tunnel completion and liner installation the computed seepage rate into the tunnel immediately decreased to 100 gpm (53 gpm/mile of tunnel) and remained steady.

2.3.1.3.2 Preakness Valley Model

The Preakness Valley model area is located near the confluence of the Passaic and Pompton Rivers, near Two Bridges. Bedrock units found in the Preakness Valley model area include the Boonton Formation, the Hook Mountain Basalt, and the Towaco Formation. See Fig. E.2.2 for model area location and Fig. E.2.4 for a geologic section. Surficial units in the Preakness Valley model area include continuous till, lake-bottom deposits, and deltaic and lacustrine fan deposits. Two major aquifers are present at the Preakness Valley study area, the unconsolidated overburden aquifer and the bedrock aquifer.

The model domain is 10,000 feet by 7,500 feet and includes areas within the Passaic and Morris counties. Workshaft 3 is located to the eastern boundary of the model area, and approximately 4,000 feet from the southern boundary. The tunnel alignment runs northwest-southeast along the eastern boundary of the model domain. The spur tunnel also runs east-west across the southern end of the model domain.

The Preakness Valley model was used to simulate groundwater flow conditions in the Towaco Formation, and the Preakness and Hook Mountain Basalts. Results of the steady-state tunnel simulations using the Preakness Valley model indicate that maximum drawdowns in the unfractured rock directly adjacent to the tunnel will be approximately 50 feet. Less than 9 feet of drawdown is predicted to occur in fractured rock layers directly adjacent to the tunnel, while zero drawdown is predicted in the glacial overburden. The drawdown at 1,000 feet horizontal distance from the tunnel is less than 30 feet in the unfractured rock; about 6 feet is expected in the fractured rock layers.

The computed seepage rate for the tunnel construction scenario indicates that seepage rate increases with increasing length of the tunnel and averages approximately 1,056 gpm/mile. The simulation representing tunnel operation indicates that the seepage rate into the tunnel decreased immediately after the placement of the liner and remained steady at an average rate of 121 gpm/mile.

2.3.1.3 Little Falls Model

The Little Falls model area is located southeast of the Spur Inlet where the main tunnel alignment bends twice. Three rock formations are present near the surface in the Little Falls model area, the Passaic Formation, the Orange Mountain Basalt, and the Feltville Formation. See Figure E.2.2 for model area location and Figure E.2.5 for a geologic section. Deltaic sand and gravel represent the most abundant surficial deposit in the area and are principally located along the Peckman Valley floor. Sand and gravel deposits are thin or absent along the Passaic River Valley in the northern corner of the model area. A thin reddish brown layer of till is found at the surface of the valley between the Peckman River and the Cedar Grove Reservoir. The model area is rectangular (10,000 feet x 7,500 feet) and the major axis is oriented northwest-southeast. The main tunnel alignment runs along the northeast edge of the model area.

The Little Falls model was used to simulate groundwater conditions in the Feltville Formation and the Orange Mountain Basalt. The results of this model can be used to evaluate potential impacts to well users from the Second Watchung Mountain southeast to the First Watchung Mountain and Workshaft 2 location.

The maximum drawdowns predicted for unfractured bedrock were 155 feet immediately adjacent to the tunnel alignment. However, maximum drawdowns calculated for the fractured permeable rock layers were significantly less (about 3 feet of drawdown was predicted). At 1,000 feet distance from the tunnel alignment, drawdowns predicted for unfractured and fractured layers were less than 10 feet and less than 1 foot, respectively. Zero drawdown is expected in the shallow sand and gravel aquifer.

The results of the tunnel construction simulation indicate a maximum seepage rate into the tunnel of about 2,708 gpm (1,430 gpm/mile of tunnel) when the 10,000-foot section of unlined tunnel is completed. The simulated flow rate into the tunnel declined immediately following the installation of the liner to 240 gpm (127 gpm/mile of tunnel) and remained steady.

2.3.1.3.4 Kearny Model

The Kearny model area is located at Kearny, Hudson County, starting from approximately 14,000 feet north of Kearny Point and extending north to include Workshaft 2B. Surficial materials within this model are of glacial and post glacial origin. Post-glacial surficial materials include fill and estuarine deposits and glacial deposits include till and lacustrine deposits of glacial Lake Bayonne. Bedrock is comprised of the Passaic Formation which underlies all of the Kearny model area. See Figure E.2.2 for model area location and Fig. E.2.6 for geologic section.

The model domain is rectangular, with longitudinal axis oriented parallel to the tunnel alignment. The tunnel runs along the eastern boundary of the model area. The model domain is approximately 10,000 feet long and 7,500 feet wide. The Passaic River intersects the model area at the northwest corner.

During tunnel construction, the model indicates that the maximum predicted drawdown, 138 feet, occurred in the unfractured rock zones immediately adjacent to the tunnel. The drawdown rapidly dissipated to less than 1 foot at a horizontal distance of 600 feet from the tunnel alignment. Within the fractured aquifers, the maximum drawdown at the tunnel was approximately 43 feet. The dissipation of drawdown with distance away from the tunnel was gradual. The overburden aquifer does not show any significant drawdown impacts due to tunnel construction.

The predicted seepage rate into the tunnel during construction generally increases linearly with increasing length of the tunnel and reaches a maximum of approximately 754 gpm/mile of tunnel when 10,000 feet of tunnel is completed. Following tunnel construction and liner installation the model indicates that the average seepage into a "dry" tunnel would be 96 gpm/mile of tunnel. The estimated steady-state seepage rate into a "wet" tunnel would be only 15 gpm/mile.

2.3.1.3.5 Newark Bay Model

The Newark Bay model area is located at Kearny Point, Hudson County, New Jersey. Post-glacial surficial materials located within the Newark Bay model area include fill and estuarine deposits, which are underlain in most areas by lake-bottom sediments including silt, clay, and fine sand. The Passaic Formation underlies all of Newark Bay model area. See Figure E.2.2 for model area location and Figure E.2.7 for a geologic section. The model domain begins at Newark Bay and extends north to include Workshaft 2C area. The model area is rectangular (10,000 feet x 7,500 feet), and oriented in the northwest-southeast direction. The main tunnel also runs northwest-southeast along the western boundary of the model area.

Results of the transient tunnel simulations indicated that short-term drawdown in the fractured aquifer will be less than 20 feet directly adjacent to the tunnel alignment, and less than 3 ft at 1,000 feet distance from the tunnel. The highest drawdowns were predicted for the unfractured bedrock at the tunnel. However, rapid dissipation of drawdown in the low permeability zones results in less than 12 feet of predicted drawdown at 3,000 feet distance from the tunnel alignment. No impact from the tunnel was predicted for the glacial overburden aquifer.

The model-computed seepage rate as the tunnel construction traversed the model area generally indicate that the seepage rate increases linearly with increasing length of the tunnel. The maximum seepage rate at the end of the tunnel construction is approximately 422 gpm/mile of tunnel. Following liner installation the model indicates a rapid decrease in seepage rate to 52 gpm/mile for a "dry" tunnel. The computed seepage rate for a "wet" tunnel was less than one gpm/mile.

The lowest seepage rates during construction simulations were predicted by the Packanack Lake model located toward the northern end of the tunnel alignment. The maximum seepage rate for this model was estimated to be 158 gpm/mile. Low seepage into the tunnel in this model area is explained by the presence of low permeability rocks with a lesser degree of fracturing than found in other rock formations to the south.

The long-term steady-state simulations indicate that the maximum seepage, 127 gpm/mile, will occur in the lined tunnel in the Little Falls area due to the presence of several fracture zones and a bedrock valley filled with permeable sand and gravel. In other areas, the steady-state seepage into a dry, lined tunnel will probably be less than 100 gpm/mile. At the southern end of the tunnel during normal operation, seepage rates into the "wet" tunnel sections will be almost nonexistent. See Table E.2.2 for a summary of estimated seepage rates for the model area.

2.3.2 Spur Inlet Geohydrologic Study

2.3.2.1 Introduction The purpose of the study was to determine the hydrologic characteristics of the Spur Tunnel Inlet area and to utilize a model to estimate groundwater inflows into the proposed spur tunnel. See Figure E.2.8 for a plan view of the study area. The center of the study area is located at well DC-122 immediately adjacent to the Passaic River and approximately 1500 ft. downstream from the confluence of the Passaic and Pompton Rivers. The spur tunnel would be bored entirely in the Towaco Formation of the Brunswick Group. The Towaco Formation consists primarily of Jurassic-age red, gray and black sedimentary rock. See Figure E.2.9 for a geologic cross section through the site. A summary of the results of this study is presented below. The full study is available at the Passaic River Division office.

2.3.2.2 Summary of Results

A hydrologic framework was developed based on new and existing data including well records, drillers logs, analysis of continuous core, pressure test data, geophysical logging and long term water-level monitoring. Three confined aquifers were defined as the primary water producing units in the spur inlet area. These included a glacial sand, gravel and till aquifer and two zones of water bearing fractured bedrock in the Towaco Formation of the Brunswick Group.

A 48 hour aquifer test was conducted in June of 1994 to determine aquifer and confining unit permeabilities. Results of aquifer test showed less than a foot of drawdown in 6 observation wells used for water-level measurements. Analysis of the aquifer test was complicated by the small drawdowns measured, diurnal fluctuations in water-levels and regionally declining water levels. The aquifer test data was corrected for these factors and analyzed using a 3 dimensional groundwater flow model. Best-fit simulations indicate transmissivities of 2400 ft^2/day in the glacial and upper fractured rock aquifers and 1600 ft^2/day in the lower fractured rock aquifer.

The model was used to simulate for 10 days a 1000-foot section of tunnel constructed at a rate of 100 feet per day. The model simulation indicates that a liner with a permeability of 1×10^{-7} with a thickness of 1 foot and is surrounded by 15 feet of material with a permeability of 1×10^{-5} will effectively reduce leakage of ground-water to the tunnel during construction. The estimated rate of inflow to the 1000 ft. section of lined tunnel at the end of 10 days is 2.3 gallons per minute.

2.3.3 Pompton Inlet Geohydrologic Study

2.3.3.1 Introduction The purpose of this study was to determine the hydrogeologic characteristics of the Pompton Inlet area and to utilize a model to estimate groundwater inflows into a section of tunnel. The Pompton Inlet area as shown on Figure E.2.10 is located at the confluence of the Ramapo, Pequannock, and Pompton Rivers. The center of the study area is located at well DC-147, immediately adjacent to the Ramapo River and 200 feet upstream from a concrete weir. A summary of the results of this study is presented below. The full study is available at the Passaic River Division office.

2.3.3.2 Summary of Results The Boonton Formation of the Brunswick Group is the bedrock formation underlying the inlet area. The Boonton Formation consists primarily of Jurassic sandstone, siltstone, and shale. The Hook Mountain basalt lies to the southeast of the study area and roughly parallels the eastern boundary of the study area. To the west of the inlet area the bedrock elevations quickly decrease toward the area occupied by glacial lake Passaic during the Pleistocene age. Pleistocene age unconsolidated clay-silt sand and gravel deposited primarily by stratified glacial drift and glacial lake-bed sediments overly the Boonton Formation in most of the study area (See Figure E.2.11 for geologic sections).

The study began with a Phase I development of the conceptual geologic framework and a preliminary three-dimensional ground-water flow model. This Phase II study involved aquifer testing, refinement of the conceptual geologic framework and ground-water flow model, and the use of this model to analyze the aquifer test results for a better understanding of the flow system and its hydraulic properties.

The degree of rock fracturing was found to be a poor indicator of water-transmitting properties. Aquifer testing demonstrated that sharply contrasting hydraulic properties exist within rock with similar fracture density. Discrete-zone pressure testing of the rock during drilling was found to be a better indicator of water-transmitting properties.

While there are sharp contrasts in the permeabilities of various bedrock zones in the study area, all the bedrock aquifers in the study area have very low yields. Wells DC-147 and DC-114, both with an open interval of more than 160 feet, yielded 3.0 or less gallons per minute. Any conclusions about the relative permeability or impermeability of the rock material should be taken with respect to the overall low yield of the aquifers.

The ground-water flow model was used to simulate the proposed tunnel under unlined conditions. A simulation of a 100-foot section of tunnel resulted in a ground-water inflow of 35.7 gpm after four days with no tunnel liner. With the addition of a tunnel liner with a permeability of 1.0×10^{-5} cm/s, the inflow to the 100-foot tunnel section was reduced to 20.2 gpm after four days. A reduction in tunnel liner permeability by one-half, to 5.0 x 10^{-6} cm/s, further reduced the ground-water inflow to tunnel, to 14.0 gpm after four days. The model, because of its size, is limited in the length of tunnel it can adequately represent. With simulated tunnel lengths greater than 100 feet, the drawdown reaches the model boundaries before the inflow to the tunnel stabilizes sufficiently.

2.3.4 Potential Impacts to Groundwater Users

The groundwater modeling analysis indicated that, along the tunnel alignment, there will be less than one foot of drawdown in wells open solely to overburden aquifers, both during construction and operation of the tunnel. As a result, there will be no significant impacts to overburden aquifer well users.

The southern half of the tunnel will be below sea level in elevation and will be flooded during normal operation conditions after construction is completed. Because of the flooded conditions, the groundwater seepage rate into the tunnel will be substantially less than seepage rates predicted for a "dry" portion of the tunnel. As a result of lower seepage rates after construction in "wet" tunnel areas, the drawdown in bedrock layers caused by tunnel construction will rebound after construction and will be minimal during normal operation conditions. Thus, no long-term impacts are predicted for groundwater levels, water wells, or groundwater usage in the southern one half of the tunnel.

Many bedrock wells are located within 5,000 feet of the tunnel alignment along the southern end of the proposed tunnel. These wells could experience drawdown impacts ranging from 10 to 50 feet during construction activities. Once construction is completed the tunnel will be lined and inflow will be significantly diminished. Additionally, if the tunnel is operated in a wet condition (i.e., the tunnel will remain filled with water to an elevation of 0.0 feet msl), significant long term drawdown impacts from the tunnel do not exist. The wells along the alignment would only be impacted for short periods of time during dewatering and maintenance activities.

If a well were to be significantly affected by drawdown, current estimates to hook-up to municipal water supplies is roughly $700 (Passaic Valley Water Commission, personal communication, 1995). The $700 estimate is considered an average installation cost for a 2-inch line from curb to building. Therefore, the $700 estimate ordinarily applies to single family residences and other small volume water users. Hence, small capacity water wells that are impacted temporarily by tunnel construction or operation could be mitigated inexpensively by connecting the user to a public water supply.

Distribution lines for public water supplies are common throughout the southern portion of the tunnel alignment and virtually the entire nearby population is serviced by purveyor supplied surface water. In the northern portion of the alignment, there is less urbanization, and consequently, the density of distribution lines are less. However, most of the population in the north has convenient access to nearby distribution lines. See Table E.2.3 for a listing of project area water suppliers.

2.3.5 Interaction of Tunnel Construction and Operation With HTRW Sites

In conjunction with the groundwater investigation, an investigation of hazardous, toxic, and radioactive waste (HTRW) was conducted along the tunnel alignment and near other project features, such as levees and floodwalls. This information is included in Appendix F (Hazardous, Toxic and Radioactive Waste Investigation). The investigation included collection and analysis of soil and groundwater samples at proposed workshaft and tunnel inlet locations, and evaluation of known HTRW sites in the vicinity . Various levels of groundwater contamination were identified at one workshaft location and several known HTRW sites along the tunnel alignment.

The results of the HTRW investigation are incorporated with the groundwater modeling results to evaluate the potential for mobilization of known groundwater contaminants toward the tunnel or workshafts and inlets during construction and operation.

With the exception of one location all shaft and inlet locations at which bedrock groundwater samples were collected showed minor or no contamination. Groundwater collected from the highly permeable zone at Workshaft 2B was shown to be contaminated with up to 900 ppb of chlorinated solvents in an area which is projected to intersect both the tunnel and workshaft. Engineering controls will be employed during construction and operation to mitigate potential infiltration and migration of contaminants.

Shallow groundwater contamination was identified at several existing HTRW sites along the alignment. The groundwater models indicate that there will be negligible or no drawdown in overburden units resulting from tunnel construction and operation and, as a result, the tunnel is not expected to impact the distribution or movement of these contaminants. Other sites may or may not present problems in terms of chemical contamination. Additional investigations should be performed to assess the uncertainties and risks associated with these sites.

2.3.6 Potential Impacts to Surface Structures Since less than one foot of drawdown of overburden water levels is predicted, no drawdown induced foundation settlements or other related damage to surface structures is expected. If unexpected overburden drawdown should occur, recharge wells can be utilized to restore groundwater levels and eliminate the potential for structural settlements.

2.3.7 Tunnel Seepage Control Measures

Based on the results of the geohydrologic/modeling studies summarized above, a tunnel construction procedure has been developed to limit seepage into the tunnel to acceptable levels both during and after construction. Based on the model study results, long term steady state seepage into the grouted-lined tunnel, filled with water to El 0.0, is estimated to be on the order of 1,000 to 2,000 gpm.

Reduction of seepage inflows into the tunnel will be accomplished by cement grouting and concrete liner placement. The cement grouting requirements for the tunnel used for construction cost estimating purposes are shown on Table E.2.1. Grouting ahead of the tunnel boring machine (TBM) will be performed in the most pervious rock zones. A series of probe holes will be drilled radially and ahead of the TBM. If the seepage from these probe holes exceeds a specified amount, the TBM will be stopped and grouting will be performed ahead of the TBM. After placement of the concrete tunnel liner, contact grouting will be performed to fill any voids between the liner and rock. Consolidation grouting refers to drilling holes through the liner and into rock and grouting pervious rock zones.

The grouting procedures were developed based on input from our tunnel consultants and discussions with design and construction personnel working on the Milwaukee sewer (MMSD) tunnels. Grouting effectiveness, cost and production data were obtained from the Milwaukee project.

2.3.8 Tunnel Shaft Seepage Control In order to excavate the tunnel shafts through the overburden soils, either slurry/concrete walls or freeze walls will be utilized for structural support and seepage control. For the portion of the tunnel excavated through rock, cement grouting will be utilized to control seepage. As a result of these controls, no significant drawdown of groundwater levels around shaft excavations is expected.

2.4 GROUNDWATER STUDY - OTHER PROJECT FEATURES

2.4.1 Great Piece Meadows

A weir will be constructed on the Passaic River upstream of the Spur Tunnel Inlet to control water levels in Great Piece Meadows. During tunnel operation, the weir would maintain a 2 year flood level in Great Piece Meadows. Some concern has been expressed over the loss of aquifer recharge in Great Piece Meadow during tunnel operation due to the reduction in flood water depth over the meadow. We have, therefore, performed an evaluation to determine if any significant aquifer recharge occur from floodwaters.

See Figure E.2.12 for a Great Piece Meadow location map and Figure E.2.13 for a geologic section. Our recharge analysis indicates that due to the thick layer of glacial clay overlying the aquifer (Figure E.2.13) and the relatively short flood duration, no significant aquifer recharge occurs during flood events. A volume of water equal to only about 0.004 % of the total aquifer volume would potentially recharge the aquifer during a 100 year flood. In fact, our deep observation wells located in the eastern portion of Great Piece Meadow indicate that the piezometric level in the bedrock is higher than in the shallow overburden aquifer. This observation would tend to support the conclusion that the Great Piece Meadows is a discharge area for the bedrock aquifer, not a recharge area. It has been suggested that the recharge of the overburden and bedrock aquifers in the Central Basin is from runoff and infiltration from higher elevations surrounding the basin.

2.4.2 Pequannock Weir and Channel Work

2.4.2.1 A new Pequannock River Weir will be constructed upstream of the Pompton Inlet to maintain river levels during low flows at the same level as the existing weir. During flood periods, the weir gates will open to allow additional flow into the inlet. In addition to the weir, channel work will be performed on the Pequannock, Wanaque, and Ramapo Rivers to improve conveyance and thus lower water levels during flood periods.

2.4.2.2 The weir and channel work described above will have no significant effect on groundwater resources. First, the thick glacial lake deposits underlying the alluvial sands prevent significant recharge from the river into the deep overburden or bedrock aquifer. Refer to the discussion above on Great Piece Meadow concerning aquifer recharge. Secondly, the observation wells installed at the Pompton Inlet indicate that this area is a discharge area for the bedrock aquifer. Piezometer levels in the bedrock aquifer are several feet higher than in the recent alluvial sands and gravels (See USGS geohydrologic report for the Pompton Inlet).

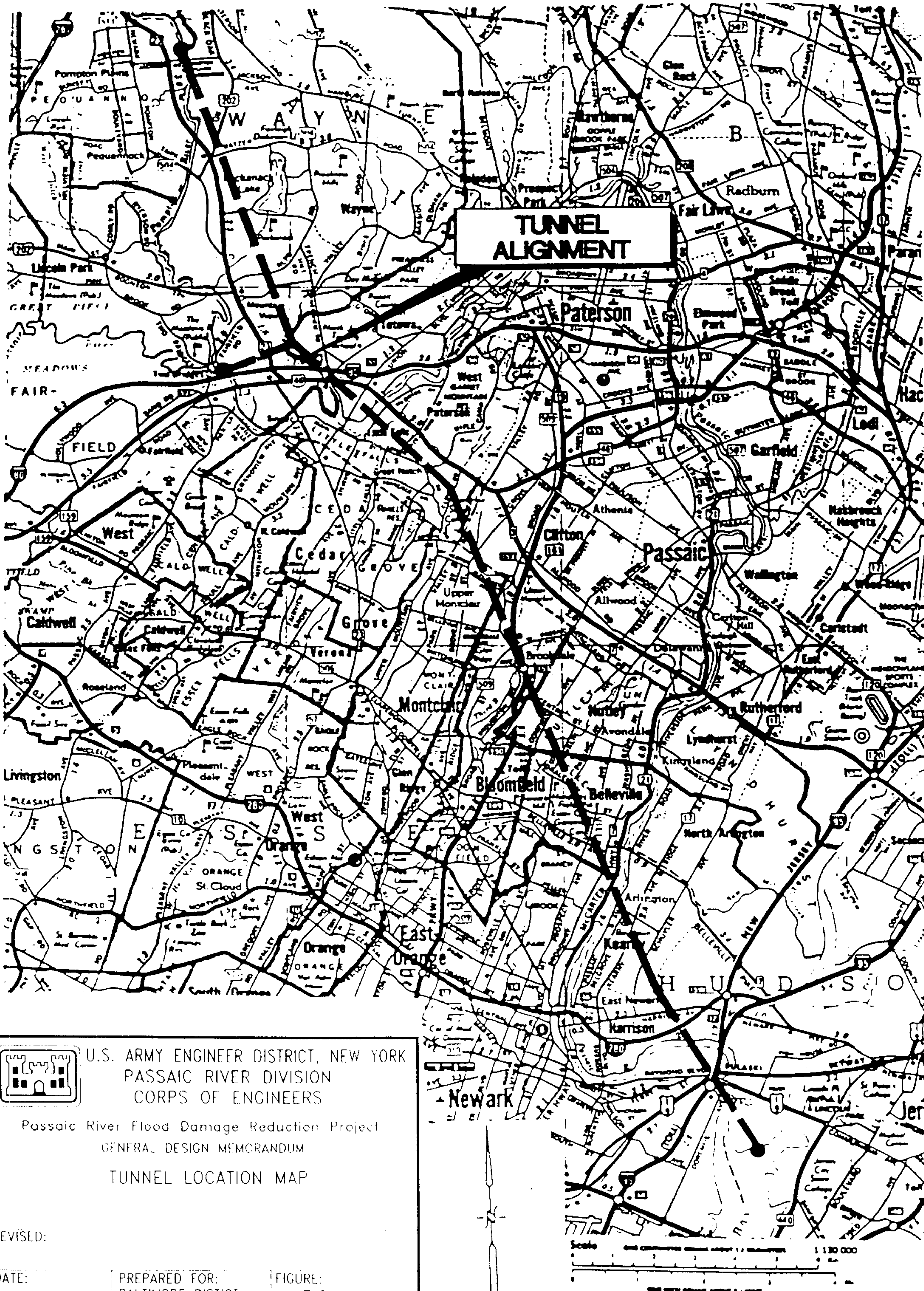
2.4.2.3 The proposed channelization will lower river levels about 2 to 5 ft. during flood periods and about 1 to 4 ft during normal flows. This slight lowering of the river levels will have no significant impact on recharge of or storage in the shallow alluvial sand and gravel aquifer. In fact, there are very few wells located in this shallow recent alluvial aquifer.

2.4.3 Levees/Floodwalls Approximately 20 miles of levees and floodwalls will be constructed as part of the project. These features will prevent inundation of floodplain areas during high water periods. As discussed above, recharge of aquifers during flood periods is insignificant for the Central basin, Pompton, and Hurricane Levee areas.

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 2
GROUNDWATER

FIGURES



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CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

TUNNEL LOCATION MAP

REVISED:

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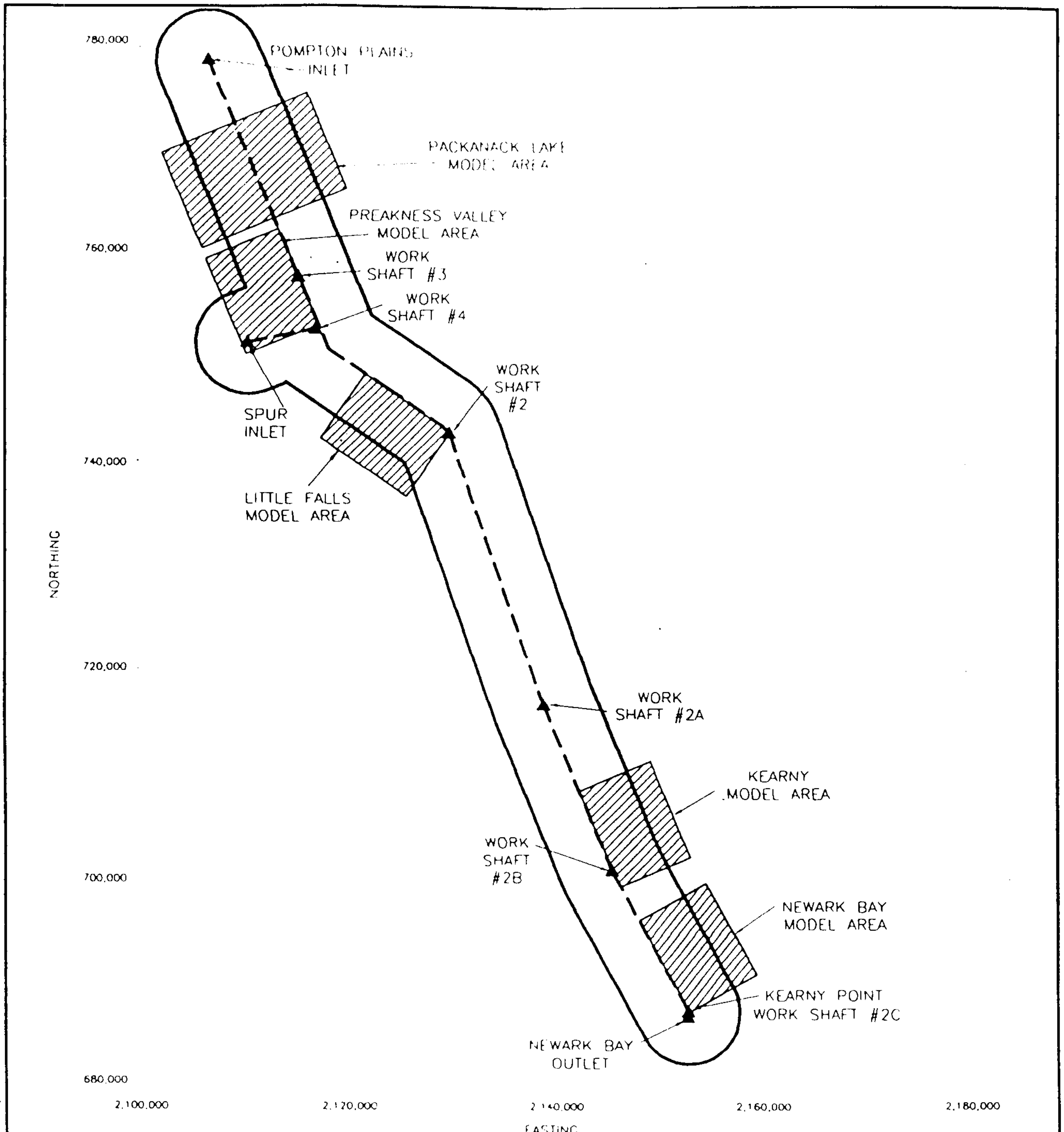

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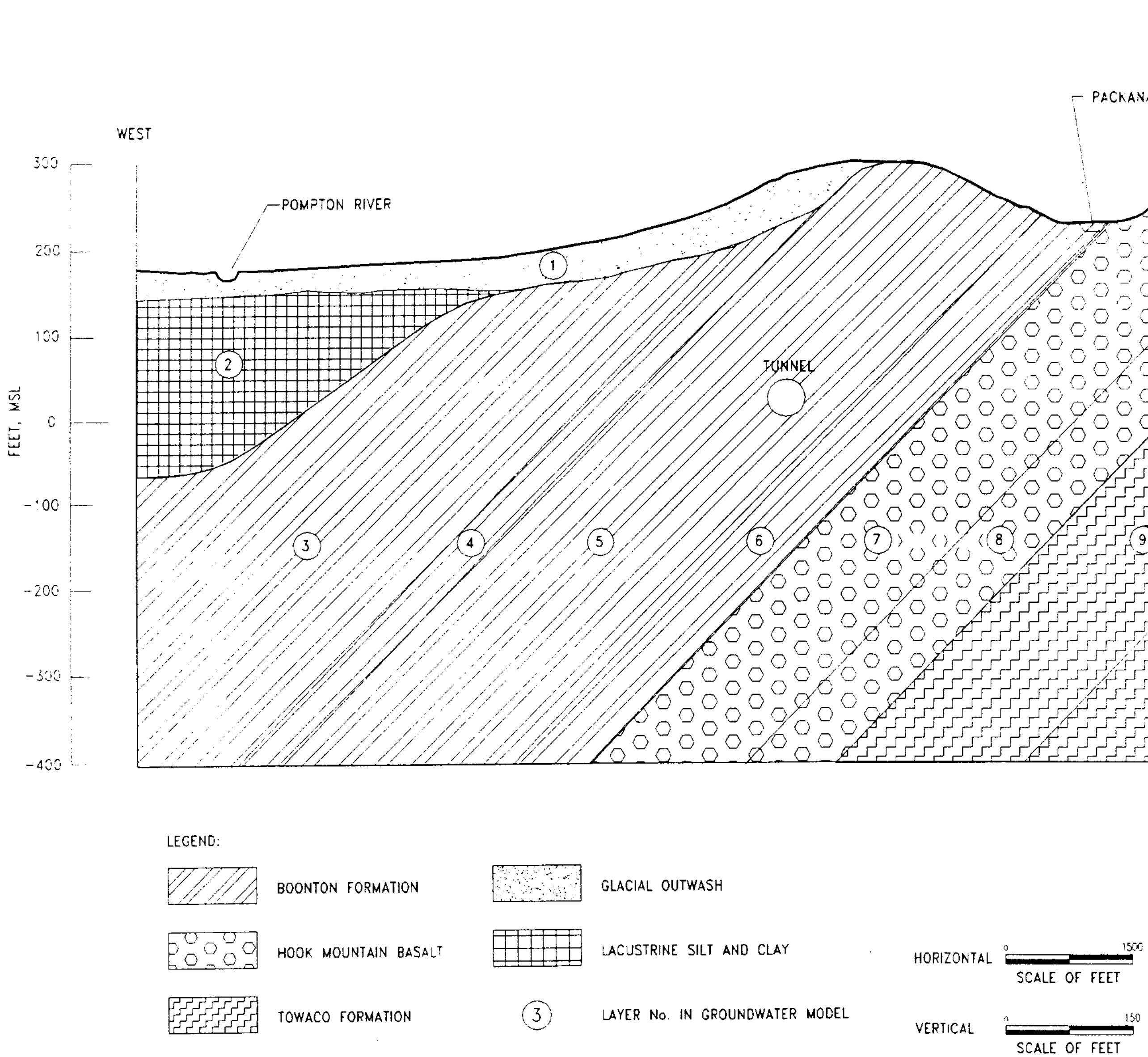
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

MODEL AREAS AND
TUNNEL ALIGNMENT

REVISED:

DATE:	PREPARED FOR: BALTIMORE DISTRICT	FIGURE: E.2.2



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGY AND MODEL CROSS SECTION,
PACKANACK LAKE

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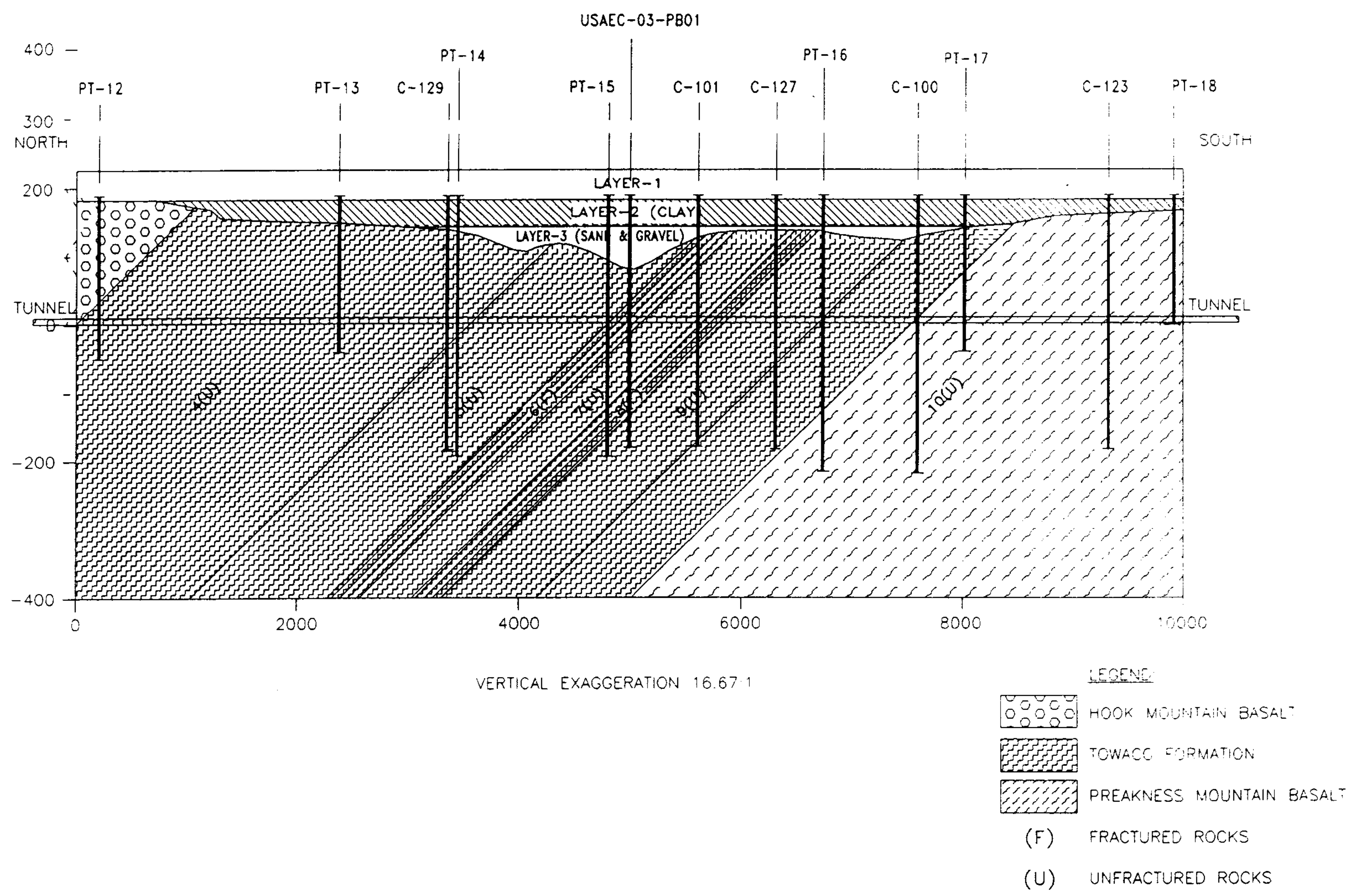

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Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

GEOLOGY AND MODEL CROSS SECTION, PREKNESS VALLEY

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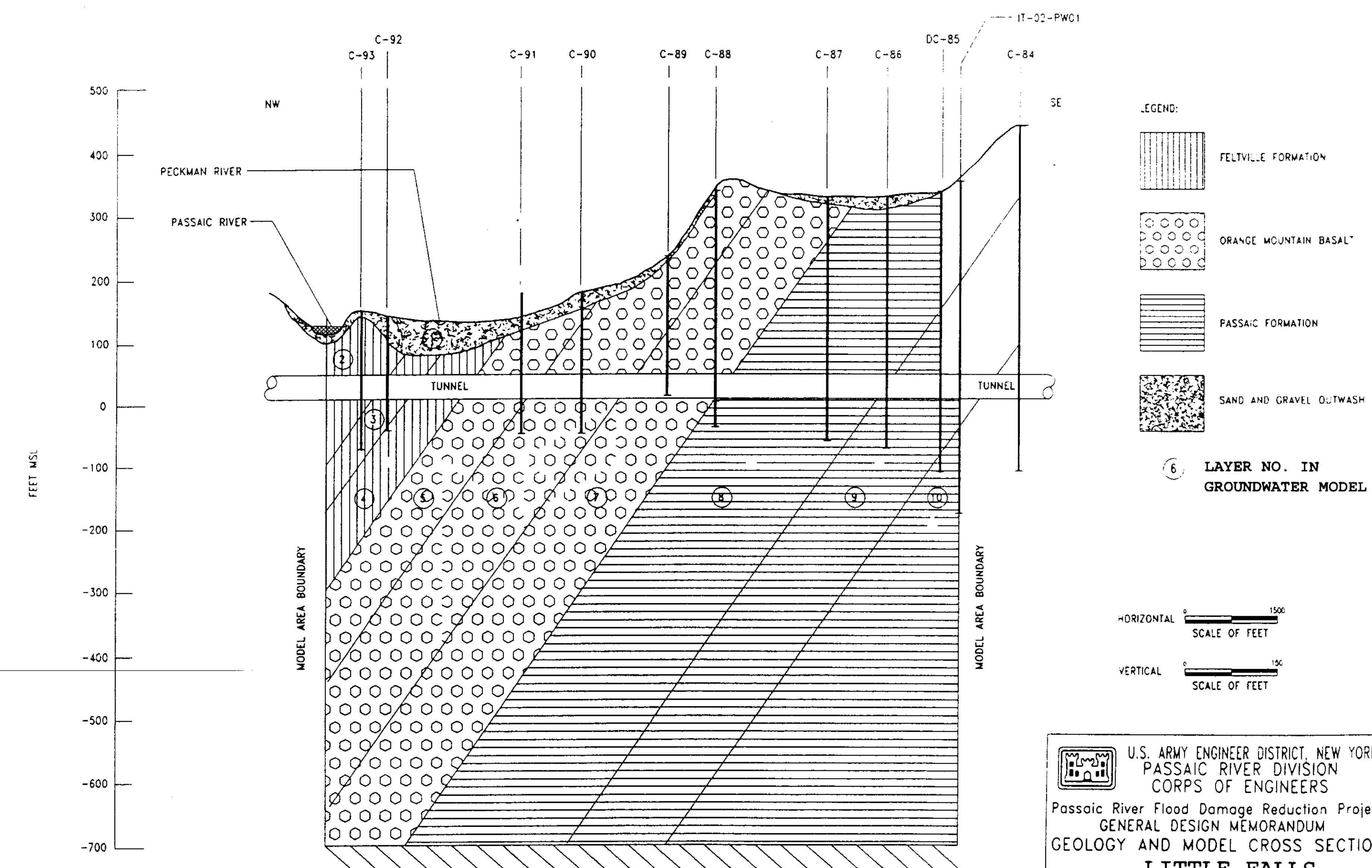

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Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

GEOLOGY AND MODEL CROSS SECTION

LITTLE FALLS

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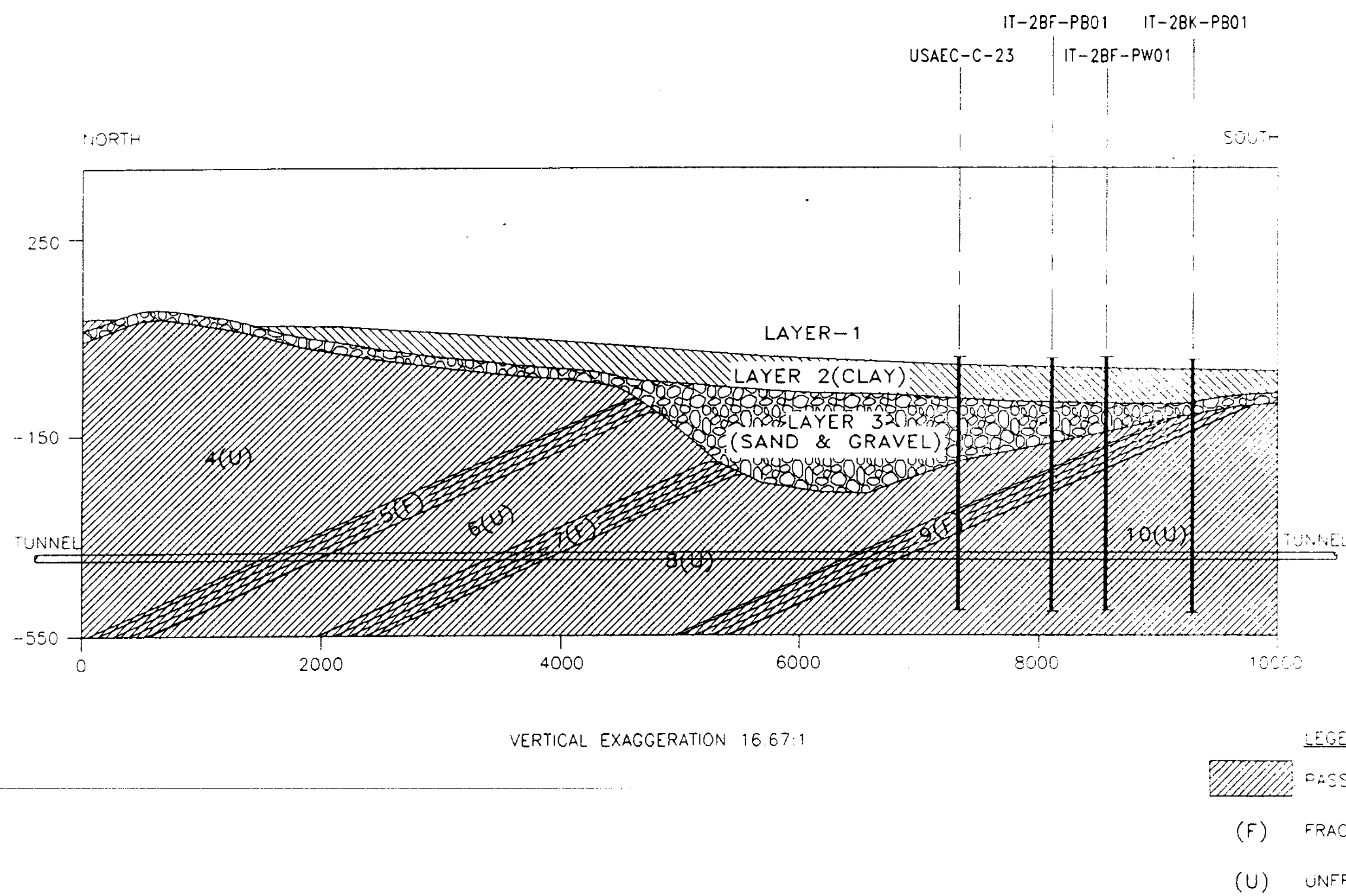

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Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
GEOLOGY AND MODEL CROSS SECTION
KEARNY

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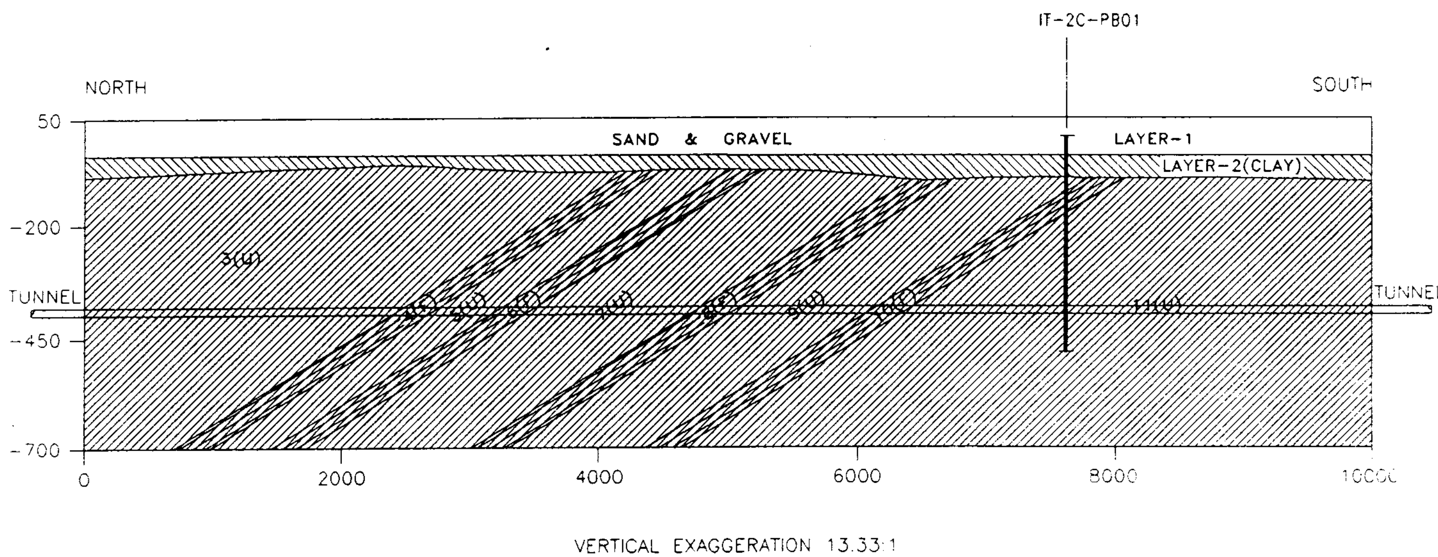


LEGEND

PASSAIC FORMATION

(F) FRACTURED ROCKS

(U) UNFRACTURED ROCKS

U.S ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

GEOLOGY AND MODEL CROSS SECTION, NEWARK BAY

DATE:	PREPARED FOR: BALTIMORE DISTRICT	FIGURE: E.2.7

/dwgfiles/netdwgs/dwgs/529740/59740a94 02/09/95 8:54pm graphics

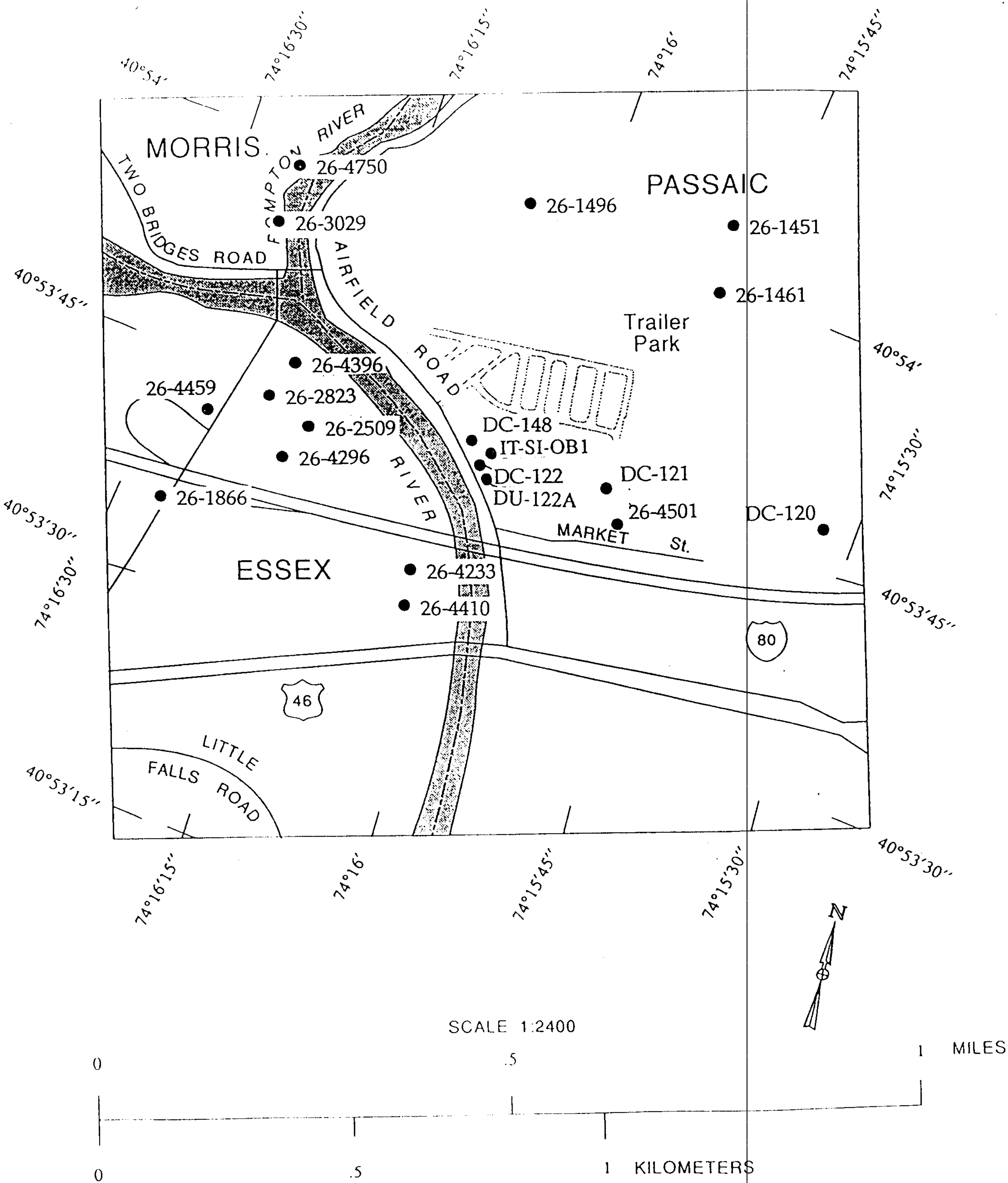


Figure E.2.8 Map showing the location of selected wells in the study area.

Geologic Cross-Section along Spur Tunnel Alignment

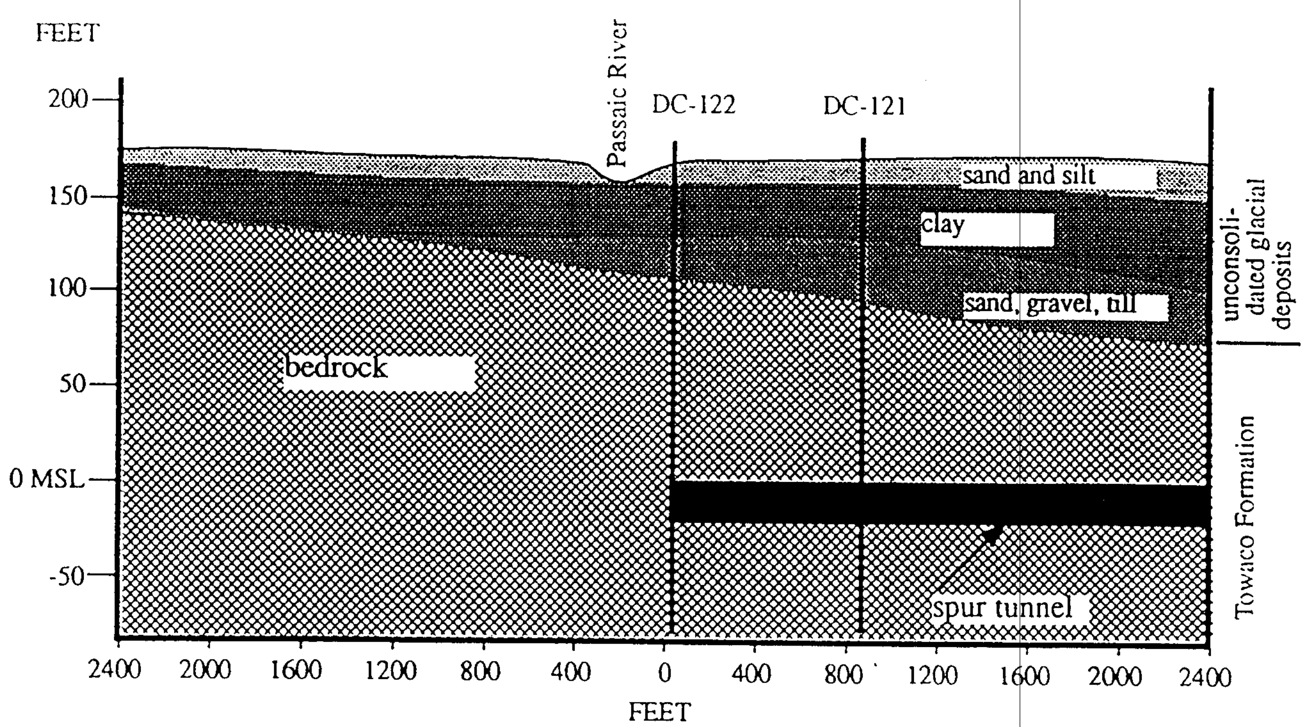


Figure E.2.9 - - Generalized geologic cross-section along spur tunnel alignment.

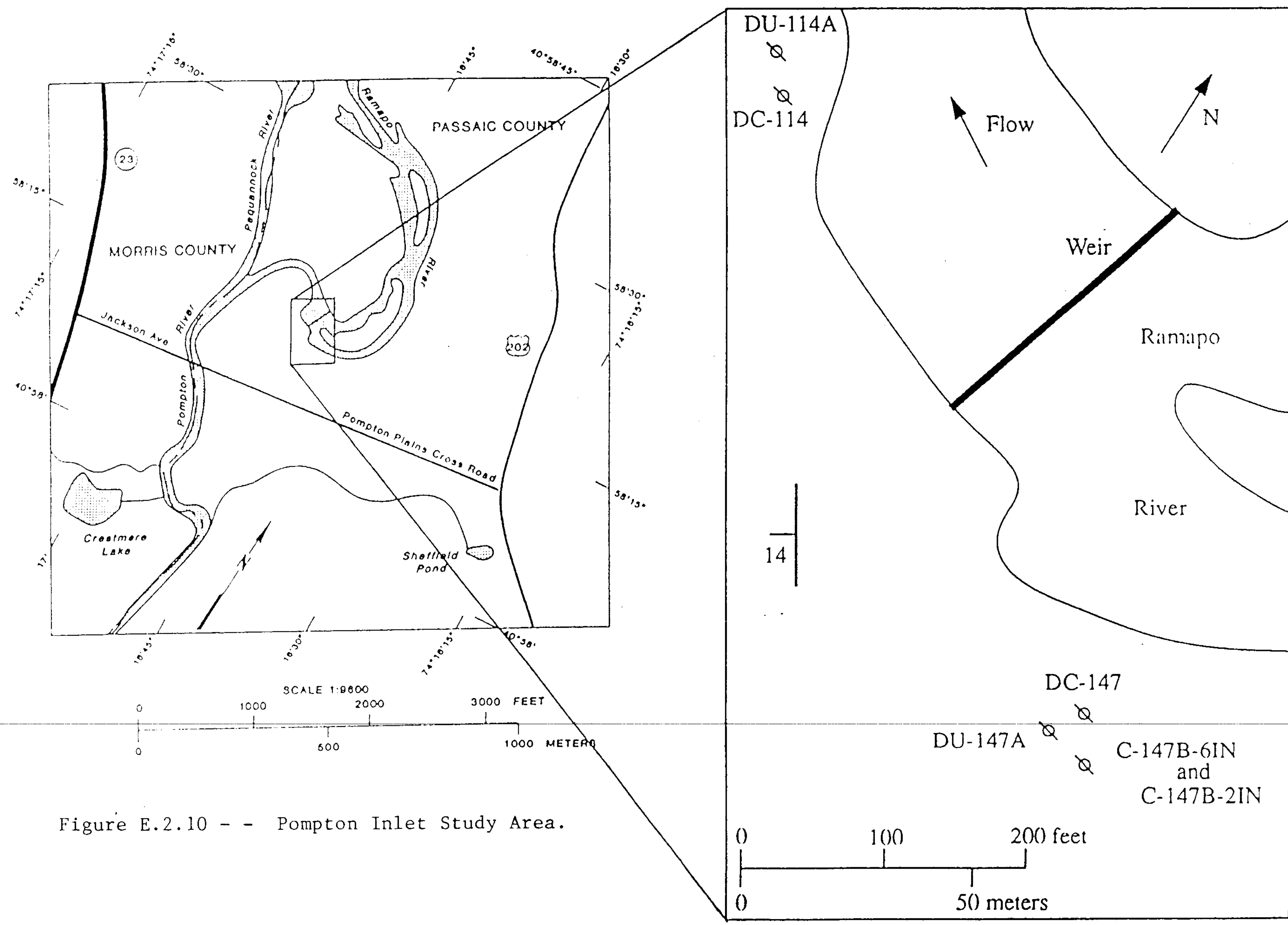
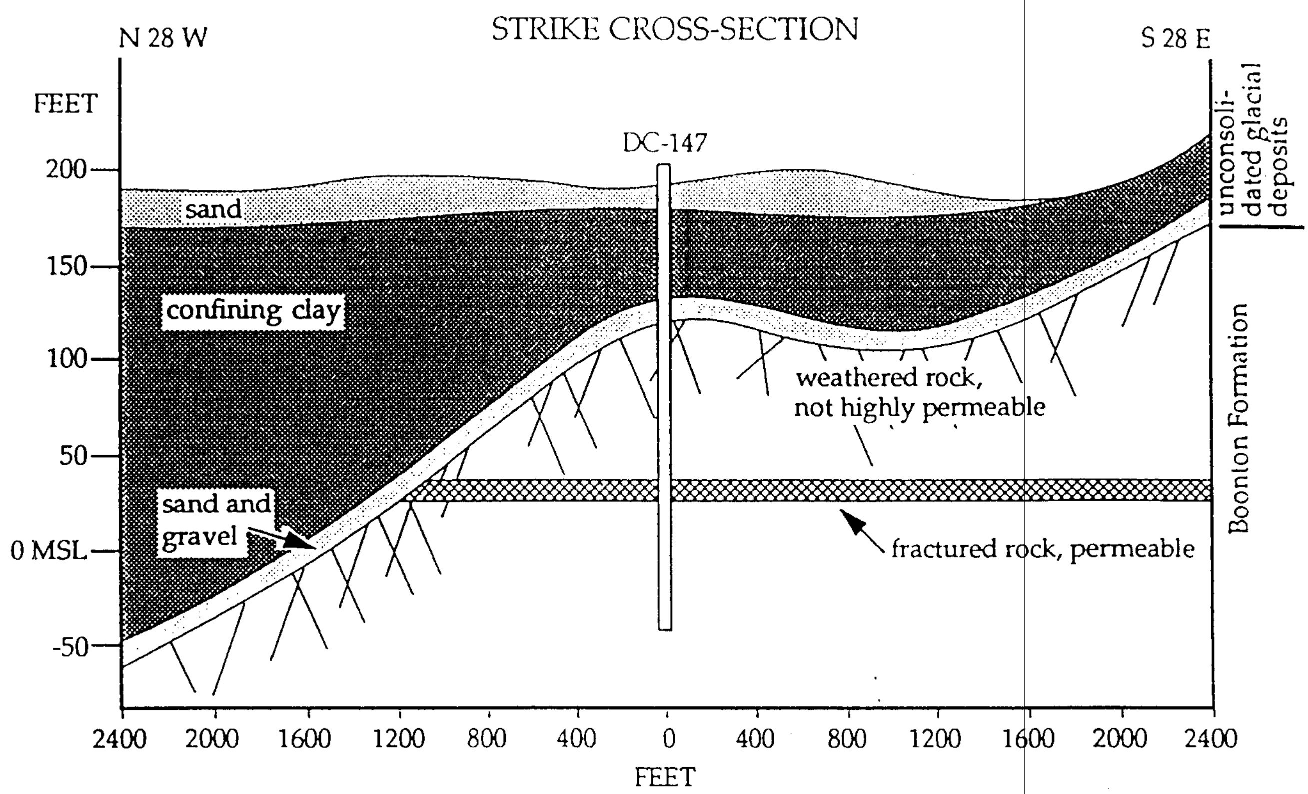
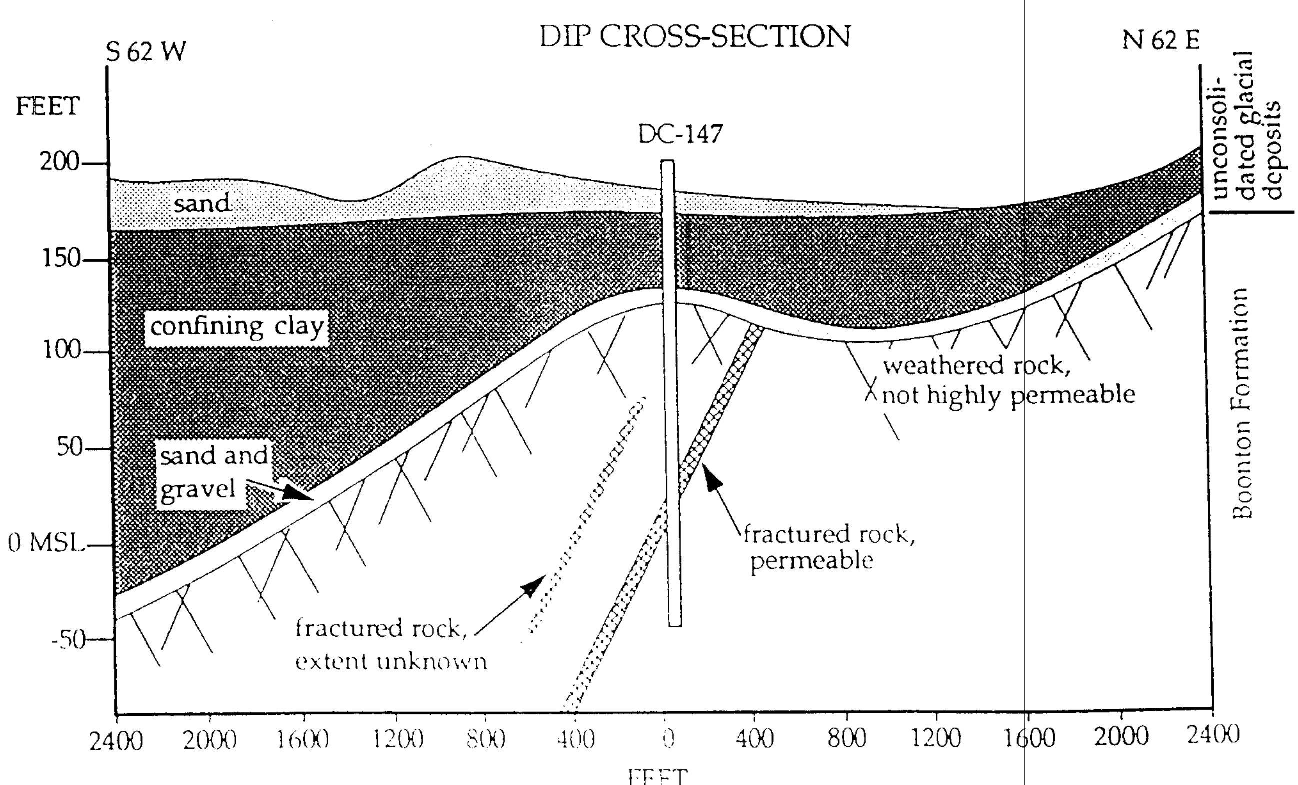


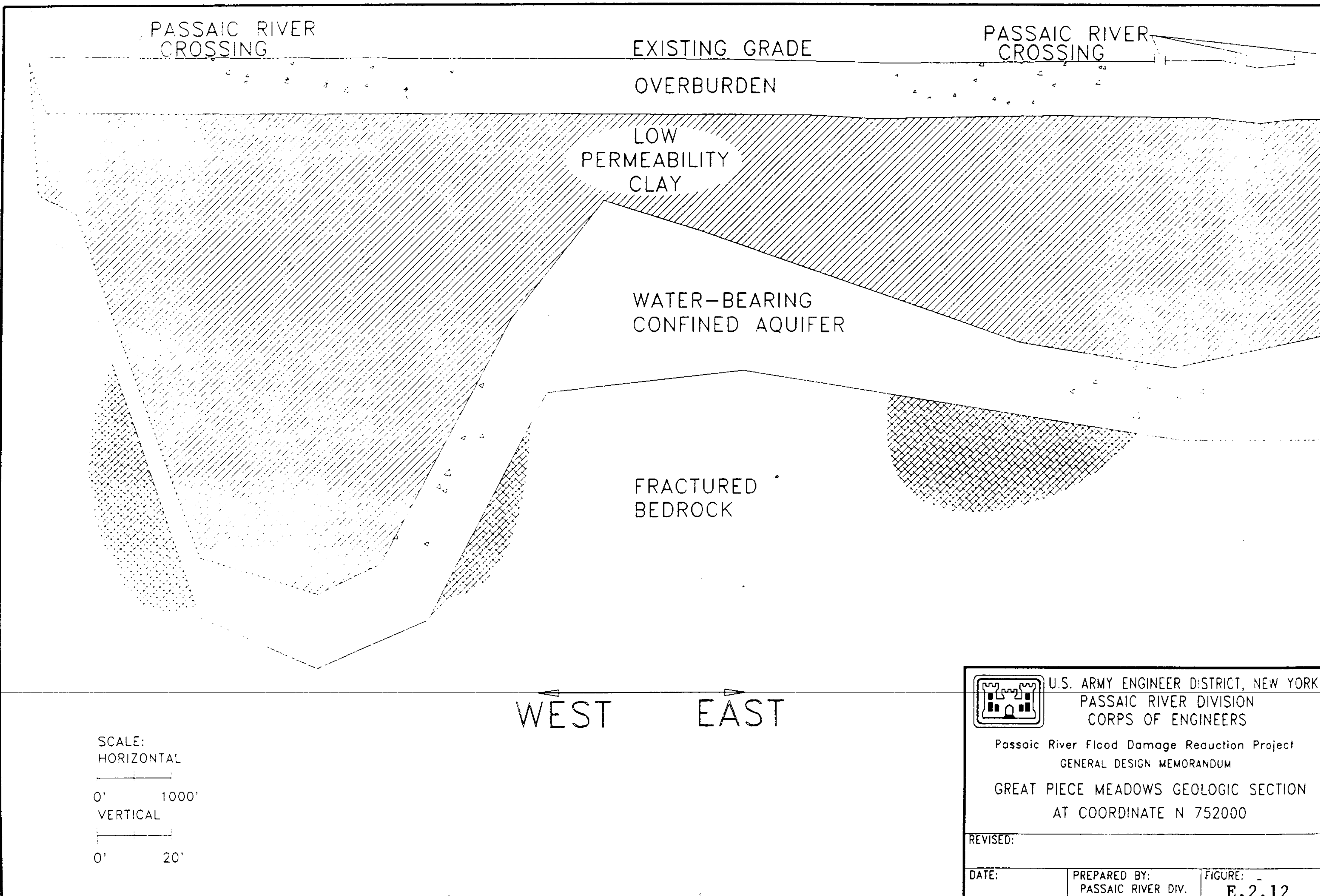
Figure E.2.10 - - Pompton Inlet Study Area.





NOT TO SCALE

Figure E.2.11 - - Pompton Inlet Geologic Sections



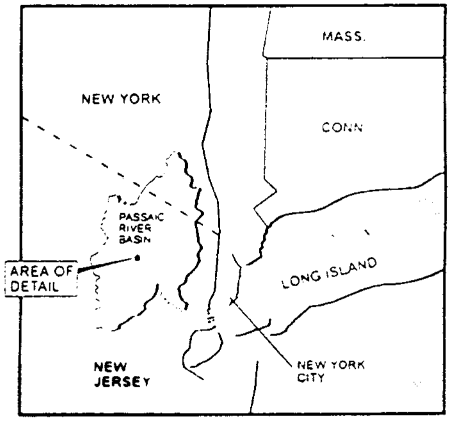
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

GREAT PIECE MEADOWS GEOLOGIC SECTION
AT COORDINATE N 752000

REVISED:		
DATE:	PREPARED BY: PASSAIC RIVER DIV.	FIGURE: E.2.12

GPM-PRO.dwg



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

GREAT PIECE MEADOWS
LOCATION MAP

REVISED:		
DATE:	PREPARED BY: PASSAIC RIVER DIV.	FIGURE: E.2.13

GPM-MAP.dwg

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 2
GROUNDWATER

TABLES

GROUTING REQUIREMENTS*

Lower Tunnel (48,398 LF)

a. Grouting ahead of face - rock with k greater 1×10^{-3} cm/sec = 9% of tunnel = 4,400 LF.

b. Consolidation grouting of rock outside concrete lining - rock with k greater than 1×10^{-4} cm/sec = 40% of tunnel = 19,400 LF.

c. Contact grouting behind lining - everything not consolidation grouted = 60% of tunnel. = 29,000 LF.

Upper Tunnel (59,349 LF)

a. Grouting ahead of face rock with k greater than 1×10^{-3} cm/sec = 4% of tunnel = 2,400 LF.

b. Consolidation grouting of rock outside concrete lining - rock with k greater than 1×10^{-4} cm/sec = 31% of tunnel = 18,400 LF.

c. Contact grouting behind lining - everything not consolidation grouted = 69% of tunnel = 40,950 LF.

* The grouting requirements were developed based on histograms of borehole packer test permeability data.

TABLE E.2.1

TABLE E.2.2

Summary of Predicted Tunnel Seepage Rates for Model Areas
(in gpm / mile of tunnel)

	Packanack Lake Model	Preaknesss Valley Model	Little Falls Model	Kearney Model	Newark Bay Model
Peak Flow into Unlined Tunnel during Construction	158	1056	1430	754	413
Steady state seepage into dry, lined tunnel after construction	53	121	127	96	52
Steady state seepage into wet, lined tunnel after construction				15	1
Seepage into lined tunnel, during dewatering and maintenance				180	133

TABLE E.2.3

Population, Water Suppliers, and Residential Wells Along Tunnel Alignment and Buffer Passaic River Flood Protection Project

	Population	Primary Water Company	Secondary Water Company	Percent of Population Hooked-up	Population Served by Residential Wells[a]
Morris County					
Pequannock	14,000	Municipal (wells)	City of Newark	100	none
Lincoln Park	10,720	PVWC	none	95	536
Riverdale	1,200	Municipal (wells)	none	100	none
Essex County					
North Caldwell	12,000	Jersey City	PVWC, Essex Fells	99	120
Cedar Grove	12,600	NJDWC	PVWC	100	none
Montclair	38,000	NJDWC	Municipal (wells)	95	1,900
Glen Ridge	7,600	NJDWC (via Montclair)	Municipal (via Montclair)	100	none
Nutley	8,000	PVWC 75% Newark 25%	none	100	none
Bloomfield	45,061	City of Newark	none	100	none
Belleville	34,213	City of Newark	none	100	none
Newark	275,000	City of Newark	none	100	none
Passaic County					
Wayne	52,000	NJDWC	Municipal (wells)	98	1,040
Totowa	11,000	PVWC	none	100	none
West Paterson	10,982	PVWC	none	100	none
Little Falls	12,000	Essex Fells (wells)	NJAWC	99.8	24
Clifton	70,000	PVWC	none	100	none
Pompton Lakes	10,539	Municipal (wells)	none	100	none
Bergen County					
Lyndhurst	18,300	Jersey City	none	100	none
North Arlington	13,790	PVWC	none	100	none
Hudson County					
Kearny	34,700	NJDWC	none	100	none
East Newark	2,000	NJDWC (via Kearny)	none	100	none
Harrison	13,425	PVWC	none	100	none
Jersey City	228,537	Jersey City	none	100	none

Note:
There are 935,667 people in the study area.
pulation data are from 1990 census.

PVWC - Passaic Valley Water Commission
NJDWC - New Jersey District Water Commission
NJAWC - New Jersey American Water Company

a - Population served by residential wells is estimated from the percent population not hooked up to the municipal water supply for each community

APPENDIX E

SECTION 3

TUNNELS

PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

GEOTECHNICAL DESIGN APPENDIX E

SECTION 3 TUNNELS

FIGURES

PLATES

ATTACHMENTS

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 3

TUNNELS

3.1 SCOPE

This portion of the report outlines the studies and results of the current level of geotechnical design on the Main and Spur Tunnels. Supporting information from other disciplines may be found in the Main Report and in Appendixes A through D, and F through I. The amount of data gathered in the geotechnical field to date is too voluminous to include in this report but is available in the Passaic River Division Office of the New York District (PRD). This report is intended to summarize the most important information gained through the field investigations and present the methods of analysis which were used and the results obtained. Computation sheets are provided as examples only or presented in sub appendices. The complete set of computations is also available at the PRD.

3.2 FEATURE DESCRIPTION

3.2.1 Main Tunnel

The Main Tunnel runs from the inlet at Pompton Lake to the outlet at Newark Bay, a horizontal distance of 107,747 feet from the center line of the Pompton inlet to the center line of the outlet. The tunnel is circular with an inside diameter of 42 feet. In order to accommodate a 15 inch thick concrete liner the minimum excavated dimension of the tunnel will be 44½ feet. Details of the proposed alignment are presented in Attachment E.3.1.

The alignment for the tunnel was defined in concept in the Water Resources Development Act of 1990 and detailed in a study by the New York District titled "Tunnel Alignment Selection" finalized in late 1991. Several factors influenced the current location of the Main Tunnel. The availability of work shaft locations and their proximity to roads or railroads suited to transportation of the tunnel muck was critical in this highly urbanized area. Minimizing the length of tunnel which had to be driven through the hardest rock was also an important consideration in minimizing construction costs. Hydraulic considerations dictated that no curve in the tunnel be constructed with a radius of less that 500 feet measured at the inside of the bend. Maneuverability of the tunnel boring machine (TBM) favored having no curve with a radius of less than 1,500 feet. The invert of the Main Tunnel varies between elevation 9 at the inlet to -408 at Work Shaft 2C near the outlet. This variation stems from hydraulic, geotechnical, and operational considerations. The need to avoid deep, glacially generated buried valleys in the lower portion of the tunnel forced the lowering of the invert to elevation -408. A practical rule of thumb is to keep a minimum of one tunnel diameter of sound rock above the crown of the tunnel. To facilitate dewatering of the tunnel the invert climbs in either direction from a low point at the dewatering pump station location at Work Shaft 2C. The degree of slope to accommodate the elevation changes were the result of mathematical hydraulic modeling. Future refinements to the proposed location of the Main Tunnel should be relatively minor provided the work shaft locations are available in the future and the current outlet in Newark Bay is not altered.

Four separate contracts are required for construction of the Main Tunnel under the currently proposed plan. Contract A goes from tunnel station 0+00 at the outlet to 161+15 at the end of a drill and blast section connecting to Work Shaft 2B where the TBM will be removed and transported to the Pompton Inlet. It is estimated that 2,257 feet of tunnel, from Work Shaft 2C to the Outlet, will be excavated by drill and blast methods using multiple drifts. The remainder of the excavation in contract A will be by TBM. Contract B goes from station 161+15 at Work Shaft 2B to 484+73 at the end of a drill and blast section connecting to the "hook hole", Work Shaft 5. Drill and blast sections will be excavated from work shafts on either end to facilitate start up with the TBMs and their disassembly. Total drill and blast footage is estimated at 550 feet. The remaining 31,810.4 feet will be excavated using a TBM. Contract C extends from station 484+73 at Work Shaft 5 to 842+97 at the end of a drill and blast section connecting to Work Shaft 3. A drill and blast section 654 feet long is proposed in the middle of this contract at Work Shaft 2. This drill and blast section is to be advanced through the faulted zone by multiple drift methods and to provide a starter tunnel for the TBM. The TBM is to complete the drive from Work Shaft 2 to Work Shaft 5, be partially disassembled at 5, and returned to shaft 2, where it will be turned around to make the drive toward Work Shaft 3. In this way the remaining 35,170 feet of tunnel in this contract will be excavated by TBM. The final contract for the Main Tunnel, D, goes from station 842+97, at Work Shaft 3 to 1077+47.00, at the center line of the Pompton Inlet. It has drill and blast sections on either end totaling 563 feet. Much of this at the inlet end to excavate the 52' diameter portion of the tunnel required by hydraulic considerations and to provide a short starter section for the TBM. The remaining 22,887 feet in contract D will be excavated by TBM southward from the inlet shaft to Workshaft 3. For this construction sequence it will be necessary to have three TBMs for the Main Tunnel.

3.2.2 Spur Tunnel The Spur Tunnel connects an inlet on the Passaic River near State Route 46 and Fairfield Road to the Main Tunnel at station 785+15.6 a total horizontal distance of 7,015 feet. The inside diameter of the Spur is to be 23 feet. It is to have a 15 inch thick concrete liner so the minimum excavated diameter will be 25½ feet. The proposed alignment, described in Attachment E.3.1, is roughly the shortest straight line distance between the Spur Inlet and the Main Tunnel that will still accommodate the construction of Work Shaft 4. A curved transition section having a minimum radius of 250 feet will redirect the straight portion of the Spur to intercept the Main Tunnel at an acute angle for hydraulic efficiency. The invert of the Spur Tunnel is straight and slopes at 0.0015 ft/ft to connect with the invert elevation of the Main Tunnel at -10.53 feet MSL. It is anticipated that the Spur will be constructed under a separate contract with a drill and blast sections through the curved section for 785 feet, and, for 360 feet at the inlet to produce a 30 foot diameter intake section. For the purposes of estimating cost the remaining 5,870 feet is to excavated by TBM from Workshaft 4 to the inlet.

3.3 SUBSURFACE INVESTIGATIONS

3.3.1 Earlier Studies

3.3.1.1 Exploration

The Corps of Engineers first began studying solutions to the areas' chronic flooding problem after the devastating flood of 1936. Support for the project quickly waned until the next major flood in 1984 which catalyzed local support and led to the development of the feasibility study out of which the current study evolved. A complete discussion of the exploration for feasibility is covered in Parts II and III-Addenda to the Feasibility Report. The exploration conducted during the Feasibility study was performed along the then proposed tunnel alignment from the Pompton Inlet to an Outlet on the Passaic River at Third River. The Spur Tunnel Location was roughly the same as the currently proposed alignment. Since the alignment for the lower portion of the tunnel changed to an outlet in Newark Bay three of the feasibility level holes are no longer applicable. Holes in this program were designated PT- or PTI-and were completed in 1985. These holes are included in the current design and are summarized in paragraph 3.3.1. below.

Water pressure testing was performed on some of these holes and all were backfilled with cement grout upon completion. These locations were not tied in with surveying until recently.

3.3.1.2 Geophysical Exploration During the Feasibility Phase the U.S. Geological Survey conducted subsurface seismic profiling along the tunnel alignments near the spur to define the top of rock elevations in this area. The results of this survey are presented in US Geological Survey Water-Resources Investigations Report 88-4061. Downhole television camera surveys were run on many holes to acquire orientation data on discontinuities.

3.3.1.3 Geologic Mapping Reconnaissance field mapping was conducted on outcroppings, quarries, and road cuts. This work was aimed at gathering information on the orientation of discontinuities and structural geology. Glaciation and cultural development has limited the number of outcroppings in the sedimentary rocks. Much of the surface exposures are in the basalt because it is more resistant and because it is frequently quarried.

3.3.2 Current Study

3.3.2.1 Exploration The exploration program was primarily laid out to provide information on the tunnels. Some of the borings located to serve as exploration for work shafts, inlets and the outlet as well as the tunnel. The drilling was begun in late 1989 and completed in late 1994. It consisted primarily of unsampled drilling through overburden although standard penetration testing and 3" Shelby tube sampling were done in some holes. Rock was sampled using 2½ inch diameter HQ coring. Hole designations generally follow the format D - drive sampled, U - Shelby Tube sampled, and C - cored. In the original layout they were numbered consecutively from the Outlet to the Pompton Inlet. Subsequent, supplementary holes are out of this numbering sequence. For the geotechnical exploration on the Main Tunnel, including the feasibility level holes, a total of 119 borings have been drilled. These borings totaled 4,593.2 feet of unsampled overburden drilling, 1,349 feet of intermittent standard penetration test drilling, 49.2 feet of Shelby tube sample, and 31,828.4 feet of rock core the vast majority of which is HQ (2.5 inch diameter). For the Spur Tunnel 10 borings were made totaling 397 feet of unsampled overburden drilling, 292.9 feet of intermittent standard penetration test drilling, 57 feet of Shelby tube sample, and 1,836.3 feet of rock core. A table summarizing the basic data on this exploration and plan of exploration sheets showing the borings in relation to the tunnel alignments are included in Attachment E.3.2. The exploration drilled by the IT Corporation for the ground water study is summarized in Section 2 of this appendix. The rock cores are currently stored in boxes at a specially prepared warehouse facility at the Military Ocean Terminal at Bayonne, New Jersey.

3.3.2.2 <u>Geophysical Exploration</u>

3.3.2.2.1 <u>Downhole Television Camera</u> Because of the limited number of surface outcroppings available it was decided to log as many of the holes as possible with a down hole television camera equipped with an orientation head designed and assembled by the Corp's Southwestern Division Laboratory. The camera and supporting equipment was manufactured by Reese Equipment Co. Videotapes of the holes were analyzed in the Corp's Southwestern Division Lab to measure the orientation and character of any breaks in the rock mass. Individual reports on each of these logs were prepared and submitted. The information from the video tape analysis was used in the discontinuity study described in paragraph 3.4.3.2.3.

3.3.2.2.2 <u>Downhole Geophysics</u> Limited down hole geophysical logging was performed during this phase of exploration. Natural Gamma, Caliper, spontaneous potential, and resistivity logs were run on the holes in Newark Bay and stratigraphic imaging was performed. The USGS performed caliper, natural gamma, fluid resistivity, and fluid temperature logging on five boreholes in conjunction with their geohydrologic studies at the Pompton and Spur Inlets. IT Corporation performed natural gamma, spontaneous potential, multipoint resistivity, caliper, temperature, and delta temperature geophysical logging for their tunnel groundwater study.

3.3.2.3 <u>Geologic Mapping</u> Very limited surface mapping was conducted to supplement the work done during the feasibility stage. Joint scan lines were measured at one of the quarries in the Montclaire area. In addition discontinuity orientation and lithologic data were obtained from 9 locations scattered around the project area.

3.3.3 <u>Suggested Future Subsurface Investigations</u>

3.3.3.1 <u>Exploration</u> Subsurface exploration should be expanded in the next phase of design especially in the lower part of the tunnel. Large gaps in the coverage still exist. Several angled borings should be drilled in the areas where faulting is inferred or where the correlation of rock units from hole to hole is questionable. Information on the area where the Spur Tunnel joins the Main Tunnel is not well developed. All of the future exploration that extends into the rock should be conducted to provide as much additional information as possible on the presence of water bearing zones. Grouting of these zones during construction has a major effect on cost so defining them during design is very important. Defining the depth and extent of buried valleys along the tunnel route is a critical goal for future exploration. Ensuring sufficient rock cover over the tunnel is vital. Additional information will be critical at this location. A buried valley which is very close to the Spur Tunnel crown is indicated by boring C-118. Additional borings are needed to define this situation and to determine if the currently proposed Spur alignment is parallel to a fault zone. The deep buried valley near Work Shaft 2B is also not well defined. Additional exploration around shafts, inlets,and the outlet are discussed later in the text.

3.3.3.2 Geophysics

It was planned to use surface methods to try to delineate the buried valleys along the tunnel alignment. The methods considered were refraction seismic, Vibroseis, and microgravity. All of these methods have been used successfully in similar applications; however, difficulty with outside noise interference was anticipated because of the level of industrial development in the basin. When these methods were investigated further it was found that the current scope of work and an inability to acquire rights-of-entry in the areas of interest prevented their use. It is recommended that future studies utilize one of these methods to help target sites of subsurface exploration in the next phase of design.

Down hole geophysical logging should be performed on some of the future exploratory holes. As a minimum Natural Gamma, SP and Resistivity logs should be run.

3.3.3.3 <u>In-situ Stress Measurement</u> To date no attempt has been made to measure the in-situ state of stress along the project alignment. Regional data is available which indicates that high horizontal stress is present at some East Coast locations so it is recommended that some future effort be aimed at measuring this condition. However, this data is notoriously difficult to acquire and even a horizontal to vertical stress ratio of 3, which is close to the upper range of the available data for the East Coast, would not prove critical to the tunnel design. However, it could result in some cracking and spalling in the crown and invert of the bore resulting in a nuisance and potential worker safety hazard. It is planned during the FDM studies to make stress measurement using hydraulic jacking, hydraulic fracturing, or, prossibly borehole pressure cells.

3.4. SITE GEOLOGY

3.4.1 Stratigraphy of the Main Tunnel

3.4.1.1 Overburden The overburden is as described in the Introduction section of this appendix.

3.4.1.2 Bedrock

The alignment of the Passaic River Basin Tunnel cuts across the strike of the Newark Basin. Over the 107,746.4 foot length of the tunnel, over 14,000 ft of strata will be crossed. A separate folio of drawings showing the detailed logs of the borings on individual sheets is available in the Passaic River Division Office along with the original field geologists logs. This text portion provides a narrative description of the rock types to be traversed by the tunneling however a clearer picture of the geology may be gained by referring to the geologic cross section along the tunnel alignment which shows the boring locations and stratigraphy. This section is presented in Attachment E.3.3 drawings E.3.2 through E.3.22. The thicknesses of formations reported below are only approximate because of variations in the dip of the beds along the tunnel alignment and gaps in the exploration. A discussion of the amount of tunneling in each formation is presented in paragraph 3.8.1.3.3.

The tunnel outlet, in the vicinity of Kearny Point on Newark Bay, is the lowest point in the geologic sequence. Proceeding up-section from the outlet, the tunnel will begin in the Passaic Formation, a sequence of Triassic "red-beds" composed primarily of sandstone with associated siltstone and shale. The Passaic Formation in the lower 20,000 to 30,000 feet of the tunnel is generally finer grained rock than the upper material. It is primarily siltstone, claystone, and shale with lesser amounts of sandstone. It is characterized by significant quantities of secondary gypsum which is present as joint fillings, bands up to 6 inches thick, and as "blebs" of disseminated material. The bands and joint filings are typically dense and "sparry". The disseminated material produces a characteristic green staining in the red claystone in which it is usually found. Some of this material has irregular voids up to four inches in maximum dimension and associated "pin hole" porosity. X-ray analysis of these fine grained rocks indicate it is 40 to 60% clay consisting of illite/sericite and chlorite, 15% quartz, 10 to 12% feldspar, 5 to 14 % gypsum, 5 to 6% carbonate and the balance opaque minerals. Near tunnel station 222+00 the sediments are cut by three closely spaced diabase sills which have a maximum individual thickness of 16 feet and a total thickness of the zone of over 27 feet. The hole which sampled this material did not extend through the entire thickness of the sills so the maximum total thickness of the zone is not known. Two additional sills of similar composition were intercepted by boring DC-38 near tunnel station 277+60. They are between one and three feet thick and are separated by about 35 feet of sediment. A thin band of contact metamorphism surrounds these intrusive layers. Progressing up section the coarser grained red-brown sandstone, which is characteristic of the Passaic Formation, predominates. Based on the lithologic distribution indicated on the field logs for this portion of the Passaic Formation it consists of 15% fine grained rock and 85% sandstone which includes infrequent, discontinuous beds of conglomerate. The sandstone is feldspathic, frequently micaceous, and calcareous. It varies from very fine grained to coarse grained with the individual grains being angular to subrounded. These characteristics are typical for continental sediments which have not been transported over great distances. The sharpness of the grains indicated on the thin sections is significant from the standpoint of cutter wear on the TBMs. The thin section photomicrographs included in Attachment E.3.4 reveal the nature of this material. The x-ray diffraction analysis indicates that this rock is cemented with ferruginous clay, calcite, and ankerite in various proportions. The quartz/feldspar content varies from 45 to over 70%. Some of the clay in the 14 degree range is interpreted to be expansive smectite. Assuming that 200 vertical feet of the formation will be repeated along the tunnel drive because of faulting, the estimated total thickness of Passaic Formation traversed by the tunnel is 6,750 feet. A contact zone at the top of the Passaic has been heat altered by the overlying Orange Mountain Basalt. This altered material varies in thickness up to several feet and is very hard and abrasive.

The Orange Mountain Basalt of First Watchung Mountain overlies the Passaic Formation. The base of this igneous unit is interpreted to be the contact between the Triassic and Jurassic aged rocks. Based on measurements in the borings, this formation is approximately 515 feet thick along the tunnel alignment. It is made up of at least three lava flows each of which contain beds of pillow lava structure at the base with thick sections of massive, fresh basalt capped by vesicular basalt in the upper portions of the flows. The bottom of the middle flow unit contains some thin sandstone beds surrounded by pillow basalt which indicates a prolonged interruption during this extrusive episode. A total of approximately 125 feet of pillow basalt in 4 separate layers is indicated by the core. The voids between the pillow structures are normally partially filled with crystalline secondary mineralization consisting of quartz, calcite, prenite, chlorite, and various zeolites. Different samples show different degrees of alteration. The x-ray analysis of the dense basalt detected 35 to 40% feldspar, 25 to 35% pyroxenes, 5 to 10% opaque ores and the balance alteration products and accessory minerals. The close intergrowth of pyroxene and feldspar make the fresh, unaltered basalt of this formation a very tough, strong rock. A thin section photomicrograph of the dense basalt is presented in Attachment E.3.4.

The next overlying formation is the Feltville which is approximately 435 feet thick. This formation is composed mainly of fine grained sandstone with about 40 to 45% shale in beds of 1 to 25 feet in thickness. Cyclic sequences of interbeds going from coarser to finer grained sediments are a characteristic of this formation. Thin calcareous bands up to two feet thick are present in the lower portion of the Feltville formation. The formation is fresh and tightly cemented, however the shale is highly susceptible to separation along its bedding.

Above the Feltville Formation is approximately 1,080 feet of Preakness Mountain Basalt of the Second Watchung Mountain. This basalt formation consists of three units, separated by sedimentary deposits. The lowest unit appears to be a single flow approximately 625 feet thick. The base of the flow apparently encountered some standing water as indicated by an eighteen foot thick layer of pillow basalt. The upper nineteen feet is vesicular. The remainder of the flow is quite massive with thick sections of coarser crystalline gabbro resulting from slower cooling rates. X-ray and thin section analysis of this material determined that it was 40 to 45% feldspar, 30 to 40% pyroxene, 15% ore (mostly magnetite), and the balance accessory and alteration minerals. Some of this rock has a characteristic "dusky red" appearance that results from oxidation of the iron in the minerals. Most of the voids in this rock are either partially or totally filled with secondary mineralization similar to that described in the Orange Mountain Basalt above. This unit is overlain by a persistent, 9 foot thick section of fine grained sandstone containing a small amount of shale. The next unit is a also a single flow unit approximately 190 feet thick capped by a three foot thick layer of shale. It has a vesicular zone in the upper 16 feet and is similar to the lower unit in other respects. Based on the presence of vesicular layers the uppermost unit is made up of several flows that grow progressively thinner toward the top of the Preakness Mountain Basalt. This is consistent with the model that the magma chamber supplying the basalt was becoming exhausted of extrusive material. The contact with the overlying Towaco formation is marked by a widespread brecciated or mixed layer that is probably a preserved aa or pahoehoe surface which has been backfilled by sediments. Boreholes also indicated that the contact is very irregular on a larger scale. Photomicrographs of representative sample of the Preakness Mountain Basalt are included in Attachment E.3.4.

The Towaco Formation is about 1,125 foot thick and is made up of a series of cyclic sediments similar but, on average, coarser textured than the Feltville Formation beneath it. These sediments consist essentially of siltstone and very fine sandstone containing about 20% shale in beds up to 12 feet thick with red and black shale being about equal in amounts. The clastic particles making up this rock are a heterogeneous mixture of quartz, feldspar, mica, calcite and rock particles in a clay matrix leading to a classification as a graywacke. The rock is fresh, tight, hard and massive. A thin section photomicrograph of the graywacke is presented in Attachment E.3.4.

The youngest basalt flows belong to the Hook Mountain Basalt which underlies the Third Watchung Mountain and is composed of at least two separate flows. The total thickness of these flows is between 260 and 280 feet. This formation contains beds of massive, blocky, and fresh, as well as, vesicular basalt. The contacts continue to be at dips of between 7 and 8 degrees as with all the other units. X-ray and thin section analysis was not performed on any of this material.

The youngest formation along the tunnel alignment is the Boonton. This unit is approximately 1,640 feet thick but only the basal 740 ft will be encountered by the tunnel. The formation is made up of about 57% fine sandstone, 19% siltstone and 24% red and black shale, with red shale constituting about 30% of that total. The sandstone and siltstone is feldspathic and contains abundant mica. It is calcareous with some of the carbonate being ankerite. Some of the x-ray data suggests that the clay is expansive smectite. Hydrocarbons and carbonaceous material is present in some of the grey sandstone and black shale beds and is discussed further in paragraph 3.8.7.

3.4.2 <u>Bedrock Stratigraphy of the Spur Tunnel</u> The spur tunnel alignment is a straight route about 6,500 feet in length connecting with the main tunnel above its mid-point and extending toward the west. This alignment is approximately 40 degrees off the regional strike and will result in the strata dipping at about 3.5 degrees into the tunnel. The excavation along the alignment will encounter about 238 vertical feet of the upper Preakness Mountain Basalt and about 215 vertical feet of the basal Towaco Formation. Much of the Preakness Mountain Basalt along the alignment appears to have been altered and fractured. The altered material is dull greenish grey and contains abundant carbonate and chlorite. A complex mesh of fracturing has been healed with calcite in many of the cores and in some cases thin layers of basalt have disintegrated to a granular paste. Thin section photomicrographs of the altered basalt are presented in Attachment E.3.4. The Towaco formation is as described for the Main Tunnel but is more fractured.

3.4.3 Structure of the Main Tunnel

3.4.3.1 Top of Rock and Weathering Profile

The top of rock profile along the tunnel route is largely a reflection of the underlying geology. The toughest material, the basalt has resisted erosion most and forms ridges. Progressively weaker rocks are subject to progressively more erosion and, in general, the low points in the top of rock are underlain by weak shales and claystones. The top of rock profile is also a product of the amplified erosive action of the glaciers and meltwater derived from them. Deep pockets of erosion and weathering in the sedimentary rocks adjacent to the basalts are attributed to plunge pools formed by glacial meltwater runoff through narrow water gaps in the basalt ridges. In the lower valley of the Passaic there is at least one buried valley which extends to elevation -300 ±. This indicates that sea level was much lower at that time. These scoured areas extend to considerable depth below the current stream baseline elevation which is superimposed on this older topography.

A measure of the depth of weathering in the various rock types is sometimes given by the RQD values versus depth below the top of rock. Plots of this data for each rock formation are presented as Figures E.3.1 through E.3.8. As can been seen from these plots there is not a clear cut demarcation in the RQD data which indicates a limit of the depth of weathering. Within the various rock formations there is considerable variation depending on the presence of faulting,folding,or buried valleys but as a general rule the RQD increases with depth. Much of the severely weathered material at top of rock was removed by scour from the glaciers as they over-rode the region.

3.4.3.2 Discontinuities

3.4.3.2.1 General The intact strength of the rocks along the tunnel is sufficient to support the openings proposed. The occurrence of weakness planes in the rock are therefore the key issue in evaluating the stability of the proposed openings. These planes of weakness or discontinuities

are of two primary types, bedding and jointing. A limited number of shears and fault zones have also been encountered in the investigations. Their influence on the tunnel design, though profound, is very localized.

3.4.3.2.2 Bedding

Strata along the tunnel alignment strike primarily to the northeast and dip to the northwest at approximately 7 to 9 degrees. However, the northern most 1.9 miles of the tunnel will be excavated through the Watchung Syncline where beds of the Boonton Formation exhibit a reversal of dip as can be seen on the geologic cross-sections in Attachment 3 on drawings E.3.2 through E.3.22. Pronounced cross-bedding is present in the Passaic and Boonton Formations as a result of fore-set beds in delta deposition. This results in local steepening of the dips to up to 20 degrees ± and variation in the strike.

The average thickness of the bedding varies from thin to massive. Based on the current information, the Passaic and Boonton Formations exhibit the thickest bedding and the Feltville and Towaco Formation have the thinnest bedding. A plot of the average bedding thickness for the portion of the rock mass involved in the tunneling versus approximate tunnel station is shown on figure E.3.9. This data was developed by dividing the length of the each run in the bottom 90 feet ± of each boring by the number of open bedding planes recorded on the geologists' field log in that run. This data was then averaged per hole. Because of the method used the extreme values are not reflected in the data. In some areas the rock is massive with zones over ten feet thick with no bedding present. In other areas the beds are less than 0.1 feet thick. These are not reflected in the data but figure E.3.9 does present an accurate representation of the relative bedding characteristics encountered along the tunnel. Bedding is a factor in the design of the initial rock support system described below in paragraph 3.8.3.

3.4.3.2.3 Jointing

Jointing, in varying degrees, was encountered in all of the exploratory borings. It varied from tight to open, and healed with various minerals to clay filled. Staining from dissolved minerals indicated the movement of water through many of the joints. Soft clay fillings or coatings were less frequent but not uncommon. Surface textures also varied from slickensided to very rough. On a larger scale the joints varied from planar to curving or undulating. The continuity of the jointing was also highly variable and difficult to determine from the core holes. The northeasterly striking joints were usually the most persistent as observed in the limited outcroppings but other joints did show some continuity. In the interbedded material many of the joints were very irregular and often ended abruptly on shale beds. All of these characteristics were influenced by the nature of the rock, the causative mechanisms, and the subsequent action of glacial loading and ground water.

Most of the measurement of the orientation and condition of the jointing was accomplished by use of a down hole television camera with an orientation head and by logging the cores. Limited data was developed from surface outcroppings but these tended to be in the basalts. The data from the core holes was plotted as poles on an equal area stereo net using the procedure outlined in Hoek and Bray. The stereo plots of poles for the various formations are included in Attachment E.3.5. These poles were contoured and composite poles plotted based on areas of greatest concentration. Each of these composite joints has been given an identification number for use in subsequent slope stability analyses. A table listing the composite joints by formation and orientation along with great circle plots on a stereo net for these composite joints are included in Attachment E.3.5. The intersection points of the composite joints that form potentially unstable wedges for the cut slopes and shafts are identified on the great circle plots. Discussion of the wedge failure analysis for the shafts and inlets is presented in paragraphs 4.8.1, 5.8.2, and 6.8.2. For the tunnel crown it takes three or more joints in combination to form potentially unstable wedges. Discussion of the wedge analysis for the tunnel is presented in paragraph 3.7.3 however, the actual analysis has been deferred until a later stage of design.

Figure E.3.10 shows the distribution of dips for all the borehole discontinuity readings taken. Figures E.3.11 and E.3.12 show the distribution of discontinuity data poles for all of this data.

The nature of the jointing in the basalt formations is significantly different from that in the sedimentary rocks. They may have developed from contraction or convection as the lava cooled, or as tectonic joints that have resulted from regional stress conditions on the rock mass. The cooling joints are completely independent of the tectonic joints with their frequency and geometry being dependant upon where in the flow they are located. In general each flow unit can be divided into a series of distinct zones which are either in sharp contact or grade into one another. The top of each flow is usually characterized by a vesicular zone with indistinct jointing that produces a rectangular, slabby pattern. Below this is a columnar zone characterized by columns, commonly six or five-sided polygons. Individual, joint bounded columns vary from .5 to 1 foot wide and 4 feet or more tall. Columnar zones are most well developed in the Orange Mountain and Preakness Mountain basalts where they may form striking fan structures many tens of feet in maximum dimension. Underlying the columnar zones there is the blocky zone which is remarkable for its planarity and the persistence of its jointing. The resulting rock mass is composed of large blocks 10 feet or more on a side. Beneath the blocky zone is the curvilinear zone which typically constitutes the majority of the thicker flows. Jointing in this zone follows a curved helical surface. The pitch of this surface varies from one area to another. In appearance the curvilinear zone resembles the columnar zone except that the joint surfaces are curved. Another interesting feature of this zone is that the overall vertical extension of the joints suggests that they are bounded by a surface which is almost cylindrical. The bounding surface of these large cylinders are thought to mark the boundaries of convection cells within the cooling flow and within the limits of the flows these cells are of fairly uniform dimension. The lowermost unit is a thin vesicular zone which cooled more rapidly because of its contact with the underlying rock. This is often indistinct and hard to identify. These different zones are seen in various degrees in each of the three basalt units. Superimposed on this system of joints are tectonic joints. They may be the product of regional stress field or localized unloading of the rock mass. Stress related joints may be modified by the preexisting set of cooling joints but are normally oriented with a principal set parallel to the Ramapo Fault and the axis of the Newark Basin and a conjugate set roughly perpendicular. The conjugate set is most apt to be modified by the cooling joints. It appears that the tectonic jointing is most likely to be open and exhibit the highest degree of weathering, staining, and mineral or clay filling. They are the most persistent and therefore the most likely ground water conduits and the most likely to influence the stability of the tunnel opening. Cooling joints are most often tight and rough in texture having developed largely from tensional rather than shear failure. Sheet jointing is well developed in some areas and is the product of unloading of the rock mass. The stress relief toward the surface results in the formation of rough somewhat persistent joints sub-parallel with the ground surface. This type of jointing is less likely to be found at tunnel depth but is well exposed in outcrop especially near Little Falls. Figure E.3.13 shows the results of some joint scan lines which were measured in the Orange Mountain basalt. They give some indication of the joint spacing which is anticipated in the columnar zones of all of the basalts.

Jointing in the sedimentary rocks is primarily of tectonic origin although some is the product of consolidation of the sediments. The continuity of the jointing is influenced by the nature of the material through which it passes. It is generally anticipated and has been seen from the down hole video logs that the joints become tighter with increasing depth. The water producing zones discussed in Section 2 of this appendix are notable exceptions to this rule.

3.4.3.2.4 <u>Shears and Faults</u> Faults are present along the tunnel alignment. The most evident of these are located in the Montclair, Great Notch area. This zone is made up of several parallel and subparallel failure planes which have been observed both in outcrop and in the drill holes. Gouge zones consisting of brecciated rock in clay and secondary mineralization have been observed and recovered in both the Orange Mountain Basalt of the First Watchung Mountain and in the underlying Passaic sandstone. The combined vertical displacement on this series of north easterly striking faults is approximately 200 feet. The individual faults comprising this zone are quite well defined. Most consist of a brecciated layer, normally one foot or less thick, in sharp contact with relatively fresh wall rock. The rock mass surrounding these faults does contain closer spaced jointing than unfaulted rock and numerous slickensided surfaces. At least one of the faults in this area has a zone of wet clay gouge and breccia about 4 ft thick and highly disturbed rock for about 50 ft on either side. The borings in the disturbed zone indicate that it has been healed to a large extent by calcite filling the fractures. Another fault is inferred by correlation of the beds in the Boonton Formation. A highly disrupted zone was encountered near the Spur Tunnel intersection that may be related to the condition of the rock encountered along the Spur alignment. While the actual plane of a fault may not have been encountered in this location it is very likely that one exists here. All the faults that have been identified with any certainty appear to normal faults with high angle dips greater than 70 degrees and normally in a north west direction. Other faults will no doubt be discovered during future exploratory programs. Additional evidence of movement in the rock mass is provided by high angle clay seams, alteration of the rock, and slickensided surfaces. Numerous low angle slickensided surfaces were found in the cores of the Boonton Formation. Most of these are interpreted to be relative movement which occurred along bedding planes during the folding that produced the Watchung Syncline. A minor fault was noted in boring C-106 with a displacement of 0.02'. Additional low-angle slickensides, not related to any of the structures previously mentioned, were recovered in the Hook Mountain Basalt, the upper Preakness Mountain Basalt, and in the Passaic Formation. Boring C-62 penetrated a zone of mineralized and brecciated rock at tunnel station 457+62. A similar, but less disturbed zone was encountered in boring C-59 at Station 439+23. Slickensides were also associated with the emplacement of the igneous intrusives at stations 277+60 and 222+72.

3.4.4 Structure of the Spur Tunnel

3.4.4.1 Top of Rock and Weathering Profile A buried valley is indicated by boring DC-118. Local residents indicated that a brick manufacturing plant and clay pit had been operated in this same area for many years and had subsequently been backfilled. This information has not yet been substantiated. Regardless, there is indication that the top of rock is low in this area. The depth of weathering indicated by the RQD values appears to be greater also.

3.4.4.2 Discontinuities

3.4.4.2.1 Bedding and Jointing Bedding and jointing are basically as recorded for the Main Tunnel except the frequency of jointing is greater along the Spur Tunnel alignment.

3.4.4.2.2 Shears, Faults, and Folding The Preakness Mountain Basalt sampled by the drilling along the Spur Tunnel has been chemically altered in many places as described in paragraph 3.4.2 above and it is suspected that this is the result of mineralizing fluids present in a fault zone. The spur tunnel alignment is sub-parallel to the major structural axis in the basin so it is suspected that it is paralleling a fault zone. Numerous low angle shears, unaccountable loss zones, slickensided planes, and clay filled joints are present in the cores and in one place the core barrel dropped a foot during drilling indicating an open or at least very weak area. Additional exploration is needed, angled perpendicular to the alignment of the Spur to confirm the presence of faulting.

3.5 LABORATORY AND FIELD ROCK MECHANICS TESTING

3.5.1 General

As is true of the exploration program, the rock mechanics testing conducted thus far has been aimed primarily at the tunnel design. Despite this, much of the data is equally applicable to the characterization and design of the shafts and surface works. The test program procedures and results are covered in this portion of the text and not repeated in subsequent sections covering the geotechnical design of the Shafts, Inlets and Outlet other than as a reference. A summary of the rock testing that has been performed thus far is summarized on a table in Attachment E.3.6. Graphical presentation of some of the testing is also presented in this appendix. During the early feasibility studies the testing focused on unconfined compressive strength and unit weight tests. This program was expanded in the current phase of design to provide information on other rock properties as described in the introduction. Most of the samples in this later phase of testing were wrapped in polyethylene tubing to preserve the natural moisture content. They were tested either in an as received moisture condition, or submerged for two weeks in water and tested saturated-surface dry. Some of the later shear strength testing was done in a submerged-saturated condition. For the finer grained sedimentary units drying of the samples during the first phase of testing resulted in strength gains except for the fissile shales.

The applicability of the testing and a summary of the results are presented in the following paragraphs. The complete laboratory reports are on file with the New York District Corps of Engineers and present details that are important but too lengthy to be included in this report.

3.5.2 Unconfined Compressive Strength The Unconfined Compressive Strength Test (U_c) is a widely used index test which gives some indication of the rocks resistance to breakage. This is not the definitive parameter for excavation since many other factors are influential but it is one of the fundamental parameters used to give some indication of the TBM production rate discussed in paragraph 3.8.1.3.3. These tests have been run on intact samples however many had incipient fractures present that controlled the breakage. On some of the tests the deformation of the samples was measured to determine the Elastic Modulus and Poisson's Ratio of the materials. In all, 153 Uc strength tests have been performed. Deformation readings were taken on 66 of these. As would be expected the more compact, dense rocks such as the very fine grained sandstones and the basalts have very high elastic moduli exceeding 10^7 PSI but even the siltstones and mudstones are not highly compressible. The greatest compressibility was exhibited by the loosely cemented, coarse grained sandstones of the Passaic Formation. Figures in Attachment E.3.6 show the Uc data on a formation by formation basis. As can be seen from these figures there is significant variation of strength within each formation.

3.5.3 <u>Brazilian Tensile (Splitting) Test</u> This type of test is widely used to determine the tensile strength of material by splitting a disk of specified dimensions. This index test is also indicative of TBM production rates. Eight of these tests were run on the various lithologies. The sedimentary rocks ranged from 602 PSI to 1,651 PSI with the shale and micaceous rocks being weakest. The two basalt samples broke at 1,198 PSI for the amygdaloidal rock and 2,043 PSI for the dense basalt.

3.5.4 <u>Direct Shear Testing</u> The direct shear testing program consisted of testing both intact samples and samples with a pre-sawed failure surface, in a direct shear box. Most of these tests were run in as received moisture content. Those tested at Missouri River Division were run submerged. Normal loads of between 100 and 450 PSI were utilized for most of the testing however a few tests were run at lower normal loads. For the intact strength determinations, samples were selected which had a visible plane of weakness on which failure was expected to occur. A peak strength at failure and residual strength or sliding friction after failure were recorded on each test at a given normal load. In most cases three samples from each rock core were taken so that a suite of tests at three different normal loads could be run. These three results provided three points for drawing the peak strength envelope and three for the sliding friction envelope. In some cases failure did not occur on the predicted plane of weakness. Also, variations from sample to sample were unavoidable. For these reasons there is considerable scatter in the intact direct shear test results. This was anticipated so numerous tests were run to try to even out the discrepancies. The sliding friction data shows less scatter than the intact strength data. In addition selected samples were prepared by pre-sawing and lapping a failure surface. Each sawed surface sample was tested using three different normal loads and a failure envelope developed for each set of data. This test represents a lower bound for the shear strength of a particular rock type and normally is most consistent from the standpoint of variations. The direct shear strength test data and sliding friction test data is shown on figures in Attachment E.3.6 for five of the formations. For approximating the shear strength of jointing in the stability analyses the sliding friction values were used. These tests were run at high normal loads which results in lower bound values for the friction angles. This may be overly conservative and subsequent testing should be conducted with lower normal load ranges.

3.5.5 Triaxial Test Limited triaxial testing has been performed so far. The results are not presented here but are available through the New York District Office of the Corps of Engineers.

3.5.6 X-Ray Diffraction

To adequately define the mineralogy of much of this rock it was necessary to use x-ray diffraction. Much of the clay mineralogy of the samples does not indicate a swelling tendency being mostly illite and kaolinite with trace amounts of chlorite. However the x-ray data shows the presence of smectite in many of the samples. The swelling characteristics of the clay fraction were determined by the ethylene glycolation test. This information conflicts with the findings of the jar slake testing described later but suggests that the proportion of expansive clays not sufficient to cause a problem in most cases.

It is also interesting to note that the percentage of feldspar in the sandstone samples ranges from 3% to 7%. This is an indication that the material tested is not highly weathered.

The mineralogy of the igneous rocks is also revealing in showing the amount of weathering and alteration that has occurred and as an indicator of the rock strength.

3.5.7 LA Abrasion Testing LA Abrasion tests were performed on Passaic sandstone, Towaco shale, and Preakness Mountain gabbro. The shale exhibited the highest loss at 31%. Next was the sandstone at 16.67% then the Gabbro at 10.16%. As expected, the igneous rock is relatively tough and resistant to abrasion losses. These values are probably representative of these rock types regardless of which formation they are from however the greatest variability should be expected from the sandstones.

3.5.8 Unit Weight Testing Unit weight determinations have been performed on 165 samples from the various lithologies. The data is summarized on a Table and shown on figures in Attachment E.3.6. The samples were tested with as received moisture content which affected the results slightly. The unit weight testing has been used to estimate the vertical stress at the tunnel level discussed in paragraph 3.6.3 and it is needed in performing the plane failure and wedge failure analyses discussed in paragraphs 4.8, 5.8 and 6.8.

3.5.9 Schmidt Hammer Testing The Schmidt hammer rebound test is a commonly used index test. Tests were run on 18 samples of various lithologies. The samples were submerged in a water bath for two weeks prior to testing. Seven of the tests produced no reading because of sample saturation. Rebound numbers for the basalt samples ranged from 24.6 for vesicular material to 36.7 for finely crystalline Orange Mountain basalt. Most values were above 30. The rebound number for the sedimentary rocks ranged from 12.2 to 27.7. None of the shales tested provided a reading.

3.5.10 Specialized Testing Because of the individual performance characteristics of various types of tunnel excavation machinery, different manufacturers have devised different test methods to evaluate what is commonly referred to as a "Drillability Index". These tests are specialized and not usually run by most laboratories. Results of this testing, when completed, will be interpreted by the manufacturer to predict machine productivity, cutter disk wear, and other pertinent design considerations. To this point a limited amount of this type of testing has been accomplished. Some by the manufacturers and some by the independent laboratory of the Norwegian Institute of Technology. This testing was focused on the toughest, most abrasive rock cores that were available. Information from this testing was used in the project cost estimate to predict cutter wear and costs.

3.5.11 Future Rock Mechanics Testing

Additional rock mechanics testing is required to provide a better statistical base for some of the rock properties. Future testing will include a large number of unconfined compressive tests as well as point load testing of core as it is recovered from the bore hole. Additional direct shear testing is also required along with more specialized testing for predicting cutter wear and machine production.

In the next stage of design, sources of construction materials will have to be identified, sampled, and tested for suitability for use as concrete aggregate and slope protection stone

3.5.12 Field Testing Rock

3.5.12.1 Point Load Testing

3.5.12.1.1 Test Equipment and Methods As part of the rock mechanics test program, point load tests were run on cores from the tunnel section. The point load test is an index test which is easy to run in the field and which, through the simple conversion, can be compared to the unconfined compressive strength. The comparative ease of the point load test allows a broader coverage of the rock strength than is possible with laboratory unconfined compressive tests and allows testing of cores immediately after they are recovered. The test equipment used, shown schematically on Figure E.3.14, consisted of a model PLT-10 instrument manufactured by the Structural Behavior Engineering Laboratory of Phoenix, Arizona. The machine was fitted with a load frame, a hydraulic jack, a hand-pump, and a gage which read directly in pounds force exerted at the platens. The gage is graduated up to 10,000 pounds. Tests are run both across the diameter of the core and along the axis of the core so a measure of anisotropy is developed. The sample is placed, as much as possible, such that the load acts along the central axis of the preferred failure plane. The load is applied by the hand pump and measured by the hydraulic gage. The gage is fitted with a maximum deflection needle which marks the maximum load sustained by the sample before failure.

3.5.12.1.2 Records and Data Reduction The location, and orientation of the sample along with the test results were entered in the remarks column of the field log (ENG FORM 1836) roughly in the area of the test. Unusual breakage patterns were recorded by a sketch. Data on each of the tests were entered in a data base and analyzed using a micro-computer. The point load index for these tests was computed according to the International Society of Rock Mechanics recommended procedures and corrected for size so the Index reported is the $Is_{(50)}$. The $Is_{(50)}$ values were multiplied by a correlation factor of 24 to give an approximation of the Unconfined Uniaxial Compressive Strength. This correlation factor is based on work by Bieniawski, D'Andrea and various other authors. The comparison with Unconfined Uniaxial Compressive Strength is less valid for the diametral test where the breaks are occurring parallel with the bedding.

3.5.12.1.3 Test Results

A clean, planar failure surface is frequently not possible with this test which results in a considerable amount of scatter in the data. In the thinly bedded material the axial tests usually broke on a diagonal or in a "stair stepped" surface to the side of the core. Failure of weaker rock were often progressive with the cone shaped platens gradually penetrating the sample before failure. The strength of the weakest material was often below the sensitivity of the pressure gage, especially for the diametral tests in shales. In the more homogeneous rock the diametral tests sometimes broke axially and in some cases the strength of the rock exceeded the range of the gage. There is a lot of scatter in the data. This is expected because of normal variation in rock strengths. This results from differences in cementation, bedding, mineralogy, cohesion, texture, etc. In the sedimentary rock the axial strength is normally much higher than the diametral strength because they are generally perpendicular to the bedding. For the basalts the opposite is generally true although the difference in strength is less.

The amount of data for the point load testing is too great to be presented in full in this document. Instead it is summarized on the figures contained in Attachment E.3.7. The field data compares favorably with the laboratory test data.

3.5.12.2 Jar Slake Testing

3.5.12.2.1 Methods A crude measure of the slake durability of the various rock types is provided by this method. A short piece of core from a rock type which is suspected of being susceptible to slaking is placed in a jar full of water and observed over time. Periodic observations of the sample are made to see if any material has spalled off and an estimate of the % of material lost and the time of the observation is made.

3.5.12.2.2 Results Generally, while many of the rocks present along the tunnel alignment contain significant clay fractions, few of them show a susceptibility to slaking. Most of the clay minerals present are Illite and, consequently are fairly stable. A further measure of this is the condition of the rock cores after many years of storage. Only a small percentage of it has deteriorated significantly because of slaking despite being exposed to the effects of weather. Similar observations can be made on the condition of the few surface exposures in these rocks.

3.5.12.3 Water Pressure Testing

3.5.12.3.1 General This data is most pertinent to the ground water study presented in section 2 of this appendix. It is presented here because it was part of the geotechnical exploration program. The majority of the pressure testing consisted of using a wire line packer in HQ diameter core holes. For most of the holes only the zone of major influence was tested. Generally this was assumed to extend from 45 feet above the proposed crown elevation to 45 feet below the proposed invert elevation. Selected holes were tested for the full depth to try to determine the connection between the overburden and the rock. The borings drilled in Newark Bay were tested using 5 foot straddle packer.

3.5.12.3.2 Data Reduction

The aim of the pressure testing was to provide information on the ground water transmission characteristics of the tunnel rock mass, specifically the coefficient of permeability, k. The recommended procedures in, Bennett, R.D., and Anderson, R.F. Technical Report GL-82-3, "New Pressure Test for Determining Coefficient of Permeability of Rock Masses", July, 1982, Waterways Experiment Station and Goodman, R. E., Moya, D. G., Van Schalkwyk, A., and Javandel, I., "Ground Water Inflows During Tunneling" have been used for preparation of the data base for pressure test information. It is consistent with the other references and with Corps guidelines. The equation used is for a continuum analysis (ie. assumed uniform seepage along entire test length) using the constant head test. It is;

$$K_e = \left({}^{Q}/{2\pi L H_o} \right) * \ln \left({}^{R}/{r_o} \right)$$

where: K_e = Equivalent coefficient of permeability (LT^{-1})

Q = Volume flow rate at equilibrium (L^3T^{-1})

r_o = Borehole radius (L)

R = Radius of influence of the pressure test (Distance from borehole at which excess pressure is zero)

H_o = Excess pressure head at center of test section

$H_o = {}^{(P_t - P_o)}/\gamma_w$ (Force L^{-2})

L = Length of test section (L)

(Consistent units should be used for all the variables)

In this equation R, the radius of influence is not known. However, research has shown that in fractured media;

$$L>R>L/_2$$

Generally R will be smaller in a fissured mass because head loss occurs more rapidly with distance from the hole. For this data R is assumed to be .75L. The pressure test data is presented in Attachment E.3.8.

3.5.12.3.3 <u>Analysis</u> The pressure test data were used to estimate the water inflow which is expected during construction of the tunnels and the amount of grouting ahead of the TBM that would be required. Grouting ahead of the TBM adds considerably to the cost. In addition the analysis provided input into the prediction of ground water draw-down that would result from the tunnel construction and operation. For purposes of this analysis the tunnel was divided into two reaches, one from the outlet to station 485+00 and the other from station 485+00 to the main inlet. This division was based on an approximation of the depth of cover over, and hence the hydrostatic pressure on, the tunnel (see paragraph 3.6.3, Stress Analysis). The permeability data was presented as a histogram for the two tunnel reaches. Each histogram, included as figures in Attachment E.3.8, presents the data for a series of permeability ranges expressed as a percentage of the total length of borehole tested. Shortcomings of this approach include over representation of high permeability zones. For example, a short reach of high permeability rock in a long test section will be represented as a high permeability for the entire length of the test section. Also, because of the method used in the field operations very low permeability zones, with K less than $1x10^{-5}$ cm/sec are below the sensitivity of measurement. Overall, however, these are thought to have a minor influence on the analysis.

3.5.12.3.4 <u>Conclusions</u> All analysis of the results was conducted as described in Technical Report GL-82-3. The pressure test data has been compiled in a data base and an equivalent k computed for each test. This was done using the continuum model constant head test. The length of tunnel involved suggests that a continuum method would be acceptable for overall modeling. The permeability data and groundwater level data are presented in Attachment E.3.8.

3.6 DESIGN PARAMETERS

3.6.1 General Many of the design parameters needed for the tunnel have already been discussed in previous paragraphs. These include Unconfined strength, shear strength, unit weight, structural geology, and permeability. Additional parameters not discussed previously are outlined below. The specific impact of these parameters on the design process which was utilized is discussed in greater detail in paragraph 3.7.1.

3.6.2 In-Situ Stress The principal stress direction has not been determined at this level of design because it is not likely to add substantially to the cost of construction. It is an important safety issue and should be addressed in the next phase of design. The problem lies in the cost of acquiring the data at depth and its reliability. It is probable that there will be significant differences in both the magnitude and direction of the principal stress at various locations along the tunnel. This is influenced by the localized geology. Several tests will have to be run at several locations to acquire a statistically significant sample and to account for variations in

the stress. Hydro-fracturing is the most likely method which might be used to determine the in situ state of stress. To get a reading using the hydro-fracture technique it is necessary to have a thick, homogeneous section of rock to set the packers in.

3.6.3 Tunnel Stress Distribution The hydrostatic pressure is assumed to result from the weight of the water column above the tunnel as measured in the borehole. This is an over simplification but as a rule would yield the maximum possible hydrostatic pressure. It is likely that fractures in the rock are the main conduit for water transmission and therefore the hydrostatic pressure is not uniform throughout the rock mass. The vertical stress is assumed to be the result of the weight of the overlying materials. No attempt has been made in this analysis to measure or estimate stresses resulting from tectonic forces and it is assumed that any vertical stress that was produced by the glaciation has already dissipated. To compute the weight of material at a location each boring log was checked to measure the ammount of each different lithology above the tunnel. For example feet of Boonton Formation shale, feet of overburden etcetera. Each of these footage values was multiplied by the average unit weight of the material as determined in the laboratory and all of the values added together to give the total vertical stress measured at the tunnel crown. To get the effective stress the hydrostatic pressure should be subtracted from the vertical stress. Plots of this data versus tunnel station are presented on figures and tables in Attachment E.3.10. The hydrostatic pressure was used to design the tunnel liner as described in Appendix G.

3.7 STABILITY ANALYSES

3.7.1 General

The ultimate goal of the tunnel design is to ensure a stable opening of the required dimension, with the desired hydraulic characteristics, and at the least cost. As the opening is made it will disturb the existing, at-rest state of the surrounding rock mass. Over some variable time period dependant upon rock mass conditions and state of stress, the rock mass and stress within it will redistribute in such a way to produce a new at-rest state. The design should assure that this occurs without loosing the project function. To ensure this, the rock mass is provided initial support to control movement within it such that a self supporting rock arch is developed around the desired opening. The goal is to install an adequate amount of an appropriate type of initial support that will safely accomplish this. Determination of the type and amount of initial support is central to the design. The final liner is to be installed to provide the required hydraulic characteristics and to resist the hydrostatic pressure acting on it.

Among the many factors influencing the selection and design of the initial support are, method of excavation, size of the opening, purpose for which the opening is being made, practical construction and compatibility issues, groundwater conditions, rock strength and deformation characteristics, state of stress, rock unit weight, discontinuity characteristics, discontinuity spacing, and more. Analysis must take all of these factors into account. Two primary methods of performing these analyses exist, the numeric or analytical and the empirical. Numeric methods utilize the principals of rock mechanics and a knowledge of the site conditions to evaluate the problem. Several numerical methods are available and fall into two main categories based on how they model the rock mass. The continuum methods treat the rock mass, more or less as if it was free to deform in any direction. Some of the more sophisticated of these methods allow for modeling anisotropic strengths but they all treat the mass as a continuous medium so movement can occur in any direction favored by the mass strength and induced stress field. The discontinuum models treat the rock as a fractured medium and constrain movement to the discontinuities which occur in set orientations. Examples of both of these methods are described in greater detail below. The empirical methods evaluate the rock mass based on a data base of past experience using a set of parameters that allow quantifying of mass properties. The numbers assigned to these parameters allow for a relative comparison of various properties of the rock mass but generally do not define a measurable physical property. For the current study the empirical method was used to select the support requirements and formulate the estimate. A detailed description of this is provided below. It will be appropriate to perform the numerical analyses on the proposed support system selected by the empirical methods at a later date. What should be kept in mind is that many of the parameters that actually effect the stability of the opening are highly variable over the length of the tunnel and also very hard to quantify precisely. It is completely impractical to provide a precise analytical design for every point along the tunnel. For this reason a reasonable degree of conservatism is warranted in selecting the support to account for uncertainties. Each of the numerical evaluations of the support designs should be conducted at several places along the tunnel alignment to provide a reasonable measure of assurance.

3.7.2 Numerical Methods, Continuum Model

One generic type of numerical analysis is called the boundary element stress analysis. Using a finite element grid this method is used to approximate the stress field surrounding the tunnels and how the rock mass will respond. Input into this analysis includes the rock mass strengths and the in-situ stress field after the tunnel opening is made. It predicts the stability by dividing the available rock mass strength by the induced stress field around the tunnel at each node point of the grid. Where this ratio is less than 1 the rock mass is over-stressed and in need of additional support. The location and length of the support, ie. rock bolts, shotcrete, or steel ribs can then be evaluated. For this method the rock mass is assumed to isotropic and homogeneous.

Modeling of non-linear, non-elastic behavior which is not possible with the boundary element analysis requires a more sophisticated modeling technique known as a finite difference model. Several variations of this type of analysis are available. The Fast Lagrangian Analysis of Continua or FLAC is one of these programs. Using this model it is possible to evaluate the interaction of the support system with the rock mass, discontinuities, and stress field. Out put from this program includes a displacement diagram which shows the predicted magnitude and direction of movement within the rock mass as a result of the tunneling after the support is installed.

3.7.3 Numerical Methods, Discontinuum Model

A recently developed numerical model that evaluates the rock mass as a fractured medium is called Dynamic Discontinuous Deformation Analysis or DDDA. This method incorporates the properties and orientations of discontinuities as well as the intact rock mass. Since the strengths along the discontinuities are normally much lower than the intact rock, failure in this model will occur along these weakness planes. The effects of the support system can be input to this model by increasing normal loads across discontinuities. This is an alternate method that uses a more accurate model of failure that the continuum methods.

Another type of discontinuum method that evaluates wedge failure in a tunnel is conceptually similar to the DDDA but handles the analysis one wedge of rock at a time. For the tunnel, unstable wedges may form in the crown with 3 or more joints in combination or in the walls with two joints in combination with a bedding plane. The maximum size of the wedge is limited by the size of the opening. The orientation data is fed into the computer which then computes, geometrically, the largest wedge that is possible under the configuration which is input. The stability of this wedge is then analyzed. "Unwedge" or "Phases" are both public domain programs which perform this type of analysis. The wedge analysis is described in greater detail in Section 4 for the shafts.

3.7.4 Empirical Methods

3.7.4.1 General

Because of the large number of parameters and their variability it has proven practical to evaluate the stability and support requirements of tunnels based on experience developed over many years. Key properties of the rock mass such as strength, fracturing, groundwater conditions, and state of stress have been numerically characterized with these systems to provide a comparative basis on which to evaluate design and to provide a basis for future design. These rock mass classification systems, as they are known, have been developed using large empirical data bases and are normally a very conservative design approach.

For this phase of design two rock mass classification systems have been used, the Q-system, developed by the Norwegian Geotechnical Institute, and the RMR-system, developed by Bieniawski. In addition a combination of the two systems has been tried. The two systems were used to serve as a cross check and confirmation. The combined system was used to level inconsistencies between the two systems on individual readings. The strength of this approach is that instead of basing the design and estimate of support on a few rigorous numerical evaluations it is based on the data acquired by the exploration program, virtually in its entirety. During the next phase of design it is recommened that Terzaghi's classification system be used also. This system has been widely used in the past and is valuable in communicating rock mass conditions for tunneling.

In this evaluation each run of core in the bottom ninety feet of drilling for each hole was evaluated with both the Q and RMR systems. The rock above this zone was not considered because it may have been of lesser quality being closer to the top of rock and more effected by weathering. It was felt that this would imbalance the analysis towards inferior rock that would not be involved in the tunneling. For each system the strength characterization was based on the point load test data axial tests. This was felt to be specifically representative of the rock within the pull. The RQD did not require interpretation. Groundwater conditions were evaluated based on the water pressure testing that was performed on the pull being evaluated. Threshold permeability values were selected for each groundwater category. The discontinuity data was taken from the visual descriptions on the geologist's field logs. Conservative fixed values were selected for those items such as joint persistence and stress condition that could not be discerned from the available data. In this way a consistent evaluation from hole to hole was achieved. A more detailed discussion of each of the two systems is provided below. Comparative plots of the Q versus RMR data are presented, by formation, in Attachment E.3.11. As can be seen this data is very consistent with the literature.

3.7.4.2 Q The Q-System is based on a numerical assessment of the rock mass quality using six different parameters and based on the following formula:

$$Q = RQD/J_n * J_r/J_a * J_w/SRF$$

J_n is the joint set number and is based on the number of different discontinuity sets that are present. J_r is the joint roughness number and is based on the joint surface characteristics for the critical joint set. J_a is the joint alteration number and is based on the surface condition of the joints. J_w is the joint water reduction number and is based on the groundwater conditions. This number was selected based on the permeability computed from the pressure testing. The SRF is the stress reduction factor and is based on an approximation of the in situ stress condition. Since this has not yet been defined for the project a conservative value of 2.5 was used in all of the Q determinations. This corresponds to a low stress, near surface condition, a reasonable model for the Passaic project. If higher stress is assumed the value of Q will go up. Based on the data bases certain ranges of Q correspond to descriptive rock qualities. The following table lists these ranges and qualitative statements:

0.01	to	0.1	=	EXTREMELY POOR
0.1	to	1.0	=	VERY POOR
1.0	to	4.0	=	POOR
4.0	to	10.0	=	FAIR
10.0	to	40.0	=	GOOD
40.0	to	100.0	=	VERY GOOD
100.0	to	400.0	=	EXTREMELY GOOD
	>	400.0	=	EXCEPTIONALLY GOOD

Histograms of the distribution of the Q values by formation are included in Attachment E.3.11. The rocks along the spur tunnel were treated separately because they were visibly of poorer quality and because the Spur Tunnel is to be a smaller diameter than the Main Tunnel.

3.7.4.3 <u>RMR</u> The RMR or Geomechanics Classification System is also based on a numerical assessment of six parameters; one is for intact strength, one is for RQD, one is for spacing of discontinuities, one is for condition of discontinuities, one is for groundwater conditions, and finally there is an adjustment for the orientation of the jointing. Each of these parameters is assigned a point value based on where in the range of possible conditions the observation falls. The RMR is computed by adding these six parameter values together. The unadjusted maximum value for RMR is 100, that representing the best quality rock possible. The lower the value of RMR the poorer the rock quality. To speed the evaluation of RMR, a computer program developed by Bieniawski, the author of the method, was used. Because of the limitations of using a 2½ inch diameter core for this evaluation certain assumptions had to be made. The persistence of the jointing was assumed to be 50 feet in all cases. As a rule this is reasonable and, since it spans the tunnel dimension, it provides a conservative basis for design. The joint spacing was computed for each pull by dividing the length of the pull by the number of fractures in it. This often contained breaks that were induced by the drilling so, here again a conservative value was used. The general groundwater condition was figured by using the pressure test data and selecting reasonable values for the range values. The computed permeability in centimeters per second times 10^{-5} was the basis for this selection as follows; K = 0 was completely dry, K from 0 to 10 was damp, K from 10 to 50 was wet, K from 50 to 100 was dripping, and K > 100 was flowing. All of the other relevant parameters could be taken from the field logs without much interpretation. Histograms of the distribution of the RMR values by formation are included in Attachment E.3.11. The rocks along the spur tunnel were treated separately because they were visibly of poorer quality and because the Spur Tunnel is to be a smaller diameter than the Main Tunnel.

3.7.4.4 <u>Combined Q and RMR</u> In an effort to normalize some of the inconsistencies for individual readings the data was also evaluated based on the product of the two classification systems. It was felt that this would tend to normalize the data in cases where the Q was low compared to the RMR and visa versa. Because of the nature of Q it had the greatest influence in this process but the results were interesting. This procedure has not been discussed in the literature and does have the shortcoming of being controlled largely by Q but it does seem to have a leveling effect. Again, histogram plots of this data are presented in Attachment E.3.11.

3.8 CONSTRUCTION CONSIDERATIONS

3.8.1 <u>Excavation</u>

3.8.1.1 <u>General</u> Excavation in the tunnel is to be performed by two different methods under the proposed plan. The majority of excavation will be by TBM with starter tunnels being excavated by drill and blast.

3.8.1.2 <u>Drill and Blast Sections</u> The drill and blast sections in the proposed plan were based on an assumed minimum length of opening at the work shafts for machine assembly and support equipment layout. This length is subject to contractor preferences and therefore only a best guess at this point. The oversized sections at the inlets will, most probably, be excavated with drill and blast along with the section from Work Shaft 2C to the outlet shaft. At Work Shaft 2 a long section of drill and blast was assumed in order to get through the worst of the faulted rock. The advantages of drill and blast through this section are increased flexibility in reacting to bad ground conditions, and the avoidance of risk in getting the TBM stuck in such bad ground. The section from Work Shaft 4 to the tie-in between the Spur and Main Tunnels will probably also be excavated by drill and blast because of improved flexibility in the operation. Because of the size of the tunnels the drill and blast operation is assumed to take place as multiple drifts. Possibly two upper drifts above spring line and one lower drift that may be removed as a bench. The exact final configuration will be left to the contractor's discretion.

3.8.1.3 Machine Boring Issues

3.8.1.3.1 General

Although conventional excavation using drilling and blasting techniques is clearly feasible and would offer flexibility in the case that adverse geologic conditions were encountered, it appears that a TBM is clearly the most economical method to approach the Passaic River Basin Tunnel Project. Even though a hardrock TBM 44½ feet in diameter has not yet been utilized, TBMs 40 feet in diameter have been used in Europe; larger machines are being considered for Taiwan, and it is well within the technology of the manufacturers to produce such a machine. The geologic conditions present along the alignment are also believed to be suitable for this alternative. All the well known positive reasons for TBM excavation, such as increased production, less temporary support, and less concrete lining as the result of reduced overbreak, are present on this project. With such a large project it will be necessary to have several contracts. Under the proposed plan there will be 5 contracts involving tunnel construction, 4 on the Main Tunnel an 1 on the Spur. In the proposed plan 4 different TBMs would be needed. Prosecution of the work would ultimately be controlled by the funding stream.

These three contractual precautions are recommended: a) Pre-qualification of the bidders. b) Placing bid documents in escrow. c) Use of a Contract Disputes Review Board. It is common practice now to include a Geotechnical Design Summary Report or GDSR as part of the bid documents.

3.8.1.3.2 Service Requirements

The TBM's proposed for use on the Passaic Tunnel are larger than any hard rock machines built to date. Naturally the specific design details will be left to the manufacturers but certain requirements are anticipated. They include probe hole capability for drilling ahead of the face, drilling rigs installed on either side of the main beam for rock support installation immediately behind the cutter head and, possibly, a rib-erector system. The cutter heads may be equipped with back-mounted, recessed cutters. Among other things, the specific cutter head design will have to take into account, mixed face conditions, rock hardness, and, in some of the basalts, the closely jointed blocky nature of the rock mass. The muck gathering system may have to accommodate some larger loose blocks that dislodge from the face during boring. Generally a flat faced design is likely to provide greater face stability. Variable speed drive and automatic thrust control are two features that will be highly desirable if not essential on the TBMs to accommodate the wide range of toughness and strength in the rocks to be bored.

Delivery of a TBM of the size being considered is likely to take 15 months. It is advisable that a spare main bearing be available during the life of the contract since replacement of this item could result in a significant loss of time otherwise.

3.8.1.3.3 <u>Production Rates</u> The production rates for the TBM are dependant on the machine penetration rate and utilization. Estimates of the penetration rates discussed below are based on the laboratory testing performed to date and the jointing characteristics of the rock mass. Figure E.3.15 shows the estimated TBM penetration rate versus the Unconfined Compressive Strength based on the experience of the Robbins Co. and Atlas Copco, Jarva. This is only a first approximation of the instantaneous penetration rate for the intact material. The degree of jointing and bedding also has a tremendous influence on the penetration rate. The method of computation of daily production rates assuming a homogeneous face condition for a TBM is summarized below.

The instantaneous penetration rates for a TBM are calculated as follows:

$$\Sigma = \Omega \div (\pi \times D) \times \delta$$

Where: Σ = **instantaneous penetration rate per hour**
Ω = **maximum head speed.**
(This is limited by the capacity of the cutting disk

bearings. They are subjected to tremendous pressure and should not travel much faster than 600 feet per minute or the bearings will be damaged. Therefore the maximum RPMs for a machine may be calculated by dividing Ω by the circumference of the TBM.)

π = 3.1416
D = diameter of cutter head
δ = penetration per revolution

The penetration per revolution is the key variable in this relationship and is dependant upon numerous factors relating to both the rock and the machine. The cutting disk size and thrust per cutter are two obvious machine factors. The main lithologic factors include unconfined and tensile strength, toughness, and degree of jointing. At this stage the Unconfined Compressive Strength is used as a index for estimating the penetration rate. Using the upper bound values of the strength data for the sedimentary materials, the average penetration rate should range between 7 feet per hour for machines equipped with 17 inch diameter cutters and 13 feet per hour for 19 inch cutters. Assuming an average strength of around 24,000 PSI for the basalt, the penetration rate using 17 inch cutters is 3 feet per hour and 5 feet per hour for 19 inch cutters. The new 19 inch cutter will have decided advantages when working in basalt. Although this size cutter has had limited use to date, it is believed that within the next five years large machines with 19 inch cutters will be readily available. Because of the low dips much of the excavation will be in mixed face conditions.

Several factors affect machine utilization. Case histories show that since there is more time to perform associated tasks, the lower the penetration rate the higher the utilization. A decrease in utilization always occurs when there is need for extra support in fault and shear zones, when there are large water inflows, through reaches of highly abrasive formation, or when there are major machine breakdowns. Muck gathering capability is another important consideration when estimating utilization and production rates. It is possible, especially in the most "boreable" rocks with a machine of this size, for the muck production to exceed the capability of the muck gathering and transport system. This "muck bound" condition, indicated on figure E.3.15 is a limiting case on the machine production. Experience has shown that the maximum mucking capacity for a 45 foot diameter machine is slightly over 11 feet per hour but it is believed that with some modifications about 15 feet per hour is achievable.

If a TBM with 17 inch cutters is operating in sedimentary rock on the Passaic Project, a penetration rate of 7 feet per hour with a utilization of 50% will result in an advance estimated at 89 feet per day. If 19 inch cutters are available, an advance rate of 13 feet per hour is possible, however, utilization would decrease. Assuming a utilization rate of 35%, one could expect an advance of about 109 feet per day. When operating in basalt the penetration rate will be slower but the utilization rate will be higher. If one uses 3 feet per hour penetration rate with a 70% utilization, it is estimated the advance will be about 47 feet per day for a machine using 17 inch cutters. If 19 inch cutters are available, an advance rate in excess of 67 feet per day may be possible. The cost estimates are based on the lesser advance rates.

When boring in mixed face conditions, where there is a marked difference in the strength of material being excavated, the production is controlled by the harder or more resistant material. If the operator attempts to force through the mixed conditions there will be excessive vibration as the cutter disks jump from soft to hard materials. This will also result in excessive wear and chipping of the disks, and extra stress on the entire system. All of these factors limit the production rate through mixed face so for the purposes of the production and cost estimate any portion of the tunnel that had any "hard rock" in it was assumed to be excavated as if it were all hard rock. Only the dense basalts are considered hard rock in the estimate. Amygdaloidal, vesicular, pillow, and weathered basalt are considered soft rock that could be excavated at the same rate as the sedimentary rocks. From the standpoint of penetration and production rates all of the sediments are considered equal for this estimate even though some of them will undoubtedly cut quicker than others. The table in Appendix E.3.11 shows how the production estimate was formulated and presents the reaches of tunnel that fall in each rock category divided out by contract.

3.8.1.3.4 Cutter Wear Estimates Very little work has been done so far to predict cutter wear so conservative estimates of cutter wear have been used in the cost estimate. Many of the rocks to be excavated have a high silica content and, based on the thin section analysis, much of this silica is sharp. This suggests that these rocks will be very abrasive.

3.8.2 Muck

3.8.2.1 General The tunnel muck produced during construction will generally fall into two categories, blasted muck and TBM derived muck. Only a very small percentage of the tunnel is to be driven by drill and blast methods.

3.8.2.2 Nature of Materials The muck produced by the drill and blast operations is likely to contain a wide range of particle sizes with the coarse fraction dominating. The precise size distribution will be dependant upon the nature of the rock being shot and the layout of the blasting pattern. The muck produced by the TBMs will be comprised of a very fine fraction mixed with flat and elongate fragments. The gradation is dependant upon the rock type being bored but normally the higher the penetration rate the larger the flake dimensions. Figure E.3.16 was taken from a promotional pamphlet distributed by WIRTH, a German TBM manufacturer. It gives a rough idea of the range of particle sizes that can be expected from a TBM. The fine fraction results from crushing of the material under the cutter disks. The balance of the material removed flakes off between the grooves cut by the disks. The width of the flakes is dependant on the cutter spacing and the thickness is dependant on the penetration rate.

3.8.2.3 Methods of Removal It is most likely that a horizontal tunnel and vertical shaft conveyor system would be used in conjunction with the TBM driven tunnel for muck removal. Use of other haulage methods will not be excluded by the specifications however other methods will probably not be able to keep pace with the muck production of the machine in the more boreable rocks. Some recent experience indicates that about 5% improvement in TBM utilization can be realized when conveyors are used instead of mucking trains. Muck removal in the drill and blast sections will probably be by loaders and possibly some rubber tired vehicles. As mentioned previously this does not represent a very large percentage of the tunneling.

3.8.2.4 Surface Transportation This issue can not be completely resolved until construction begins because of uncertainty in the long term availability of different transportation systems. Depending on the shaft location muck will be transported from the site either by train, truck, or barge.

3.8.2.5 Potential Usage Refer to Section 16, Appendix E, for a full discussion of the disposal of the tunnel muck. Adequate disposed areas have been located for the tunnel muck within 10 miles of the workshaft. All muck is suitable for use as levee fill, engineered fill, and quarry fill. The basalt tunnel muck could be processed into concrete or asphalt aggregate or road base material.

3.8.3 Initial Rock Support

3.8.3.1 Design and General Considerations

The initial support of the tunnel opening is that which is installed prior to placement of the liner. It will consist primarily of rock bolts on regular spacings installed in the crown of the tunnel approximately 30 degrees above the spring line. The spacing of the bolts will be dependent upon the quality of the rock in the crown and will be supplemented with spot bolting, welded wire mesh, and strapping. In cases where rock bolts are not sufficient to support the rock, for example in faulted areas, 8 x 48 steel ribs on four foot centers will be used. Typical section of the various initial support layouts for both the main tunnel and the spur are shown on Plates E.3.1 and E.3.2. Shotcrete is not generally regarded as compatible with Tunnel Boring Machines because of the amount of dust and latence that it produces and because it can not be applied very close to the face. Recent advances in the technology have lead to some contractors using shotcrete with TBMs but it is still not in wide usage. It is likely to be very useful in the portions of the tunnels that are driven using Drill and Blast methods. Here it is anticipated that steel fiber reinforced shotcrete will be used as part of the initial support system before the final lining is formed and poured.

The initial rock support requirements have been selected by using the rock mass classification systems. The rock support category is based on the rock mass classification value and an equivalent dimension of the excavation. The equivalent dimension is a function of the span and purpose of the opening. For example a temporary mine opening would not require the same amount of support as a permanent water tunnel. Based on the literature the span of the opening should be divided by a factor called the Engineering Support Ratio or ESR to derive an appropriate equivalent dimension. The ESR is based on the proposed use of the opening. In the case of the Passaic tunnel an ESR of 1.3 was used which is pertinent for storage caverns, water treatment plants, minor highway and railroad tunnels, surge chambers, and access tunnels. A higher ESR of 1.6 might also be appropriate since the larger tunnels are to excavated with a TBM. Ranges of Q and RMR, for the appropriate equivalent dimension, were selected based on recommendations from the consultant group and from the literature for each support class of rock. Some modification of the support classes described in the literature was necessary because of the size of this tunnel. As a minimum it was assumed that pattern bolting on five foot centers would be needed. For poorer quality rock classes this spacing would be tightened up to 4 feet. This support was supplemented with welded wire mesh, strapping, and steel ribs based on the tunnel class. In all, seven classes of rock were used to characterize the mass and each class had a corresponding initial support design. Instead of trying to characterize specific tunnel reaches which would require specific support treatment the data was evaluated statistically. The percentage of data in each Q or RMR class range was assumed to be equivalent to an equal percentage of the total footage of tunnel to be bored in each formation per contract. In this way, for each contract the total footage requiring support class I, II, III etc. treatment could be figured without specifying precisely what reach of tunnel this corresponded to. It was not felt to be realistic to be this specific since normal practice is to evaluate support requirements as the tunnel is bored. Being too specific based on the limited data available will very likely result in differing site conditions claims. This method just provides a systematic, supportable approach to estimate the amount of each class that can be expected overall. The limits of the different support classes for Q and RMR by contract and the design description of these classes is presented in Attachment E.3.12.

3.8.3.2 <u>Rock Bolts</u> Rock bolts are to be the primary means of initial support in the tunnel. They are to be installed immediately behind the head of the TBM using drill rigs installed especially for this purpose. Several different types of rock bolts are available and each provides specific advantages and disadvantages. Swellex® and split set bolts belong to the broad category known as friction anchored bolts. The disadvantages of these bolts are they are not regarded as long lived as solid bar reinforcement, they will normally require corrosion protection for permanent installations, the length of bolt that can be installed is limited, and their material cost is relatively high. For split sets the hole diameter is very critical. Their advantages are they are more able to deform with the rock mass and still retain their strength, and they are quick and relatively cheap to install. The use of Swellex® bolts or split sets has not been encouraged in the Corps of Engineers. Resin encapsulated bolts provide the advantages of quick installation, adaptability, and resistance to corrosion. Their disadvantages include, high cost, occasional problems with anchorage reliability, and resin can be messy and toxic without adequate ventilation. Cement grouted bolts using mechanical anchors and Hollow Core® bolts are another option for support. Their disadvantage is that they are more difficult to install, and they do not develop their maximum support as quickly as the other systems described. For the purposes of the cost estimate the resin encapsulated bolts were used. This will put the cost on the high side so that savings may be achieved during the next phase if some less expensive type of bolt is deemed acceptable at that time.

3.8.3.3 Strapping Strapping is to consist of 12 to 124 gage steel plate deformed as shown on Plates E.3.1 and E.3.2. This is to provide necessary support between the rock bolts as needed. The key is to hold intermediate loose blocks of rock in place so that the self supporting rock arch can form. This supplemental support is normally fabricated specifically for a job. It must be installed so that it is contact with the rock to be effective. Many experts prefer curved, light sections of channel or I-beam to strapping.

3.8.3.4 Welded Wire Mesh This material is to be used to prevent small rock falls from occurring between the bolted and strapped material. Even a small rock constitutes a serious threat to worker safety in a tunnel of this size. The estimate is based on welded wire mesh with a 4 inch square opening. Welded wire is preferred over chain link mesh because it will not unravel if a strand is severed. The mesh will be held in place with the rock bolts and intermediate short pins as needed. The mesh is to be installed over the upper 90 degrees of the tunnel as needed based on the frequency of jointing and bedding. Mesh on the side walls and invert is not thought to be necessary.

3.8.3.5 Steel Ribs Steel ribs will be needed primarily in the faulted areas where the disturbed rock will not allow bolting to develop adequate strength. The ribs will be installed in segments immediately behind the TBM head. Some type of lagging between the ribs will be needed. Mats of welded rebars (Melbourne type) are beginning to take the place of convetional wooden lagging in the U.S. and offer several advantages. For the cost estimate it was assumed that 8" ribs weighing 48 pounds per foot would be installed on 4 foot centers through the bad ground. The ribs would eventually be encapsulated in the final concrete liner.

3.8.4 Tunnel Lining A detailed discussion of the tunnel liner design is presented in Appendix G, Structural Design. Initially three alternative treatments were considered, unlined, pre-cast-segmental, and cast-in-place. Because of environmental and longevity considerations the unlined option was eliminated early in the study even though it is the cheapest alternative. Based on its more desirable hydraulic characteristics and the recommendations of the consultant group, a cast-in-place liner was assumed to be most appropriate at this stage of design. The option to use a precast-segmental liner should be an option that is left to the contractors, possibly as a bidding option. The cast-in-place liner was designed to withstand full hydrostatic head assuming groundwater elevation equal to the top of ground elevation. To accommodate the variation in this hydrostatic pressure the strength of the concrete was varied and the liner thickness was held constant at 15". Under the proposed plan the liner within a given contract reach would not be placed until the excavation in that reach is completed.

3.8.5 <u>Water Inflow and Control</u>

A critical issue for the successful completion of tunneling is the control of water inflow. This is of importance from both an operational and environmental standpoint. The results of the groundwater study indicate that some water bearing zones are present which would produce enough flow to stop tunneling operations. In addition, the drawdown of water levels within these zones caused by unobstructed flow into the tunnel is environmentally unacceptable. For these reasons it will be necessary to anticipate and treat these high yield water bearing zones before encountering them with the excavation.

In order to anticipate areas of heavy inflow the TBMs will be fitted with the capability to drill and probe hole in advance of the drive as described in paragraph 3.8.1.3.2. Indication of high flow from the probe hole will initiate a grouting operation by drilling a series of holes in advance of the TBM and pre-grouting the rock mass before boring through it. A successful grouting program will be much easier before making the tunnel opening, however this will result in keeping the TBM idle while the grouting is performed. This adds very significantly to the cost of tunneling. The added costs related to grouting have been based on the analysis described earlier.

Panning may be used for localized seeps as necessary to allow for placement of the concrete tunnel liner as described in paragraph 3.8.4. After liner placement is complete, consolidation grouting of the rock behind the liner will be performed in the more pervious zones. Finally contact grouting through the liner is performed to ensure good contact between the liner and the surrounding rock.

3.8.6 <u>Ventilation and Dust Control</u> Adequate ventilation of any underground works is strictly controlled by OSHA regulation. The contract will require that all pertinent regulations are met. Driving the tunnel with TBMs produces a lot of dust. Since much of the rock to be bored has a high silica content this dust poses a serious medical threat to workers if it is not adequately controlled. Dust suppression systems using water are commonly used but will be left to the contractor's discretion so long as a system which meets the minimum requirements is utilized. The potential for medical problems inherent in silica dust will be addressed in the contract.

3.8.7 Potential for Encountering Gassy Conditions

This issue is related to ventilation but constitutes a separate, and potentially serious condition. There were indications during the exploration work that hydrocarbons are naturally occurring in some of this rock. Strong hydrocarbon odor and "crude oil" seepage was noted on the log for boring C-130 at a depth of 280 to 290 feet in the Towaco Formation. The possibility of gassy ground is obviously in this formation and should be strongly suspected in all of the Jurassic sedimentary rocks which contain dark grey beds with high organic content. Further evaluation of the red-beds and basalts is necessary to reach any conclusions regarding gassy ground while excavating in them. However monitoring will be a requirement for all of the tunneling work.

Monitoring during the shaft sinking is also a wise precaution. Some of the overburden may be contaminated to the point where gassy conditions could pose a threat to worker health and safety.

REFERENCES

American Society of Civil Engineers, 1989, "Avoid and Resolving Disputes in Underground Construction, Successful Practices and Guidelines", ASCE Publication, pp 1-24 with 3 appendicies

Bieneawski, Z.T., 1989, "Engineering Rock Mass Classifications", John Wiley & Sons, pp 1-251

Bowcock, J. B., Boyd, J. M., Hoek, E., and Sharp, J. C., 1977, "Drakensberg Pumped Storage Scheem - Rock Engineering Aspects.", Proceedings Symposium Exploration for Rock Engineering, Z.T. Bieniawski, editor, A.A. Balkema, Rotterdam, Volume 2, pp 121-139

Brekke, T.L., Heuer, R.E., and Korbin, G.E., 1990, "Passaic River Diversion Tunnel Project, New Jersey", Final Report, Phase A, Jul., U. S. Army Corps of Engineers, Nashville District, Nashville, TN, 20 pp.

Broch, E. and Franklin, J.A., 1972, "The Point Load Strength Test", International Journal of Rock Mechanics, , Min. Sci. Vol 9, Pergamon Press, pp 669-697

Broch, E., Franklin, J.A., and Walton, G., 1971, "Logging the Mechanical Character of Rock.", Transactions/Section A of the Institution of Mining and Metalurgy, Vol 80, pp A1-A9

Buro, M. R., May, 1970, "Prestressed Anchors and Shotcrete for Large Underground Powerhouse.", Civil Engineering, ASCE, Volume 40, No. 5, pp 60-64

East, J. H. and Gardner, E. D., 1964, "Oil SHale Mining, Rifle, Colorado, 1944-56.", U.S. Bureau of Mines, Bulletin 611, pp 1-163

Endersbee, L. A. and Hofto, E. O., 1963, "Civil Engineering Design and Studies in Rock Mechanics for Poatina Underground Power Station, Tasmania.", Journal of the Institution of Engineers, Austrailia, Volume 35, pp 187-209

Faulst, George T., 1978, "Joint Systems in the Watchung Basalt Flows, New Jersey", US Geological Survey Professional Paper 864-B, US Government Printing Office, Washington D.C., 45 pp.

Hoek, E. and Bray, J., 1977, "Rock Slope Engineering", Revised 2nd Edition, The Institution of Mining and Metalurgy, London, pp 1-402

Hoek, E., and Brown, E.T., 1980, "Underground Excavations in Rock", The Institution of Mining and Metalurgy, London, pp 1-527

International Society of Rock Mechanics, June 10, 1984, "Suggested Method for Determining Point Load Strength", pp 53-60.

Imprie, A. S. and Jory, L. T., 1968, "Behavior of the Underground Powerhouse Arch at the W.A.C. Bennett Dam During Excavation.", Proceedings, 5th Canadian Rock Mechanics Symposium, Toronto, pp 19-39

Kaiser, P. K., and Hutchinson, D. E., 1982, "Effects of Construction Proceedure on Tunnel Performance", Proceedings of the Fourth International Conference on Numerical Methods in Geomechanics, Edmonton, Canada, pp 561-569

Kimmons, G. H., Mar-April, 1972, "Pumped Storage Plant at Racoon Mountain in U.S.A.", Tunnels and Tunneling, Volume 4, Number 2, pp 108-113

University of Trondheim, The Norwegian Institute of Technology, The Division of Construction Engineering, October, 1988, "Hard Rock Tunneling, Project Report I-88", pp

U.S. Army Corps of Engineers, 1987, "Flood Protection Feasibility Main Stem Passaic River", Phase I, General Design Memorandum, Part III, Dec., U.S. Army Corps of Engineers, New York District, NY, 63 pp.

U.S. Army Corps of Engineers, 1982, "Park River Local Protection Project, As-Built Foundation Report, Auxiliary Conduit Tunnel", Volumes I and II, U.S. Army Corps of Engineers New England Division.

Stillborg, B., 1986, "Professional Users Handbook for Rock Bolting", Trans Tech Publications, Series on Rock and Soil Mechanics, Vol 15, pp 1-145

Serafim, J. L. and Periera, J. P., 1983, "Considerations on the Geomechanical Classification of Bieniawski", International Symposium on Engineering Geology and Underground Construction, Proceedings, Volume 1, pp II.33-II.42

Toolanen, B., Hartwig, S., and Janzon, H., "Design Considerations for Large Hard Rock TBMs When Used in Bad Ground", Atlas Copco Mechanical Rock Excavation AB, Stockholm, Sweden. 16 pp.

Toombs, A. F., Snowdon, R. A., and O'Reilly, M.P., 1976, "The Particle-Size Distributions of Debris Produced During Tunneling Trials", Transport and Road Research Laboratory, Department of the Environment, TRRL Report # 714, pp 1-24

Van Houten, F.B., 1969, "Late Triassic Newark Group, North Central New Jersey and Adjacent Pennsylvania and New York", <u>Geology of Selected Areas in New Jersey and Eastern Pennsylvania and Guidebook of Excursions</u>, Subitzky, S., ed., The Geological Society of America, Rutgers University Press, New Brunswick, NJ, pp. 314-347.

Various, October 1978, "Analysis of Tunnel Stability by the Convergence-Confinement Method", Proceedings AFTES Conference, Paris, Underground Space, Volume 4, Number 6, pp 361-402

Wijk, Gunnar, 1980, "The Point Load Test for the Tensile Strength of Rock", Geotechnical Testing Journal, GTJODJ, Vol. 3, No. 2, pp 49-54.

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EM 1110-2-2907	ROCK REINFORCEMENT
EM 1110-2-4300	INSTRUMENTATION FOR CONCRETE STRUCTURES

TECHNICAL MANUALS (NAVFAC)

TM 5-818-4	BACKFILL FOR SUBSURFACE STRUCTURES
TM 5-818-4	DEWATERING GROUNDWATER CONTROL

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

APPENDIX E

SECTION 3

FIGURES

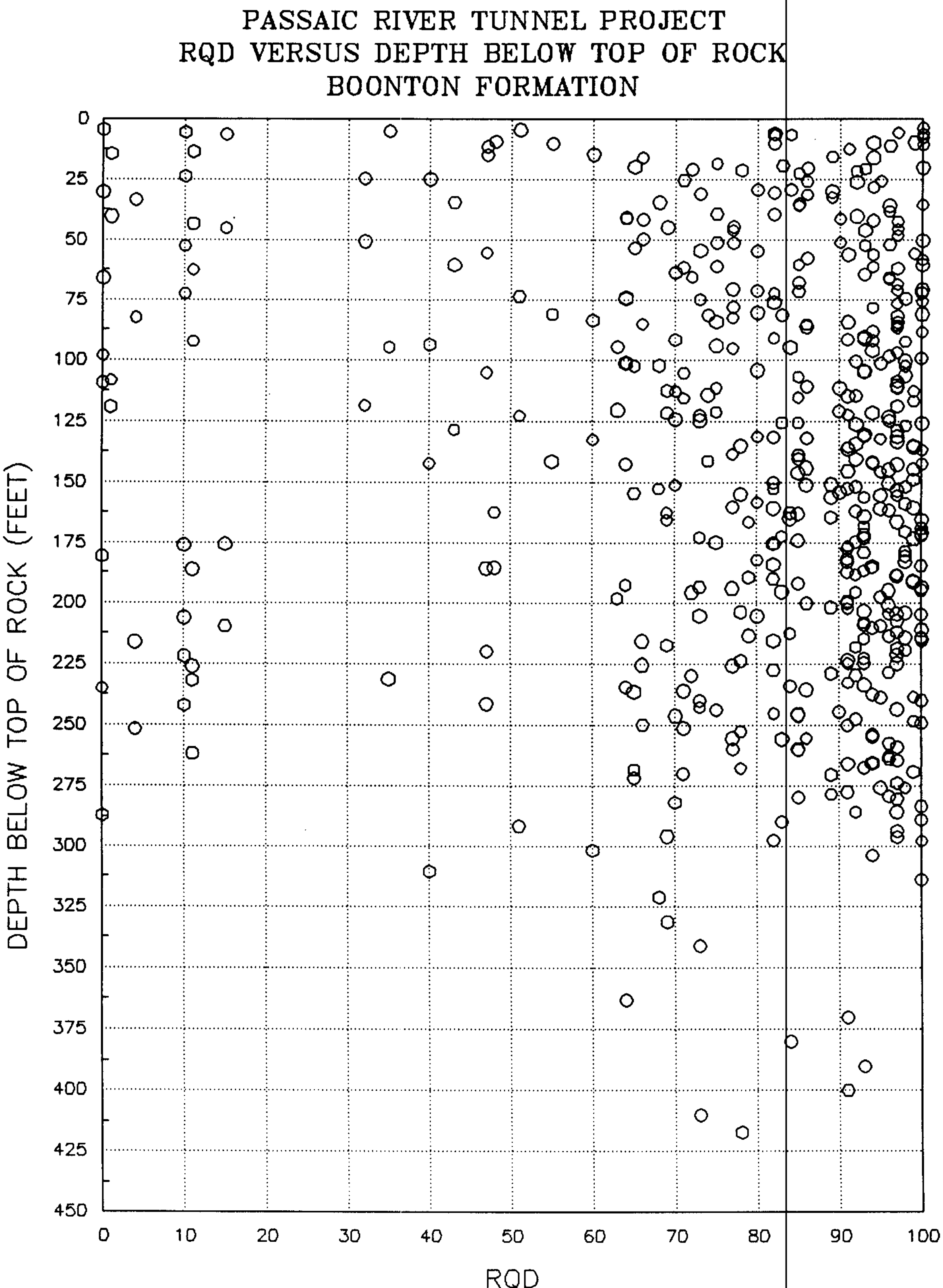

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
BOONTON FORMATION
DEPTH BELOW TOP OF ROCK (FEET)
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
450
0
10
20
30
40
50
60
70
80
90
100
RQD

Figure E.3.1

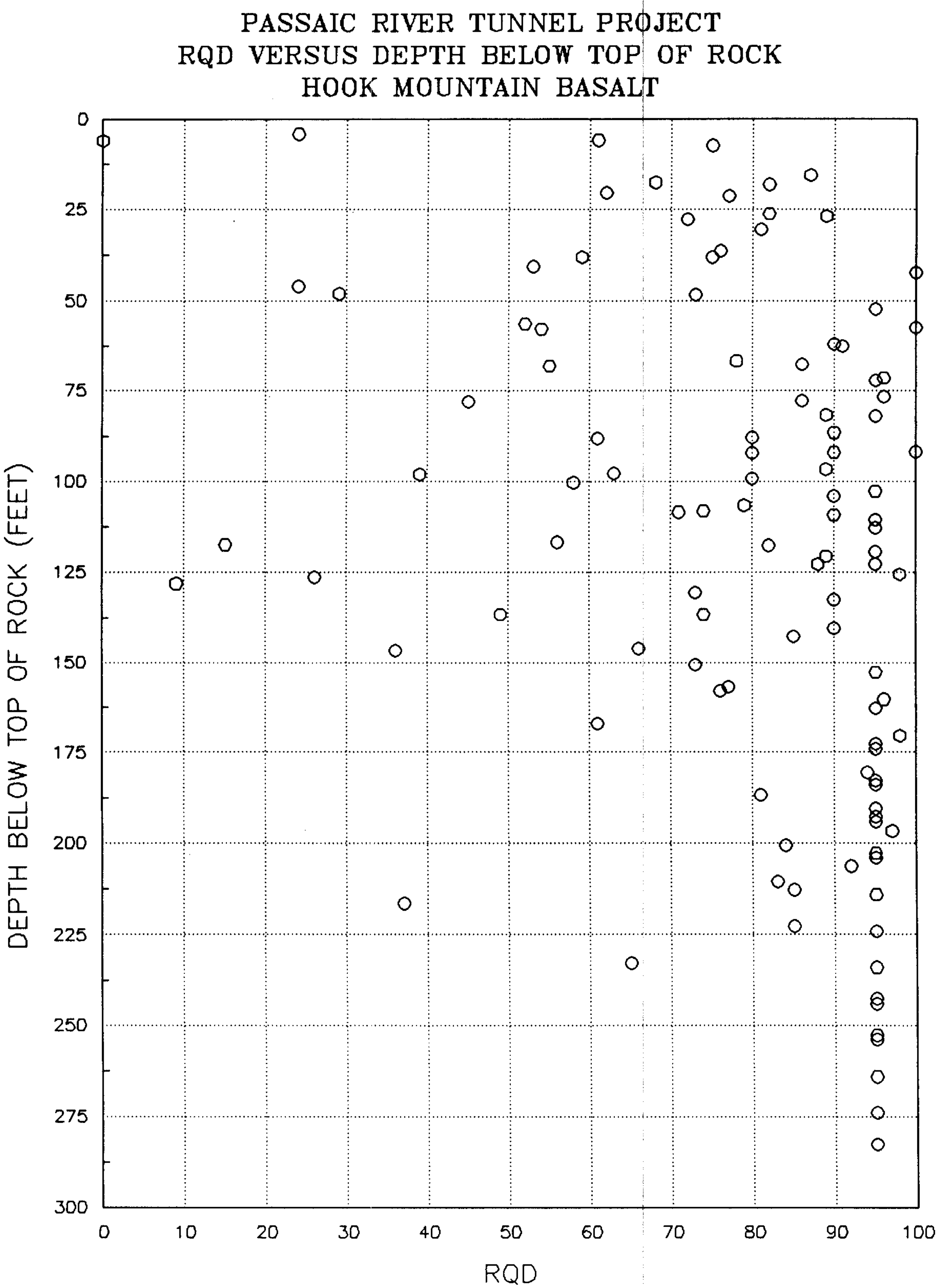

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
HOOK MOUNTAIN BASALT
DEPTH BELOW TOP OF ROCK (FEET)
RQD
0
25
50
75
100
125
150
175
200
225
250
275
300
0
10
20
30
40
50
60
70
80
90
100

Figure E.3.2

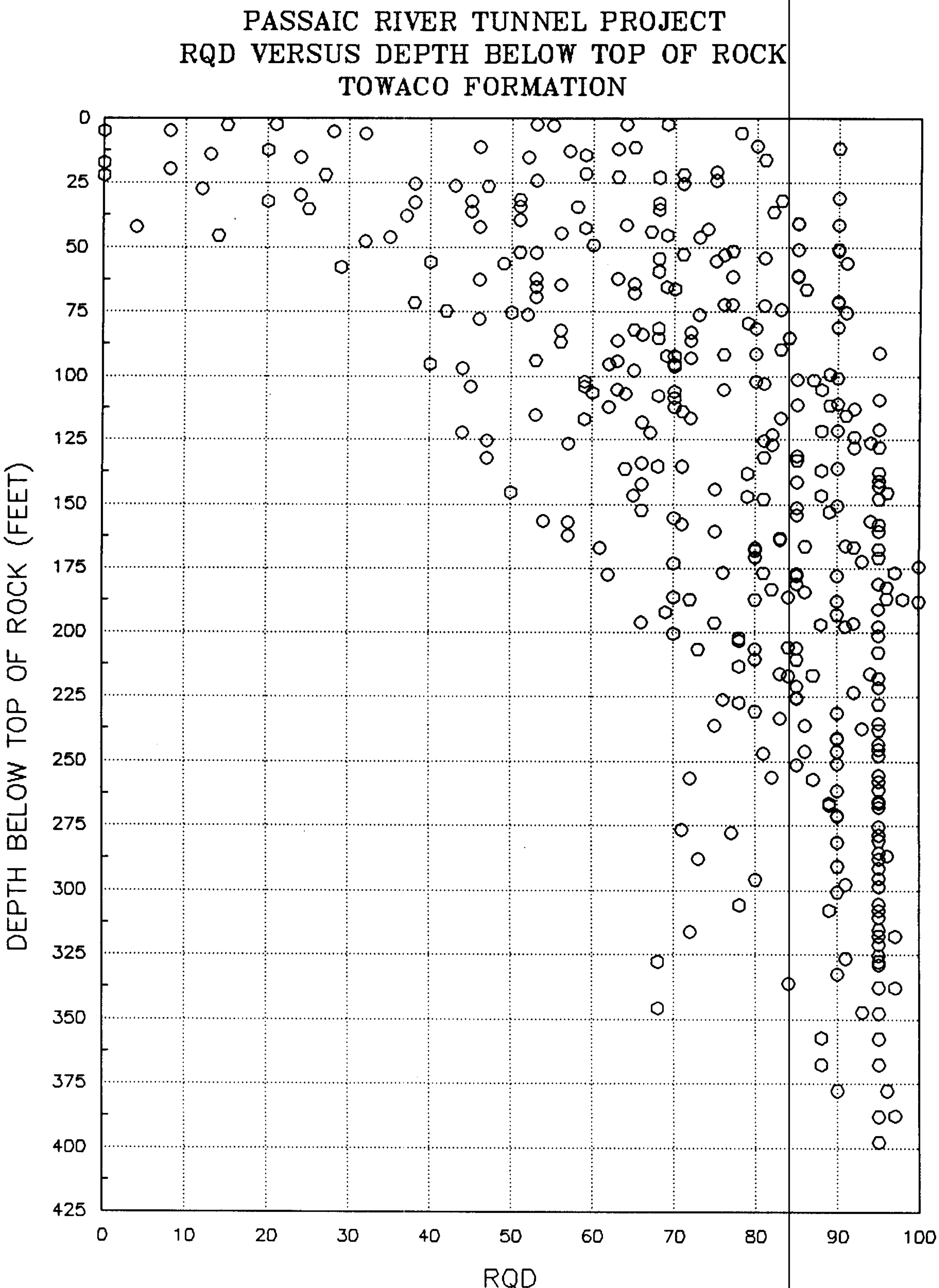

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
TOWACO FORMATION
DEPTH BELOW TOP OF ROCK (FEET)
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
0
10
20
30
40
50
60
70
80
90
100
RQD

Figure E.3.3

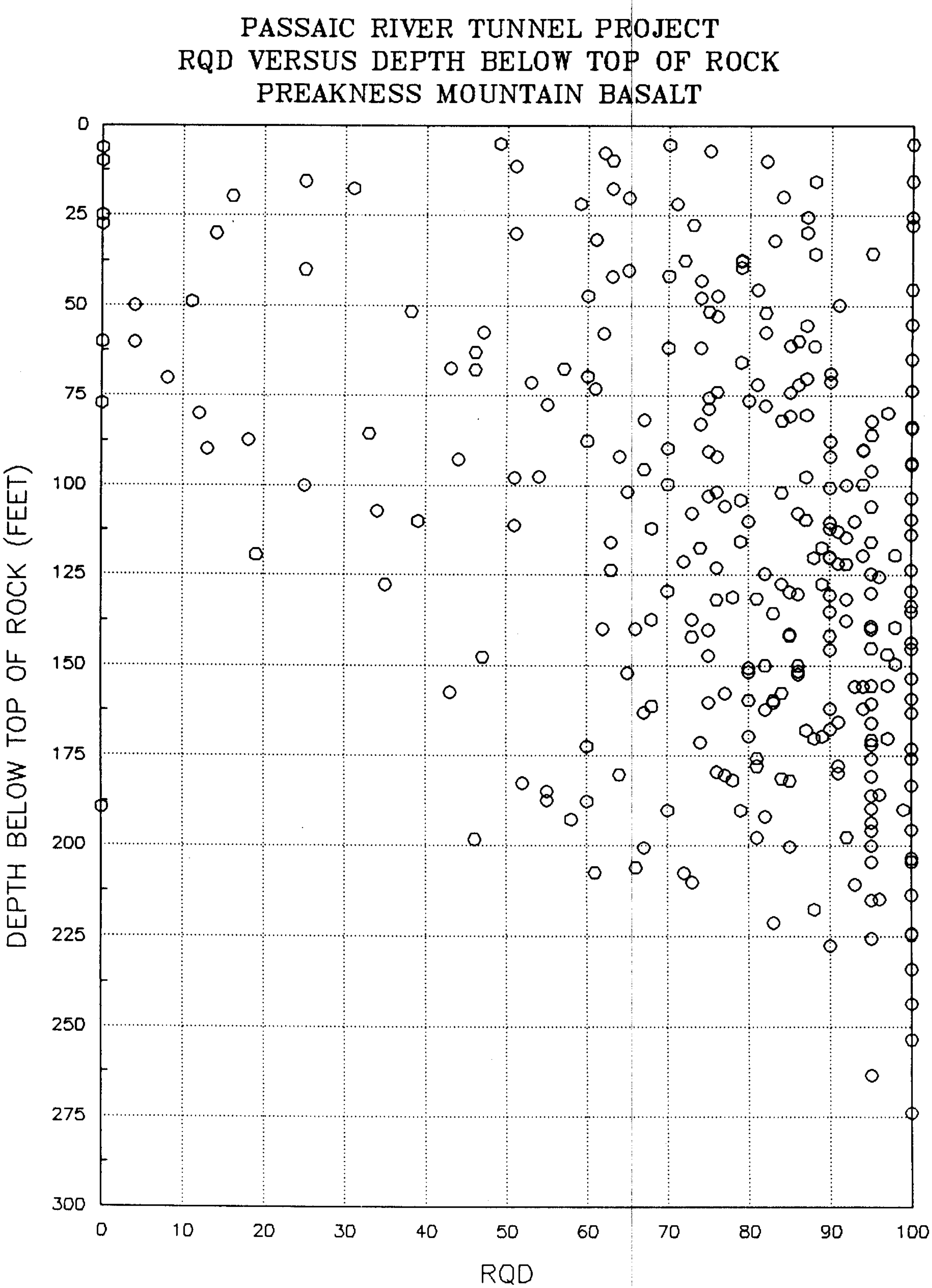

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
PREAKNESS MOUNTAIN BASALT
DEPTH BELOW TOP OF ROCK (FEET)
0
25
50
75
100
125
150
175
200
225
250
275
300
RQD
0
10
20
30
40
50
60
70
80
90
100

Figure E.3.4

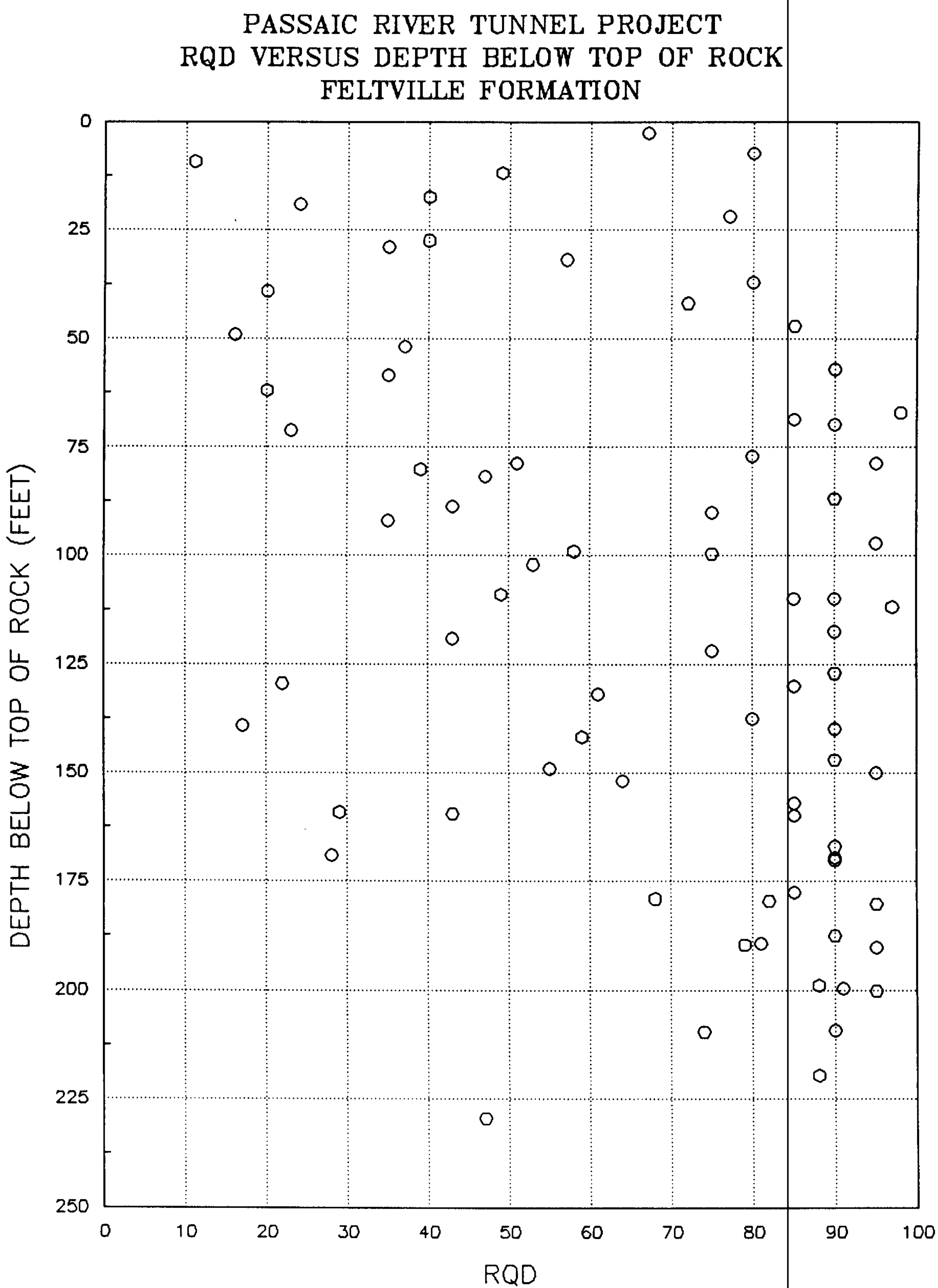

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
FELTVILLE FORMATION
DEPTH BELOW TOP OF ROCK (FEET)
0
25
50
75
100
125
150
175
200
225
250
0
10
20
30
40
50
60
70
80
90
100
RQD

Figure E.3.5

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
ORANGE MOUNTAIN BASALT

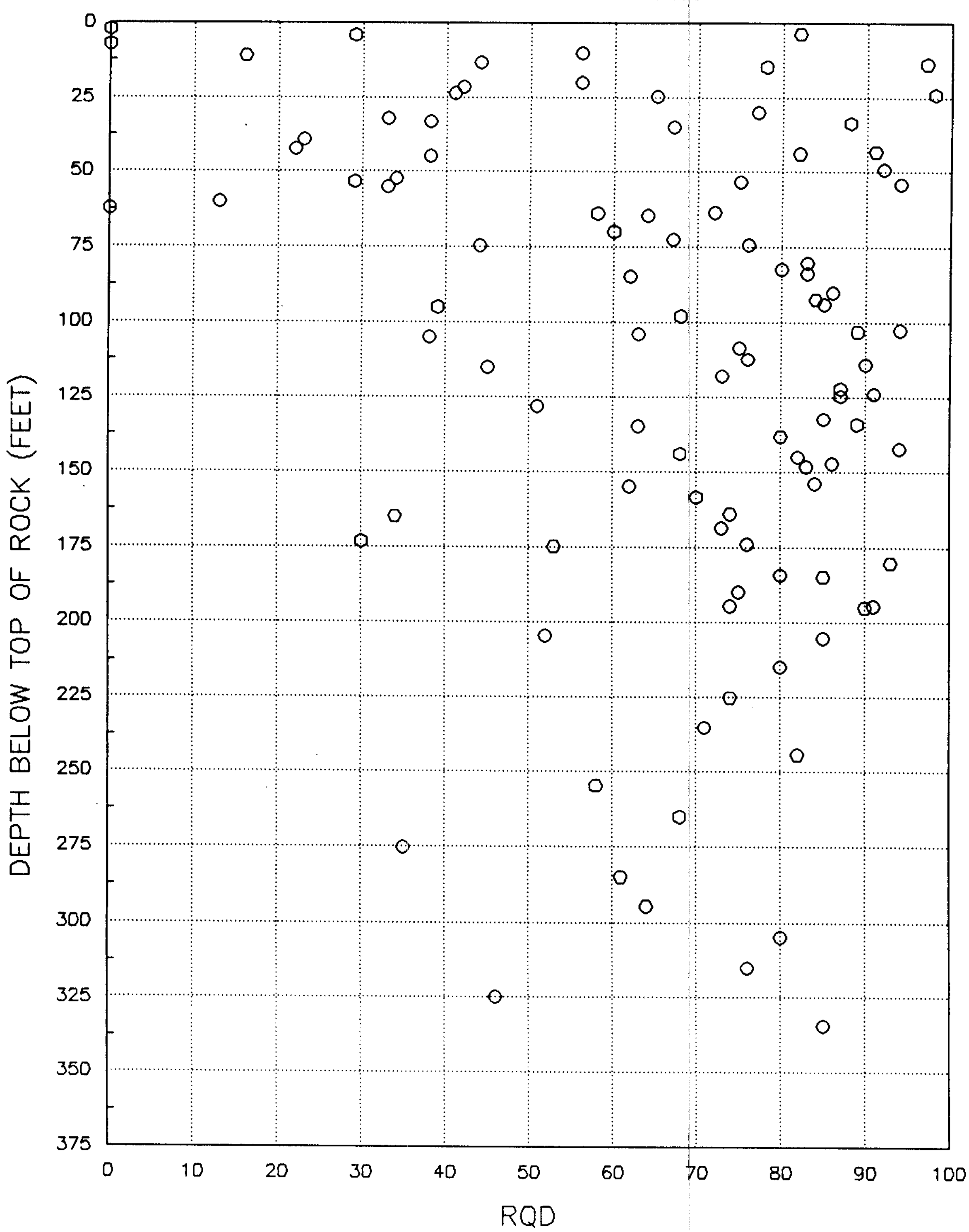


Figure E.3.6

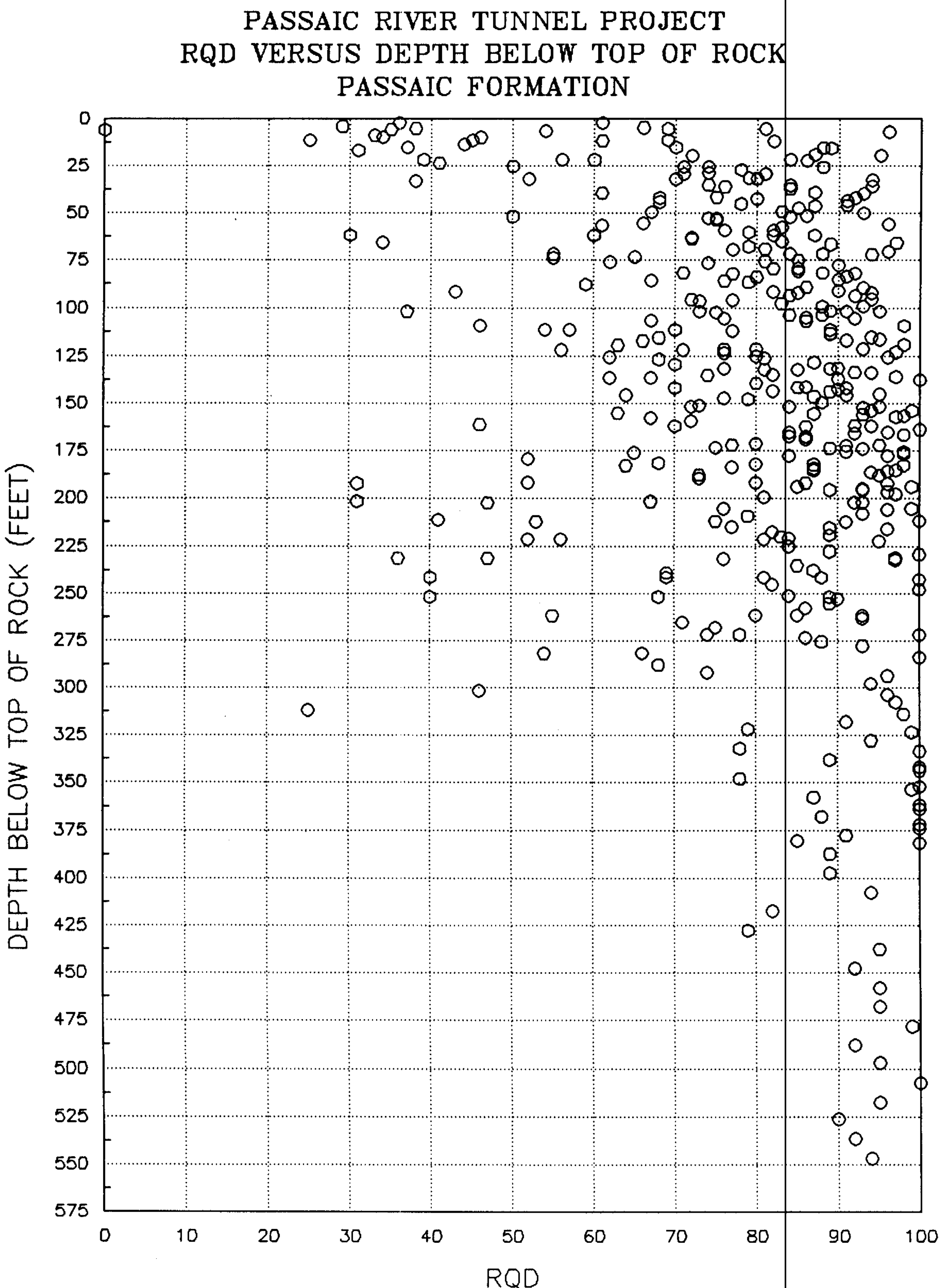

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
PASSAIC FORMATION
DEPTH BELOW TOP OF ROCK (FEET)
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
450
475
500
525
550
575
0
10
20
30
40
50
60
70
80
90
100
RQD

Figure E.3.7

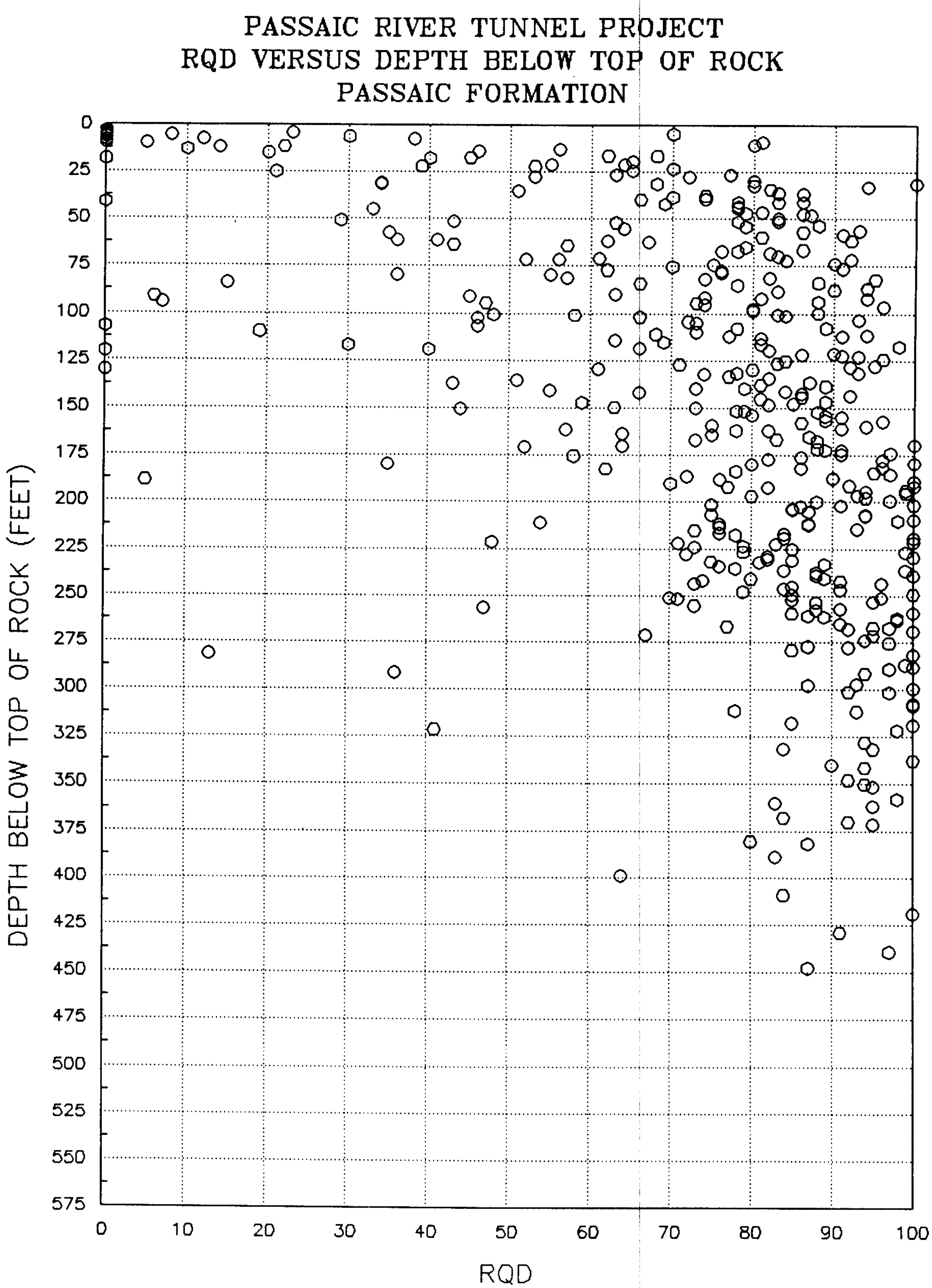
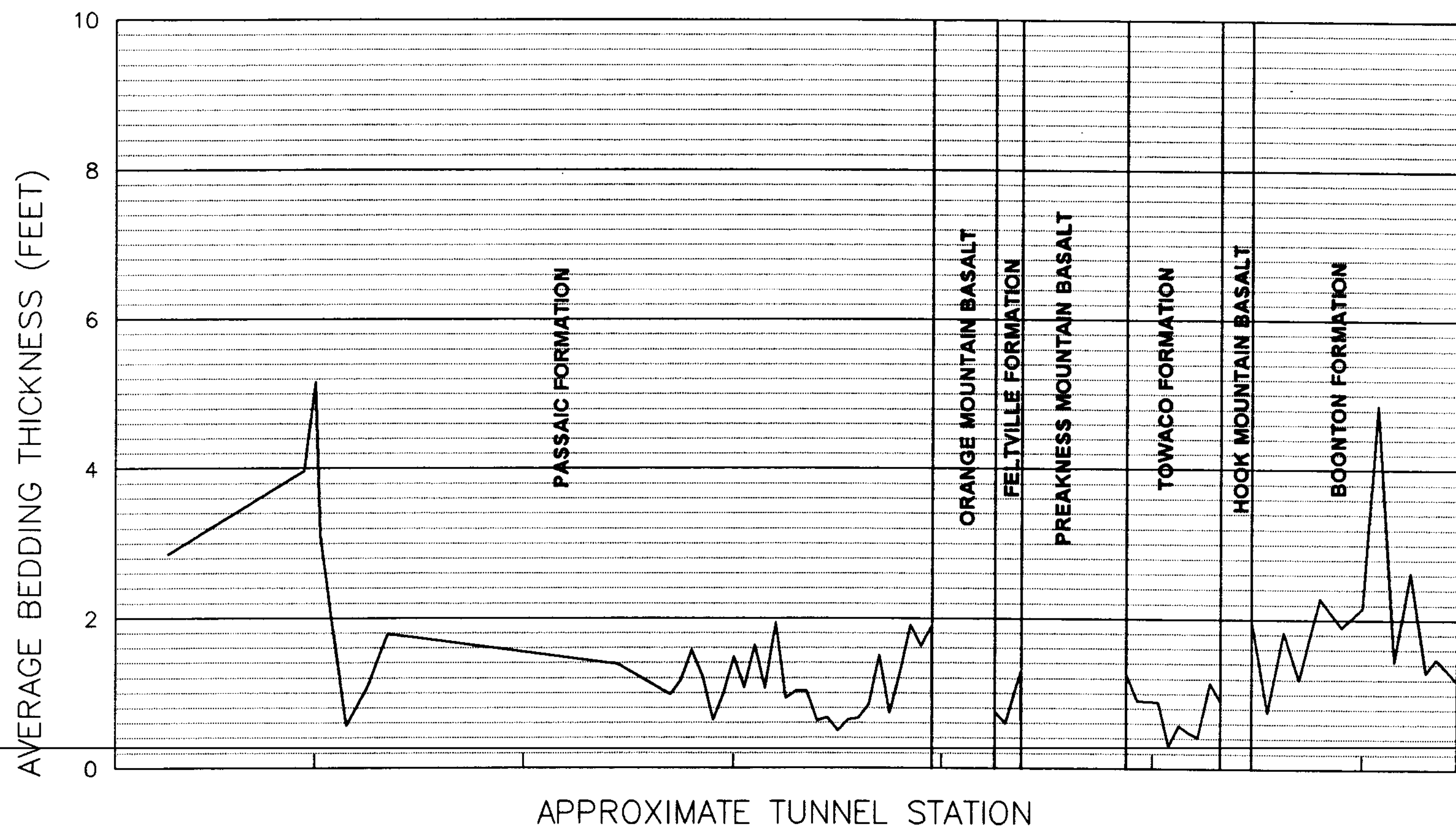

PASSAIC RIVER TUNNEL PROJECT
RQD VERSUS DEPTH BELOW TOP OF ROCK
PASSAIC FORMATION
DEPTH BELOW TOP OF ROCK (FEET)
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
400
425
450
475
500
525
550
575
0
10
20
30
40
50
60
70
80
90
100
RQD

Figure E.3.8

PASSAIC TUNNEL PROJECT
APPROXIMATE AVERAGE BEDDING THICKNESS IN FEET



The average bedding thickness was computed by dividing the total core length by the number of open bedding planes encountered.

Figure E.3.9

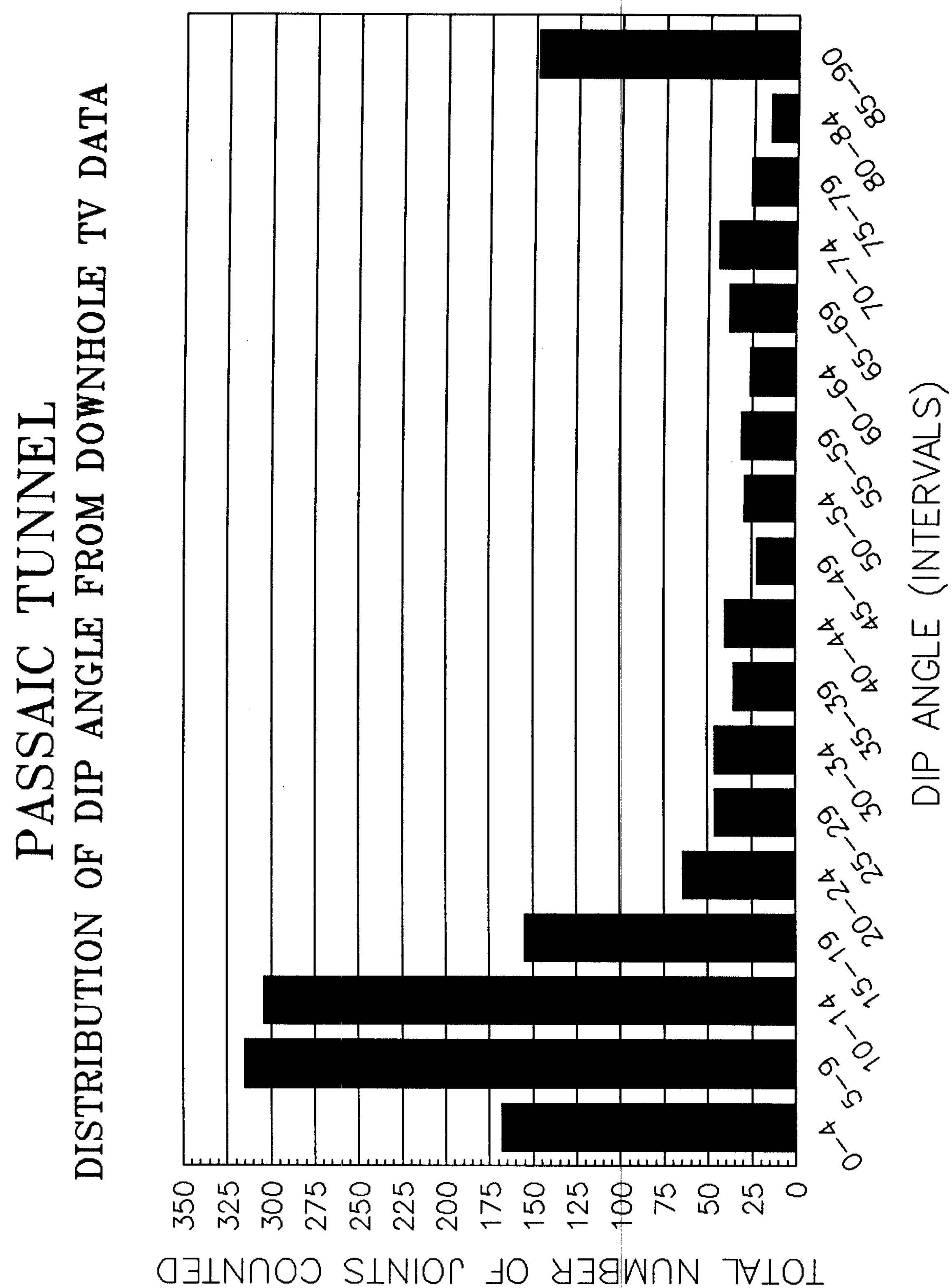
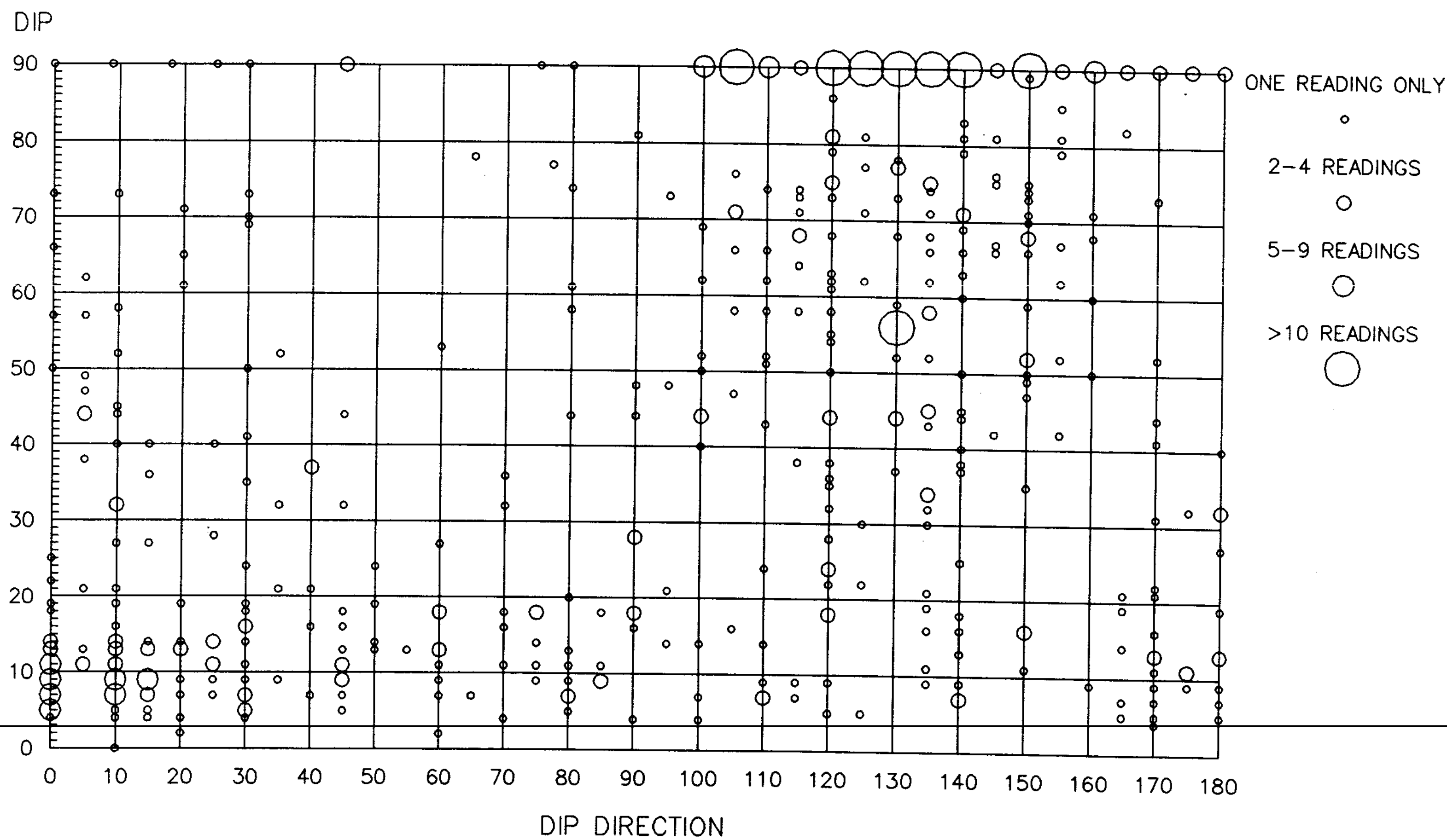

PASSAIC TUNNEL
DISTRIBUTION OF DIP ANGLE FROM DOWNHOLE TV DATA
TOTAL NUMBER OF JOINTS COUNTED
350
325
300
275
250
225
200
175
150
125
100
75
50
25
0
DIP ANGLE (INTERVALS)
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
50-54
55-59
60-64
65-69
70-74
75-79
80-84
85-90

Figure E.3.10

PASSAIC TUNNEL PROJECT
DISCONTINUITY DATA FROM BOREHOLE TV CAMERA

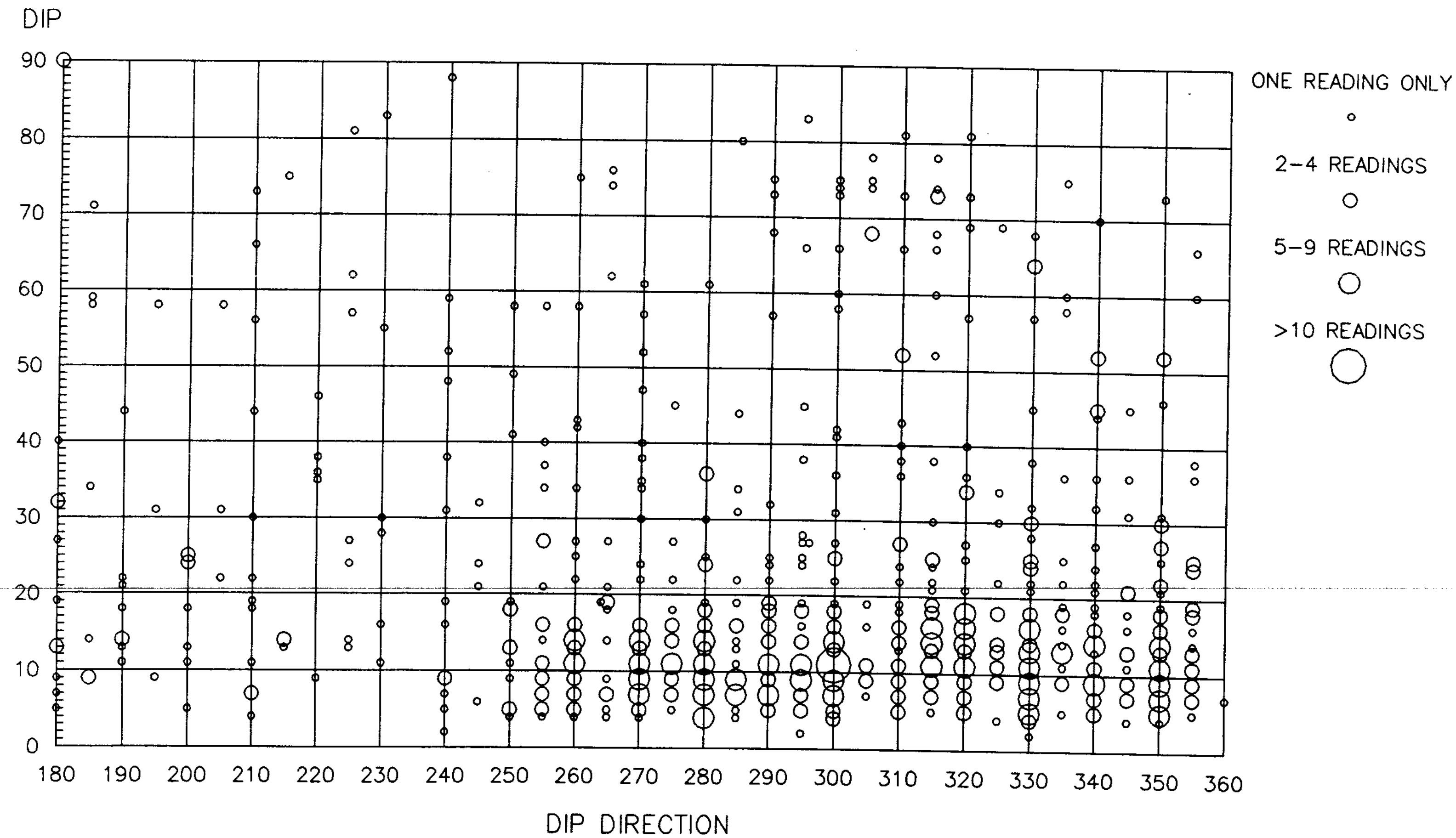


There are over 1,500 points in this data base.
131 readings were at orientation 0 – 0.

SHEET 1 OF 2

Figure E.3.11

PASSAIC TUNNEL PROJECT
DISCONTINUITY DATA FROM BOREHOLE TV CAMERA



There are over 1,500 points in this data base.
131 readings were at orientation 0 - 0.

SHEET 2 OF 2

Figure E.3.12

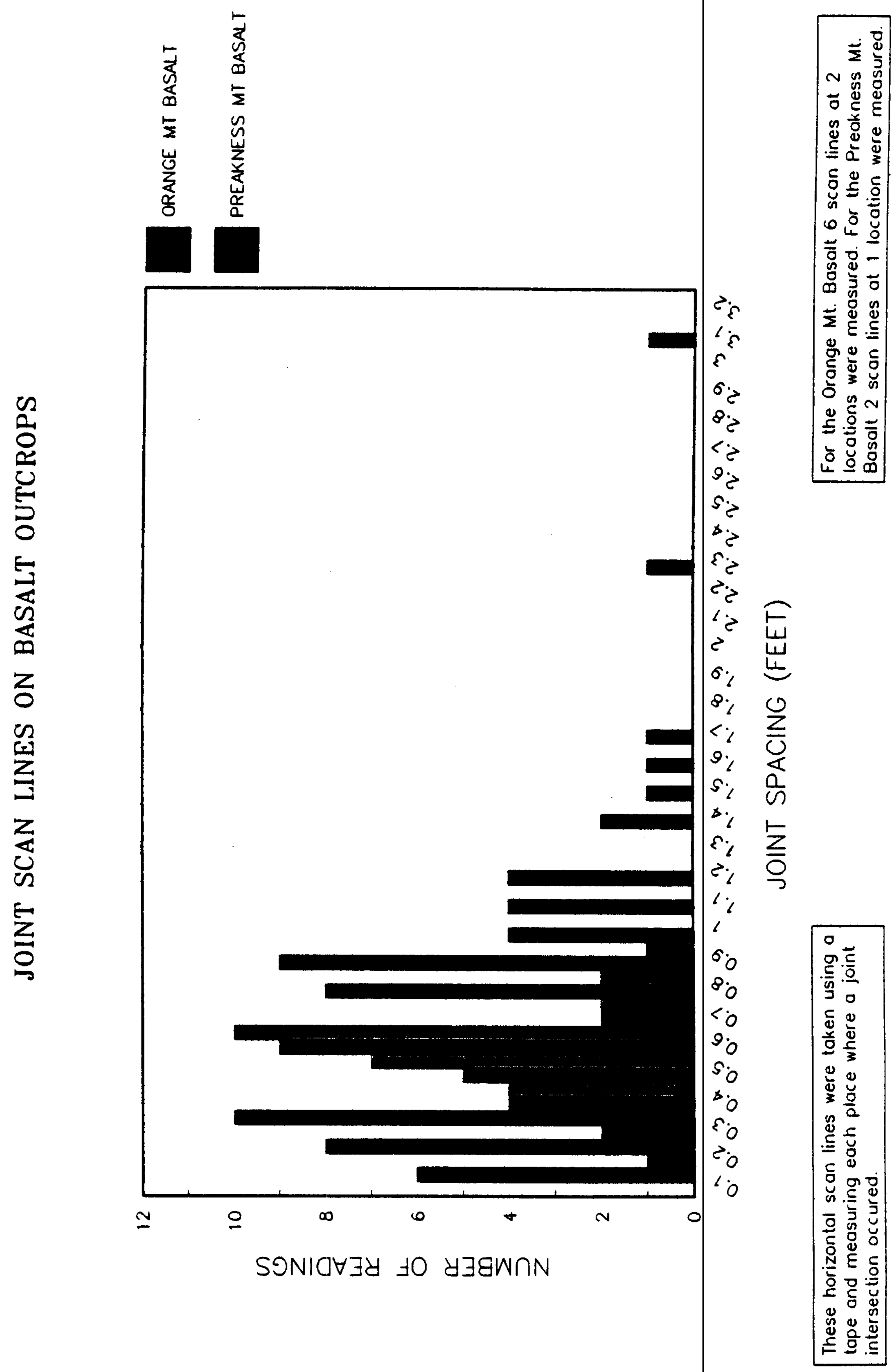
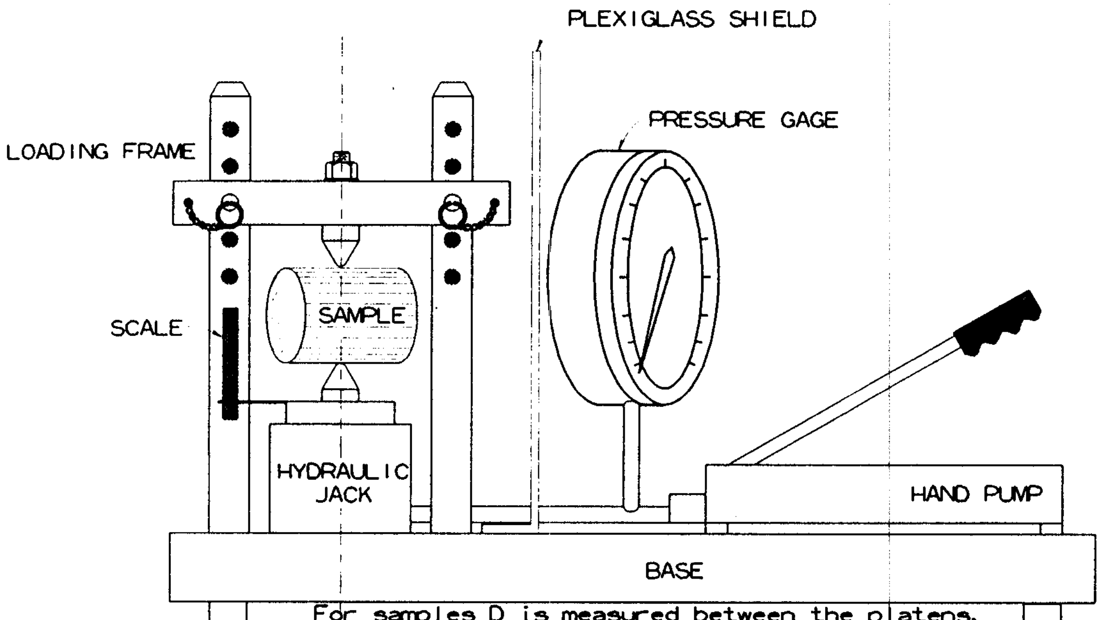
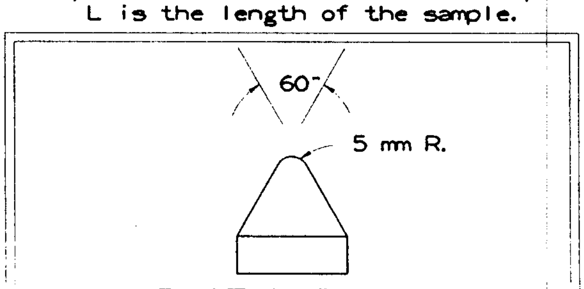

JOINT SCAN LINES ON BASALT OUTCROPS
ORANGE MT BASALT
PREAKNESS MT BASALT
NUMBER OF READINGS
0
2
4
6
8
10
12
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2
2.1
2.2
2.3
2.4
2.5
2.6
2.7
2.8
2.9
3
3.1
3.2
JOINT SPACING (FEET)
These horizontal scan lines were taken using a tape and measuring each place where a joint intersection occured.
For the Orange Mt. Basalt 6 scan lines at 2 locations were measured. For the Preakness Mt. Basalt 2 scan lines at 1 location were measured.

Figure E.3.13

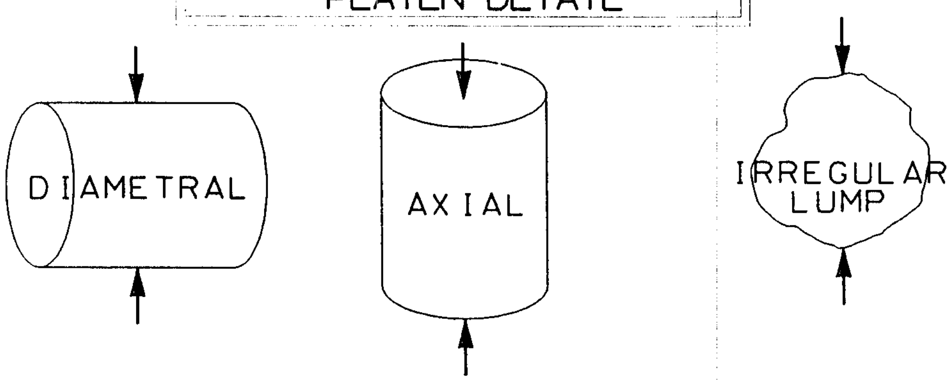
POINT LOAD TEST APARATUS



For samples D is measured between the platens.
L is the length of the sample.



PLATEN DETAIL



SAMPLE ORIENTATION

The point load strength index is computed by dividing the failure load by the dia. squared.

Empirical studies indicate that the Unconfined Compressive strength is 24 times the point load strength index.

PASSAIC FLOOD DAMAGE REDUCTION STUDY

POINT LOAD TEST EQUIPMENT

FIGURE E.3.14

PASSAIC RIVER PROJECT

ESTIMATED TUNNEL BORING MACHINE PENETRATION RATE

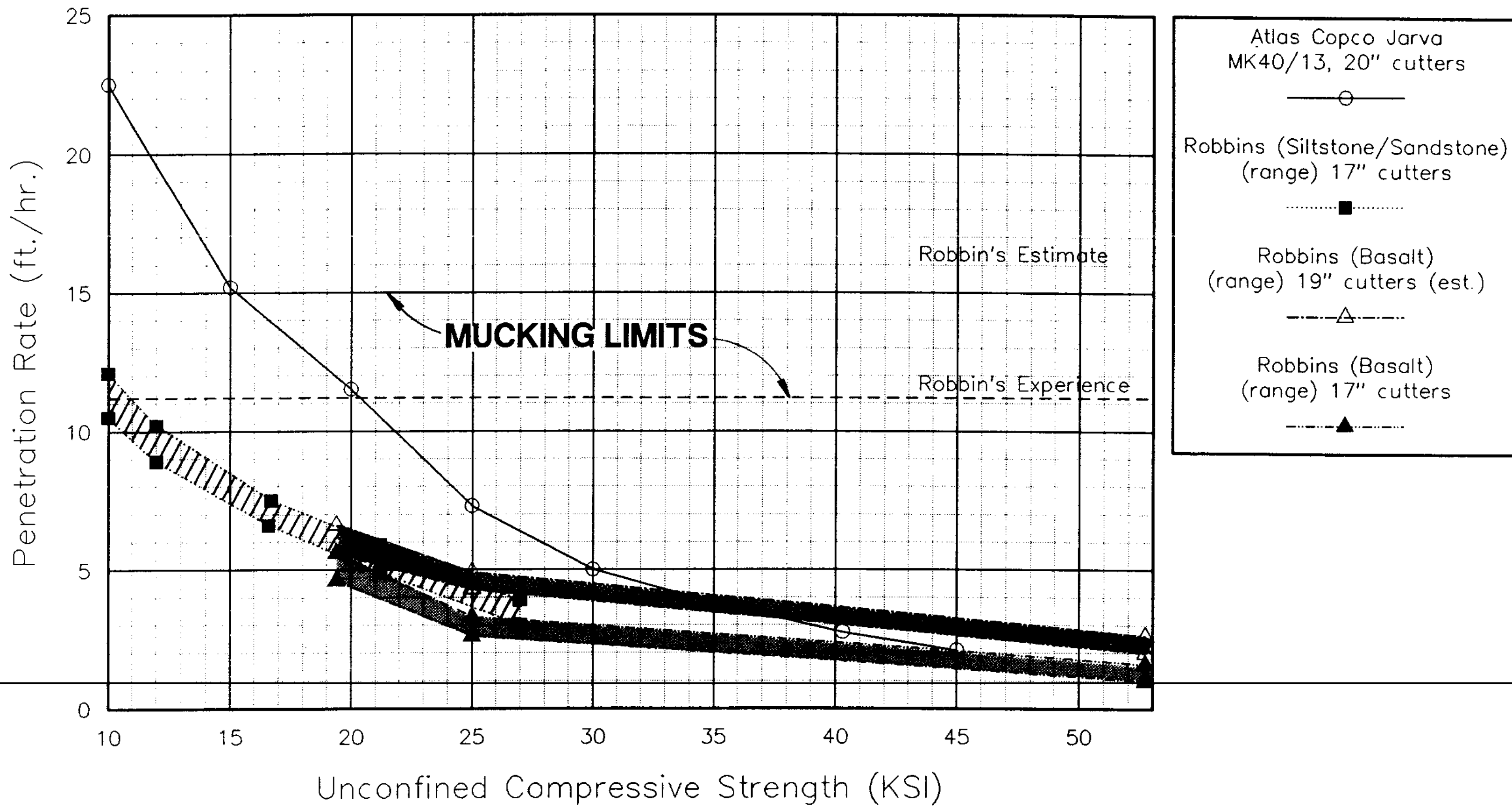
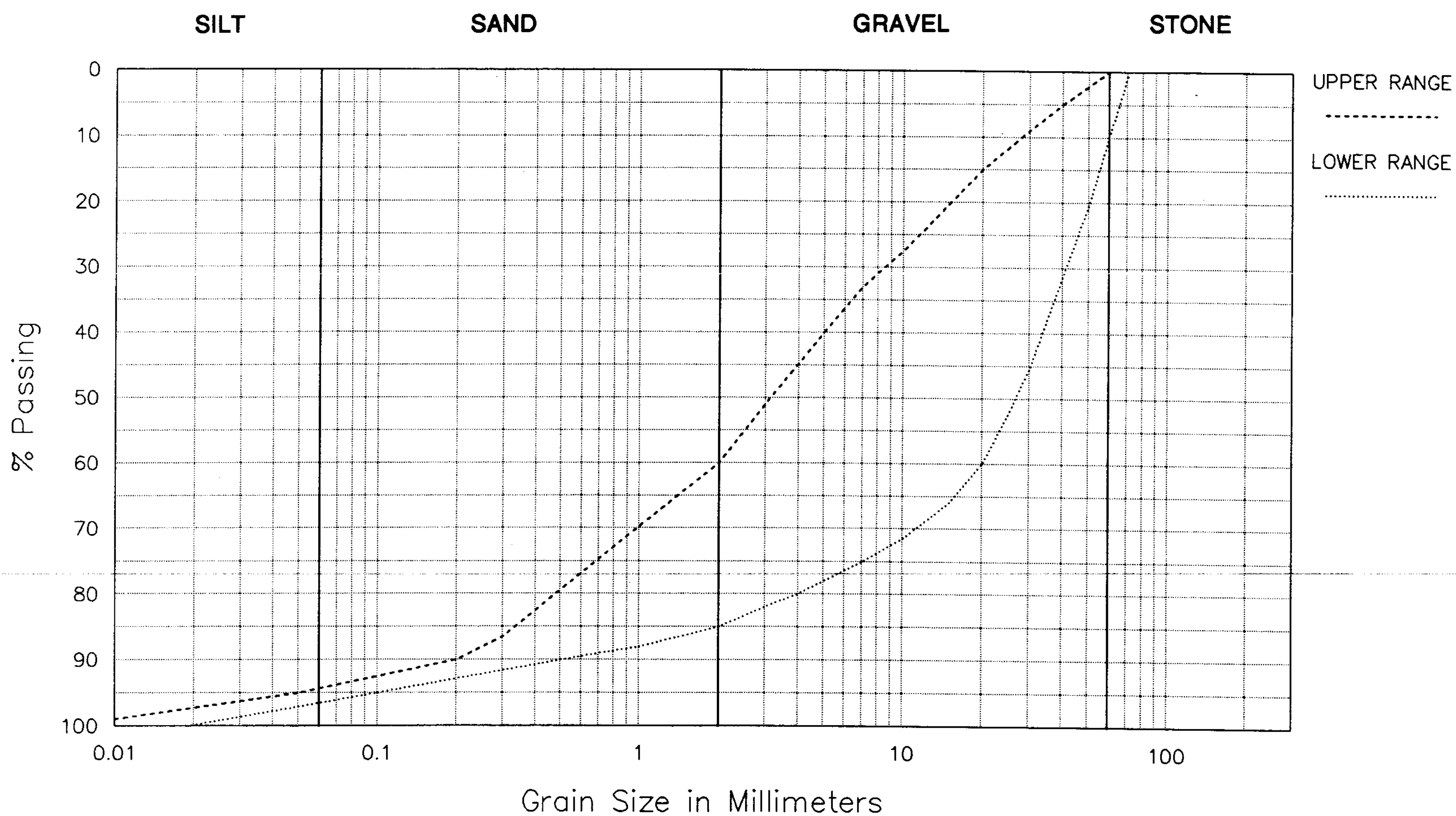


Figure E.3.15

PASSAIC TUNNEL PROJECT
TYPICAL GRADATION RANGES FOR TBM CUTTINGS



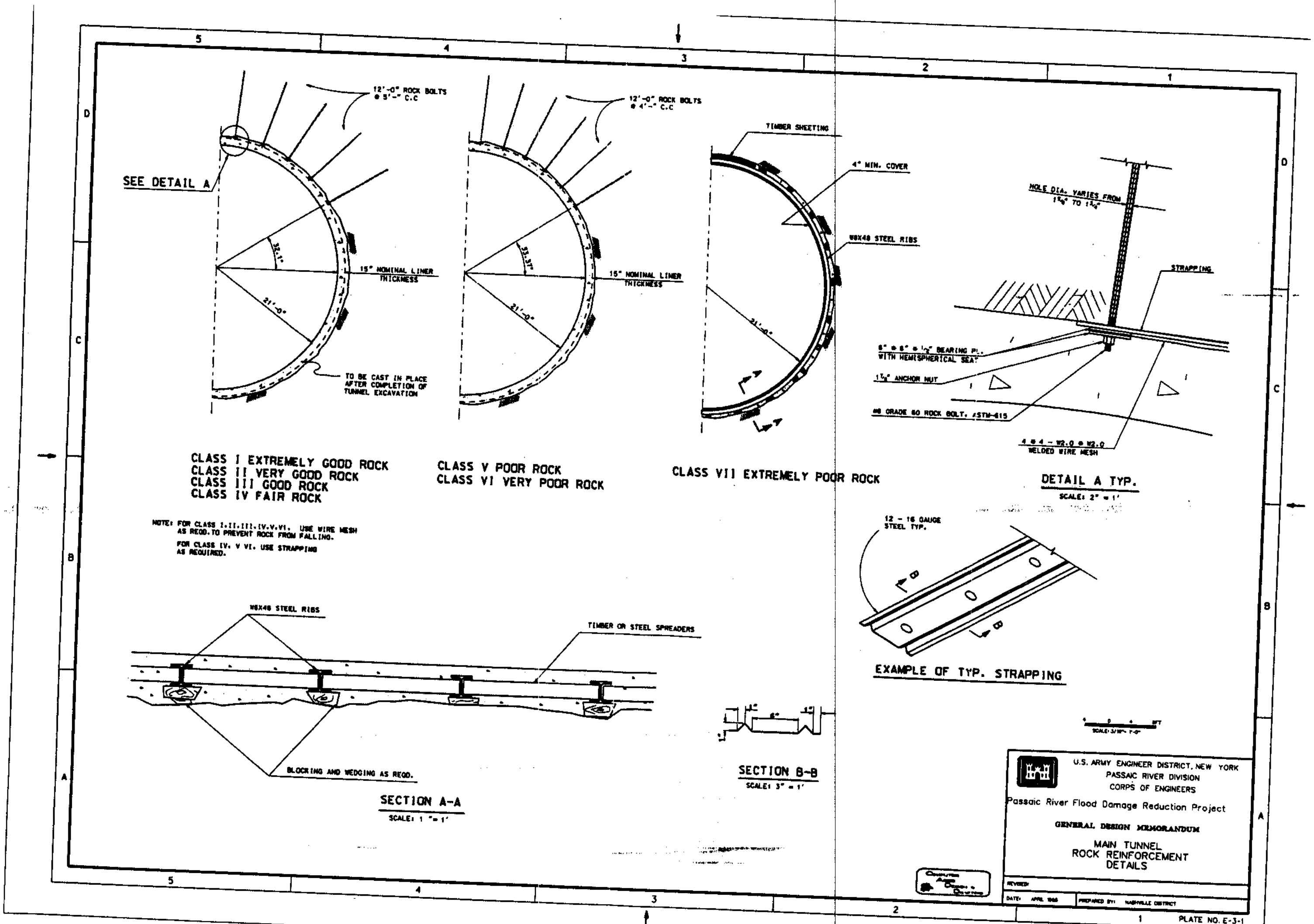
This information was taken from a WIRTH promotional pamphlet.

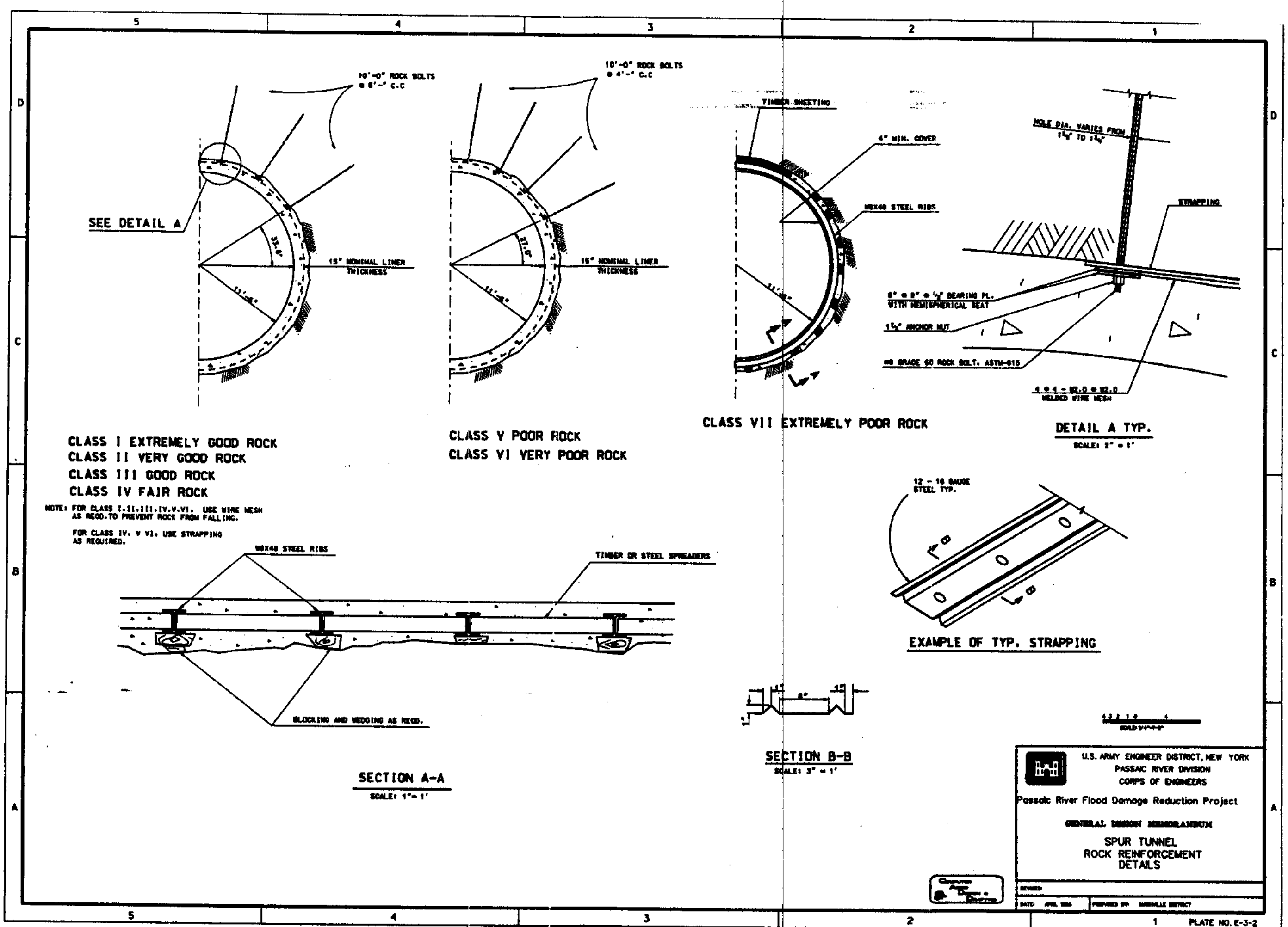
Figure E.3.16

PASSAIC RIVER FLOOD
DAMAGE REDUCTION STUDY

APPENDIX E
SECTION 3

PLATES


12'-0" ROCK BOLTS @ 5'-" C.C
12'-0" ROCK BOLTS @ 4'-" C.C
SEE DETAIL A
15" NOMINAL LINER THICKNESS
TO BE CAST IN PLACE AFTER COMPLETION OF TUNNEL EXCAVATION
TIMBER SHEETING
4" MIN. COVER
W8X48 STEEL RIBS
CLASS I EXTREMELY GOOD ROCK
CLASS II VERY GOOD ROCK
CLASS III GOOD ROCK
CLASS IV FAIR ROCK
CLASS V POOR ROCK
CLASS VI VERY POOR ROCK
CLASS VII EXTREMELY POOR ROCK
STRAPPING
ANCHOR NUT
DETAIL A TYP.
EXAMPLE OF TYP. STRAPPING
TIMBER OR STEEL SPREADERS
BLOCKING AND WEDGING AS REQD.
SECTION A-A
SECTION B-B
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
MAIN TUNNEL
ROCK REINFORCEMENT
DETAILS
PLATE NO. E-3-1



10'-0" ROCK BOLTS
SEE DETAIL A
15" NOMINAL LINER THICKNESS
CLASS I EXTREMELY GOOD ROCK
CLASS II VERY GOOD ROCK
CLASS III GOOD ROCK
CLASS IV FAIR ROCK
CLASS V POOR ROCK
CLASS VI VERY POOR ROCK
TIMBER SHEETING
4" MIN. COVER
W8X48 STEEL RIBS
CLASS VII EXTREMELY POOR ROCK
STRAPPING
1½" ANCHOR NUT
DETAIL A TYP.
12 - 16 GAUGE STEEL TYP.
EXAMPLE OF TYP. STRAPPING
W8X48 STEEL RIBS
TIMBER OR STEEL SPREADERS
BLOCKING AND WEDGING AS REQD.
SECTION A-A
SCALE: 1"= 1'
SECTION B-B
SCALE: 3" = 1'
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
SPUR TUNNEL
ROCK REINFORCEMENT
DETAILS
PLATE NO. E-3-2

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.1

TUNNEL ALIGNMENT DATA

PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

			Main Tunnel Layout Data						
Location	Tunnel Sta.	Northing	Easting	C-Ln EL.	Invert EL.	Distance	Bearing	Slope	
Newark Bay Outlet	0+00	684,850.00	2,152,830.00	-377.6	-398.6				
						1,764.60	N8.8014^W	0.005	
Kearney Point PC	17+64.6	686,593.79	2,153,100.00	-386.4	-407.4				
Curve Data	*LC=315.4'*	*R=500'*	*D=686,670.30N*	*2,152,605.89E*	*Ang=36.1522^*				
Work Shaft 2C PT	20+80.0	686,900.00	2,153,050.00	-388	-409				
V1 PC	20+80.0	686,900.00	2,153,050.00	-388	-409				
V1 Curve Data	*LC=2.4'*	*R=500'*	*D=686,900.0N*	*2,153,050.0E*	*Ang=0.2743^*				
V1 PT	20+82.4	686,902.14	2,153,048.89	-387.99	-408.99				
						14,232.60	N27.3513^W	0.0005	
Work Shaft 2B PT	163+15.0	699,543.60	2,146,509.80	-380.88	-401.88				
Curve Data	*LC=46.75'*	*R=500'*	*D=699,768.39N*	*2,146,944.36E*	*Ang=3.9104^*				
Work Shaft 2B PC	163+61.77	699,586.08	2,146,490.34	-379.74	-400.74				
V2 PT	163+61.77	699,586.08	2,146,490.34	-379.74	-400.74				
V2 Curve Data	*LC=11.45'*	*R=500'*	*D=701,547.66N*	*2,145,639.77E*	*Ang=2,7402^*				
V2 PC	163+73.23	699,596.61	2,146,485.72	-380.19	-401.19				
						20,115.77	N23.4409^W	0.0117	

PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

Location	Tunnel Sta.	Northing	Easting	C-Ln EL.	Invert EL.	Distance	Bearing	Slope
			Main Tunnel Layout Data (Cont.)					
Work Shaft 2A PT	364+89.00	718,041.70	2,138,477.18	-144.84	-165.84			
Curve Data	*LC=36.29'*	*R=500'*	*D=718,240.60N*	*2,138,946.92E*	*Ang=4.1582^*			
Work Shaft 2A PC	365+25.29	718,083.81	2,138,471.18	-143.7	-165.42			
						11,866.84	N19.2827^W	0.0117
V3 PC	483+92.13	729,276.04	2,134,555.43	-5.57	-26.57			
V3 Curve Data	*LC=5.87'*	*R=500'*	*D=729,322.55N*	*2,134,609.27E*	*Ang=0.6417^*			
Shaft #5 & V3 PT	483+98.00	729,281.93	2,135,553.46	-4.6	-25.6			
						13,978.00	N19.2827^W	0.0005
Work Shaft 2	623+76.00	742,475.82	2,129,937.48	2.38	-18.61			
						188.64	N19.2827^W	0.0005
PC-1	625+64.64	742,488.50	2,129,933.06	2.48	-18.52			
Curve Data	*LC=953.19'*	*R=1500'*	*D=741,986.37N*	*2,128,519.58E*	*Ang=36.4099^*			
PT-1	635+17.83	743,229.42	2,129,359.14	2.95	-18.04			
						13,689.33	N55.9616^W	0.0005
PC-2	772+07.16	750,892.00	2,118,015.31	9.81	-11.19			
Curve Data	*LC=905.3'*	*R=1500'*	*D=752,151.47N*	*2,118,866.06E*	*Ang=34.1529^*			

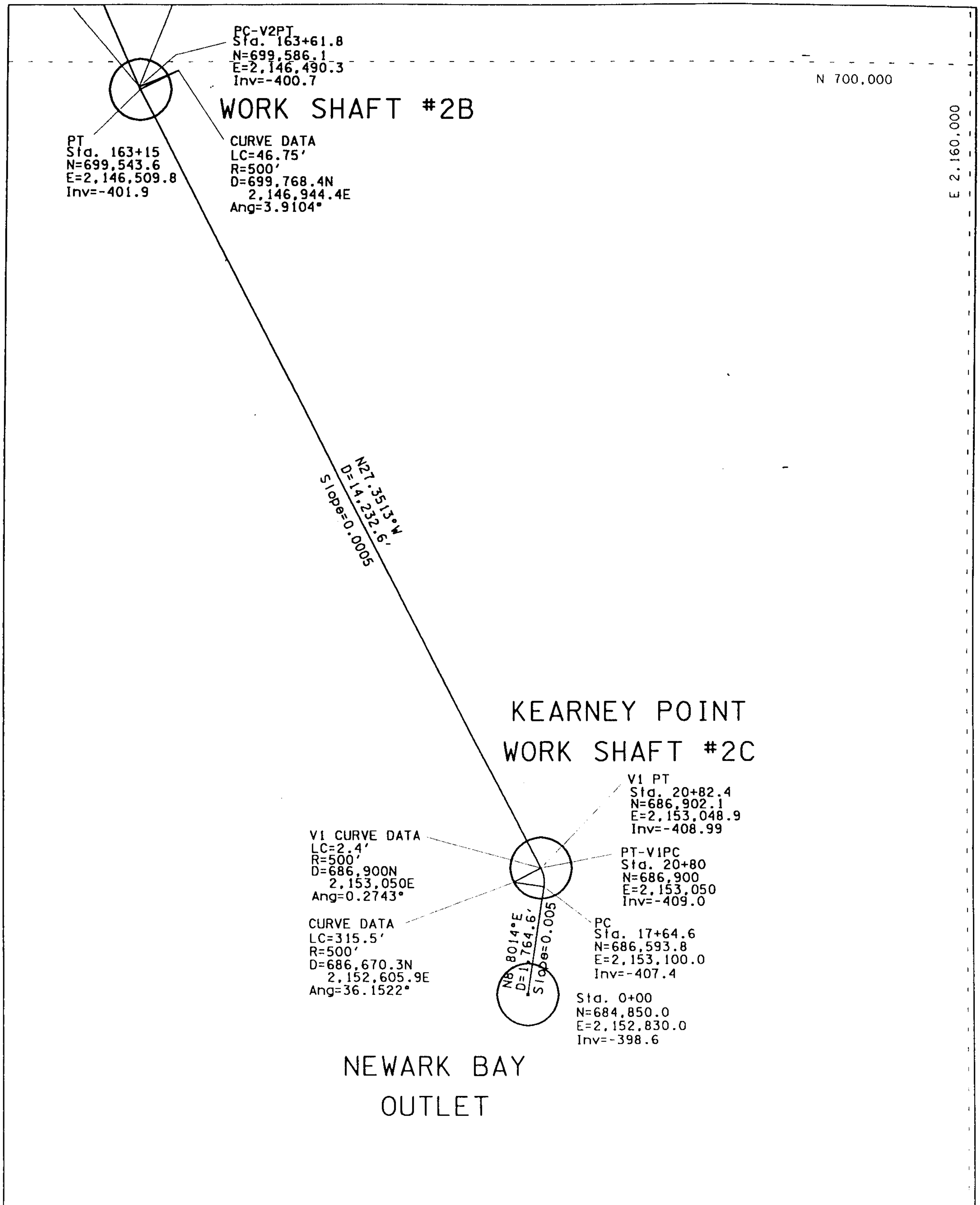
PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

Location	Tunnel Sta.	Northing	Easting	C-Ln EL.	Invert EL.	Distance	Bearing	Slope
			Main Tunnel Layout Data (Cont.)					
PT-2	781+12.46	751,588.12	2,117,458.21	10.26	-10.74			
						187.54	N21.8087^W	0.0005
Vent Shaft #6	783+00.00	751,936.77	2,117,321.70	10.35	-10.64			
						215.64	N21.8087^W	0.0005
Spur Tunnel Inlet	785+15.64	751,952.29	2,117,312.48	10.47	-10.53			
						5,831.36	N21.8087^W	0.0005
Work Shaft 3	843+47.00	757,459.96	2,115,072.60	13.39	-7.61			
						22,900.00	N21.8087^W	0.0005
Main Tnl Dia Incrs.	1072+47.00	778,626.79	2,106,638.75	24.25	3.25			
(from 42' to 52')				29.25	3.25			
						300	N21.8087^W	0.0005
15' Dia. Inlet Vent	1075+47.00	778,905.31	2,106,527.30	29.4	3.4			
						200	N21.8087^W	0.0005
Pompton Inlet CL	1077+47.00	779,091.00	2,106,543.00	N/A	3.5			

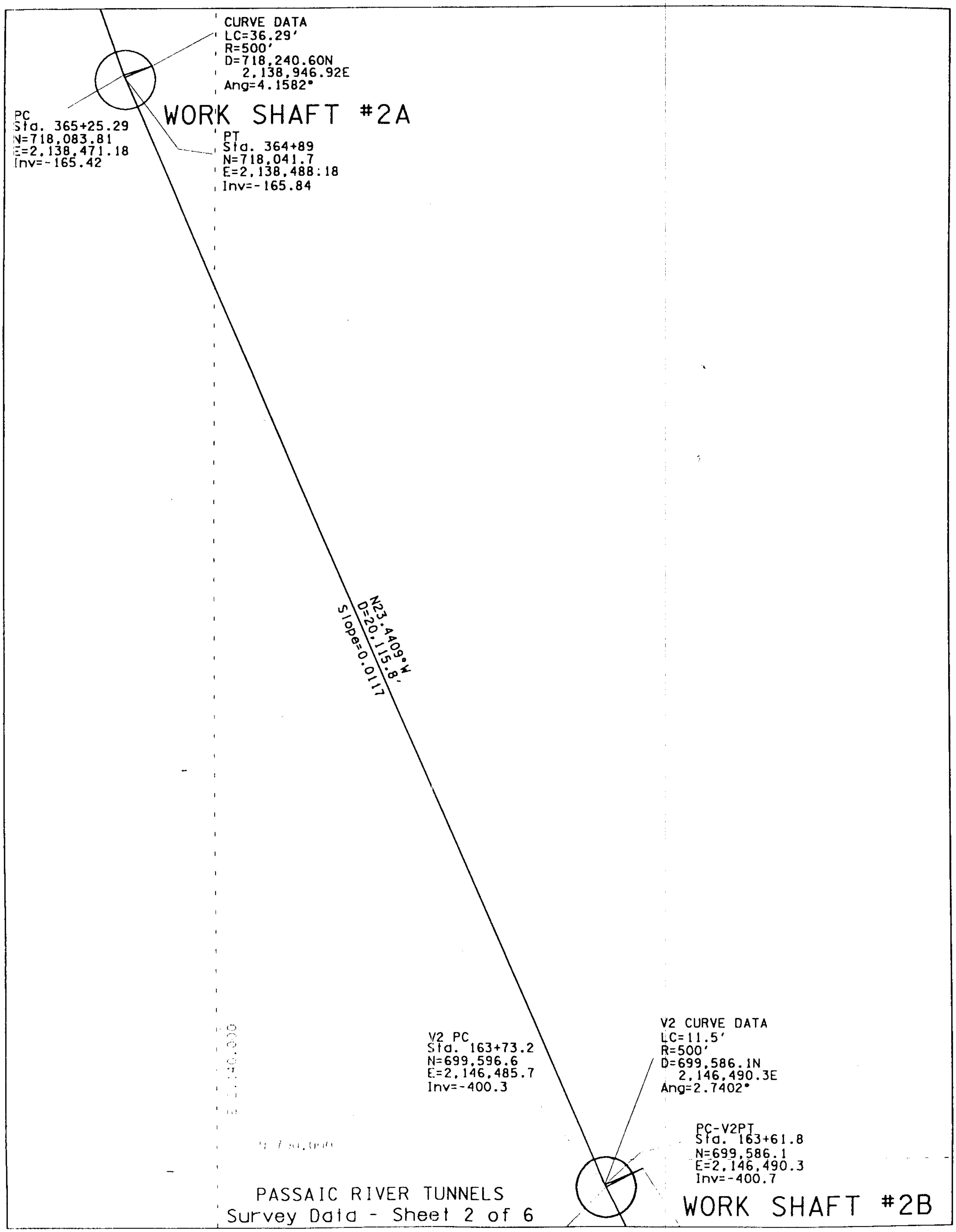
PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

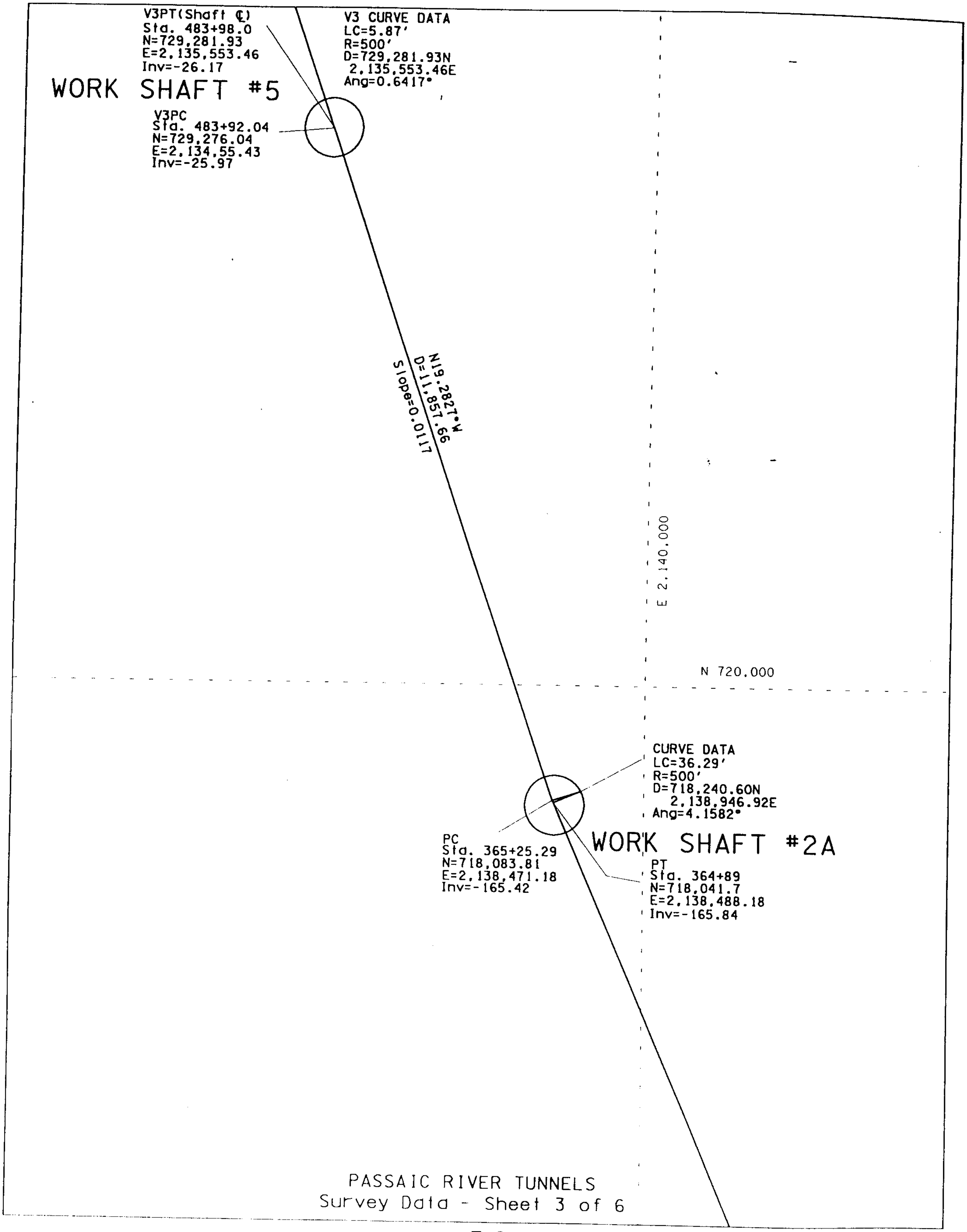
Spur Tunnel Layout Data

Location	Tunnel Sta.	Northing	Easting	C-Ln EL.	Invert EL.	Distance	Bearing	Slope
Spur Tunnel	0+00	751,952.29	2,117,312.47	0.97	-10.53	(Angled 18.8111^W from Main Tunnel Alignment.)		
						326.36	N40.6198^W	0.0015
Work Shaft 4	3+26.36	752,200.00	2,117,100.00	1.46	-10.04			
						204.78	N40.6198^W	0.0015
SPC	5+31.14	752,355.43	2,116,966.68	1.77	-9.73			
Curve Data	*LC=253.61'*	*R=250'*	*D=752,192.68N*	*2,116,776.92E*	*Ang=58.1248^*			
SPT	7+84.75	752,439.77	2,116,738.91	2.15	-9.35			
						5,870.25	W8.6998^S	0.0015
Tunnel Dia. Chg.	66+55.00	751,551.85	2,112,936.21	10.96	-0.54			
(from 23' to 30')				14.46	-0.54			
						260	W8.6998^S	0.0015
12' Dia. Vent Shaft	69+15.00	751,512.52	2,110,679.20	14.85	-0.15			
						100	W8.6998^S	0.0015
Spur Inlet Shaft CL	70+15.00	751,497.40	2,110,580.35	N/A	0			

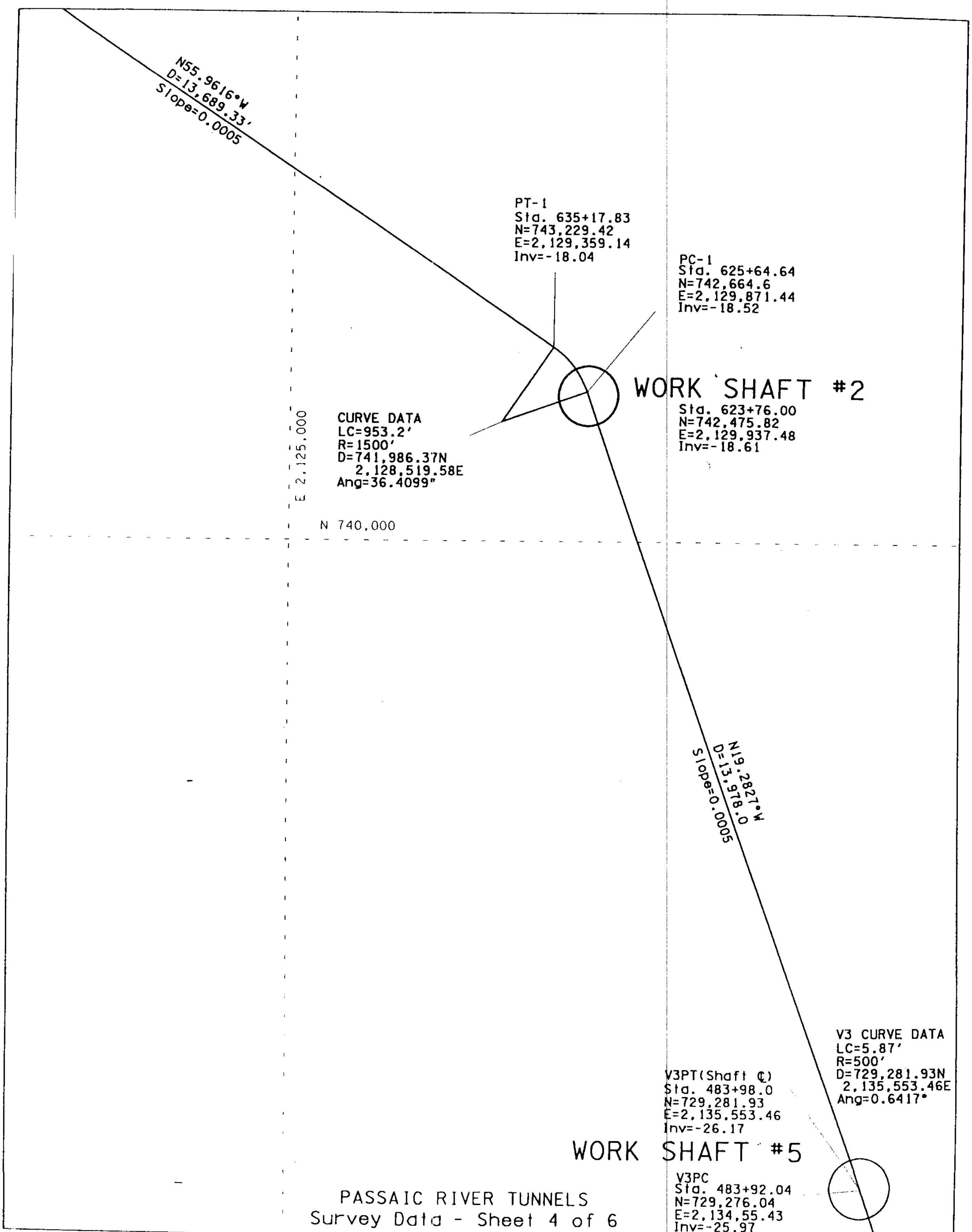


PASSAIC RIVER TUNNELS
Survey Data - Sheet 1 of 6

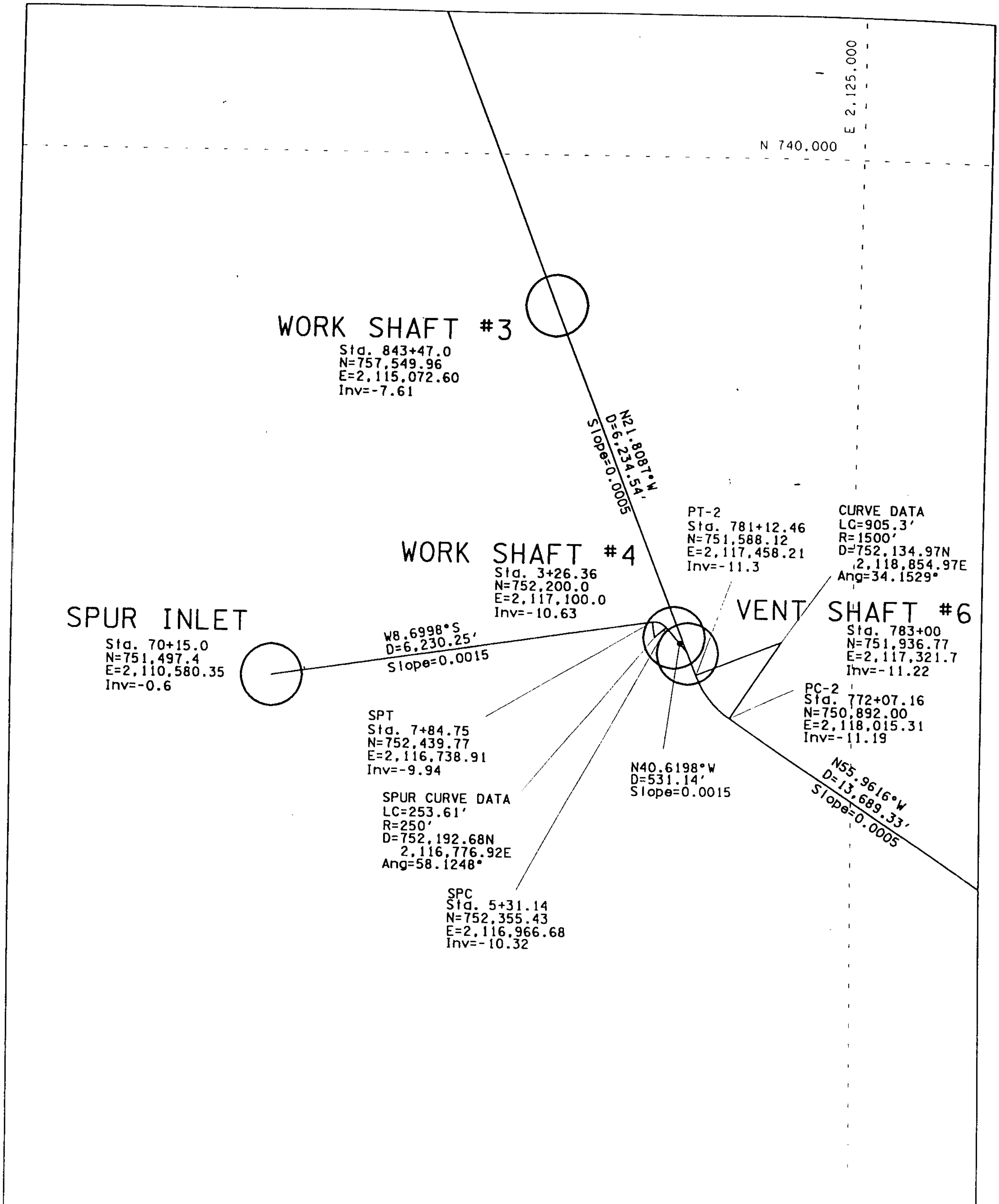

CURVE DATA
LC=36.29'
R=500'
D=718,240.60N
2,138,946.92E
Ang=4.1582°
WORK SHAFT #2A
PC
Sta. 365+25.29
N=718,083.81
E=2,138,471.18
Inv=-165.42
PT
Sta. 364+89
N=718,041.7
E=2,138,488:18
Inv=-165.84
N23.4409°W
D=20,115.8'
Slope=0.0117
V2 PC
Sta. 163+73.2
N=699,596.6
E=2,146,485.7
Inv=-400.3
V2 CURVE DATA
LC=11.5'
R=500'
D=699,586.1N
2,146,490.3E
Ang=2.7402°
PC-V2PT
Sta. 163+61.8
N=699,586.1
E=2,146,490.3
Inv=-400.7
WORK SHAFT #2B
PASSAIC RIVER TUNNELS
Survey Data - Sheet 2 of 6



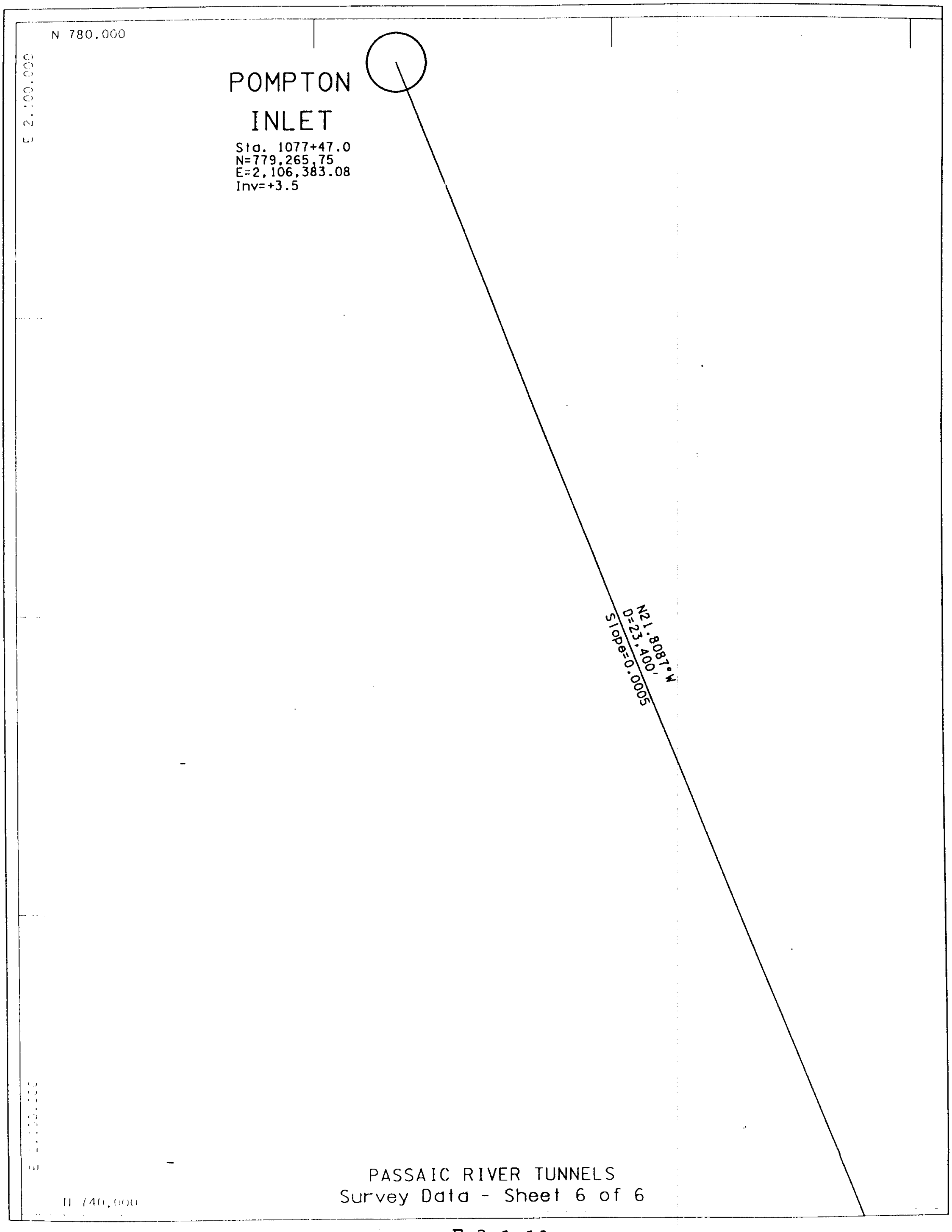
PASSAIC RIVER TUNNELS
Survey Data - Sheet 3 of 6



PASSAIC RIVER TUNNELS
Survey Data - Sheet 4 of 6



PASSAIC RIVER TUNNELS
Survey Data - Sheet 5 of 6



PASSAIC RIVER TUNNELS
Survey Data - Sheet 6 of 6

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.2

BORING DATA

ATTACHMENT E.3.2

BORING DATA

PASSAIC RIVER TUNNEL PROJECT

BOREHOLE BASIC DATA SHEET

PRINT DATE 01-May-95

HOLE NUMBER	ACTUAL TOP ELEV.	ACTUAL DEPTH TO ROCK	ASSUME HTRW	HTRW PROTO.	TUNNEL STATION	STATE PLANE COORD. (1923 DATUM) EASTING	NORTHING	INVERT ELEV. (FT)	ACTUAL ROCK CORE	ACTUAL SPT FOOTAGE	ACTUAL UNSAMP FOOTAGE	DATES START	FINISH	REMARKS (Rock type)
C-01-D	-12	76	N	N/A	316	2,153,020.0	686,170.0	-409.0	4.8	76	0			Newark Bay outlet(Tp)
C-145	-4	87	Y	D	995	2,153,050.0	686,900.0	-408.5	415.2	87	0			(Tp)
C-146	-4	103	Y	C-D	1,698	2,152,810.0	687,295.0	-408.2	396	103	0			Work shaft 2C(Tp)
C-06	5	99.6	Y	C-D	4809	2,151,750.0	689,300.0	-406.6	403.4	56.5	43.1			(Tp)
C-15	10	63.5	Y	D	10,716	2,148,780.0	694,810.0	-403.5	438.6	63.5	0			(Tp)
C-19	8.61	112	Y	D	13,913	2,147,219.6	697,206.3	-402.0	318.9	0	112	23-Jan-91	14-Mar-91	(Tp)
PR-4C	10	151.4	N	N/A	14908.4	2,147,307.9	698,372.8	-401.6	252.8	16.1	135.3	08-Jun-90	11-Jul-90	(Tp)
PR-7	10	161	N	N/A	16,315	2,146,509.8	699,543.6	-400.8	240.9	22.5	138.5	15-Aug-90	22-Aug-90	Work Shaft 2B(Tp)
PR-6A	10	N/A	N	N/A	16,900	2,146,143.5	699,799.6	-400.7	0	131.5	0	28-Jul-990	28-Jul-990	(Tp)
PR-6	10	260	N	N/A	16,905			-400.7	141.5	0	260	17-Jul-90	26-Jul-90	(Tp)
PR-5CA	15	289.3	N	N/A	17013.8	2,145,785.0	699,990.9	-400.5	189.5	0	302.7	23-Jul-90	04-Aug-90	(Tp)
C-21	6.22	155	N	N/A	17,200	2,146,370.0	699,320.0	-400.0	355	0	155			(Tp)
C-22	10	280	Y	D	17,350	2,145,670.0	700,620.0	-400.0	230	0	280			(Tp)
C-23	10.6	290	N	N/A	17,400			-400.0	363.5	0	290			(Tp)
DC-23	8	170	N	N/A	18258.6	2,145,472.0	701,212.0	-399.9	333.6	150.9	19.1			(Tp)
DC-24	7.6	70.7	N	N/A	18,955	2,144,919.5	701,731.4	-394.0	380.6	0	70.7	08-Oct-92	26-Oct-92	(Tp)
C-26	15.18	59.8	N	N/A	19897.8	2,145,194.4	702,878.2	-382.2	381.8	0	59.8	15-Mar-91	04-Apr-91	(Tp)
C-28	107.6	32	N	N/A	22272.3	2,143,703.9	704,819.8	-352.5	470.4	0	32	05-May-92	21-May-92	(Tp)
C-38	26.7	48.8	N	N/A	27759.7	2,141,792.1	709,959.4	-283.8	342.85	41.3	7.5	07-May-94	31-May-94	(Tp)
C-41	94.5	31.6	N	N/A	30,000	2,141,200.3	711,850.9	-253.3	401.4	31.6	0			(Tp)
C-42	90.8	32	N	N/A	30536.1	2,140,646.8	712,699.4	-249	390.8	32	0	22-Jun-94	27-Jul-94	(Tp)
C-48	130.54	45.5	N	N/A	35,000	2,139,234.5	716,434.1	-195.2	356.1	45.5	0			Work Shaft 2A(Tp)
C-53	83.2	41.1	N	N/A	37906.1	2,138,097.4	719,419.4	-156.7	253.6	41.1	8.9	08-Jun-94	22-Jun-94	(Tp)
C-59	228.1	64.6	N	N/A	43923.1	2,136,165.0	725,120.0	-81.4	297.1	0	64.6	29-Apr-92	13-May-92	(Tp)
C-60	177.85	20	N	N/A	44773.4	2,135,844.2	725,908.4	-70.8	271.7	0	20	08-Feb-92	17-Feb-92	(Tp)
DC-61	174.48	13.5	N	N/A	45033.6	2,135,878.4	726,196.6	-67.5	288.2	13.5	0	21-Jan-92	03-Feb-92	(Tp)
C-62	169.36	19.5	N	N/A	45762.5	2,135,400.3	726,800.2	-58.4	261.5	0	19.5	08-Nov-91	15-Nov-91	(Tp)
C-58	183.3	28	N	N/A	46103.9	2,136,446.2	724,224.5	-54.1	304.75	28	0	02-Sep-94	16-Sep-94	(Tp)
C-63	176.86	19.7	N	N/A	46286.5	2,135,216.2	727,290.9	-51.8	262.2	0	19.7	04-Dec-91	11-Dec-91	(Tp)
C-64	176.44	17.1	N	N/A	46804.2	2,135,282.4	727,863.7	-45.3	216.7	0	17.1	05-Dec-91	16-Dec-91	(Tp)
C-67	165.94	27	N	N/A	48417.2	2,134,594.7	729,331.1	-25.6	221	0	27	09-Nov-91	18-Nov-91	Work Shaft 5(Tp)
C-68	172.28	46.2	N	N/A	49013.7	2,134,422.1	729,902.8	-25.3	196.4	0	46.2	02-Nov-91	20-Nov-91	(Tp)
C-69	165.66	40.5	N	N/A	49577.2	2,134,193.8	730,419.7	-25	201.9	0	40.5	21-Oct-91	01-Nov-91	(Tp)
C-70	168.79	39.2	N	N/A	50301.2	2,133,970.4	731,108.6	-24.7	193.8	0	39.2	01-Oct-91	09-Oct-91	(Tp)
C-71	173.44	43.5	N	N/A	50910.4	2,133,773.2	731,685.1	-24.4	187.8	0	43.5	12-Oct-91	02-Nov-91	(Tp)
C-72	202.84	3	N	N/A	51732.5	2,133,520.5	732,467.7	-23.9	249	0	3	27-Feb-92	05-Mar-92	(Tp)
C-73	207.91	2.9	N	N/A	52194.4	2,133,294.0	732,877.4	-23.7	252.1	0	9.4	19-Feb-92	23-Feb-92	(Tp)
C-74	237.1	15	N	N/A	53192.5	2,132,997.7	733,831.4	-23.2	277.1	15	0	01-Dec-92	16-Dec-92	(Tp)
C-76	222.3	6	N	N/A	53841.4	2,132,931.2	734,496.4	-22.9	286	6	0	11-Nov-92	24-Nov-92	(Tp)
C-75	228.17	11.8	N	N/A	54,950	2,132,402.5	735,485.2	-22.3	274.1	0	11.8	17-Jun-91	26-Jun-91	(Tp)

PASSAIC RIVER TUNNEL PROJECT
BOREHOLE BASIC DATA SHEET

PRINT DATE 27-Apr-95

HOLE NUMBER	ACTUAL TOP ELEV.	ACTUAL DEPTH TO ROCK	ASSUME HTRW	HTRW PROTO.	TUNNEL STATION	STATE PLANE COORD. (1923 DATUM) EASTING	NORTHING	INVERT ELEV. (FT)	ACTUAL ROCK CORE	ACTUAL SPT FOOTAGE	ACTUAL UNSAMP FOOTAGE	DATES START	FINISH	REMARKS (Rock type)
C-77	239.42	15	N	N/A	55,392	2,132,174.8	735,873.5	-22.1	266.3	0	15.4	01-May-91	09-May-91	(Tp)
C-78	243.08	16	N	N/A	56,073	2,132,036.4	736,546.7	-21.8	275.7	0	16	24-Apr-91	01-May-91	(Tp)
C-80	245.2	18.9	N	N/A	56747.2	2,131,892.6	737,211.2	-21.4	253.7	0	18.9	02-Jul-92	09-Jul-92	(Tp)
C-79	230.21	6	N	N/A	57167.6	2,131,575.0	737,544.5	-21.2	281.7	0	9.2	16-Apr-91	23-Apr-91	(Tp)
C-81	239.8	9	N	N/A	57885.4	2,131,651.7	738,333.4	-20.9	260	0	10.5	13-Jun-92	20-Jun-92	(Tp)
C-82	242.62	14.9	N	N/A	58356.3	2,131,291.1	738,705.1	-20.6	275.4	0	16	13-May-92	21-May-92	(Tp)
C-83	317.7	13.7	N	N/A	59511.7	2,130,930.0	739,808.1	-20.1	368.4	0	13.2	05-Apr-91	16-Apr-91	(Tp)
PT-40	398.9	22.9	N	N/A	60,113			-19.7	424.2	16	2	01-Jan-85		(Tp)
C-84	433.3	0.3	N	N/A	60461.2	2,130,581.1	740,686.6	-19.6	549.7	0	0.3	27-Aug-91	13-Sep-91	(Tp)
PT-39	385.9	22.8	N	N/A	60,495			-19.6	410.5	0	22.8	01-Jan-85		(Tp)
PT-38	469.6	28.3	N	N/A	61,028			-19.3	472.9	16	0	01-Jan-85		(Tp)
C-85	330.2	1.7	N	N/A	62074.7	2,130,043.0	742,207.8	-18.8	444.3	1.7	3.3	29-May-92	12-Jun-92	Work Shaft 2(Tp)
C-86	322.35	20.2	N	N/A	63163.1	2,129,560.0	742,922.6	-18.2	381.6	0	20.2	11-Oct-91	23-Oct-91	(Tp)
C-87	324.48	8.5	N	N/A	64099.1	2,128,856.2	743,503.0	-17.8	380.6	0	8.5	24-Oct-91	05-Nov-91	(Contact of Tp/Jo)
PT-35	377.8	11.8	N	N/A	64,422			-17.6	401.4	7.3	4.5	01-Jan-85		(Contact of Tp/Jo)
PT-29	330.9	19	N	N/A	65,941			-16.8	350.8	0	11.7	01-Jan-85		(Contact of Tp/Jo)
C-88	332.78	6.5	N	N/A	65953.4	2,127,384.8	744,637.4	-16.8	368.1	0	8.2	05-Mar-92	20-Mar-92	(Contact of Tp/Jo)
C-89	235.3	30.4	N	N/A	67009.9	2,126,514.2	745,236.0	-16.3	195.7	0	30.4	09-May-91	17-May-91	(Jo)
C-90	175.05	19.2	N	N/A	68023.4	2,125,644.1	745,758.5	-15.8	204.5	0	19.2	17-Jul-91	06-Aug-91	(Jo)
C-91	138.28	49.7	N	N/A	68869.9	2,124,937.0	746,224.0	-15.4	147.3	0	49.7	19-Jul-91	27-Jul-91	(Jo)
PT-26	184.5	52.6	N	N/A	70,341			-14.6	200.3	0	52.7	01-Jan-85		(Contact of Jo/Jf)
C-92	140.89	40	N	N/A	71076.1	2,123,291.5	747,729.5	-14.3	146	0	40	29-May-91	05-Jun-91	(Jf)
C-93	147.35	12.6	N	N/A	71520.6	2,122,741.8	747,709.8	-14	209.3	0	12.6	21-Apr-92	27-Apr-92	(Jf)
PT-23	192.8	55	N	N/A	71,616			-14.0	187.5	0	55	01-Jan-85		(Jf)
C-94	181.05	2.9	N	N/A	73145	2,121,377.1	748,591.4	-13.2	222.4	0	5	21-Aug-91	26-Aug-91	(Contact of Jf/Jp)
C-95	184.77	11	N	N/A	74335	2,120,475.2	749,382.3	-12.6	224.3	0	14	17-May-91	21-May-91	(Jp)
DC-141	152.2	27.3	N	N/A	75022.1	2,119,876.8	749,723.8	-12.3	224.4	0	27.3	26-Oct-92	11-Nov-92	(Jp)
C-97	172.6	25.4	N	N/A	75825.6	2,119,230.6	750,203.0	-11.9	207.6	0	25.4	31-Mar-92	03-Apr-92	(Jp)
PT-20	169.6	29	N	N/A	75,873			-11.9	274	0	29	01-Jan-85		(Jp)
PT-19	171.3	16.5	N	N/A	77,357			-11.1	225.5	0	16.5	01-Jan-85		(Jp)
C-98	169.1	33.4	N	N/A	78,041	2,117,496.9	751,516.7	-10.7	189.9	0	33.4	17-Aug-91	27-Aug-91	(Jp)
C-99-D	176.6	16.5	N	N/A	79,023	2,117,128.2	752,433.1	-10.3	214.7	12.5	0.5	06-Jun-91	15-Jun-91	Work Shaft 6(Jp)
PT-18	175.8	32.4	N	N/A	79,477			-10.1	140	0	32.3	01-Jan-85		(Jp)
C-123	175.35	28.2	N	N/A	80,012	2,116,779.4	753,350.4	-9.8	204.1	0	28.9	14-Mar-92	27-Mar-92	(Jp)
C-124	175.7	50.4	N	N/A	80,627	2,116,576.7	753,931.4	-9.5	181.8	0	50.4	22-May-92	29-May-92	(Contact of Jp/Jt)
PT-17	176.5	42	N	N/A	80,909			-9.4	160.2	0	62.3	01-Jan-85		(Contact of Jp/Jt)
C-125	179.2	52.1	N	N/A	81,204	2,116,319.4	75,450.2	-9.2	181.2	0	52.1	27-Apr-92	05-May-92	(Contact of Jp/Jt)
DC-136	177.7	69	N	N/A	81,686	2,116,129.7	754,893.5	-9.0	232.7	66.5	3.9	31-Mar-93	19-Apr-93	(Contact of Jp/Jt)
C-100A	177.4	145	N	N/A	81,737	2,116,113.9	754,942.0	-8.9	128	0	145	19-Feb-92	14-Mar-92	(Contact of Jp/Jt)
C-126	172.21	67.8	N	N/A	82,591	2,115,830.9	755,748.7	-8.5	155.2	0	67.8	09-Jul-92	15-Jul-92	(Jt)

PASSAIC RIVER TUNNEL PROJECT
BOREHOLE BASIC DATA SHEET

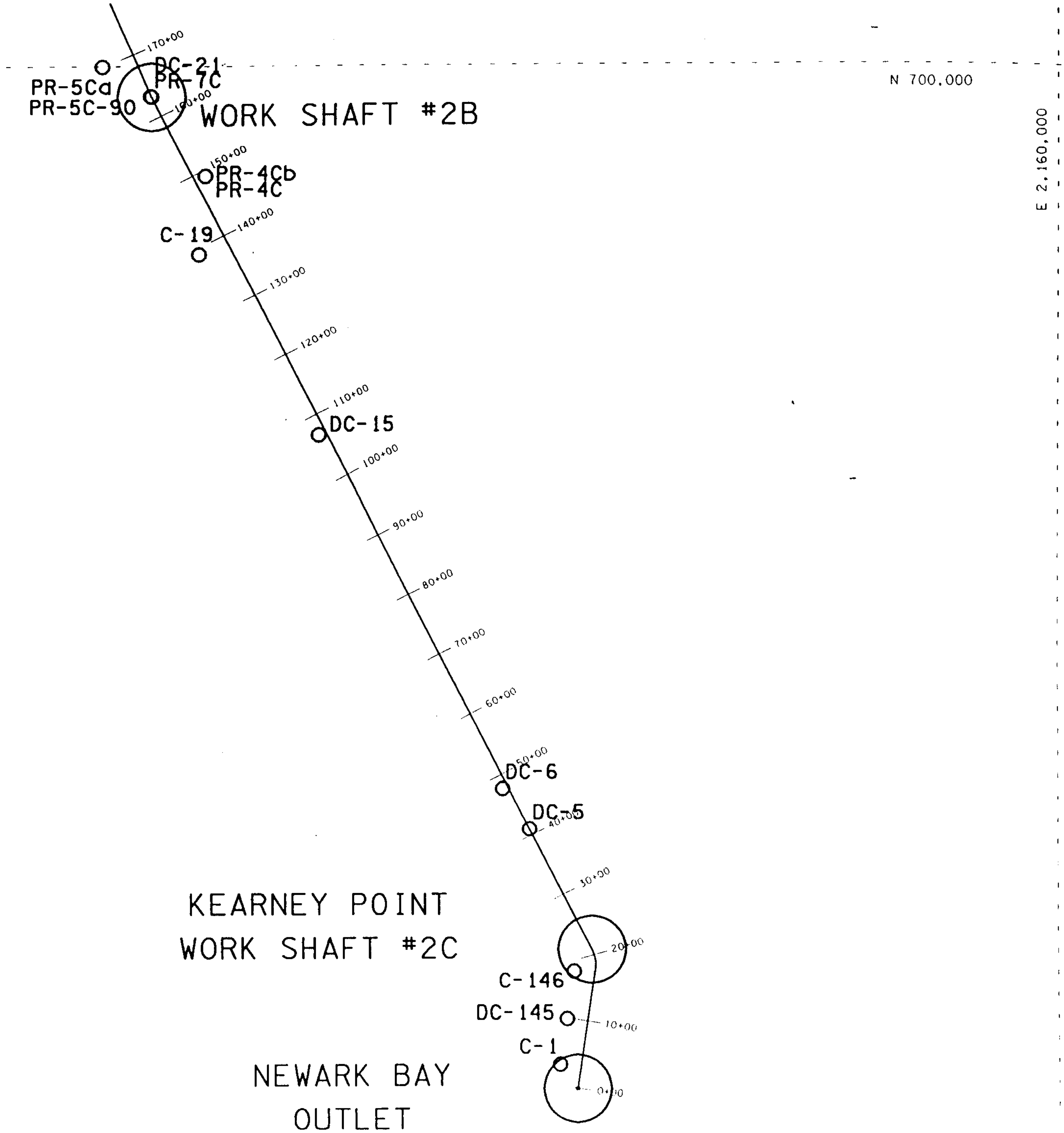
PRINT DATE 01-May-95

HOLE NUMBER	ACTUAL TOP ELEV.	ACTUAL DEPTH TO ROCK	ASSUME HTRW	HTRW PROTO.	TUNNEL STATION	STATE PLANE COORD. (1923 DATUM) EASTING	NORTHING	INVERT ELEV. (FT)	ACTUAL ROCK CORE	ACTUAL SPT FOOTAGE	ACTUAL UNSAMP FOOTAGE	DATES START	FINISH	REMARKS (Rock type)
PT-16	172.3	52.3	N	N/A	82,682			-8.5	344.3	36	14.3	01-Jan-85		(Contact of Jp/Jt)
C-127	174.7	90.8	N	N/A	83,009	2,115,596.2	756,106.7	-8.3	131.9	0	90.8	16-Jul-92	21-Jul-92	(Jt)
C-101	177.5	118	N	N/A	83,721	2,115,368.7	756,818.5	-7.9	108.6	0	118	07-Aug-91	16-Aug-91	(Jt)
DC-102	180	86.7	N	N/A	84,337	2,115,000.0	757,750.0	-7.4	266.5	60.5	26.2			Work Shaft 3(Jt)
PT-15	175.6	81.3	N	N/A	84,524			-7.5	292.5	0	80	01-Jan-85		(Jt)
C-128	188.4	35.4	N	N/A	85,307	2,114,708.0	758,225.2	-7.2	197.8	0	35.4	11-Aug-92	20-Aug-92	(Jt)
PT-14	229.4	44.5	N	N/A	85,987			-6.8	389.3	0	43	01-Jan-85		(Jt)
C-129	237	40.6	N	N/A	86,083	2,114,586.0	759,012.4	-6.8	243.2	0	40.6	24-Jul-92	03-Aug-92	(Jt)
PT-13	313.7	12.5	N	N/A	86,944			-6.3	339	0	12.5	01-Jan-85		(Jt)
C-103	344.6	5.5	N	N/A	87,995	2,113,814.2	760,762.1	-5.8	397.8	0	5.5	04-Jun-92	16-Jun-92	(Contact of Jt/Jh)
C-130	311.3	5.5	N	N/A	88,342	2,113,817.2	761,137.2	-5.6	346.6	0	5.5	16-Jun-92	01-Jul-92	(Contact of Jt/Jh)
PT-12	241.3	8	N	N/A	89,101			-5.3	279.5	0	11.3	01-Jan-85		(Contact of Jt/Jh)
C-131	177.4	12.5	N	N/A	89,896	2,113,139.7	762,540.2	-4.9	216.4	0	12.5	07-Jul-92	14-Jul-92	(Jh)
PT-11	180.9	10	N	N/A	90,154			-4.7	252.8	0	10	01-Jan-85		(Contact of Jt/Jh)
C-104	167.6	101.2	N	N/A	91,065	2,112,635.2	763,598.3	-4.3	125.5	0	101.2	18-Sep-91	10-Oct-91	(Contact of Jh/Jb)
PT-10	168	91	N	N/A	91,324			-4.1	117.2	0	93	01-Jan-85		(Contact of Jh/Jb)
DC-132	167.1	65.6	N	N/A	92,018	2,112,324.9	764,499.5	-3.8	206.2	58.8	6.8	21-Feb-93	27-Mar-93	(Contact of Jh/Jb)
C-105	184.8	32.5	N	N/A	92,749	2,112,058.1	765,180.7	-3.4	213.6	0	33	12-Sep-91	18-Sep-91	(Jb)
PT-9	228.5	40.9	N	N/A	93,483			-3.1	417.2	0	40.8	01-Jan-85		(Contact of Jh/Jb)
C-106	258.6	16.8	N	N/A	94,491	2,111,308.6	766,756.5	-2.6	310.9	0	16.8	30-Jul-91	06-Aug-91	(Jb)
PT-8	269.6	15	N	N/A	95,171			-2.2	293.3	0	19.2	01-Jan-85		(Jb)
C-107	265.3	35.2	N	N/A	95,974	2,110,808.9	768,154.2	-1.8	296.1	0	35.2	27-Aug-91	10-Sep-91	(Jb)
C-133	265.7	27.3	N	N/A	96,760	2,110,601.0	768,917.4	-1.4	295.8	0	27.3	15-Jul-92	22-Jul-92	(Jb)
PT-7	271.9	22.3	N	N/A	97,285			-1.2	290	0	22.3	01-Jan-85		(Jb)
C-108	255.8	24.7	N	N/A	97,955	2,110,101.9	770,004.8	-0.8	297.8	0	24.7	27-May-92	03-Jun-92	(Jb)
C-109	239.1	9	N	N/A	98,713	2,109,965.8	770,766.4	-0.5	265.8	0	10.8	27-Jun-91	16-Jul-91	(Jb)
C-134	237	26.7	N	N/A	99,315	2,109,263.2	771,255.1	-0.2	246.2	0	26.7	21-Jul-92	31-Jul-92	(Jb)
PT-5	251.7	31.4	N	N/A	99,772			0.1	280.2	0	31.3	01-Jan-85		(Jb)
C-110	263.6	47.7	N	N/A	101,373	2,108,833.1	773,178.6	0.9	263.5	0	48.7	14-Sep-91	21-Sep-91	(Jb)
C-111	228.8	30.5	N	N/A	102,281	2,108,302.8	773,944.0	1.3	276.4	0	30.5	07-Aug-91	17-Aug-91	(Jb)
PT-3	210.4	18.6	N	N/A	103,221			1.8	243.6	0	18.5	01-Jan-85		(Jb)
C-112	209.5	16.3	N	N/A	103,725	2,108,000.3	775,379.0	2.1	285.7	0	16.3	23-Sep-91	01-Oct-91	(Jb)
PT-2	222.9	13.6	N	N/A	104,852			2.6	259.9	0	13.5	01-Jan-85		(Jb)
C-135	195.6	8	N	N/A	105,729	2,107,226.9	777,226.9	3.1	232.9	0	8	22-Jun-92	01-Jul-92	(Jb)
PT-1	187.3	38.5	N	N/A	106,109			3.2	214.2	0	0			(Jb)
C-113	179.7	81.8	N	N/A	106,785	2,106,778.1	778,185.1	3.6	151.2	0	81.8	22-May-92	27-May-92	(Jb)
C-147B	185.7	61	N	N/A	107,159	2,107,098.2	778,691.0	3.8	171.1	0	61			Pompton Inlet(Jb)
DC-147	186.1	61.7	N	N/A	107,159	2,107,085.1	778,711.7	3.8	170.9	61.7	0			Pompton Inlet(Jb)
DU-114A	179.9	49.2	N	N/A	107,571	2,106,669.2	779,013.5	4.0	0	49.2	0			Pompton Inlet(Jb)
DC-114	180	51	N	N/A	107,576	2,106,679.0	778,997.7	4.0	172.5	51	10	24-Aug-92	05-Oct-92	Pompton Inlet(Jb)

PASSAIC RIVER TUNNEL PROJECT
BOREHOLE BASIC DATA SHEET

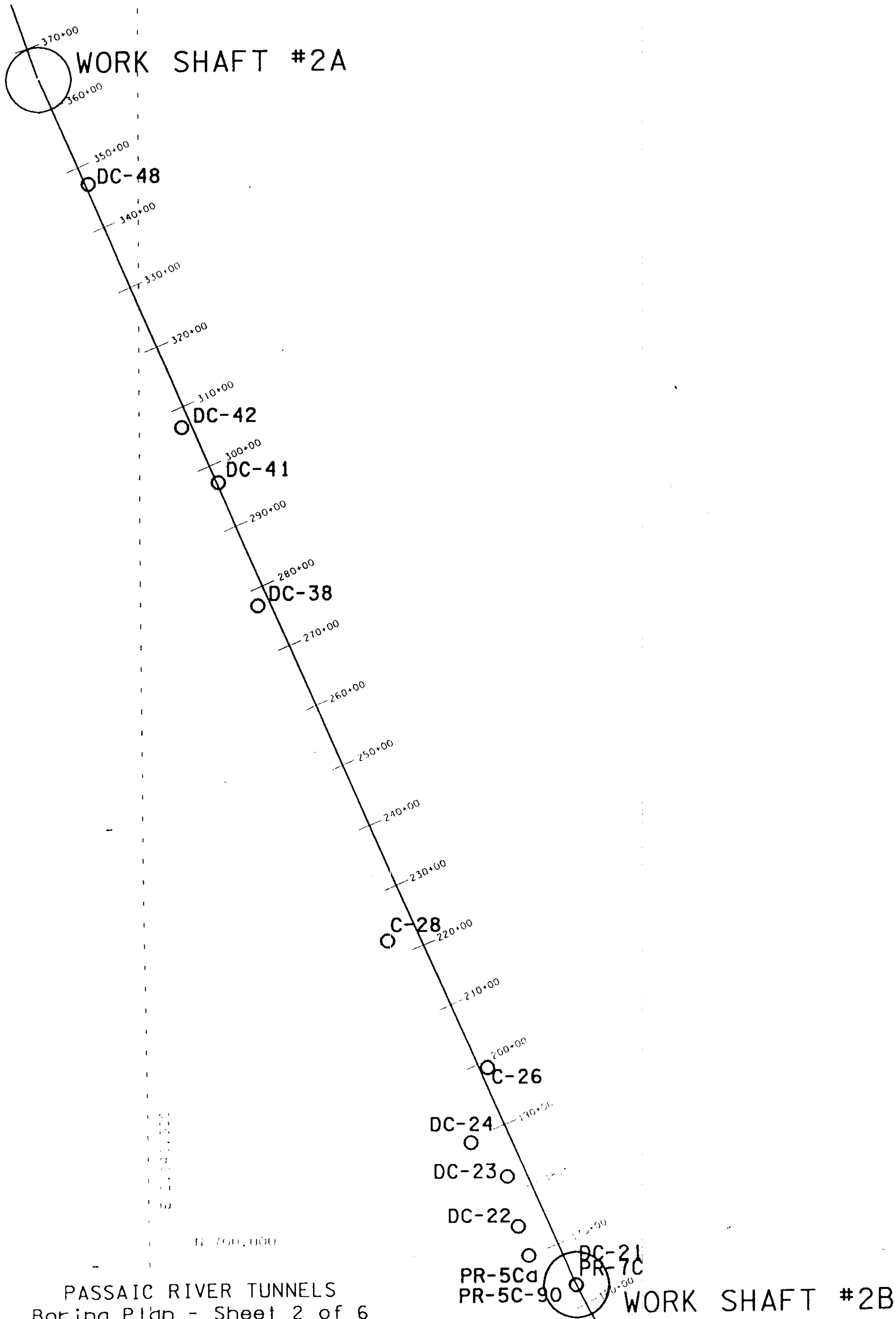
PRINT DATE 01-May-95

HOLE NUMBER	ACTUAL TOP ELEV.	ACTUAL DEPTH TO ROCK	ASSUME HTRW	HTRW PROTO.	TUNNEL STATION	STATE PLANE COORD. (1923 DATUM)		INVERT ELEV. (FT)	ACTUAL ROCK CORE	ACTUAL SPT FOOTAGE	ACTUAL UNSAMP FOOTAGE	DATES		REMARKS (Rock type)
						EASTING	NORTHING					START	FINISH	
DC-115	178.1	40	N	N/A	900	2,116,578.1	752,731.6	0.0	192.7	14	26	10-Apr-92	24-Apr-92	Work Shaft 4(Jp)
C-116	176.5	46	N	N/A	1,153	2,116,330.8	752,686.0	0.0	184.7	0	46	01-Apr-92	10-Apr-92	Spur Tunnel(Jp)
C-117	171.7	42.5	N	N/A	1,912	2,115,576.4	752,534.0	0.0	178.6	0	42.5	24-Mar-92	01-Apr-92	Spur Tunnel(Jp)
C-118	166.3	140.1	N	N/A	3,663	2,113,864.0	752,199.3	0.0	123	0	140.1	06-Apr-92	14-Apr-92	Spur Tunnel(Contact Jt/Jp)
DC-119	180	74	N	N/A	3,889	2,113,640.0	752,155.0	0.0	188.2	74	10			Spur Tunnel(Contact Jt/Jp)
DC-120	170	107.6	N	N/A	5,218	2,112,340.0	751,880.0	0.0	163.1	71.5	36.1	5-Aug-93	17-Aug-93	Spur Tunnel(Contact Jt/Jp)
DC-121	170.8	71	N	N/A	6,222	2,111,381.0	751,691.9	0.0	192	40	41.3			Spur Tunnel(Jt)
DC-122	169.4	57	N	N/A	7,330	2,110,391.3	751,417.7	0.0	216	57	0			Spur Tunnel Inlet(Jt)
DU-122A	168.9	57	N	N/A	7,325	2,110,411.7	751,410.2	0.0	216	57	0			Spur Tunnel Inlet(Jt)
DC-148	175	80	Y	N/A	7,300			0.0	182	36.4	55			Spur Tunnel Inlet(Jt)

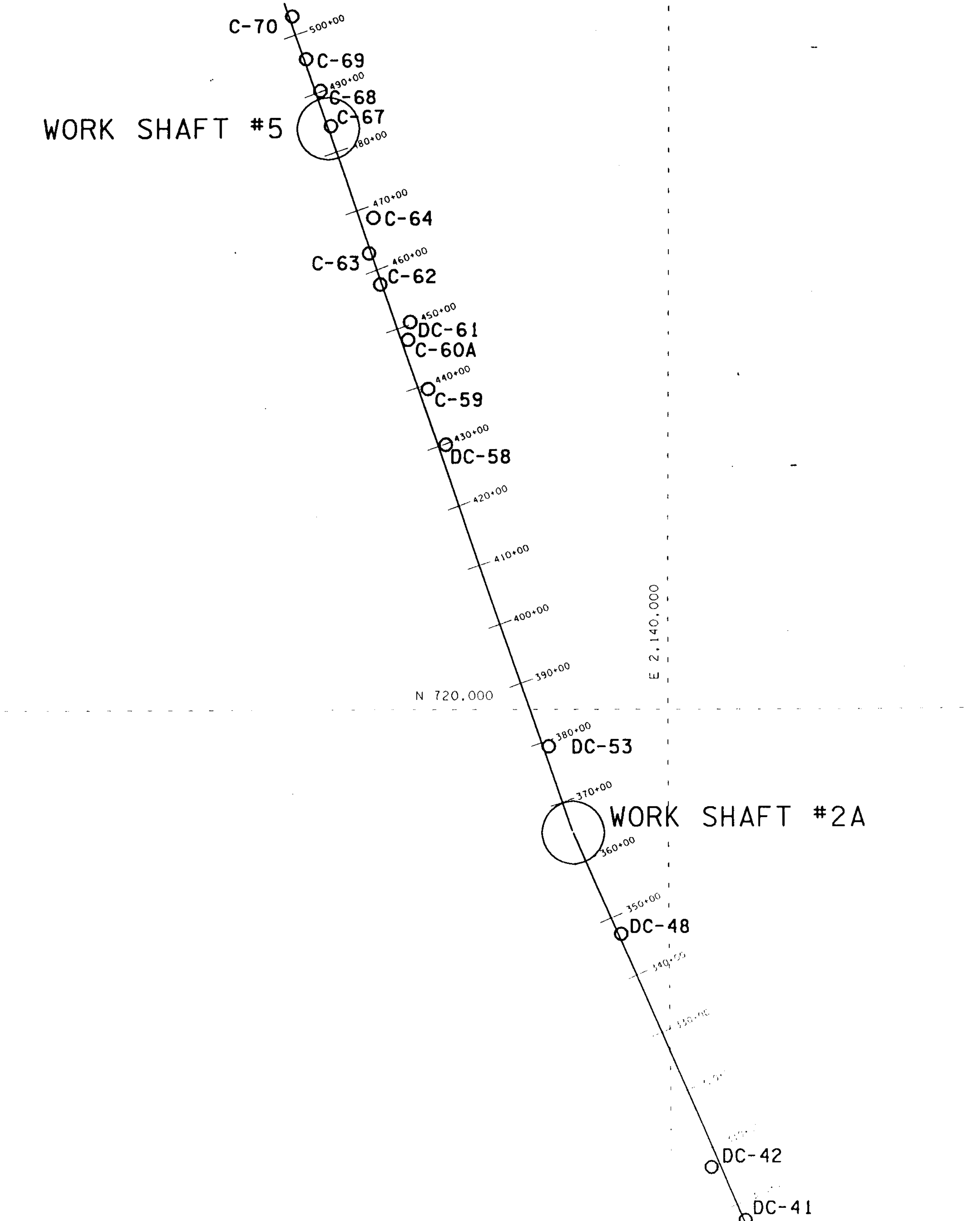


PASSAIC RIVER TUNNELS
Boring Plan - Sheet 1 of 6

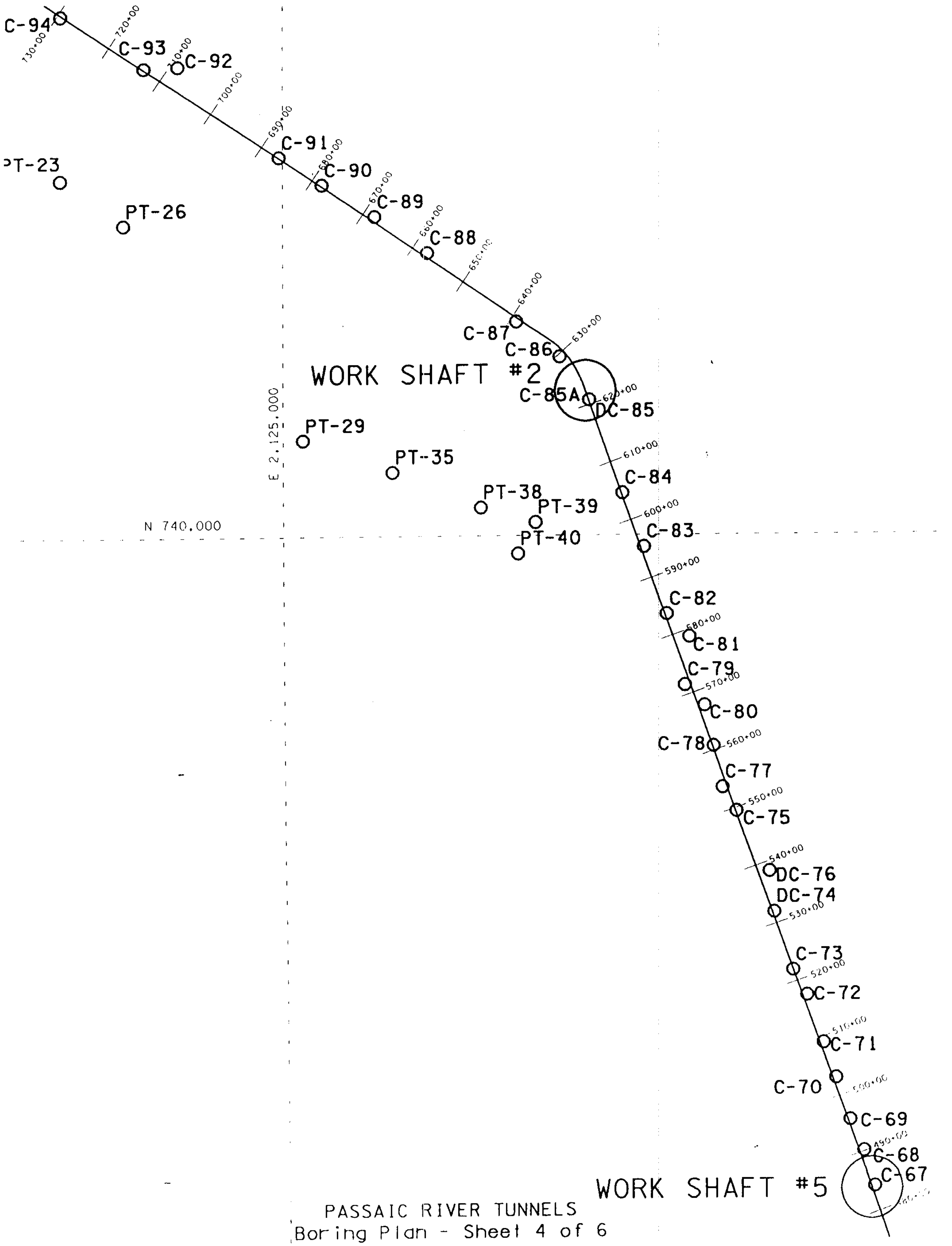
E.3.2-5



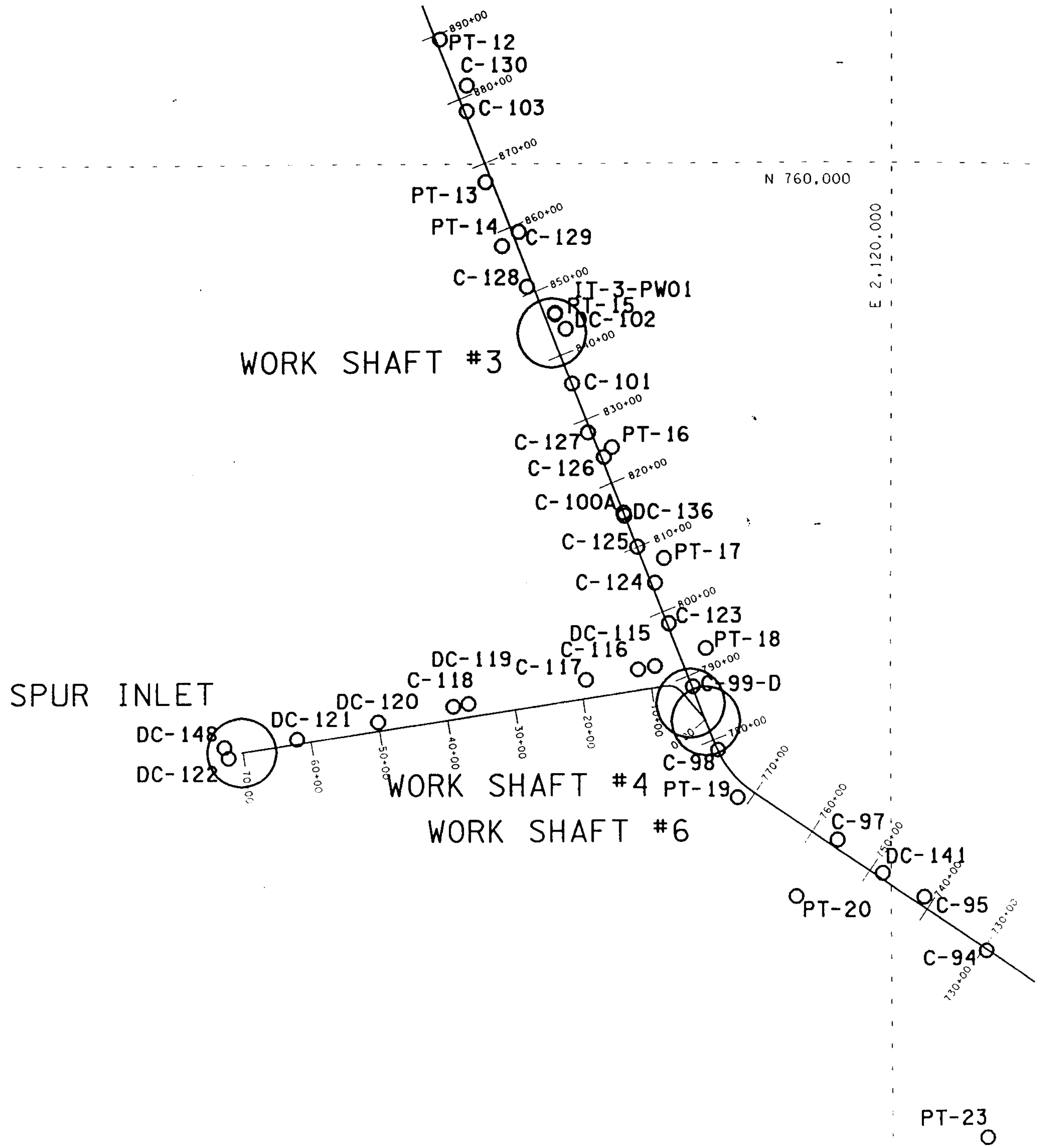
PASSAIC RIVER TUNNELS
Boring Plan - Sheet 2 of 6



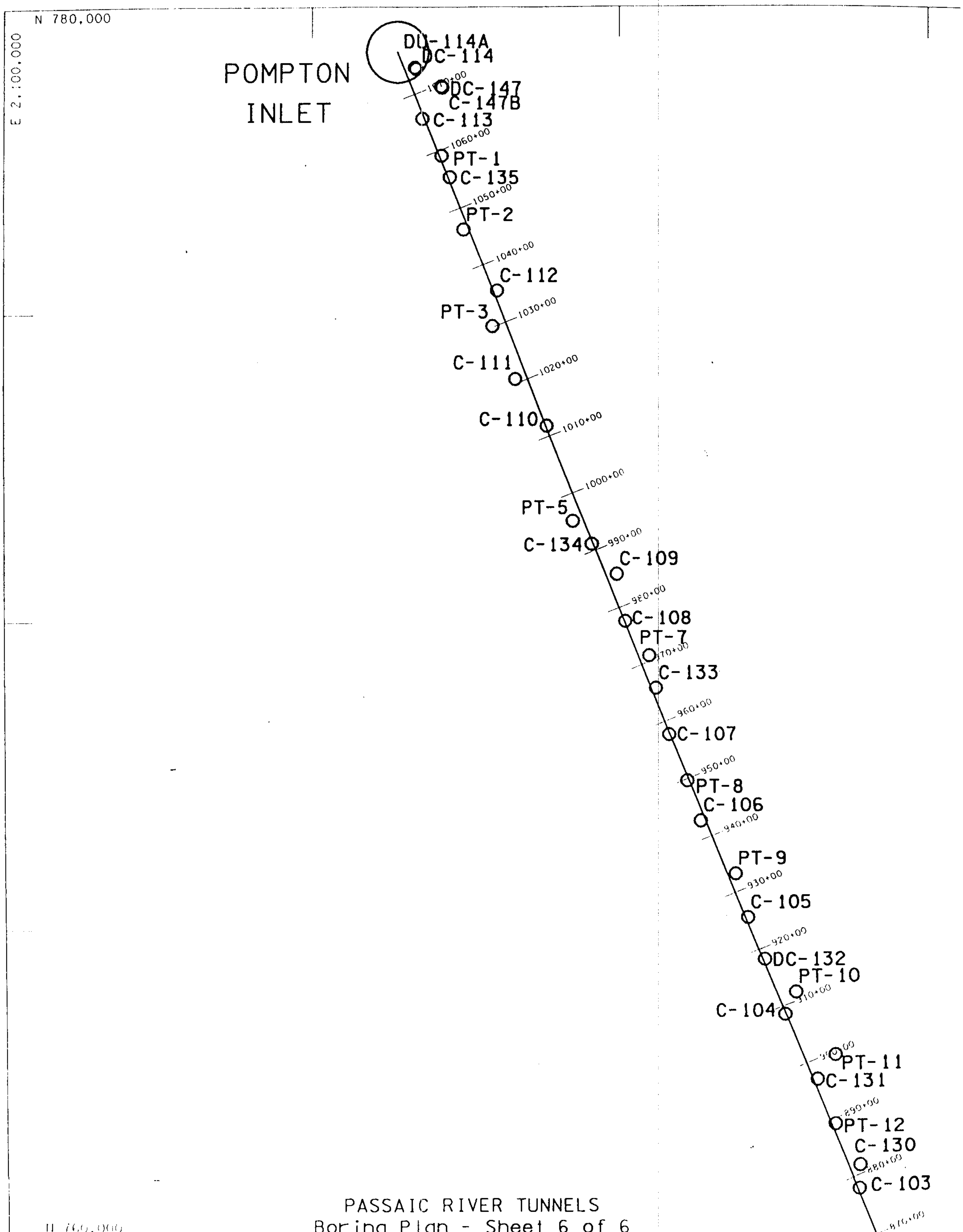
PASSAIC RIVER TUNNELS
Boring Plan - Sheet 3 of 6



PASSAIC RIVER TUNNELS
Boring Plan - Sheet 4 of 6



PASSAIC RIVER TUNNELS
Boring Plan - Sheet 5 of 6



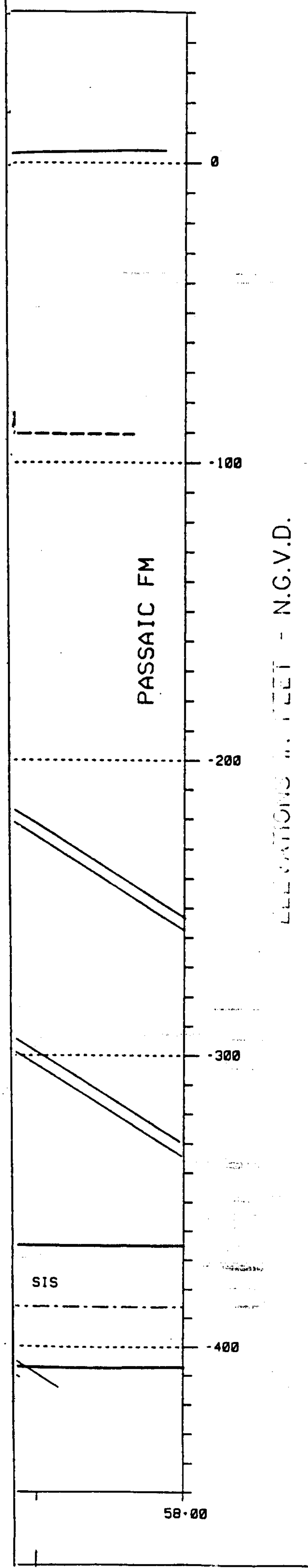
PASSAIC RIVER TUNNELS
Boring Plan - Sheet 6 of 6

PASSAIC RIVER FLOOD
DAMAGE REDUCTION PROJECT

ATTACHMENT E.3.3

TUNNEL GEOLOGIC PROFILES

(NOTE: GEOLOGIC PROFILES START WITH DWG. NO. E.3.2)



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. -2+00 TO 58+00

REVISED:		
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT	JS/RU

DRAWING NO. E.3.2

U.S. ARM

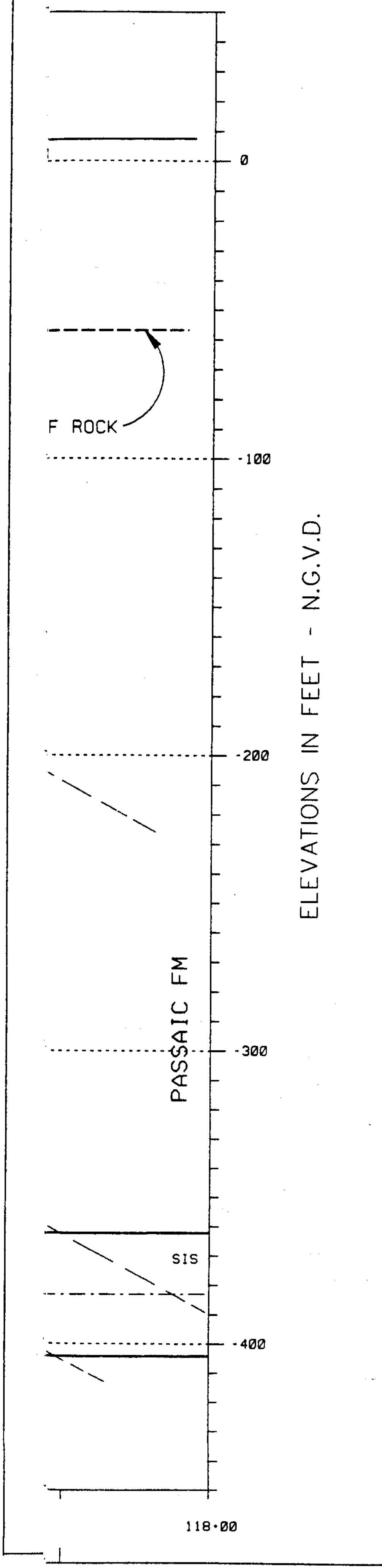
Passaic River Flood D

GENERAL D

GEOL
ALONG FLO
MAIN TUNNEL

REVISED:

DATE: JAN 1995 | PREPARE



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 58+00 TO 118+00

REVISED:	
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DRAWING NO. E.3.3

U.S. ARM

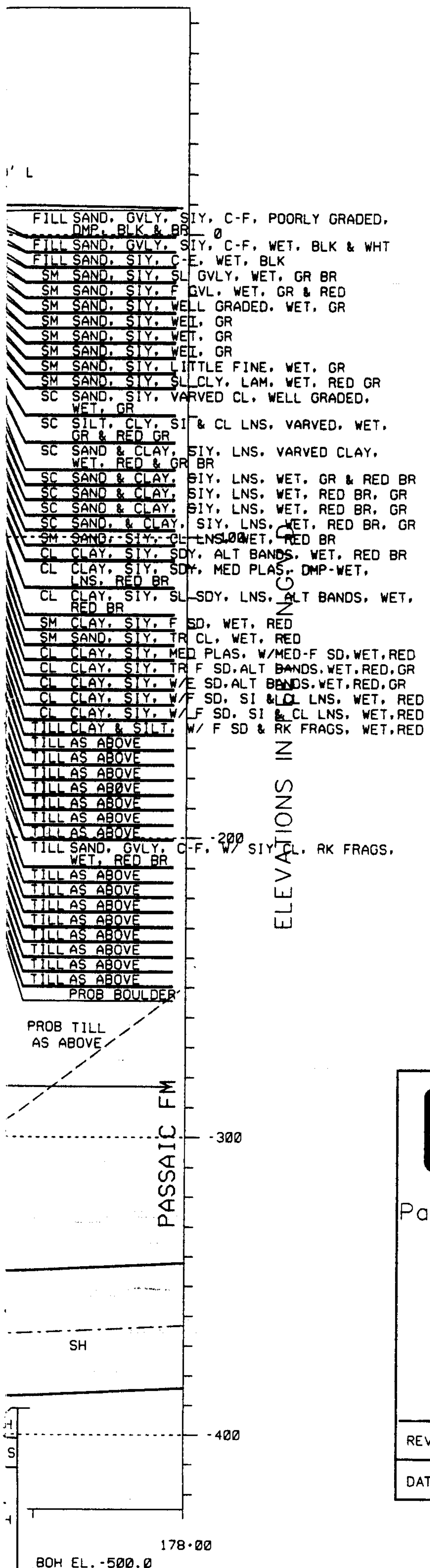
Passaic River Flood

GENERAL

GEOL
ALONG FLO
MAIN TUNNEL

REVISED:

DATE: JAN 1995 | PREPARE



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA. 118+00 TO 178+00

REVISED:		
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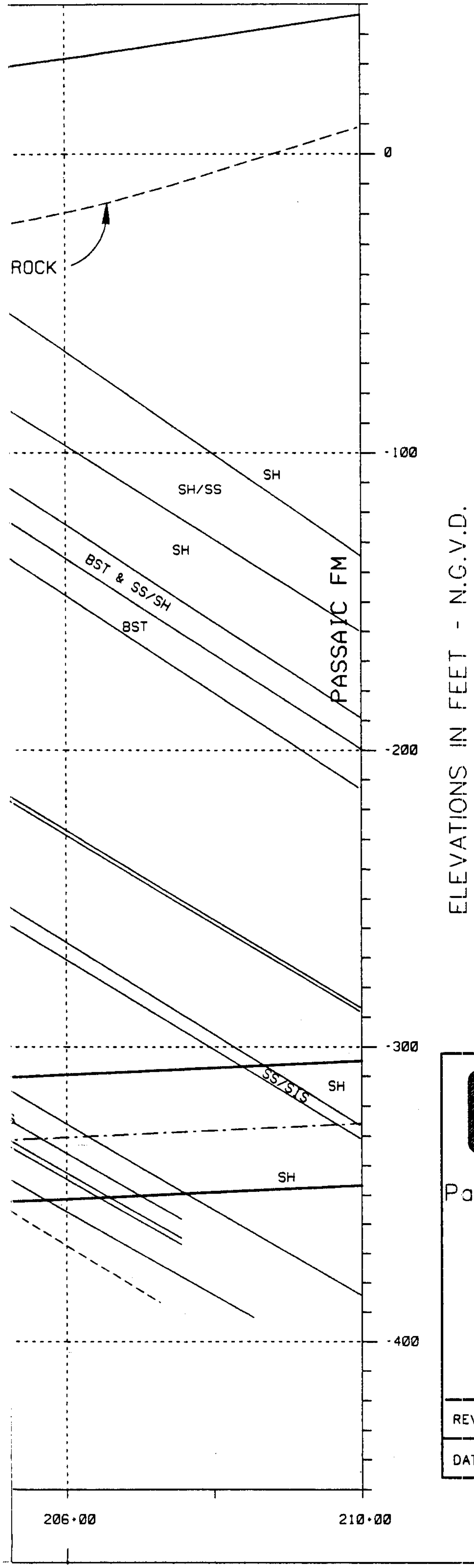
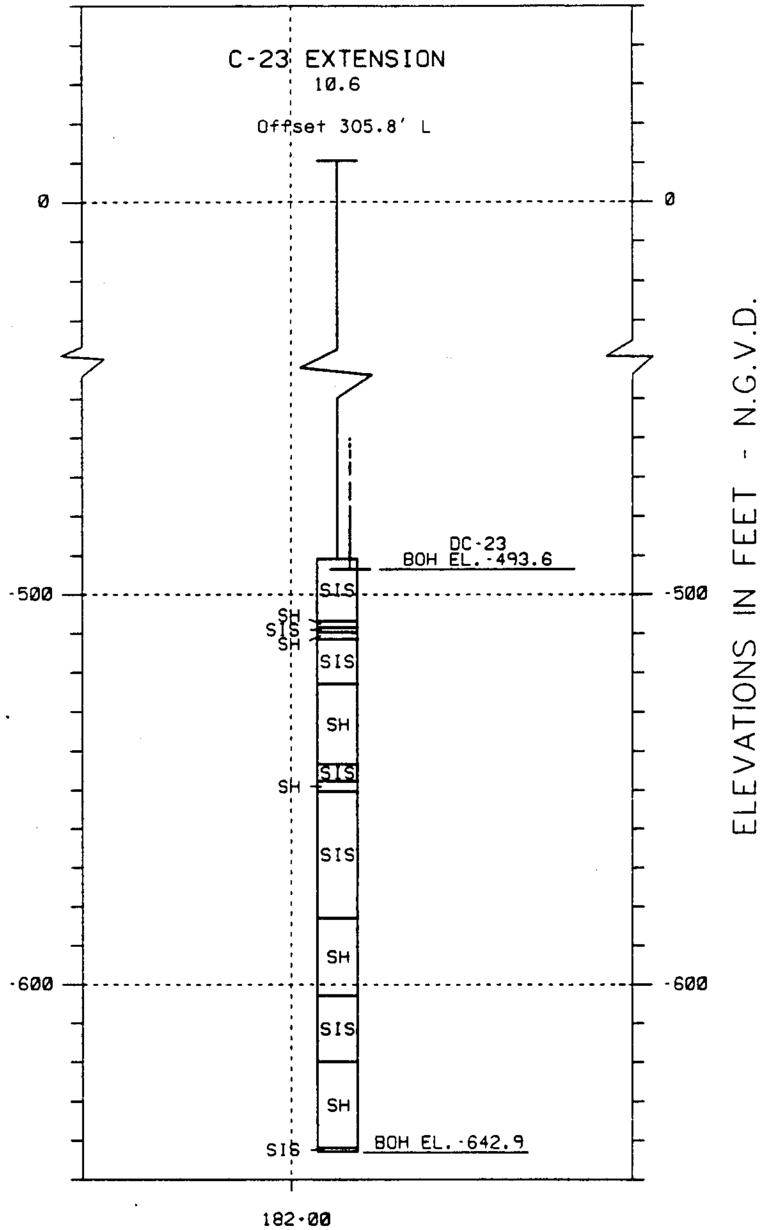
DRAWING NO. E.3.4

ELEVATIONS IN FEET - N.G.V.D.
PASSAIC FM
APPROXIMATE TOP OF GROUND
APPROXIMATE TOP OF ROCK
C-19
8.6
Offset 443.41' L
03/12/91
PR-4CB
10.0
Offset 170.96' R
07/14/90
WORK SHAFT #28
(45' ID)
STA. 163+15
PR-7
10.0
Offset 0.0'
DC-21
6.2
Offset 8.0' L
PR-5CA
10.0
PR-5C-90
10.0
Offset 486.04' L
DC-22
10.0
Offset 452.1' L
10/24/94
TUNNEL CENTER LINE
TUNNEL INVERT
SLOPE - 0.0005
SLOPE - 0.0117
INV. -401.88
BOH EL. -504.0
BOH EL. -482.2
BOH EL. -500.0
BOH 205.5'
PROB TILL
AS ABOVE
PROB BOULDER
SIS
SS
SH
SH/SIS
SS/SH
SI/SS
0
-100
-200
-300
-400
118+00
122+00
126+00
130+00
134+00
138+00
142+00
146+00
150+00
154+00
158+00
162+00
166+00
170+00
174+00
178+00
U.S. ARMY
Passaic River Flood D
GENERAL D
GEOL
ALONG FLOO
MAIN TUNNEL
REVISED
DATE: JAN 1995
PREPARE





U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

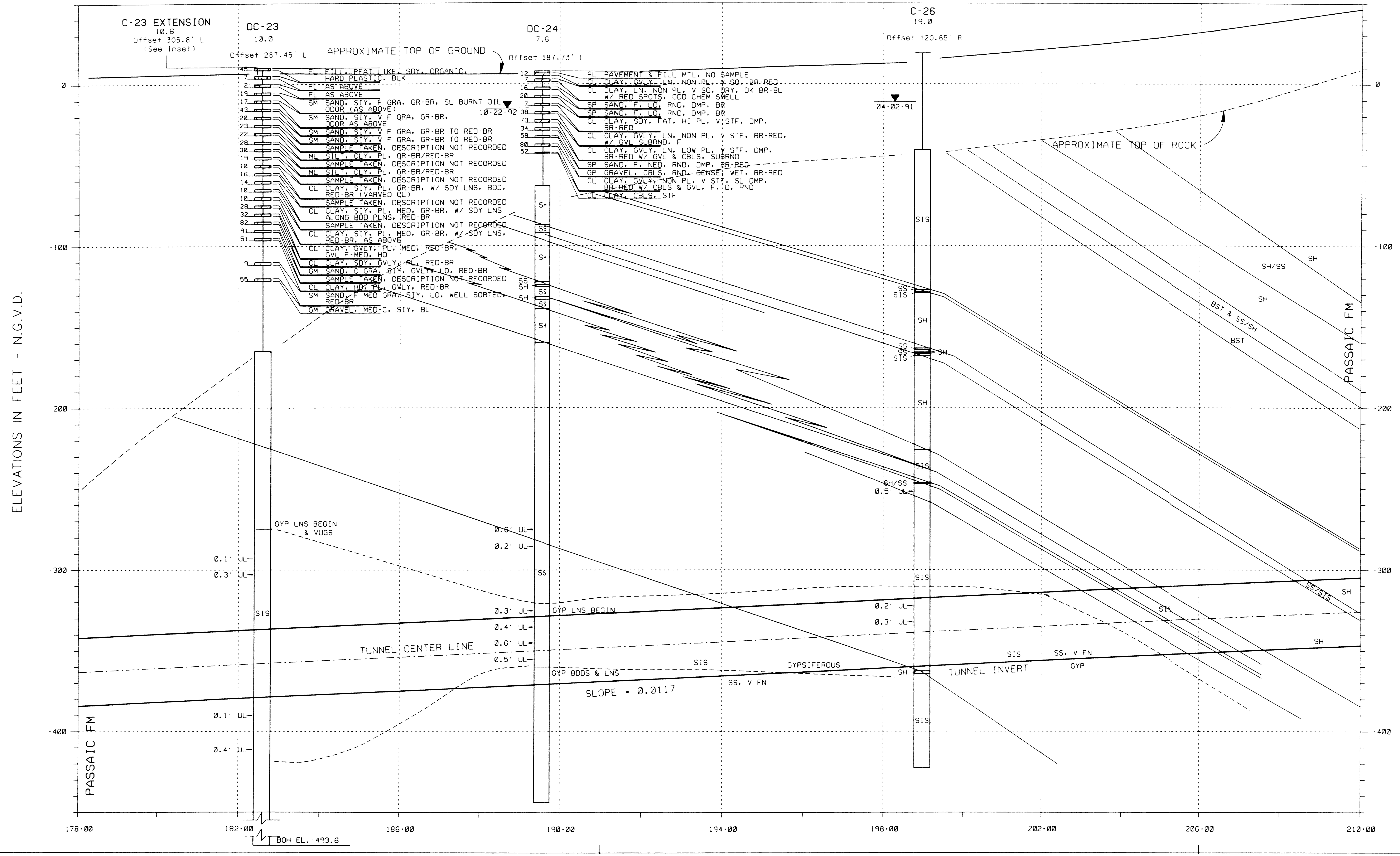
Passaic River Flood Damage Reduction Project

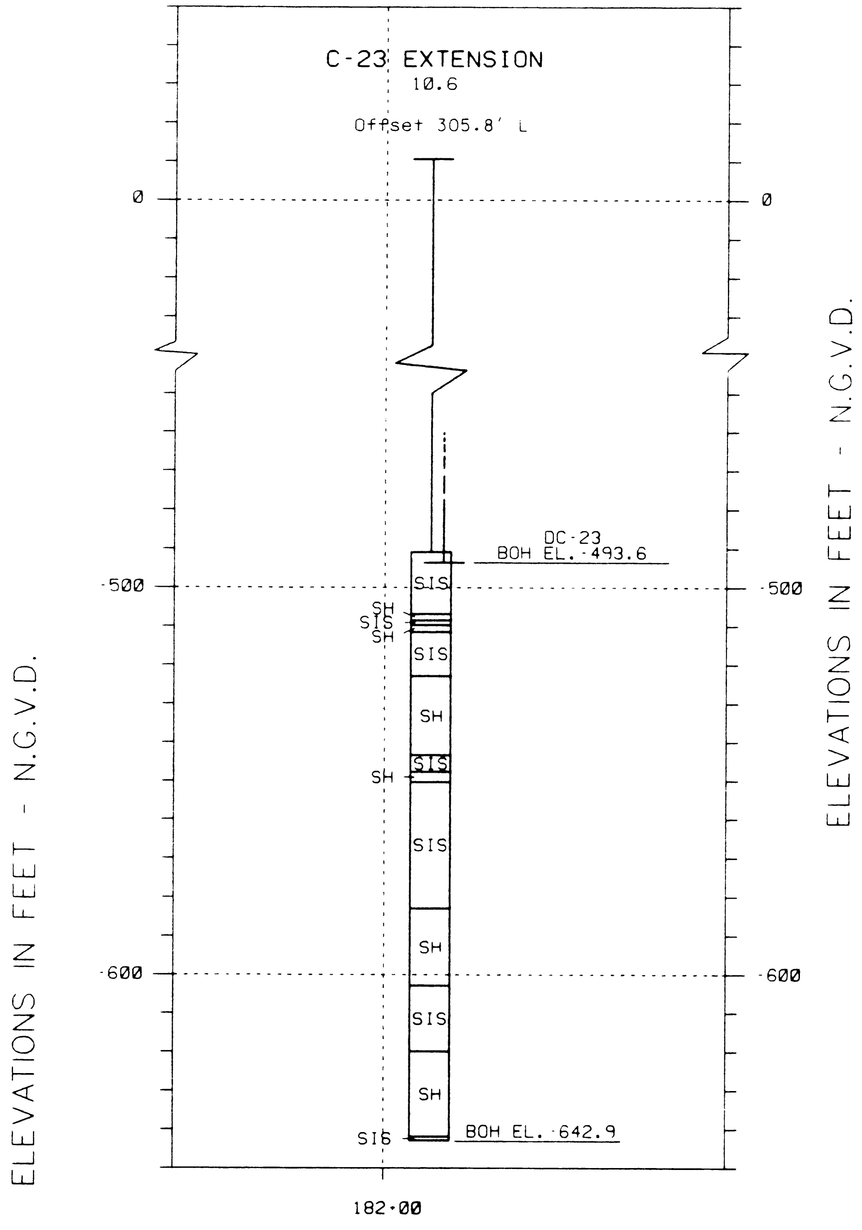
GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 178+00 TO 210+00

REVISED:	
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT JS/RU

DRAWING NO. E.3.5





U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

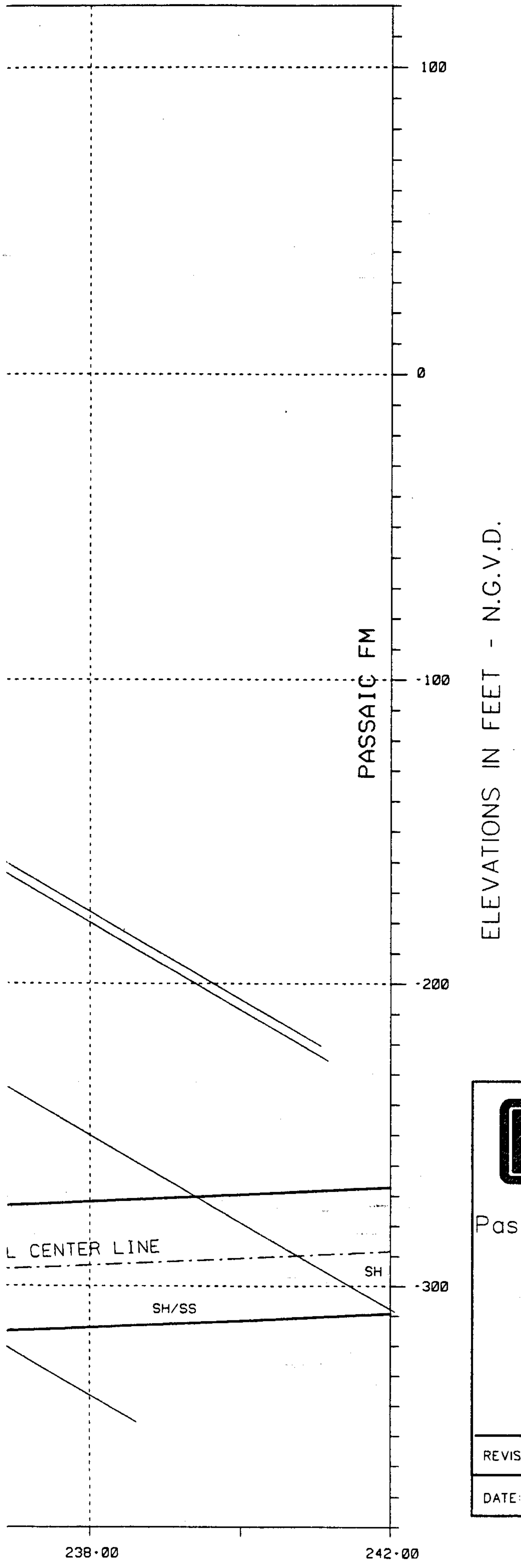
GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA. 178+00 TO 210+00

REVISED:		
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT	JS/RU

DRAWING NO. E.3.5



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

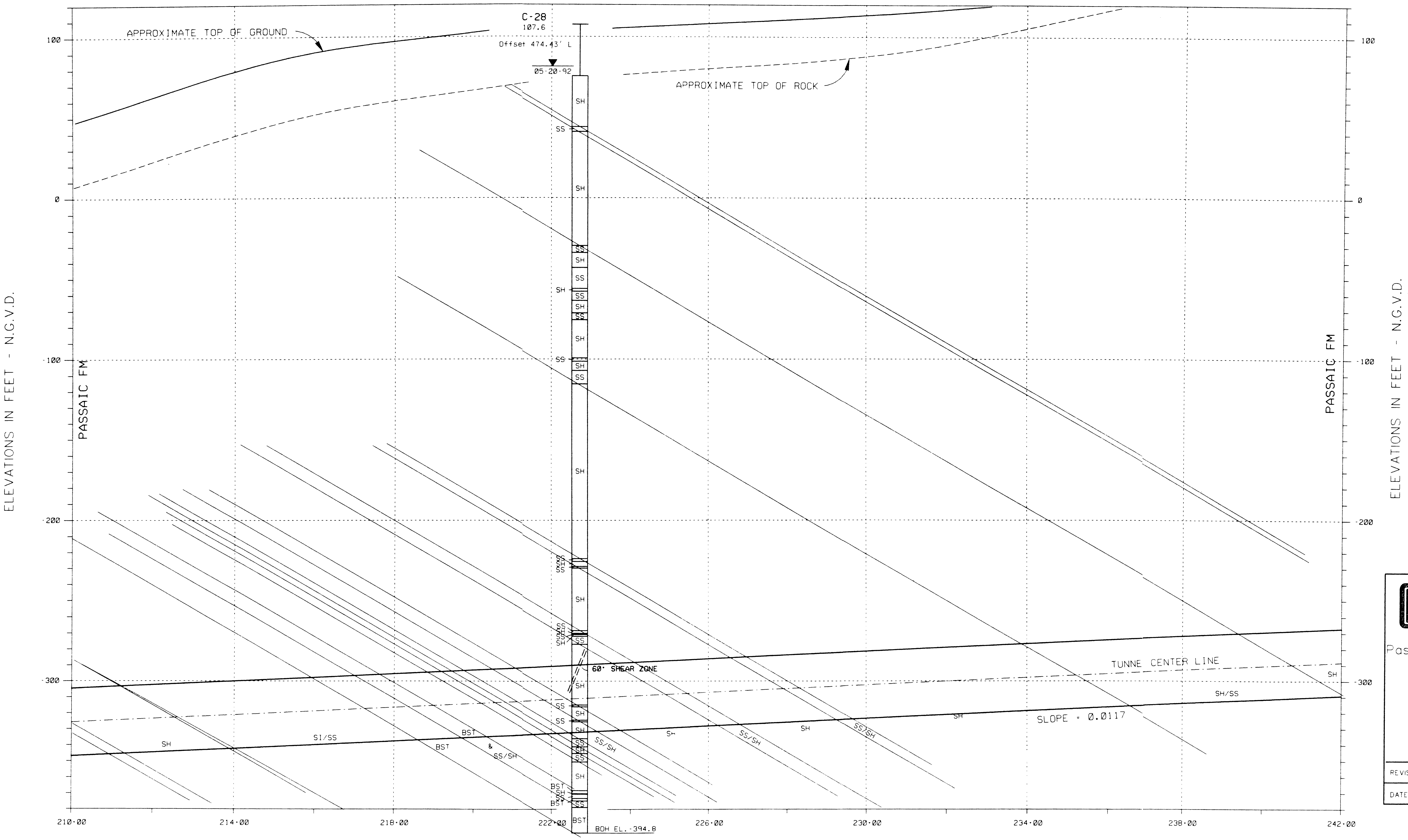
GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 210+00 TO 242+00

REVISED:

DATE: JAN 1995 | PREPARED BY: NASHVILLE DISTRICT | JS/RU

DRAWING NO. E.3.6



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

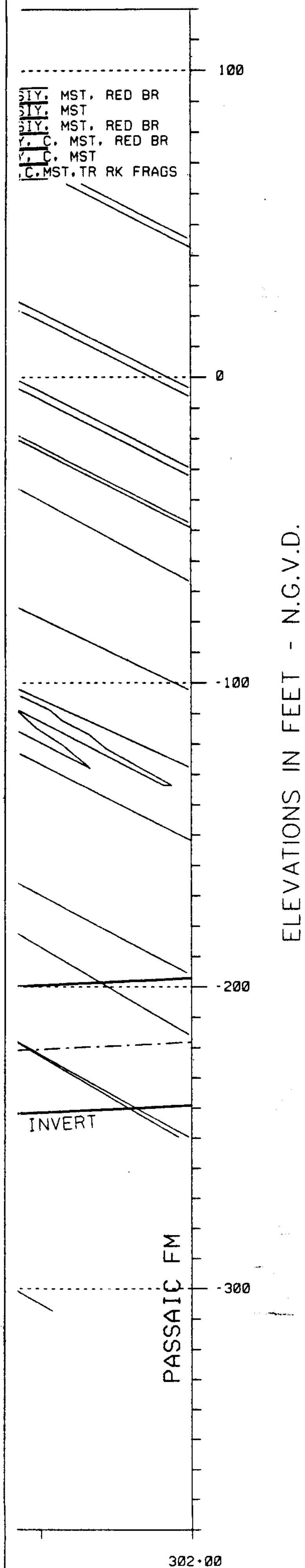
Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 210+00 TO 242+00

REVISED		
DATE JAN 1995	PREPARED BY: NASHVILLE DISTRICT	JS/RU

DRAWING NO. E.3.6



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA. 242+00 TO 302+00

REVISED:	
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT JS/RU

DRAWING NO. E.3.7

U.S. ARM

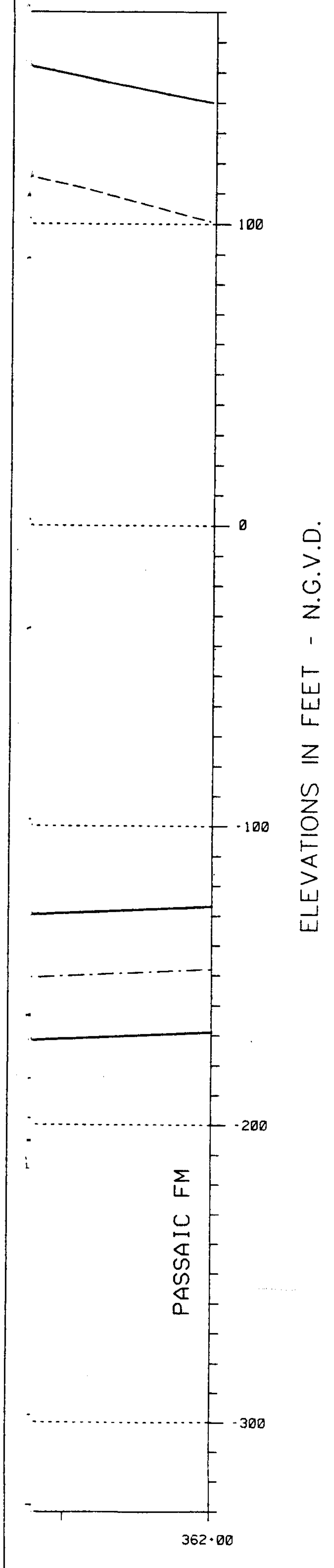
Passaic River Flood

GENERAL

GEO
ALONG FLO
MAIN TUNNEL

REVISED:

DATE: JAN 1995 PREPA



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA.302+00 TO 362+00

REVISED:	
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT JS/RU

DRAWING NO. E.3.8

U.S. ARM

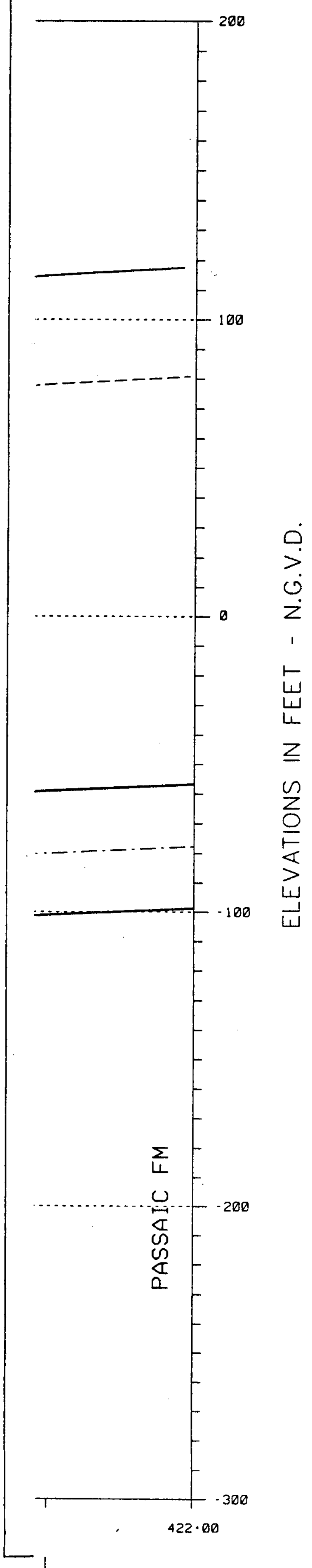
Passaic River Flood

GENERAL

GEO
ALONG FLO
MAIN TUNNEL

REVISED:

DATE: JAN 1995	PREPAR



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

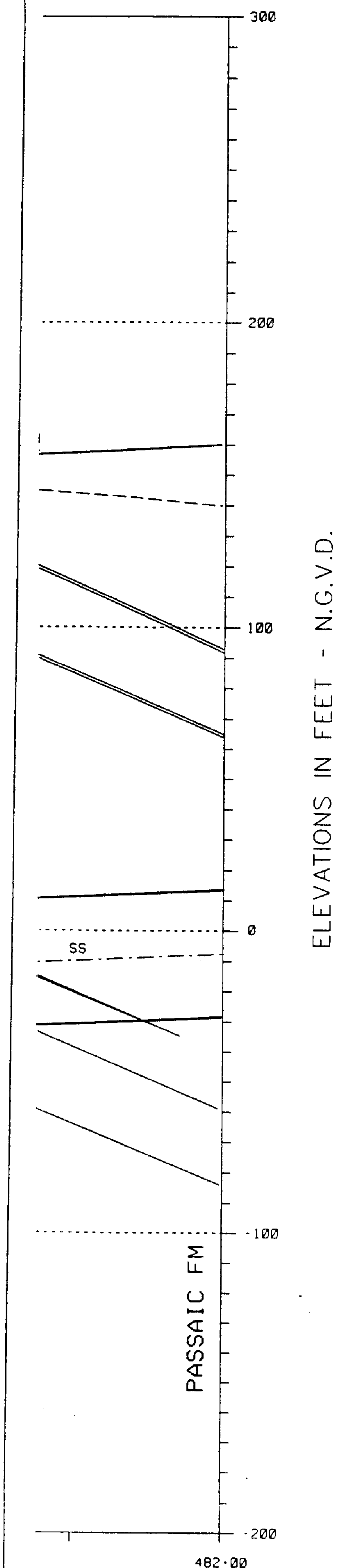
GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA.362+00 TO 422+00

REVISED:		
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT	JS/RU

DRAWING NO. E.3.9


ELEVATIONS IN FEET - N.G.V.D.
EMERGENCY SHAFT 2A
(NAN REVISION DATED JAN 27, 1995)
DC-48
130.5
Offset 47.0' R
10/17/94
ML SILT, CLY, MED D, BR-DK BR
CL CLAY, SIY, DRY, SO, MOT, RED BR-BR
SM SILT, SDY, CLY, DRY, RED BR, BECOMES GVLY WITH DEPTH
GP GRAVEL, SDY, F-MED, WET, RED BR, SUBRND-SUBANG
SM SAND, SIY, F GRA, LO, WET-SAT, BR
ML SAND, SIY, CLY, VF-F GRA, WET-SAT, BR
SM SAND-SILT, VF GRA, SL CLY, WET, BR, BECOMES MORE GVLY WITH DEPTH
DC-53
75.0
Offset 34.46' L
ASPHALT - PAVEMENT 0.2' W/ 0.3' OF ANG CBLS FOR ROAD FOUNDATION
ML SILT, SDY, SO, LOW PL, SL DMP, BR-RED
ML SILT, (FILL), MOTT, W/ MED GRA SD, NO PATTERN, HD
ML SILT, (FILL), WET, SO
ML SILT, SDY, GVLY, (FILL CONT.)
SM SAND, SIY, GVLY, V F GRA, BR-RED, WET, GVL SUBRND, C GRA, MED D.
ML SILT, LOW PL, SDY, BR-RED, V STF, WET, W/ SUBANG F GRA-C GRA GVL
SM SAND, SIY, F-MED GRA, D, SUBRND, WET
SM SAND, SIY, W/ SUBRND-ANG, F-C GRA GVL
GM GRAVEL, F-C GRA, D, ANG, WET, BR-RED, BST, SS, SIY, SDY
ATTEMPTED SAMPLE - NO PENETRATION
APPROXIMATE TOP OF GROUND
APPROXIMATE TOP OF ROCK
NO DRILLING
TUNNEL CENTER LINE
TUNNEL INVERT
SLOPE - 0.0117
PASSAIC FM
362+00
366+00
370+00
374+00
378+00
382+00
386+00
390+00
394+00
398+00
402+00
406+00
410+00
414+00
418+00
422+00
U.S. ARM
Passaic River Flood
GENERAL
GEO
ALONG FLO
MAIN TUNNEL
REVISED
DATE: JAN 1995
PREPAR



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA.422+00 TO 482+00

REVISED:		
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DRAWING NO. E.3.10

U.S. ARM

Passaic River Flood D

GENERAL D

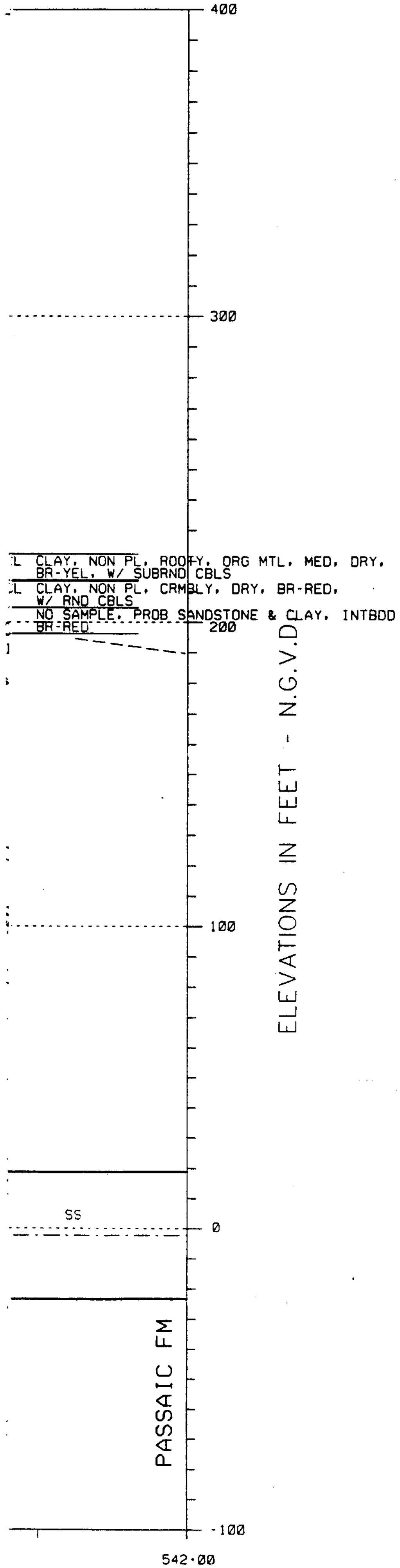
GEOL
ALONG FLOO

MAIN TUNNEL S

REVISED

DATE: JAN 1995

PREPARED



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA.482+00 TO 542+00

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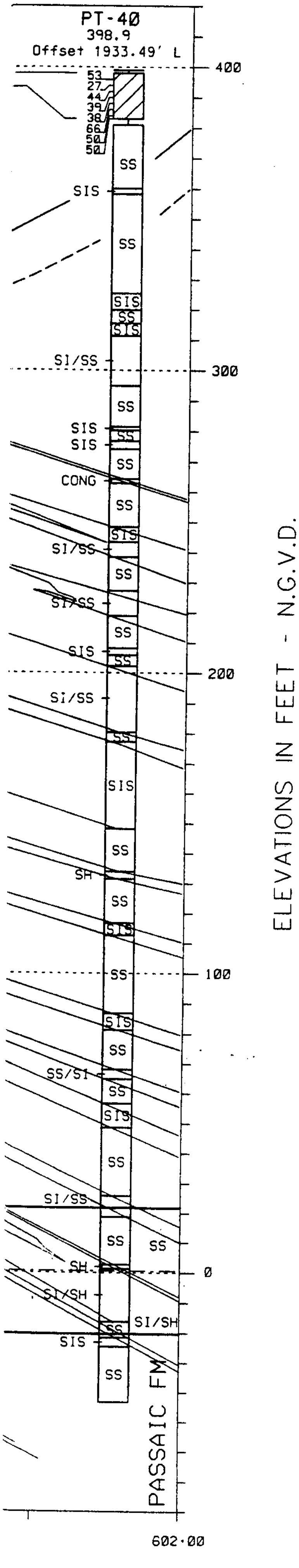
U.S. ARM

Passaic River Flood D

GENERAL D

GEOL
ALONG FLOO
MAIN TUNNEL

REVISED	
DATE JAN 1995	PREPARE



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA.542+00 TO 602+00

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DRAWING NO. E.3.12

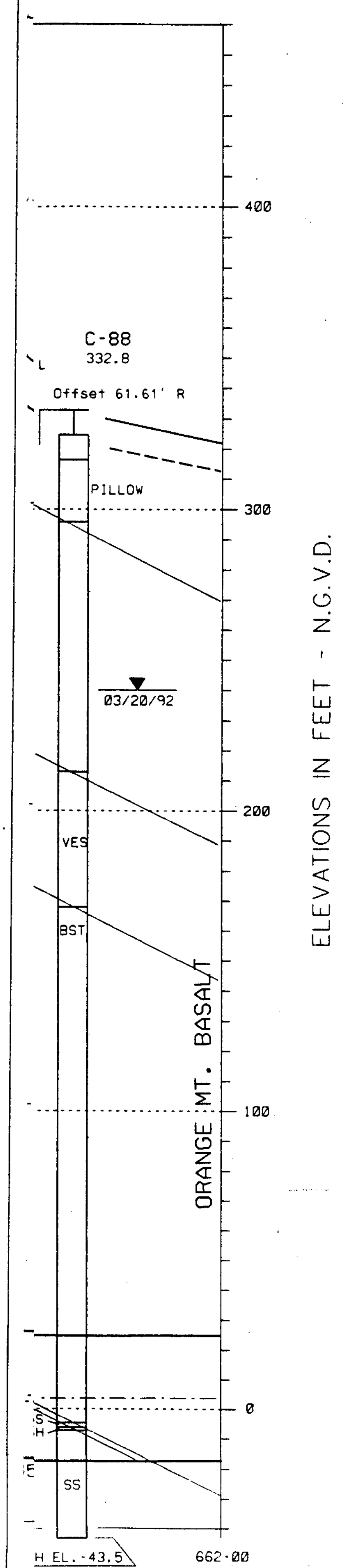
U.S. ARM

Passaic River Flood D

GENERAL D

GEOL
ALONG FLO
MAIN TUNNEL S

REVISED	
DATE JAN 1995	PREPARE



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA.602+00 TO 662+00

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DRAWING NO. E.3.13

U.S. ARM

Passaic River Flood

GENERAL

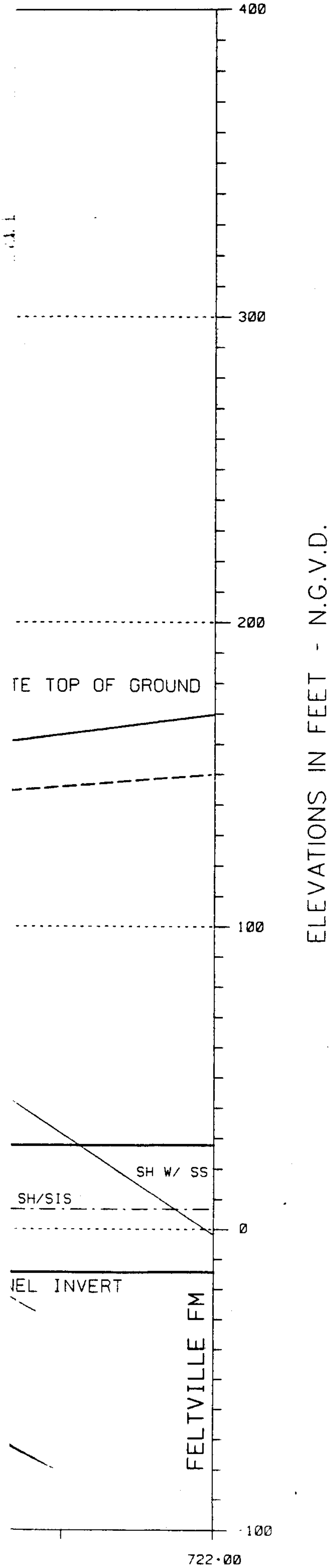
GEO

ALONG FLO

MAIN TUNNEL

REVISED

DATE JAN 1995



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA.662+00 TO 722+00

REVISED:	
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DRAWING NO. E.3.14

U.S. AR

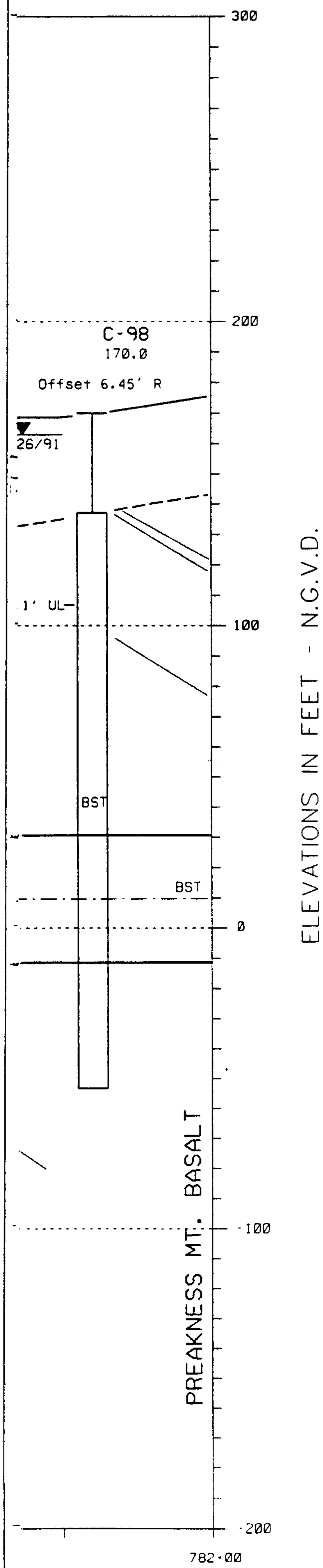
Passaic River Flood

GENERAL

GEO
ALONG FLO
MAIN TUNNEL

REVISED

DATE: JAN 1991 PREPA



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA.722+00 TO 782+00

REVISED:

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DRAWING NO. E.3.15

U.S. ARM

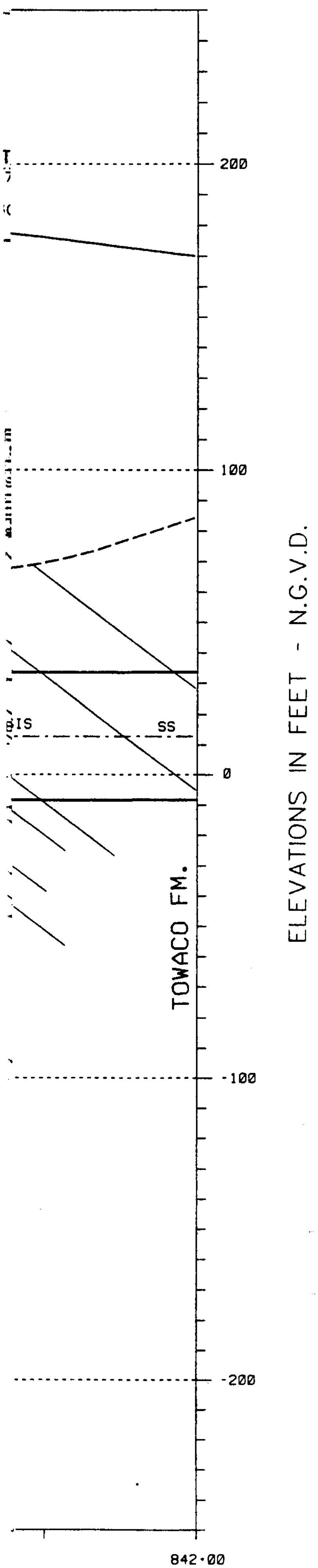
Passaic River Flood

GENERAL

GEO
ALONG FLO
MAIN TUNNEL

REVISED

DATE JAN 1995 PREPAR



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA.782+00 TO 842+00

REVISED:		
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DRAWING NO. E.3.16

U.S. ARM

Passaic River Flood

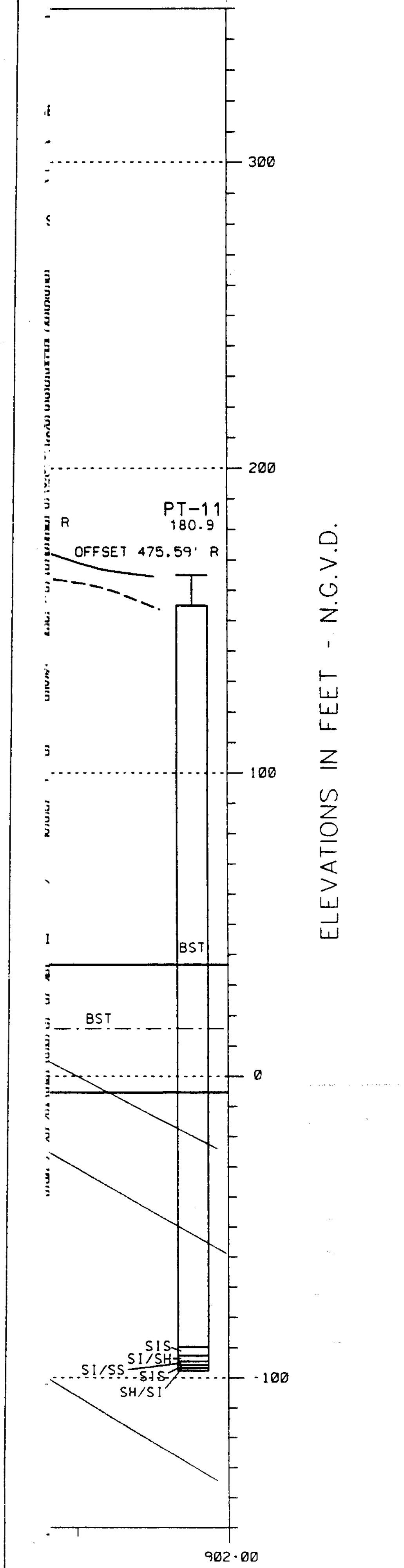
GENERAL

GEO
ALONG FLO
MAIN TUNNEL

REVISED

DATE: JAN 1995

PREPA



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

MAIN TUNNEL STA.842+00 TO 902+00

REVISED:		
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DRAWING NO. E.3.17

U.S. ARM

Passaic River Flood

GENERAL

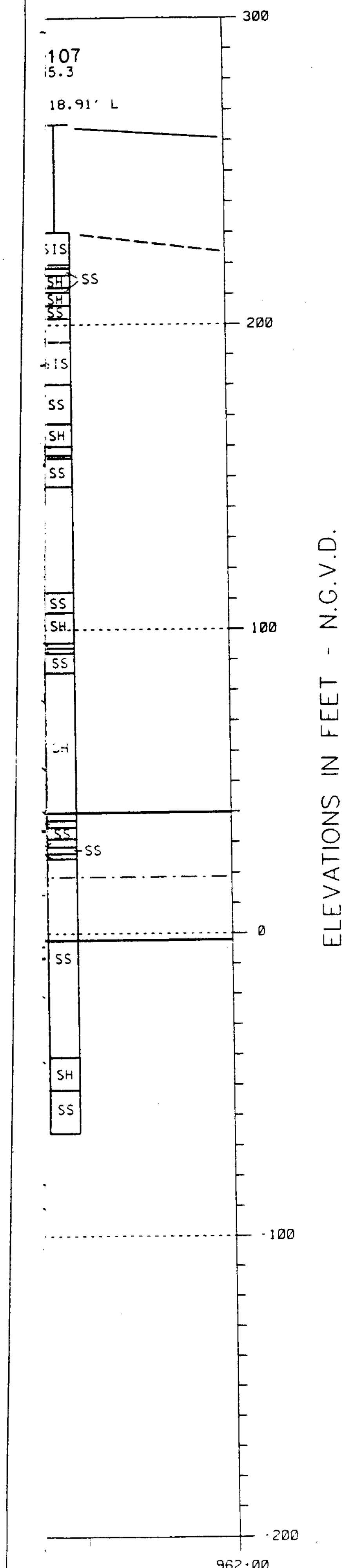
GEO

ALONG FLO

MAIN TUNNEL

REVISED

DATE: JAN 1995 | PREPAR



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 902+00 TO 962+00

REVISED:	
DATE: JAN 1995	PREPARED BY: NASHVILLE DISTRICT JS/RU

DRAWING NO. E.3.18

U.S. A

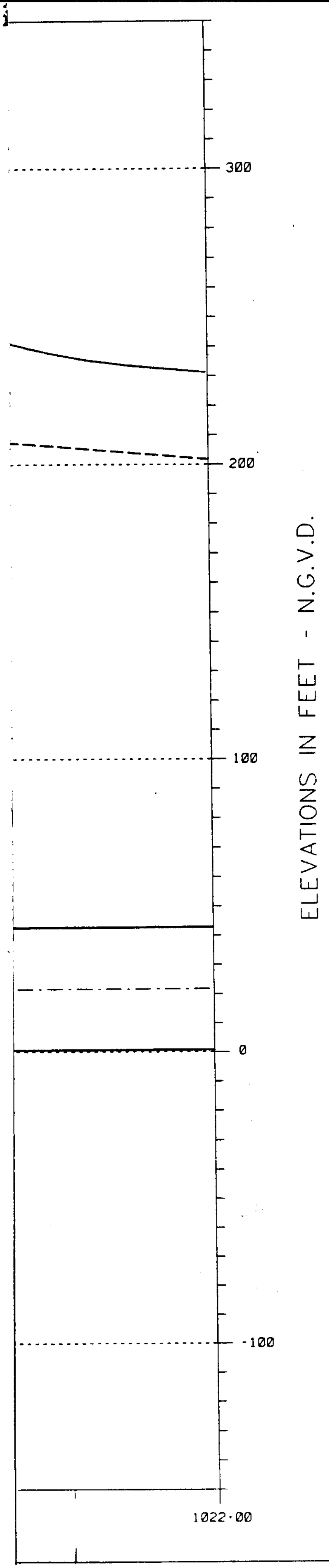
Passaic River Floo

GENERA

GE
ALONG F
MAIN TUNNE

REVISED

DATE JAN 1995



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 962+00 TO 1022+00

REVISED:	
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DRAWING NO. E.3.19

U.S.

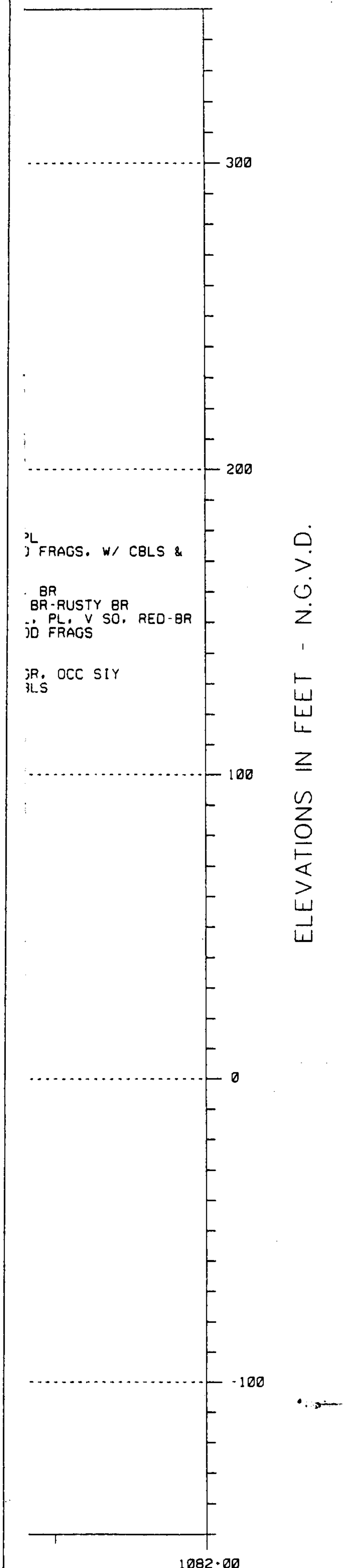
Passaic River Floo

GENERA

G
ALONG F
MAIN TUNNEL

REVISED

DATE JAN 1995 PR



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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
MAIN TUNNEL STA. 1022+00 TO 1082+00

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DRAWING NO. E.3.20

U.S. ARM

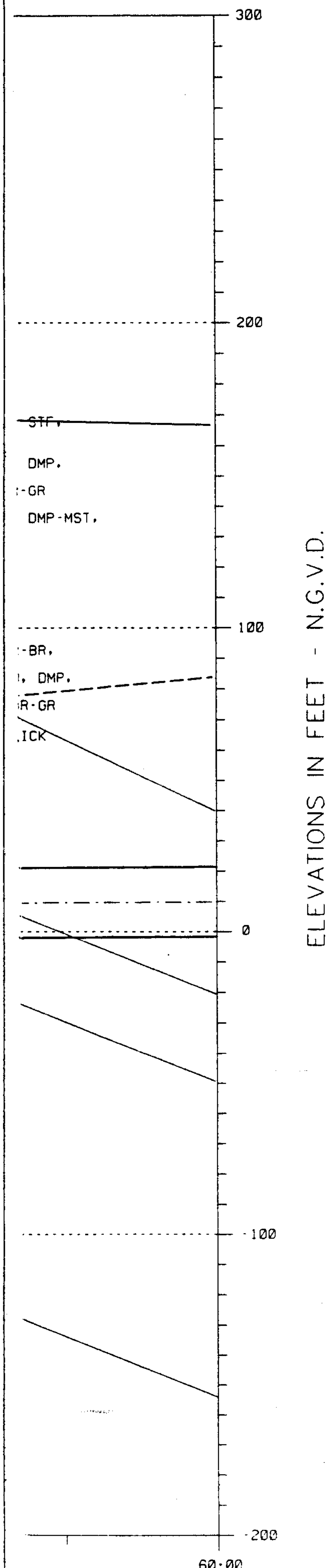
Passaic River Flood

GENERAL

GEO
ALONG FLO
MAIN TUNNEL S

REVISED:

DATE: JAN 1995 | PREPAR



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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
SPUR TUNNEL STA. 0+00 TO 60+00

REVISED:	
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DRAWING NO. E.3.21

U.S. AR

Passaic River Flood

GENERAL

GEO
ALONG FLO
SPUR TUNNE

REVISED

DATE JAN 1995 PREPAR

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Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS

SPUR TUNNEL STA. 60+00 TO 76+00

REVISED:	
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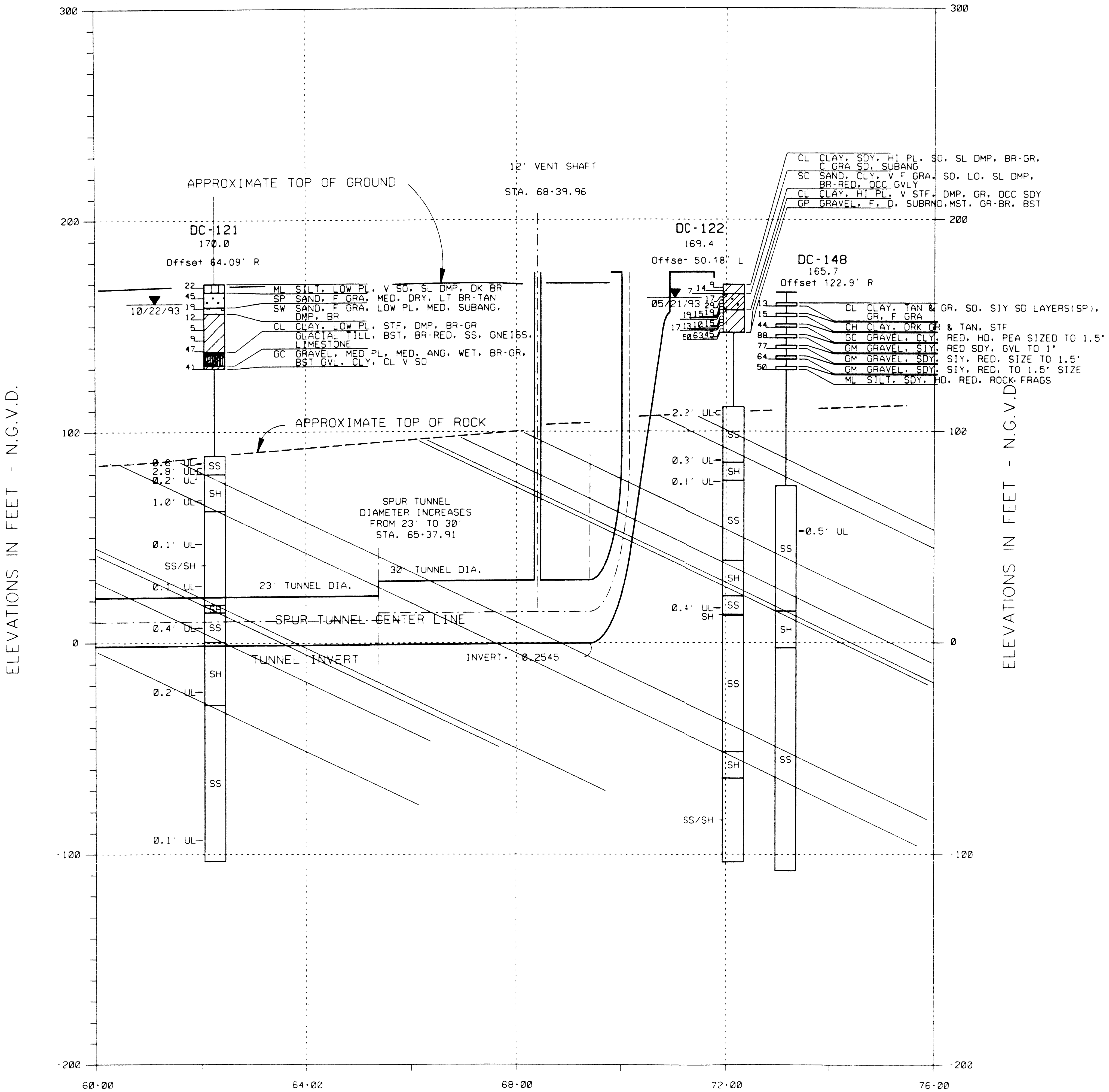
DRAWING NO. E.3



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PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project

GENERAL DESIGN MEMORANDUM

GEOLOGIC PROFILES
ALONG FLOOD DIVERSION TUNNELS
SPUR TUNNEL STA. 60+00 TO 76+00

REVISED:		
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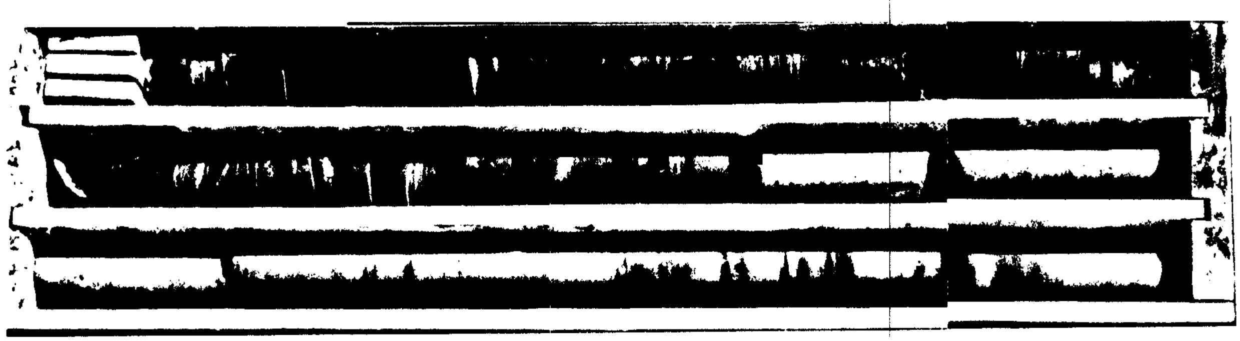
DRAWING NO. E.3.22

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

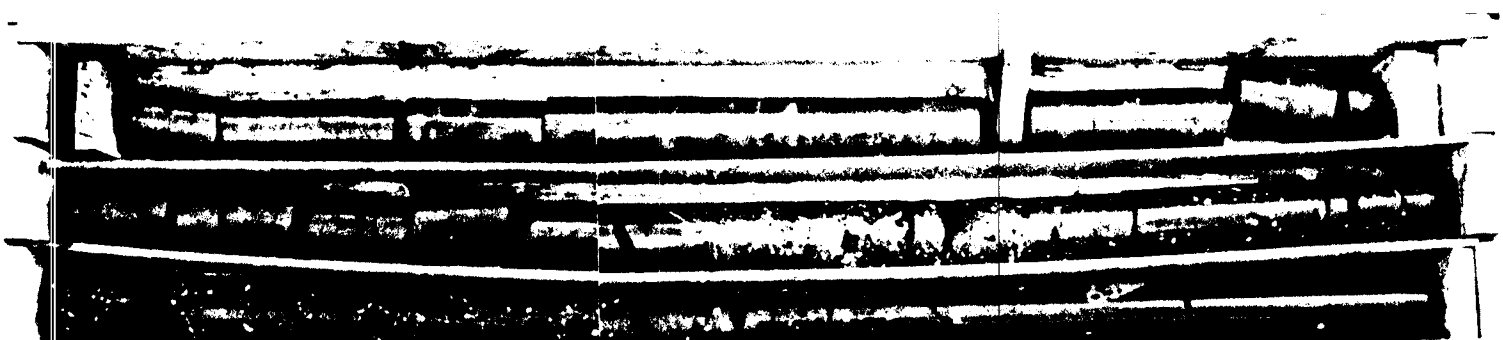
ATTACHMENT E.3.4

ROCK CORE PHOTOGRAPHS AND THIN SECTION MICROGRAPHS

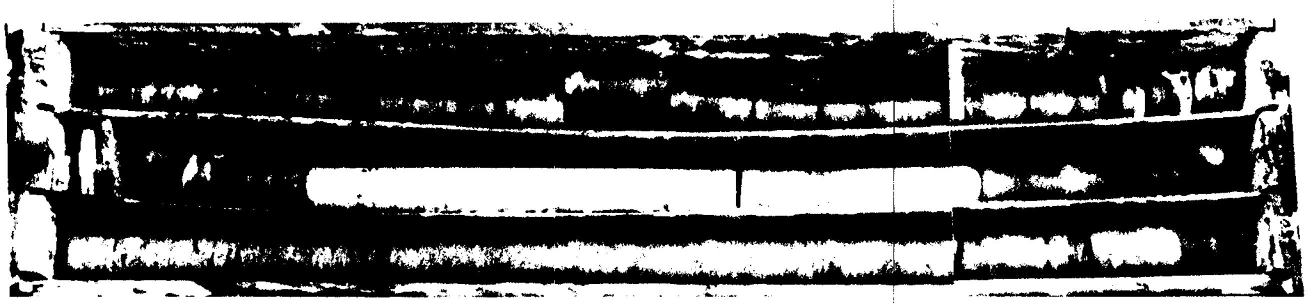
PASSAIC RIVER PROJECT
ROCK CORE PHOTOGRAPHS



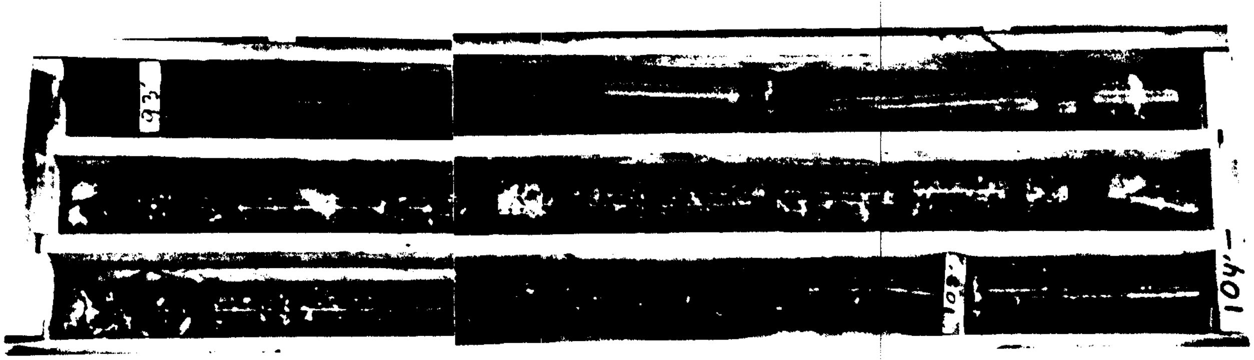
BOONTON FORMATION Inter-bedded sandstone and shale



CONTACT OF BOONTON FM. w/ HOOK MT. BASALT Sandstone – Vesicular basalt.

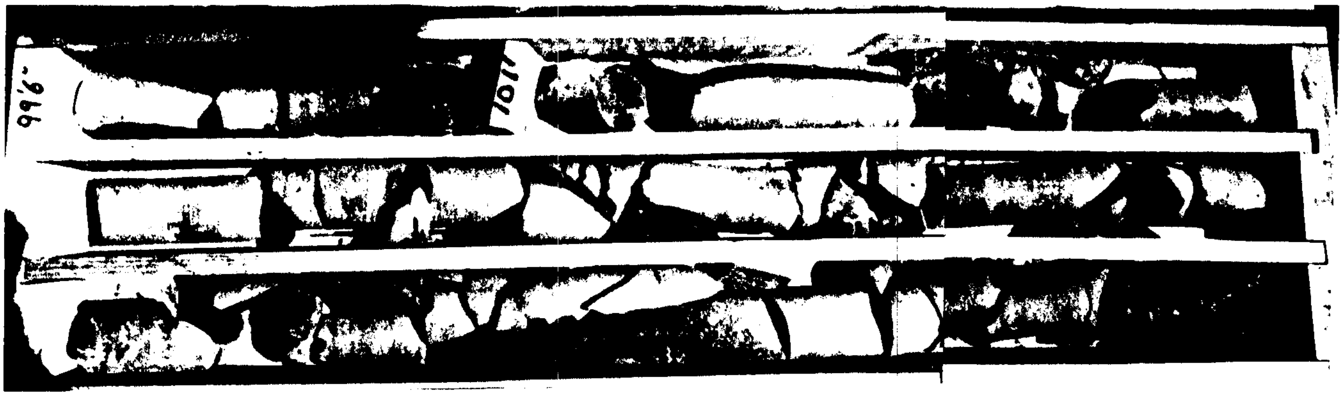


TOWACO FORMATION Inter-bedded sandstone and shale.

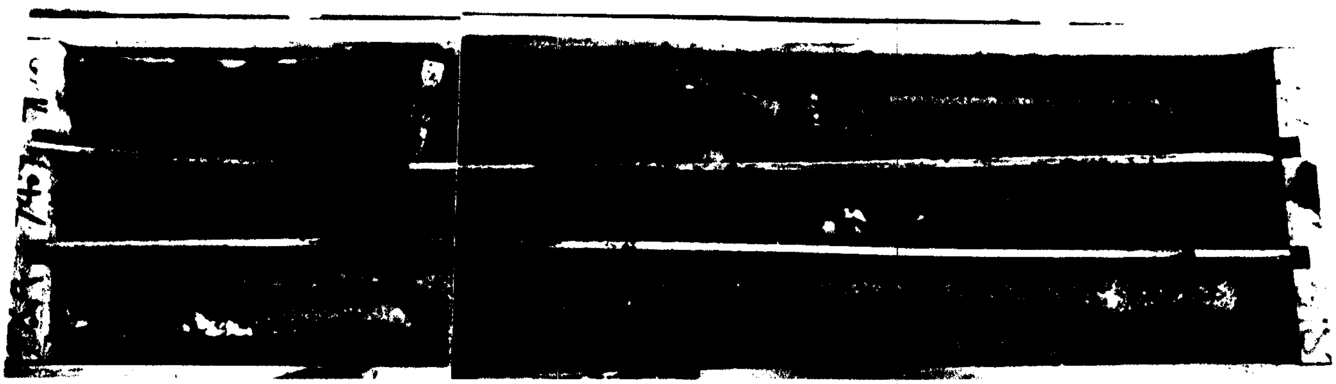


CONTACT OF TOWACO FM. w/ PREAKNESS MT. BASALT Sandstone – weathered basalt.

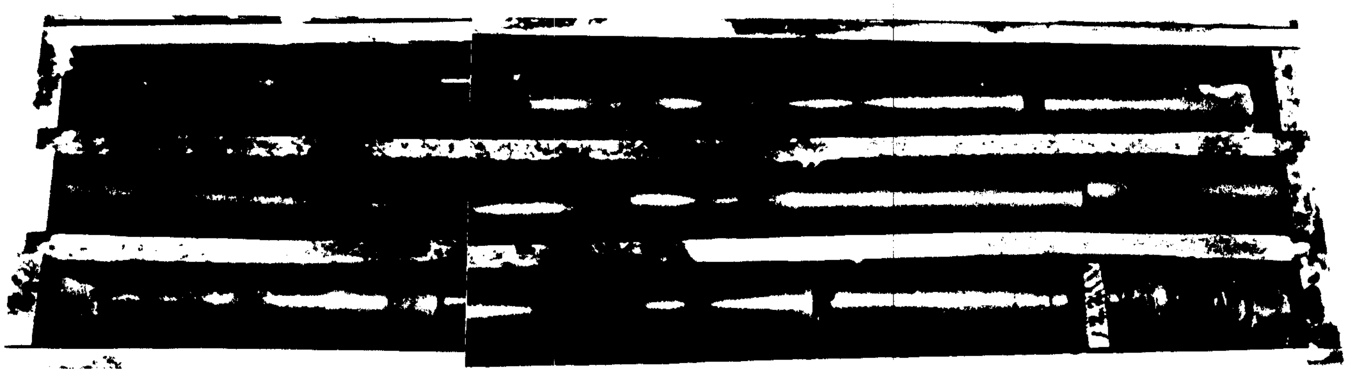
PASSAIC RIVER PROJECT
ROCK CORE PHOTOGRAPHS



PREAKNESS MT. BASALT Dense basalt showing frequency of cooling joints.

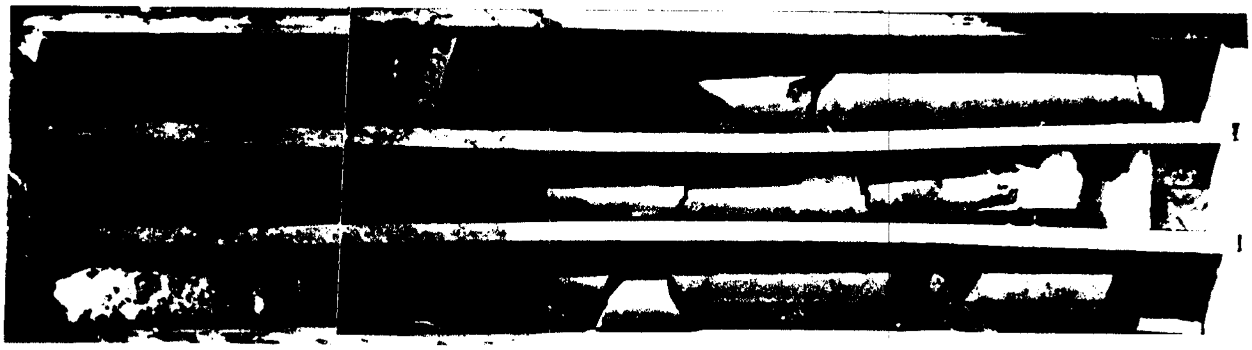


PREAKNESS MT. BASALT Dense and coarse crystalline basalt.



FELTVILLE FORMATION Sandstone w/ occasional shale bands.

PASSAIC RIVER PROJECT
ROCK CORE PHOTOGRAPHS



ORANGE MT. BASALT   Pillow basalt showing mineralization between pillows.

FAULT ZONE IN PASSAIC FORMATION   Highly mineralized fault zone in Passaic sandstone. Mineralization consists mainly of calcite w/ accessory quartz and zeolites.

PASSAIC FORMATION   Sandstone w/ corse grained arkosic bands.

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS
611 SOUTH COBB DR.
MARIETTA, GEORGIA 30060-3112

PETROGRAPHIC REPORT

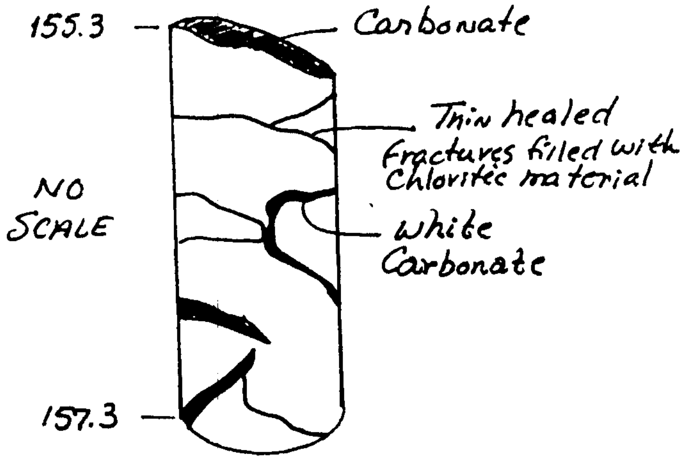
PROJECT: Passaic Tunnel Project

BORING NO: DC 115

SAMPLE NO: 22

DEPTH (ft.): 155.3-157.3

SAD LAB. NO: 128/1569



ROCK DESCRIPTION

BASALT, altered, dull greenish gray (5G 4/1), amygdaloidal, hard. Color and abundant carbonate suggest altered state. Core has numerous thin, randomly distributed calcite and chlorite healed fractures. There are numerous filled amygdules (filled voids-typically 5 mm or less in size).

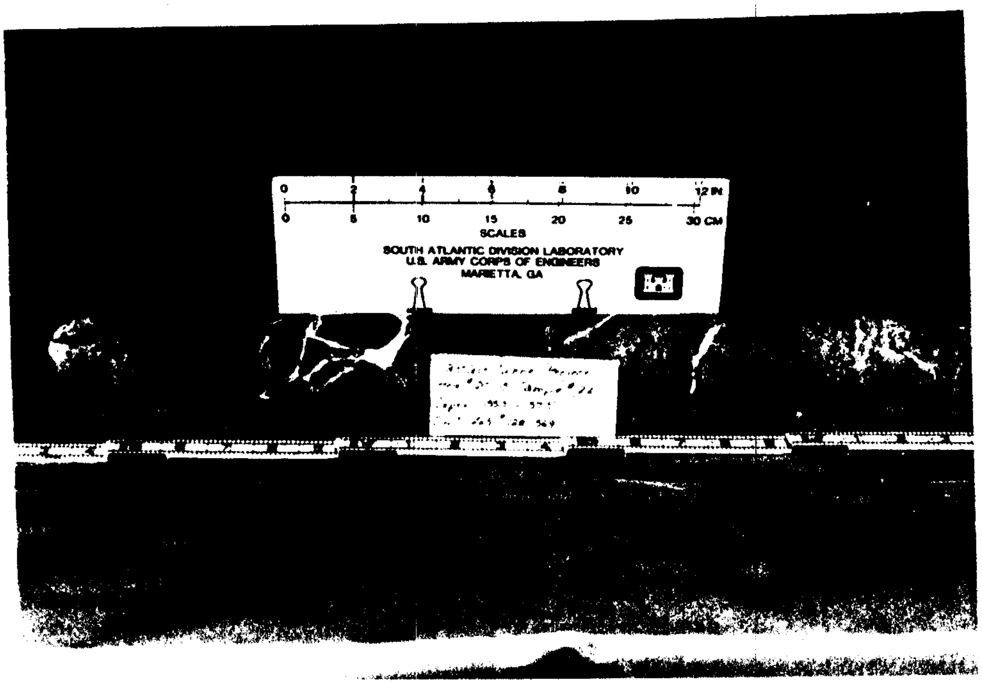


Figure 1. Boring DC 115, Sample 22, d. 155.3-157.3 ft.

Preakness Mountain Basalt, Spur Tunnel

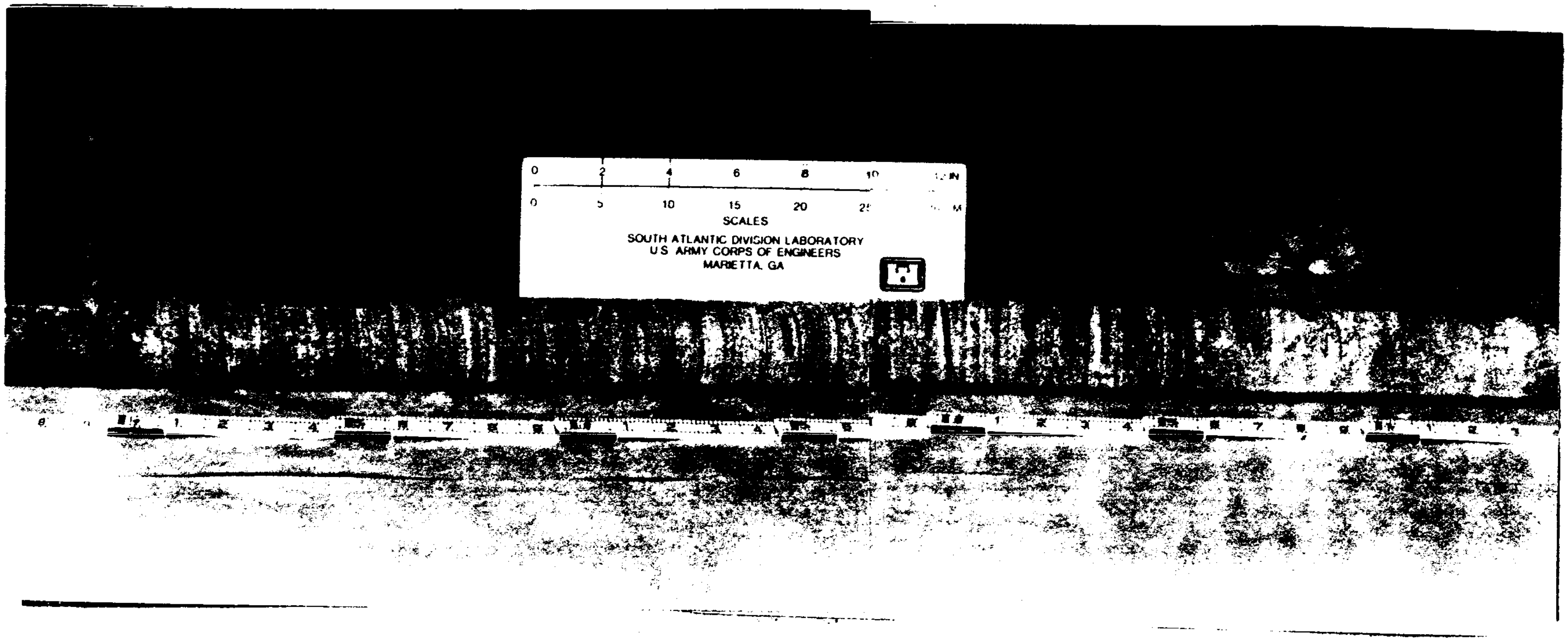
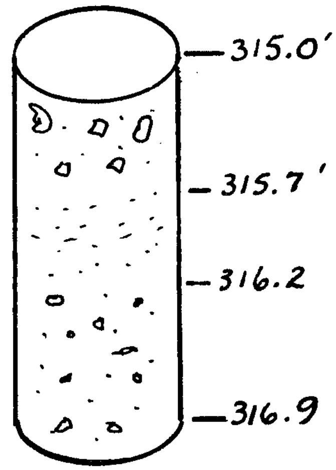


Figure 1. Boring C-59, Sample 3, Depth 311.6 to 315.4 ft.

Passaic Formation

DEPARTMENT OF THE ARMY
SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS
611 SOUTH COBB DRIVE
MARIETTA, GEORGIA 30060-3112

PETROGRAPHIC REPORT



Project: Passaic River

Boring No: C-83

Sample No: 6

Depth (ft.): 315.0-316.9

SAD Lab. No.: 128/1543

General Rock Description

SANDSTONE PEBBLE CONGLOMERATE, grayish red (5R 4/2), moderately hard to hard, fresh, sound. Poorly sorted and porous. Pebble concentration at d. 315.0 to 315.7 ft. and 316.2 to 316.9 ft. Source of much of the detrital material appears to be from a metamorphic or highly strained granitic environment. A few altered volcanic (?) rock particles are also represented.

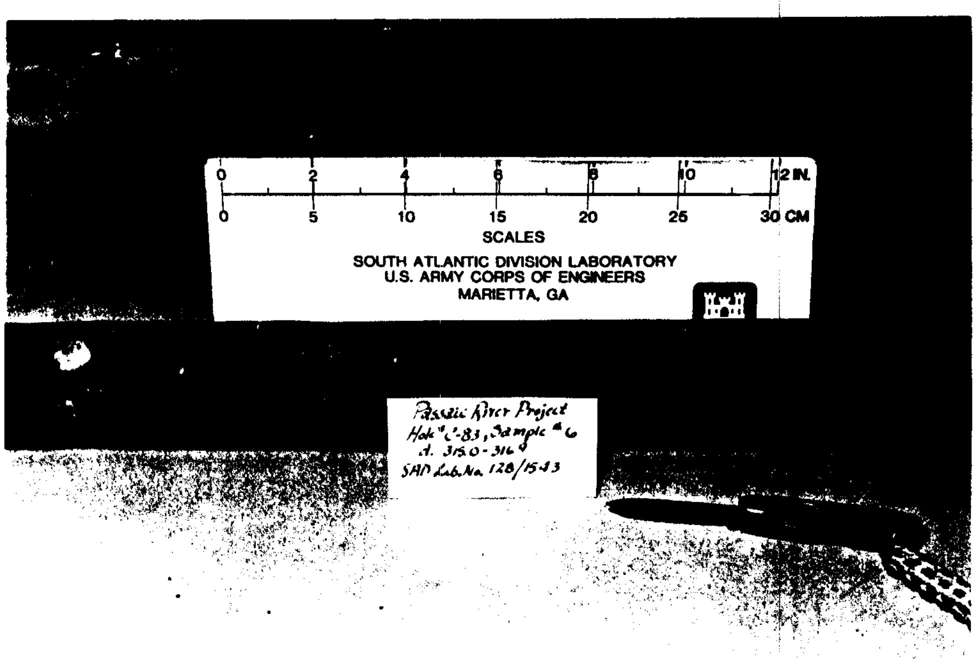


Figure 1. Top (d.315.0") is just out of view at left. Pebbles up to 1/2 inch maximum size are restricted primarily to top portion. Rock is very porous. Pebble sizes grade to coarse sandy areas.

Passaic Formation
Conglomeritic

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SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS
611 SOUTH COBB DRIVE
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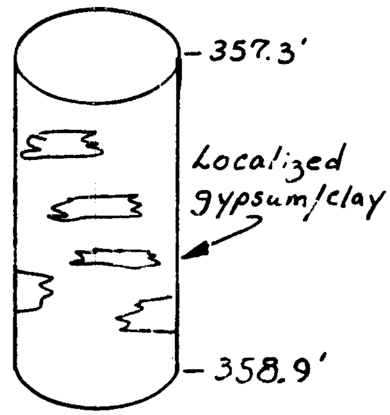
PETROGRAPHIC REPORT



Project: Passaic River

Boring No: C-19

Sample No: 1

Depth (ft.): 357.3-358.9

SAD Lab. No.: 128/1538

General Rock Description

SILTY SHALE, dark reddish brown (10R 3/4), indurated. Somewhat mottled appearance due to blebs and streaky zones of pale green clayey material associated with white to clear gypsum as shown below. Shale is fresh, sound and moderately hard (easily scratched with a knife blade). Strong hammer strikes yield breaks along bedding laminae, however, fissile character is not as well developed as found in some organic rich shales.

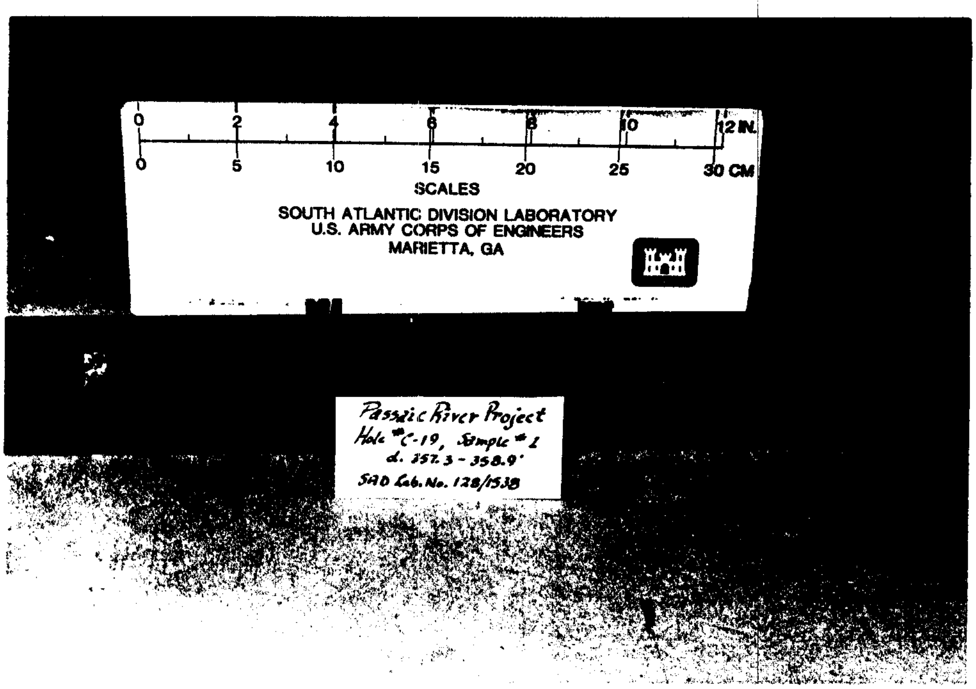
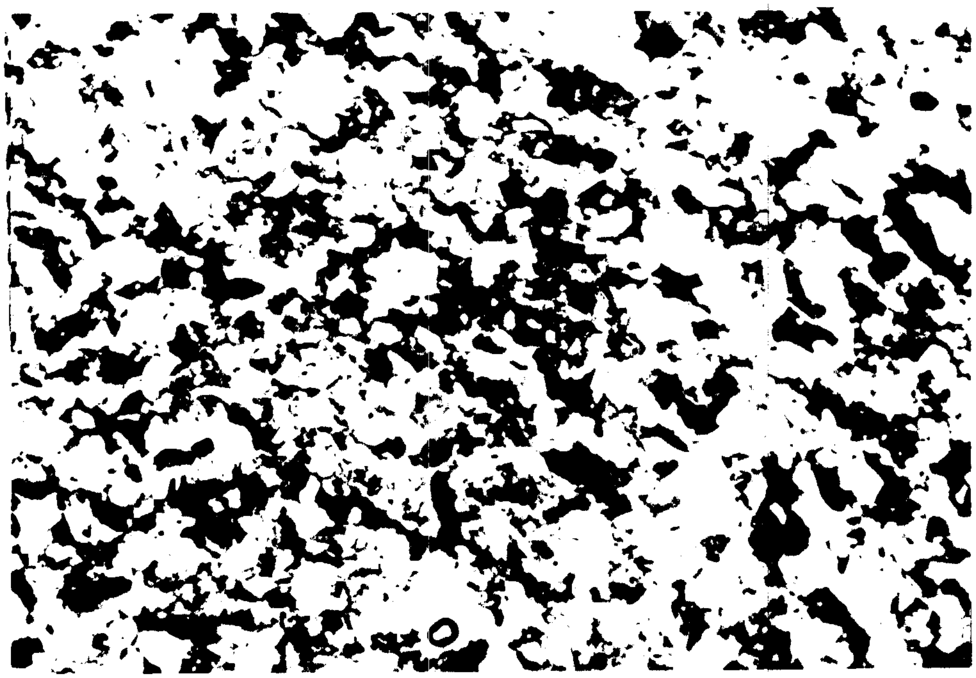


Figure 1. This views shows almost the entire core length. Top (d.357.3') is to left. Note abundance of lenticular blebs and streaks of intergrown gypsum/clay.

Passaic Formation, Gypsiferous

Towaco Formation



—— 1mm ——

Figure 2. Thin section Photomicrograph (25X, PPL), oriented normal to bedding. Based on optical and XRD analyses, the primary mineral grains are quartz, feldspar, calcite, mica and chloritic clay. However, the dark particles in this view are fine grained rock particles (clay rich). Rock particles also include quartzite. This sandstone could perhaps be classified as a feldspathic wacke or graywacke.

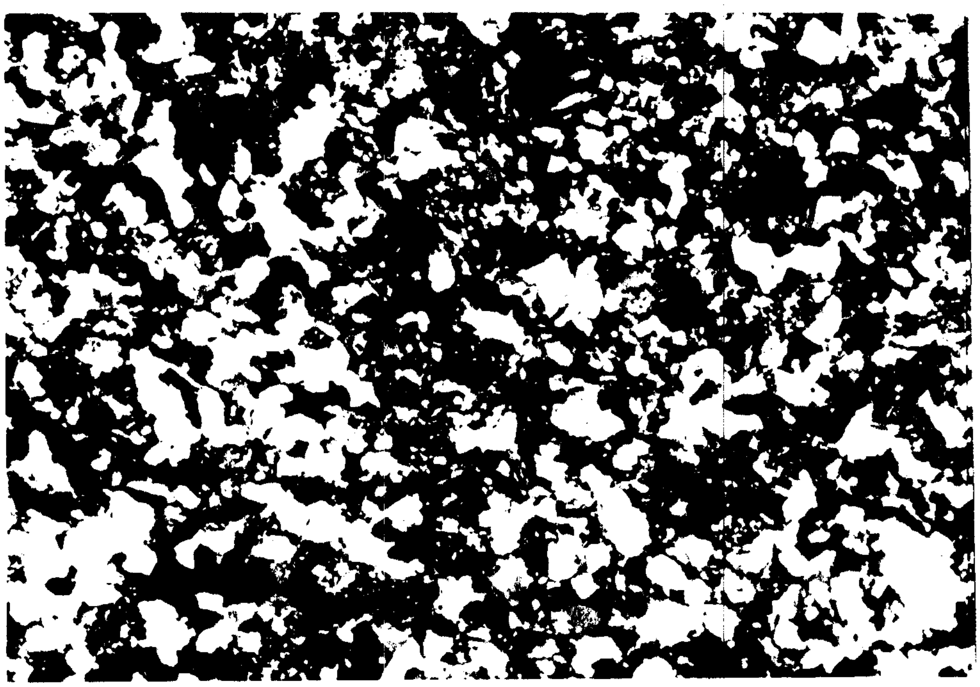
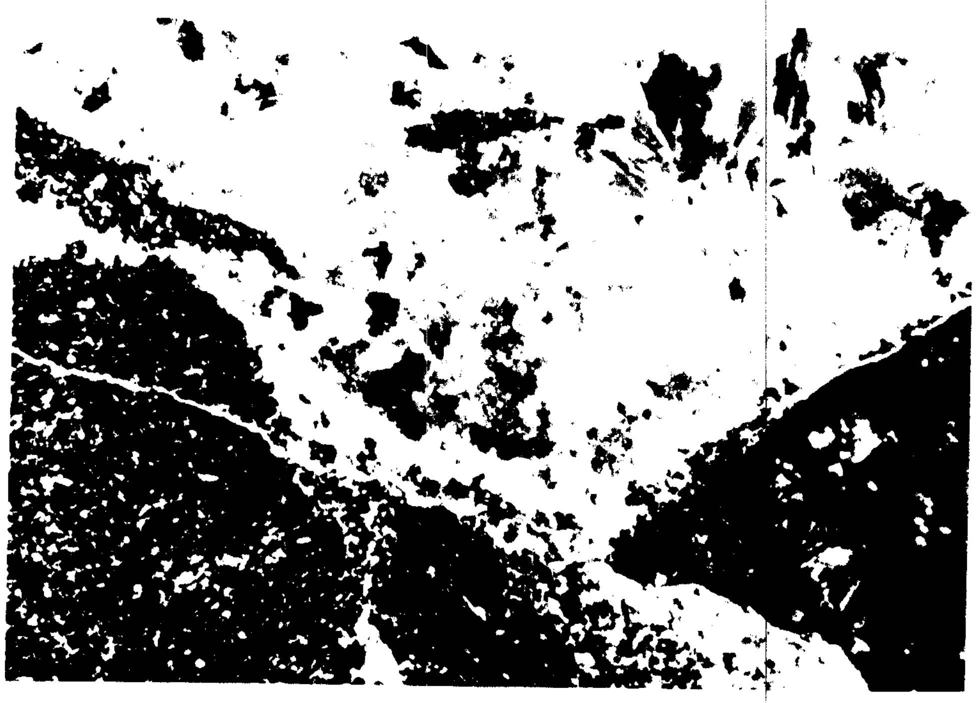


Figure 3. Same as above but in Polarized light.

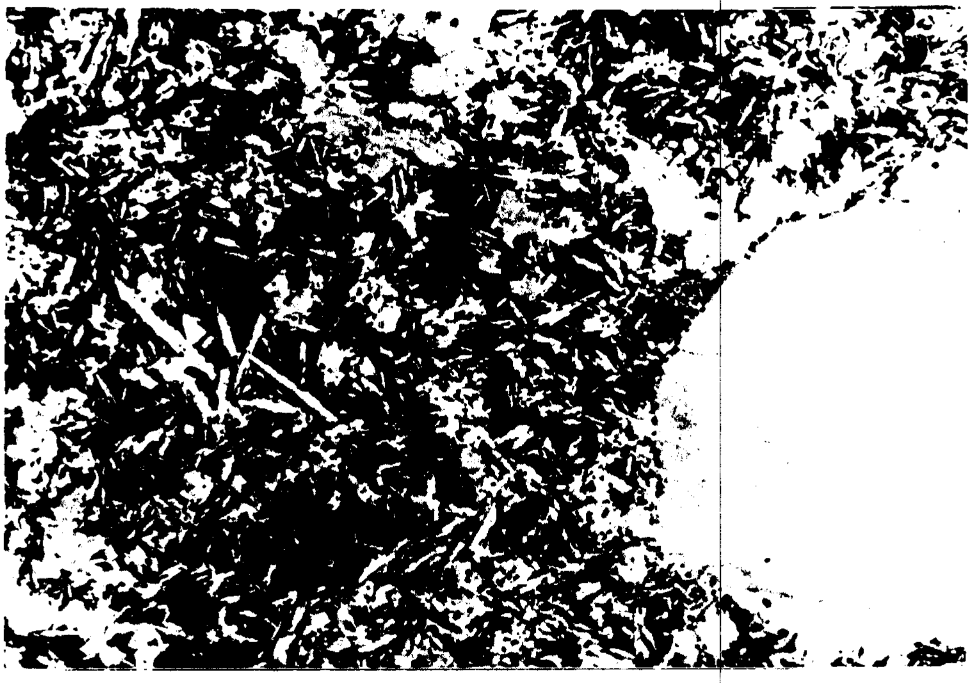
Preakness Mountain Basalt, Spur Tunnel



1mm

Figure 2. Thin section photomicrograph (25X, PL), random orientation. This is a view of chlorite and carbonate vein filling material. In addition, a zeolite or perhaps prehnite fills the center of the calcite healed fracture (multi-colored mineral, top right). It appears that this zone has undergone movement, hence, the presence of chlorite/carbonate along with rock fragments which fill these complex fractures. Branching, curving pyroxene crystals occur in the rock groundmass of this extrusive rock. In this view, chloritic material is at left, carbonate is center, and a rock fragment is at bottom right.

Preakness Mountain Basalt, Spur Tunnel



—— 1mm ——

Figure 3. Thin section photomicrograph (25X, PPL), random orientation. This is a section of the top fine crystalline basalt. It shows the fine crystalline groundmass next to a chlorite filled void. The groundmass has abundant white lath-shaped feldspar, curved, branching pyroxene in some areas, abundant opaque dust, and what appear to be globulites of epidote(?). These characteristics suggest rapid cooling.

—— 1mm ——

Figure 4. Thin section Photomicrograph (25X, PPL), random orientation. In contrast to the above, this is a section representing the speckled bottom half. Pyroxenes are better developed, however, feldspars are more altered, and ore dust now form crystallites. The speckled appearance comes from the lighter feldspar/pyroxene clusters surrounded by dark chloritic material replacing volcanic glass(?) and filling voids.

Microscopic Characteristics (d. 182.8 ft.)

Plagioclase feldspar, pyroxene, and ore make up the bulk of the primary minerals in this basalt. Alteration of the basalt has resulted in a variety of secondary minerals such as the cavity filled with a zeolite mineral or prehnite below.

Estimated Composition (relative abundance)

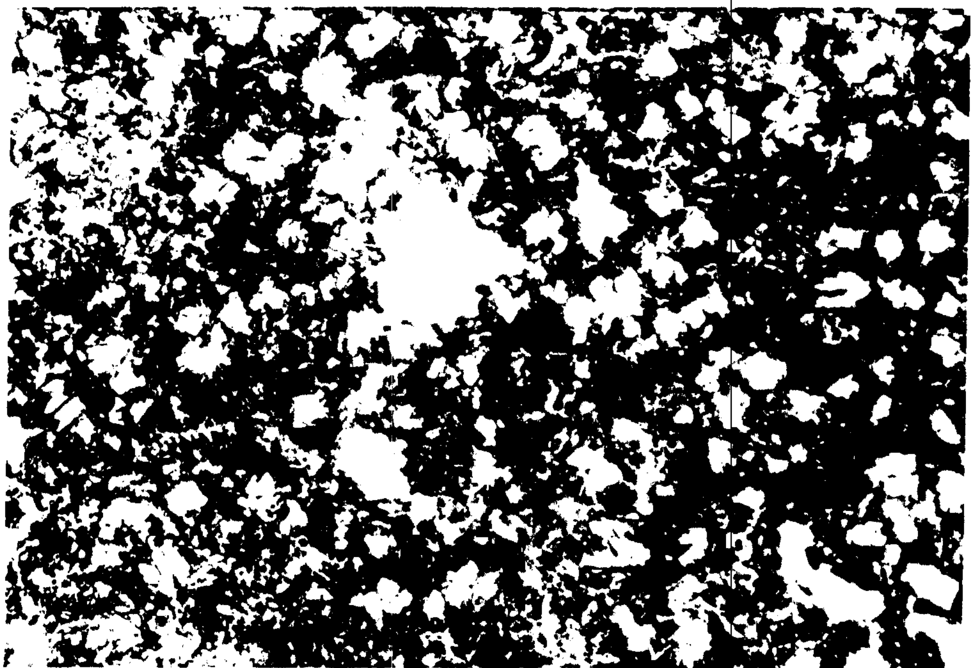
Plagioclase Feldspar.............Major
Pyroxene.........................Major
Iron Ore.........................Moderate
Secondary Minerals...............Moderate
(calcite, amphibole, quartz, epidote, zeolite, chlorite, other)

Preakness Mountain Basalt, Amygdaloidal

0.5 mm

Figure 2. Thin Section Photomicrograph (63X, P.L.). Oriented normal to core length. This is a small gas cavity rimmed with chlorite and filled with an unidentified zeolite (center) mineral (could also be prehnite). Lath-shaped grains outside cavity is plagioclase feldspar. Other colored grains are pyroxenes.

Preakness Mountain Basalt



—— 1mm ——

Figure 2. Thin section photomicrograph (25X, PPL), random orientation. This extrusive rock is fine grained as shown and contains abundant chloritic material (green) which fill scattered voids and replaces original interstitial volcanic glass(?). In this view, the white grains are clinopyroxenes. Small lath-shaped feldspar grains are abundant in the groundmass, however, they are intensely altered and less discernable in the photomicrograph at this magnification. Also in the groundmass are small opaque crystallites (globulites) that may be ores and perhaps some epidote. The mineralogy and texture suggest alteration before significant crystal growth. The speckled character is due to clusters of lighter colored feldspar/pyroxene rich zones in which the feldspars have been highly kaolinized. These zones contain much less dark chloritic material and are in marked contrast to areas with the dark chloritic material filling voids and replacing interstitial material. This rock is very similar in textural characteristics to the bottom core section of Boring C-117, Sample No. 24.

PETROGRAPHIC REPORT CONTINUED

Microscopic Characteristics (d.173.6 ft.)

This rock is made up almost entirely of plagioclase feldspar (ranges from andesine to labradorite), pyroxene, and magnetite. These minerals are arranged in "ophitic" fashion, hence, the diabase classification. It has several mineralogical characteristics that suggest that it may be a tholeiitic diabase.

Estimated Composition (%)

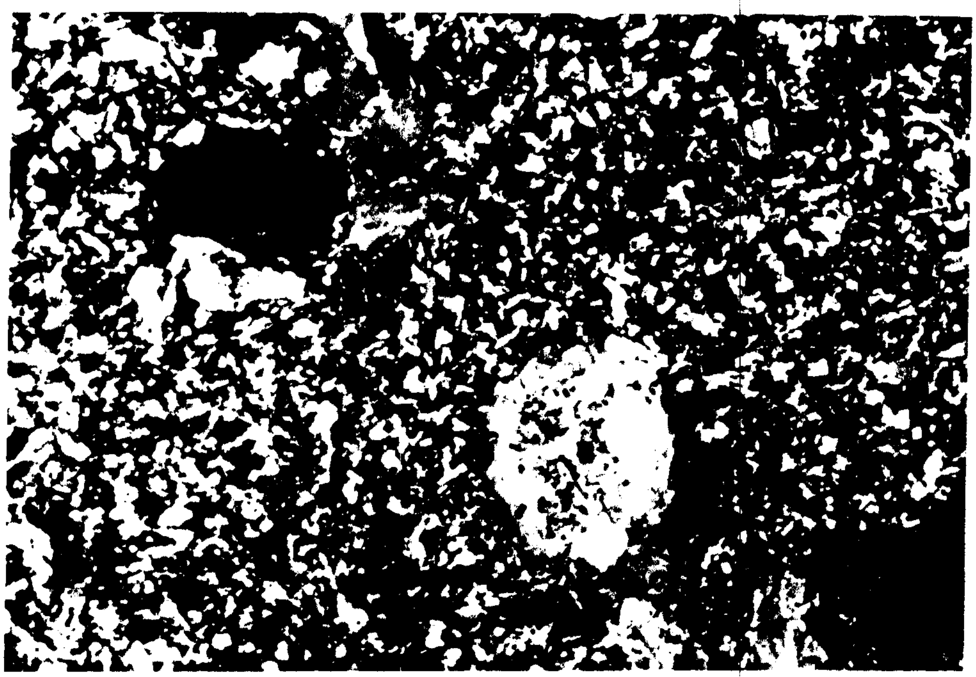
Plagioclase Feldspar................45
(andesine - labradorite)
Pyroxene(ortho+clino)...............30
Magnetite...........................15
Quartz..............................<5
Other (minor glass, alteration
products) ....<5

Preakness Mountain Basalt

—— 1 mm ——

Figure 2. Thin Section Photomicrograph (25X, P.L.). Oriented normal to core length. Pyroxene (anhedral colored grains) and magnetite (black) formed in between well-formed, lath-shaped plagioclase (gray lath-shaped grains), hence, the "diabase" texture. Plagioclase/quartz intergrowths occur interstitially (not shown).

Orange Mountain Basalt



—— 1mm ——

Figure 2. Thin section photomicrograph (25X, PL), random orientation. This microporphyritic basalt is fine grained as shown and consist primarily of altered feldspar (white, small lath shaped grains in groundmass) and pyroxene (interstitial between feldspar in groundmass). In this view, larger clinopyroxene and orthopyroxene crystals occur in clusters (glomeroporphyritic fashion). Epidote may also be present as small crystallites in groundmass.

Estimated Rock Composition (%)

Feldspar (altered)	35
Pyroxenes (epidote?)	35
Ore (skeletal magnetite)	5
Other Accessory Minerals	5
Alteration Products (clay*, mica, tr. calcite)	20

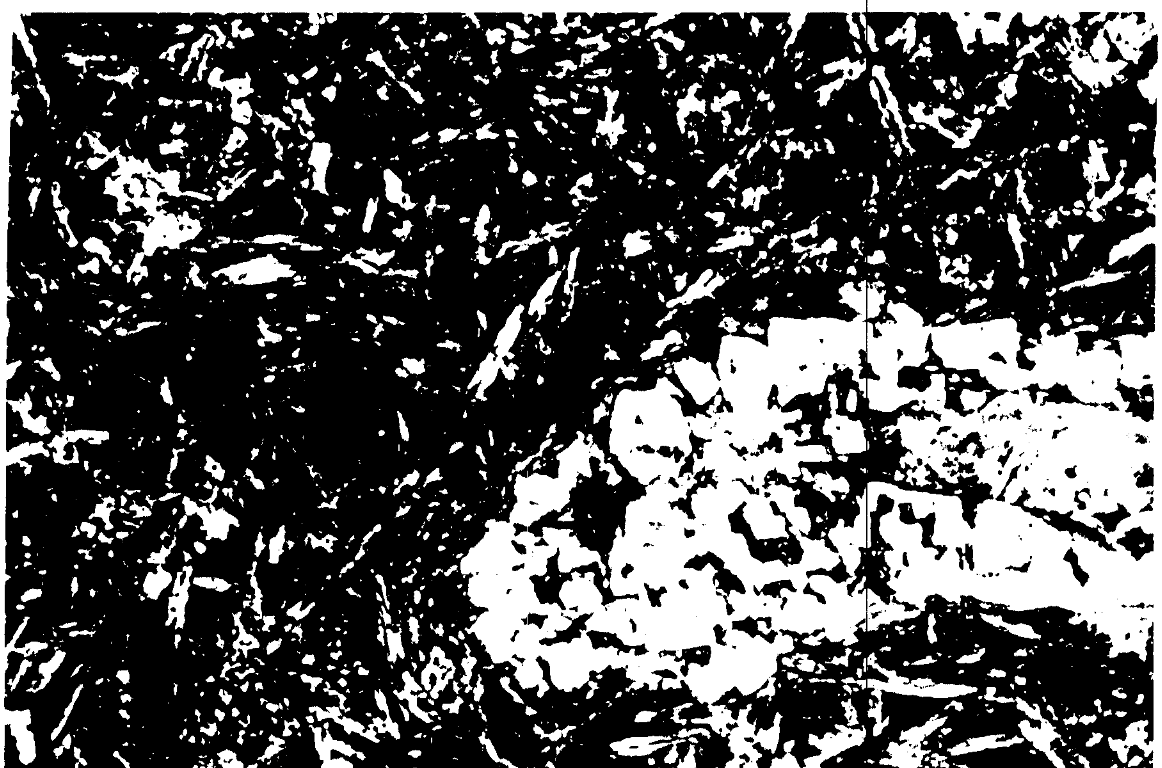
* XRD analyses indicates chlorite, kaolin and mica/sericite to be abundant

Microscopic Characteristics (d. 154.0 ft.)
The Basalt is fine grained, hence, essentially no minerals can be seen without magnification. It is relatively fresh and unweathered, however, much of the original "glass" has altered to more stable constituents (typically chloritic material).

Estimated Composition (%)

Feldspar	40
Pyroxene	25
Alteration Products (chlorite, other clays, some carbonate, possible zeolites)	15
Ore (opaque minerals)	10
All Other	10

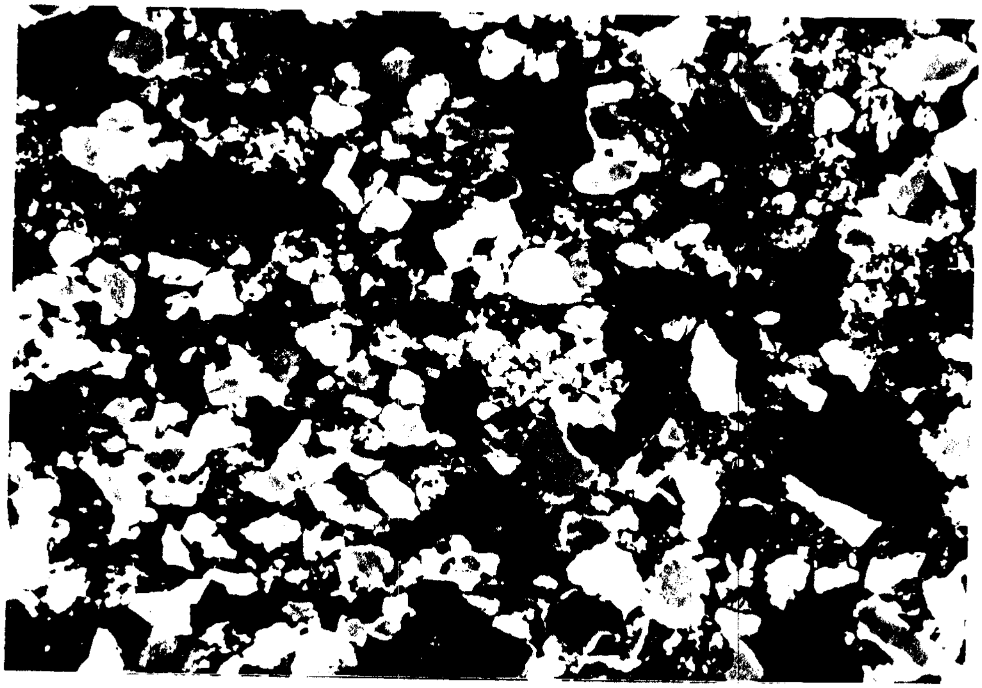
Orange Mountain Basalt



——— 0.5 mm ———

Figure 2. Thin Section Photomicrograph (63X, P.P.L.). Oriented normal to core length. The fine grained "felty" groundmass is apparent in this view. Branching feldspar microlites (and pyroxene in some areas) are common in this section (variolitic texture). Abundant dark "crystallites" occurring interstitially in this view are thought to be ore but could also be another pryroxene species. Large pyroxene grain with glass inclusions appears bottom right.

Passaic Formation



—— 1 mm ——

Figure 2. Thin section photomicrograph (25X, P.L.), oriented normal to bedding. This sandstone is made up predominantly of quartz particles (individual grains as well as quartzite and other rock particles), with lessor amounts of feldspar, calcite, and clay material. Note angular character of grains and grain support network. There is minimal cement in this rock. The cementing material is primarily fine clayey material and some carbonate.

Estimated Rock Composition (%)

Quartz (single grains, rock particles etc.)	75
Feldspar	10
Calcite (as individual grains and cement)	5
Clay*	5
All Other (misc. minerals suite)	5

* Most of this clay is 14 angstrom expandable clay (smectite-see Plate 1)

PETROGRAPHIC REPORT CONTINUED

Microscopic Characteristics (d.316.5 ft.)
The rock core sample is poorly sorted, that is, a range of particle sizes occurs. Grains vary from angular to well rounded. Carbonate and clay cement is sparse and not too well developed where present.

Estimated composition

All Quartz *	40
Feldspar	20
Carbonate (calcite)	10
Ferruginous Clay Matrix	15
All Others	15

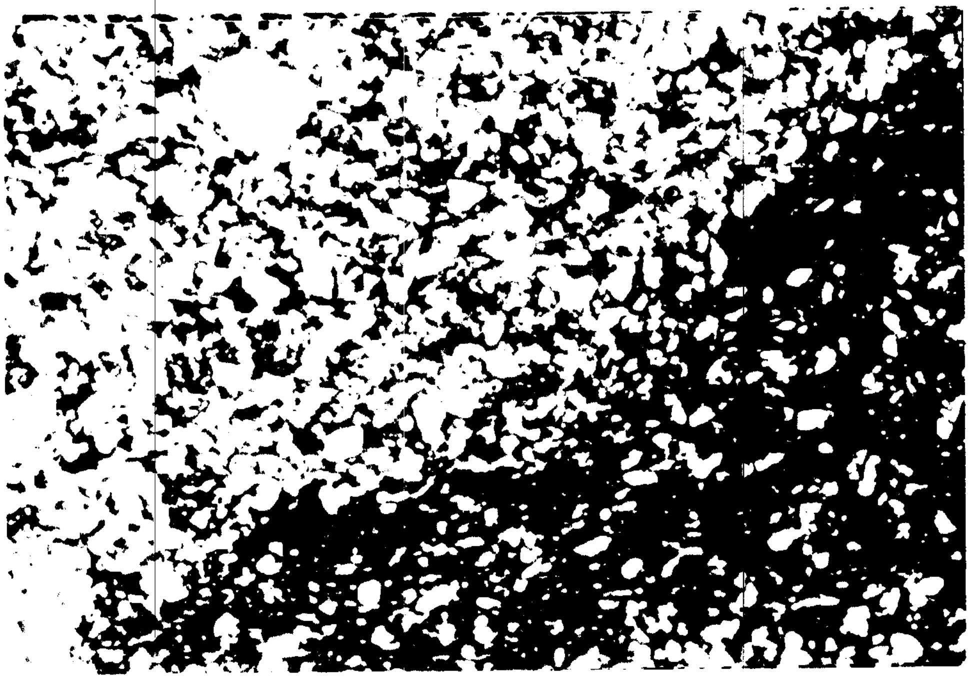
* Quartz & feldspar occur as single crystals as well as in rock particles

Passaic Formation, Conglomeritic

—— 1.0 mm ——

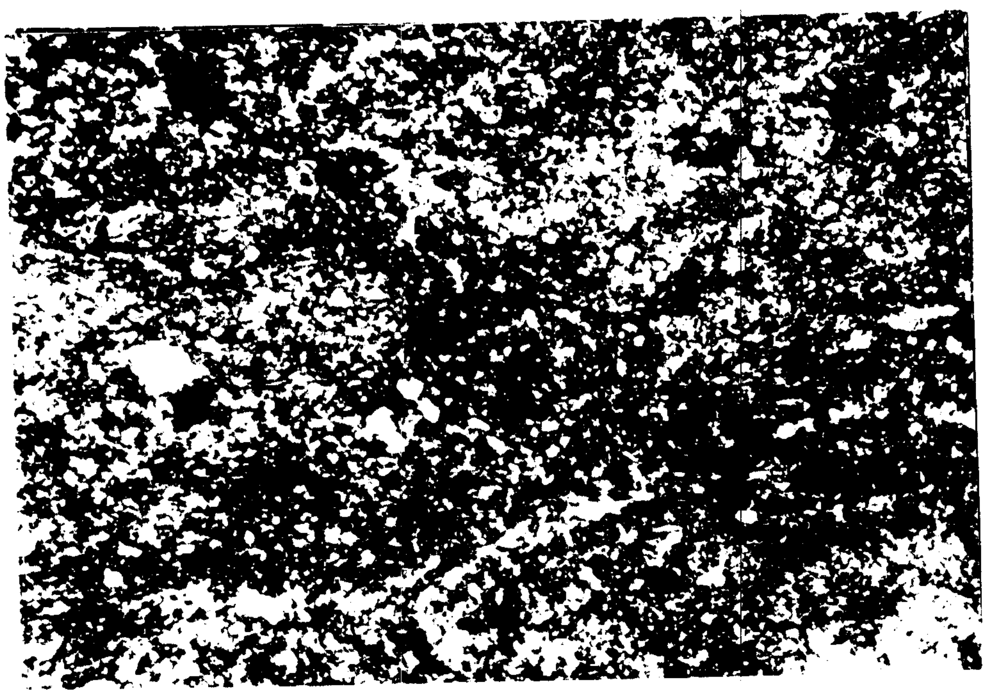
Figure 2. Thin Section Photomicrograph (25X, P.P.L.). Oriented normal to core length. This view demonstrates the diversity of grain sizes present in this coarse grained rock. Igneous rock particles (altered black in this view) and other rock particles are common in this core. Note small amount of matrix material. The small amount of calcite identified in the XRD study occurs interstitually.

Passaic Formation



—— 1 mm ——

Figure 2. Thin section photomicrograph (25X, PPL). This is the sandstone (far left in Figure 1) near the sandstone/shale contact. The sandstone has considerable quartz, calcite and feldspar, hence, it can generally be classified as a feldspathic sandstone. Individual grains are typically cemented by calcium carbonate with lessor amounts of clay, however, XRD data shows increase in clay with depth.



—— 1 mm ——

Figure 3. Thin section photomicrograph (25X, PPL). Based on XRD and thin section analyses, the bulk of the shale is quartz, carbonate, feldspar, mica and clay. The shale is sandy as shown in this view.

Microscopic Characteristics (depth 357.8 ft.)

Figure 2 below shows the fine grained character of the core at depth indicated. White angular grains are primarily quartz with subordinant feldspar, carbonate and tr. glauconite (?). Washed out area at bottom right in photomicrograph is gypsum. Reddish brown matrix is ferrigenous clay (illite/sericite, chlorite), and very fine opaque material.

ESTIMATED COMPOSITION (%)

Clay (illite/sericite chlorite, other)	43
Quartz	15
Gypsum (tr. Anhydrite)	14
Feldspar	12
Opaques (ore)	10
Other (carb., glauc. etc)	6

Passaic Formation, Gypsiferous

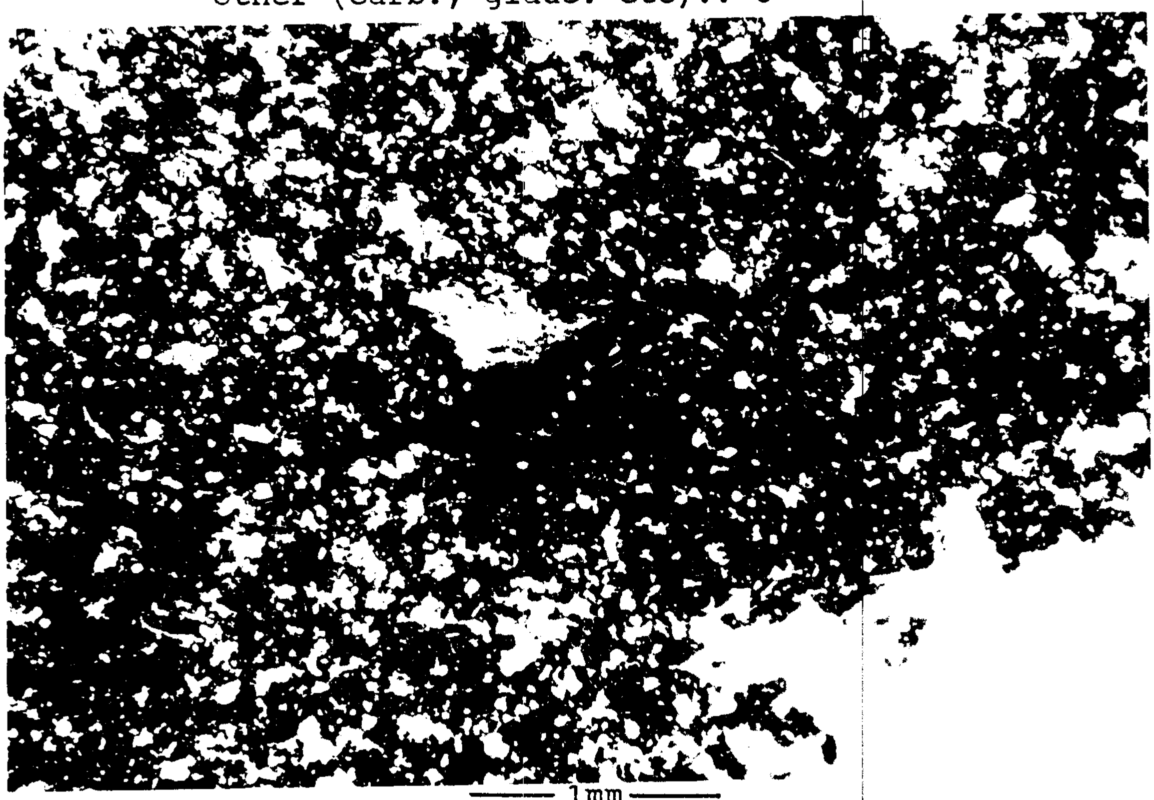


Figure 2. Thin Section Photomicrograph (25X, P.P.L.). Oriented normal to core length. Patches of gypsum (bottom right) are common and typically occur intergrown with well developed chloritic (?) clay. Traces of anhydrite and other possible sulfates where noted. The ferrigenous matrix has abundant very fine opaque "ore". Relic fossil forms are occasionally discernable. Dark clay rich streaks known as "microstylolites" (solution tails) are conspicuous on core surfaces.

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.5

DISCONTINUITY STUDIES

Attachment E.3.5

Discontinuity Studies

Passaic Flood Protection Project
Discontinuity Study
Tabulation of Composite Joints From Down-Hole TV Camera

PASSAIC FORMATION

Joint Number	strike	dip
p1	N 64° E	vert.
p2	N 30° E	vert.
p3	N 84° W	24° NE
p4	N 42° E	24° NW
p5	N 46° E	46° NW

Joint Number	strike	dip
p6	N 16° E	vert.
p7	N 42° E	39° SE
p8	N 42° E	18° SE
p9	N 43° E	vert.
p10	N 60° E	70° SE

FELTVILLE / TOWACO FORMATIONS

Joint Number	strike	dip
t1	N 43° E	vert.
t2	N 46° W	67° NE
t3	N 16° E	84° NW
t4	N 44° E	22° NW
t5	N 12° E	79° SE

Joint Number	strike	dip
t6	N 15° E	63° SE
t7	N 73° E	19° SE
t8	N 44° E	43° SE
t9	N 44° E	77° SE
t10	N 84° E	58° SE

BOONTON FORMATION

Joint Number	strike	dip
b1	N 86° E	37° NW
b2	N 45° E	44° NW
b3	N 6° W	78° NE
b4	N 12° E	80° SE
b5	N 8° E	16° SE

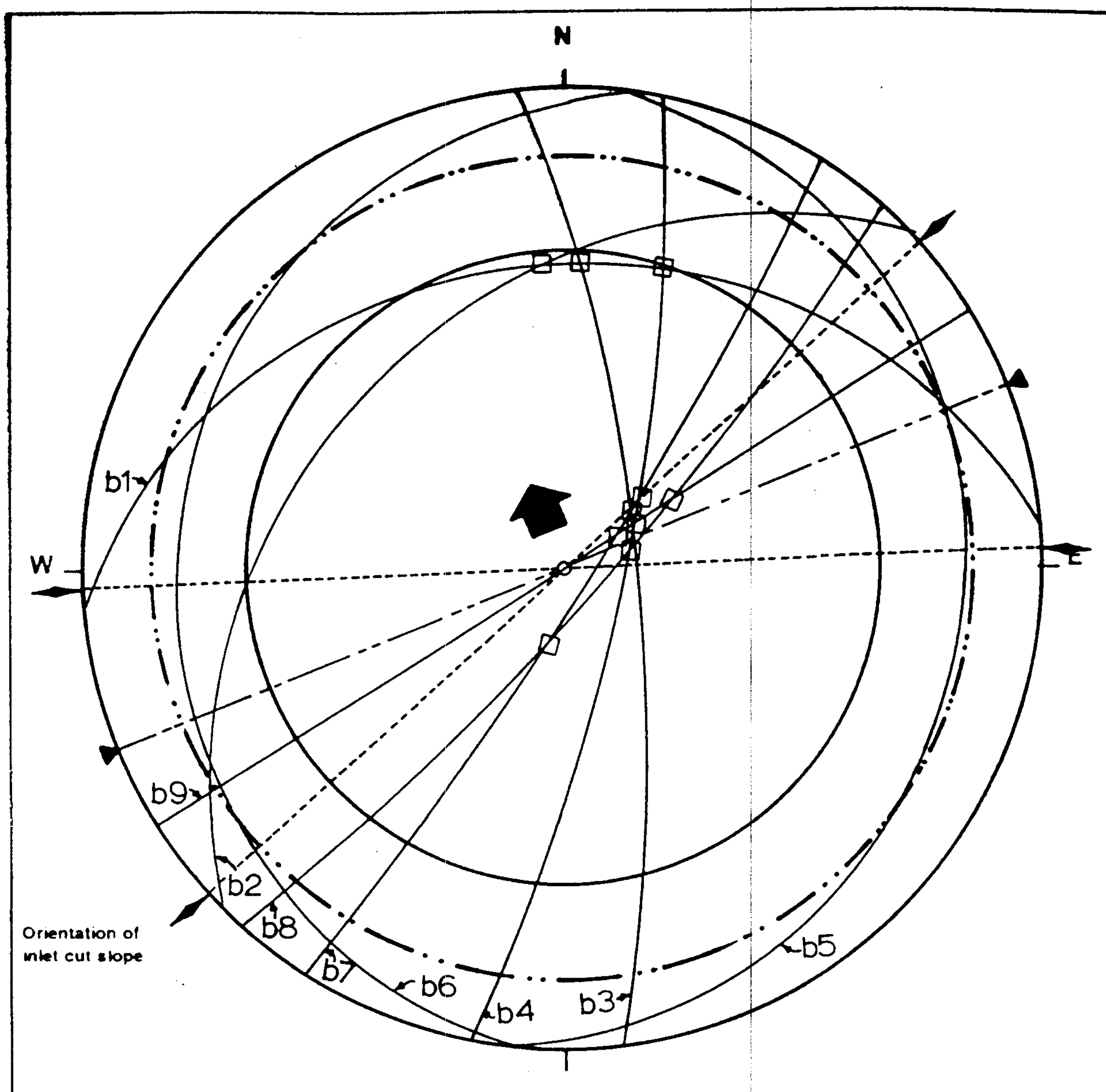
Joint Number	strike	dip
b6	N 4° E	20° NW
b7	N 33° E	86° SE
b8	N 42° E	84° SE
b9	N 58° E	vert.

ALL BASALT FORMATIONS
(ORANGE MT., PREAKNESS MT., and HOOK MT.)

Joint Number	strike	dip
i1	N 90° E	vert.
i2	N 84° W	55° NE
i3	N 61° E	62° NW
i4	N 48° E	85° NW
i5	N 48° W	33° NE
i6	N 90° E	32° N
i7	N 48° E	48° NW
i8	N 32° E	84° NW
i9	N 7° E	66° SE

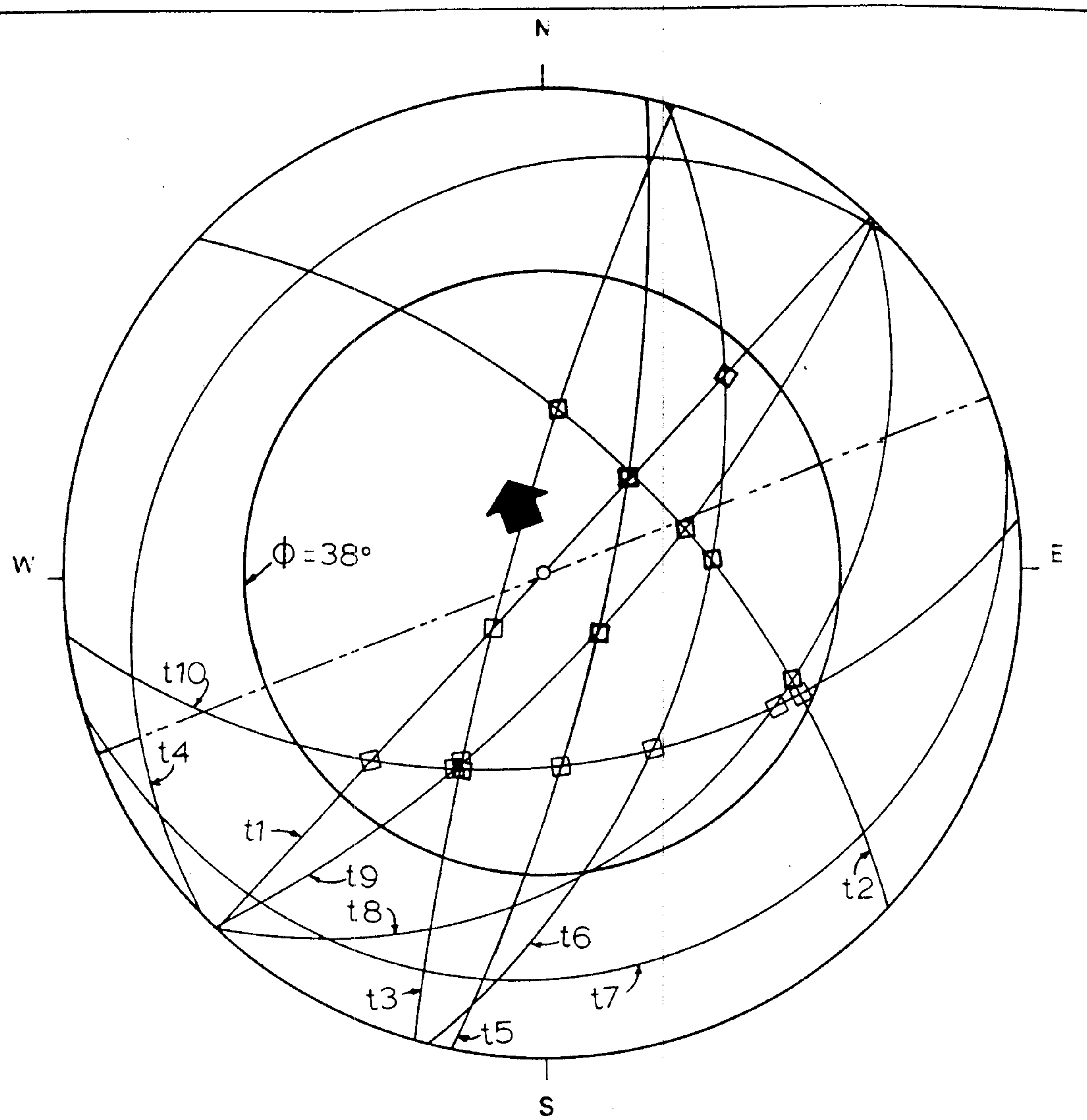
Joint Number	strike	dip
i10	N 44° E	18° SE
i11	N 16° E	23° NW
i12	N 28° E	82° SE
i13	N 80° W	50° SW
i14	N 30° W	62° SW
i15	N 48° E	87° SE
i16	N 90° E	70° S
i17	N 44° W	vert

Joint numbers refer to composite joints as determined by contouring poles of joints measured using the down-hole television camera. See the attached figures for locations of these joints.



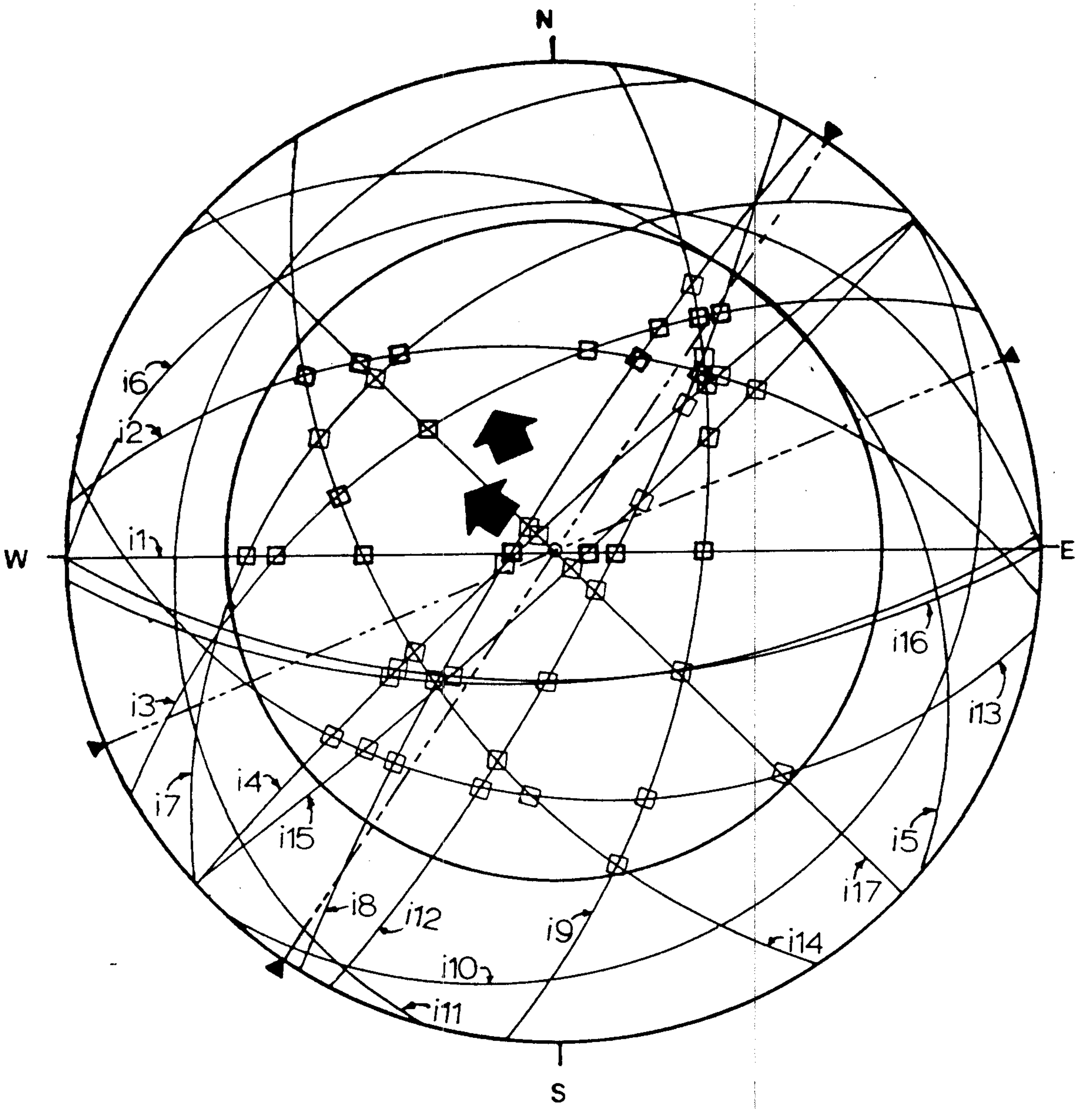
These are great circle plots of composite joints determined by contouring of joint poles. The numbering of the joints corresponds to the attached table and contour plot. Chained lines show the orientation of the tunnel face. Intersections of wedge-forming joint combinations that are free to move and have intersection lines that dip at more than the Φ angle (Markland's criteria) are indicated with a square. Stability computations are attached (After Hoek and Bray)

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
BOONTON FORMATION



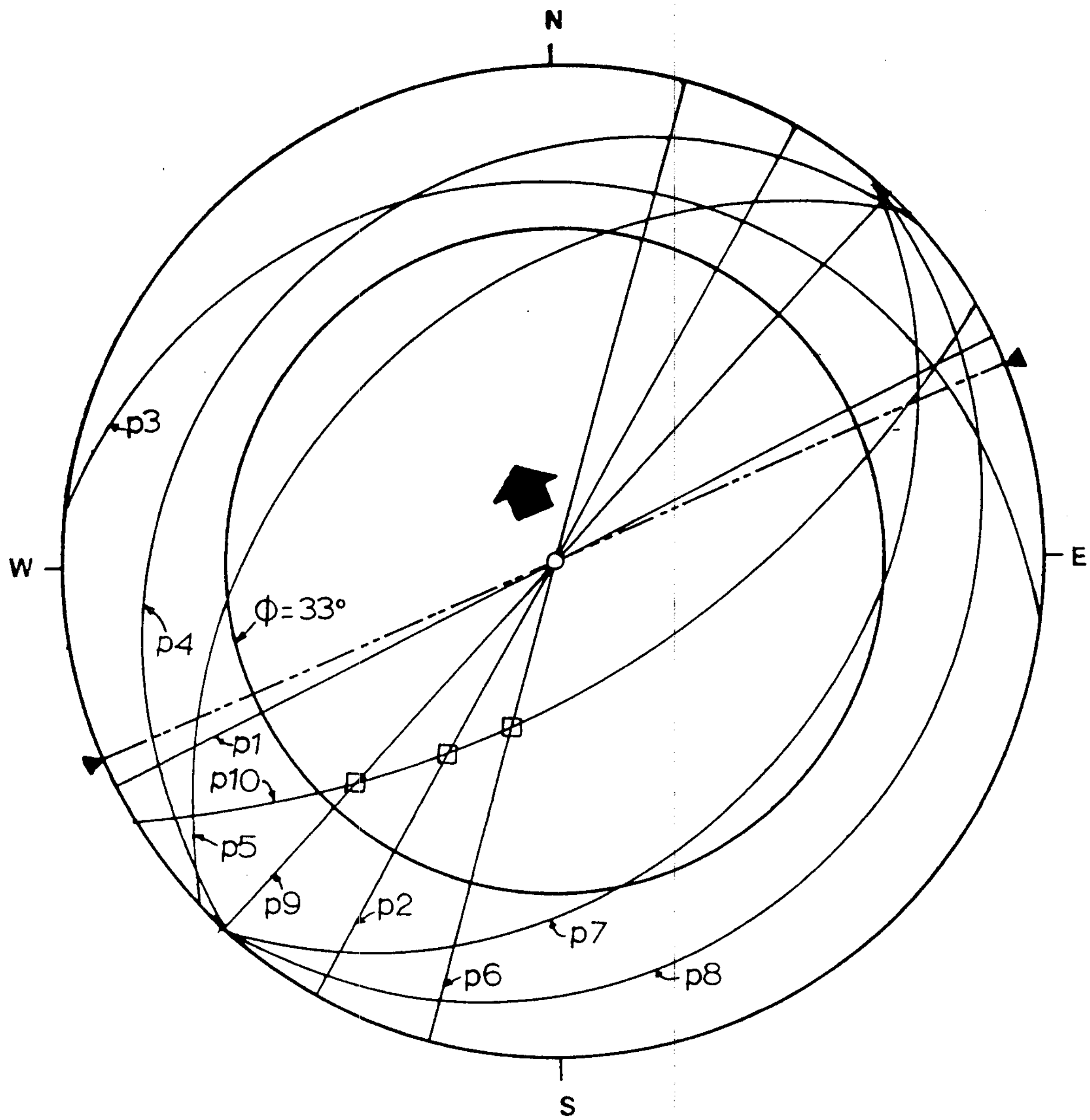
These are great circle plots of composite joints determined by contouring of joint poles. The numbering of the joints corresponds to the attached table and contour plot. Chained lines show the orientation of the tunnel face. Intersections of wedge-forming joint combinations that are free to move and have intersection lines that dip at more than the Φ angle (Markland's criteria) are indicated with a square. Stability computations are attached (After Hoek and Bray)

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
TOWACO/FELTVILLE FORMATION



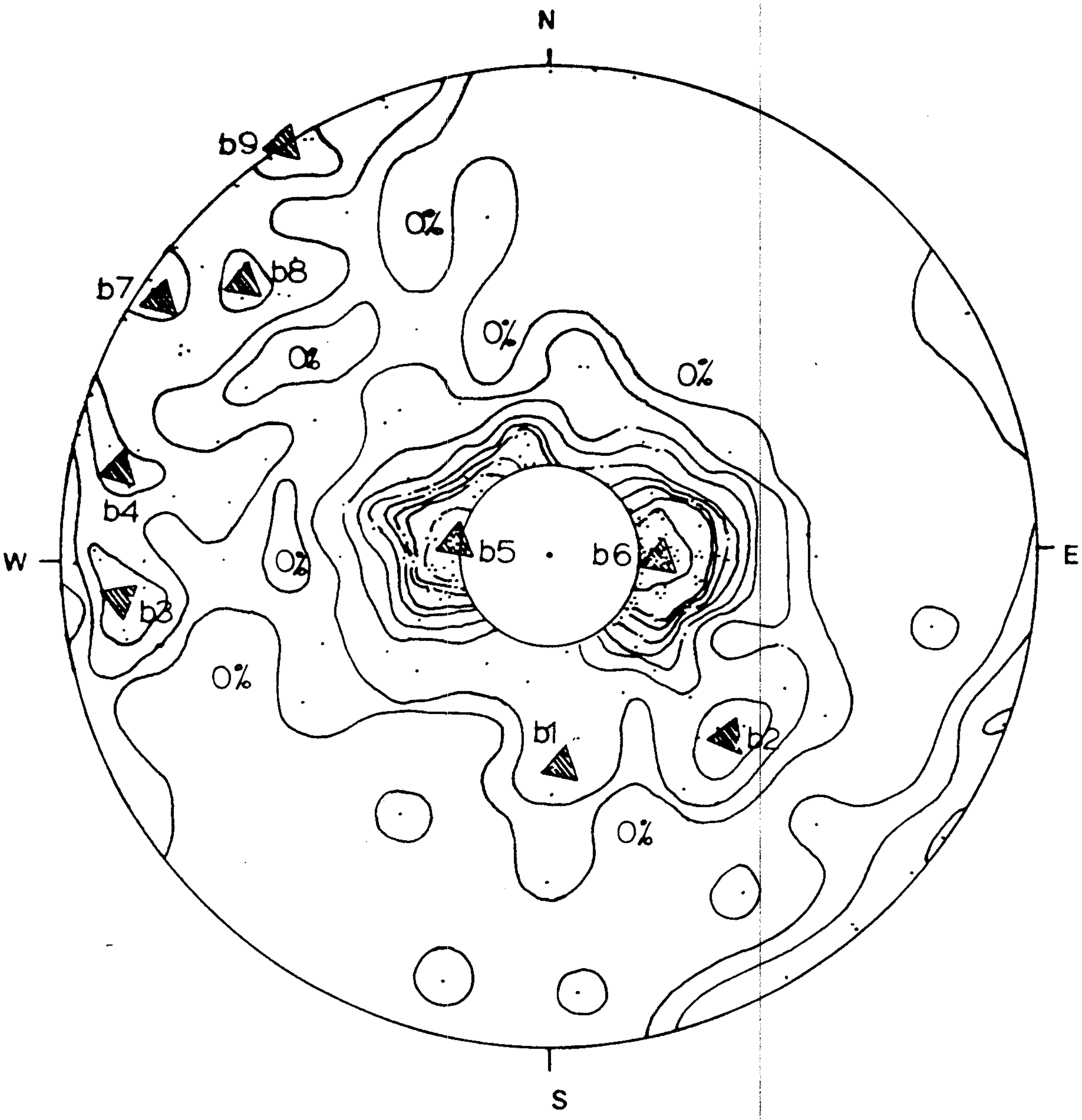
These are great circle plots of composite joints determined by contouring of joint poles. The numbering of the joints corresponds to the attached table and contour plot. Chained lines show the orientation of the tunnel face. Intersections of wedge-forming joint combinations that are free to move and have intersection lines that dip at more than the Φ angle (Markland's criteria) are indicated with a square. Stability computations are attached (After Hoek and Bray)

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
ALL BASALT FORMATIONS



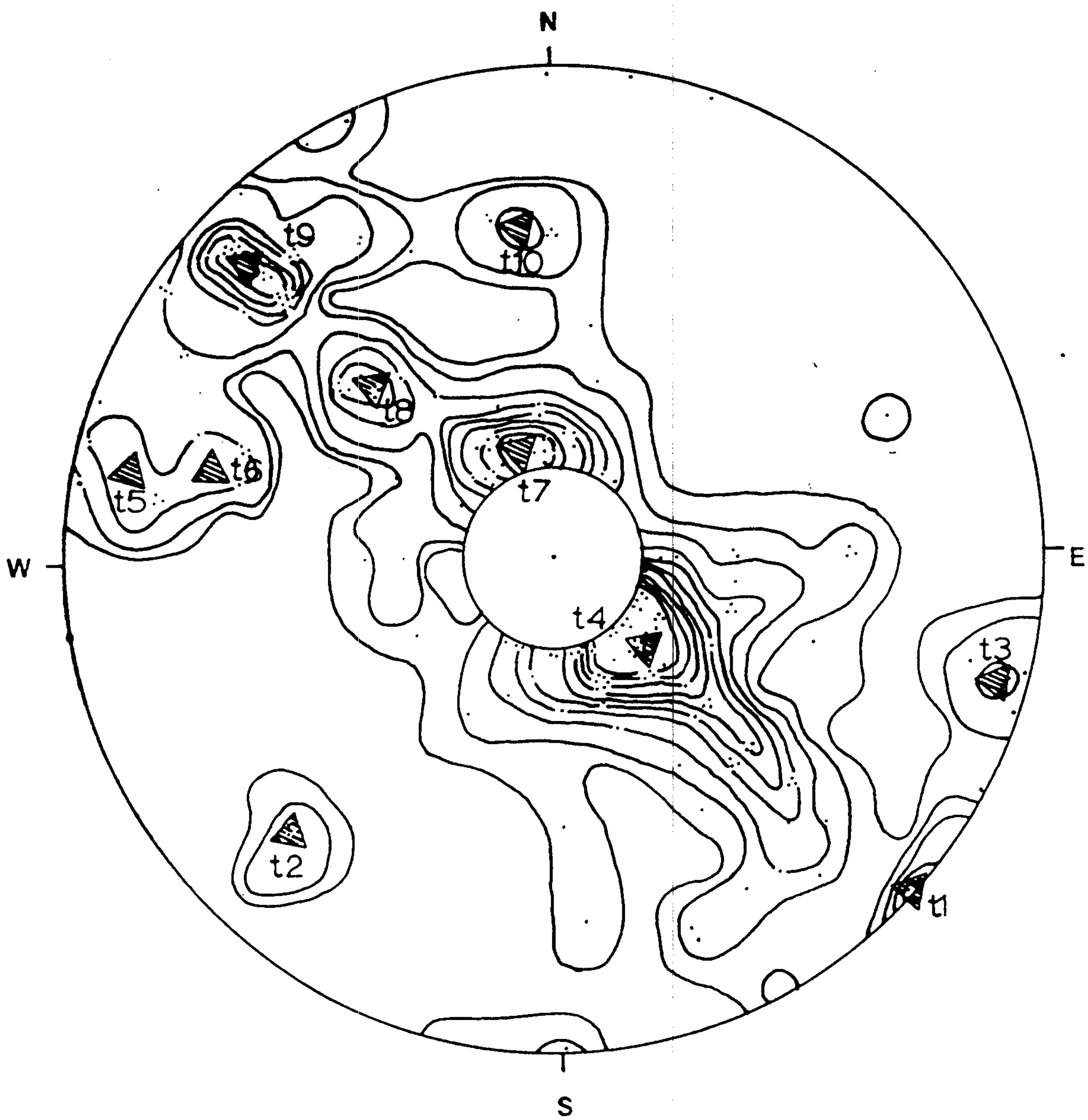
These are great circle plots of composite joints determined by contouring of joint poles. The numbering of the joints corresponds to the attached table and contour plot. Chained lines show the orientation of the tunnel face. Intersections of wedge-forming joint combinations that are free to move and have intersection lines that dip at more than the Φ angle (Markland's criteria) are indicated with a square. Stability computations are attached (After Hoek and Bray)

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
PASSAIC FORMATION



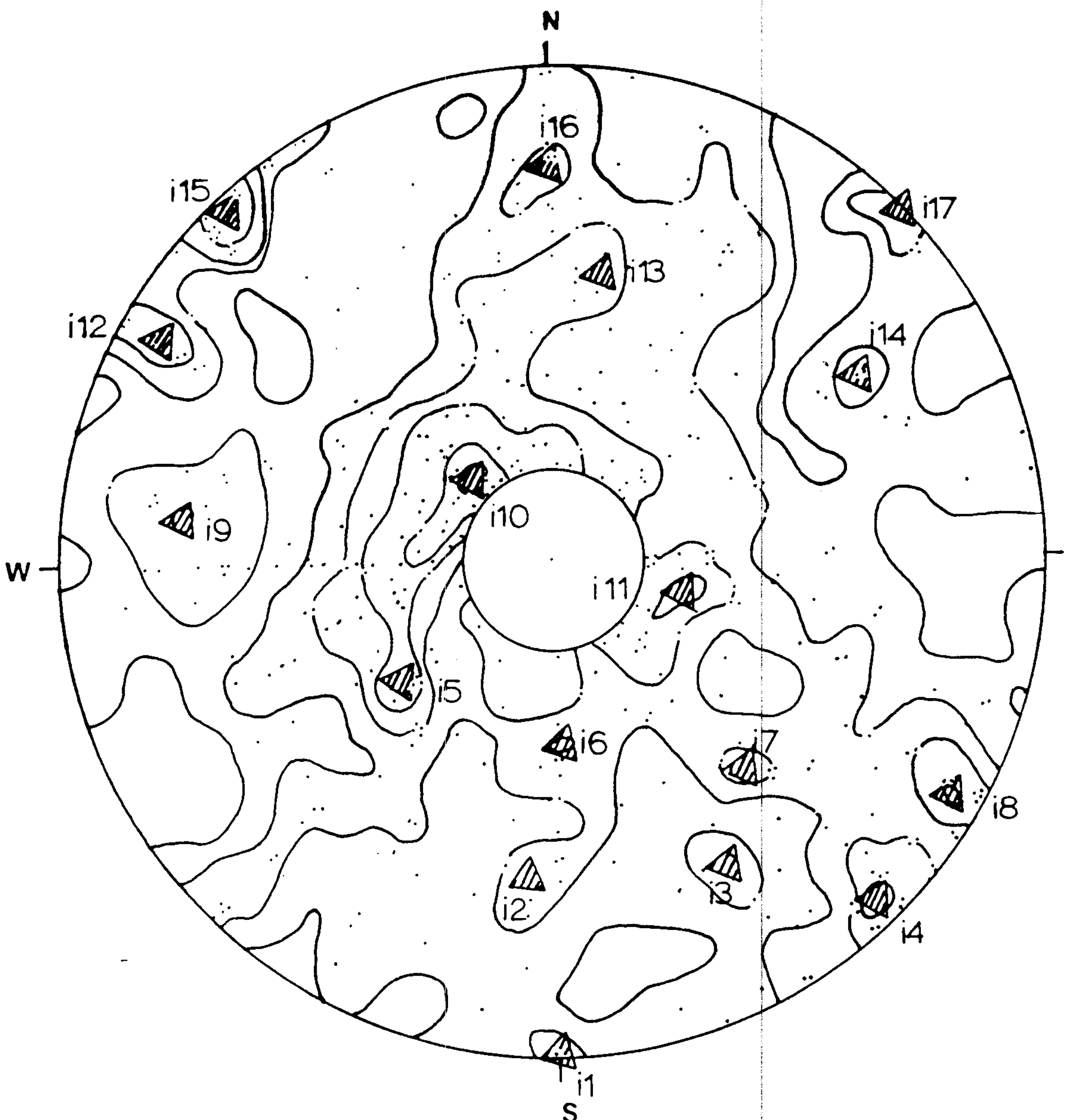
The discontinuity data presented here was gathered using a Reese Equipment Co. down-hole television camera. A total of 204 joint readings are represented in this diagram. Joints inside the inner 15o are considered to be bedding and present in all sets. Contour interval is 2 % based on total number of joints contained within a square representing 1% of the equal area stereonet area (After Hoek and Bray).

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
BOONTON FORMATION



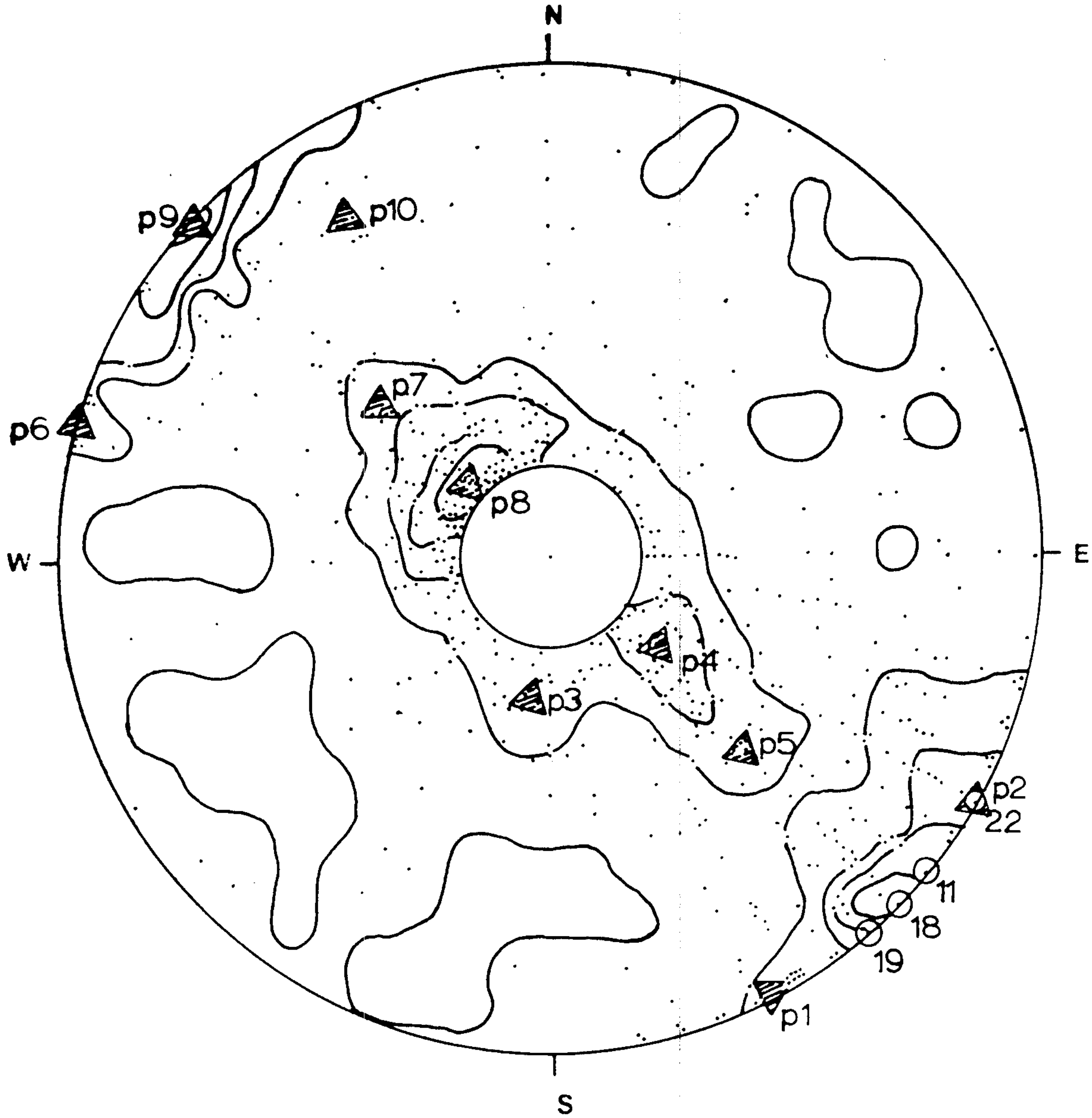
The discontinuity data presented here was gathered using a Reese Equipment Co. down-hole television camera. A total of 194 joint readings are represented in this diagram. Joints inside the inner 15o are considered to be bedding and present in all sets. Contour interval is 2 % based on total number of joints contained within a square representing 1% of the equal area stereonet area (After Hoek and Bray).

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
TOWACO/FELTVILLE FMS.



The discontinuity data presented here was gathered using a Reese Equipment Co. down-hole television camera. A total of 389 joint readings are represented in this diagram. Joints inside the inner 15o are considered to be bedding and present in all sets. Contour interval is 2 % based on total number of joints contained within a square representing 1% of the equal area stereonet area (After Hoek and Bray).

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
ALL BASALT FORMATIONS



The discontinuity data presented here was gathered using a Reese Equipment Co. down-hole television camera. A total of 630 joint readings are represented in this diagram. Joints inside the inner 15o are considered to be bedding and present in all sets. Contour interval is 2 % based on total number of joints contained within a square representing 1% of the equal area stereonet area (After Hoek and Bray).

PASSAIC FLOOD PROTECTION PROJECT
Tunnel - Discontinuity Study
Joint Pole Contour Diagram
PASSAIC FORMATION

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.6

ROCK MECHANICS LABORATORY TESTING

Attachment E.3.6

Rock Mechanics Laboratory Testing

PASSAIC TUNNEL PROJECT
LABORATORY TESTING
LIST OF ABBREVIATIONS

01-May-95

LABORATORIES

HAM = HAMILTON ENGINEERING
MRD = MISSOURI RIVER DIVISION, CORPS
ROBBIN = THE ROBBINS CO.
SAD = SOUTH ATLANTIC DIVISION, CORPS
SINTEF = NORWEGIAN INSTITUTE OF TECHNOLOGY
WES = WATERWAYS EXPERIMENT STATION, CORPS

ROCK TYPE

FORMATION

Tp - PASSAIC FM.
Jo - ORANGE MT. BASALT
Jf - FELTVILLE FM.
Jp - PREAKNESS MT. BASALT
Jt - TOWACO FM.
Jh - HOOK MT. BASALT
Jb - BOONTON FM.

LITHOLOGY

MS = MUDSTONE
SS - SANDSTONE
SH - SHALE
SIS - SILTSTONE
CS - CLAYSTONE
GYP - GYPSIFEROUS
/ - INTERBEDDED
AM - AMYGDALOIDAL
VS - VESICULAR
GB - GABBRO, COARSE XLN.
GG - GOUGE OR DISRUPTED
WEA - WEATHERED

COLOR

R - RED
B - BROWN
G - GRAY
BK - BLACK
W - WHITE
GN - GREEN
BL - BLUE

GRAIN SIZE

VF - VERY FINE
F - FINE
M - MEDIUM
C - COARSE
CG - CONGLOMERITE
AK - ARKOSIC

BEDDING
Ms - MASSIVE
Tb - THICK BEDDED
Thb - THIN BEDDED

MODIFIERS

Sl - SLIGHTLY
Dk - DARK
V - VERY
Ca - CALCAREOUS
Sly - SILTY
MIC - MICACEOUS
X - CROSS-BEDDED
Sd - SANDY

EXAMPLE FORMAT: Tp,SS/GYP,R,M,Th or Jo,VS,GN,Ms or Jf,SS/SH,G,VF,Int,MIC

TEST TYPE

AB = ABRASION
CUTTER STEEL
ABS = ABSORPTION
BR = BRITTLENESS
BT = TENSILE STRENGTH, BRAZILIAN
CI = CONE INDENTER TEST
CLI = CUTTER LIFE INDEX
DRI = DRILLING RATE INDEX
FL = FLAKINESS
HD 1-4 = HARDNESS
HD 5-6 = HARDNESS
HD 7 = HARDNESS
LA = LOS ANGELES ABRASION
P = PERMEABILITY
POR = APPARENT POROSITY
PL = POINT LOAD TEST
PT = PETROGRAPHIC ANALYSIS
PTS = THIN SECTION PETROGRAPHIC
PTX = X-RAY DIFFRACTION
RH = SCHMIDT HAMMER
SF = SLIDING FRICTION, NATURAL BREAK
SH = SHEAR STRENGTH
SW = SLIDING FRICTION, SAWED SURFACE
SV = SIEVERS J NUMBER
SWF = FREE SWELL TEST
Uc = UNCONFINED, NO DEFORMATION
UcE = UNCONFINED WITH ELASTIC MODULUS
UcER = UNCONFINED WITH ELASTIC MODULUS AND POISSON'S RATIO
UW = UNIT WEIGHT

UNITS

PSI = POUNDS PER SQUARE INCH
PCF = POUNDS PER CUBIC FOOT
CM/S = CENTIMETERS PER SECOND
MPA = MEGA PASCALS
KN = KILONEWTONS

PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
SINTEF	PT-19	81.5	82.4	Jp,F	AB		1.5				
SINTEF	PT-20	150.2	151.1	Jp,F	AB		4.5				
SINTEF	PT20	113	114.1	Jp,GB	AB		2.5			%	
MRD	DC-119	246.2	246.6	Jp,WEA,VS,AM,	ABS		3.4			%	
MRD	DC-121	251	251.4	Jt,SiySH,G-R	ABS		4.4			%	
MRD	C-146	217.6	218.1	Tp,Siy SS,R-B,F	ABS		4.6			%	
MRD	C-146	161.5	162.1	Tp,SiyCS,R-B	ABS		1			%	
MRD	DC-38	343.6	344	Tp,SiyCS,R-B	ABS		1.1			%	
MRD	C-145	351.5	352	Tp,SiyCS,R-B	ABS		0.8			%	
MRD	DC-23	467.5	467.9	Tp,SiyCS,R-B	ABS		3.7			%	
MRD	C-146	325.4	326	Tp,SiyCS/SH,R-B	ABS		3.2			%	
MRD	C-145	350.4	350.9	Tp,SiySH,R-B	ABS		1			%	
SINTEF	PT-19	81.5	82.4	Jp,F	BR		21.4				
SINTEF	PT-20	150.2	151.1	Jp,F	BR		23				
SINTEF	PT20	113	114.1	Jp,GB	BR		22.6				
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	BT		1148			PSI	
SAD	C-88	316.8	318.2	Jo,DkG,F	BT		2043			PSI	
SAD	C-116	163.4	165.2	Jp,GR-G,AM,WEA	BT		1198			PSI	
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	BT		989			PSI	
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	BT		602			PSI	
SAD	C-28	448.4	449.9	Tp,SH,R-B,Ca	BT		671			PSI	
SAD	C-63	218.1	219.1	Tp,SS,G-R,F,MIC	BT		988			PSI	
SAD	C-71	178.8	180.7	Tp,SS,G-R,M	BT	52000	1651			PSI	
HAM	C-131	151.8	152.8	Jh,Ms,F	CI	52000	0.117			INCH	
HAM	C-131	151.8	152.8	Jh,Ms,F	CI	58500	0.134			INCH	
SINTEF	PT-19	81.5	82.4	Jp,F	CLI		56.1				
SINTEF	PT-20	150.2	151.1	Jp,F	CLI		39.6				
SINTEF	PT20	113	114.1	Jp,GB	CLI		48.8				
SINTEF	PT-19	81.5	82.4	Jp,F	DRI		31				VERY LOW TO LOW
SINTEF	PT-20	150.2	151.1	Jp,F	DRI		33				VERY LOW TO LOW
SINTEF	PT20	113	114.1	Jp,GB	DRI		32				VERY LOW TO LOW
SINTEF	PT-19	81.5	82.4	Jp,F	FL		1.29				
SINTEF	PT-20	150.2	151.1	Jp,F	FL		1.26				
SINTEF	PT20	113	114.1	Jp,GB	FL		1.33				
ROBBIN	PT-2	237.9	238.9	Jb,SS,G,Siy,F	HD 1-4		15				QUALITY - HIGH
ROBBIN	PTI-3	142.2	143	Jf,SS,R,F	HD 1-4		60				QUALITY - HIGH
ROBBIN	PT-10	180.3	181	Jh,F	HD 1-4		10				QUALITY - HIGH
ROBBIN	PT-19	105.2	106.1	Jp,F	HD 1-4		10				QUALITY - HIGH
ROBBIN	PT-19	130.2	131	Jp,F	HD 1-4		10				QUALITY - HIGH
ROBBIN	PT-20	109.3	110.4	Jp,GB,M	HD 1-4		10				QUALITY - HIGH
ROBBIN	PT-20	149.3	150.2	Jp,M	HD 1-4		10				QUALITY - HIGH
ROBBIN	PT-12	227.3	228	Jt,SS,R,Siy	HD 1-4		30				QUALITY - HIGH
ROBBIN	PT-44	107.9	108.4	Tp,SS,R	HD 1-4		30				QUALITY - HIGH
ROBBIN	PTI-4	207.8	208.7	Tp,SS,R	HD 1-4		20				QUALITY - HIGH
ROBBIN	PT-38	442.6	443.4	Tp,SS,R	HD 1-4		20				QUALITY - HIGH
ROBBIN	PT-10	180.3	181	Jh,F	HD 5-6		90				QUALITY - HIGH
ROBBIN	PT-19	105.2	106.1	Jp,F	HD 5-6		90				QUALITY - HIGH
ROBBIN	PT-19	130.2	131	Jp,F	HD 5-6		90				QUALITY - HIGH
ROBBIN	PT-20	109.3	110.4	Jp,GB,M	HD 5-6		90				QUALITY - HIGH
ROBBIN	PT-20	149.3	150.2	Jp,M	HD 5-6		90				QUALITY - HIGH
ROBBIN	PT-2	237.9	238.9	Jb,SS,G,Siy,F	HD-7		85				QUALITY - HIGH
ROBBIN	PTI-3	142.2	143	Jf,SS,R,F	HD-7		40				QUALITY - HIGH
ROBBIN	PT-10	180.3	181	Jh,F	HD-7		0				QUALITY - HIGH
ROBBIN	PT-19	105.2	106.1	Jp,F	HD-7		0				QUALITY - HIGH
ROBBIN	PT-12	227.3	228	Jt,SS,R,Siy	HD-7		70				QUALITY - HIGH
ROBBIN	PT-44	107.9	108.4	Tp,SS,R	HD-7		70				QUALITY - HIGH

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT

SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
ROBBIN	PTI-4	207.8	208.7	Tp,SS,R	HD-7		80				QUALITY - HIGH
ROBBIN	PT-38	442.6	443.4	Tp,SS,R	HD-7		80				QUALITY - HIGH
MRD	DC-119	217	223.5	Jp,G	LA		10.16			%	
MRD	DC-121	247	257.7	Jt,SiySH,G-R	LA		31.05			%	
MRD	DC-53	248.9	255.5	Tp,SS,ms,R-B	LA		16.67			%	
SAD	C-83	315	316.9	Tp,CG,G-R	P		0.000000			CM/S	
ROBBIN	PT-2	237.9	238.9	Jb,SS,G,Siy,F	PL		6.4			PSI	QUALITY - HIGH
ROBBIN	PT-10	180.3	181	Jh,F	PL		9			PSI	QUALITY - HIGH
ROBBIN	PT-19	105.2	106.1	Jp,F	PL		11			PSI	QUALITY - HIGH
ROBBIN	PT-19	130.2	131	Jp,F	PL		11			PSI	QUALITY - HIGH
ROBBIN	PT-20	109.3	110.4	Jp,GB,M	PL		8.5			PSI	QUALITY - HIGH
ROBBIN	PT-20	149.3	150.2	Jp,M	PL		8.5			PSI	QUALITY - HIGH
ROBBIN	PT-12	227.3	228	Jt,SS,R,Siy	PL		1.8			PSI	QUALITY - HIGH
ROBBIN	PT-44	107.9	108.4	Tp,SS,R	PL		7.4			PSI	QUALITY - HIGH
ROBBIN	PTI-4	207.8	208.7	Tp,SS,R	PL		7.4			PSI	QUALITY - HIGH
ROBBIN	PT-38	442.6	443.4	Tp,SS,R	PL		9.5			PSI	QUALITY - HIGH
MRD	DC-119	246.2	246.6	Jp,WEA,VS,AM,	POR		8.1			%	
MRD	DC-121	251	251.4	Jt,SiySH,G-R	POR		10.8			%	
MRD	C-146	217.6	218.1	Tp,Siy SS,R-B,F	POR		10.5			%	
MRD	C-146	161.5	162.1	Tp,SiyCS,R-B	POR		2.6			%	
MRD	DC-38	343.6	344	Tp,SiyCS,R-B	POR		2.8			%	
MRD	C-145	351.5	352	Tp,SiyCS,R-B	POR		2			%	
MRD	DC-23	467.5	467.9	Tp,SiyCS,R-B	POR		9.1			%	
MRD	C-146	325.4	326	Tp,SiyCS/SH,R-B	POR		8.3			%	
MRD	C-145	350.4	350.9	Tp,SiySH,R-B	POR		2.8			%	
SAD	C-113	167	168.2	Jb,SH,DkG,MICCa	PT						
SAD	C-108	240	242.9	Jb,SS,G-R,F,XCa	PT						
SAD	C-88	316.8	318.2	Jo,DkG,F,Ms	PT						
SAD	C-89	152.8	154.8	Jo,DkG,VF,Ms	PT						
SAD	C-118	184.9	186.6	Jp,DkG,AM	PT						
SAD	C-123	147.6	149.4	Jp,DkG,Ms	PT						
SAD	C-97	156	159.7	Jp,GB,R-G,M	PT						
SAD	C-95	171.7	174.2	Jp,GR-BK,M	PT						
SAD	C-99D	182	183.1	Jp,GR-BK,VS,AM	PT						
SAD	C-117	156.4	158.6	Jp,GR-G,SIWEAAM	PT						
SAD	DC-115	155.3	157.3	Jp,GR-G,WEA,AM	PT						
SAD	C-116	163.4	165.2	Jp,GR-G,WEA,AM	PT						
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	PT						
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	PT						
SAD	C-100A	167.7	169.2	Jt,SS,G-R,F,MIC	PT						
SAD	C-125	158.7	160.3	Jt,SSDkR-B,F,Ca	PT						
SAD	C-124	164.7	166.4	Tp,DkG,AM	PT						
SAD	C-28	448.4	449.9	Tp,SH,R,Sd,Ca	PT						
SAD	C-19	357.3	358.9	Tp,SH,Si,DkRGYP	PT						
SAD	C-26	358.8	360.1	Tp,SiySH,DkR-B	PT						
SAD	C-77	204.7	207.3	Tp,SiySS,R-B,F	PT						
SAD	C-79	211.2	212.9	Tp,SS,B-G,CaMIC	PT						
SAD	C-83	315	316.9	Tp,SS,CG,G-R	PT						
SAD	C-71	178.8	180.7	Tp,SS,G-R,AK,Ca	PT						
SAD	C-62	181.7	183.2	Tp,SS,G-R,AK,Ca	PT						
SAD	C-68	184.1	186.6	Tp,SS,G-R,AK,Ca	PT						
SAD	C-62	197	198.1	Tp,SS,G-R,AK,Ca	PT						
SAD	DC-61	214.8	216.6	Tp,SS,G-R,AK,Ca	PT						
SAD	C-59	311.6	315.4	Tp,SS,G-R,AK,Ca	PT						
SAD	C-68	159.4	161.8	Tp,SS,G-R,MIC	PT						
SAD	C-75	218.1	219.8	Tp,SS,G-R,MIC	PT						

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT

SUMMARY OF ROCK MECHANICS LABORATORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
SAD	C-82	237.2	239.8	Tp,SS,G-R,MICCa	PT						
SAD	C-78	211.7	213.6	Tp,SS,R,F,Ca	PT						
SAD	C-72	171.9	173.7	Tp,SS,R-B,AF.Ca	PT						
SAD	C-60A	239.3	241	Tp,SS,R-B,AK,Ca	PT						
SAD	C-63	200.2	201.1	Tp,SSG-R,MIC	PT						
SAD	C-63	218.1	219.1	Tp,SSG-R,MIC,Ca	PT						
SAD	C-63	222.6	224	Tp,SSG-R,MIC,Ca	PT						
SAD	C-28	459.9	461.9	Tp,SS/SH,R-B,Ca	PT						
SAD	C-88	316.8	318.2	Jo,DkG,F,Ms	PTS						
SAD	C-89	152.8	154.8	Jo,DkG,VF,Ms	PTS						
SAD	C-118	184.9	186.6	Jp,DkG,AM	PTS						
SAD	C-123	147.6	149.4	Jp,DkG,Ms	PTS						
SAD	C-97	156	159.7	Jp,GB,R-G,M	PTS						
SAD	C-95	171.7	174.2	Jp,GR-BK,M	PTS						
SAD	C-99D	182	183.1	Jp,GR-BK,VS,AM	PTS						
SAD	C-117	156.4	158.6	Jp,GR-G,SIWEAAM	PTS						
SAD	DC-115	155.3	157.3	Jp,GR-G,WEA,AM	PTS						
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	PTS						
SAD	C-19	357.3	358.9	Tp,SH,Si,DkRGYP	PTS						
SAD	C-26	358.8	360.1	Tp,SiySH,DkR-B	PTS						
SAD	C-77	204.7	207.3	Tp,SiySS,R-B,F	PTS						
SAD	C-79	211.2	212.9	Tp,SS,B-G,CaMIC	PTS						
SAD	C-83	315	316.9	Tp,SS,CG,G-R	PTS						
SAD	DC-61	214.8	216.6	Tp,SS,G-R,AK,Ca	PTS						
SAD	C-75	218.1	219.8	Tp,SS,G-R,MIC	PTS						
SAD	C-78	211.7	213.6	Tp,SS,R,F,Ca	PTS						
SAD	C-28	459.9	461.9	Tp,SS/SH,R-B,Ca	PTS						
SAD	C-113	167	168.2	Jb,SH,DkG,MICCa	PTX						
SAD	C-108	240	242.9	Jb,SS,G-R,F,XCa	PTX						
SAD	C-88	316.8	318.2	Jo,DkG,F,Ms	PTX						
SAD	C-89	152.8	154.8	Jo,DkG,VF,Ms	PTX						
SAD	C-123	147.6	149.4	Jp,DkG,Ms	PTX						
SAD	C-97	156	159.7	Jp,GB,R-G,M	PTX						
SAD	C-95	171.7	174.2	Jp,GR-BK,M	PTX						
SAD	C-99D -	182	183.1	Jp,GR-BK,VS,AM	PTX						
SAD	C-117	156.4	158.6	Jp,GR-G,SIWEAAM	PTX						
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	PTX						
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	PTX						
SAD	C-100A	167.7	169.2	Jt,SS,G-R,F,MIC	PTX						
SAD	C-28	448.4	449.9	Tp,SH,R,Sd,Ca	PTX						
SAD	C-19	357.3	358.9	Tp,SH,Si,DkRGYP	PTX						
SAD	C-26	358.8	360.1	Tp,SiySH,DkR-B	PTX						
SAD	C-77	204.7	207.3	Tp,SiySS,R-B,F	PTX						
SAD	C-79	211.2	212.9	Tp,SS,B-G,CaMIC	PTX						
SAD	C-83	315	316.9	Tp,SS,CG,G-R	PTX						
SAD	C-71	178.8	180.7	Tp,SS,G-R,AK,Ca	PTX						
SAD	C-68	184.1	186.6	Tp,SS,G-R,AK,Ca	PTX						
SAD	C-62	197	198.1	Tp,SS,G-R,AK,Ca	PTX						
SAD	DC-61	214.8	216.6	Tp,SS,G-R,AK,Ca	PTX						
SAD	C-59	311.6	315.4	Tp,SS,G-R,AK,Ca	PTX						
SAD	C-68	159.4	161.8	Tp,SS,G-R,MIC	PTX						
SAD	C-75	218.1	219.8	Tp,SS,G-R,MIC	PTX						
SAD	C-82	237.2	239.8	Tp,SS,G-R,MICCa	PTX						
SAD	C-78	211.7	213.6	Tp,SS,R,F,Ca	PTX						
SAD	C-72	171.9	173.7	Tp,SS,R-B,AF.Ca	PTX						
SAD	C-63	222.6	224	Tp,SSG-R,MIC,Ca	PTX						

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	RH		25.7				
SAD	C-88	316.8	318.2	Jo,DkG,F	RH		36.7				
SAD	C-118	184.9	186.6	Jp,AM,DkG	RH		31.4				
SAD	C-124	164.7	166.4	Jp,DkG,VS	RH		24.6				
SAD	C-97	156	159.7	Jp,GB,M	RH		35.7				
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	RH		30.6				
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	RH		18				
SAD	C-28	448.4	449.9	Tp,SH,R-B,Ca	RH		*				No reading, saturated
SAD	C-28	459.9	461.9	Tp,SH,R-B,Ca	RH		*				No reading, saturated
SAD	C-60A	239.3	241	Tp,SS,DkR-B,F	RH		*				No reading, saturated
SAD	C-72	171.9	173.7	Tp,SS,DkR-B,F-M	RH		27.7				
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	RH		12.2				
SAD	C-62	181.7	183.2	Tp,SS,G-R,F,Siy	RH		*				No reading, saturated
SAD	DC-61	214.8	216.6	Tp,SS,G-R,M	RH		*				No reading, saturated
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	RH		22				
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	RH		*				No reading, saturated
SAD	C-63	200.2	201.1	Tp,SS,G-R,VF,MIC	RH		*				No reading, saturated
SAD	C-63	222.6	224	Tp,SS,G-R,VF,MIC	RH		32.5				
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	SF	300	151.4			PSI	
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	SF	150	143.1			PSI	
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	SF	450	223.6			PSI	
SAD	C-89	152.8	154.8	Jo,DkG,F	SF	200	276.4			PSI	
SAD	C-89	152.8	154.8	Jo,DkG,F	SF	100	150.0			PSI	
SAD	C-89	152.8	154.8	Jo,DkG,F	SF	400	548.6			PSI	
SAD	C-97	156	159.7	Jp,GB,M	SF	150	366.7			PSI	
SAD	C-97	156	159.7	Jp,GB,M	SF	300	387.5			PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	SF	400	241.7			PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	SF	100	101.4			PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	SF	200	136.1			PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	SF	100	119.4			PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	SF	200	168.1			PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	SF	400	250.0			PSI	
MRD	C-146	154.5	154.8	Tp,CS,Siy,R-B	SF	27.78	18.3			PSI	
MRD	C-146	154.8	155.1	Tp,CS,Siy,R-B	SF	13.89	16.4			PSI	
MRD	C-146	155.7	156	Tp,CS,Siy,R-B	SF	55.56	27.6			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SF	359.722	293.1			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SF	100	175.0			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SF	200	276.4			PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	SF	400	322.2			PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	SF	200	206.9			PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	SF	100	141.7			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	SF	450	611.1			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	SF	300	631.9			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	SF	150	265.3			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	SF	300	448.6			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	SF	150	244.4			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SF	400	551.4			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SF	100	213.9			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SF	200	277.8			PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	SF	100	76.4			PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	SF	400	400.0			PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	SF	200	130.6			PSI	
MRD	C-146	213.6	214	Tp,SS,Siy,R-B,X	SF	13.89	13.6			PSI	
MRD	C-146	214.1	214.4	Tp,SS,Siy,R-B,X	SF	27.78	13.9			PSI	
MRD	C-146	214.8	215.1	Tp,SS,Siy,R-B,X	SF	55.56	27.8			PSI	
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	SH	450	640.3			PSI	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	SH	150	375.0			PSI	
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	SH	300	448.6			PSI	
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	SH	300	769.4			PSI	
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	SH	150	741.7			PSI	
SAD	C-124	164.7	166.4	Jp,DkG,VS	SH	300	1650.0			PSI	
SAD	C-124	164.7	166.4	Jp,DkG,VS	SH	150	1563.9			PSI	
SAD	C-124	164.7	166.4	Jp,DkG,VS	SH	450	1894.4			PSI	
SAD	C-97	156	159.7	Jp,GB,M	SH	150	2627.8			PSI	
SAD	C-97	156	159.7	Jp,GB,M	SH	300	2851.4			PSI	
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	SH	150	1955.6			PSI	
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	SH	300	2179.2			PSI	
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	SH	150	1006.9			PSI	
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	SH	300	1365.3			PSI	
SAD	C-83	315	316.9	Tp,CG,G-R	SH	300	1197.2			PSI	
SAD	C-83	315	316.9	Tp,CG,G-R	SH	400	1384.7			PSI	
SAD	C-83	315	316.9	Tp,CG,G-R	SH	100	544.4			PSI	
MRD	C-146	154.5	154.8	Tp,CS,Siy,R-B	SH	27.78	58.1			PSI	
MRD	C-146	154.8	155.1	Tp,CS,Siy,R-B	SH	13.89	35.2			PSI	
MRD	C-146	155.7	156	Tp,CS,Siy,R-B	SH	55.56	80.6			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SH	200	733.3			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SH	100	529.2			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SH	400	1087.5			PSI	
SAD	C-60A	239.3	241	Tp,SS,DkR-B,F	SH	450	1731.9			PSI	
SAD	C-60A	239.3	241	Tp,SS,DkR-B,F	SH	300	231.9			PSI	
SAD	C-60A	239.3	241	Tp,SS,DkR-B,F	SH	150	509.7			PSI	
SAD	C-72	171.9	173.7	Tp,SS,DkR-B,F-M	SH	300	2138.9			PSI	
SAD	C-72	171.9	173.7	Tp,SS,DkR-B,F-M	SH	450	1670.8			PSI	
SAD	C-72	171.9	173.7	Tp,SS,DkR-B,F-M	SH	150	1426.4			PSI	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	SH	300	1384.7			PSI	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	SH	150	1323.6			PSI	
SAD	DC-61	214.8	216.6	Tp,SS,G-R,M	SH	450	1365.3			PSI	
SAD	DC-61	214.8	216.6	Tp,SS,G-R,M	SH	300	1083.3			PSI	
SAD	DC-61	214.8	216.6	Tp,SS,G-R,M	SH	150	741.7			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	SH	450	1405.6			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	SH	150	1558.3			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	SH	300	1605.6			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	SH	300	1711.1			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,CAL	SH	150	1262.5			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SH	200	1006.9			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SH	100	755.6			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SH	400	1429.2			PSI	
MRD	C-146	213.6	214	Tp,SS,Siy,R-B,X	SH	13.89	27.4			PSI	
MRD	C-146	214.1	214.4	Tp,SS,Siy,R-B,X	SH	27.78	40.0			PSI	
MRD	C-146	214.8	215.1	Tp,SS,Siy,R-B,X	SH	55.56	61.8			PSI	
SINTEF	PT-19	81.5	82.4	Jp,F	SV		57				
SINTEF	PT-20	150.2	151.1	Jp,F	SV		69				
SINTEF	PT20	113	114.1	Jp,GB	SV		66				
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	SW	300	204.2			PSI	
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	SW	450	309.7			PSI	
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	SW	150	93.1			PSI	
SAD	C-89	152.8	154.8	Jo,DkG,F	SW	400	288.9			PSI	
SAD	C-89	152.8	154.8	Jo,DkG,F	SW	100	73.6			PSI	
SAD	C-89	152.8	154.8	Jo,DkG,F	SW	200	123.6			PSI	
SAD	C-88	316.8	318.2	Jo,DkG,F	SW	150	144.4			PSI	
SAD	C-88	316.8	318.2	Jo,DkG,F	SW	300	212.5			PSI	
SAD	C-88	316.8	318.2	Jo,DkG,F	SW	450	301.4			PSI	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT

SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
SAD	C-97	156	159.7	Jp,GB,M	SW	300	226.4			PSI	
SAD	C-97	156	159.7	Jp,GB,M	SW	450	354.2			PSI	
SAD	C-97	156	159.7	Jp,GB,M	SW	150	101.4			PSI	
SAD	C-115	155.3	157.3	Jp,GR-G,AM,WEA	SW	300	218.1			PSI	
SAD	C-115	155.3	157.3	Jp,GR-G,AM,WEA	SW	150	116.7			PSI	
SAD	C-115	155.3	157.3	Jp,GR-G,AM,WEA	SW	450	330.6			PSI	
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	SW	150	105.6			PSI	
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	SW	300	212.5			PSI	
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	SW	450	311.1			PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	SW	400	245.8			PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	SW	200	119.4			PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	SW	100	48.6			PSI	
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	SW	450	345.8			PSI	
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	SW	150	130.6			PSI	
SAD	C-125	144.5	146.2	Jt,SS,DkG,M,MIC	SW	300	261.1			PSI	
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	SW	450	375.0			PSI	
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	SW	300	201.4			PSI	
SAD	C-100A	148.2	149.4	Jt,SS,G-R,F,MIC	SW	150	109.7			PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	SW	400	291.7			PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	SW	100	81.9			PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	SW	200	147.2			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SW	100	55.6			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SW	200	109.7			PSI	
SAD	C-19	357.3	358.9	Tp,SH Siy	SW	400	209.7			PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	SW	200	134.7			PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	SW	100	70.8			PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	SW	400	273.6			PSI	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	SW	150	104.2			PSI	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	SW	300	191.7			PSI	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	SW	450	301.4			PSI	
SAD	C-62	181.7	183.2	Tp,SS,G-R,F,Siy	SW	300	183.3			PSI	
SAD	C-62	181.7	183.2	Tp,SS,G-R,F,Siy	SW	450	315.3			PSI	
SAD	C-62	181.7	183.2	Tp,SS,G-R,F,Siy	SW	150	87.5			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	SW	150	81.9			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	SW	450	266.7			PSI	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	SW	300	173.6			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SW	400	255.6			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SW	200	133.3			PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	SW	100	63.9			PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	SW	400	322.2			PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	SW	200	173.6			PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	SW	100	94.4			PSI	
WES	PT 2	206	186.6	Jb,MS/SIS,R	Uc		16,540			PSI	
WES	PT 7	234	224	Jb,MS/SIS,R	Uc		17,070			PSI	
WES	PT 1	161	108.4	Jb,SIS,G	Uc		17,330			PSI	
WES	PT 9	188	108.4	Jb,SIS,R	Uc		15,130			PSI	
WES	PT 9	226	208.7	Jb,SIS,R	Uc		17,540			PSI	
WES	PT 8	255	208.7	Jb,SIS,R	Uc		15,120			PSI	
WES	PTI 1	167	443.4	Jb,SS,G,M-F	Uc		9,550			PSI	
ROBBIN	PT-2	237.9	238.9	Jb,SS,G,Siy,F	Uc		18,730			PSI	QUALITY = HIGH
ROBBIN	PT-2	237.9	238.9	Jb,SS,G,Siy,F	Uc		14,380			PSI	QUALITY = HIGH
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	Uc		14,710			PSI	
WES	PT 7	254		Jb,SS,R,F	Uc		10,220			PSI	
WES	PTI 1	96	360.1	Jb,SS,R,M	Uc		18,950			PSI	
WES	PT 3	171		Jb,SS,R,VF	Uc		13,630			PSI	
WES	PT 2	216		Jb,SS,R-G,M	Uc		10,690			PSI	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT

SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
WES	PT 2	224		Jb,SS,R–G,VF	Uc		17,580			PSI	
WES	PTI 1	96		Jb,SS/SIS,R,M	Uc		7,060			PSI	
WES	PTI 3	143		Jf,SH,DkR–B	Uc		8,530			PSI	
WES	PT 23	163	219.8	Jf,SS,DkR,F	Uc		14,010			PSI	
WES	PTI 3	155	207.3	Jf,SS,GG,R–B,F	Uc		6,290			PSI	
WES	PTI 3	132	213.6	Jf,SS,M–C	Uc		6,900			PSI	
ROBBIN	PTI–3	142.2	143	Jf,SS,R,F	Uc		24,210			PSI	QUALITY = HIGH
ROBBIN	PTI–3	142.2	143	Jf,SS,R,F	Uc		28,330			PSI	QUALITY = HIGH
WES	PTI 3	130		Jf,SS,R–B,M	Uc		9,010			PSI	
ROBBIN	PT–10	180.3	181	Jh,F	Uc		44,180			PSI	QUALITY = HIGH
ROBBIN	PT–10	180.3	181	Jh,F	Uc		51,730			PSI	QUALITY = HIGH
SAD	C–118	184.9	186.6	Jp,AM,DkG	Uc		6,653			PSI	
ROBBIN	PT–19	105.2	106.1	Jp,F	Uc		55,970			PSI	QUALITY = HIGH
ROBBIN	PT–19	105.2	106.1	Jp,F	Uc		53,070			PSI	QUALITY = HIGH
ROBBIN	PT–19	130.2	131	Jp,F	Uc		56,690			PSI	QUALITY = HIGH
ROBBIN	PT–19	130.2	131	Jp,F	Uc		54,420			PSI	QUALITY = HIGH
ROBBIN	PT–20	109.3	110.4	Jp,GB,M	Uc		29,170			PSI	QUALITY = HIGH
ROBBIN	PT–20	109.3	110.4	Jp,GB,M	Uc		32,070			PSI	QUALITY = HIGH
SAD	C–97	156	159.7	Jp,GB,M	Uc		13,705			PSI	
SAD	C–117	156.4	158.6	Jp,GR–G,AM,WEA	Uc		12,526			PSI	
ROBBIN	PT–20	149.3	150.2	Jp,M	Uc		30,830			PSI	QUALITY = HIGH
ROBBIN	PT–20	149.3	150.2	Jp,M	Uc		34,530			PSI	QUALITY = HIGH
MRD	DC–119	246.2	246.6	Jp,WEA,VS,AM,	Uc		1,969			PSI	Large vesicle in failure.
WES	PTI 2	63		Jt,SH	Uc		3,800			PSI	
WES	PTI 2	35		Jt,SH,WEA,R–B	Uc		3,910			PSI	
WES	PT 15	136		Jt,SIS,R	Uc		11,810			PSI	
WES	PT 12	214		Jt,SIS,R	Uc		16,070			PSI	
WES	PT 12	237	183.1	Jt,SIS,R	Uc		15,550			PSI	
MRD	DC–121	251	251.4	Jt,SiySH,G–R	Uc		2,711			PSI	
WES	PT 16	135	238.9	Jt,SS,DkR,F	Uc		10,820			PSI	
WES	PT 14	229	242.9	Jt,SS,DkR,F	Uc		10,690			PSI	
WES	PT 13	297		Jt,SS,DkR,VF	Uc		13,220			PSI	
WES	PT 13	312		Jt,SS,DkR,VF	Uc		15,640			PSI	
WES	PTI 2	173		Jt,SS,DkRB	Uc		8,450			PSI	
ROBBIN	PT–12	227.3	228	Jt,SS,R,Siy	Uc		12,830			PSI	QUALITY = HIGH
ROBBIN	PT–12	227.3	228	Jt,SS,R,Siy	Uc		13,040			PSI	QUALITY = HIGH
WES	PT 46	152		Tp,MS,DkR	Uc		17,130			PSI	
WES	PT 35	363		Tp,MS/SIS,DkR	Uc		16,710			PSI	
SAD	C–28	459.9	461.9	Tp,SH,R–B,Ca	Uc		6,738			PSI	
WES	PT 39	258	212.9	Tp,SIS,DkR,GG	Uc		6,420			PSI	
WES	PT 39	276	316.9	Tp,SIS,DkR,GG	Uc		3,510			PSI	
WES	PT 38	455		Tp,SIS,R	Uc		9,090			PSI	
WES	PT 44	237	360.1	Tp,SIS/MS,DkR	Uc		16,390			PSI	
WES	PT 38	428		Tp,SIS/SS,DkR,F	Uc		34,220			PSI	
WES	PT 38	435		Tp,SIS/SS,DkR,F	Uc		34,220			PSI	
MRD	C–146	217.6	218.1	Tp,Siy SS,R–B,F	Uc		3,892			PSI	Failed on healed frac.
MRD	C–146	161.5	162.1	Tp,SiyCS,R–B	Uc		6,493			PSI	Failed on healed frac.
MRD	DC–38	343.6	344	Tp,SiyCS,R–B	Uc		11,138			PSI	
MRD	C–145	351.5	352	Tp,SiyCS,R–B	Uc		10,522			PSI	Failed on healed frac.
MRD	DC–23	467.5	467.9	Tp,SiyCS,R–B	Uc		2,049			PSI	Broke along a gypsum sea
MRD	C–146	325.4	326	Tp,SiyCS/SH,R–B	Uc		4,670			PSI	Bedding plane failures.
MRD	C–145	350.4	350.9	Tp,SiySH,R–B	Uc		6,107			PSI	Bedding plane failures.
WES	PT 40	396		Tp,SS,DkR,C	Uc		13,630			PSI	
WES	PT 39	386		Tp,SS,DkR,F	Uc		13,670			PSI	
WES	PT 38	445	181	Tp,SS,DkR,F	Uc		7,580			PSI	
WES	PT 38	445		Tp,SS,DkR,F	Uc		13,410			PSI	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
WES	PT 39	173		Tp,SS,DkR,GG	Uc		1,680			PSI	
WES	PT 35	334		Tp,SS,DkR,M	Uc		18,640			PSI	
WES	PT 44	224		Tp,SS,DkR,VF	Uc		13,360			PSI	
SAD	C-60A	239.3	241	Tp,SS,DkR-B,F	Uc		12,027			PSI	
WES	PTI 4	163		Tp,SS,DkR-B,F-M	Uc		15,840			PSI	
SAD	C-72	171.9	173.7	Tp,SS,DkR-B,F-M	Uc		18,264			PSI	
WES	PTI 4	173	154.8	Tp,SS,G,F	Uc		15,990			PSI	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	Uc		5,860			PSI	
SAD	DC-61	214.8	216.6	Tp,SS,G-R,M	Uc		5,440			PSI	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	Uc		9,387			PSI	
SAD	C-63	222.6	224	Tp,SS,G-R,VF,MIC	Uc		10,173			PSI	
ROBBIN	PT-44	107.9	108.4	Tp,SS,R	Uc		21,420			PSI	QUALITY = HIGH
ROBBIN	PT-44	107.9	108.4	Tp,SS,R	Uc		22,350			PSI	QUALITY = HIGH
ROBBIN	PTI-4	207.8	208.7	Tp,SS,R	Uc		25,140			PSI	QUALITY = HIGH
ROBBIN	PTI-4	207.8	208.7	Tp,SS,R	Uc		15,930			PSI	QUALITY = HIGH
ROBBIN	PT-38	442.6	443.4	Tp,SS,R	Uc		24,930			PSI	QUALITY = HIGH
ROBBIN	PT-38	442.6	443.4	Tp,SS,R	Uc		29,070			PSI	QUALITY = HIGH
WES	PT 1	179	114.1	Jb,MS,BK	UcER		11,470	1.6	0.18	PSI	
WES	PT 1	168	238.9	Jb,MS,DkG	UcER		8,130	1.6	0.25	PSI	
WES	PTI 1	184	242.9	Jb,MS,DkG	UcER		11,440	1.9	0.21	PSI	
WES	PT 5	215		Jb,MS,DkG	UcER		10,660	2.3	0.17	PSI	
WES	PTI 1	134		Jb,MS/SS,DkB,F	UcER		18,570	3.35	0.28	PSI	
WES	PT 3	206	110.4	Jb,MS/SS,DkG,VF	UcER		16,400	7.9	0.23	PSI	
WES	PT 3	182	159.7	Jb,SIS,R	UcER		16,110	3.9	0.19	PSI	
WES	PT 9	194	157.3	Jb,SIS,R	UcER		14,900	2.4	0.29	PSI	
WES	PT 5	240	158.6	Jb,SIS,R	UcER		16,600	3.55	0.27	PSI	
WES	PT 7	250		Jb,SIS,R	UcER		13,830	2.45	0.21	PSI	
WES	PT 8	255		Jb,SIS,R	UcER		16,160	2.3	0.23	PSI	
WES	PT 7	273	150.2	Jb,SIS,R	UcER		17,170	3.25	0.29	PSI	
WES	PT 9	211	174.2	Jb,SS,R,M	UcER		6,110	1.55	0.34	PSI	
WES	PT 2	186		Jb,SS,R,VF	UcER		17,070	7.15	0.27	PSI	
WES	PT 8	224		Jb,SS,R,VF	UcER		9,080	2.1	0.26	PSI	
WES	PT 26	167		Jf,MS,DkR	UcER		16,270	8	0.25	PSI	
WES	PT 23	177		Jf,SIS,DkR	UcER		5,450	2.2	0.1	PSI	
WES	PT 26	189		Jf,SS,DkR,F	UcER		13,820	2.9	0.22	PSI	
WES	PT 11	153		Jh,DkG	UcER		21,830	9.65	0.28	PSI	
WES	PT 10	164	143	Jh,DkG	UcER		12,390	9.4	0.22	PSI	
WES	PT 11	177		Jh,DkG	UcER		9,150	6.3	0.33	PSI	
WES	PT 12	20		Jh,DkGN/G	UcER		48,400	26.25	0.25	PSI	
WES	PT 10	143		Jh,slVS	UcER		23,100	8.35	0.34	PSI	
WES	PT 10	157		Jh,slVS	UcER		22,300	8.65	0.24	PSI	
WES	PT 10	137		Jh,VS	UcER		14,410	4.25	0.31	PSI	
WES	PT 29	286		Jo,DkG	UcER		10,060	12.35	0.16	PSI	
WES	PT 29	306	181	Jo,DkG	UcER		11,540	10.65	0.26	PSI	
WES	PT 18	143		Jp,DkG	UcER		15,250	12.8	0.37	PSI	
WES	PT 18	154		Jp,DkG	UcER		48,580	13.35		PSI	
WES	PT 17	156		Jp,DkG	UcER		16,480	22.35	0.25	PSI	
WES	PT 20	42		Jp,GB,DkG	UcER		34,210	11.65	0.26	PSI	
WES	PT 20	110		Jp,GB,DkG	UcER		29,540	19.25	0.29	PSI	
WES	PT 20	174		Jp,GB,DkG	UcER		27,040	9.3	0.28	PSI	
WES	PT 20	174		Jp,GB,DkG	UcER		26,940	12.8	0.26	PSI	
WES	PT 19	132	154.8	Jp,G/BK	UcER		46,850	12.2	0.28	PSI	
SAD	C-95	171.7	174.2	Jp,M-GB	UcER		16,521	3.52	0.61	PSI	
WES	PT 17	148		Jp,slAM,DkG	UcER		24,950	7.2	0.24	PSI	
WES	PT 19	157		Jp,slVS,BL/G	UcER		24,000	8.05	0.22	PSI	
WES	PT 19	157		Jp,slVS,BL/G	UcER		20,130		0.38	PSI	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT

SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
WES	PT 17	180		Jp,slVS,BL/G	UcER		7,720	5.45	0.13	PSI	
WES	PT 17	166	166.4	Jp,slVS,DkG	UcER		19,470	10.75	0.22	PSI	
WES	PT 19	166	82.4	Jp,slVS,G/BK	UcER		23,120	8.3	0.27	PSI	
SAD	C-99D	182	183.1	Jp,VS,GR-BK	UcER		11,250	2.47	0.06	PSI	
WES	PT 15	181	131	Jt,MS,BK	UcER		14,010	9.55	0.18	PSI	
WES	PT 13	275	151.1	Jt,MS,BK	UcER		18,040	4.95	0.28	PSI	
WES	PT 14	206	114.1	Jt,MS,R	UcER		16,280	2.25	0.36	PSI	
WES	PT 15	147		Jt,SIS,R	UcER		13,220	2.4	0.21	PSI	
WES	PT 12	221		Jt,SIS,R	UcER		13,600	3.9	0.18	PSI	
WES	PT 16	157		Jt,SIS/MS,DkR	UcER		14,700	2.45	0.31	PSI	
WES	PT 16	165	461.9	Jt,SIS/MS,DkR	UcER		12,650	2.8	0.27	PSI	
WES	PT 15	164	110.4	Jt,SS,DkG,VF	UcER		18,920	10.7	0.17	PSI	
WES	PT 16	146	159.7	Jt,SS,DkR,VF	UcER		9,780	5.6	0.29	PSI	
WES	PT 14	218	157.3	Jt,SS,DkR,VF	UcER		16,820	3.1	0.2	PSI	
WES	PTI 2	147	158.6	Jt,SS,R-W,C	UcER		14,050	2.65	0.29	PSI	
SAD	C-79	211.2	212.9	Tp,B-G,M	UcER		14,470	1.42	0.12	PSI	
SAD	C-83	315	316.9	Tp,CG,G-R	UcER		6,276	0.009	0.12	PSI	
WES	PT 46	172	150.2	Tp,MS,DkR	UcER		13,350	6.35	0.22	PSI	
SAD	C-26	358.8	360.1	Tp,SH Siy	UcER		18,997	1.5	0.11	PSI	
WES	PT 35	349		Tp,SS,C-AK	UcER		17,260	5.7	0.24	PSI	
WES	PT 39	349		Tp,SS,DkR,C-VC	UcER		9,600	3.1	0.23	PSI	
WES	PT 40	362		Tp,SS,DkR,C-VC	UcER		6,790	3.5	0.24	PSI	
WES	PT 35	329		Tp,SS,DkR,F	UcER		11,430	2.85	0.34	PSI	
WES	PT 39	355		Tp,SS,DkR,GG F	UcER		2,860	2.6	0.15	PSI	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	UcER		9,724	1.35	0.13	PSI	
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	UcER		16,670	2	0.14	PSI	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	UcER		19,714	1.72	0.57	PSI	
WES	PT 1	179		Jb,MS,BK	UW		160			PCF	
WES	PT 1	168		Jb,MS,DkG	UW		159.3			PCF	
WES	PTI 1	184		Jb,MS,DkG	UW		159.2			PCF	
WES	PT 5	215		Jb,MS,DkG	UW		162.6			PCF	
WES	PT 2	206		Jb,MS/SIS,R	UW		161.9			PCF	
WES	PT 7	234		Jb,MS/SIS,R	UW		161.7			PCF	
WES	PTI 1	134		Jb,MS/SS,DkB,F	UW		159.4			PCF	
WES	PT 3	206	241	Jb,MS/SS,DkG,VF	UW		167.7			PCF	
SAD	C-113	167	168.2	Jb,SH,DkG,MIC	UW		149			PCF	
WES	PT 1	161	173.7	Jb,SIS,G	UW		163.4			PCF	
WES	PT 3	182		Jb,SIS,R	UW		162.2			PCF	
WES	PT 9	188	219.8	Jb,SIS,R	UW		153.8			PCF	
WES	PT 9	194	251.4	Jb,SIS,R	UW		152.5			PCF	
WES	PT 9	226	183.2	Jb,SIS,R	UW		155			PCF	
WES	PT 5	228	216.6	Jb,SIS,R	UW		155.1			PCF	
WES	PT 5	240	186.6	Jb,SIS,R	UW		151.3			PCF	
WES	PT 8	241	315.4	Jb,SIS,R	UW		162.2			PCF	
WES	PT 5	248	224	Jb,SIS,R	UW		161.1			PCF	
WES	PT 7	250	108.4	Jb,SIS,R	UW		161.8			PCF	
WES	PT 8	255	208.7	Jb,SIS,R	UW		159			PCF	
WES	PT 8	255	443.4	Jb,SIS,R	UW		161.2			PCF	
WES	PT 7	273	169.2	Jb,SIS,R	UW		159.2			PCF	
WES	PTI 1	167	228	Jb,SS,G,M-F	UW		156.9			PCF	
ROBBIN	PT-2	237.9	238.9	Jb,SS,G,Siy,F	UW		163.5			PCF	QUALITY - HIGH
SAD	C-108	240	242.9	Jb,SS,G-R,F,X	UW		161.1			PCF	
WES	PT 7	254	212.9	Jb,SS,R,F	UW		146.6			PCF	
WES	PTI 1	96	316.9	Jb,SS,R,M	UW		155.5			PCF	
WES	PT 9	211		Jb,SS,R,M	UW		142.8			PCF	
WES	PT 3	171		Jb,SS,R,VF	UW		161.9			PCF	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
WES	PT 2	186		Jb,SS,R,VF	UW		152.6			PCF	
WES	PT 8	224	461.9	Jb,SS,R,VF	UW		144.6			PCF	
WES	PT 2	216	358.9	Jb,SS,R-G,M	UW		161.3			PCF	
WES	PT 2	224	360.1	Jb,SS,R-G,VF	UW		161			PCF	
WES	PTI 1	96		Jb,SS/SIS,R,M	UW		155.3			PCF	
WES	PT 26	167		Jf,MS,DkR	UW		164.2			PCF	
WES	PTI 3	143		Jf,SH,DkR-B	UW		166.6			PCF	
WES	PT 23	177		Jf,SIS,DkR	UW		143.2			PCF	
WES	PT 23	163		Jf,SS,DkR,F	UW		151.3			PCF	
WES	PT 26	189		Jf,SS,DkR,F	UW		151.3			PCF	
WES	PTI 3	155		Jf,SS,GG,R-B,F	UW		164.5			PCF	
WES	PTI 3	132	218.1	Jf,SS,M-C	UW		142.1			PCF	
ROBBIN	PTI-3	142.2	143	Jf,SS,R,F	UW		166			PCF	QUALITY - HIGH
WES	PTI 3	130	344	Jf,SS,R-B,M	UW		150.3			PCF	
WES	PT 11	141	352	Jh,DkG	UW		186.8			PCF	
WES	PT 11	153	467.9	Jh,DkG	UW		181.3			PCF	
WES	PT 10	164	326	Jh,DkG	UW		175.1			PCF	
WES	PT 11	177	350.9	Jh,DkG	UW		167.2			PCF	
WES	PT 12	20		Jh,DkGN/G	UW		184.3			PCF	
ROBBIN	PT-10	180.3	181	Jh,F	UW		182.8			PCF	QUALITY - HIGH
WES	PT 10	143		Jh,slVS	UW		178			PCF	
WES	PT 10	157		Jh,slVS	UW		176.9			PCF	
WES	PT 10	137		Jh,VS	UW		166.8			PCF	
WES	PT 11	165		Jh,VS	UW		156.5			PCF	
WES	PT 29	286		Jo,DkG	UW		182.3			PCF	
WES	PT 29	297		Jo,DkG	UW		181.8			PCF	
WES	PT 29	306		Jo,DkG	UW		181			PCF	
SAD	C-89	152.8	154.8	Jo,DkG,F	UW		179			PCF	
SAD	C-118	184.9	186.6	Jp,AM,DkG	UW		168.6			PCF	
WES	PT 18	136		Jp,DkG	UW		181.6			PCF	
WES	PT 18	143		Jp,DkG	UW		181.5			PCF	
WES	PT 18	154		Jp,DkG	UW		182.8			PCF	
WES	PT 17	156	241	Jp,DkG	UW		177.7			PCF	
SAD	C-124	164.7	166.4	Jp,DkG,VS	UW		161.1			PCF	
SINTEF	PT-19	81.5	82.4	Jp,F	UW		181.6			PCF	
ROBBIN	PT-19	105.2	106.1	Jp,F	UW		185.3			PCF	QUALITY - HIGH
ROBBIN	PT-19	130.2	131	Jp,F	UW		182.2			PCF	QUALITY - HIGH
SINTEF	PT-20	150.2	151.1	Jp,F	UW		182.2			PCF	
SINTEF	PT20	113	114.1	Jp,GB	UW		182.2			PCF	
WES	PT 20	42	216.6	Jp,GB,DkG	UW		182.2			PCF	
WES	PT 20	110	186.6	Jp,GB,DkG	UW		180.6			PCF	
WES	PT 20	174	315.4	Jp,GB,DkG	UW		179.8			PCF	
WES	PT 20	174	224	Jp,GB,DkG	UW		183.9			PCF	
ROBBIN	PT-20	109.3	110.4	Jp,GB,M	UW		177.2			PCF	QUALITY - HIGH
SAD	C-97	156	159.7	Jp,GB,M	UW		183.7			PCF	
SAD	C-115	155.3	157.3	Jp,GR-G,AM,WEA	UW		167			PCF	
SAD	C-117	156.4	158.6	Jp,GR-G,AM,WEA	UW		172.9			PCF	
WES	PT 19	132	213.6	Jp,G/BK	UW		181.9			PCF	
WES	PT 18	170		Jp,G/BK	UW		182.5			PCF	
ROBBIN	PT-20	149.3	150.2	Jp,M	UW		181.6			PCF	QUALITY - HIGH
SAD	C-95	171.7	174.2	Jp,M-GB	UW		186			PCF	
WES	PT 17	148		Jp,slAM,DkG	UW		174			PCF	
WES	PT 19	157		Jp,slVS,BL/G	UW		172.3			PCF	
WES	PT 19	157		Jp,slVS,BL/G	UW		172.3			PCF	
WES	PT 17	180		Jp,slVS,BL/G	UW		154.7			PCF	
WES	PT 17	166		Jp,slVS,DkG	UW		177.6			PCF	

For an explanation of abbreviations see the attached legend sheet.

PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^{6}	POISSONS RATIO	UNITS	REMARKS
WES	PT 19	166		Jp,slVS,G/BK	UW		174.3			PCF	
SAD	C-99D	182	183.1	Jp,VS,GR-BK	UW		166			PCF	
MRD	DC-119	246.2	246.6	Jp,WEA,VS,AM,	UW		152.3			PCF	Low unit weight = wea. rk
WES	PT 15	181		Jt,MS,BK	UW		165			PCF	
WES	PT 13	275		Jt,MS,BK	UW		168.4			PCF	
WES	PT 14	206		Jt,MS,R	UW		159.8			PCF	
WES	PTI 2	63		Jt,SH	UW		154.6			PCF	
WES	PTI 2	35		Jt,SH,WEA,R-B	UW		140.1			PCF	
WES	PT 15	136		Jt,SIS,R	UW		142			PCF	
WES	PT 15	147		Jt,SIS,R	UW		148.3			PCF	
WES	PT 12	214		Jt,SIS,R	UW		165			PCF	
WES	PT 12	221		Jt,SIS,R	UW		160.2			PCF	
WES	PT 12	237		Jt,SIS,R	UW		163.3			PCF	
WES	PT 16	157		Jt,SIS/MS,DkR	UW		144.2			PCF	
WES	PT 16	165		Jt,SIS/MS,DkR	UW		156.9			PCF	
MRD	DC-121	251	251.4	Jt,SiySH,G-R	UW		159.1			PCF	
WES	PT 15	164		Jt,SS,DkG,VF	UW		151.4			PCF	
WES	PT 16	135		Jt,SS,DkR,F	UW		143			PCF	
WES	PT 14	229		Jt,SS,DkR,F	UW		152.6			PCF	
WES	PT 16	146		Jt,SS,DkR,VF	UW		158.9			PCF	
WES	PT 14	218		Jt,SS,DkR,VF	UW		150.9			PCF	
WES	PT 13	297		Jt,SS,DkR,VF	UW		144.1			PCF	
WES	PT 13	312		Jt,SS,DkR,VF	UW		165.5			PCF	
WES	PTI 2	173		Jt,SS,DkRB	UW		156.5			PCF	
SAD	C-100A	167.7	169.2	Jt,SS,G-R,F,MIC	UW		156.8			PCF	
ROBBIN	PT-12	227.3	228	Jt,SS,R,Siy	UW		164.7			PCF	QUALITY = HIGH
SAD	C-125	158.7	160.3	Jt,SS,R-B,F-M	UW		159.8			PCF	
WES	PTI 2	147		Jt,SS,R-W,C	UW		153.1			PCF	
SAD	C-79	211.2	212.9	Tp,B-G,M	UW		147			PCF	
SAD	C-83	315	316.9	Tp,CG,G-R	UW		150			PCF	
WES	PT 46	152		Tp,MS,DkR	UW		161.1			PCF	
WES	PT 46	172		Tp,MS,DkR	UW		161.8			PCF	
WES	PT 35	363		Tp,MS/SIS,DkR	UW		160.6			PCF	
SAD	C-28	459.9	461.9	Tp,SH,R-B,Ca	UW		165.9			PCF	
SAD	C-19	357.3	358.9	Tp,SH Siy	UW		158			PCF	
SAD	C-26	358.8	360.1	Tp,SH Siy	UW		164			PCF	
WES	PT 39	258		Tp,SIS,DkR,GG	UW		153.8			PCF	
WES	PT 39	276		Tp,SIS,DkR,GG	UW		154.8			PCF	
WES	PT 46	177		Tp,SIS,R	UW		160.2			PCF	
WES	PT 38	455		Tp,SIS,R	UW		154.9			PCF	
WES	PT 44	237		Tp,SIS/MS,DkR	UW		162.5			PCF	
WES	PT 38	428		Tp,SIS/SS,DkR,F	UW		162.9			PCF	
WES	PT 38	435		Tp,SIS/SS,DkR,F	UW		158.6			PCF	
MRD	C-146	217.6	218.1	Tp,Siy SS,R-B,F	UW		148.5			PCF	
MRD	C-146	161.5	162.1	Tp,SiyCS,R-B	UW		161.6			PCF	
MRD	DC-38	343.6	344	Tp,SiyCS,R-B	UW		161.6			PCF	
MRD	C-145	351.5	352	Tp,SiyCS,R-B	UW		166.6			PCF	
MRD	DC-23	467.5	467.9	Tp,SiyCS,R-B	UW		160.4			PCF	
MRD	C-146	325.4	326	Tp,SiyCS/SH,R-B	UW		167.2			PCF	
MRD	C-145	350.4	350.9	Tp,SiySH,R-B	UW		167.2			PCF	
WES	PT 35	349		Tp,SS,C-AK	UW		156.7			PCF	
WES	PT 40	396		Tp,SS,DkR,C	UW		162			PCF	
WES	PT 39	349		Tp,SS,DkR,C-VC	UW		149.2			PCF	
WES	PT 40	362		Tp,SS,DkR,C-VC	UW		142.6			PCF	
WES	PT 35	329		Tp,SS,DkR,F	UW		147.9			PCF	
WES	PT 39	386		Tp,SS,DkR,F	UW		158.5			PCF	

For an explanation of abbreviations see the attached legend sheet.

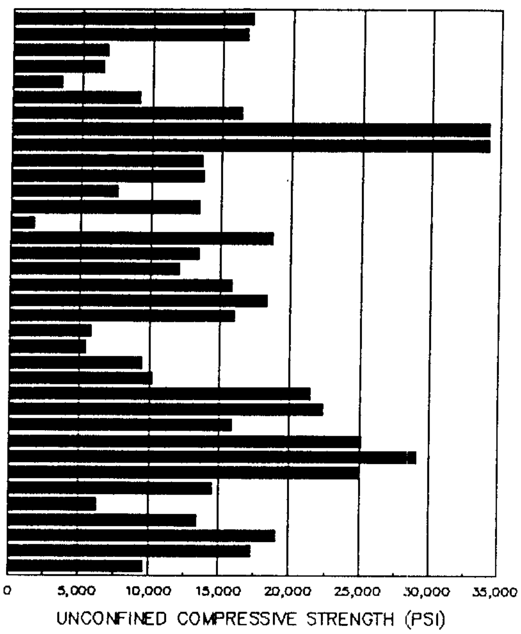
PASSAIC RIVER TUNNEL PROJECT
SUMMARY OF ROCK MECHANICS LABORITORY TESTING

LAB	HOLE NUMBER	DEPTHS TOP	DEPTHS BOTTOM	ROCK TYPE	TEST TYPE	NORMAL LOAD (PSI)	TEST RESULT	"E" MODULUS x10^6	POISSONS RATIO	UNITS	REMARKS
WES	PT 38	445		Tp,SS,DkR,F	UW		157.4			PCF	
WES	PT 38	445		Tp,SS,DkR,F	UW		158.9			PCF	
WES	PT 39	173		Tp,SS,DkR,GG	UW		146.3			PCF	
WES	PT 39	355		Tp,SS,DkR,GG F	UW		154.9			PCF	
WES	PT 35	334		Tp,SS,DkR,M	UW		164.7			PCF	
WES	PT 44	224		Tp,SS,DkR,VF	UW		164.4			PCF	
WES	PT 44	230		Tp,SS,DkR,VF	UW		159.9			PCF	
WES	PT 44	253		Tp,SS,DkR,VF	UW		162.5			PCF	
SAD	C-60A	239.3	241	Tp,SS,DkR-B,F	UW		164.6			PCF	
WES	PTI 4	163		Tp,SS,DkR-B,F-M	UW		142.2			PCF	
SAD	C-72	171.9	173.7	Tp,SS,DkR-B,F-M	UW		155.6			PCF	
WES	PTI 4	173		Tp,SS,G,F	UW		163.2			PCF	
SAD	C-75	218.1	219.8	Tp,SS,G-R,F	UW		167			PCF	
SAD	C-82	237.2	239.8	Tp,SS,G-R,F,MIC	UW		165			PCF	
SAD	C-62	181.7	183.2	Tp,SS,G-R,F,Siy	UW		148.2			PCF	
SAD	DC-61	214.8	216.6	Tp,SS,G-R,M	UW		145.8			PCF	
SAD	C-68	184.1	186.6	Tp,SS,G-R,M,Ca	UW		149.8			PCF	
SAD	C-59	311.6	315.4	Tp,SS,G-R,M,Ca	UW		156.9			PCF	
SAD	C-63	222.6	224	Tp,SS,G-R,VF,MIC	UW		163.9			PCF	
ROBBIN	PT-44	107.9	108.4	Tp,SS,R	UW		159.7			PCF	QUALITY = HIGH
ROBBIN	PTI-4	207.8	208.7	Tp,SS,R	UW		157.2			PCF	QUALITY = HIGH
ROBBIN	PT-38	442.6	443.4	Tp,SS,R	UW		162.9			PCF	QUALITY = HIGH
SAD	C-77	204.7	207.3	Tp,SS,R-B,F	UW		164			PCF	
SAD	C-78	211.7	213.6	Tp,SS,R-B,F	UW		163			PCF	

For an explanation of abbreviations see the attached legend sheet.

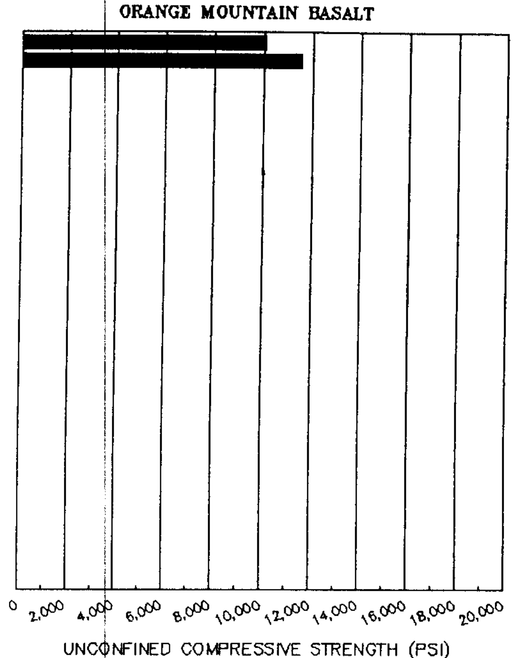
PASSAIC TUNNEL PROJECT
UNCONFINED COMPRESSION STRENGTH TESTING

PASSAIC FORMATION



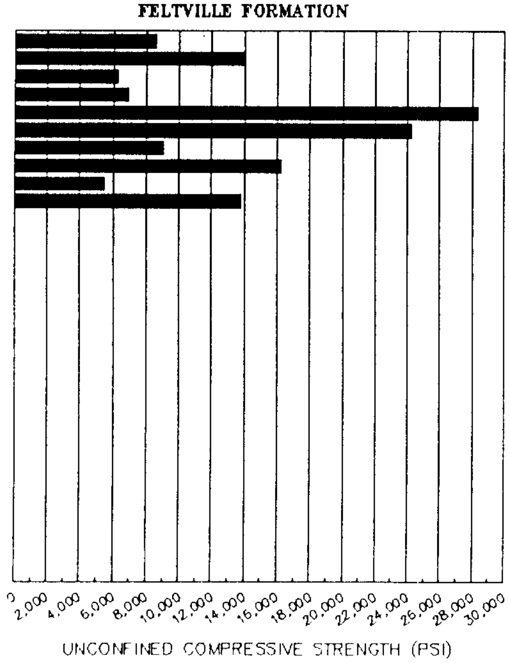
Laboratory test data from all sources.

ORANGE MOUNTAIN BASALT



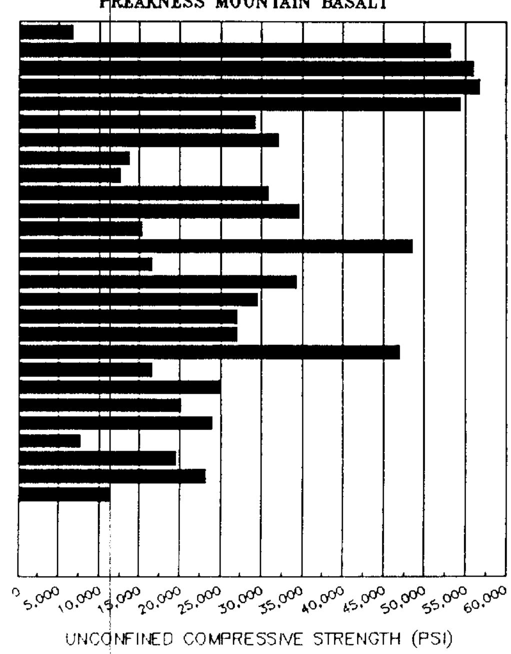
Laboratory test data from all sources.

FELTVILLE FORMATION



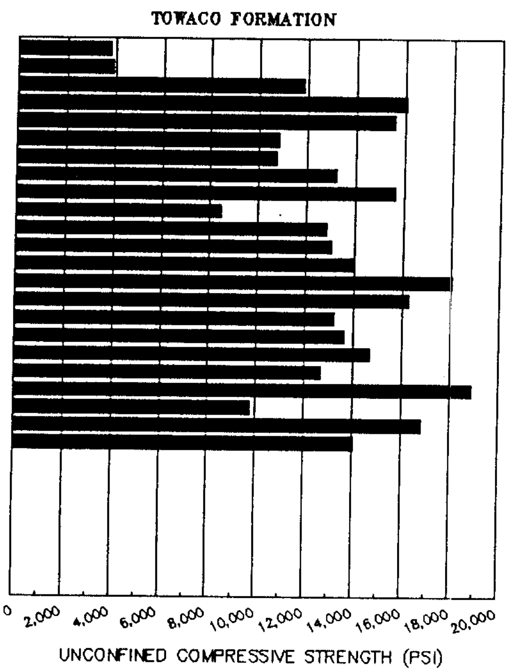
Laboratory test data from all sources.

PREAKNESS MOUNTAIN BASALT



Laboratory test data from all sources.

PASSAIC TUNNEL PROJECT
UNCONFINED COMPRESSION STRENGTH TESTING



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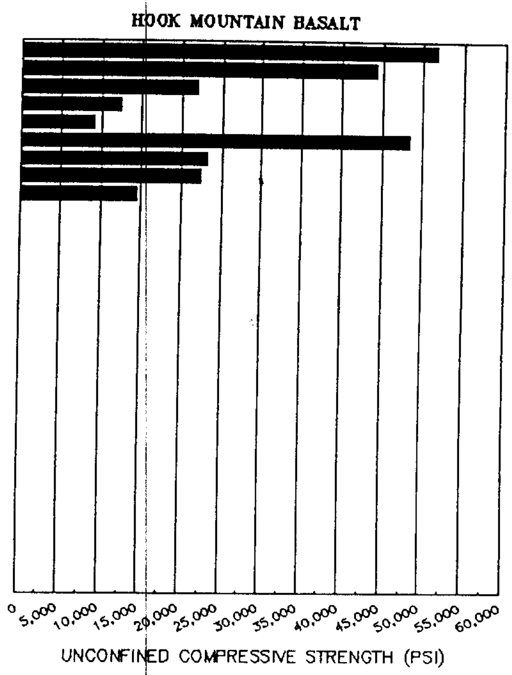

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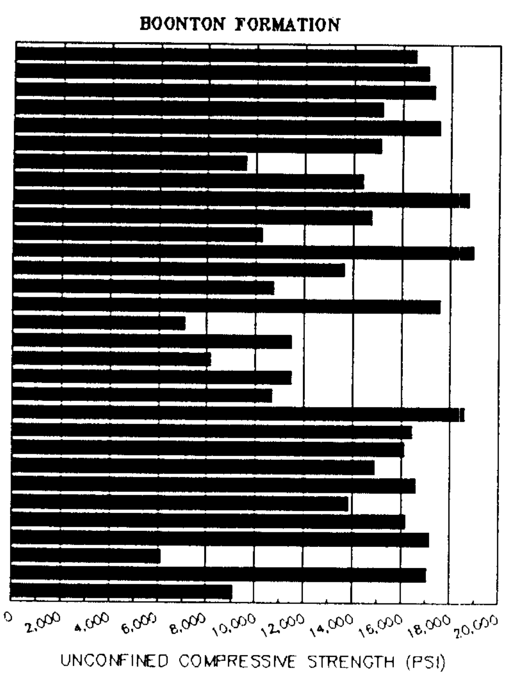
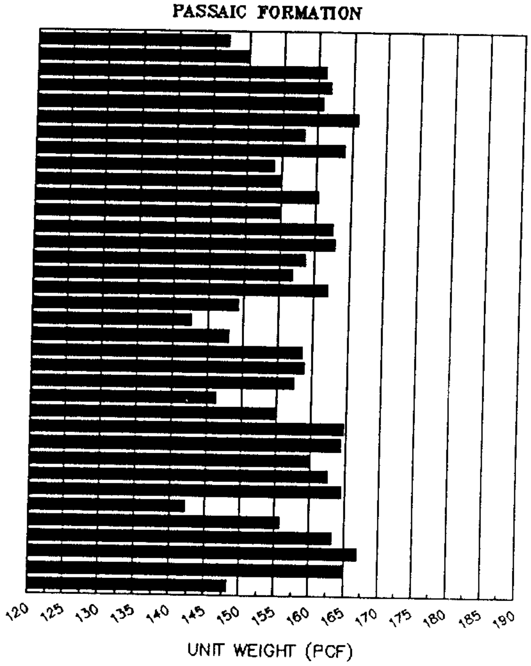


Laboratory test data from all sources.

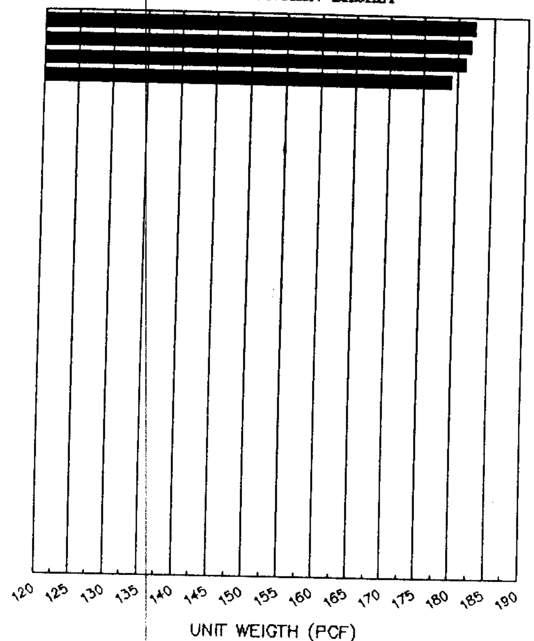
PASSAIC TUNNEL PROJECT
UNIT WEIGHT

PASSAIC FORMATION



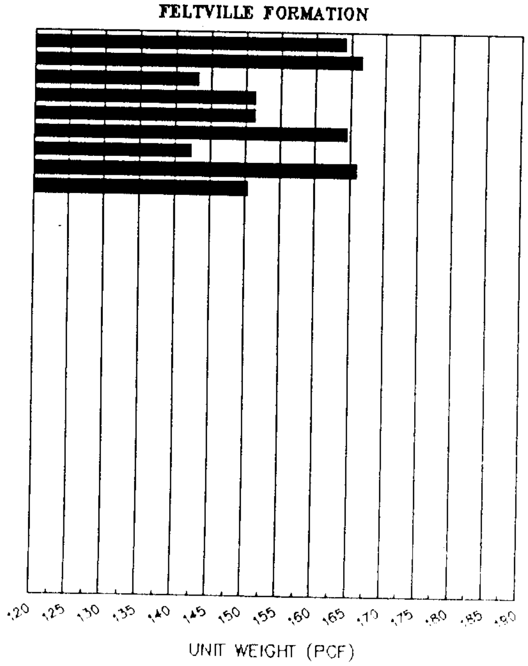
Laboratory test data from all sources.

ORANGE MOUNTAIN BASALT



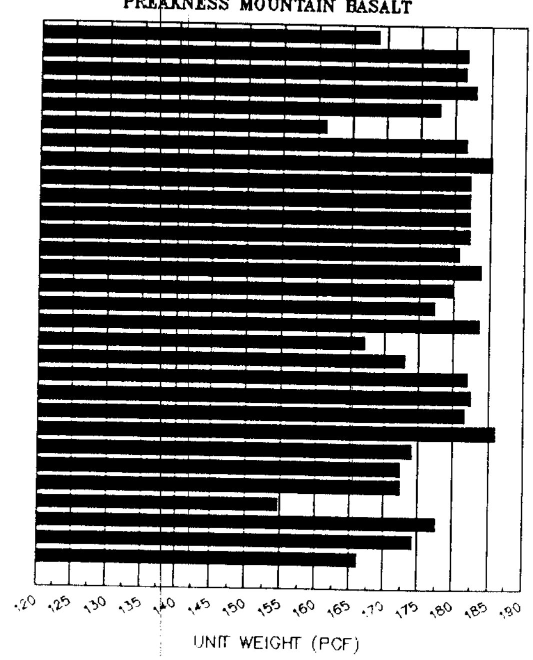
Laboratory test data from all sources.

FELTVILLE FORMATION



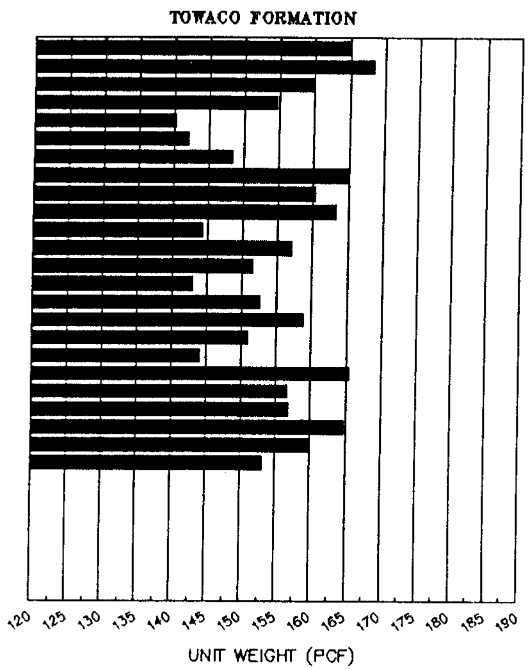
Laboratory test data from all sources.

PREAKNESS MOUNTAIN BASALT

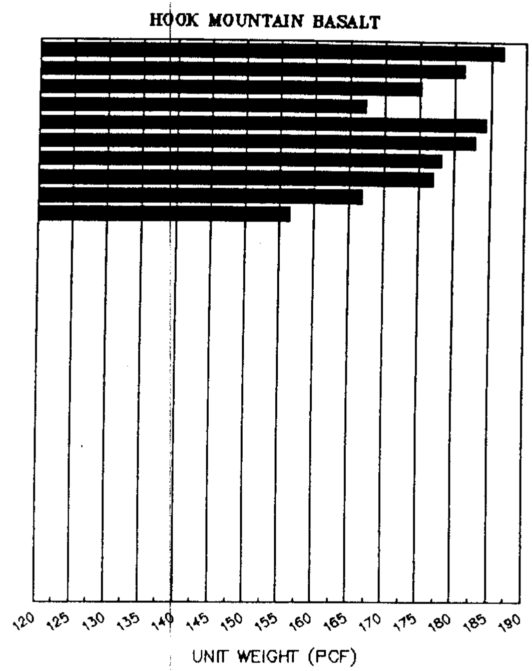


Laboratory test data from all sources.

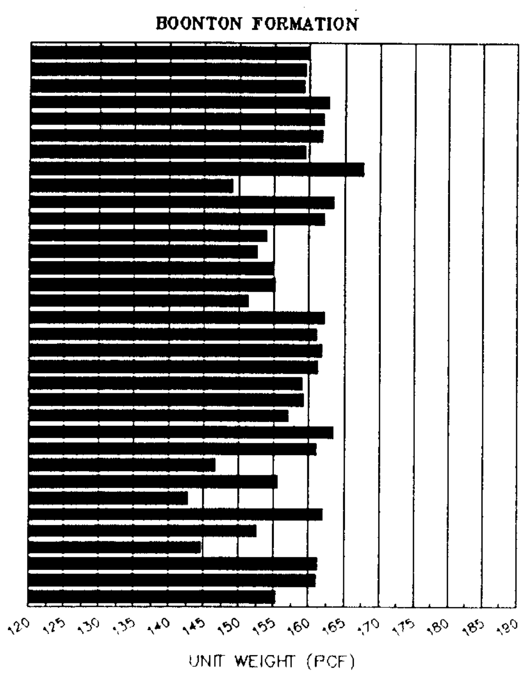
PASSAIC TUNNEL PROJECT
UNIT WEIGHT



Laboratory test data from all sources.



Laboratory test data from all sources.

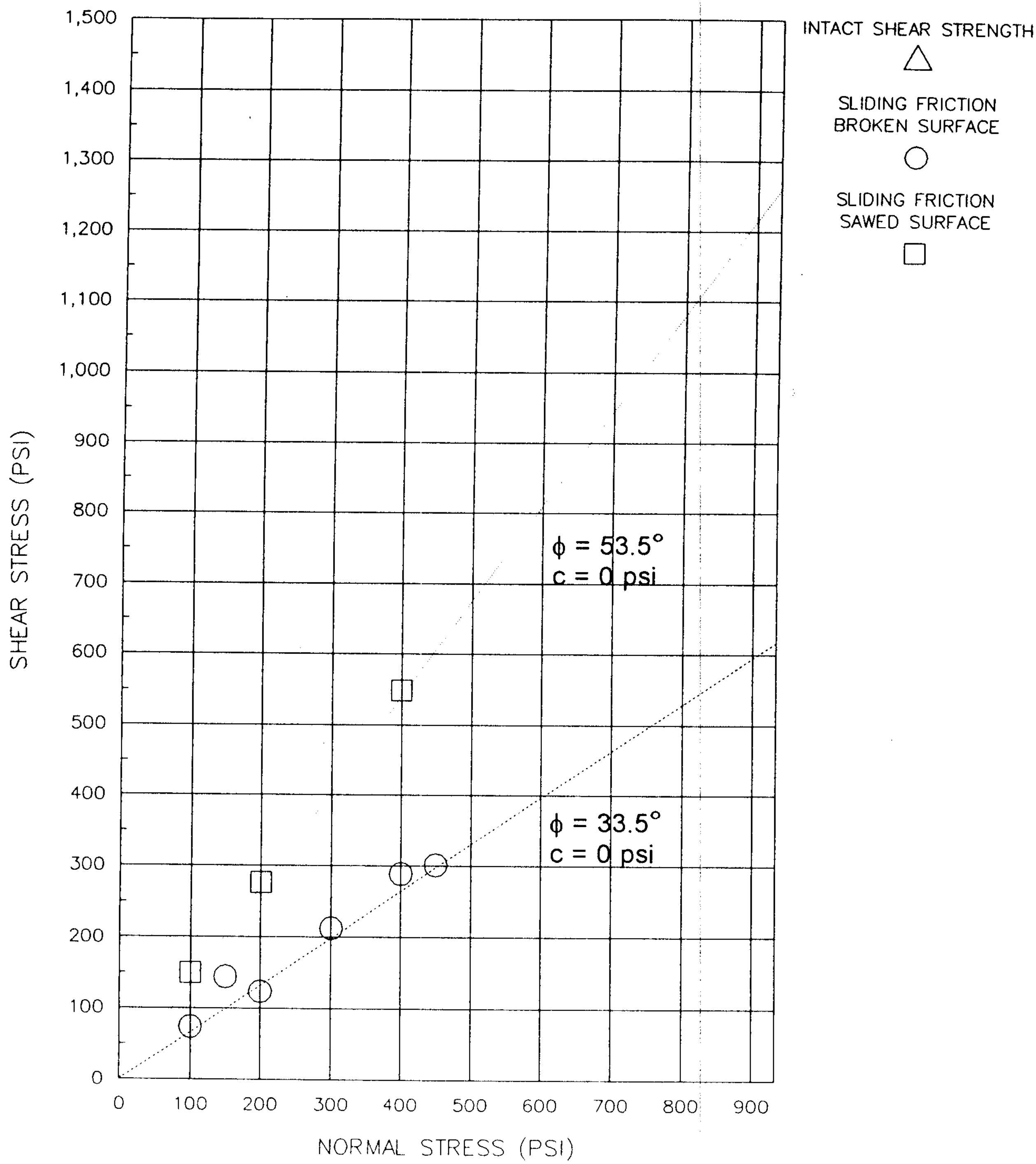


Laboratory test data from all sources.

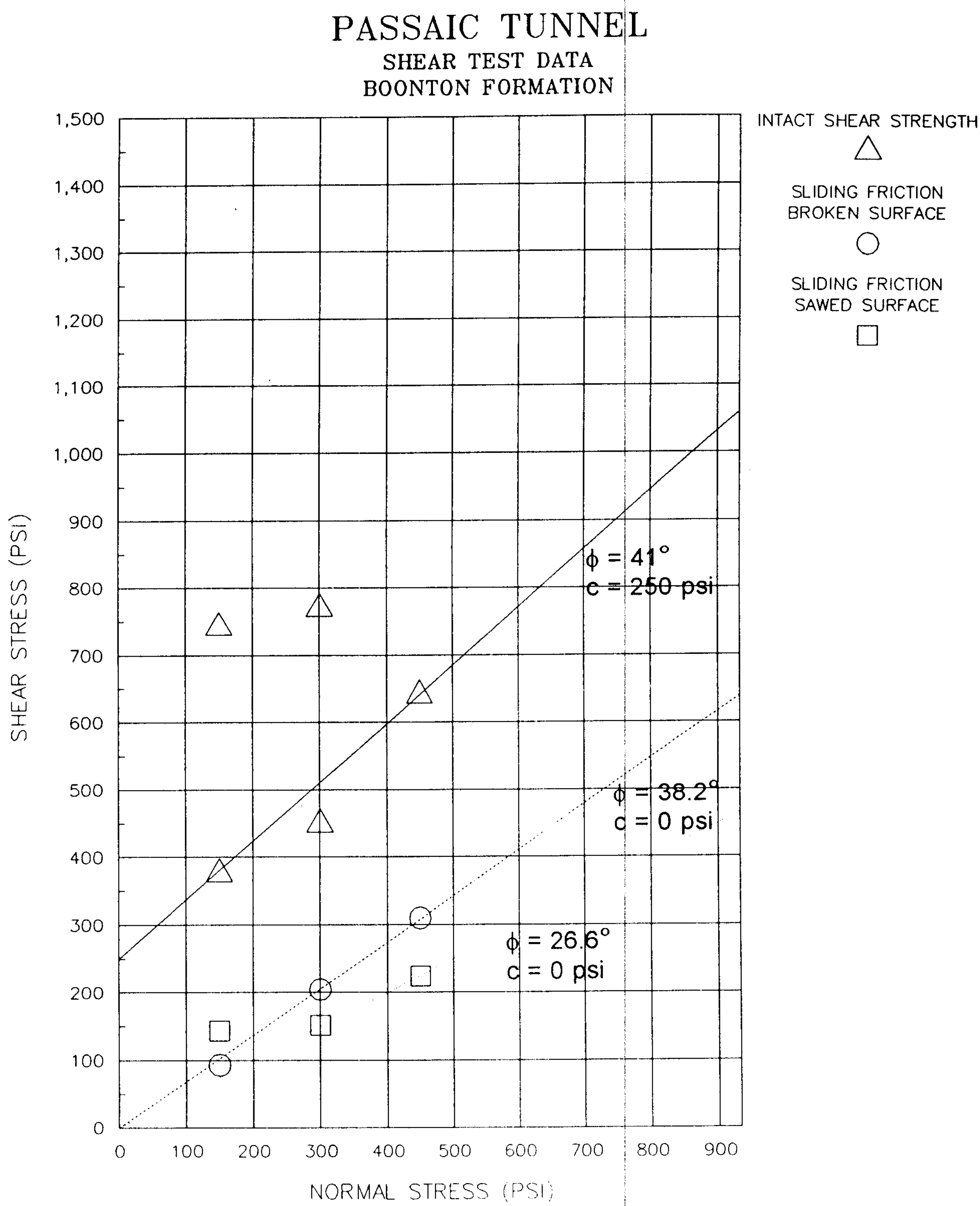
PASSAIC TUNNEL PROJECT
AVERAGE UNIT WEIGHTS FOR LITHOLOGIES

LITHOLOGY	AVERAGE UNIT WT. (LBS/CUBIC FOOT)
OVERBURDEN, FILL	120
OVERBURDEN, ALLUVIUM	120
OVERBURDEN, VARVED	120
OVERBURDEN, GRAVEL TILL	120
BOONTON SANDSTONE	155.3
BOONTON SILTSTONE	158.3
BOONTON SHALE	160.1
HOOK MOUNTAIN BASALT	179.6
HOOK MOUNTAIN VUGGY BASALT	169.6
TOWACO SANDSTONE	154.8
TOWACO SHALE	155.7
PREAKNESS MOUNTAIN BASALT	181.5
PREAKNESS MOUNTAIN VUGGY BAS	170.2
FELTVILLE SANDSTONE/SILTSTONE	152.7
FELTVILLE SHALE	165.4
ORANGE MOUNTAIN BASALT	181
ORANGE MOUNTAIN VUGGY BASALT	170
PASSAIC SANDSTONE/SILTSTONE	157.4
PASSAIC SHALE	158

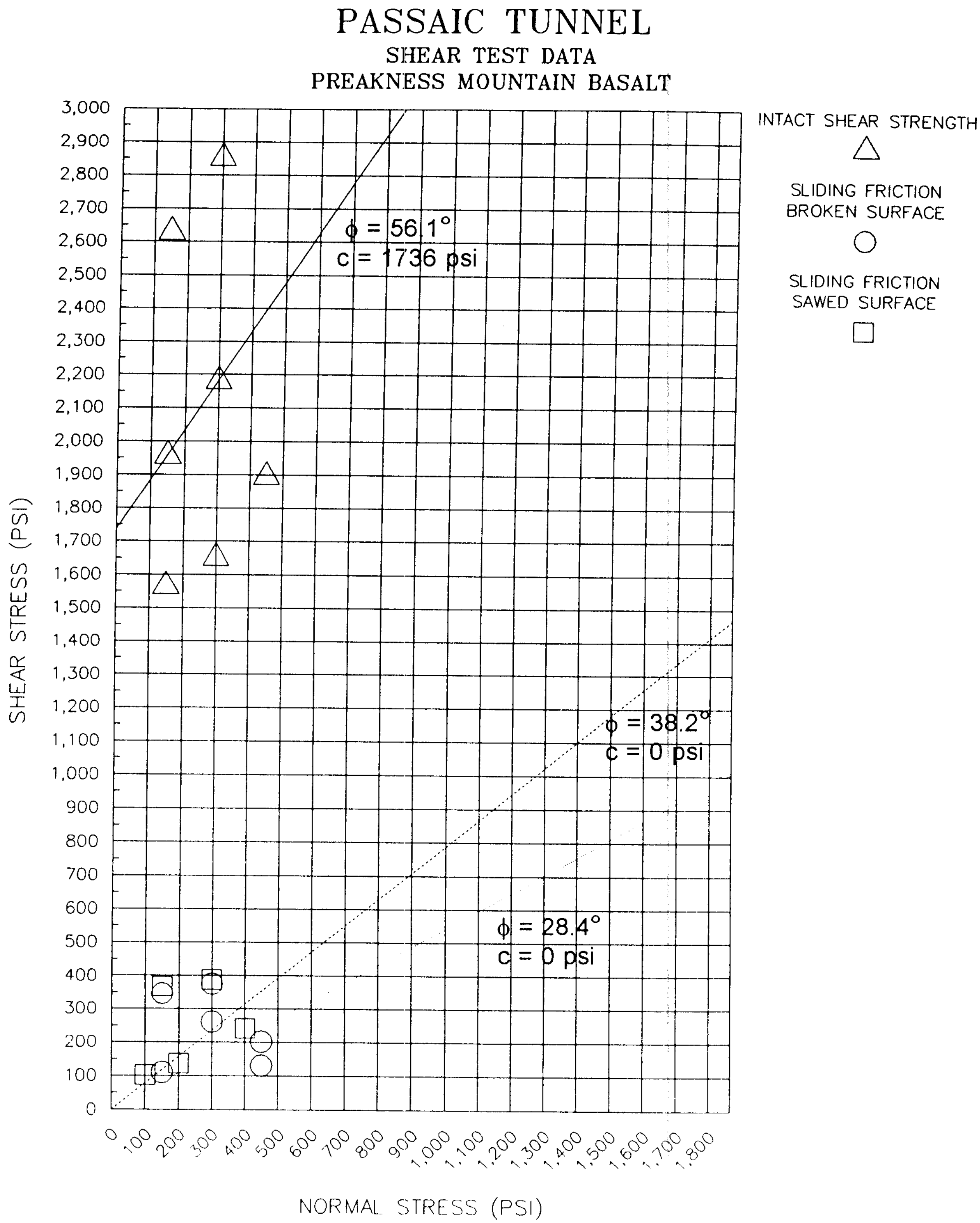
PASSAIC TUNNEL
SHEAR TEST DATA
ORANGE MOUNTAIN BASALT



Shear tests performed by SAD lab using a direct shear device



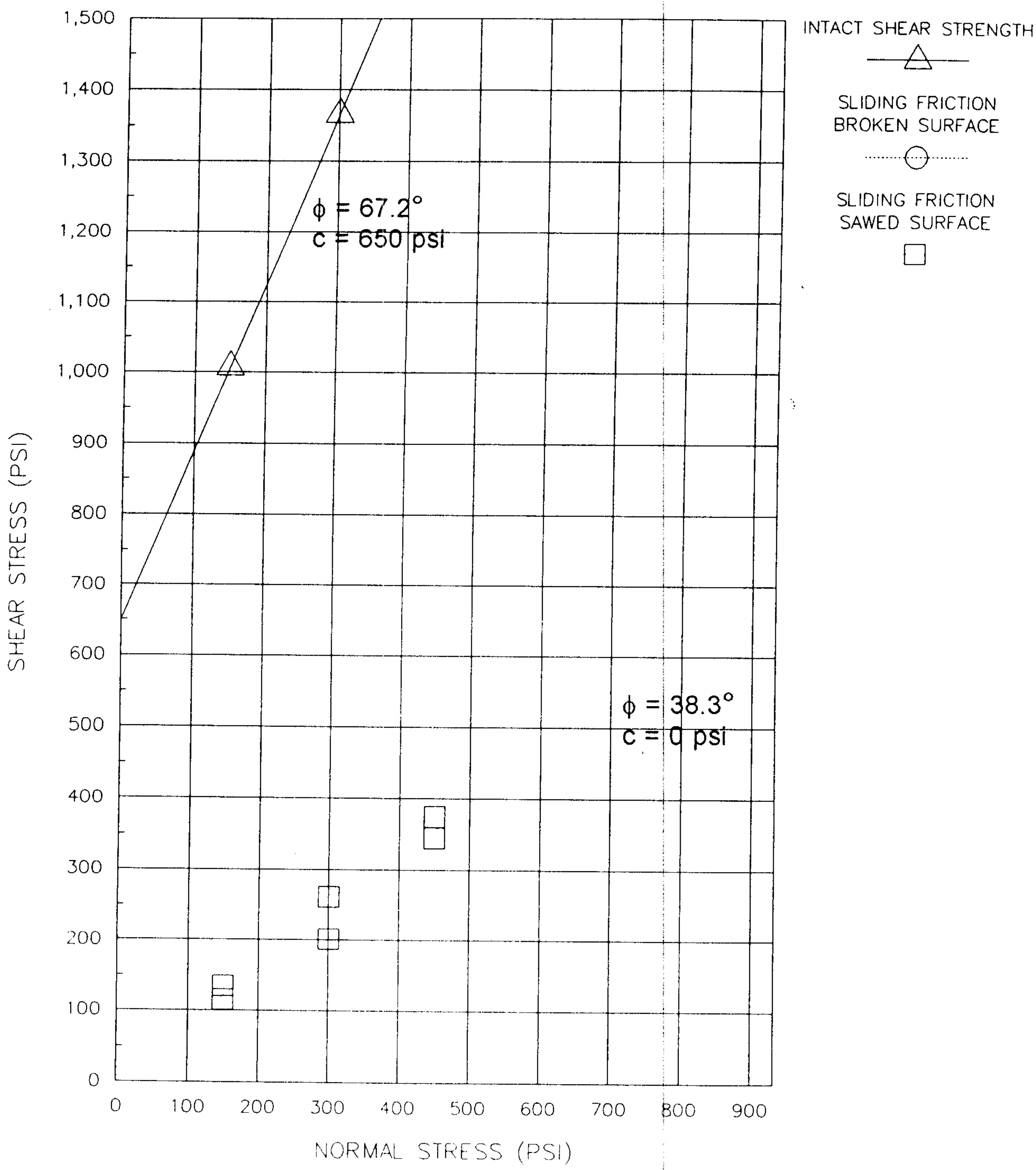
Shear tests performed by SAD lab using a direct shear device.



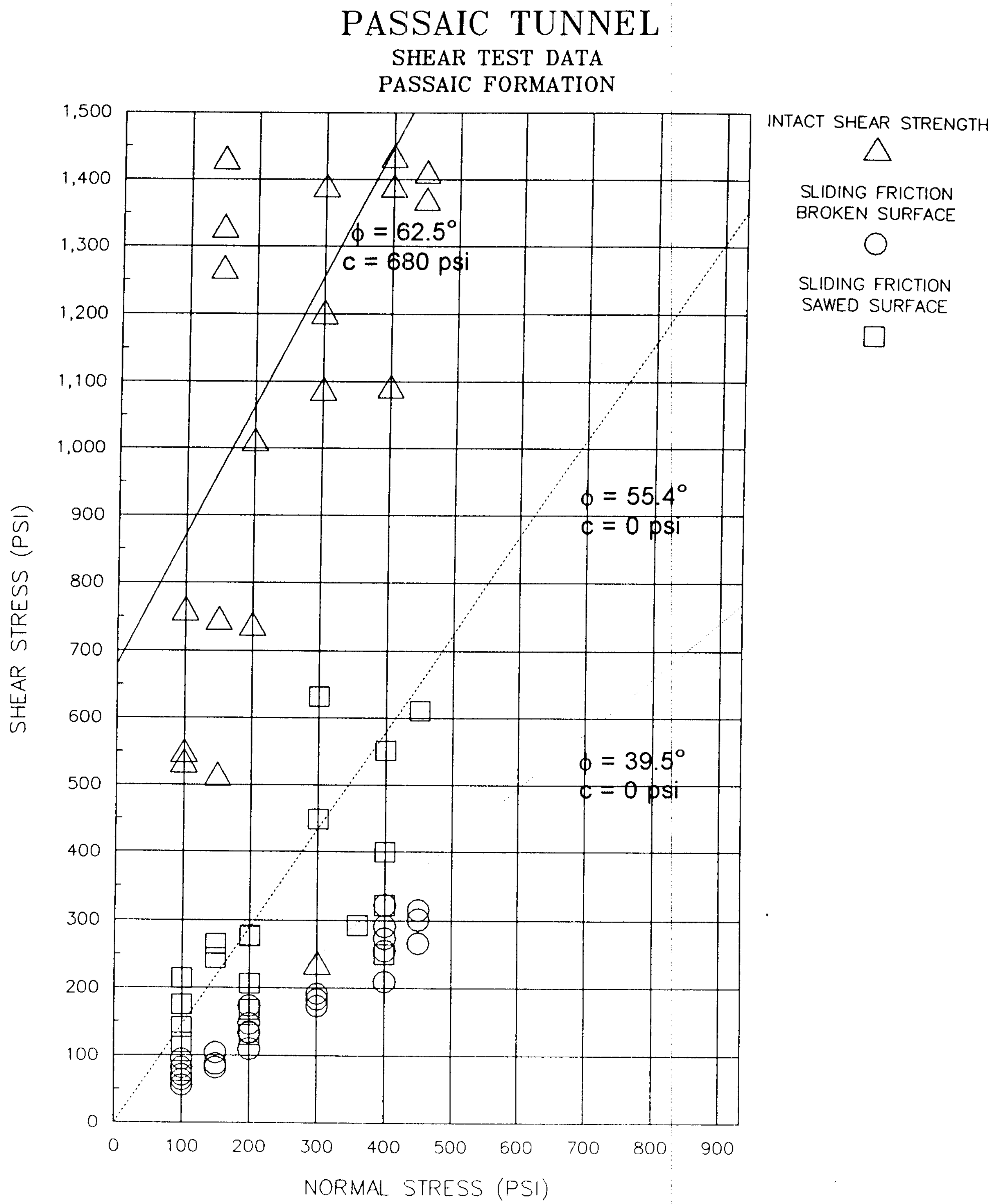
Shear tests performed by SAD lab using a direct shear device.

PASSAIC TUNNEL

SHEAR TEST DATA
TOWACO FORMATION



Shear tests performed by SAS lab using a direct shear device



Shear tests performed by SAD lab using a direct shear device.

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

ATTACHMENT E.3.7

POINT LOAD TEST DATA

Attachment E.3.7

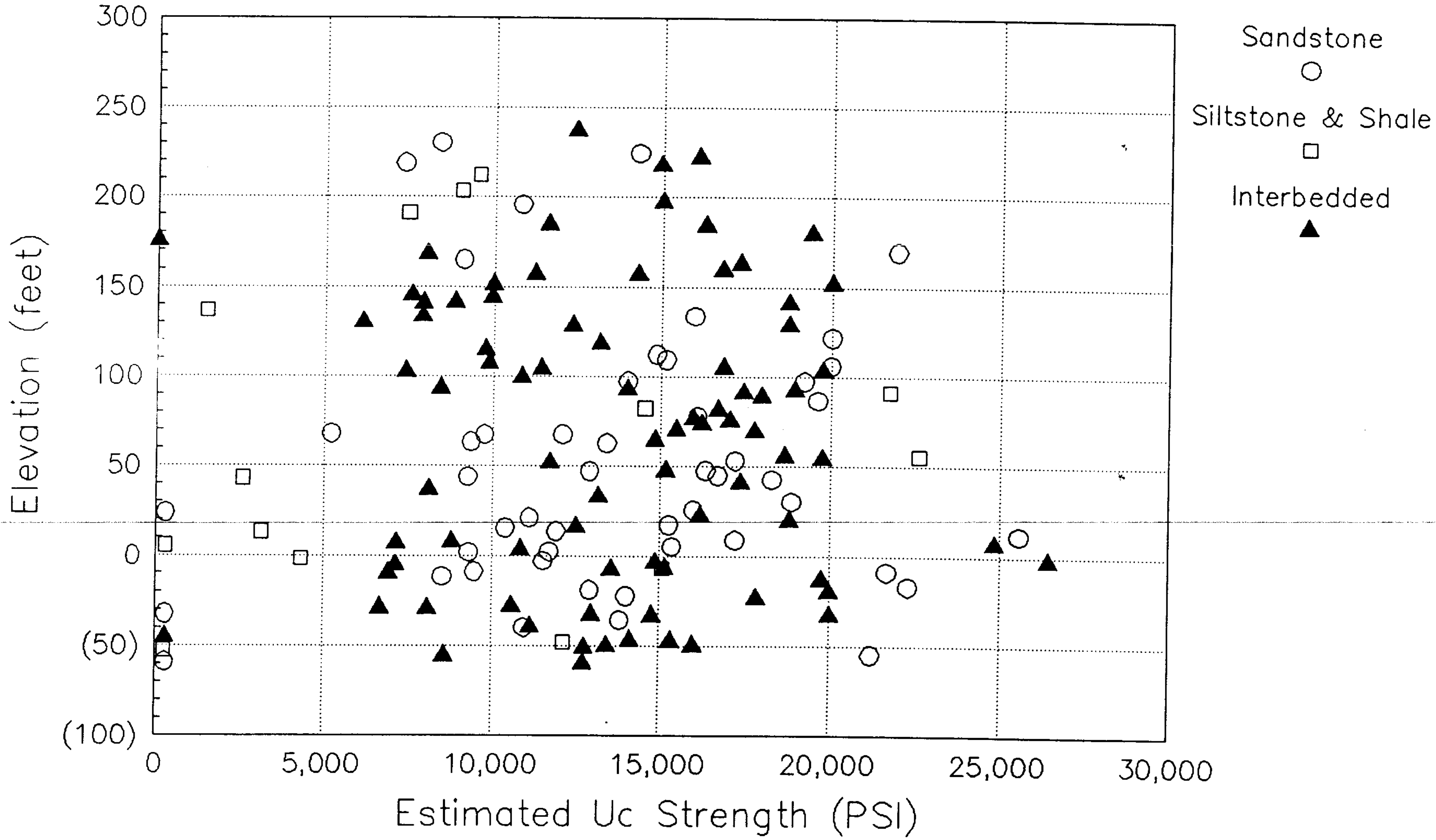
Point Load Test Data

Title

Page

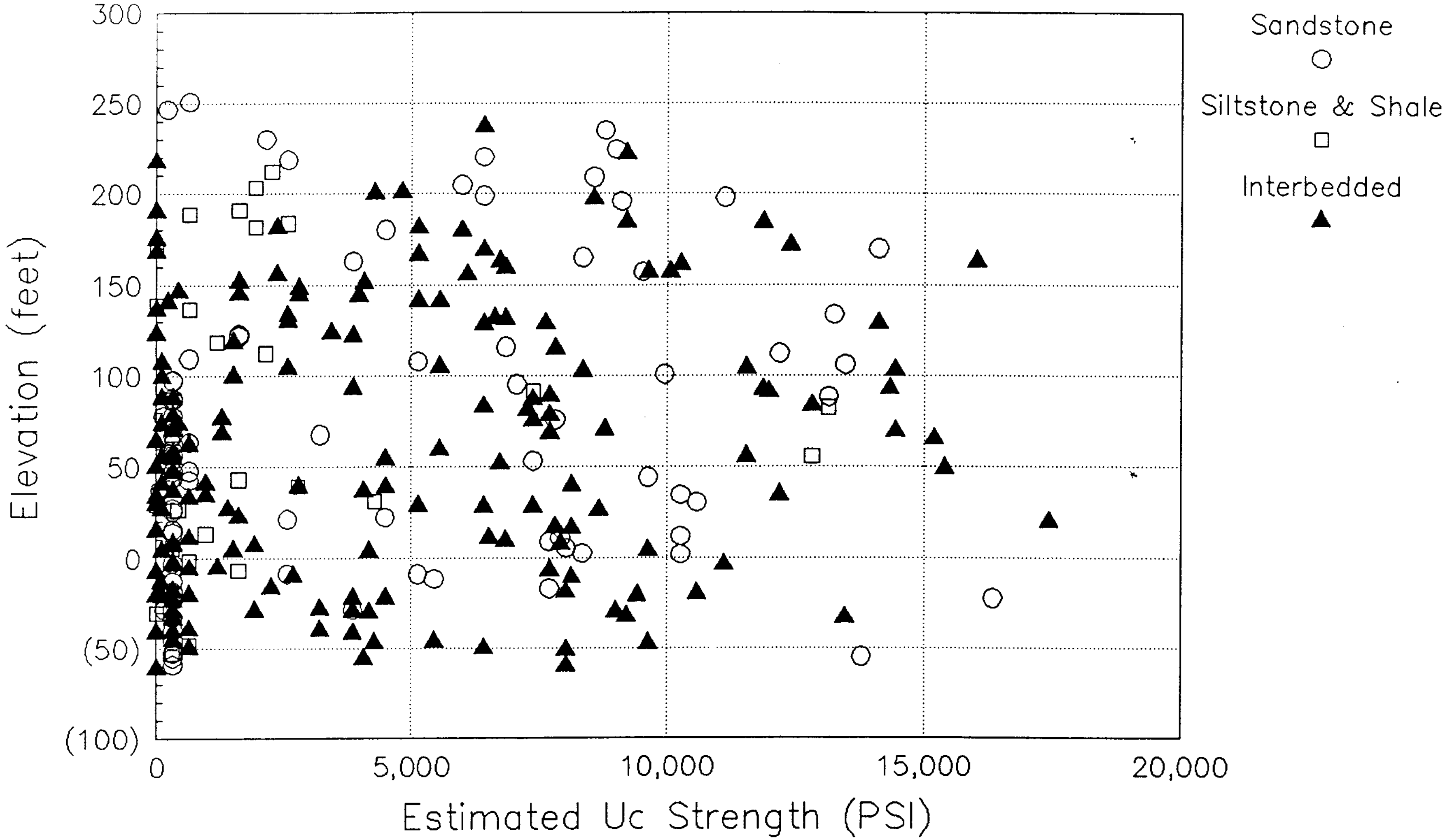
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH AXIAL TESTS ON THE BOONTON FORMATION



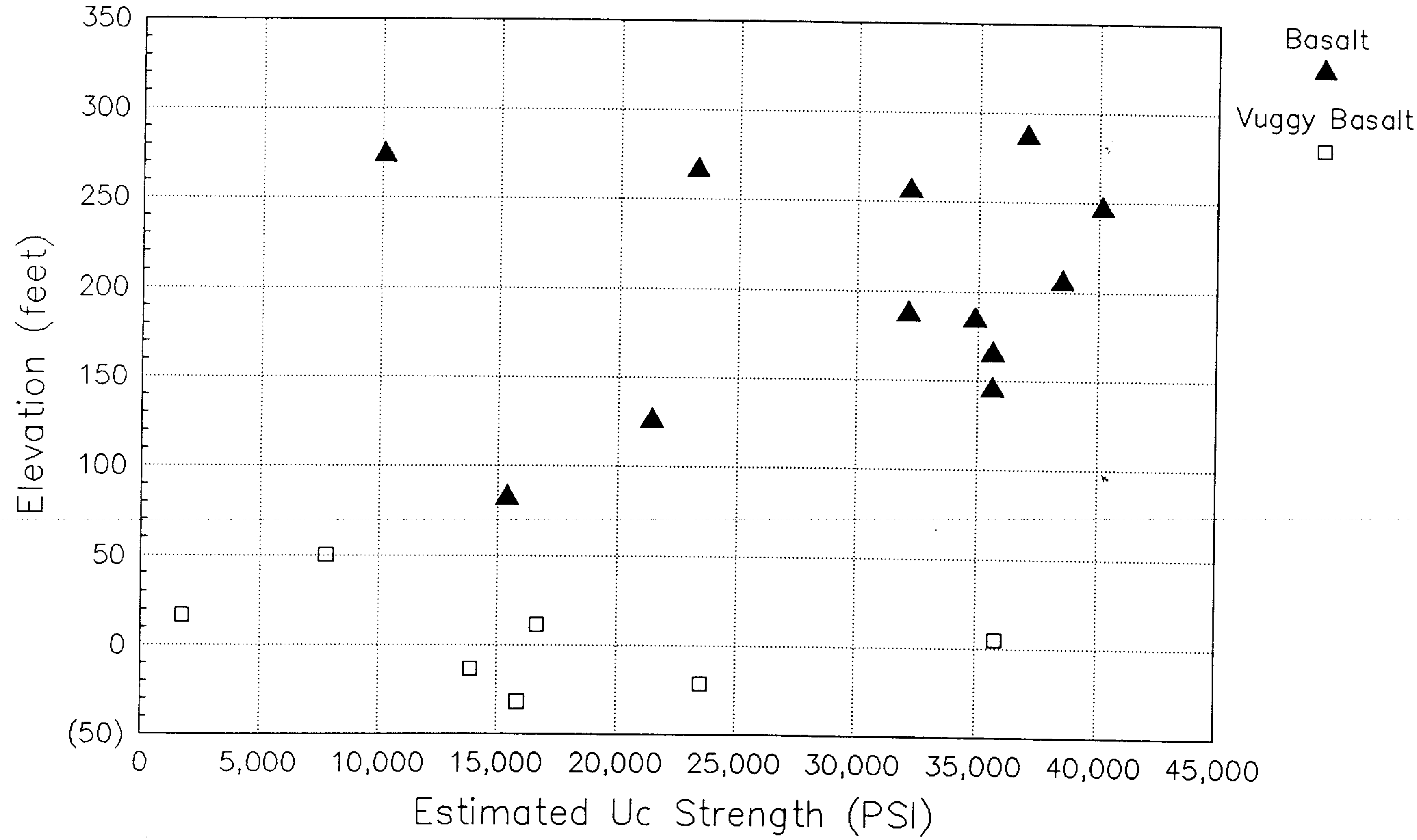
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
DIAMETRAL TESTS ON THE BOONTON FORMATION



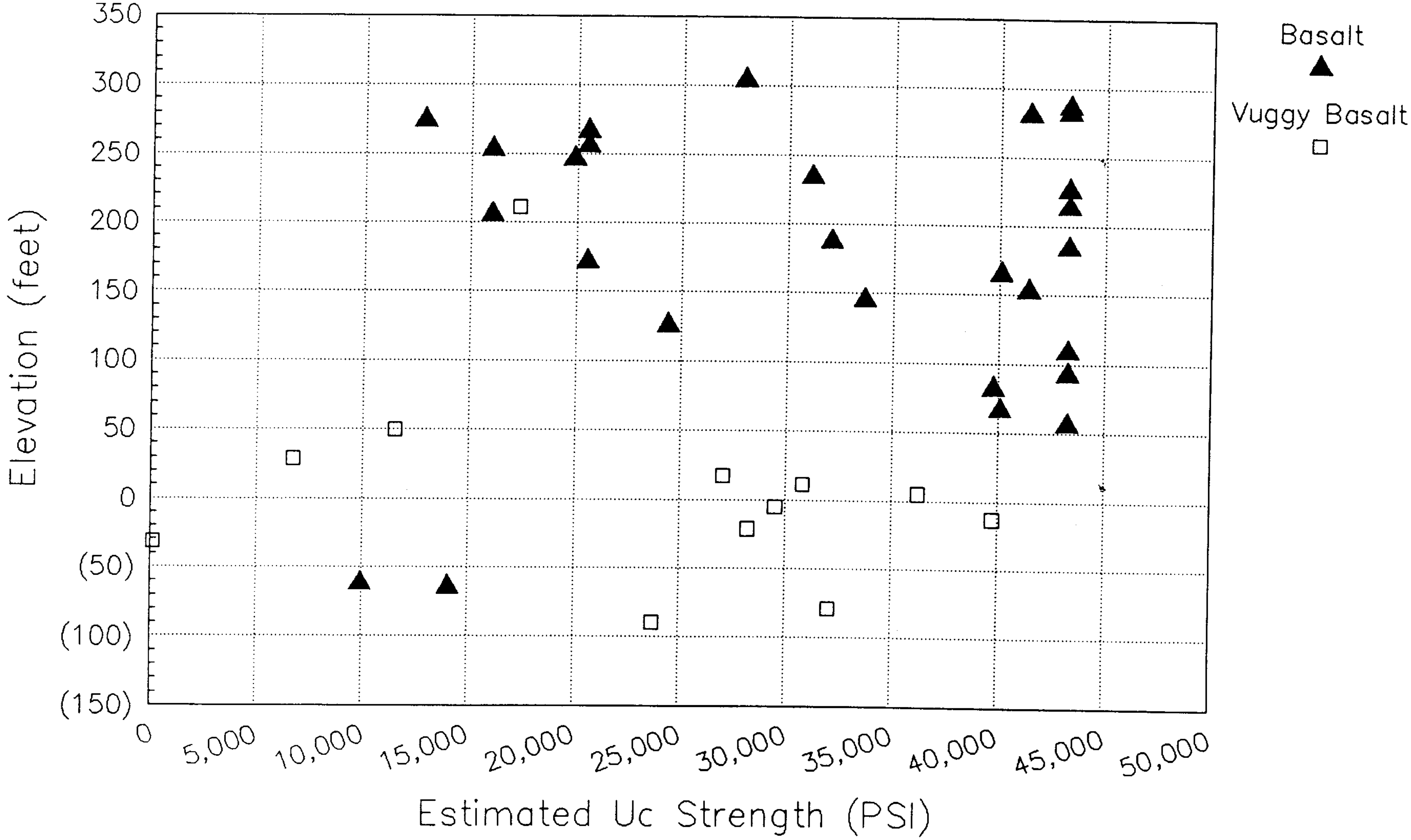
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH AXIAL TESTS ON THE HOOK MOUNTAIN BASALT



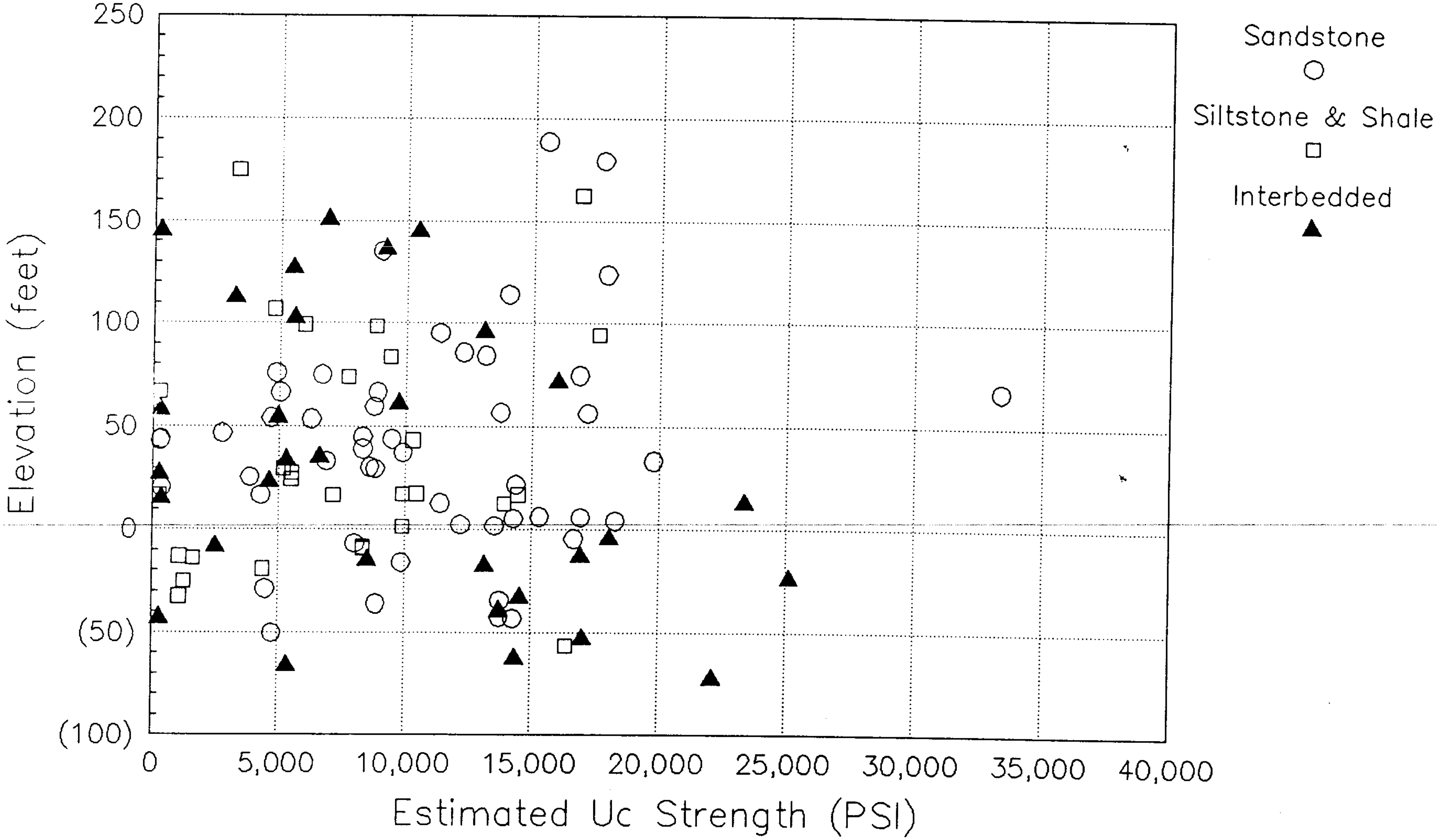
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH DIAMETRAL TESTS ON THE HOOK MOUNTAIN BASALT



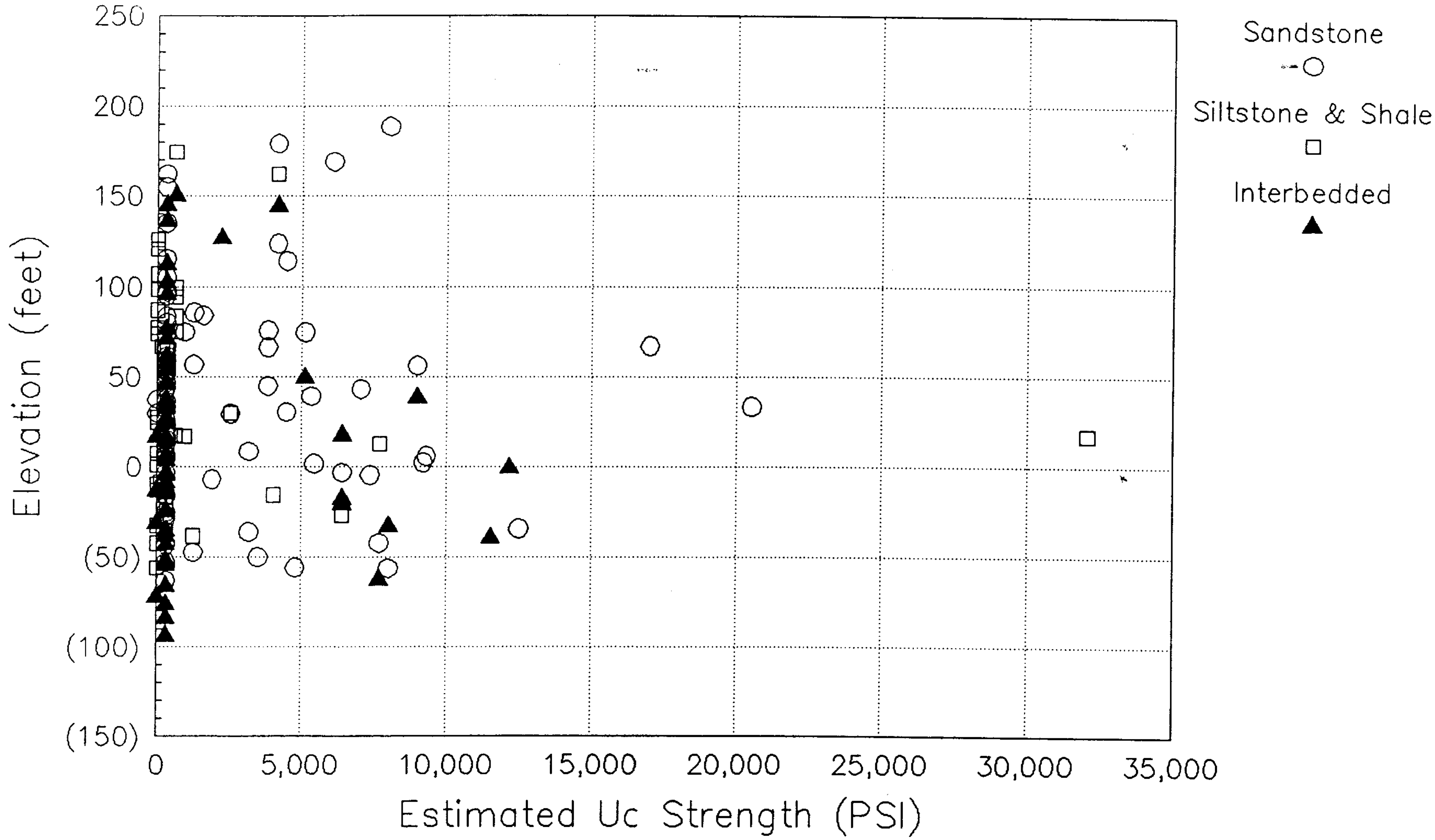
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH AXIAL TESTS ON THE TOWACO FORMATION



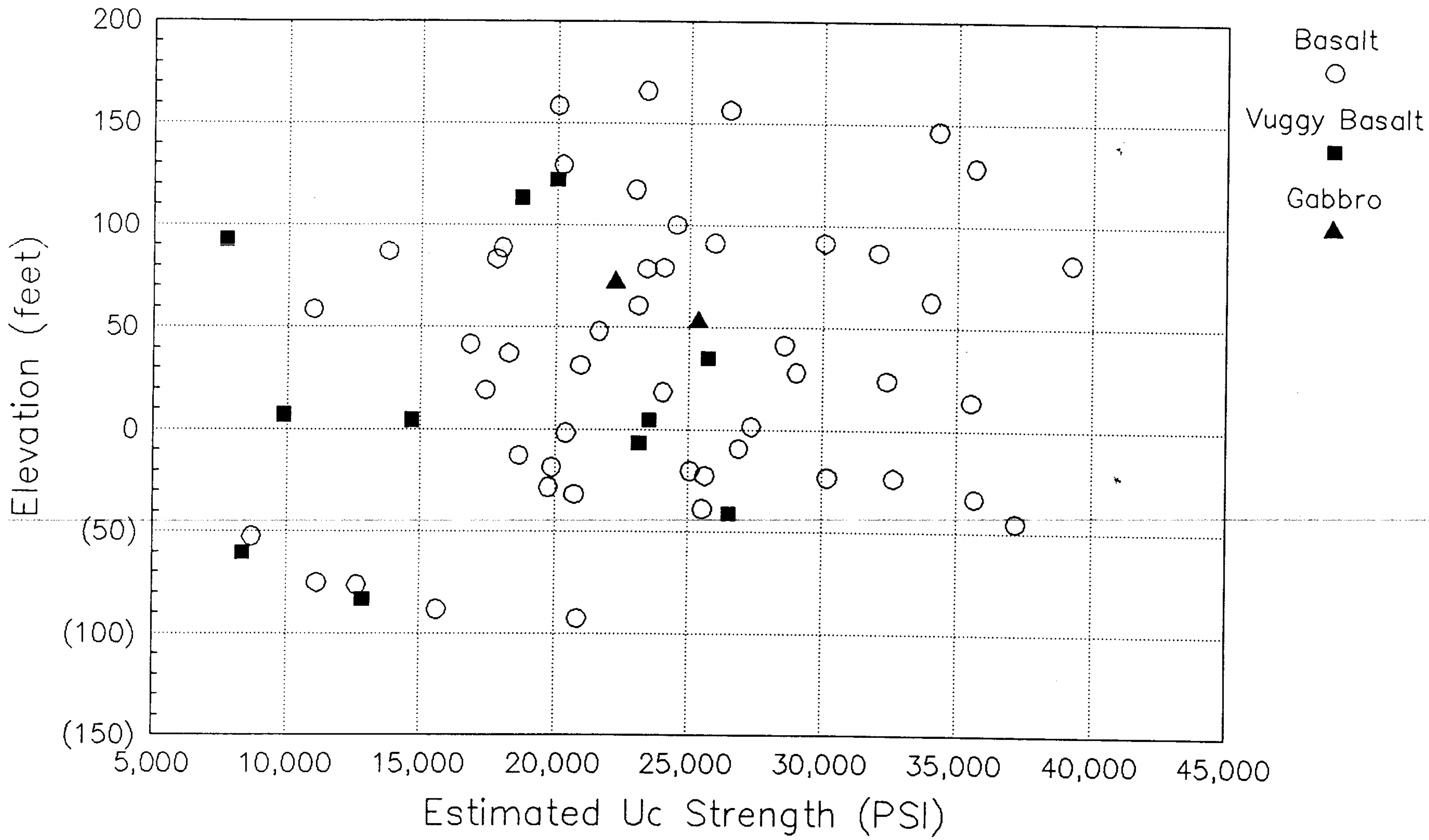
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
DIAMETRAL TESTS ON THE TOWACO FORMATION



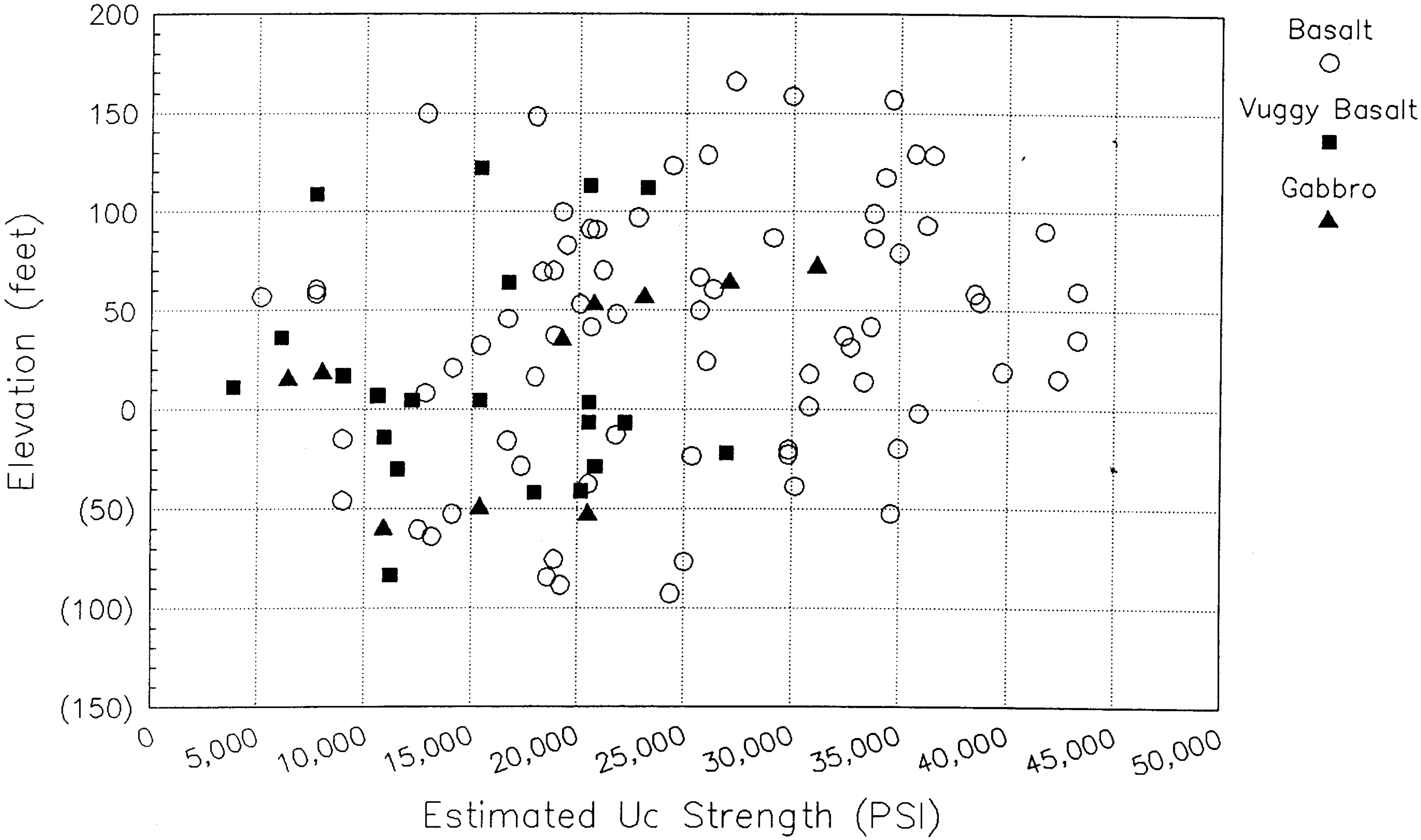
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH AXIAL TESTS ON THE PREAKNESS MT. BASALT



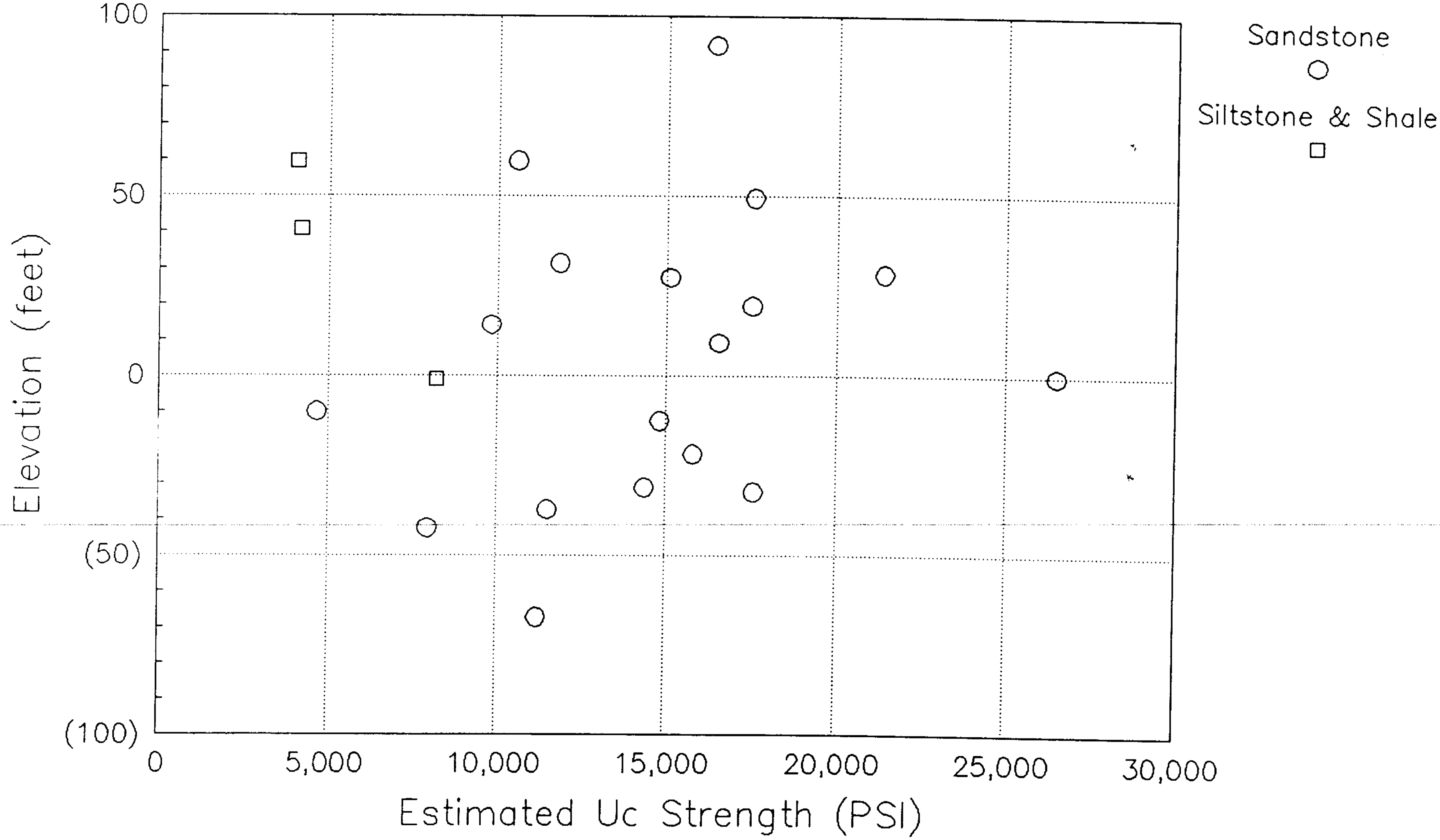
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
DIAMETRAL TESTS ON THE PREAKNESS MT. BASALT



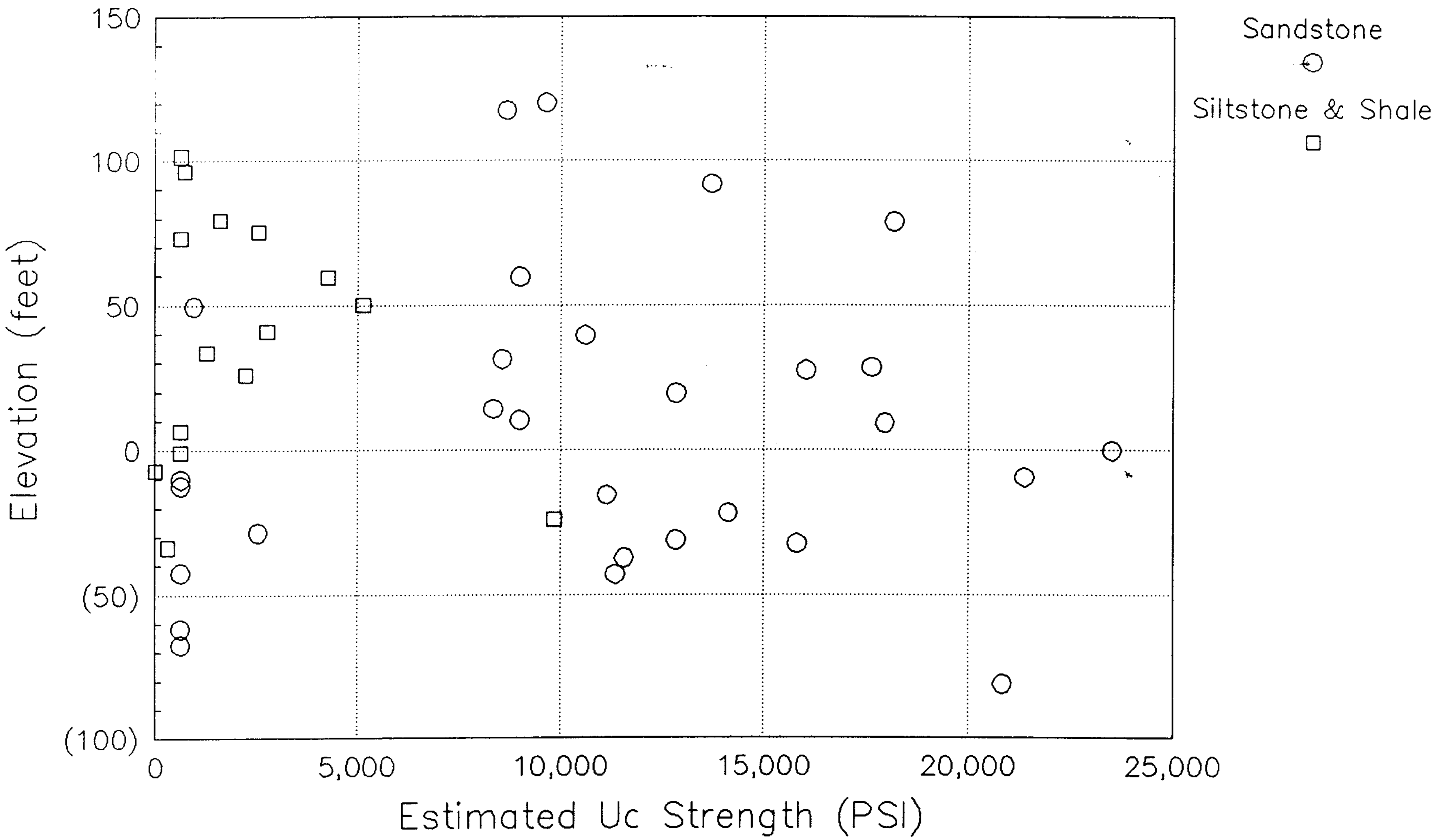
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
AXIAL TESTS ON THE FELTVILLE FORMATION



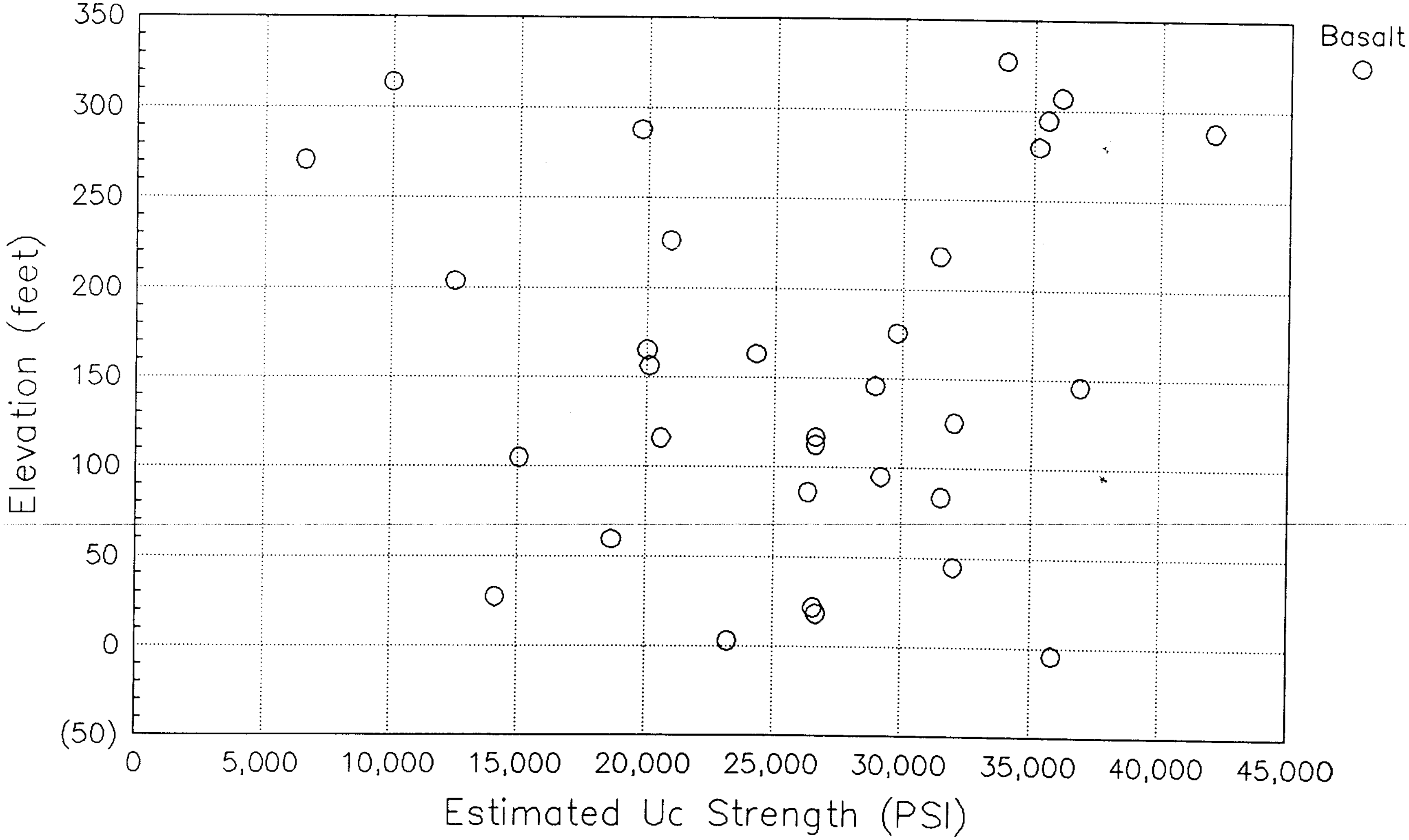
PASSAIC TUNNEL PROJECT

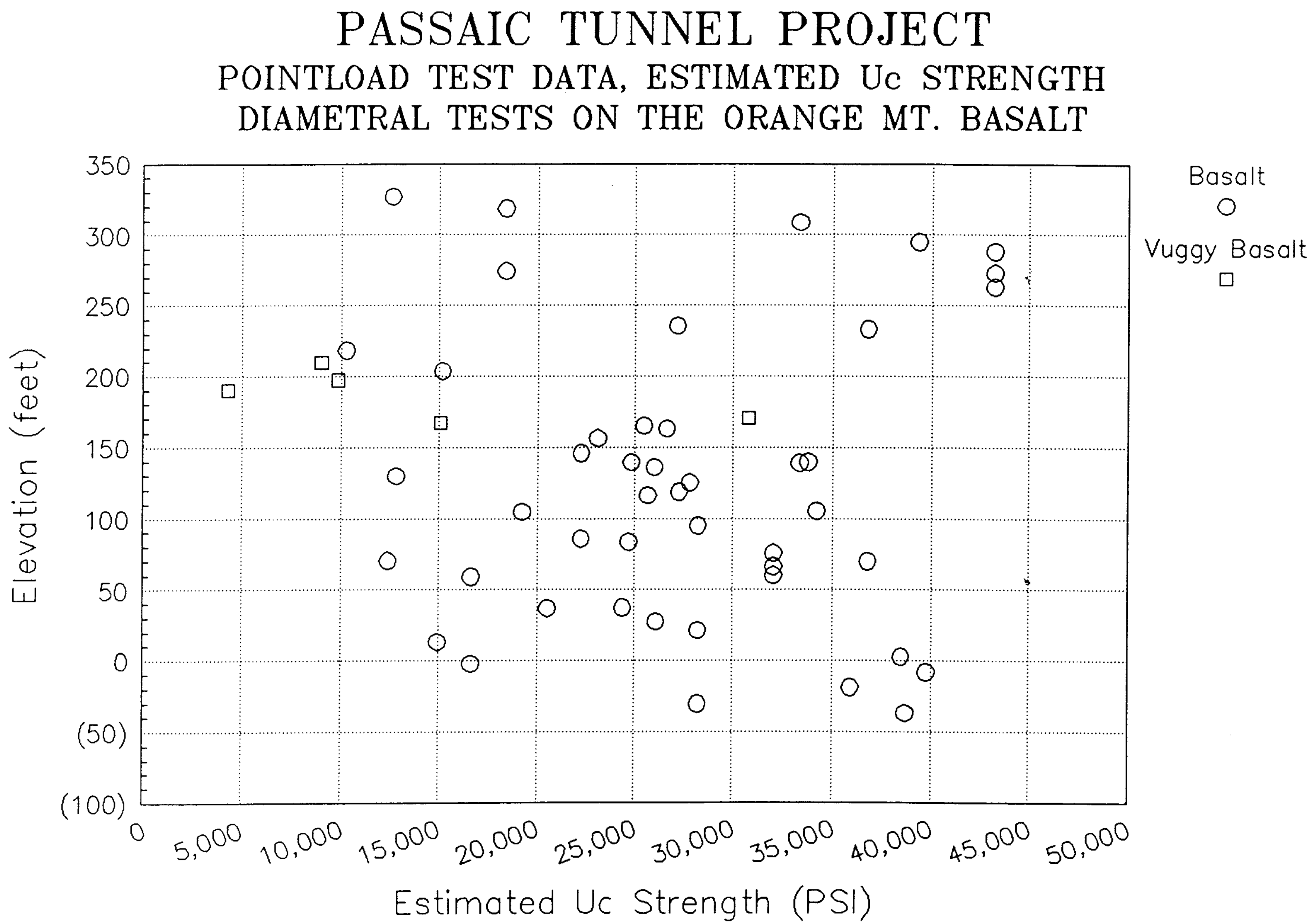
POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH DIAMETRAL TESTS ON THE FELTVILLE FORMATION



PASSAIC TUNNEL PROJECT

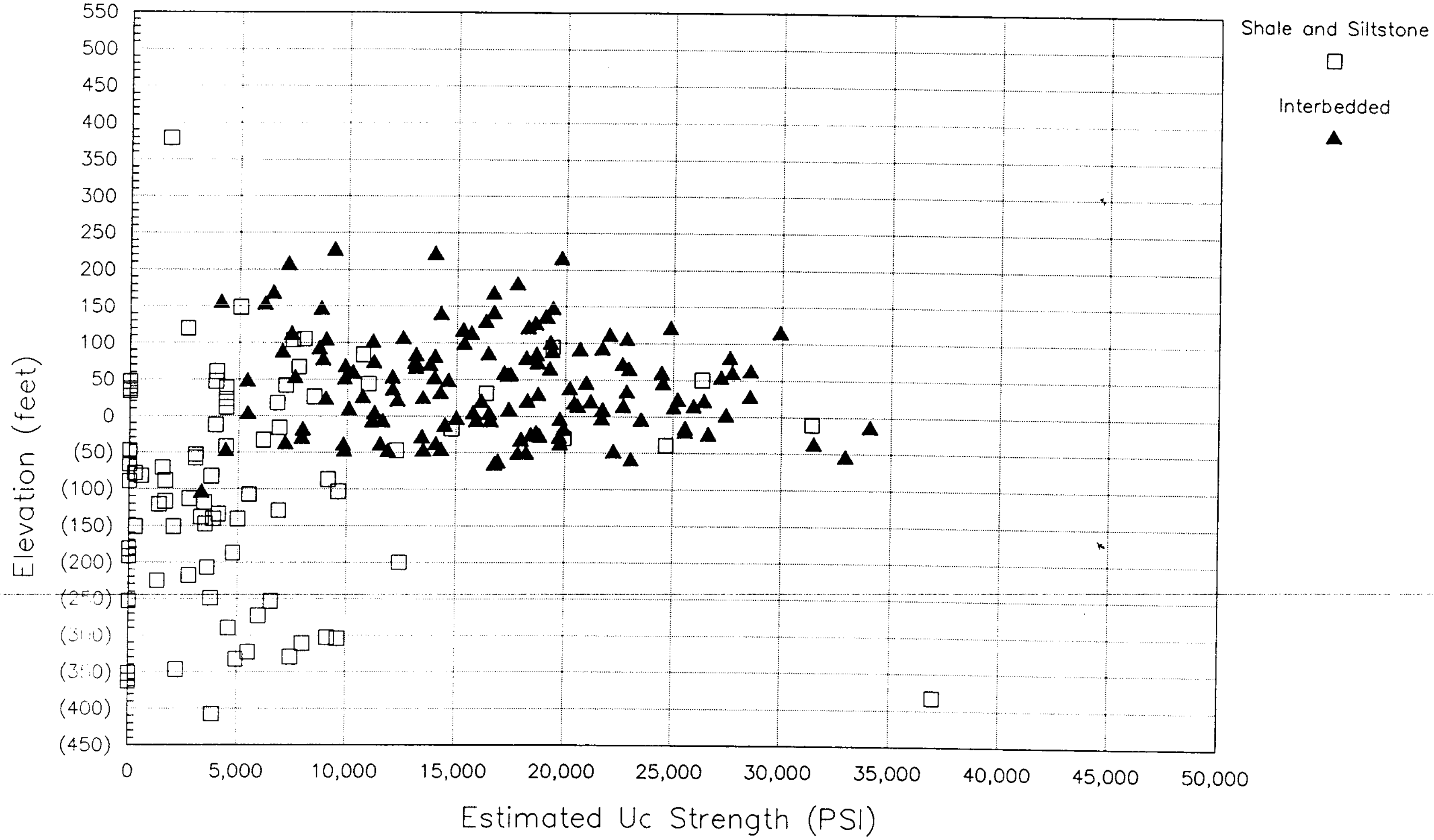
POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
AXIAL TESTS ON THE ORANGE MT. BASALT




PASSAIC TUNNEL PROJECT
POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
DIAMETRAL TESTS ON THE ORANGE MT. BASALT
Elevation (feet)
350
300
250
200
150
100
50
0
(50)
(100)
0
5,000
10,000
15,000
20,000
25,000
30,000
35,000
40,000
45,000
50,000
Estimated Uc Strength (PSI)
Basalt
Vuggy Basalt

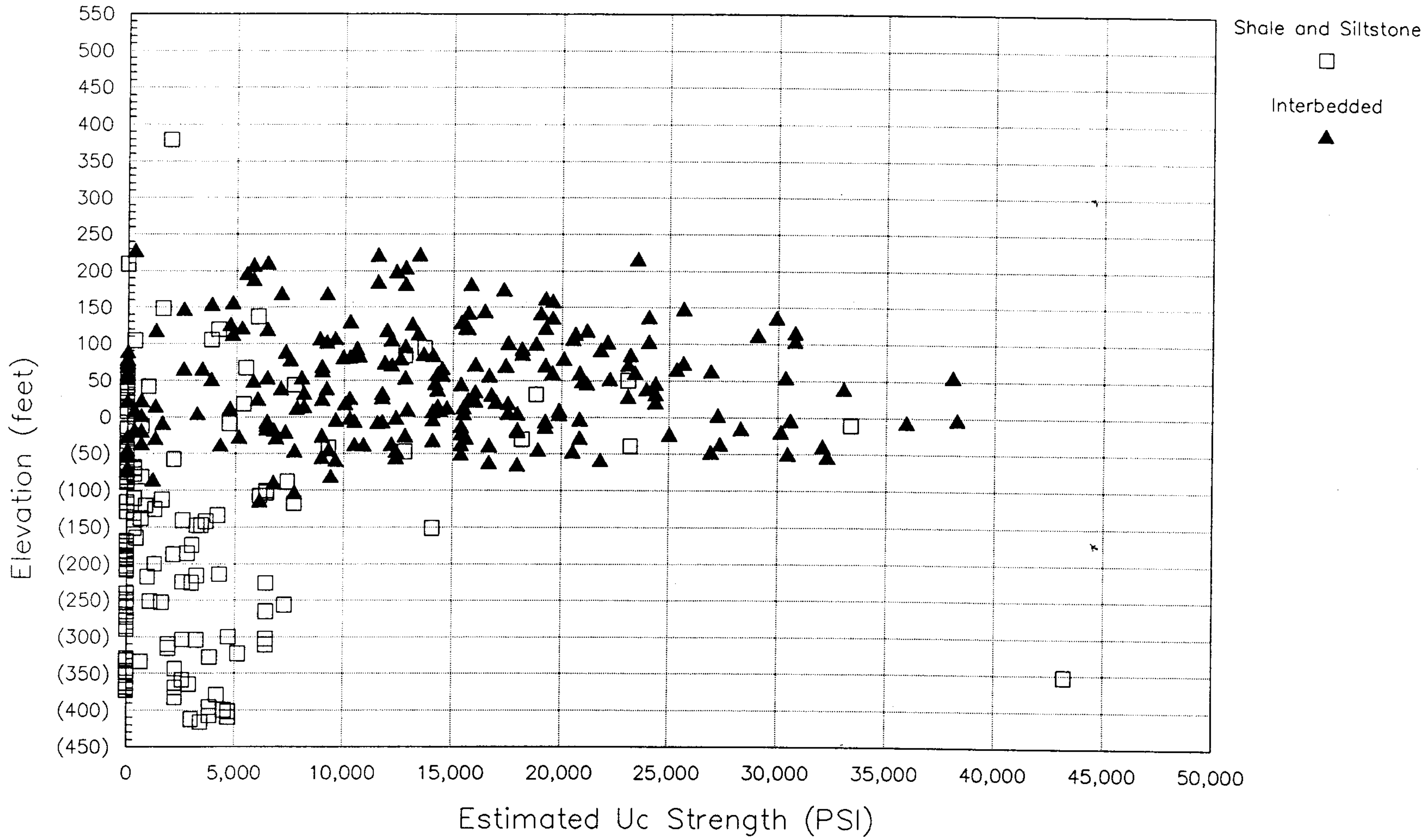
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
AXIAL TESTS ON PASSAIC FORMATION



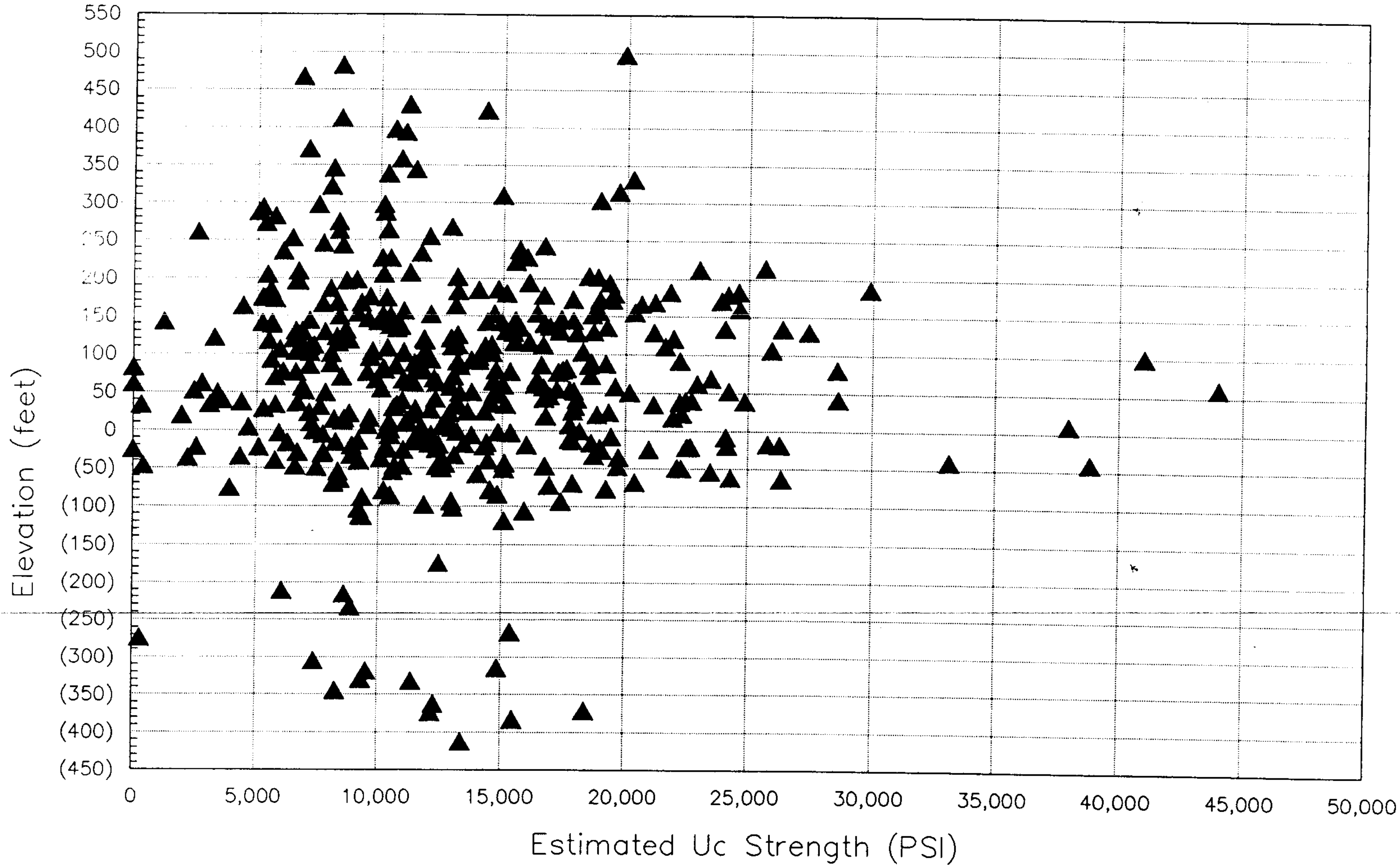
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
DIAMITRAL TESTS ON PASSAIC FORMATION



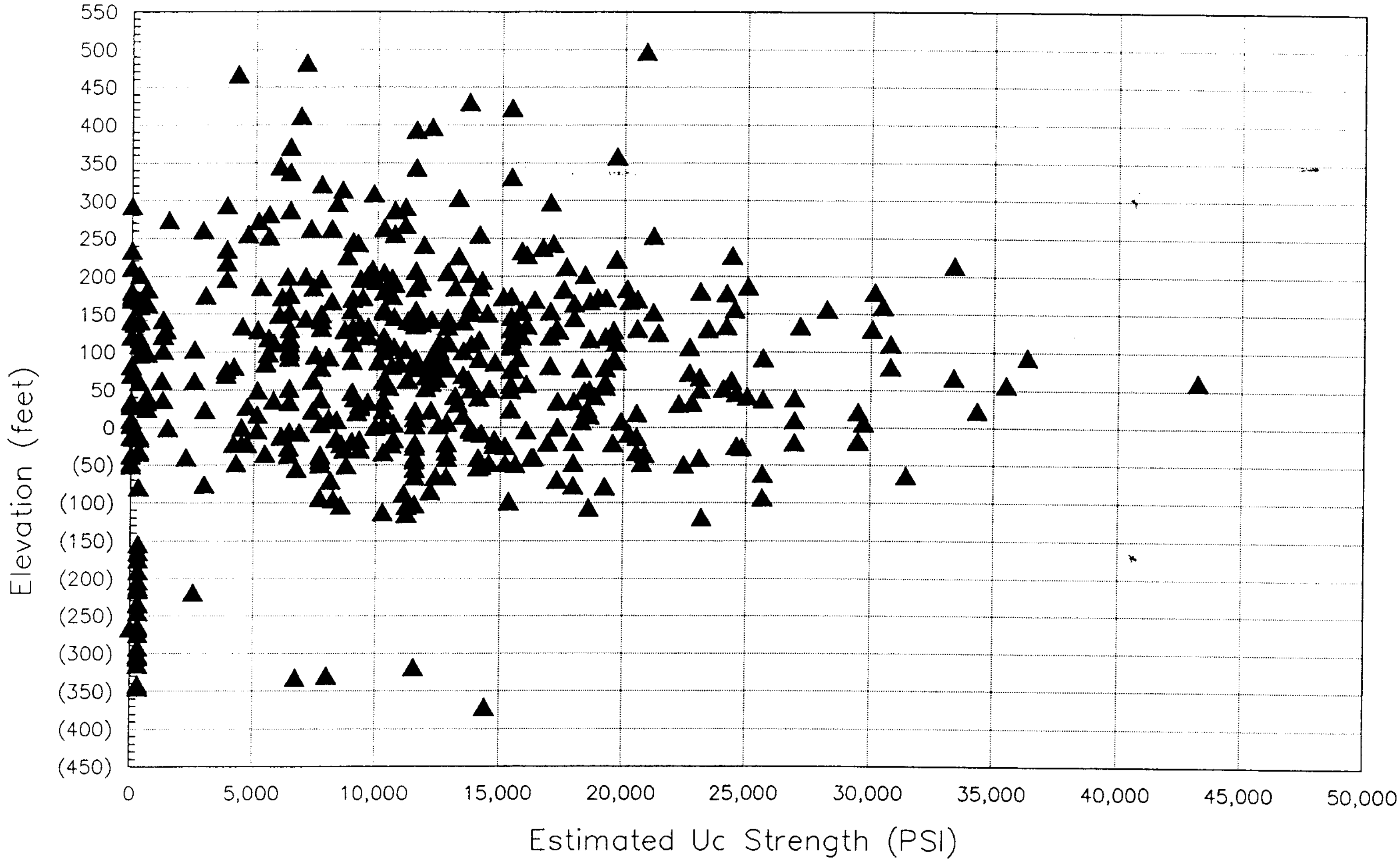
PASSAIC TUNNEL PROJECT

POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
AXIAL TESTS ON PASSAIC FORMATION, SANDSTONE

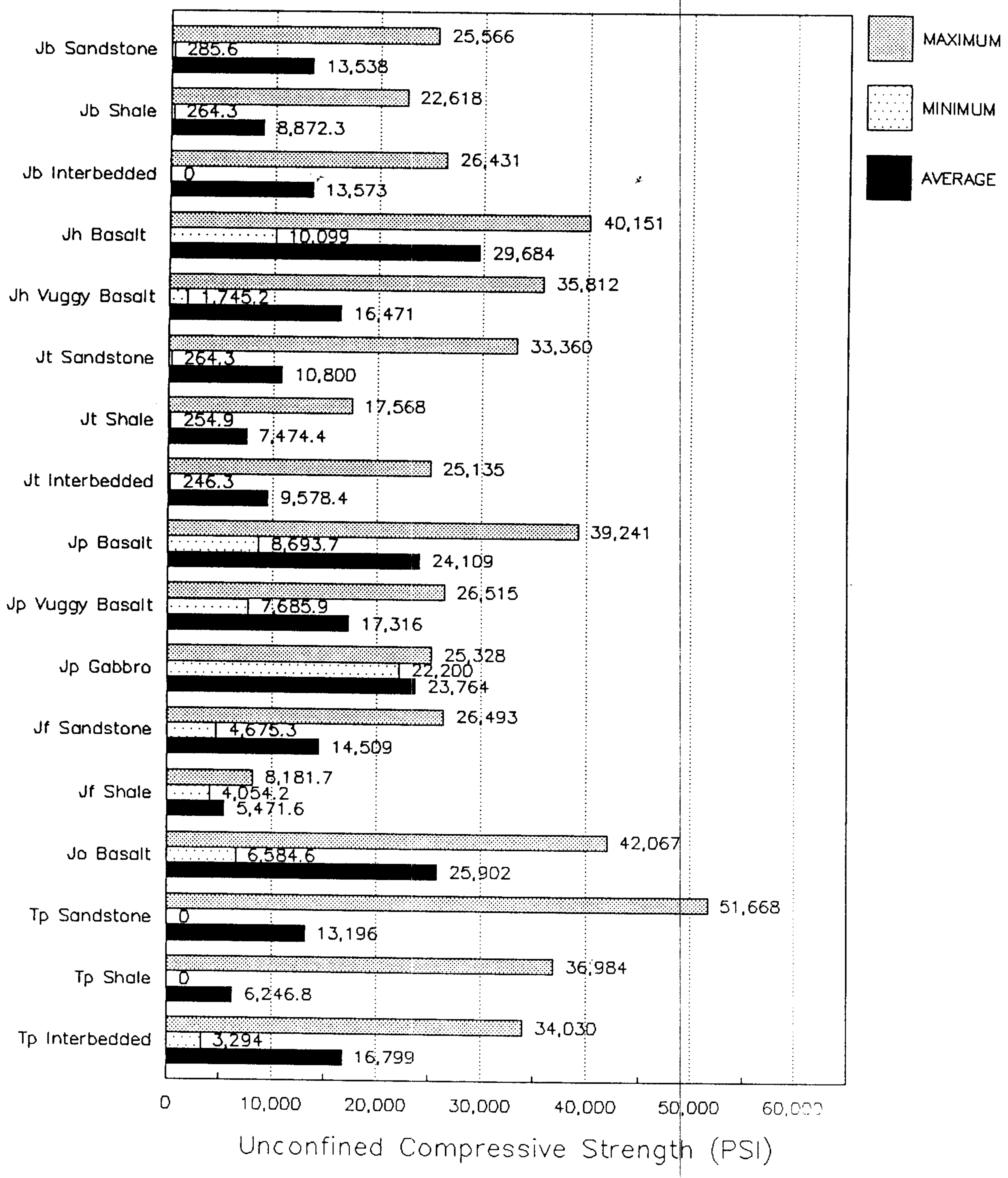


PASSAIC TUNNEL PROJECT

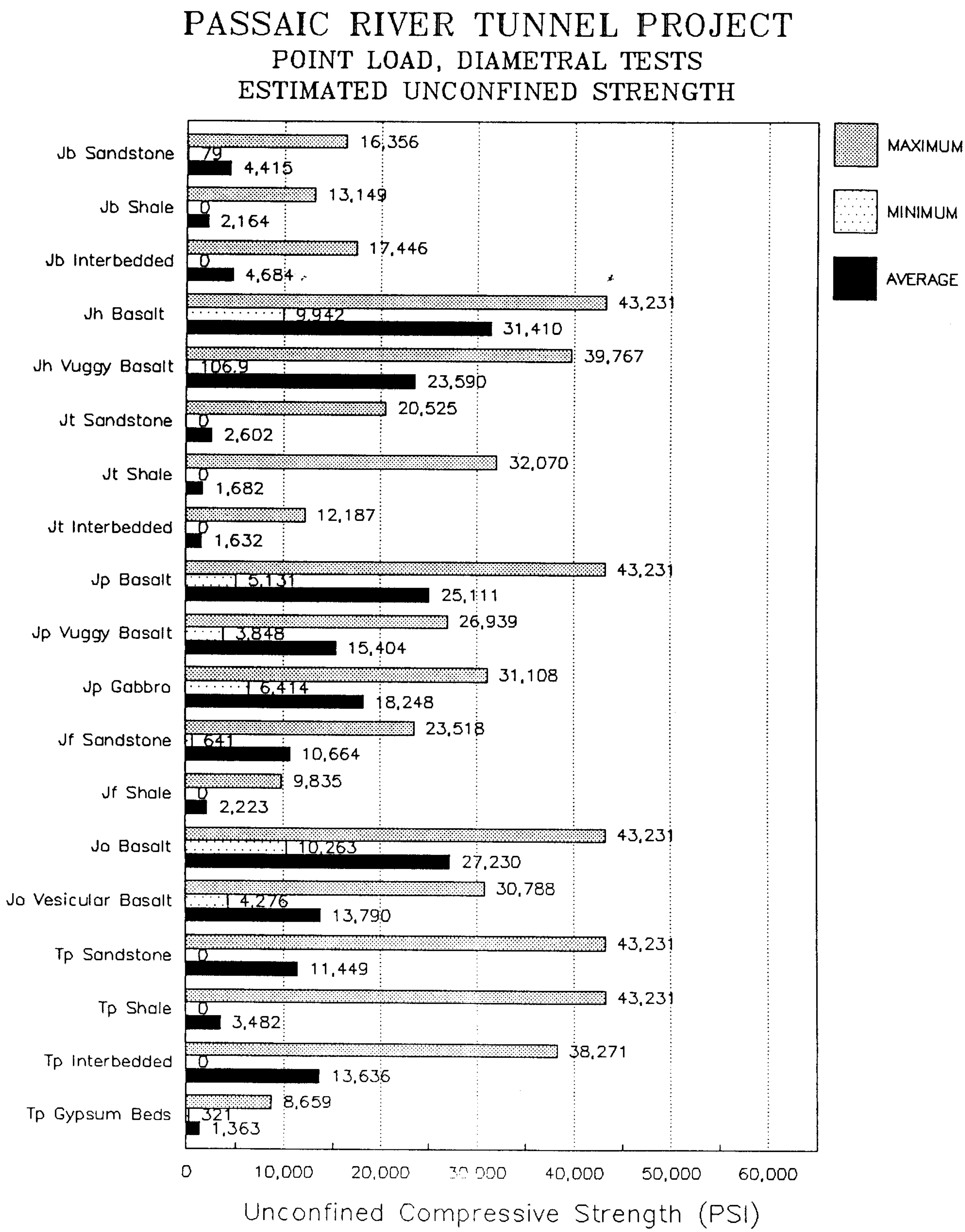
POINTLOAD TEST DATA, ESTIMATED Uc STRENGTH
DIAMITRAL TESTS ON PASSAIC FORMATION, SANDSTONE



PASSAIC RIVER TUNNEL PROJECT
POINT LOAD, AXIAL TESTS
ESTIMATED UNCONFINED STRENGTH



Unconfined strength was estimated by multiplying the point load index by 24. Only tests which produced clean breaks are included.



Unconfined strength was estimated by multiplying the point load index by 24. Only tests which produced clean breaks are included.

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

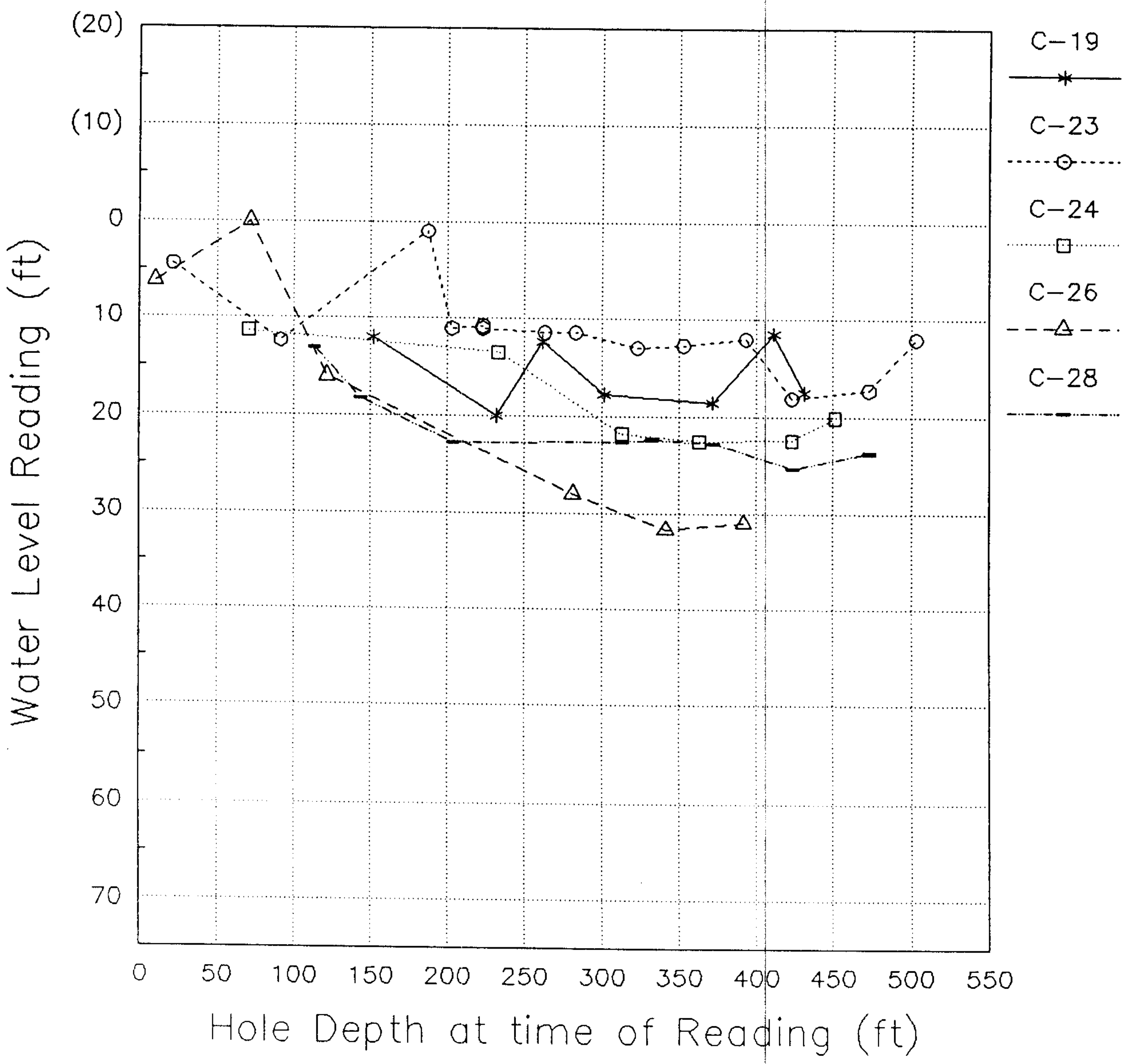
ATTACHMENT E.3.8

GROUNDWATER LEVELS
AND
PRESSURE TEST DATA

Attachment E.3.8

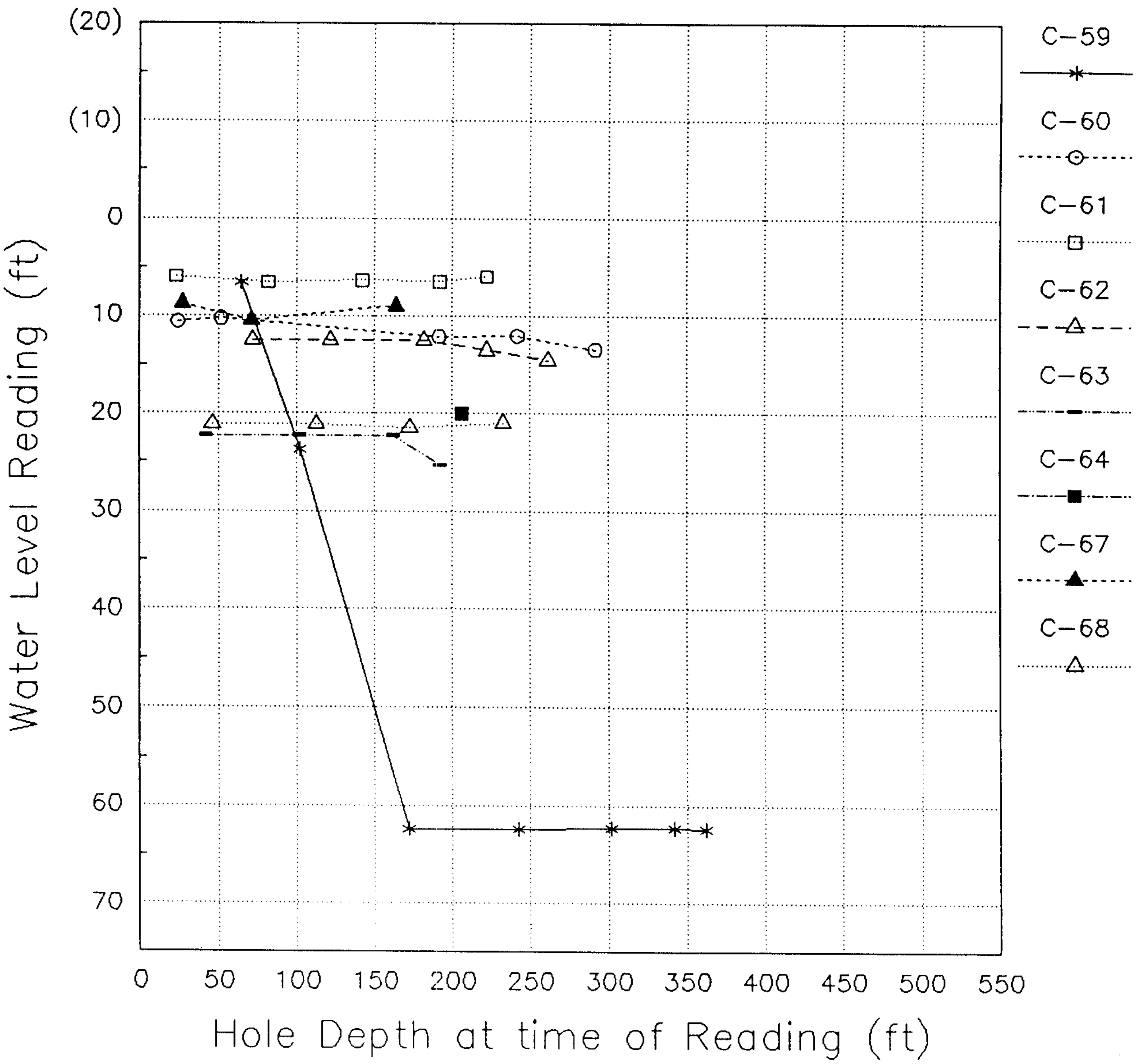
Groundwater Levels and Pressure Test Data

PASSAIC WATER LEVEL READINGS
WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



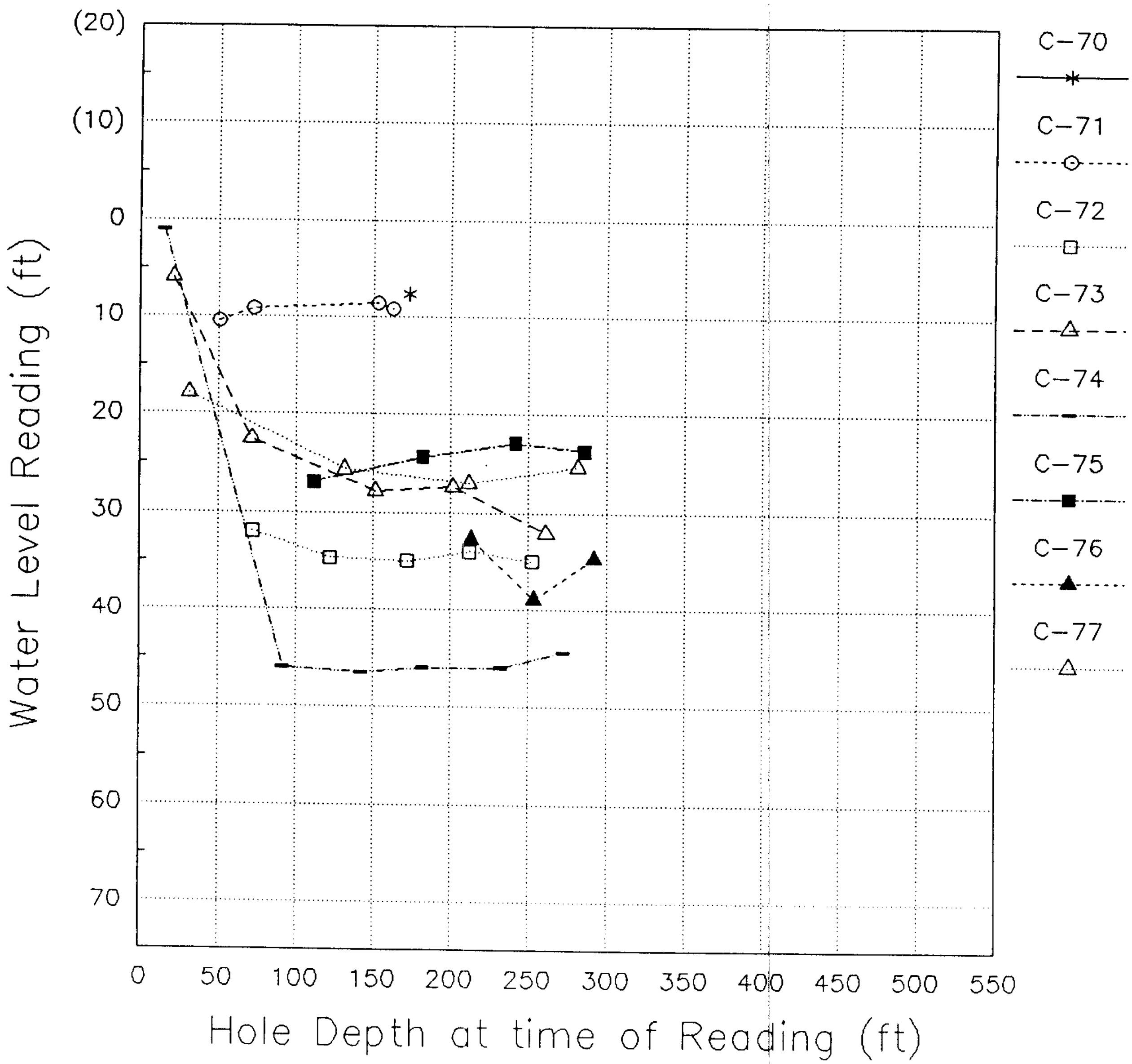
NOTE: A negative water level denotes artesian head above ground surface.

PASSAIC WATER LEVEL READINGS
WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



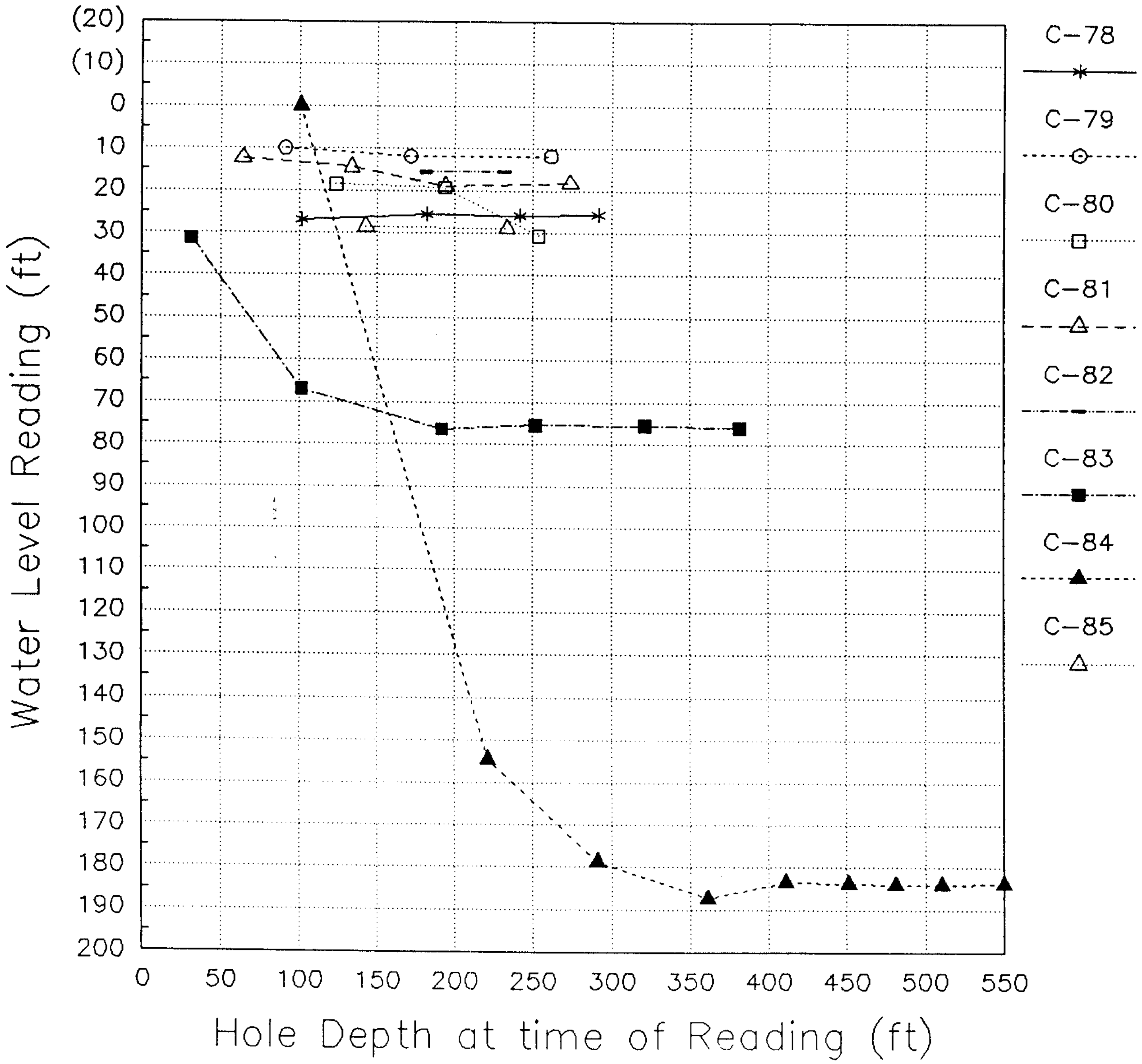
NOTE: A negative water level denotes artesian head above ground surface.

PASSAIC WATER LEVEL READINGS
WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



NOTE: A negative water level denotes artesian head above ground surface.

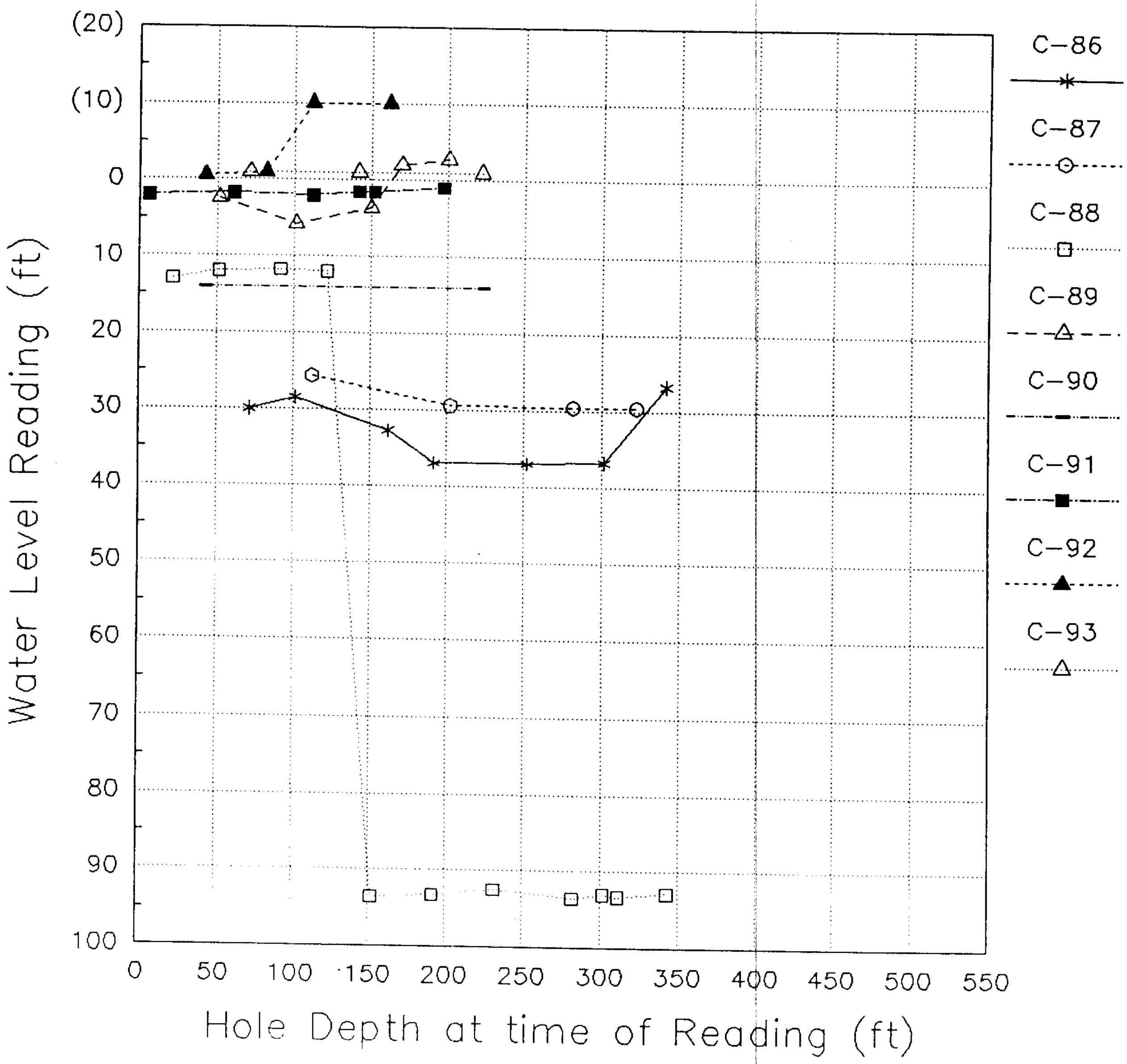
PASSAIC WATER LEVEL READINGS
WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



NOTE: A negative water level denotes artesian head above ground surface.

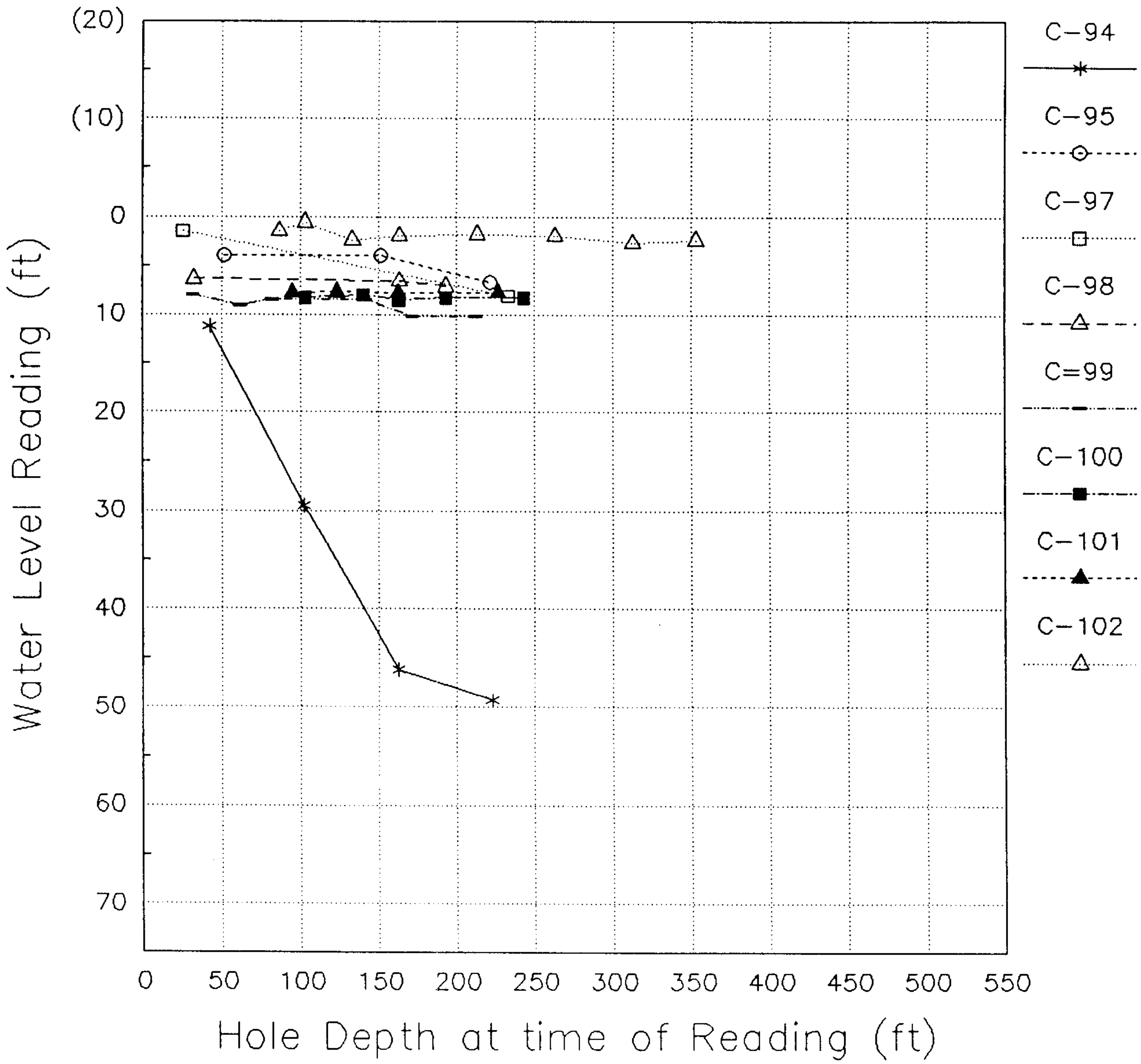
PASSAIC WATER LEVEL READINGS

WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



NOTE: A negative water level denotes artesian head above ground surface.

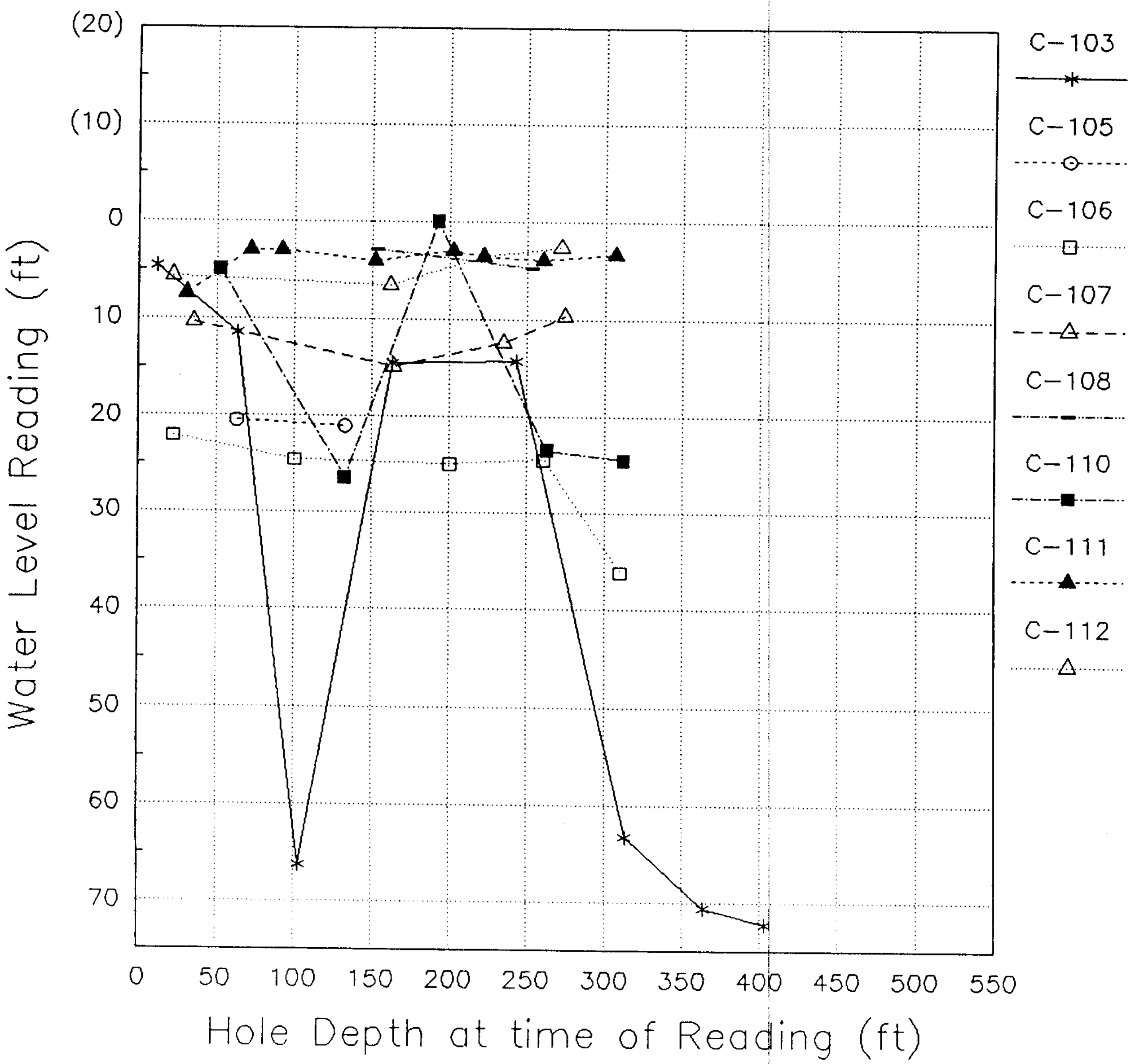
PASSAIC WATER LEVEL READINGS
WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



NOTE: A negative water level denotes artesian head above ground surface.

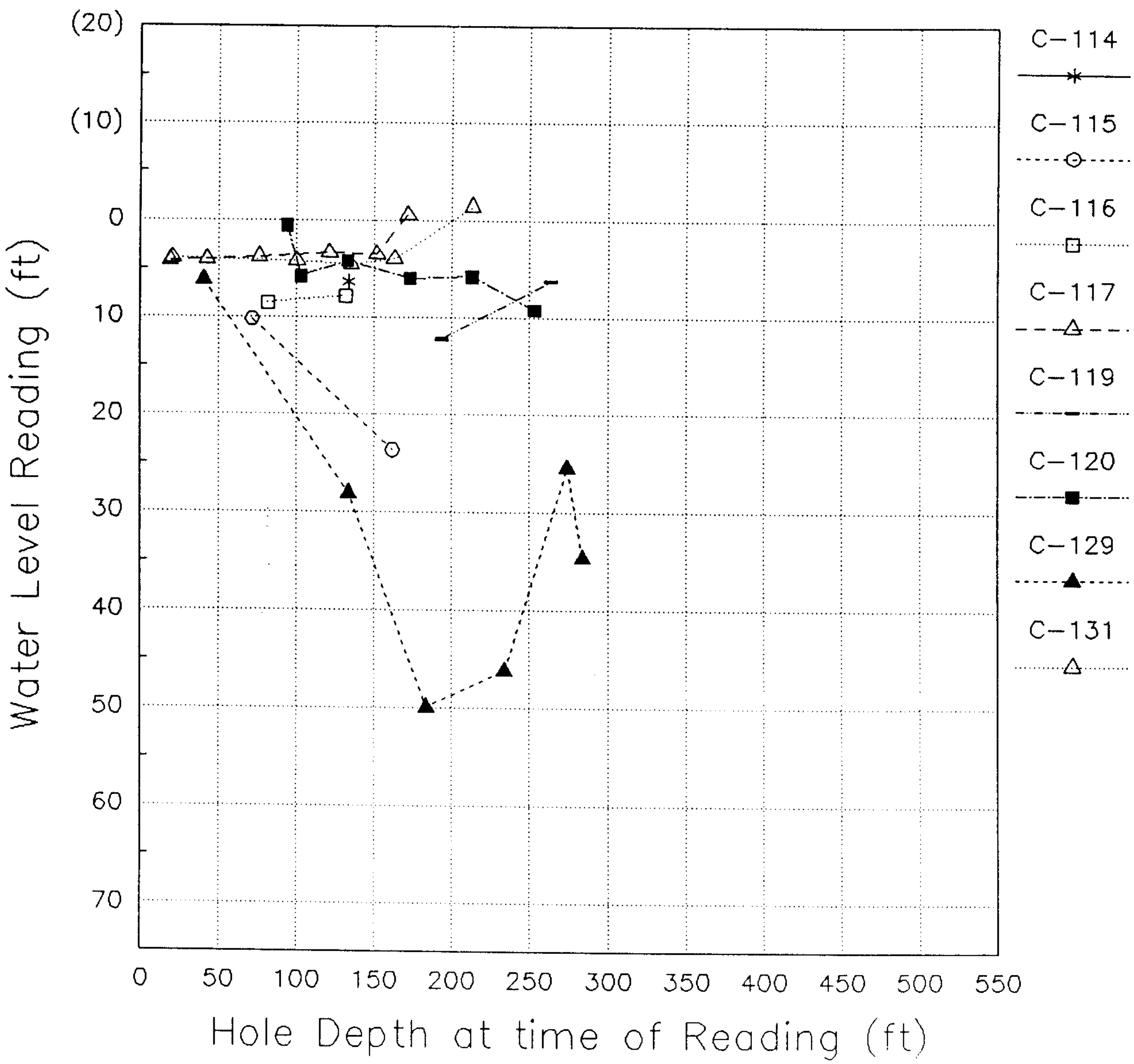
PASSAIC WATER LEVEL READINGS

WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



NOTE: A negative water level denotes artesian head above ground surface.

PASSAIC WATER LEVEL READINGS
WATER LEVEL VS. HOLE DEPTH AT TIME OF READING



NOTE: A negative water level denotes artesian head above ground surface.

PASSAIC RIVER TUNNEL PROJECT
EXPLORATION PROGRAM
PRESSURE TEST DATA

PRINT DATE
30-Jan-95

HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
DC-114	1.6	23-Sep-92	143.4	153.5	50	1	3	21.03	3566	10.1	1.1807	GOOD TEST
DC-114	1.6	24-Sep-92	153.5	163.5	50	0	3	0.00	3566	10	0.0000	GOOD TEST
DC-114	1.6	24-Sep-92	163.5	173.2	50	3	3	63.08	3566	9.7	3.6497	GOOD TEST
DC-114	1.6	24-Sep-92	173.2	183.2	50	0	3	0.00	3566	10	0.0000	GOOD TEST
DC-114	1	25-Sep-92	183.2	233.3	50	3	3	63.08	3547	50.1	1.0146	GOOD TEST
C-147	10	05-Jun-93	72	152.3	50	0	3	0.00	3822	80.3	0.0000	GOOD TEST
C-147	10.9	07-Jun-93	152.3	162.4	50	7	3	147.19	3849	10.1	7.6563	GOOD TEST
C-147	10.9	07-Jun-93	162.4	172.4	50	0	3	0.00	3849	10	0.0000	GOOD TEST
C-147	1.7	08-Jun-93	172.8	232.5	50	0	3	0.00	3569	59.7	0.0000	GOOD TEST
C-147B	13.4	22-Sep-93	71.4	81.3	50	0	3	0.00	3925	9.9	0.0000	GOOD TEST
C-147B	13.4	22-Sep-93	81.3	91.3	50	0	3	0.00	3925	10	0.0000	GOOD TEST
C-147B	13.4	22-Sep-93	91.3	101.3	50	0	3	0.00	3925	10	0.0000	GOOD TEST
C-147B	13.4	22-Sep-93	101.3	111.7	50	0	3	0.00	3925	10.4	0.0000	GOOD TEST
C-147B	13.4	22-Sep-93	111.7	121.8	50	0	3	0.00	3925	10.1	0.0000	GOOD TEST
C-147B	13.4	22-Sep-93	121.8	132	50	1	3	21.03	3925	10.2	1.0647	GOOD TEST
C-147B	13.4	22-Sep-93	132	142.2	50	0	3	0.00	3925	10.2	0.0000	GOOD TEST
C-147B	3	23-Sep-93	142.2	232.1	50	0	3	0.00	3608	89.9	0.0000	GOOD TEST
C-113	14.6	25-May-92	143	152.9	50	0	3	0.00	3962	9.9	0.0000	GOOD TEST
C-113	14.6	25-May-92	152.9	162.8	50	5	3	105.14	3962	9.9	5.3925	GOOD TEST
C-113	15.2	26-May-92	162.8	172.9	50	1	3	21.03	3980	10.1	1.0577	GOOD TEST
C-113	15.2	26-May-92	172.9	183	50	9	3	189.25	3980	10.1	9.5197	GOOD TEST
C-113	15.2	26-May-92	183	233	50	18	3	378.50	3980	50	5.4344	GOOD TEST
C-135	1.2	01-Jul-92	142.6	232.9	50	0	12	0.00	3553	90.3	0.0000	GOOD TEST, COMBINE 4 ZERO TAKE TESTS.
C-112	3.6	30-Sep-91	212.2	221.9	50	2	3	42.06	3627	9.7	2.3922	GOOD TEST
C-112	3.6	30-Sep-91	222.1	302	50	0	21	0.00	3627	79.9	0.0000	GOOD TEST, COMBINE 7 ZERO TAKE TESTS.
C-111	3	14-Aug-91	212.8	221.5	50	0	3	0.00	3608	8.7	0.0000	GOOD TEST
C-111	3.6	15-Aug-91	222.9	230.8	50	2	3	42.06	3627	7.9	2.7800	GOOD TEST
C-111	3.6	15-Aug-91	232.9	280.5	50	0	15	0.00	3627	47.6	0.0000	GOOD TEST, COMBINE 5 ZERO TAKE TESTS.

*** A negative value in the depth to water column denotes artesian pressure

PASSAIC RIVER TUNNEL PROJECT
EXPLORATION PROGRAM
PRESSURE TEST DATA

PRINT DATE
30-Jan-95

HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-111	4	16-Aug-91	282.6	290.5	50	5	3	105.14	3639	7.9	6.9267	GOOD TEST
C-111	4	16-Aug-91	292.4	306.9	50	4	3	84.11	3639	14.5	3.5246	GOOD TEST
C-110	23.5	20-Sep-91	222.6	312.2	50	0	27	0.00	4233	89.6	0.0000	GOOD TEST, COMBINE 9 ZERO TAKE TESTS.
C-134	5.5	28-Jul-92	192.4	272.9	50	0	12	0.00	3685	80.5	0.0000	GOOD TEST, COMPILED 4 ZERO TAKE TESTS.
C-109	1.8	16-Jul-91	181.6	201.6	50	0	6	0.00	3572	20	0.0000	GOOD TEST, COMBINE 2 ZERO TAKE TESTS.
C-109	1.8	16-Jul-91	202	211.3	50	3	3	63.08	3572	9.3	3.7584	GOOD TEST
C-109	1.8	17-Jul-91	212.1	251.7	50	0	12	0.00	3572	39.6	0.0000	GOOD TEST, COMBINE 4 ZERO TAKE TESTS.
C-109	1.8	17-Jul-91	222.6	231.6	50	0	3	0.00	3572	9	0.0000	GOOD TEST
C-109	1.8	17-Jul-91	233.5	241.6	50	0	3	0.00	3572	8.1	0.0000	GOOD TEST
C-109	2.5	18-Jul-91	253.1	276.6	50	0	6	0.00	3593	23.5	0.0000	GOOD TEST, COMBINE 2 ZERO TAKE TESTS.
C-109	1.8	18-Jul-91	262.7	276.6	50	0	3	0.00	3572	13.9	0.0000	GOOD TEST
C-108	4.8	02-Jun-92	242.9	253.1	50	5	3	105.14	3663	10.2	5.7045	GOOD TEST
C-108	4.8	02-Jun-92	253.1	263	50	9	3	189.25	3663	9.9	10.4979	GOOD TEST
C-108	4.8	02-Jun-92	263	273	50	3	3	63.08	3663	10	3.4733	GOOD TEST
C-108	4.8	02-Jun-92	273	283	50	1	3	21.03	3663	10	1.1578	GOOD TEST
C-108	4.8	03-Jun-92	283	322.5	50	23	3	483.64	3663	39.5	9.1390	GOOD TEST
C-133	6	20-Jul-92	242.6	323.1	50	0	12	0.00	3700	80.5	0.0000	GOOD TEST, COMPILED 4 ZERO TAKE TESTS.
C-107	12.4	07-Sep-91	243.8	263.9	50	0	6	0.00	3895	20.1	0.0000	GOOD TEST, COMBINE 2 ZERO TAKE TESTS.
C-107	12.4	07-Sep-91	263.6	274	50	6	3	126.17	3895	10.4	6.3460	GOOD TEST
C-107	9.7	09-Sep-91	273.8	284.2	50	0	3	0.00	3813	10.4	0.0000	GOOD TEST
C-107	9.7	09-Sep-91	284	294.1	50	2	3	42.06	3813	10.1	2.2085	GOOD TEST
C-107	9.7	09-Sep-91	293.9	304.4	50	5	3	105.14	3813	10.5	5.3642	GOOD TEST
C-107	9.7	10-Sep-91	304.1	331.3	50	12	3	252.33	3813	27.2	6.1791	GOOD TEST
C-106	25	02-Aug-91	231.7	240.6	50	1	3	21.03	4279	8.9	1.0801	GOOD TEST
C-106	36.2	05-Aug-91	232	327.7	50	7	3	147.19	4620	95.7	1.0640	GOOD TEST
C-106	25	02-Aug-91	242.4	250.6	50	0	3	0.00	4279	8.2	0.0000	GOOD TEST
C-106	25	02-Aug-91	252.2	260.5	50	1	3	21.03	4279	8.3	1.1366	GOOD TEST
C-106	24.5	03-Aug-91	262.2	270.6	50	0	3	0.00	4264	8.4	0.0000	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-106	24.5	03-Aug-91	272.2	280.6	50	8	3	168.22	4264	8.4	9.0460	GOOD TEST
C-106	24.5	03-Aug-91	282.1	290.6	50	12	3	252.33	4264	8.5	13.4524	GOOD TEST
C-106	24.5	03-Aug-91	292.1	300.6	50	13	3	273.36	4264	8.5	14.5734	GOOD TEST
C-106	24.5	03-Aug-91	302.2	310.4	50	17	3	357.47	4264	8.2	19.5629	GOOD TEST
C-106	36.2	05-Aug-91	312	327.7	50	14	3	294.39	4620	15.7	9.1414	GOOD TEST
C-105	21	14-Sep-91	173.6	182.5	50	5	3	105.14	4157	8.9	5.5589	GOOD TEST
C-105	21	14-Sep-91	182.2	192.6	50	11	3	231.31	4157	10.4	10.9008	GOOD TEST
C-105	21	16-Sep-91	192.1	202.8	50	5	3	105.14	4157	10.7	4.8511	GOOD TEST
C-105	21	16-Sep-91	202.2	213	50	9	3	189.25	4157	10.8	8.6715	GOOD TEST
C-105	21	16-Sep-91	212.6	223.2	50	0	3	0.00	4157	10.6	0.0000	GOOD TEST
C-105	21	16-Sep-91	223	246.6	50	22	3	462.61	4157	23.6	11.6251	GOOD TEST
DC-132	-12	18-Mar-93	65.6	91.5	50	0	3	0.00	3151	25.9	0.0000	GOOD TEST
DC-132	-12	19-Mar-93	91.5	101.5	50	42	3	883.17	3151	10	56.5286	GOOD TEST
DC-132	-12	19-Mar-93	101.5	111.7	50	0	3	0.00	3151	10.2	0.0000	GOOD TEST
DC-132	-12	19-Mar-93	111.7	121.9	50	1	3	21.03	3151	10.2	1.3263	GOOD TEST
DC-132	-12	19-Mar-93	121.9	131.9	50	6	3	126.17	3151	10	8.0755	GOOD TEST
DC-132	-12	19-Mar-93	131.9	142.2	50	3	3	63.08	3151	10.3	3.9502	GOOD TEST
DC-132	-12	19-Mar-93	142.2	152.4	50	0	3	0.00	3151	10.2	0.0000	GOOD TEST
DC-132	-12	20-Mar-93	152.4	162.4	50	14	3	294.39	3151	10	18.8429	GOOD TEST
DC-132	-12	20-Mar-93	162.4	172.4	50	6	3	126.17	3151	10	8.0755	GOOD TEST
DC-132	-12	20-Mar-93	172.4	182.4	50	0	3	0.00	3151	10	0.0000	GOOD TEST
DC-132	-12	20-Mar-93	182.4	192.4	50	14	3	294.39	3151	10	18.8429	GOOD TEST
DC-132	-12	20-Mar-93	192.4	202.3	50	0	3	0.00	3151	9.9	0.0000	GOOD TEST
DC-132	-12	20-Mar-93	202.3	211.6	50	0	3	0.00	3151	9.3	0.0000	GOOD TEST
DC-132	-12	21-Mar-93	211.6	221.9	50	0	3	0.00	3151	10.3	0.0000	GOOD TEST
DC-132	-12	21-Mar-93	221.9	232.1	50	2	3	42.06	3151	10.2	2.6526	GOOD TEST
DC-132	-12	21-Mar-93	232.1	242	50	0	3	0.00	3151	9.9	0.0000	GOOD TEST
DC-132	-12	21-Mar-93	242	252.3	50	6	3	126.17	3151	10.3	7.9003	GOOD TEST

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			TOP	BOTTOM								
DC-132	-12	22-Mar-93	252.3	262.3	50	8	3	168.22	3151	10	10.7674	GOOD TEST
DC-132	-12	22-Mar-93	262.3	271.8	50	14	3	294.39	3151	9.5	19.5712	GOOD TEST
C-104	-8	26-Sep-91	143.3	153.6	50	2	3	42.06	3273	10.3	2.5353	GOOD TEST, ARTESIAN HOLE.
C-104	-8	28-Sep-91	153.4	226.1	50	0	21	0.00	3273	72.7	0.0000	GOOD TEST, COMBINE 7 ZERO TAKE TESTS IN 3 DAYS. ARTESIAN.
C-131	4.6	11-Jul-92	142.6	163	50	0	6	0.00	3657	20.4	0.0000	COMBINE 2 ZERO TAKE TESTS
C-131	4	13-Jul-92	162.8	172.8	50	11	3	231.31	3639	10	10.4698	GOOD TEST
C-131	4	13-Jul-92	172.8	228.9	50	0	6	0.00	3639	56.1	0.0000	COMBINE 2 ZERO TAKE TESTS
C-130	17.6	25-Jun-92	262.1	271.6	50	2	3	42.06	4053	9.5	2.1736	GOOD TEST
C-130	17.6	25-Jun-92	271.6	282	50	0	3	0.00	4053	10.4	0.0000	GOOD TEST
C-130	17.6	25-Jun-92	282	291.9	50	1	3	21.03	4053	9.9	1.0542	GOOD TEST
C-130	17.6	25-Jun-92	291.9	301.4	50	48	3	1009.34	4053	9.5	52.1658	GOOD TEST
C-130	42.4	26-Jun-92	301.4	351.3	15	100	3	2102.78	2347	49.9	51.2748	GOOD TEST
C-103	63.2	13-Jun-92	323.2	333.3	30	87	3	1829.42	4036	10.1	90.7405	GOOD TEST
C-103	63.2	13-Jun-92	333.3	343.2	40	1	3	21.03	4740	9.9	0.9015	GOOD TEST
C-103	63.2	13-Jun-92	343.2	352.9	42	0	3	0.00	4881	9.7	0.0000	GOOD TEST
C-103	63.2	13-Jun-92	352.9	362.9	37	5	3	105.14	4529	10	4.6824	GOOD TEST
C-103	70.6	15-Jun-92	362.9	403.1	35	6	3	126.17	4614	40.2	1.8662	GOOD TEST
C-129	49.9	30-Jul-92	203.7	213.6	50	0	3	0.00	5038	9.9	0.0000	GOOD TEST
C-129	49.9	30-Jul-92	213.6	223.7	50	94	3	1976.62	5038	10.1	78.5536	GOOD TEST
C-129	46.2	31-Jul-92	223.7	233.7	50	27	3	567.75	4925	10	23.2508	GOOD TEST
C-129	46.2	31-Jul-92	233.7	243.9	50	1	3	21.03	4925	10.2	0.8486	GOOD TEST
C-129	25.4	03-Aug-92	243.9	283.8	50	7	3	147.19	4291	39.9	2.3552	GOOD TEST
C-102	0.6	10-Mar-94	88.5	103.1	50	0	3	0.00	3535	14.6	0.0000	GOOD TEST
C-102	0.6	11-Mar-94	103.4	113.1	50	1	3	21.03	3535	9.7	1.2271	GOOD TEST
C-102	0.6	11-Mar-94	113.1	273.1	50	0	3	0.00	3535	160	0.0000	COMBINED SEVERAL 0 TAKE TESTS OVER SEVERAL DAYS
C-101	7.8	15-Aug-91	130	226.6	50	98	3	2060.73	3755	96.6	18.1881	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-101	7.7	13-Aug-91	142.5	152.3	50	23	3	483.64	3752	9.8	26.3934	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-101	7.7	13-Aug-91	151.4	162.5	50	78	3	1640.17	3752	11.1	81.5869	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST

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			TOP	BOTTOM								
C-101	7.8	14-Aug-91	161.7	172.3	50	22	3	462.61	3755	10.6	23.7979	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-101	7.8	14-Aug-91	171.2	182.5	50	27	3	567.75	3755	11.3	27.8440	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-101	7.8	14-Aug-91	181.4	192.7	50	50	3	1051.39	3755	11.3	51.5630	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-101	7.8	15-Aug-91	191.3	202.9	50	80	3	1682.23	3755	11.6	80.8957	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-101	7.8	15-Aug-91	202.6	212	50	32	3	672.89	3755	9.4	37.8379	LEAKAGE AROUND PACKER, PROBABLY A POOR TEST
C-102	2	16-Mar-94	273.6	283	50	18	3	378.50	3578	9.4	22.3355	GOOD TEST
C-102	2	16-Mar-94	283.4	293.1	50	4	3	84.11	3578	9.7	4.8497	GOOD TEST
C-102	2	16-Mar-94	293.5	302.9	50	7	3	147.19	3578	9.4	8.6860	GOOD TEST
C-102	2	16-Mar-94	303.6	353.1	50	0	3	0.00	3578	49.5	0.0000	COMBINED SEVERAL 0 TAKE TESTS
C-127	8.8	20-Jul-92	142.9	152.9	50	23	3	483.64	3785	10	25.7712	GOOD TEST
C-127	8.8	20-Jul-92	152.9	162.9	50	3	3	63.08	3785	10	3.3615	GOOD TEST
C-127	8.8	20-Jul-92	162.9	172.7	50	60	3	1261.67	3785	9.8	68.2425	GOOD TEST
C-127	8.8	20-Jul-92	172.7	183	50	48	3	1009.34	3785	10.3	52.6165	GOOD TEST
C-127	8.8	21-Jul-92	183	222.7	50	46	3	967.28	3785	39.7	17.6172	GOOD TEST
C-126	5.6	13-Jul-92	143	152.9	50	34	3	714.95	3688	9.9	39.3964	GOOD TEST
C-126	5.6	14-Jul-92	152.9	163	50	172	3	3616.79	3688	10.1	196.3673	GOOD TEST
C-126	5.6	14-Jul-92	163	173	50	7	3	147.19	3688	10	8.0509	GOOD TEST
C-126	5.6	14-Jul-92	173	183	50	10	3	210.28	3688	10	11.5012	GOOD TEST
C-126	12.3	15-Jul-92	183	223	50	30	3	630.84	3892	40	11.1066	GOOD TEST
C-100A	8.3	03-Mar-92	146	163	50	9	3	189.25	3770	17	6.7742	GOOD TEST
C-100A	8.3	04-Mar-92	162.7	173	50	14	3	294.39	3770	10.3	15.4085	GOOD TEST
C-100A	8.3	04-Mar-92	172.8	183	50	13	3	273.36	3770	10.2	14.4120	GOOD TEST
C-100A	8.3	05-Mar-92	182.7	203	50	0	6	0.00	3770	20.3	0.0000	GOOD TEST, COMBINE 2 ZERO TAKE TESTS.
C-100A	8.3	05-Mar-92	202.9	213	50	23	3	483.64	3770	10.1	25.6852	GOOD TEST, SLIGHT AIR LEAK ON PACKER.
C-100A	8.3	06-Mar-92	212.8	273	50	0	15	0.00	3770	60.2	0.0000	GOOD TEST, COMBINE 5 ZERO TAKE TESTS.
DC-136	0.3	14-Apr-93	80.5	93	50	0	3	0.00	3526	12.5	0.0000	GOOD TEST
DC-136	0.3	14-Apr-93	93	103	50	0	3	0.00	3526	10	0.0000	GOOD TEST
DC-136	0.3	14-Apr-93	103	112.9	50	0	3	0.00	3526	9.9	0.0000	GOOD TEST

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			TOP	BOTTOM								
DC-136	5.4	15-Apr-93	112.9	123	50	0	3	0.00	3682	10.1	0.0000	GOOD TEST
DC-136	5.4	15-Apr-93	123	133	50	0	3	0.00	3682	10	0.0000	GOOD TEST
DC-136	5.4	15-Apr-93	133	143	50	56	3	1177.56	3682	10	64.5136	GOOD TEST
DC-136	5.4	15-Apr-93	143	152.7	50	122	3	2565.40	3682	9.7	143.7518	GOOD TEST
DC-136	5.4	15-Apr-93	152.7	162.9	50	22	3	462.61	3682	10.2	24.9751	GOOD TEST
DC-136	4.8	16-Apr-93	162.9	173.1	50	10	3	210.28	3663	10.2	11.4090	GOOD TEST
DC-136	4.8	16-Apr-93	173.1	182.9	50	19	3	399.53	3663	9.8	22.3294	GOOD TEST
DC-136	4.8	16-Apr-93	182.9	192.9	50	11	3	231.31	3663	10	12.7356	GOOD TEST
DC-136	4.8	16-Apr-93	192.9	203	50	29	3	609.81	3663	10.1	33.3288	GOOD TEST
DC-136	4.8	16-Apr-93	203	213	50	0	3	0.00	3663	10	0.0000	GOOD TEST
DC-136	5.3	17-Apr-93	213	223	50	4	3	84.11	3678	10	4.6119	GOOD TEST
DC-136	5.3	17-Apr-93	223	232.9	50	11	3	231.31	3678	9.9	12.7776	GOOD TEST
DC-136	5.3	17-Apr-93	232.8	243	50	11	3	231.31	3678	10.2	12.4979	GOOD TEST
DC-136	5.3	17-Apr-93	243	253.1	50	0	3	0.00	3678	10.1	0.0000	GOOD TEST
DC-136	5.3	17-Apr-93	253.1	263.1	50	0	3	0.00	3678	10	0.0000	GOOD TEST
DC-136	5.3	17-Apr-93	263.1	273.1	50	0	3	0.00	3678	10	0.0000	GOOD TEST
DC-136	5.4	19-Apr-93	273.1	283.1	50	0	3	0.00	3682	10	0.0000	GOOD TEST
DC-136	5.4	19-Apr-93	283.1	293.1	50	0	3	0.00	3682	10	0.0000	GOOD TEST
DC-136	5.4	19-Apr-93	293.2	303.1	50	0	3	0.00	3682	9.9	0.0000	GOOD TEST
C-125	7.5	30-Apr-92	123.5	133.5	50	8	3	168.22	3746	10	9.0587	GOOD TEST
C-125	7.5	30-Apr-92	133.7	143.6	50	0	3	0.00	3746	9.9	0.0000	GOOD TEST
C-125	7.5	30-Apr-92	142.4	153.6	50	7	3	147.19	3746	11.2	7.2848	GOOD TEST
C-125	6.8	01-May-92	152.8	163.6	50	8	3	168.22	3724	10.8	8.6038	GOOD TEST
C-125	6.8	01-May-92	162.4	173.6	50	52	3	1093.45	3724	11.2	54.4255	GOOD TEST
C-125	6.8	01-May-92	172.5	193.4	50	0	6	0.00	3724	20.9	0.0000	GOOD TEST, COMBINE 2 ZERO TAKE TESTS.
C-125	6.8	01-May-92	192.4	203.5	50	8	3	168.22	3724	11.1	8.4295	GOOD TEST
C-125	9.6	02-May-92	203.7	213.3	46	221	3	4647.15	3528	9.6	273.8082	GOOD TEST
C-125	9.6	02-May-92	213.6	233.3	50	176	3	3700.90	3810	19.7	116.9230	GOOD TEST

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			TOP	BOTTOM								
PT-17	11	237	93	103	20	4.6	1	290.18	1742	10	35.6125	GOOD TEST
PT-17	11	242	103	113	20	0.94	1	59.30	1742	10	7.2773	GOOD TEST
PT-17	11	247	122.5	132.5	20	5	1	315.42	1742	10	38.7093	GOOD TEST
PT-17	11	252	142.5	152.5	20	1.86	1	117.34	1742	10	14.3998	GOOD TEST
PT-17	11	257	182.5	192.5	20	1	1	63.08	1742	10	7.7419	GOOD TEST
C-124	7.8	26-May-92	142	232.2	50	0	9	0.00	3755	90.2	0.0000	GOOD TEST, COMBINE 3 ZERO TAKE TESTS.
C-123	6.1	23-Mar-92	141.6	173	50	0	9	0.00	3703	31.4	0.0000	GOOD TEST, COMBINE 3 ZERO TAKE TESTS.
C-123	6.1	24-Mar-92	173.5	183	50	7	3	147.19	3703	9.5	8.3276	GOOD TEST
C-123	6.1	24-Mar-92	183.4	193	50	8	3	168.22	3703	9.6	9.4440	GOOD TEST
C-123	7	25-Mar-92	193.7	203	50	33	3	693.92	3730	9.3	39.5860	GOOD TEST
C-123	7	25-Mar-92	203.2	233	50	207	3	4352.76	3730	29.8	101.3029	GOOD TEST
C-123	7	25-Mar-92	213	233	50	40	3	841.11	3730	20	26.8199	GOOD TEST
C-99-D	10.1	15-Jun-91	122.4	227.7	60	20	3	420.56	4528	105.3	2.8632	GOOD TEST
C-99-D	8.5	13-Jun-91	132.5	161.7	70	0	6	0.00	5183	29.2	0.0000	GOOD TEST, COMBINE 3 ZERO TAKE TESTS.
C-99-D	8.5	13-Jun-91	162.5	171.6	80	9	3	189.25	5886	9.1	6.9522	GOOD TEST
C-99-D	10.1	14-Jun-91	172.8	181.7	85	4	3	84.11	6287	8.9	2.9406	GOOD TEST
C-99-D	10.1	14-Jun-91	182.6	191.7	90	0	3	0.00	6638	9.1	0.0000	GOOD TEST
C-99-D	10.1	14-Jun-91	192.6	201.7	95	2	3	42.06	6990	9.1	1.3010	GOOD TEST
C-99-D	10.1	14-Jun-91	203.6	211.8	100	14	3	294.39	7342	8.2	9.3562	GOOD TEST
C-99-D	10.1	15-Jun-91	211.5	227.7	50	10	3	210.28	3825	16.2	7.6998	POOR SEAL ON PACKER
C-99-D	10.1	15-Jun-91	213.6	227.1	50	10	3	210.28	3825	13.5	8.8521	POOR SEAL ON PACKER
C-98	7.1	23-Aug-91	132.8	223.4	50	0	27	0.00	3733	90.6	0.0000	GOOD TEST COMBINED 9 ZERO TAKE TESTS.
PT-19	5	347	131.7	151.7	30	1.6	1	100.93	2263	20	5.5760	GOOD TEST
PT-19	5	352	131.7	141.2	30	1.04	1	65.61	2263	9.5	6.4438	GOOD TEST
PT-19	5	357	141.5	151.7	30	0.56	1	35.33	2263	10.2	3.2884	GOOD TEST
C-97	7.8	21-Apr-92	141.7	149.7	50	2	3	42.06	3755	8	2.6608	GOOD TEST
C-97	7.8	21-Apr-92	149.7	163	50	0.5	3	10.51	3755	13.3	0.4560	GOOD TEST
C-97	7.8	21-Apr-92	163	173	50	0	3	0.00	3755	10	0.0000	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-97	7.8	21-Apr-92	173	183	50	2	3	42.06	3755	10	2.2592	GOOD TEST
C-97	7.8	21-Apr-92	183	233	50	0	3	0.00	3755	50	0.0000	GOOD TEST
DC-141	3.2	10-Nov-92	61.5	67.5	50	2	3	42.06	3614	6	2.9268	GOOD TEST USING A DOUBLE PACKER ON REAMED HOLE
DC-141	3.2	10-Nov-92	67.5	77.5	50	2	3	42.06	3614	10	2.0664	GOOD TEST
DC-141	3.2	10-Nov-92	77.5	87.5	50	6	3	126.17	3614	10	6.1993	GOOD TEST
DC-141	3.2	10-Nov-92	87.5	97.5	50	2	3	42.06	3614	10	2.0664	GOOD TEST
DC-141	3.2	10-Nov-92	97.5	107.5	50	7	3	147.19	3614	10	7.2325	GOOD TEST
DC-141	3.2	10-Nov-92	107.5	117.5	50	0	3	0.00	3614	10	0.0000	GOOD TEST
DC-141	3.2	10-Nov-92	117.5	127.5	50	3	3	63.08	3614	10	3.0996	GOOD TEST
DC-141	3.2	10-Nov-92	127.5	137.5	50	2	3	42.06	3614	10	2.0664	GOOD TEST
DC-141	3.2	10-Nov-92	137.5	147.5	50	5	3	105.14	3614	10	5.1660	GOOD TEST
DC-141	5.3	02-Nov-92	172.6	251.7	50	0	3	0.00	3678	79.1	0.0000	GOOD TEST, COMPOSITE OF 5 ZERO TAKE TESTS.
C-95	6.7	21-May-91	100	238.3	50	0	3	0.00	3721	138.3	0.0000	GOOD TEST
C-94	29.5	23-Aug-91	141.2	152.4	50	0	3	0.00	4416	11.2	0.0000	GOOD TEST
C-94	29.5	23-Aug-91	153.3	162.4	50	7	3	147.19	4416	9.1	7.2073	GOOD TEST
C-94	46.2	24-Aug-92	162.8	172.4	50	5	3	105.14	4925	9.6	4.4377	GOOD TEST
C-94	46.2	24-Aug-92	173	182.4	50	5	3	105.14	4925	9.4	4.5072	GOOD TEST
C-94	46.2	24-Aug-92	182.7	192.5	50	7	3	147.19	4925	9.8	6.1188	GOOD TEST
C-94	46.2	24-Aug-92	192.5	202.5	50	0	3	0.00	4925	10	0.0000	GOOD TEST
C-94	46.2	24-Aug-92	202.6	212.5	50	1	3	21.03	4925	9.9	0.8676	GOOD TEST
C-94	49.3	26-Aug-91	212.7	222.5	50	0	3	0.00	5020	9.8	0.0000	GOOD TEST
PT-23	15	152	122	132	20	1.5	1	94.63	1864	10	10.8532	GOOD TEST
PT-23	15	157	222	232	20	2.8	1	176.63	1864	10	20.2593	GOOD TEST
C-93	-11	24-Apr-92	142	152	50	50	3	1051.39	3182	10	66.6513	GOOD TEST
C-93	-11	24-Apr-92	152	162	50	93	3	1955.59	3182	10	123.9714	GOOD TEST
C-93	-11	24-Apr-92	162	172	50	66	3	1387.84	3182	10	87.9797	GOOD TEST
C-93	-11	24-Apr-92	172	182	50	90	3	1892.51	3182	10	119.9723	GOOD TEST
C-93	-11	27-Apr-92	182	222	50	9	3	189.25	3182	40	4.0757	GOOD TEST

*** A negative value in the depth to water column denotes artesian pressure

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-92	-1	01-Jun-91	91.1	102	45	22	3	462.61	3135	10.9	27.9168	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	01-Jun-91	101.6	112	45	178	3	3742.95	3135	10.4	233.9166	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	02-Jun-91	111.1	122	55	23	3	483.64	3838	10.9	23.8371	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	02-Jun-91	120.3	132	60	83	3	1745.31	4190	11.7	74.7290	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	02-Jun-91	131.2	142.1	65	12	3	252.33	4542	10.9	10.5106	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	02-Jun-91	141.2	152	70	22	3	462.61	4893	10.8	18.0079	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	02-Jun-91	150.8	162	75	10	3	210.28	5245	11.2	7.4318	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	04-Jun-91	160.3	172	58	145	3	3049.04	4049	11.7	135.0863	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
C-92	-1	04-Jun-91	170.1	182	80	36	3	757.00	5597	11.9	23.9581	LEAKAGE AROUND PACKER, ARTESIAN HOLE, 10 GPM, "SALTY"
PT-26	53	232	133	143	20	3	1	189.25	3022	10	13.3876	GOOD TEST
PT-26	53	237	142.5	152.5	20	1.6	1	100.93	3022	10	7.1400	GOOD TEST
PT-26	53	242	172.7	182.7	20	3	1	189.25	3022	10	13.3876	GOOD TEST
C-91	2.1	23-Jul-91	103.1	112.1	50	5	3	105.14	3581	9	6.4005	GOOD TEST
C-91	2.1	24-Jul-91	113	122.1	50	1	3	21.03	3581	9.1	1.2697	GOOD TEST
C-91	2.1	24-Jul-91	123	162	50	0	12	0.00	3581	39	0.0000	GOOD TEST COMBINED 4 ZERO TAKE TESTS
C-91	2.1	26-Jul-91	162.5	172	50	1	3	21.03	3581	9.5	1.2302	GOOD TEST
C-91	2.1	26-Jul-91	172.8	197	50	0	6	0.00	3581	24.2	0.0000	GOOD TEST COMBINED 2 ZERO TAKE TESTS
C-90	-5.8	01-Aug-91	130.5	223.7	50	110	3	2313.06	3340	93.2	23.6470	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	140.7	223.7	50	117	3	2460.26	3340	83	27.7056	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	142.1	153.3	50	70	3	1471.95	3340	11.2	81.6891	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	152.7	163.3	50	80	3	1682.23	3340	10.6	97.2774	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	162.5	173.3	50	115	4	1813.65	3340	10.8	103.4257	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	173	183.3	50	75	3	1577.09	3340	10.3	93.1666	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	182.8	193.3	50	75	3	1577.09	3340	10.5	91.8435	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	192.7	203.5	50	110	5	1387.84	3340	10.8	79.1432	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	202.2	213.7	50	97	4	1529.77	3340	11.5	83.2332	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-90	-5.8	01-Aug-91	213	223.7	50	75	3	1577.09	3340	10.7	90.5616	ARTESIAN HOLE SOME FLOW AROUND PACKER NOTED
C-89	-2.7	15-May-91	90	226.1	55	100	3	2102.78	3786	136.1	13.7931	GOOD TEST HOLE IS ARTESIAN.

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-89	-2.7	15-May-91	90	226.1	55	100	3	2102.78	3786	136.1	13.7931	GOOD TEST HOLE IS ARTESIAN.
C-89	-2.7	15-May-91	140	226.1	62	123	3	2586.42	4279	86.1	22.0532	GOOD TEST HOLE IS ARTESIAN
C-88	93.5	16-Mar-92	291.8	321.5	50	0	9	0.00	6367	29.7	0.0000	GOOD TEST COMBINED 3 ZERO TAKE TESTS
C-88	93.5	17-Mar-91	321.9	331.6	50	1	3	21.03	6367	9.7	0.6813	GOOD TEST
C-88	93.5	17-Mar-91	331.5	341	50	6	3	126.17	6367	9.5	4.1514	GOOD TEST
C-88	93.5	20-Mar-91	341.9	376.3	50	9	3	189.25	6367	34.4	2.3003	GOOD TEST
C-87	29.6	30-Oct-91	280	300	50	0	6	0.00	4419	20	0.0000	GOOD TEST COMBINED 2 ZERO TAKE TESTS
C-87	29.6	30-Oct-91	300	310	50	1	3	21.03	4419	10	0.9597	GOOD TEST
C-87	29.6	30-Oct-91	310	330	50	0	6	0.00	4419	20	0.0000	GOOD TEST COMBINED 2 ZERO TAKE TESTS
C-87	29.6	01-Nov-91	330	340	50	2	3	42.06	4419	10	1.9195	GOOD TEST
C-87	29.6	01-Nov-91	340	350	50	0	3	0.00	4419	10	0.0000	GOOD TEST
C-87	29.6	01-Nov-91	350	360	50	2	3	42.06	4419	10	1.9195	GOOD TEST
C-87	29.6	02-Nov-91	360	389.1	50	3	3	63.08	4419	29.1	1.2630	GOOD TEST
C-86	36.8	21-Oct-91	302.2	311.8	50	0	3	0.00	4639	9.6	0.0000	GOOD TEST
C-86	36.8	21-Oct-91	312.1	321.9	50	5	3	105.14	4639	9.8	4.6406	GOOD TEST
C-86	36.8	21-Oct-91	321.8	331.9	50	0	3	0.00	4639	10.1	0.0000	GOOD TEST
C-86	36.8	21-Oct-91	331.9	341.9	50	2	3	42.06	4639	10	1.8287	GOOD TEST
C-86	36.8	22-Oct-91	341.7	361.9	50	0	6	0.00	4639	20.2	0.0000	GOOD TEST COMBINED 2 ZERO TAKE TESTS.
C-86	36.8	22-Oct-91	361.8	371.8	50	1	3	21.03	4639	10	0.9143	GOOD TEST
C-86	36.8	22-Oct-91	371.6	401.8	50	0	9	0.00	4639	30.2	0.0000	GOOD TEST COMBINED 3 ZERO TAKE TESTS
C-85	29	11-Jun-93	362.7	402.2	50	9	3	189.25	4401	39.5	2.9768	BOTTOM OF THIS TEST CHANGED BECAUSE OF TIGHT TEST @ BOH.
C-85	29	11-Jun-93	402.2	449.3	50	0	3	0.00	4401	47.1	0.0000	GOOD TEST
C-84	-2.6	11-Jun-92	362.7	449.3	50	9	3	189.25	3438	86.6	1.9987	GOOD TEST
C-84	-2.6	11-Jun-92	402.2	449.3	50	0	3	0.00	3438	47.1	0.0000	GOOD TEST
C-84	184	11-Sep-91	440.3	481.1	52	8	3	168.22	9266	40.8	1.2242	GOOD TEST
C-84	184	11-Sep-91	473	481.1	50	10	2	315.42	9125	8.1	8.1375	TEST IN QUESTION, ADDED GAS PRESSURE TO PACKER.
C-84	184	11-Sep-91	481.9	491.8	50	1	3	21.03	9125	9.9	0.4683	GOOD TEST
C-84	184	11-Sep-91	491.8	501.1	50	5	3	105.14	9125	9.3	2.4519	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET TOP	BOTTOM	GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
C-84	184	11-Sep-91	501.8	510.6	50	1	3	21.03	9125	8.8	0.5107	GOOD TEST
C-84	184	12-Sep-91	511.9	520.8	50	1	3	21.03	9125	8.9	0.5065	GOOD TEST
C-84	184	12-Sep-91	521.9	531	50	0	3	0.00	9125	9.1	0.0000	GOOD TEST
C-84	184	12-Sep-91	532	539.7	50	1	3	21.03	9125	7.7	0.5628	GOOD TEST
C-84	184	12-Sep-91	542	550	50	0	3	0.00	9125	8	0.0000	GOOD TEST
PT-40	56	402.2	342.2	352.2	40	0.72	1	45.42	4520	10	2.1481	GOOD TEST
PT-40	56	412.2	392	402.2	60	0.2	1	12.62	5927	10.2	0.4483	GOOD TEST
PT-40	56	381.6	402.2	412.2	50	1.5	1	94.63	5224	10	3.8727	GOOD TEST
C-83	75.8	15-Apr-91	240	381.6	60	78	3	1640.17	6531	141.6	6.0320	GOOD TEST
C-83	75.8	15-Apr-91	290	381.6	60	35	3	735.97	6531	91.6	3.9043	GOOD TEST
C-83	75.8	15-Apr-91	290	381.6	60	55	3	1156.53	6531	91.6	6.1353	GOOD TEST
C-82	15.6	19-May-92	207.8	221.3	50	3	3	63.08	3992	13.5	2.5441	GOOD TEST
C-82	15.6	19-May-92	221.3	231.3	50	30	3	630.84	3992	10	31.8695	GOOD TEST
C-82	15.5	20-May-92	231.3	241.4	30	116	3	2439.23	2583	10.1	189.0983	GOOD TEST
C-82	15.5	20-May-92	241.4	251.4	50	0	3	0.00	3989	10	0.0000	GOOD TEST
C-82	15.5	20-May-92	251.4	291.4	30	98	3	2060.73	2583	40	54.6746	GOOD TEST
C-81	18.9	19-Jun-92	193.9	204	50	10	3	210.28	4093	10.1	10.2860	GOOD TEST
C-81	18.9	18-Jun-92	203.7	213.9	50	0	3	0.00	4093	10.2	0.0000	GOOD TEST
C-81	18.9	18-Jun-92	213.8	223.9	51	10	3	210.28	4163	10.1	10.1122	GOOD TEST
C-81	18.9	18-Jun-92	223.5	233.9	50	37	3	778.03	4093	10.4	37.2396	GOOD TEST
C-81	18.9	18-Jun-92	233.7	243.9	50	14	3	294.39	4093	10.2	14.2954	GOOD TEST
C-81	18.9	19-Jun-92	243.6	254	50	12	3	252.33	4093	10.4	12.0777	GOOD TEST
C-81	18.9	19-Jun-92	253.7	273.7	50	0	3	0.00	4093	20	0.0000	GOOD TEST
C-79	11.8	20-Apr-91	150	290.9	60	142	3	2985.95	4580	140.9	15.7243	GOOD TEST
C-79	11.8	20-Apr-91	200	290.9	60	122	3	2565.40	4580	90.9	19.5304	GOOD TEST
C-80	19.3	06-Jul-92	193.3	203.3	50	23	3	483.64	4105	10	23.7621	GOOD TEST
C-80	19.3	06-Jul-92	203.3	213.4	40	96	3	2018.67	3402	10.1	118.8086	GOOD TEST
C-80	19.3	06-Jul-92	213.4	223.2	50	18	3	378.50	4105	9.8	18.8767	GOOD TEST

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			TOP	BOTTOM								
C-80	19.3	06-Jul-92	223.2	233.3	52	0	3	0.00	4246	10.1	0.0000	GOOD TEST
C-80	30.7	07-Jul-92	233.3	272.6	20	114	3	2397.17	2343	39.3	71.1285	GOOD TEST
C-78	25.6	29-Apr-91	150	291.7	60	165	3	3469.59	5001	141.7	16.6544	GOOD TEST
C-78	25.8	29-Apr-91	200	291.7	60	70	3	1471.95	5007	91.7	10.1761	GOOD TEST
C-77	25.3	08-May-91	140	281.7	60	45	3	946.25	4991	141.7	4.5504	GOOD TEST
C-77	25.3	08-May-91	190	281.7	60	17	3	357.47	4991	91.7	2.4789	GOOD TEST
C-75	24.3	21-Jun-91	182.2	201.9	50	0	6	0.00	4258	19.7	0.0000	GOOD TEST COMPILATION OF 3 ZERO TAKE TESTS.
C-75	24.3	22-Jun-91	182.4	285.9	50	74	3	1556.06	4258	103.5	11.4313	GOOD TEST
C-75	24.3	21-Jun-91	212	221.8	50	3	3	63.08	4258	9.8	3.0335	GOOD TEST
C-75	24.3	21-Jun-91	221.7	231.8	50	29	3	609.81	4258	10.1	28.6761	GOOD TEST
C-75	24.3	21-Jun-91	232.1	241.5	50	15	3	315.42	4258	9.4	15.6414	GOOD TEST
C-75	24.3	22-Jun-91	242.1	250.9	50	75	3	1577.09	4258	8.8	82.0898	GOOD TEST
C-75	24.3	22-Jun-91	252.1	261.5	50	17	3	357.47	4258	9.4	17.7270	GOOD TEST
C-75	24.3	22-Jun-91	262.1	271.6	50	7	3	147.19	4258	9.5	7.2426	GOOD TEST
C-75	24.3	22-Jun-91	272.2	285.9	50	0	3	0.00	4258	13.7	0.0000	GOOD TEST
DC-76	25	17-Nov-92	23.2	32.9	50	35	3	735.97	4279	9.7	35.4824	GOOD TEST, NOTE ON LOG SUGGESTS PERCHED WATER IN THE OVB
DC-76	25	17-Nov-92	32.9	42.5	50	42	3	883.17	4279	9.6	42.9061	GOOD TEST
DC-76	25	17-Nov-92	42.5	53	50	29	3	609.81	4279	10.5	27.7214	GOOD TEST
DC-76	25	17-Nov-92	53	62.7	50	19	3	399.53	4279	9.7	19.2619	GOOD TEST
DC-76	29.8	18-Nov-92	62.7	72.9	50	22	3	462.61	4425	10.2	20.7777	GOOD TEST
DC-76	29.8	18-Nov-92	72.9	83	50	144	3	3028.01	4425	10.1	136.9975	TEST QUESTIONED BUT 0.4' LOSS IN CORE - OPEN HORIZ.?
DC-76	29.8	18-Nov-92	83	92.9	50	15	3	315.42	4425	9.9	14.4837	GOOD TEST
DC-76	29.8	18-Nov-92	92.9	103	50	2	3	42.06	4425	10.1	1.9027	GOOD TEST
DC-76	29.8	18-Nov-92	103	112.9	50	7	3	147.19	4425	9.9	6.7591	GOOD TEST
DC-76	29.8	18-Nov-92	112.9	122.9	50	0	3	0.00	4425	10	0.0000	GOOD TEST
DC-76	29.8	18-Nov-92	122.9	132.9	50	28	3	588.78	4425	10	26.8357	GOOD TEST
DC-76	29.8	19-Nov-92	132.9	143.1	50	0	3	0.00	4425	10.2	0.0000	GOOD TEST
DC-76	29.8	19-Nov-92	153.1	163	50	5	3	105.14	4425	9.9	4.8279	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
DC-76	29.8	19-Nov-92	163	173	50	0	3	0.00	4425	10	0.0000	GOOD TEST
DC-76	29.8	19-Nov-92	173	182.9	50	24	3	504.67	4425	9.9	23.1739	GOOD TEST
DC-76	29.8	19-Nov-92	182.9	192.9	50	37	3	778.03	4425	10	35.4614	GOOD TEST
DC-76	29.8	19-Nov-92	192.9	203	50	82	3	1724.28	4425	10.1	78.0125	GOOD TEST
DC-76	32.7	20-Nov-92	203	213	50	0	3	0.00	4514	10	0.0000	GOOD TEST
DC-76	32.7	20-Nov-92	213	223	50	0	3	0.00	4514	10	0.0000	GOOD TEST
DC-76	32.7	20-Nov-92	223	232.6	50	0.5	3	10.51	4514	9.6	0.4842	GOOD TEST
DC-76	32.7	20-Nov-92	232.6	242.8	50	93	3	1955.59	4514	10.2	86.1130	GOOD TEST
DC-76	32.7	20-Nov-92	242.8	253	50	218	3	4584.07	4514	10.2	201.8562	GOOD TEST
DC-76	38.9	21-Nov-92	253	262.9	50	26	3	546.72	4703	9.9	23.6243	GOOD TEST
DC-76	38.9	21-Nov-92	262.9	273	50	0	3	0.00	4703	10.1	0.0000	GOOD TEST
DC-76	38.9	21-Nov-92	273	283	50	2	3	42.06	4703	10	1.8038	GOOD TEST
DC-76	38.9	21-Nov-92	283	292	50	11	3	231.31	4703	9	10.7224	GOOD TEST
DC-74	1	04-Dec-92	15	32.1	16	132	3	2775.67	1156	17.1	322.5755	COULD NOT GET MORE THAN 16 PSI PRESSURE. PACKER SEAL GOOD
DC-74	1	04-Dec-92	32.1	42.2	16	85	3	1787.37	1156	10.1	309.5895	GOOD TEST
DC-74	1	04-Dec-92	42.2	52.1	20	162	3	3406.51	1437	9.9	481.6219	GOOD TEST
DC-74	1	04-Dec-92	52	61.9	42	28	3	588.78	2985	9.9	40.0849	GOOD TEST
DC-74	1	04-Dec-92	61.7	71.7	34	133	3	2796.70	2422	10	232.9001	GOOD TEST
DC-74	1	04-Dec-92	71.5	81.5	32	138	3	2901.84	2281	10	256.5574	GOOD TEST
DC-74	1	04-Dec-92	81.5	91.7	48	151	3	3175.20	3407	10.2	185.2466	GOOD TEST
DC-74	46	08-Dec-92	91.4	101.8	50	125	3	2628.48	4919	10.4	104.6832	GOOD TEST
DC-74	46	08-Dec-92	101.5	112	50	73	3	1535.03	4919	10.5	60.7013	GOOD TEST
DC-74	46	08-Dec-92	111.7	122.2	50	0	3	0.00	4919	10.5	0.0000	GOOD TEST
DC-74	46	08-Dec-92	121.9	131.9	50	22	3	462.61	4919	10	18.9686	GOOD TEST
DC-74	46	08-Dec-92	131.7	141.8	50	79	3	1661.20	4919	10.1	67.6139	GOOD TEST
DC-74	46.5	09-Dec-92	141.6	151.6	50	0	3	0.00	4934	10	0.0000	GOOD TEST
DC-74	46.5	09-Dec-92	151.7	161.6	50	0	3	0.00	4934	9.9	0.0000	GOOD TEST
DC-74	46.5	09-Dec-92	161.5	171.7	50	8	3	168.22	4934	10.2	6.7761	GOOD TEST

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			TOP	BOTTOM								
DC-74	46.5	09-Dec-92	171.5	181.7	50	3	3	63.08	4934	10.2	2.5410	GOOD TEST
DC-74	46	10-Dec-92	181.7	192	50	0	3	0.00	4919	10.3	0.0000	GOOD TEST
DC-74	46	10-Dec-92	191.8	202	50	16	3	336.45	4919	10.2	13.5942	GOOD TEST
DC-74	46	10-Dec-92	201.9	212.1	50	24	3	504.67	4919	10.2	20.3913	GOOD TEST
DC-74	46	10-Dec-92	211.7	222.1	50	24	3	504.67	4919	10.4	20.0992	GOOD TEST
DC-74	46	10-Dec-92	221.6	232	50	0	3	0.00	4919	10.4	0.0000	GOOD TEST
DC-74	46	14-Dec-92	232.1	242.1	50	3	3	63.08	4919	10	2.5866	GOOD TEST
DC-74	46	14-Dec-92	242.1	251.9	50	16	3	336.45	4919	9.8	14.0033	GOOD TEST
DC-74	46	14-Dec-92	251.9	262.1	26	13	3	273.36	3231	10.2	16.8164	GOOD TEST
DC-74	46	14-Dec-92	262.1	271.7	50	117	3	2460.26	4919	9.6	103.9713	GOOD TEST
DC-74	44.4	15-Dec-92	271.7	281.7	50	7	3	147.19	4870	10	6.0959	GOOD TEST
DC-74	44.4	15-Dec-92	281.7	292	50	0	3	0.00	4870	10.3	0.0000	GOOD TEST
C-73	27.8	22-Feb-92	162	171.3	50	0	3	0.00	4364	9.3	0.0000	GOOD TEST
C-73	27.8	22-Feb-92	172	181.5	50	0	3	0.00	4364	9.5	0.0000	GOOD TEST
C-73	27.8	22-Feb-92	181.7	191.5	50	0	3	0.00	4364	9.8	0.0000	GOOD TEST
C-73	27.8	22-Feb-92	192	201.5	50	3	3	63.08	4364	9.5	3.0281	GOOD TEST
C-73	27.3	24-Feb-92	201.9	211.5	50	29	3	609.81	4349	9.6	29.1481	GOOD TEST
C-73	32.1	26-Feb-92	211.6	261.5	50	50	3	1051.39	4495	49.9	13.3877	GOOD TEST
C-72	34.7	02-Mar-92	151.8	172	50	55	3	1156.53	4575	20.2	29.8384	GOOD TEST
C-72	35	03-Mar-92	171.7	181.8	50	3	3	63.08	4584	10.1	2.7554	GOOD TEST
C-72	34.7	03-Mar-92	181.8	192	50	2	3	42.06	4575	10.2	1.8272	GOOD TEST
C-72	34.7	03-Mar-92	193.2	202.1	50	0	3	0.00	4575	8.9	0.0000	GOOD TEST
C-72	34.7	04-Mar-92	203	252	50	98	3	2060.73	4575	49	26.1715	GOOD TEST
C-71	8.6	17-Oct-91	152.4	162.8	50	4	3	84.11	3779	10.4	4.3603	GOOD TEST
C-71	8.6	17-Oct-91	162.4	171.9	50	7	3	147.19	3779	9.5	8.1597	GOOD TEST
C-71	8.6	18-Oct-91	171.7	180.7	50	10	3	210.28	3779	9	12.1298	GOOD TEST
C-71	8.6	18-Oct-91	180.4	190.7	50	8	3	168.22	3779	10.3	8.7836	GOOD TEST
C-71	8.6	18-Oct-91	190.4	200.6	50	0	3	0.00	3779	10.2	0.0000	GOOD TEST

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			TOP	BOTTOM								
C-71	8.6	19-Oct-91	200.4	231.3	50	4	3	84.11	3779	30.9	1.8771	GOOD TEST
C-70	7.6	04-Oct-91	142.8	152.7	50	0	3	0.00	3749	9.9	0.0000	GOOD TEST
C-70	7.6	04-Oct-91	152.4	162.4	50	0	3	0.00	3749	10	0.0000	GOOD TEST
C-70	7.6	04-Oct-91	162	172.8	50	25	3	525.70	3749	10.8	26.7121	GOOD TEST
C-70	7.6	05-Oct-91	172.6	183	50	53	3	1114.48	3749	10.4	58.2444	GOOD TEST
C-70	7.6	05-Oct-91	182.7	193.2	50	13	3	273.36	3749	10.5	14.1850	GOOD TEST, DWR = 50%
C-70	7.6	05-Oct-91	193	203	50	0	3	0.00	3749	10	0.0000	GOOD TEST
C-70	7.6	07-Oct-91	202.8	233	50	4	3	84.11	3749	30.2	1.9274	GOOD TEST
C-69	6.6	28-Oct-91	142.1	151.7	50	5	3	105.14	3718	9.6	5.8783	GOOD TEST
C-69	6.6	28-Oct-91	152	161.7	50	19	3	399.53	3718	9.7	22.1673	GOOD TEST
C-69	6.6	28-Oct-91	161.5	171.9	50	7	3	147.19	3718	10.4	7.7557	GOOD TEST
C-69	6.6	28-Oct-91	171.6	182.1	50	0	3	0.00	3718	10.5	0.0000	GOOD TEST
C-69	6.6	29-Oct-91	191.1	202.5	50	0	3	0.00	3718	11.4	0.0000	GOOD TEST
C-69	6.6	29-Oct-91	202.3	242.4	50	0	3	0.00	3718	40.1	0.0000	GOOD TEST
C-68	21.5	06-Nov-91	152.6	192.7	40	0	12	0.00	3469	40.1	0.0000	GOOD TEST COMPOSITE OF 4 TESTS, NO TAKE.
C-67	9	13-Nov-91	152.6	192.7	35	0	12	0.00	2736	40.1	0.0000	GOOD TEST COMPILED 4 0 TAKE TESTS FOR THIS RECORD.
C-67	9	14-Nov-91	192.4	202.9	40	12	3	252.33	3088	10.5	15.8956	GOOD TEST
C-67	20	15-Nov-91	202.6	212.7	40	0	3	0.00	3423	10.1	0.0000	GOOD TEST
C-64	20.1	10-Dec-91	172.7	182.2	50	0	3	0.00	4130	9.5	0.0000	GOOD TEST
C-64	20.1	10-Dec-91	182.4	192.5	50	45	3	946.25	4130	10.1	45.8768	GOOD TEST
C-64	201	10-Dec-91	192.4	202.3	50	0	3	0.00	9643	9.9	0.0000	GOOD TEST
C-64	20	11-Dec-91	202.2	212.2	50	0	3	0.00	4127	10	0.0000	GOOD TEST
C-63	22.3	09-Dec-91	172.6	192	32	207	3	4352.76	2931	19.4	180.9158	GOOD TEST
C-63	22.3	09-Dec-91	182.4	192	50	200	3	4205.57	4197	9.6	208.3215	GOOD TEST
C-63	25.3	10-Dec-91	192	202	50	0	3	0.00	4288	10	0.0000	GOOD TEST
C-63	25.3	10-Dec-91	202	212.1	50	38	3	799.06	4288	10.1	37.3085	GOOD TEST
C-63	25.2	10-Dec-91	212.1	222.1	50	20	3	420.56	4285	10	19.7955	GOOD TEST
C-63	25.3	10-Dec-91	222.1	231.6	50	9	3	189.25	4288	9.5	9.2457	GOOD TEST

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			TOP	BOTTOM								
C-63	25.3	10-Dec-91	222.1	231.6	50	9	3	189.25	4288	9.5	9.2457	GOOD TEST
C-63	24.8	11-Dec-91	231.6	242.1	50	23	3	483.64	4273	10.5	22.0173	GOOD TEST
C-63	24.8	11-Dec-91	242.1	282.1	50	36	3	757.00	4273	40	12.1395	GOOD TEST
DC-58	0	06-Sep-94	32.6	42.6	30	151	3	3175.20	2110	10	303.4958	GOOD TEST
DC-58	0	06-Sep-94	42.6	52.8	50	63	3	1324.75	3517	10.2	74.8666	GOOD TEST
DC-58	0	06-Sep-94	63.8	73.8	19	232	3	4878.46	1336	10	736.2604	PUMPING AT MAXIMUM CAPACITY
DC-58	0	06-Sep-94	72.8	82.8	15	177	3	3721.93	1055	10	711.5068	PUMPING AT MAXIMUM CAPACITY
DC-58	0	06-Sep-94	82.8	92.5	52	7	3	147.19	3658	9.7	8.3020	GOOD TEST
DC-58	56.4	07-Sep-94	92.5	102.8	56	16	3	336.45	5658	10.3	11.7333	GOOD TEST
DC-58	56.4	07-Sep-94	102.8	112.7	46	134	3	2817.73	4955	9.9	115.5622	PUMPING AT MAXIMUM RATE
DC-58	56.4	07-Sep-94	112.7	122.8	52	56	3	1177.56	5377	10.1	43.8491	GOOD TEST
DC-58	56.4	07-Sep-94	122.8	132.8	54	94	3	1976.62	5517	10	72.2582	GOOD TEST
DC-58	56.4	07-Sep-94	132.8	142.7	51	133	3	2796.70	5306	9.9	107.0978	GOOD TEST
DC-58	76.3	08-Sep-94	142.7	152.4	17	188	3	3953.23	3521	9.7	231.5929	PUMPING AT MAXIMUM CAPACITY
DC-58	76.3	08-Sep-94	152.4	162.3	57	95	3	1997.64	6335	9.9	64.0776	GOOD TEST
DC-58	76.3	08-Sep-94	162.3	182.3	50	0	3	0.00	5843	20	0.0000	COMBINATION OF 2 TESTS
DC-58	72.3	09-Sep-94	182.3	192.9	32	176	3	3700.90	4455	10.6	160.4711	PUMPING AT MAXIMUM CAPACITY
DC-58	72.3	09-Sep-94	192.9	202.9	50	42	4	662.38	5721	10	23.3538	GOOD TEST
DC-58	72.3	09-Sep-94	202.9	212.8	52	3	3	63.08	5861	9.9	2.1870	GOOD TEST
DC-58	72.3	09-Sep-94	212.8	222.9	51	42	3	883.17	5791	10.1	30.5340	GOOD TEST
DC-58	72.3	09-Sep-94	222.9	232.9	50	9	3	189.25	5721	10	6.6725	GOOD TEST
DC-58	75.6	10-Sep-94	232.9	242.8	55	5	3	105.14	6173	9.9	3.4610	GOOD TEST
DC-58	75.6	10-Sep-94	242.8	252.8	53	0	3	0.00	6032	10	0.0000	GOOD TEST
DC-58	75.6	10-Sep-94	252.8	262.8	54	23	3	483.64	6103	10	15.9847	GOOD TEST
DC-58	77.8	12-Sep-94	262.8	272.8	57	102	3	2144.84	6381	10	67.7994	GOOD TEST
DC-58	77.8	12-Sep-94	272.8	282.8	51	1	3	21.03	5959	10	0.7118	GOOD TEST
DC-58	77.8	12-Sep-94	282.8	332.8	50	0	3	0.00	5888	50	0.0000	COMBINATION OF SEVERAL TESTS
DC-58	0	06-Sep-94	525.8	62.8	53	0	3	0.00	3728	-463	ERR	GOOD TEST

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			TOP	BOTTOM								
C-62	12.5	13-Nov-91	181.7	191.7	50	0	3	0.00	3898	10	0.0000	GOOD TEST
C-62	12.5	13-Nov-91	191.7	201.9	50	0	3	0.00	3898	10.2	0.0000	GOOD TEST
C-62	12.5	13-Nov-91	201.9	211.9	50	50	5	630.84	3898	10	32.6421	GOOD TEST
C-62	12.5	13-Nov-91	211.9	221.9	40	247	5	3116.32	3195	10	196.7569	GOOD TEST
C-62	13.5	14-Nov-91	221.9	231.6	50	0	3	0.00	3928	9.7	0.0000	GOOD TEST
C-62	13.5	14-Nov-91	231.6	240.9	50	14	3	294.39	3928	9.3	15.9471	GOOD TEST
C-62	13.5	14-Nov-91	240.9	251	50	48	3	1009.34	3928	10.1	51.4412	GOOD TEST
C-62	13.5	14-Nov-91	251	261.2	50	1	3	21.03	3928	10.2	1.0639	GOOD TEST
C-62	14.5	15-Nov-91	261.2	280.9	50	0	3	0.00	3959	19.7	0.0000	GOOD TEST
C-61	6.5	27-Jan-92	211.9	222	46	0	3	0.00	3434	10.1	0.0000	GOOD TEST
C-61	6	28-Jan-92	221.9	231.7	50	0	3	0.00	3700	9.8	0.0000	GOOD TEST
C-61	6	28-Jan-92	230.8	241.7	50	26	3	546.72	3700	10.9	27.9538	GOOD TEST
C-61	6.6	29-Jan-92	240.8	252	50	45	3	946.25	3718	11.2	47.1762	GOOD TEST
C-61	6.6	29-Jan-92	251.2	261.9	50	0	3	0.00	3718	10.7	0.0000	GOOD TEST
C-61	6.6	29-Jan-92	261	272	50	12	3	252.33	3718	11	12.7510	GOOD TEST
C-61	6.6	29-Jan-92	262.7	301.7	50	45	3	946.25	3718	39	17.7993	GOOD TEST
C-60A	12.1	14-Feb-92	203.7	211.7	50	0	3	0.00	3886	8	0.0000	GOOD TEST
C-60A	12.1	14-Feb-92	213.6	221.7	50	3	3	63.08	3886	8.1	3.8220	GOOD TEST
C-60A	12.1	14-Feb-92	223.3	231.7	50	0	3	0.00	3886	8.4	0.0000	GOOD TEST
C-60A	12.1	14-Feb-92	232.2	241.7	50	0	3	0.00	3886	9.5	0.0000	GOOD TEST
C-60A	12.1	15-Feb-92	242	291.7	50	27	3	567.75	3886	49.7	8.3910	GOOD TEST
C-59	62.4	06-May-92	281.8	291.9	50	10	3	210.28	5419	10.1	7.7692	GOOD TEST
C-59	62.4	07-May-92	291.9	301.9	50	11	3	231.31	5419	10	8.6094	GOOD TEST
C-59	62.4	07-May-92	301.9	311.9	43	10	3	210.28	4927	10	8.6090	GOOD TEST
C-59	62.4	07-May-92	311.9	321.9	50	8	3	168.22	5419	10	6.2614	GOOD TEST
C-59	62.4	09-May-92	321.9	361.9	50	4	3	84.11	5419	40	1.0636	GOOD TEST
DC-53	5.1	10-Jun-94	53.4	63.1	48	0	3	0.00	3532	9.7	0.0000	GOOD TEST
DC-53	5.7	11-Jun-94	63.1	73.3	49	1	3	21.03	3620	10.2	1.1544	GOOD TEST

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			TOP	BOTTOM								
DC-53	5.7	11-Jun-94	73.3	83.3	57	4	3	84.11	4183	10	4.0556	GOOD TEST
DC-53	5.7	11-Jun-94	83.3	93.5	54	3	3	63.08	3972	10.2	3.1566	GOOD TEST
DC-53	5.7	11-Jun-94	93.5	103.5	50	0	3	0.00	3691	10	0.0000	GOOD TEST
DC-53	1.2	13-Jun-94	103.5	123.3	50	0	3	0.00	3553	19.8	0.0000	GOOD TEST
DC-53	-2	13-Jun-94	123.3	133.5	52	89	3	1871.48	3597	10.2	103.4198	ARTESIAN FLOW
DC-53	-2	13-Jun-94	133.5	143.5	54	0	3	0.00	3737	10	0.0000	GOOD TEST
DC-53	-2	13-Jun-94	143.5	153.4	42	147	3	3091.09	2893	9.9	217.0970	ARTESIAN FLOW
DC-53	-2	14-Jun-94	153.4	163.5	50	9	3	189.25	3456	10.1	10.9638	ARTESIAN
DC-53	-2	14-Jun-94	163.5	173.3	50	21	3	441.58	3456	9.8	26.1599	GOOD TEST
DC-53	-2	14-Jun-94	173.3	193.4	50	0	3	0.00	3456	20.1	0.0000	ARTESIAN
DC-53	-2	15-Jun-94	193.4	203.4	50	13	3	273.36	3456	10	15.9538	GOOD TEST
DC-53	-2	15-Jun-94	203.4	303.1	50	0	3	0.00	3456	99.7	0.0000	COMBINATION OF 3 DAYS TESTS IN ARTESIAN HOLE.
DC-48	2.6	03-Oct-94	58.5	63.1	40	40	3	841.11	2893	4.6	101.8602	GOOD TEST
DC-48	2.6	03-Oct-94	63	73.1	48	33	3	693.92	3455	10.1	40.2059	GOOD TEST
DC-48	2.6	03-Oct-94	83	93.1	50	18	3	378.50	3596	10.1	21.0726	GOOD TEST
DC-48	10.1	04-Oct-94	93	103.1	42	130	3	2733.62	3262	10.1	167.7787	GOOD TEST
DC-48	10.1	04-Oct-94	103	113.1	45	71	3	1492.98	3473	10.1	86.0656	GOOD TEST
DC-48	10.1	04-Oct-94	113.1	123.1	50	29	3	609.81	3825	10	32.1575	GOOD TEST
DC-48	10.1	04-Oct-94	123	133.1	50	51	3	1072.42	3825	10.1	56.1372	GOOD TEST
DC-48	9	05-Oct-94	133	163.1	50	0	3	0.00	3791	30.1	0.0000	COMBINED 3 TESTS
DC-48	9.3	06-Oct-94	163	401.1	50	0	3	0.00	3800	238.1	0.0000	COMBINED SEVERAL TESTS OVER SEVERAL DAYS TIGHT HOLE
DC-42	22.5		42	52	50	150	3	3154.18	4203	10	151.3737	GOOD TEST
DC-42	22.5		52	62	53	156	3	3280.34	4414	10	149.9022	GOOD TEST
DC-42	22.5		62	72	53	51	3	1072.42	4414	10	49.0065	GOOD TEST
DC-42	22.5		72	82	56	0	3	0.00	4625	10	0.0000	GOOD TEST
DC-42	22.5		82	92	52	30	3	630.84	4343	10	29.2942	GOOD TEST
DC-42	22.5		92	102	52	39	3	820.09	4343	10	38.0824	GOOD TEST
DC-42	22.5		102	112	50	9	3	189.25	4203	10	9.0824	GOOD TEST

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			TOP	BOTTOM								
DC-42	22.5		112	122	50	0	3	0.00	4203	10	0.0000	GOOD TEST
DC-42	22.5		122	132	50	81	3	1703.25	4203	10	81.7418	GOOD TEST
DC-42	22.5		132	142	32	216	3	4542.01	2937	10	311.9567	GOOD TEST
DC-42	22.5		142	152	48	75	3	1577.09	4062	10	78.3080	GOOD TEST
DC-42	22.5		152	162	53	60	3	1261.67	4414	10	57.6547	GOOD TEST
DC-42	22.5		162	182	52	0	3	0.00	4343	20	0.0000	GOOD TEST
DC-42	22.5		182	192	52	84	3	1766.34	4343	10	82.0237	GOOD TEST
DC-42	22.5		192	202	50	3	3	63.08	4203	10	3.0275	GOOD TEST
DC-42	22.5		202	222	50	0	3	0.00	4203	20	0.0000	GOOD TEST
DC-42	22.5		222	232	52	12	3	252.33	4343	10	11.7177	GOOD TEST
DC-42	22.5		232	262	50	0	3	0.00	4203	30	0.0000	GOOD TEST
DC-42	22.5		262	272	50	9	3	189.25	4203	10	9.0824	GOOD TEST
DC-42	22.5		272	322	50	0	3	0.00	4203	50	0.0000	GOOD TEST
DC-42	22.5		322	332	52	3	3	63.08	4343	10	2.9294	GOOD TEST
DC-42	22.5		332	342	54	3	3	63.08	4484	10	2.8375	GOOD TEST
DC-42	22.5		342	382	50	0	3	0.00	4203	40	0.0000	GOOD TEST
DC-42	22.5		382	392	50	6	3	126.17	4203	10	6.0549	GOOD TEST
DC-42	22.5		392	402	61	0	3	0.00	4976	10	0.0000	GOOD TEST
DC-42	22.5		402	412.5	50	3	3	63.08	4203	10.5	2.9197	GOOD TEST
DC-42	22.5		412.5	422.8	54	0	3	0.00	4484	10.3	0.0000	GOOD TEST
DC-41	49.7	18-Oct-94	93.2	103.2	50	9	3	189.25	5032	10	7.5860	GOOD TEST
DC-41	49.7	18-Oct-94	103.2	113.2	50	6	3	126.17	5032	10	5.0573	GOOD TEST
DC-41	49.7	18-Oct-94	113.1	123.6	50	100	3	2102.78	5032	10.5	81.2887	GOOD TEST
DC-41	49.7	18-Oct-94	123.6	133.6	50	15	3	315.42	5032	10	12.6433	GOOD TEST
DC-41	49.7	18-Oct-94	133.6	143.6	50	3	3	63.08	5032	10	2.5287	GOOD TEST
DC-41	49.7	19-Oct-94	143.6	153.6	50	130	3	2733.62	5032	10	109.5751	GOOD TEST
DC-41	49.7	19-Oct-94	153.6	163.6	50	30	3	630.84	5032	10	25.2866	GOOD TEST
DC-41	49.7	19-Oct-94	163.6	173.6	50	16	4	252.33	5032	10	10.1146	GOOD TEST

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			TOP	BOTTOM								
DC-41	49.7	19-Oct-94	173.6	182.2	50	7	3	147.19	5032	8.6	6.5928	GOOD TEST
DC-41	49.7	20-Oct-94	193.3	203.6	50	10	3	210.28	5032	10.3	8.2460	GOOD TEST
DC-41	49.7	20-Oct-94	203.6	213.7	50	26	3	546.72	5032	10.1	21.7539	GOOD TEST
DC-41	49.7	20-Oct-94	213.7	233.7	50	0	3	0.00	5032	20	0.0000	GOOD TEST
DC-41	49.7	21-Oct-94	233.7	243.7	50	7	3	147.19	5032	10	5.9002	GOOD TEST
DC-41	49.7	21-Oct-94	243.7	253.7	50	8	3	168.22	5032	10	6.7431	GOOD TEST
DC-41	49.7	21-Oct-94	253.7	263.7	50	18	3	378.50	5032	10	15.1719	GOOD TEST
DC-41	49.7	21-Oct-94	263.7	273.7	50	27	3	567.75	5032	10	22.7579	GOOD TEST
DC-41	49.7	21-Oct-94	273.7	433	50	0	3	0.00	5032	159.3	0.0000	QUESTIONABLE DATA BASED ON DRILLERS MEMORY (GEOLOGIST Q
DC-38	19.9	10-May-94	48.8	63.3	50	50	4	788.54	4123	14.5	29.1593	GOOD TEST
DC-38	20.1	11-May-94	63.3	73.3	50	0	3	0.00	4130	10	0.0000	GOOD TEST
DC-38	20.1	11-May-94	73.3	83.4	50	6	3	126.17	4130	10.1	6.1169	GOOD TEST
DC-38	18.7	13-May-94	83.4	92.6	50	0	3	0.00	4087	9.2	0.0000	GOOD TEST
DC-38	19.6	14-May-94	92.6	103	50	93	3	1955.59	4114	10.4	93.1167	GOOD TEST
DC-38	19.6	14-May-94	103	113.5	50	0	3	0.00	4114	10.5	0.0000	GOOD TEST
DC-38	19.6	14-May-94	113.5	123.4	50	7	3	147.19	4114	9.9	7.2698	GOOD TEST
DC-38	20.8	16-May-94	123.4	133.4	50	0	3	0.00	4151	10	0.0000	GOOD TEST
DC-38	20.8	16-May-94	133.4	143.6	50	22.5	3	473.13	4151	10.2	22.6543	GOOD TEST
DC-38	21.9	17-May-94	143.6	153.2	50	2	4	31.54	4184	9.6	1.5670	GOOD TEST
DC-38	21.4	18-May-94	153.2	173.4	50	0	3	0.00	4169	20.2	0.0000	GOOD TEST
DC-38	21.4	18-May-94	173.4	183.4	50	7	3	147.19	4169	10	7.1209	GOOD TEST
DC-38	21.3	21-May-94	183.4	193.2	50	3	3	63.08	4166	9.8	3.1001	GOOD TEST
DC-38	21.3	21-May-94	193.2	203.3	50	9	3	189.25	4166	10.1	9.0948	GOOD TEST
DC-38	21.3	21-May-94	203.3	213.3	50	8	3	168.22	4166	10	8.1441	GOOD TEST
DC-38	21.3	21-May-94	213.3	253.4	50	0	3	0.00	4166	40.1	0.0000	GOOD TEST
DC-38	22.2	23-May-94	253.4	263.7	50	125	3	2628.48	4194	10.3	123.6769	GOOD TEST
DC-38	22.2	23-May-94	263.7	303.7	50	0	3	0.00	4194	40	0.0000	COMBINED SEVERAL TESTS
DC-38	22.7	24-May-94	303.7	311.5	50	4	3	84.11	4209	7.8	4.8353	GOOD TEST

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			TOP	BOTTOM								
DC-38	23.2	25-May-94	311.5	371.7	50	0	3	0.00	4224	60.2	0.0000	COMBINED SEVERAL TESTS
DC-38	24.7	26-May-94	371.7	381.4	50	3	3	63.08	4270	9.7	3.0479	GOOD TEST
DC-38	24.7	26-May-94	381.4	391.4	50	1	3	21.03	4270	10	0.9933	GOOD TEST
C-28	23.7	19-May-92	422.6	502.4	46	33	3	693.92	3958	79.8	6.8140	GOOD TEST
C-28	23.7	19-May-92	462.6	502.4	50	31	3	651.86	4239	39.8	10.5790	GOOD TEST
C-28	23.7	19-May-92	482.8	502.4	50	0	3	0.00	4239	19.6	0.0000	GOOD TEST
C-26	20	04-Apr-91	311.6	441.6	50	9999	1	*******	4127	130	3946.4046	COULD NOT GET PACKER TOP SEAL ON FRACTURED SIDES OF HOLE.
DC-24	22.5	21-Oct-92	373	383	50	0	3	0.00	4203	10	0.0000	GOOD TEST
DC-24	22.5	21-Oct-92	383	392.8	50	0	3	0.00	4203	9.8	0.0000	GOOD TEST
DC-24	22.5	21-Oct-92	392.8	403	50	1	3	21.03	4203	10.2	0.9944	GOOD TEST
DC-24	22.5	21-Oct-92	403	413	50	1	3	21.03	4203	10	1.0092	GOOD TEST
DC-24	22.5	22-Oct-92	413	451.3	50	4	3	84.11	4203	38.3	1.4204	GOOD TEST
DC-23	11.1	09-Apr-94	183.8	193.1	22	152	3	3196.23	1886	9.3	360.6815	GOOD TEST
DC-23	11.1	11-Apr-94	193.1	223.1	50	0	3	0.00	3855	30	0.0000	GOOD TEST
DC-23	11.1	14-Apr-94	223.1	263	50	0	3	0.00	3855	39.9	0.0000	GOOD TEST
DC-23	11.4	15-Apr-94	263	272.8	50	0	3	0.00	3864	9.8	0.0000	GOOD TEST
DC-23	11.4	15-Apr-94	272.8	283.2	50	3	3	63.08	3864	10.4	3.1980	GOOD TEST
DC-23	11	18-Apr-94	283.2	293.1	50	33	3	693.92	3852	9.9	36.6040	GOOD TEST
DC-23	11	18-Apr-94	293.1	303.1	50	0	3	0.00	3852	10	0.0000	GOOD TEST
DC-23	11	18-Apr-94	303.1	313.1	50	4	3	84.11	3852	10	4.4039	GOOD TEST
DC-23	11	18-Apr-94	313.1	322.8	50	54	3	1135.50	3852	9.7	60.8085	GOOD TEST
DC-23	13	19-Apr-94	322.8	332.8	50	0	3	0.00	3913	10	0.0000	GOOD TEST
DC-23	13	19-Apr-94	332.8	343.1	50	2	3	42.06	3913	10.3	2.1206	GOOD TEST
DC-23	12.7	20-Apr-94	343.1	363.1	50	0	3	0.00	3904	20	0.0000	GOOD TEST
DC-23	12.7	20-Apr-94	363.1	373.4	50	62	3	1303.73	3904	10.3	65.8936	GOOD TEST
DC-23	12.7	20-Apr-94	373.4	383.3	50	0	3	0.00	3904	9.9	0.0000	GOOD TEST
DC-23	12	21-Apr-94	383.3	391.9	50	10	3	210.28	3883	8.6	12.2057	GOOD TEST
DC-23	12	21-Apr-94	391.9	413.7	50	0	3	0.00	3883	21.8	0.0000	GOOD TEST

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			TOP	BOTTOM								
DC-23	12	21-Apr-94	413.7	423.7	50	108	3	2271.01	3883	10	117.9728	POSSIBLE PACKER MALFUNCTION (DRILLER COMMENT)
DC-23	18.1	22-Apr-94	423.7	433.9	50	130	3	2733.62	4069	10.2	133.5389	POSSIBLE PACKER MALFUNCTION (DRILLER COMMENT)
DC-23	18.1	22-Apr-94	433.9	473.1	50	0	3	0.00	4069	39.2	0.0000	GOOD TEST
DC-23	17.2	23-Apr-94	473.1	503.6	50	0	3	0.00	4041	30.5	0.0000	GOOD TEST
PR-5CA	20.1	07-Aug-90	282.5	492.2	20	424	10	2674.74	2019	209.7	22.7759	TESTED INSIDE CASING. PROB LEAKAGE @ BOT OF CASING.
PR-5CA	20.1	06-Aug-90	400.2	492.2	50	1	5	12.62	4130	92	0.1055	GOOD TEST
PR-6A	23	27-Jul-90	261	401.5	20	22.5	10	141.94	2108	140.5	1.6280	GOOD TEST
PR-6A	23	27-Jul-90	301	401.5	35	13	5	164.02	3163	100.5	1.6624	PUMP SURGING FROM 20 TO 50 PSI.
PR-7	23	23-Aug-90	171.3	401.8	17	348	7	3136.15	1897	230.5	26.2199	GOOD TEST
PR-7	23	23-Aug-90	317	401.8	30	247	4	3895.41	2811	84.8	51.1979	HIGH ANGLE BREAKS + ZONE @ 370
PR-7	23	23-Aug-90	327	401.8	30	296	5	3734.54	2811	74.8	54.4820	INTENSELY FRACTURED ZONE @ 370 ONLY PROB TAKE AREA.
PR-4CB	15.1	14-Jul-90	153	404.2	10	17.8	5	224.58	1164	251.2	2.8429	GOOD TEST
PR-4CB	15.1	14-Jul-90	270	404.2	40	15.2	5	191.77	3274	134.2	1.4723	GOOD TEST
PR-4CB	15.1	14-Jul-90	277	404.2	50	0.16	3	3.36	3977	127.2	0.0222	GOOD TEST
PR-4CB	15.1	14-Jul-90	277	404.2	45	1.05	1	66.24	3625	127.2	0.4804	GOOD TEST
C-19	23	11-Mar-91	290	430.9	60	122	3	2565.40	4921	140.9	12.5725	WATER LOST @ 251.7 IN 0.1' CAVITY. VERT FRAC @ 3385
C-19	23	11-Mar-91	340	430.9	70	110	3	2313.06	5625	90.9	14.3386	PUMP SURGING FROM 60 TO 80 PSI.
DC-15	6.2	10-Nov-94	93.6	103.6	45	13.5	3	283.88	3354	10	17.0700	GOOD TEST
DC-15	6.2	11-Nov-94	103.6	113.3	46	0	3	0.00	3425	9.7	0.0000	GOOD TEST
DC-15	6.2	11-Nov-94	113.3	123.6	50	6	3	126.17	3706	10.3	6.7177	GOOD TEST
DC-15	6.2	11-Nov-94	123.6	133.6	50	1	3	21.03	3706	10	1.1444	GOOD TEST
DC-15	6.2	11-Nov-94	133.6	143.6	50	3	3	63.08	3706	10	3.4333	GOOD TEST
DC-15	6.2	11-Nov-94	143.6	163.6	50	0	3	0.00	3706	20	0.0000	COMBINED SEVERAL 0 TAKE TESTS
DC-15	6.6	12-Nov-94	163.6	173.6	45	2	3	42.06	3366	10	2.5197	GOOD TEST
DC-15	6.6	12-Nov-94	173.6	183.6	45	6	3	126.17	3366	10	7.5592	GOOD TEST
DC-15	6.6	14-Nov-94	183.6	193.6	30	146	3	3070.06	2311	10	267.9059	GOOD TEST
DC-15	6.6	14-Nov-94	193.6	203.6	40	59	3	1240.64	3015	10	83.0036	GOOD TEST
DC-15	6.6	14-Nov-94	203.6	213.6	36	136	3	2859.79	2733	10	211.0246	GOOD TEST

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			TOP	BOTTOM								
DC-15	6.6	14-Nov-94	213.6	223.6	46	31	3	651.86	3437	10	38.2565	GOOD TEST
DC-15	6.6	14-Nov-94	223.6	273.4	46	0	3	0.00	3437	49.8	0.0000	COMBINED SEVERAL 0 TAKE TESTS
DC-15	6.1	15-Nov-94	273.4	283.6	48	2	3	42.06	3562	10.2	2.3465	GOOD TEST
DC-15	6.1	15-Nov-94	283.6	323.6	46	0	3	0.00	3421	40	0.0000	COMBINED SEVERAL 0 TAKE TESTS
DC-15	5.3	16-Nov-94	323.6	333.6	46	10	3	210.28	3397	10	12.4848	GOOD TEST
DC-15	5.3	16-Nov-94	333.6	343.6	46	4	3	84.11	3397	10	4.9939	GOOD TEST
DC-15	5.3	16-Nov-94	343.6	502.1	50	0	3	0.00	3678	158.5	0.0000	COMBINED SEVERAL 0 TAKE TESTS
DC-6	8.8	04-Aug-94	101.8	112.4	53	1	3	21.03	3996	10.6	1.0164	GOOD TEST
DC-6	9.8	05-Aug-94	112.4	122.8	51	0	3	0.00	3886	10.4	0.0000	GOOD TEST
DC-6	9.8	05-Aug-94	122.8	132.8	54	8	3	168.22	4097	10	8.2816	GOOD TEST
DC-6	7.1	12-Aug-94	132.8	162.8	53	0	3	0.00	3944	30	0.0000	GOOD TEST
DC-6	7.5	13-Aug-94	162.8	172.9	50	0	3	0.00	3746	10.1	0.0000	GOOD TEST
DC-6	7.5	13-Aug-94	172.8	182.9	50	14	3	294.39	3746	10.1	15.7363	GOOD TEST
DC-6	8.2	15-Aug-94	182.9	212.6	51	0	3	0.00	3837	29.7	0.0000	GOOD TEST
DC-6	8.2	15-Aug-94	212.6	222.6	52	73	3	1535.03	3908	10	79.2337	GOOD TEST
DC-6	7.5	16-Aug-94	222.6	252.8	50	0	3	0.00	3746	30.2	0.0000	GOOD TEST
DC-6	8.3	17-Aug-94	252.8	282.3	50	0	3	0.00	3770	29.5	0.0000	GOOD TEST
DC-6	6.9	18-Aug-94	282.3	292.6	54	0	3	0.00	4009	10.3	0.0000	GOOD TEST
DC-6	6.9	18-Aug-94	292.6	303	54	24	3	504.67	4009	10.4	24.6640	GOOD TEST
DC-6	6.9	18-Aug-94	303	312.9	52	1	4	15.77	3868	9.9	0.8285	GOOD TEST
DC-6	7.5	19-Aug-94	312.9	323	50	5	3	105.14	3746	10.1	5.6201	GOOD TEST
DC-6	7.5	19-Aug-94	323	353	50	0	3	0.00	3746	30	0.0000	GOOD TEST
DC-6	8.1	20-Aug-94	353	392.9	54	0	3	0.00	4045	39.9	0.0000	GOOD TEST
DC-6	7.9	23-Aug-94	392.9	412.8	50	0	3	0.00	3758	19.9	0.0000	GOOD TEST
DC-6	7.9	23-Aug-94	412.8	422.9	52	54	4	851.63	3898	10.1	43.7376	GOOD TEST
DC-6	7.9	23-Aug-94	422.9	433	53	0	3	0.00	3969	10.1	0.0000	GOOD TEST
DC-6	6	25-Aug-94	433	443	50	28	3	588.78	3700	10	32.0974	GOOD TEST
DC-6	6	25-Aug-94	443	462.8	50	0	3	0.00	3700	19.8	0.0000	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10 -5	REMARKS
			TOP	BOTTOM								
DC-6	6	25-Aug-94	462.8	472.9	51	7	3	147.19	3770	10.1	7.8168	GOOD TEST
DC-6	6	25-Aug-94	472.9	482.2	53	1	4	15.77	3911	9.3	0.8582	GOOD TEST
DC-6	6.8	26-Aug-94	472.9	502.7	55	19	3	399.53	4076	29.8	8.5099	GOOD TEST
DC-6	6.8	26-Aug-94	472.9	492.5	50	22	3	462.61	3724	19.6	15.0097	GOOD TEST
C-146	-5	14-Oct-93	107	112	90	49	3	1030.36	6178	5	51.0526	GOOD TEST
C-146	-5	14-Oct-93	112	117	95	57	3	1198.59	6530	5	56.1891	GOOD TEST
C-146	-5	14-Oct-93	117	122	90	51	3	1072.42	6178	5	53.1364	GOOD TEST
C-146	-5	14-Oct-93	122	127	95	51	3	1072.42	6530	5	50.2745	GOOD TEST
C-146	-5	14-Oct-93	127	132	90	51	3	1072.42	6178	5	53.1364	GOOD TEST
C-146	-5	14-Oct-93	132	137	97	49	3	1030.36	6670	5	47.2842	GOOD TEST
C-146	-5	14-Oct-93	137	142	97	45	3	946.25	6670	5	43.4243	GOOD TEST
C-146	-5	14-Oct-93	142	147	100	34	3	714.95	6881	5	31.8034	GOOD TEST
C-146	-5	14-Oct-93	147	152	100	26	3	546.72	6881	5	24.3202	GOOD TEST
C-146	-5	14-Oct-93	152	157	95	39	3	820.09	6530	5	38.4452	GOOD TEST
C-146	-5	14-Oct-93	157	162	90	37	3	778.03	6178	5	38.5499	GOOD TEST
C-146	-5	14-Oct-93	162	167	90	29	3	609.81	6178	5	30.2148	GOOD TEST
C-146	-5	14-Oct-93	167	172	90	31	3	651.86	6178	5	32.2986	GOOD TEST
C-146	-5	14-Oct-93	172	177	95	33	3	693.92	6530	5	32.5305	GOOD TEST
C-146	-5	14-Oct-93	177	182	100	0	3	0.00	6881	5	0.0000	GOOD TEST
C-146	-5	14-Oct-93	182	187	100	74	3	1556.06	6881	5	69.2191	GOOD TEST
C-146	-5	14-Oct-93	187	192	80	49	3	1030.36	5475	5	57.6118	GOOD TEST
C-146	-5	14-Oct-93	192	197	80	50	3	1051.39	5475	5	58.7876	GOOD TEST
C-146	-5	14-Oct-93	197	202	80	78	3	1640.17	5475	5	91.7087	GOOD TEST
C-146	-5	14-Oct-93	202	207	80	35	3	735.97	5475	5	41.1513	GOOD TEST
C-146	-5	14-Oct-93	207	212	80	36	3	757.00	5475	5	42.3271	GOOD TEST
C-146	-5	14-Oct-93	212	217	80	37	3	778.03	5475	5	43.5028	GOOD TEST
C-146	-5	14-Oct-93	217	222	80	53	3	1114.48	5475	5	62.3149	GOOD TEST
C-146	-5	14-Oct-93	222	227	80	58	3	1219.61	5475	5	68.1936	GOOD TEST

*** A negative value in the depth to water column denotes artesian pressure

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-146	-5	14-Oct-93	227	232	80	65	3	1366.81	5475	5	76.4239	GOOD TEST
C-146	-5	14-Oct-93	232	237	80	58	3	1219.61	5475	5	68.1936	GOOD TEST
C-146	-5	14-Oct-93	237	242	80	74	3	1556.06	5475	5	87.0056	GOOD TEST
C-146	-5	14-Oct-93	242	247	80	81	3	1703.25	5475	5	95.2359	GOOD TEST
C-146	-5	14-Oct-93	247	252	80	43	3	904.20	5475	5	50.5573	GOOD TEST
C-146	-5	14-Oct-93	252	257	80	86	3	1808.39	5475	5	101.1147	GOOD TEST
C-146	-5	14-Oct-93	257	262	80	85	3	1787.37	5475	5	99.9389	GOOD TEST
C-146	-5	14-Oct-93	262	267	80	84	3	1766.34	5475	5	98.7632	GOOD TEST
C-146	-5	14-Oct-93	267	272	80	80	3	1682.23	5475	5	94.0602	GOOD TEST
C-146	-5	14-Oct-93	272	277	80	35	3	735.97	5475	5	41.1513	GOOD TEST
C-146	-5	14-Oct-93	277	282	80	61	3	1282.70	5475	5	71.7209	GOOD TEST
C-146	-5	14-Oct-93	282	287	80	70	3	1471.95	5475	5	82.3026	GOOD TEST
C-146	-5	14-Oct-93	287	292	80	60	3	1261.67	5475	5	70.5451	GOOD TEST
C-146	-5	14-Oct-93	292	297	80	37	3	778.03	5475	5	43.5028	GOOD TEST
C-146	-5	14-Oct-93	297	302	80	52	3	1093.45	5475	5	61.1391	GOOD TEST
C-146	-5	14-Oct-93	302	307	80	41	3	862.14	5475	5	48.2058	GOOD TEST
C-146	-5	14-Oct-93	307	312	80	33	3	693.92	5475	5	38.7998	GOOD TEST
C-146	-5	14-Oct-93	312	317	80	41	3	862.14	5475	5	48.2058	GOOD TEST
C-146	-5	14-Oct-93	317	322	80	18	3	378.50	5475	5	21.1635	GOOD TEST
C-146	-5	14-Oct-93	322	327	80	20	3	420.56	5475	5	23.5150	GOOD TEST
C-146	-5	14-Oct-93	327	332	90	31	3	651.86	6178	5	32.2986	GOOD TEST
C-146	-5	14-Oct-93	332	337	80	58	3	1219.61	5475	5	68.1936	GOOD TEST
C-146	-5	14-Oct-93	337	342	90	39	3	820.09	6178	5	40.6337	GOOD TEST
C-146	-5	14-Oct-93	342	347	80	39	3	820.09	5475	5	45.8543	GOOD TEST
C-146	-5	14-Oct-93	347	352	90	39	3	820.09	6178	5	40.6337	GOOD TEST
C-146	-5	14-Oct-93	352	357	90	40	3	841.11	6178	5	41.6756	GOOD TEST
C-146	-5	14-Oct-93	357	362	90	45	3	946.25	6178	5	46.8851	GOOD TEST
C-146	-5	14-Oct-93	362	367	80	62	3	1303.73	5475	5	72.8966	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
C-146	-5	14-Oct-93	367	372	80	53	3	1114.48	5475	5	62.3149	GOOD TEST
C-146	-5	14-Oct-93	372	377	100	26	3	546.72	6881	5	24.3202	GOOD TEST
C-146	-5	14-Oct-93	377	382	100	29	3	609.81	6881	5	27.1264	GOOD TEST
C-146	-5	14-Oct-93	382	387	100	33	3	693.92	6881	5	30.8680	GOOD TEST
C-146	-5	14-Oct-93	387	392	100	30	3	630.84	6881	5	28.0618	GOOD TEST
C-146	-5	14-Oct-93	392	397	100	34	3	714.95	6881	5	31.8034	GOOD TEST
C-146	-5	14-Oct-93	397	402	100	23	3	483.64	6881	5	21.5141	GOOD TEST
C-146	-5	14-Oct-93	402	407	100	35	3	735.97	6881	5	32.7388	GOOD TEST
C-146	-5	14-Oct-93	407	412	100	29	3	609.81	6881	5	27.1264	GOOD TEST
C-146	-5	14-Oct-93	412	417	100	31	3	651.86	6881	5	28.9972	GOOD TEST
C-146	-5	14-Oct-93	417	422	100	22	3	462.61	6881	5	20.5787	GOOD TEST
C-146	-5	14-Oct-93	422	427	100	31	3	651.86	6881	5	28.9972	GOOD TEST
C-146	-5	14-Oct-93	427	432	100	28	3	588.78	6881	5	26.1910	GOOD TEST
C-146	-5	14-Oct-93	432	437	100	27	3	567.75	6881	5	25.2556	GOOD TEST
C-146	-5	14-Oct-93	437	442	100	28	3	588.78	6881	5	26.1910	GOOD TEST
C-146	-5	14-Oct-93	442	447	100	22	3	462.61	6881	5	20.5787	GOOD TEST
C-146	-5	14-Oct-93	447	452	100	18	3	378.50	6881	5	16.8371	GOOD TEST
C-146	-5	14-Oct-93	452	457	100	21	3	441.58	6881	5	19.6433	GOOD TEST
C-146	-5	14-Oct-93	457	462	100	21	3	441.58	6881	5	19.6433	GOOD TEST
C-146	-5	14-Oct-93	462	467	90	27	3	567.75	6178	5	28.1310	GOOD TEST
C-146	-5	14-Oct-93	467	472	100	16	3	336.45	6881	5	14.9663	GOOD TEST
C-146	-5	14-Oct-93	472	477	100	15	3	315.42	6881	5	14.0309	GOOD TEST
C-146	-5	14-Oct-93	477	482	100	25	3	525.70	6881	5	23.3848	GOOD TEST
C-146	-5	14-Oct-93	482	487	100	4	3	84.11	6881	5	3.7416	GOOD TEST
C-146	-5	14-Oct-93	487	492	100	3	3	63.08	6881	5	2.8062	GOOD TEST
C-145	-5	13-Sep-93	92	97	50	35	3	735.97	3365	5	66.9605	GOOD TEST
C-145	-5	13-Sep-93	97	102	50	41	3	862.14	3365	5	78.4395	GOOD TEST
C-145	-5	13-Sep-93	102	107	70	84	3	1766.34	4771	5	113.3228	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET TOP	BOTTOM	GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
C-145	-5	13-Sep-93	107	112	70	36	3	757.00	4771	5	48.5669	GOOD TEST
C-145	-5	13-Sep-93	112	117	70	48	3	1009.34	4771	5	64.7559	GOOD TEST
C-145	-5	13-Sep-93	117	122	70	50	3	1051.39	4771	5	67.4541	GOOD TEST
C-145	-5	13-Sep-93	122	127	70	12	3	252.33	4771	5	16.1890	GOOD TEST
C-145	-5	13-Sep-93	127	132	70	52	3	1093.45	4771	5	70.1522	GOOD TEST
C-145	-5	13-Sep-93	132	137	70	57	3	1198.59	4771	5	76.8976	GOOD TEST
C-145	-5	13-Sep-93	137	142	70	77	3	1619.14	4771	5	103.8793	GOOD TEST
C-145	-5	13-Sep-93	142	147	70	79	3	1661.20	4771	5	106.5774	GOOD TEST
C-145	-5	13-Sep-93	147	152	70	84	3	1766.34	4771	5	113.3228	GOOD TEST
C-145	-5	13-Sep-93	152	157	70	97	3	2039.70	4771	5	130.8609	GOOD TEST
C-145	-5	13-Sep-93	157	162	65	83	3	1745.31	4420	5	120.8841	GOOD TEST
C-145	-5	13-Sep-93	162	167	60	68	3	1429.89	4068	5	107.5999	GOOD TEST
C-145	-5	13-Sep-93	167	172	60	81	3	1703.25	4068	5	128.1705	GOOD TEST
C-145	-5	13-Sep-93	172	177	75	43	3	904.20	5123	5	54.0281	GOOD TEST
C-145	-5	13-Sep-93	177	182	70	50	3	1051.39	4771	5	67.4541	GOOD TEST
C-145	-5	13-Sep-93	182	187	70	62	3	1303.73	4771	5	83.6431	GOOD TEST
C-145	-5	13-Sep-93	187	192	60	62	3	1303.73	4068	5	98.1058	GOOD TEST
C-145	-5	13-Sep-93	192	197	60	40	3	841.11	4068	5	63.2941	GOOD TEST
C-145	-5	13-Sep-93	197	202	60	63	3	1324.75	4068	5	99.6882	GOOD TEST
C-145	-5	13-Sep-93	202	207	60	38	3	799.06	4068	5	60.1294	GOOD TEST
C-145	-5	13-Sep-93	207	212	60	17	3	357.47	4068	5	26.9000	GOOD TEST
C-145	-5	13-Sep-93	212	217	70	31	3	651.86	4771	5	41.8215	GOOD TEST
C-145	-5	13-Sep-93	217	222	60	45	3	946.25	4068	5	71.2058	GOOD TEST
C-145	-5	13-Sep-93	222	227	70	61	3	1282.70	4771	5	82.2940	GOOD TEST
C-145	-5	13-Sep-93	227	232	80	34	3	714.95	5475	5	39.9756	GOOD TEST
C-145	-5	13-Sep-93	232	237	80	25	3	525.70	5475	5	29.3938	GOOD TEST
C-145	-5	13-Sep-93	237	242	80	43	3	904.20	5475	5	50.5573	GOOD TEST
C-145	-5	13-Sep-93	242	247	80	17	3	357.47	5475	5	19.9878	GOOD TEST

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			TOP	BOTTOM								
C-145	-5	13-Sep-93	247	252	80	12	3	252.33	5475	5	14.1090	GOOD TEST
C-145	-5	13-Sep-93	252	257	80	30	3	630.84	5475	5	35.2726	GOOD TEST
C-145	-5	13-Sep-93	257	262	80	48	3	1009.34	5475	5	56.4361	GOOD TEST
C-145	-5	13-Sep-93	262	267	70	43	3	904.20	4771	5	58.0105	GOOD TEST
C-145	-5	13-Sep-93	267	272	70	57	3	1198.59	4771	5	76.8976	GOOD TEST
C-145	-5	13-Sep-93	272	277	70	56	3	1177.56	4771	5	75.5486	GOOD TEST
C-145	-5	13-Sep-93	277	282	70	71	3	1492.98	4771	5	95.7848	GOOD TEST
C-145	-5	13-Sep-93	282	287	60	54	3	1135.50	4068	5	85.4470	GOOD TEST
C-145	-5	13-Sep-93	287	292	60	46	3	967.28	4068	5	72.7882	GOOD TEST
C-145	-5	13-Sep-93	292	297	75	38	3	799.06	5123	5	47.7458	GOOD TEST
C-145	-5	13-Sep-93	297	302	75	48	3	1009.34	5123	5	60.3104	GOOD TEST
C-145	-5	13-Sep-93	302	307	70	64	3	1345.78	4771	5	86.3412	GOOD TEST
C-145	-5	13-Sep-93	307	312	70	50	3	1051.39	4771	5	67.4541	GOOD TEST
C-145	-5	13-Sep-93	312	317	70	62	3	1303.73	4771	5	83.6431	GOOD TEST
C-145	-5	13-Sep-93	317	322	70	57	3	1198.59	4771	5	76.8976	GOOD TEST
C-145	-5	13-Sep-93	322	327	70	64	3	1345.78	4771	5	86.3412	GOOD TEST
C-145	-5	13-Sep-93	327	332	70	72	3	1514.00	4771	5	97.1339	GOOD TEST
C-145	-5	13-Sep-93	332	337	70	37	3	778.03	4771	5	49.9160	GOOD TEST
C-145	-5	13-Sep-93	337	342	70	56	3	1177.56	4771	5	75.5486	GOOD TEST
C-145	-5	13-Sep-93	342	347	70	65	3	1366.81	4771	5	87.6903	GOOD TEST
C-145	-5	13-Sep-93	347	352	70	61	3	1282.70	4771	5	82.2940	GOOD TEST
C-145	-5	13-Sep-93	352	357	70	77	3	1619.14	4771	5	103.8793	GOOD TEST
C-145	-5	13-Sep-93	357	362	70	64	3	1345.78	4771	5	86.3412	GOOD TEST
C-145	-5	13-Sep-93	362	367	80	36	3	757.00	5475	5	42.3271	GOOD TEST
C-145	-5	13-Sep-93	367	372	70	67	3	1408.86	4771	5	90.3885	GOOD TEST
C-145	-5	13-Sep-93	372	377	70	45	3	946.25	4771	5	60.7087	GOOD TEST
C-145	-5	13-Sep-93	377	382	70	39	3	820.09	4771	5	52.6142	GOOD TEST
C-145	-5	13-Sep-93	382	387	70	72	3	1514.00	4771	5	97.1339	GOOD TEST

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HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET TOP	BOTTOM	GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
C-145	-5	13-Sep-93	387	392	70	51	3	1072.42	4771	5	68.8032	GOOD TEST
C-145	-5	13-Sep-93	392	397	65	74	3	1556.06	4420	5	107.7762	GOOD TEST
C-145	-5	13-Sep-93	397	402	60	44	3	925.22	4068	5	69.6235	GOOD TEST
C-145	-5	13-Sep-93	402	407	60	52	3	1093.45	4068	5	82.2823	GOOD TEST
C-145	-5	13-Sep-93	407	412	100	46	3	967.28	6881	5	43.0281	GOOD TEST
C-145	-5	13-Sep-93	412	417	100	0	3	0.00	6881	5	0.0000	GOOD TEST
C-145	-5	13-Sep-93	417	422	55	88	3	1850.45	3716	5	152.4249	GOOD TEST
C-145	-5	13-Sep-93	422	427	75	20	3	420.56	5123	5	25.1293	GOOD TEST
C-145	-5	13-Sep-93	427	432	100	15	3	315.42	6881	5	14.0309	GOOD TEST
C-145	-5	13-Sep-93	432	502	50	0	3	0.00	3365	70	0.0000	GOOD TEST
	SPUR TUNNEL HOLES BEGIN HERE							*SPUR TUNNEL HOLES BEGIN HERE*				
C-115D	10.2	14-Apr-92	121.7	131.8	40	157	3	3301.37	3124	10.1	211.5505	GOOD TEST
C-115D	10.2	14-Apr-92	132	141.8	38	171	3	3595.76	2984	9.8	246.7283	GOOD TEST
C-115D	10.2	14-Apr-92	141.7	151.8	50	5	3	105.14	3828	10.1	5.4993	GOOD TEST
C-115D	10.2	14-Apr-92	152.2	161.8	45	96	3	2018.67	3476	9.6	120.7203	GOOD TEST
C-115D	23.6	16-Apr-92	162	171.8	50	8	3	168.22	4236	9.8	8.1301	GOOD TEST
C-115D	23.6	16-Apr-92	171.8	182	50	0	3	0.00	4236	10.2	0.0000	GOOD TEST
C-115D	23.6	16-Apr-92	181.7	192.3	50	41	3	862.14	4236	10.6	39.3087	GOOD TEST
C-115D	23.6	16-Apr-92	192.8	232.7	50	0	12	0.00	4236	39.9	0.0000	GOOD TEST, COMBINE 4 ZERO TAKE TESTS
C-116	7.8	07-Apr-92	122.1	131.5	50	0	3	0.00	3755	9.4	0.0000	GOOD TEST
C-116	7.8	08-Apr-92	131.6	141.4	50	51	3	1072.42	3755	9.8	58.4770	GOOD TEST
C-116	7.8	08-Apr-92	141.7	151.6	50	67	3	1408.86	3755	9.9	76.2477	GOOD TEST
C-116	7.8	08-Apr-92	151.8	161.6	30	131	3	2754.65	2348	9.8	240.2031	LEAKAGE AROUND PACKER, POOR TEST.
C-116	7.8	08-Apr-92	161.6	191.5	50	0	9	0.00	3755	29.9	0.0000	GOOD TEST, COMBINE 3 ZERO TAKE TESTS.
C-116	7.8	08-Apr-92	181.6	230.7	30	126	3	2649.51	2348	49.1	65.4519	LEAKAGE AROUNG PACKER, POOR TEST.
C-117	3.4	29-Mar-92	129.8	141.7	19	100	3	2102.78	1440	11.9	258.6378	JOINTS LEAKING AROUND PACKER, 19 psi MAX SUSTAINED PRESSUR
C-117	3.4	30-Mar-92	152.1	161.9	41	88	3	1850.45	2988	9.8	126.8119	POOR TEST, JOINTED ROCK LEAKING AROUND PACKER.
C-117	3.4	30-Mar-92	162	171.8	50	0	3	0.00	3621	9.8	0.0000	GOOD TEST

*** A negative value in the depth to water column denotes artesian pressure

PASSAIC RIVER TUNNEL PROJECT
EXPLORATION PROGRAM
PRESSURE TEST DATA

PRINT DATE

31-Jan-95

HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET TOP	BOTTOM	GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^-5	REMARKS
C-117	3.5	01-Apr-92	162.1	221.1	28	93	3	1955.59	2076	59	46.9968	JOINTS LEAKING AROUND PACKER. 28 psi MAX SUSTAINED PRESSUR
C-118	4.7	09-Apr-92	140.1	153	50	63	3	1324.75	3660	12.9	60.3204	GOOD TEST
C-118	4.7	09-Apr-92	153	163	50	14	3	294.39	3660	10	16.2224	GOOD TEST
C-118	4.7	09-Apr-92	163	173	50	19	3	399.53	3660	10	22.0161	GOOD TEST
C-118	0.5	11-Apr-92	173	213	50	0	12	0.00	3532	40	0.0000	GOOD TEST, COMBINE 4 ZERO TAKE TESTS.
C-118	1.5	13-Apr-92	213	223	50	3	3	63.08	3563	10	3.5714	GOOD TEST
C-118	1.5	13-Apr-92	223	263	50	194	3	4079.40	3563	40	78.4593	GOOD TEST
DC-119	12.2	17-Jul-93	193.5	203.5	10	0	3	0.00	1075	10	0.0000	GOOD TEST BUT PROBLEMS WITH PUMP PRESSURE
DC-119	12.2	17-Jul-93	203.5	213.4	20	0	3	0.00	1779	9.9	0.0000	
DC-119	12.2	17-Jul-93	213.4	223.5	15	0	3	0.00	1427	10.1	0.0000	GOOD TEST
DC-119	12.2	17-Jul-93	223.5	233.5	28	0	3	0.00	2341	10	0.0000	GOOD TEST
DC-119	6.6	19-Jul-93	233.5	272.2	50	1000	3	*******	3718	38.7	398.0173	DUMMY DATA HIGH TAKE BUT PACKER WOULD NOT SEAL.
DC-120	5.8	11-Aug-93	107.6	122	45	0	3	0.00	3342	14.4	0.0000	GOOD TEST
DC-120	5.8	11-Aug-93	122	132.9	48	0	3	0.00	3553	10.9	0.0000	GOOD TEST
DC-120	4.2	12-Aug-93	132.9	142.9	46	0	3	0.00	3364	10	0.0000	GOOD TEST
DC-120	4.2	12-Aug-93	142.9	152.9	48	0	3	0.00	3504	10	0.0000	GOOD TEST
DC-120	4.2	12-Aug-93	152.9	163	30	0	3	0.00	2238	10.1	0.0000	GOOD TEST
~~DC-120~~	~~4.2~~	~~12-Aug-93~~	~~163~~	~~173~~	~~50~~	~~5~~	~~3~~	~~105.14~~	~~3645~~	~~10~~	~~5.8179~~	~~GOOD TEST~~
DC-120	6	13-Aug-93	173	182.9	50	0	3	0.00	3700	9.9	0.0000	GOOD TEST
DC-120	6	13-Aug-93	182.9	192.7	50	5	3	105.14	3700	9.8	5.8181	GOOD TEST
DC-120	6	13-Aug-93	192.9	202.8	50	11	3	231.31	3700	9.9	12.7039	GOOD TEST .2' unaccountable loss in interval
DC-120	5.8	14-Aug-93	202.8	212.8	50	0	3	0.00	3694	10	0.0000	GOOD TEST
DC-120	5.8	14-Aug-93	212.8	223.1	50	0	3	0.00	3694	10.3	0.0000	GOOD TEST
DC-120	5.8	14-Aug-93	223.1	233	50	0	3	0.00	3694	9.9	0.0000	GOOD TEST
DC-120	5.8	14-Aug-93	233	242.8	50	0	3	0.00	3694	9.8	0.0000	GOOD TEST
DC-120	9.2	16-Aug-93	242.8	253.1	50	0	3	0.00	3797	10.3	0.0000	GOOD TEST
DC-120	9.2	16-Aug-93	253.1	262.9	48	0	3	0.00	3657	9.8	0.0000	GOOD TEST
DC-120	9.2	16-Aug-93	262.9	270.7	48	0	3	0.00	3657	7.8	0.0000	GOOD TEST

*** A negative value in the depth to water column denotes artesian pressure

PASSAIC RIVER TUNNEL PROJECT
EXPLORATION PROGRAM
PRESSURE TEST DATA

PRINT DATE

31-Jan-95

HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
DC-121	10.6	18-Oct-93	90.4	113.2	50	0	3	0.00	3840	22.8	0.0000	GOOD TEST
DC-121	10.6	18-Oct-93	113.2	122.9	50	27	3	567.75	3840	9.7	30.5008	GOOD TEST .1' UNACCOUNTABLE LOSS IN INTERVAL
DC-121	10.6	18-Oct-93	133	143.1	50	0	3	0.00	3840	10.1	0.0000	GOOD TEST
DC-121	10.6	18-Oct-93	143.1	153.1	50	60	3	1261.67	3840	10	66.2687	GOOD TEST
DC-121	9.8	20-Oct-93	153.1	163	50	0	3	0.00	3816	9.9	0.0000	GOOD TEST
DC-121	9.8	20-Oct-93	163	173.1	50	27	3	567.75	3840	10.1	29.6017	GOOD TEST
DC-121	10.6	21-Oct-93	173.1	183.1	50	0	3	0.00	3840	10	0.0000	GOOD TEST
DC-121	10.6	21-Oct-93	183.1	193.1	50	26	3	546.72	3840	10	28.7165	GOOD TEST
DC-121	10.6	21-Oct-93	193.1	203.1	50	0	3	0.00	3840	10	0.0000	GOOD TEST
DC-121	10.6	21-Oct-93	203.1	213.2	50	16	3	336.45	3840	10.1	17.5418	GOOD TEST
DC-121	9.3	22-Oct-93	213.2	223.3	50	0	3	0.00	3800	10.1	0.0000	GOOD TEST
DC-121	9.3	22-Oct-93	223.3	233	50	141	3	2964.92	3840	9.7	159.2820	GOOD TEST
DC-121	9.3	22-Oct-93	233	243.1	50	0	3	0.00	3840	10.1	0.0000	GOOD TEST
DC-121	9.3	22-Oct-93	243.1	253.3	50	6	3	126.17	3840	10.2	6.5302	GOOD TEST
DC-121	9.3	22-Oct-93	253.3	263.1	50	0	3	0.00	3840	9.8	0.0000	GOOD TEST
DC-121	9.3	22-Oct-93	263.1	273.3	50	1	3	21.03	3840	10.2	1.0884	GOOD TEST
DC-122	9	15-May-94	70	93.1	50	0	3	0.00	3791	23.1	0.0000	GOOD TEST
DC-122	9	15-May-94	93.1	103.1	50	150	3	3154.18	3791	10	167.8029	GOOD TEST
DC-122	9	15-May-94	103.1	113.1	50	139	3	2922.87	3791	10	155.4974	GOOD TEST
DC-122	9	15-May-94	113.1	123.3	50	123	3	2586.42	3791	10.2	135.5920	GOOD TEST
DC-122	6.1	17-May-94	123.3	163.2	50	0	3	0.00	3703	39.9	0.0000	GOOD TEST
DC-122	6.1	17-May-94	163.2	173.4	50	22	3	462.61	3703	10.2	24.8312	GOOD TEST
DC-122	5.3	18-May-94	173.4	183.4	50	0	3	0.00	3678	10	0.0000	GOOD TEST
DC-122	5.3	18-May-94	183.4	193.3	50	41	3	862.14	3678	9.9	47.6256	GOOD TEST
DC-122	5.3	18-May-94	193.3	203.2	50	23	3	483.64	3678	9.9	26.7168	GOOD TEST
DC-122	5.3	18-May-94	203.4	213.4	50	14	3	294.39	3678	10	16.1418	GOOD TEST
DC-122	5.3	18-May-94	213.4	223.3	50	2	3	42.06	3678	9.9	2.3232	GOOD TEST
DC-122	5.3	18-May-94	223.3	233.4	50	0	3	0.00	3678	10.1	0.0000	GOOD TEST

*** A negative value in the depth to water column denotes artesian pressure

PASSAIC RIVER TUNNEL PROJECT
EXPLORATION PROGRAM
PRESSURE TEST DATA

PRINT DATE

31-Jan-95

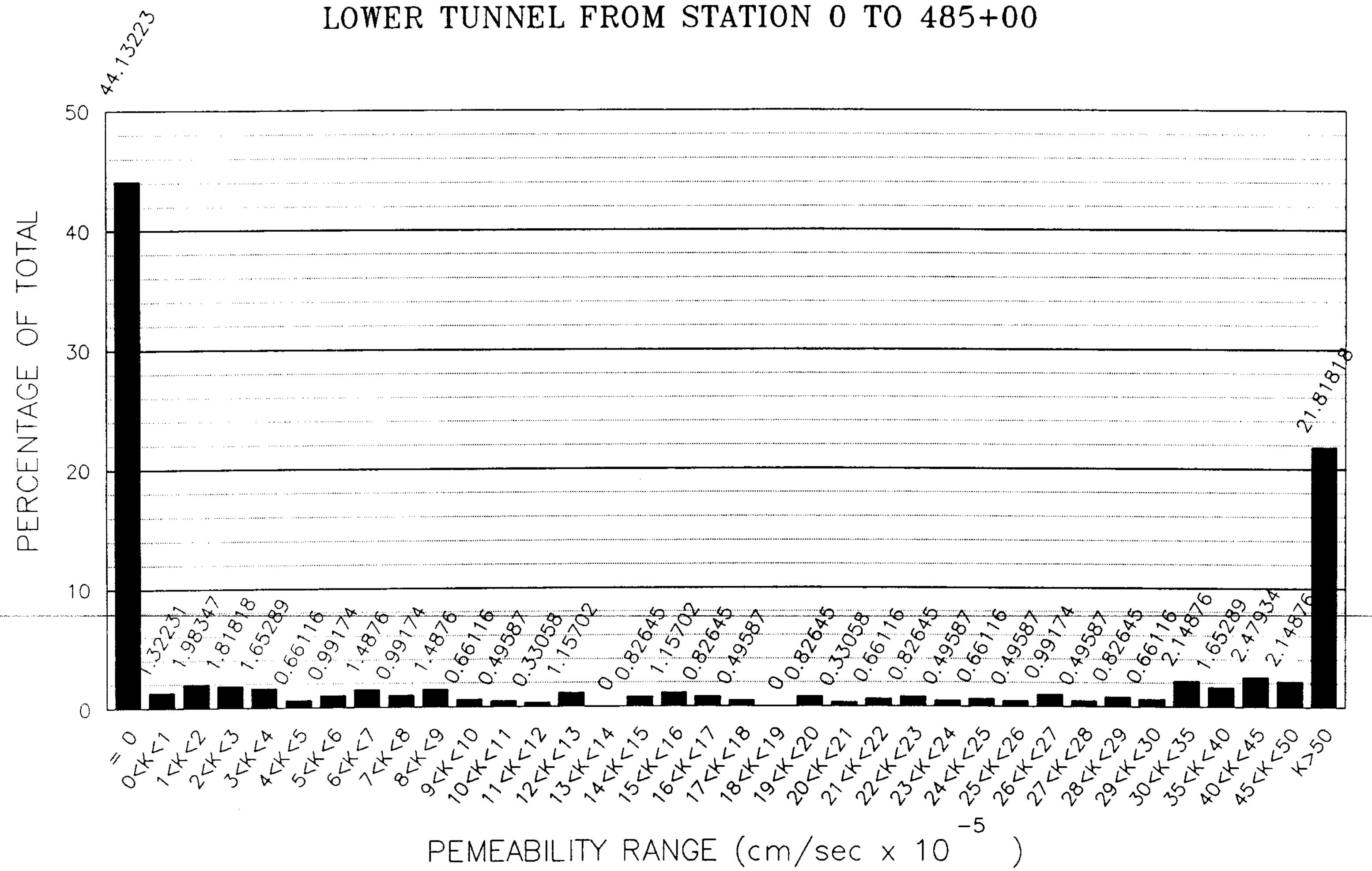
HOLE #	DEPTH TO WATER ***	DATE OF TEST	PACKER SETTING DEPTH IN FEET		GAGE PRES. (psi)	TOTAL TAKE (gal)	TIME (min)	UNIT TAKE (cm^3/sec)	SURCHARGE HEAD (cm)	TEST LENGTH (ft)	PERM. COEFF. Ke cm/sec x 10^{-5}	REMARKS
			TOP	BOTTOM								
DC-122	5.3	19-May-94	233.4	243	50	16	3	336.45	3678	9.6	19.0133	GOOD TEST
DC-122	5.3	19-May-94	243	253	50	14	3	294.39	3678	10	16.1418	GOOD TEST
DC-122	5.3	19-May-94	253	273	50	0	3	0.00	3678	20	0.0000	GOOD TEST

E.3.8-40

*** A negative value in the depth to water column denotes artesian pressure

PASSAIC TUNNEL PROJECT

PERMEABILITY FROM PRESSURE TEST DATA
LOWER TUNNEL FROM STATION 0 TO 485+00

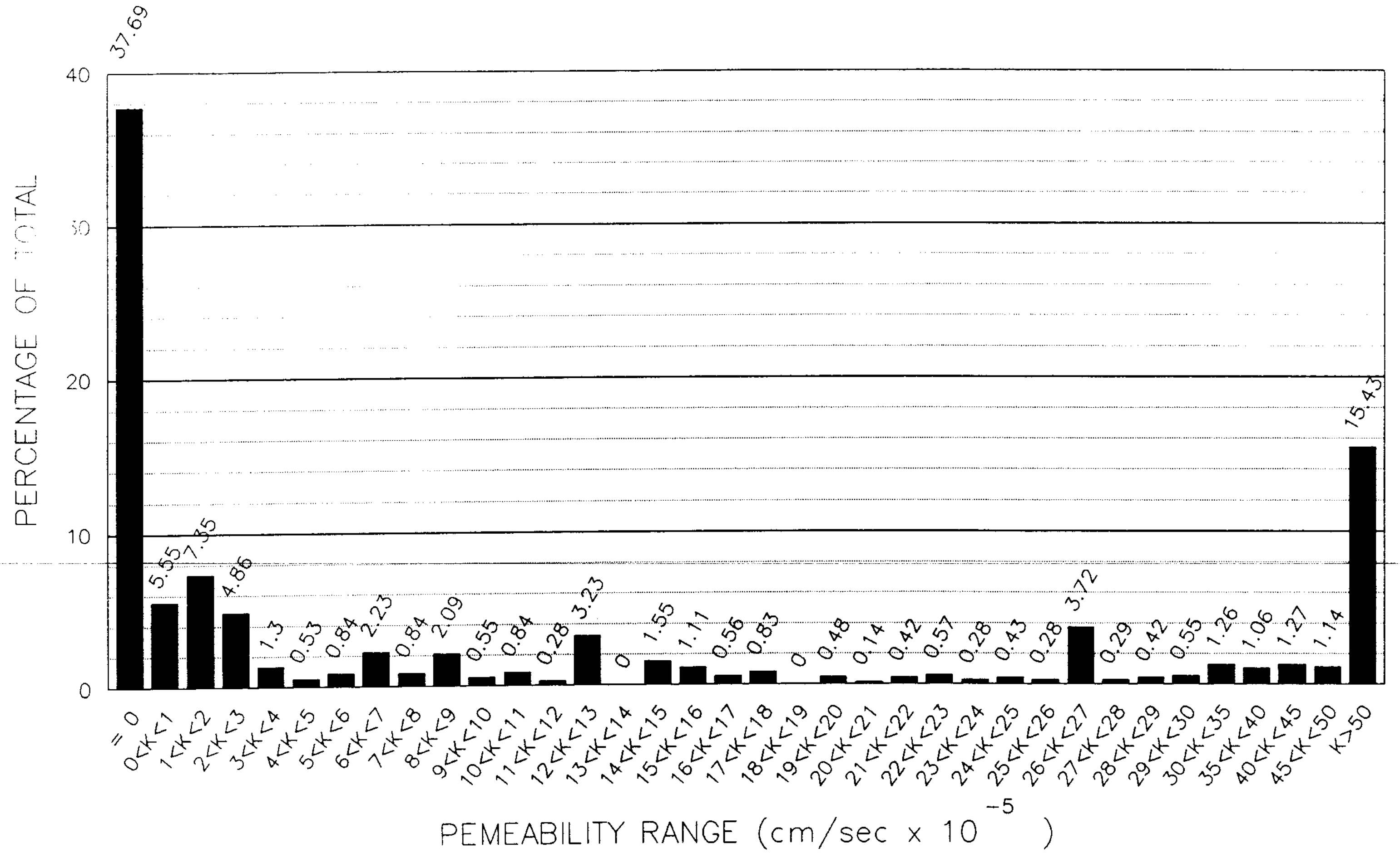


The percentages are *normalized* to the total numer of tests run. Test where packer leakage was noted have not been considered.

* See attached sheet.*

PASSAIC TUNNEL PROJECT

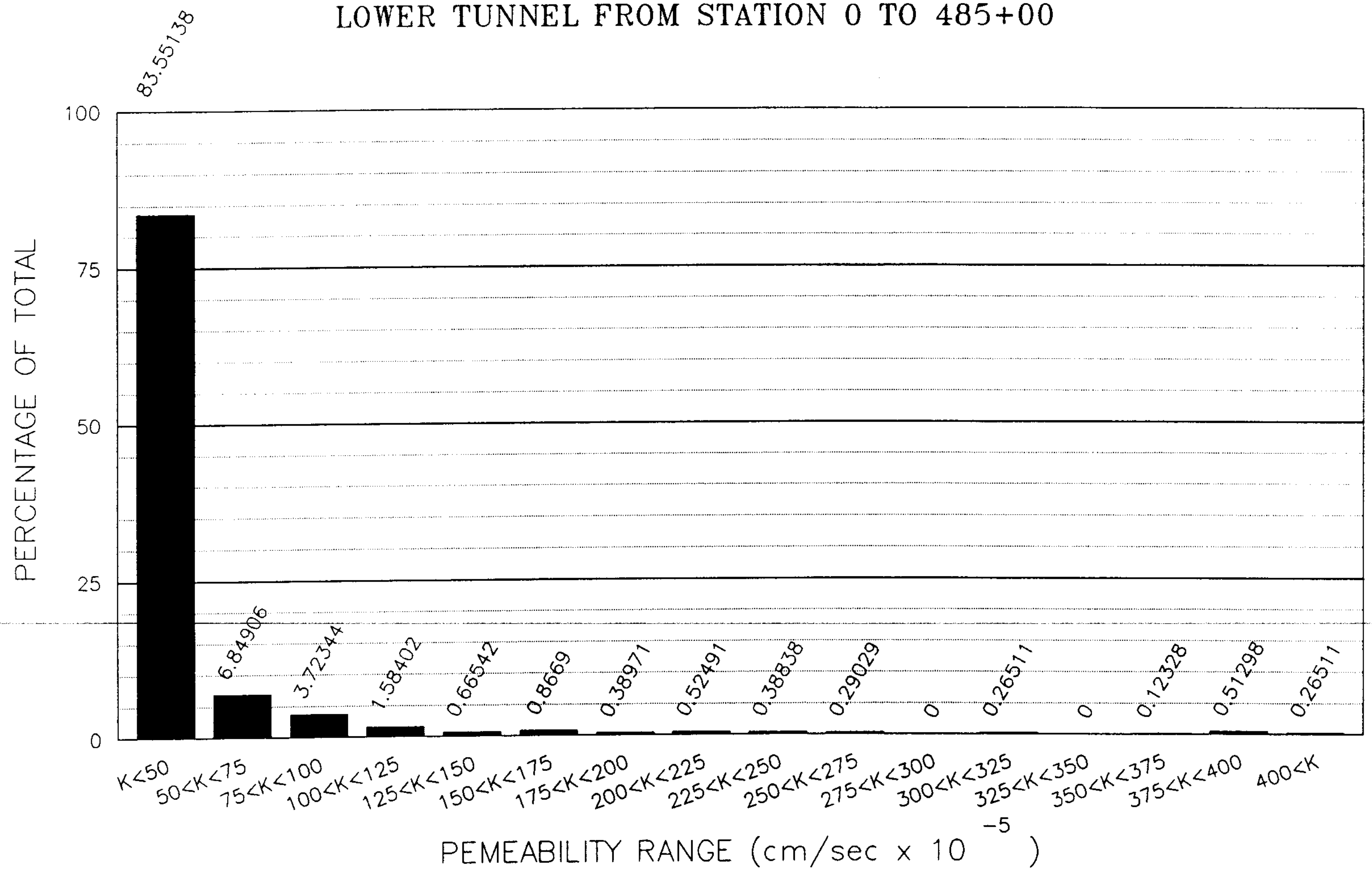
PERMEABILITY FROM PRESSURE TEST DATA
LOWER TUNNEL FROM STATION 0 TO 485+00



The percentages are *normalized* to the total footage tested. Tests where packer leakage was noted have not been considered.

* See attached sheet.*

PASSAIC TUNNEL PROJECT
PERMEABILITY FROM PRESSURE TEST DATA
LOWER TUNNEL FROM STATION 0 TO 485+00

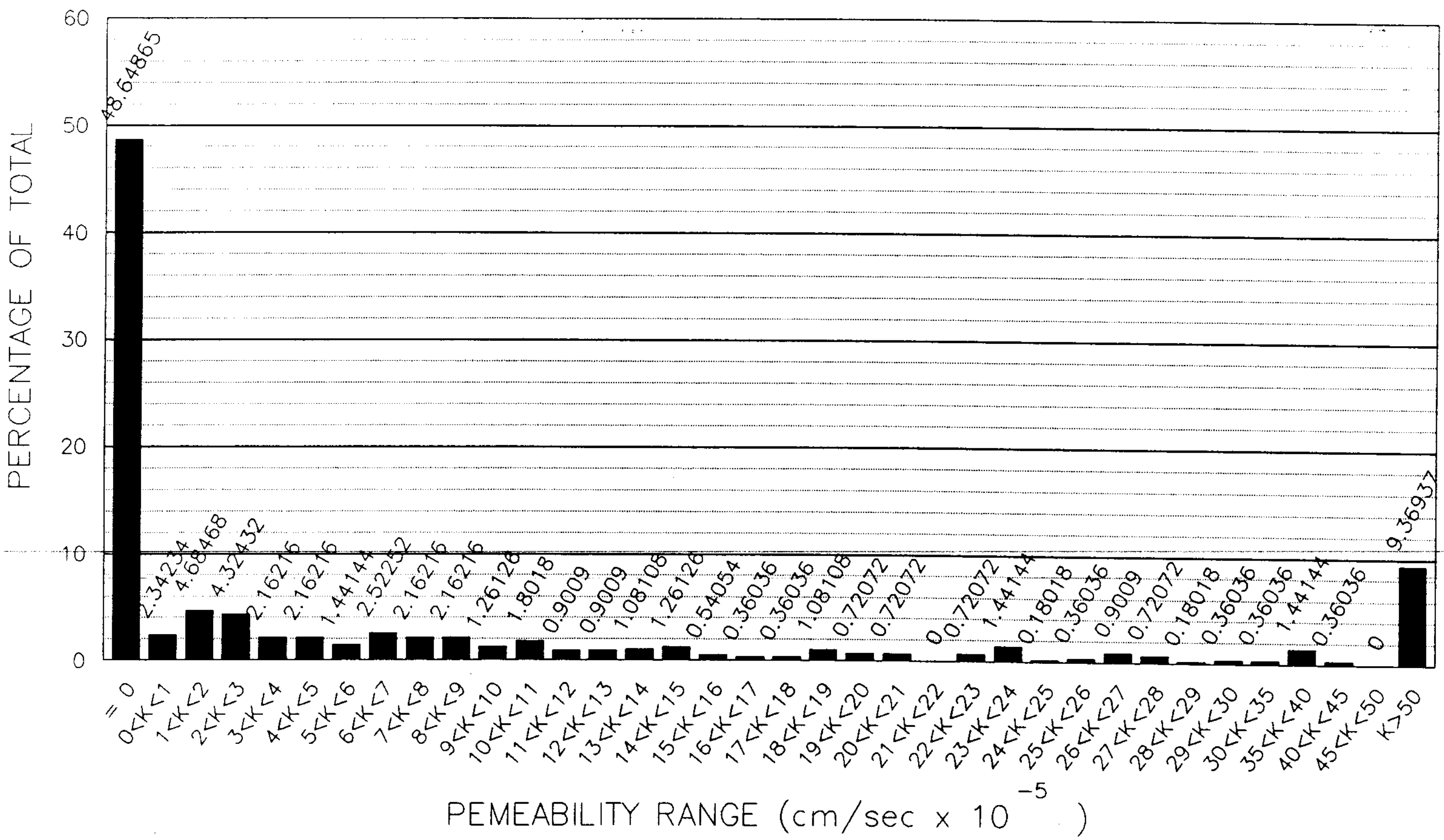


The percentages are normalized to the total footage tested. Tests where packer leakage was noted have not been considered.

PASSAIC TUNNEL PROJECT

PERMEABILITY FROM PRESSURE TEST DATA

UPPER TUNNEL FROM STATION 485+00 TO 1075+00

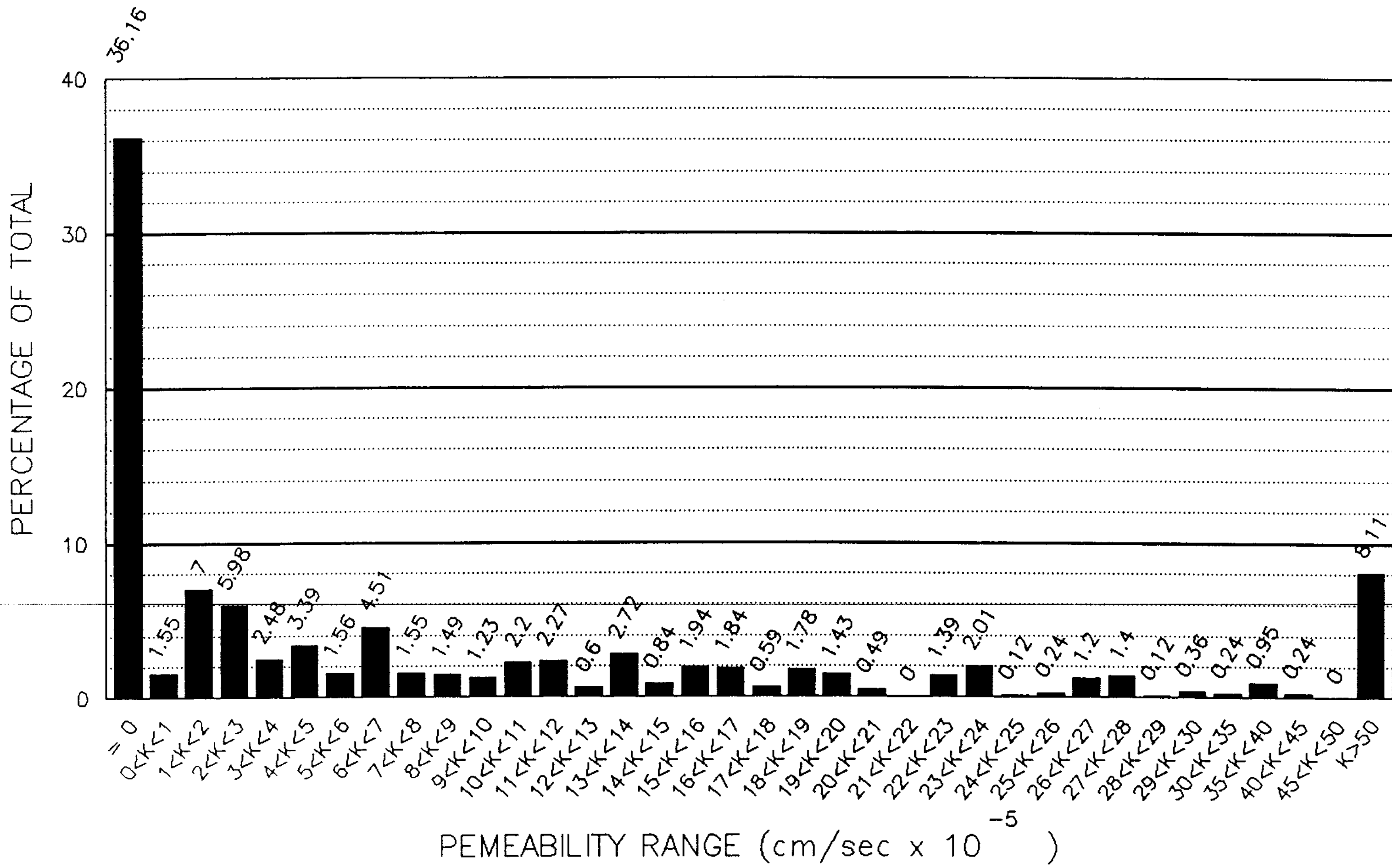


The percentages are *normalized* to the total numer of tests run. Test where packer leakage was noted have not been considered.

* See attached sheet.*

PASSAIC TUNNEL PROJECT

PERMEABILITY FROM PRESSURE TEST DATA
UPPER TUNNEL FROM STATION 485+00 TO 1075+00

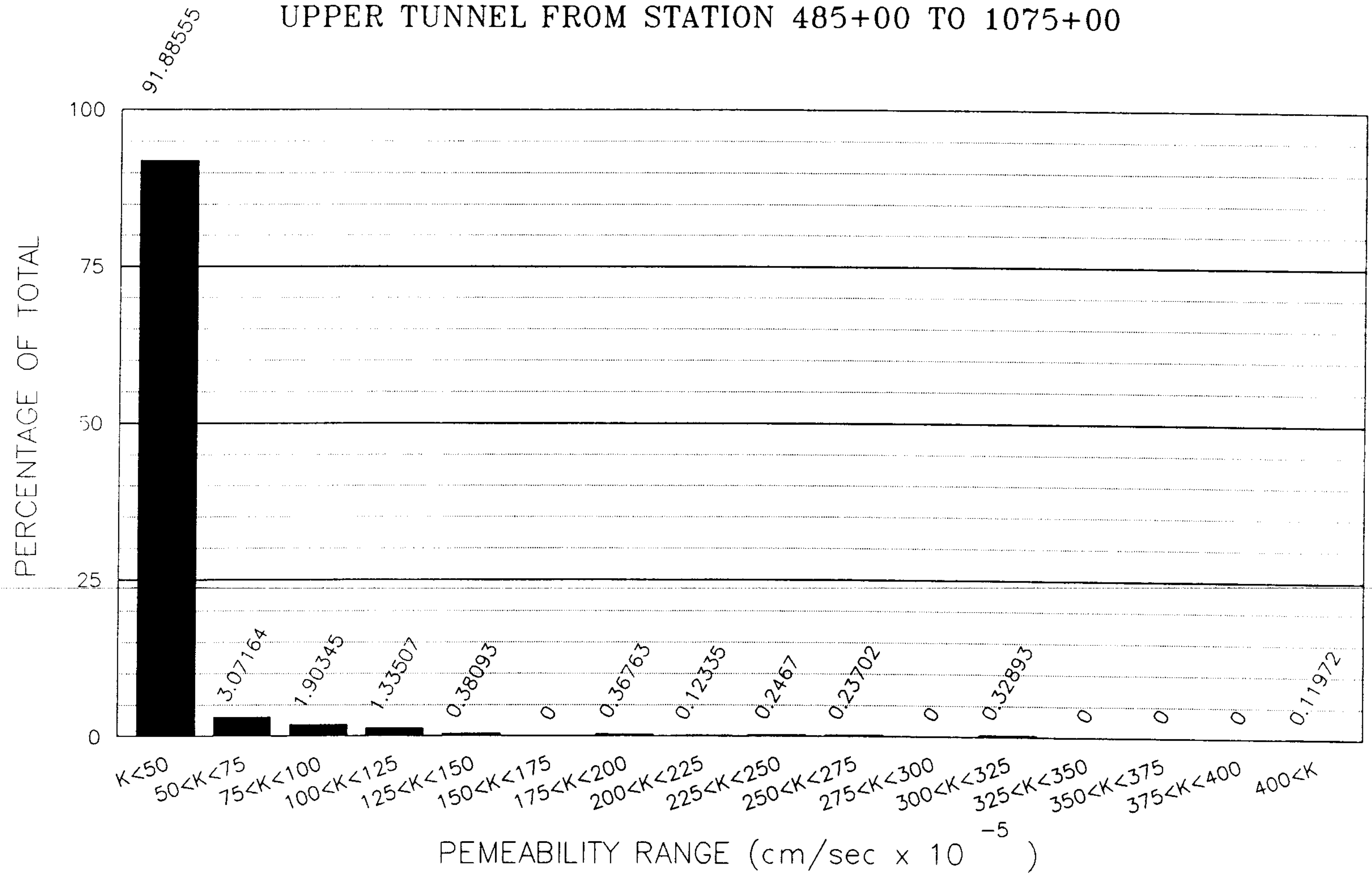


The percentages are *normalized* to the total footage tested. Tests where packer leakage was noted have not been considered.

See attached sheet.

PASSAIC TUNNEL PROJECT

PERMEABILITY FROM PRESSURE TEST DATA
UPPER TUNNEL FROM STATION 485+00 TO 1075+00



The percentages are normalized to the total footage tested. Tests where packer leakage was noted have not been considered.

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.9

RESERVED

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.10

STRESS AND PRESSURE DISTRIBUTION

Attachment E.3.10

Stress and Pressure Distribution

PASSAIC TUNNEL PROJECT
TOTAL VERTICAL STRESS AND HYDROSTATIC PRESSURE
AS MEASURED AT TUNNEL INVERT

PRINT DATE 07-Feb-95

HOLE NUMBER	TUNNEL STATION	TOP ELEV.	INVERT ELEV	THICK. OVERB.	DENSITY OVERB.	ROCK LAYER DATA LAYER 1 TYPE	LAYER 1 THICKNE	LAYER 1 DENSITY	LAYER 2 TYPE	LAYER 2 THICKNE	LAYER 2 DENSITY	LAYER 3 TYPE	LAYER 3 THICKNE	LAYER 3 DENSITY	WATER DEPTH	VERTICAL STRESS (PSI)	HYDROSTATIC PRESSURE (PSI)
S117	SPUR	171.7	0	42.5	120	Jp	129.2	181.5	JtSH	78	155.7				4	198.3	72.7
S116	SPUR	176.5	0	46	120	Jp	130.5	181.5	JtSH	99	155.7				7.5	202.8	73.2
S119	SPUR	170	0	74	120	JtSS	96	154.8	JtSH	15	155.7				5.8	164.9	71.2
S115	SPUR	178.1	0	39.2	120	Jp	138.9	181.5	JtSH	15	155.7	TpBAS	3	181	23.6	207.7	66.9
S120	SPUR	170	0	103	120	JtSS	52	154.8	JtSH	15	155.7				7	158.0	70.6
S121	SPUR	170	0	71	120	JtSS	57	154.8	JtSH	42	155.7				9.2	165.9	69.7
S118	SPUR	166.3	0	140.1	120	JtSS	10.2	154.8	JtSH	16	155.7				1.5	145.0	71.4
S122	SPUR	169.4	0	57	120	JtSS	86.4	154.8	JtSH	26	155.7				6.2	168.5	70.7
S148	SPUR	170	0	80	120	JtSS	90	154.8							3	163.4	72.4
C-1	316	-12	-409	76	120	Tp SS	333	157.4	TpSH	74	158				-12	427.3	177.2
C-145	995	-4	-408.5	87	120	Tp SS	321.5	153.7		4.5	158				-4	415.7	177.0
C-146	1,698	-4	-408.2	103	120	Tp SS	305.8	153.7		3.5	158				-4	412.2	176.9
6	4,809	5	-406.6	99.6	120	TpSS	307	157.4	TpSH	5	158				6.8	424.1	175.4
19	13,913	8.6	-402	112	120	TpSS	298.6	157.4	TpSH	3	158				17.4	419.7	170.4
PR-4C	14,908	10	-401.6	136.4	120	TpSH	275.2	158		78	158				12	415.6	178.4
PR-7	16,315	10	-400.8	161	120	TpSH	239.8	158		78	158				20	397.3	178.0
PR-6	16,905	10	-400.7	260	120	TpSH	150.7	158		99	158				22.6	382.0	168.2
PR-5CA	17,014	10	-400.5	289.3	120	JbSS	111.2	155.3		340.1	158		3	181	23.5	361.0	177.9
23	18,259	8	-399.9	170	120	TpSS	238	157.4	TpSH	9	158	TpBAS	3	181	12	401.8	171.6
24	18,955	7.6	-394	70.7	120	TpSS	252.9	157.4	TpSH	78	158				20	420.9	165.4
26	19,898	15.2	-382.2	60	120	TpSS	218.8	157.4	TpSH	99	158				31	397.8	158.8
28	22,272	107.6	-352.5	32	120	TpSS	68	157.4	TpSH	340.1	158				23.5	474.2	189.2
38	27,760	40	-283.8	48.8	120	TpSS	261.1	157.4	TpSH	9	158	TpBAS	3	181	25	339.7	129.5
42	30,536	95	-249	32	120	TpSS	312	157.4	TpSH	74	158				25	367.7	138.2
53	37,906	75	-156.7	41.1	120	TpSS	187.6	157.4	TpSH	3	158				14.5	242.6	100.4
59	43,923	228.1	-81.4	64.6	120	TpSS	136.3	157.4	TpSH	27	158				62.2	232.4	107.2
60	44,773	177.9	-70.8	20	120	TpSS	39	157.4	TpSH	189.7	158				13.6	267.4	101.9
61	45,034	197.6	-67.5	13.5	120	TpSS	154.2	157.4	TpSH	74	158				11.4	261.0	109.9
62	45,763	169.4	-58.4	19.5	120	TpSS	213.8	157.4	TpSH	4.5	158				14.5	254.9	92.4
58	46,104	225	-54.1	28	120	TpSS	251.1	157.4	TpSH	12.5	158				50.8	297.8	98.9
63	46,287	176.9	-51.8	20	120	TpSS	205.2	157.4	TpSH	3.5	158				24.8	244.8	88.4
64	46,804	176.4	-45.3	17.1	120	TpSS	180.6	157.4	TpSH	24	158	TpSH	5	158	20	238.0	87.4
67	48,417	165.9	-25.6	27	120	TpSS	105.7	157.4	TpSH	3	158	Jo	1.5	181	9	141.3	79.1
68	49,014	172.3	-25.3	46	120	TpSS	139.1	157.4	TpSH	12.5	158				21	204.1	76.5
69	49,577	165.7	-25	40.5	120	TpSS	150.2	157.4	TpSH	10	158	Jo	60.5	181	32	197.9	82.6
70	50,301	168.8	-24.7	39.2	120	TpSS	126.8	157.4	TpSH	20.5	158				42.6	193.8	83.8
71	50,910	173.4	-24.4	43.5	120	TpSS	130.3	157.4	TpSH	24	158				9	205.0	81.8
72	51,733	202.8	-23.9	12.6	120	TpSS	211.1	157.4	TpSH	3	158				35	244.5	83.1
73	52,194	207.9	-23.7	2.9	120	TpSS	218.7	157.4	TpSH	10	158				32	252.4	86.5
74	53,193	237.1	-23.2	15	120	TpSS	236.8	157.4	TpSH	8.5	158				42.6	280.7	94.3
76	53,841	222.3	-22.9	6	120	TpSS	239.2	157.4	TpSH	2	158				34.5	266.5	91.3
75	54,950	228.2	-22.3	11.8	120	TpSS	238.7	157.4	TpSH	2	158				23.8	270.7	98.2

PASSAIC TUNNEL PROJECT
TOTAL VERTICAL STRESS AND HYDROSTATIC PRESSURE
AS MEASURED AT TUNNEL INVERT

PRINT DATE 07-Feb-95

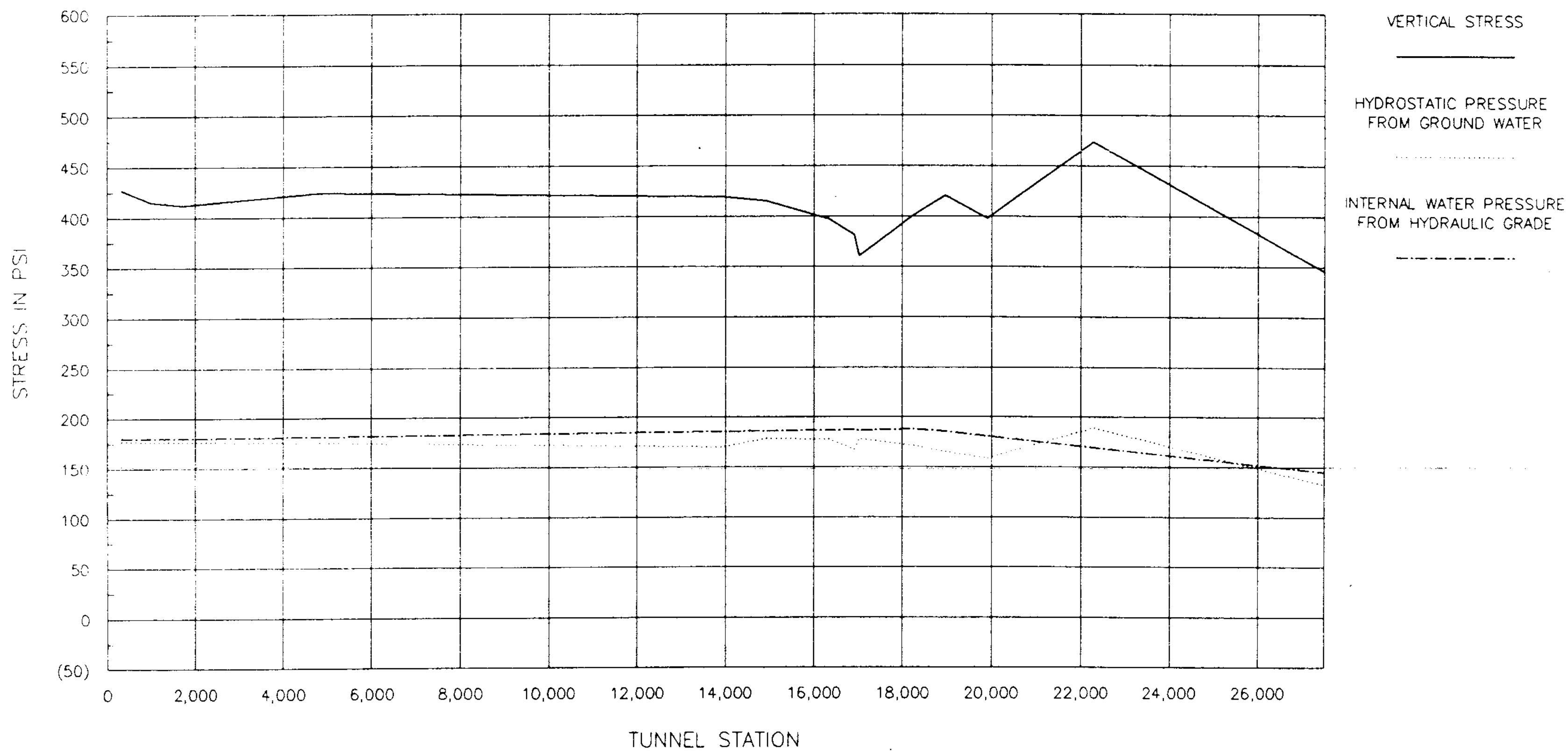
HOLE NUMBER	TUNNEL STATION	TOP ELEV.	INVERT ELEV	THICK. OVERB.	DENSITY OVERB.	ROCK LAYER DATA: LAYER 1 TYPE	LAYER 1 THICKNE	LAYER 1 DENSITY	LAYER 2 TYPE	LAYER 2 THICKNE	LAYER 2 DENSITY	LAYER 3 TYPE	LAYER 3 THICKNE	LAYER 3 DENSITY	WATER DEPTH	VERTICAL STRESS (PSI)	HYDROSTATIC PRESSURE (PSI)
77	55,392	239.4	-22.1	15	120	TpSS	242	157.4	TpSH	4.5	158				15	282.0	106.8
78	56,073	243.1	-21.8	16	120	TpSS	244.9	157.4	TpSH	4	158				25.4	285.4	103.8
80	56,747	245.2	-21.4	18.9	120	TpSS	247.7	157.4	TpSH	16	158				29	286.5	103.0
79	57,168	230.2	-21.2	6	120	TpSS	243.4	157.4	TpSH	2	158				11.7	273.2	103.9
81	57,885	239.8	-20.9	9	120	TpSS	231.7	157.4	TpSH	20	158	TpSH	2.5	158	18.1	282.7	105.1
82	58,356	242.6	-20.6	15	120	TpSS	245.6	157.4	TpSH	2.5	158	TpSH	2.5	158	15.4	283.7	107.4
83	59,512	310	-20.1	13.7	120	TpSS	300.4	157.4	TpSH	16	158	TpSH	12	158	73	357.3	111.4
PT-40	60,113	398.9	-19.7	22.9	120	TpSS	389.7	157.4	TpSH	6	158	TpSH	5	158	28.5	451.6	181.4
84	60,461	433.3	-19.6	0.3	120	TpSS	445.1	157.4	TpSH	7.5	158	Jo	1.5	181	183.5	495.0	116.7
PT-39	60,495	385.9	-19.6	22.8	120	Jo	23.5	181	TpSS	356.7	157.4	TpSH	2.5	158	36.7	441.2	175.7
PT-38	61,028	469.6	-19.3	28.5	120	Jo	55	181	TpSS	393.6	157.4	TpSH	12	158	29.7	536.3	211.9
85	62,075	330.2	-18.8	2	120	Jo	5	181	TpSS	336.8	157.4	TpSH	5	158	28.5	381.6	138.9
85a	62,075	330	-18.8	10	120	TpSS	325.8	157.4	TpSH	13	158	Jo	1.5	181	22.6	380.6	141.4
86	63,163	322.4	-18.2	20.2	120	TpSS	318.4	157.4	TpSH	2	158				36.7	367.1	131.7
87	64,099	324.5	-17.8	8.5	120	TpSS	268.3	157.4	TpSH	5	158	Jo	60.5	181	29.7	381.9	135.5
PT-35	64,422	377.8	-17.6	11.8	120	Jo	175	181	TpSS	205.6	157.4	TpSH	3	158	-5.8	457.8	171.3
PT-29	65,941	330.9	-16.8	19	120	Jo	328.7	181		60	165.4	JbSIS	20	158.3	1	429.0	150.7
88	65,953	332.8	-16.8	6.5	120	Jo	343.1	181	JtSH	60	165.4	JbSIS	40	158.3	92.4	436.7	111.5
89	67,010	235.3	-16.3	30.4	120	Jo	221.2	181		64.2	165.4				-3	303.4	110.3
90	68,023	175.1	-15.8	19.2	120	Jo	171.7	181	JtSH	75	165.4				-5.8	231.8	85.2
91	68,870	138.3	-15.4	49.7	120	Jo	104	181		43	165.4	JpBAS	93.8	181.5	1	172.1	66.2
PT-26	70,341	184.5	-14.6	52.6	120	JtSS	86.5	152.7	JtSH	60	165.4	Jp BAS	9		49.2	204.5	86.3
92	71,076	140.9	-14.3	40	120	JtSS	51	152.7	JtSH	64.2	165.4				-4	161.2	69.0
93	71,521	147.4	-14	12.6	120	JtSS	73.8	152.7	JtSH	75	165.4				-11	174.9	74.7
PT-23	71,616	192.8	-14	55	120	JtSS	108.8	152.7	JtSH	43	165.4				8	210.6	89.6
~~94~~	~~73,145~~	~~181.1~~	~~-13.2~~	~~3~~	~~120~~	~~Jp~~	~~149~~	~~181.5~~	~~JtSS~~	~~42.2~~	~~152.7~~				~~49.2~~	~~235.1~~	~~62.9~~
95	74,335	184.8	-12.6	11	120	Jp	186.4	181.5	Jp	156.9	181.5	Jh	156.6	179.6	6.6	244.1	82.7
141	75,022	152.2	-12.3	27.3	120	Jp	137.2	181.5	JpSH	3.5	155.7				4.3	195.7	69.4
97	75,826	180	-11.9	25.4	120	Jp	166.5	181.5	JpSH	3.5	155.7				8	231.0	79.7
PT-20	75,873	169.6	-11.9	29	120	Jp	152.5	181.5		136	160.1				7	216.4	78.6
PT-19	77,357	171.3	-11.1	28.2	120	JtSS	9	154.8	Jp	156.9	181.5	JpBAS	103.3	181.5	7	207.4	79.0
98	78,041	170	-10.7	33	120	Jp	137.7	181.5	JtSH	26	155.7	JpBAS	103.3	181.5	7	201.1	75.3
99	79,023	180	-10.3	16.5	120	Jp	160	191.8	JpSH	3.5	155.7	Jp	88.6	181.5	10	230.6	78.1
PT-18	79,477	175.8	-10.1	32.4	120	Jp	153.5	181.5		84.1	155.7	Jp BAS	9		9.8	220.5	80.6
123	80,012	175.4	-9.8	28.2	120	Jp BAS	157	181.5	JtSH	34	155.7				7	221.4	77.2
124	80,627	175.7	-9.5	50.4	120	JtSS	5.5	154.8	JtSH	26	155.7	JpBAS	103.3	181.5	10.1	206.2	75.9
PT-17	80,909	176.5	-9.4	62.3	120	JtSS	31	154.8	JtSH	4	155.7	Jp	88.6	181.5	10.5	201.2	80.6
125	81,204	179.2	-9.2	52.1	120	JtSS	43.2	154.8	JtSH	84.1	155.7	Jp BAS	9		9.8	180.8	77.4
136	81,686	177.9	-9	69	120	JtSS	83.9	154.8	JtSH	34	155.7				9.2	184.5	77.0
100	81,737	177.4	-8.9	145	120	Jt	41.3	155	JtSH	53.9	155.7				7.9	165.3	80.7
126	82,591	172.2	-8.5	67.8	120	JtSS	104.9	154.8	JtSH	8	155.7				10.5	177.9	73.8
PT-16	82,682	172.3	-8.5	50.3	120	JtSS	119.5	154.8	JtSH	11	155.7				2.1	182.3	78.3

PASSAIC TUNNEL PROJECT
TOTAL VERTICAL STRESS AND HYDROSTATIC PRESSURE
AS MEASURED AT TUNNEL INVERT

PRINT DATE 07-Feb-95

HOLE NUMBER	TUNNEL STATION	TOP ELEV.	INVERT ELEV	THICK. OVERB.	DENSITY OVERB.	ROCK LAYER DATA: LAYER 1 TYPE	LAYER 1 THICKNE	LAYER 1 DENSITY	LAYER 2 TYPE	LAYER 2 THICKNE	LAYER 2 DENSITY	LAYER 3 TYPE	LAYER 3 THICKNE	LAYER 3 DENSITY	WATER DEPTH	VERTICAL STRESS (PSI)	HYDROSTATIC PRESSURE (PSI)
127	83,009	174.7	-8.3	90.8	120	JtSS	90.2	154.8	JtSH	2	155.7				7.5	174.8	76.0
101	83,721	180	-7.9	118	120	JtSS	16	154.8	JtSH	53.9	155.7				7.9	173.8	78.0
PT-15	84,524	175.6	-7.5	80	120	JtSS	70.1	154.8	JtSH	33	155.7				34.5	177.7	79.3
102	84,337	180	-7.4	86.7	120	JtSS	58.2	154.8	JtSH	42.5	155.7	JtSH	12	155.7	2.1	180.8	80.3
128	85,307	188.4	-7.2	35.4	120	JtSS	128.2	154.8	JtSH	32	155.7	JtSH	12	155.7	1.2	201.9	84.2
PT-14	85,987	229.4	-6.8	43	120	JtSS	160.7	154.8	JtSH	32.5	155.7	Jh	156.6	179.6	42	243.7	102.4
129	86,083	237	-6.8	40.6	120	JtSS	186.2	154.8	JtSH	17	155.7	Jh	205.3	179.6	34.5	252.4	90.7
PT-13	86,944	313.7	-6.3	12.5	120	JtSS	224.5	154.8	JtSH	83	155.7				1.5	341.5	138.7
103	87,995	344.6	-5.8	5.5	120	Jh	150.5	179.6	JtSS	182.4	154.8	JtSH	12	155.7	70.5	401.3	121.3
130	88,342	311.3	-5.6	5.5	120	JtSS	99.2	154.8	JtSH	55.6	155.7	Jh	156.6	179.6	42	366.7	119.1
PT-12	89,101	241.3	-5.3	8	120	JtSS	29.3	154.8	JtSH	4	155.7	Jh	205.3	179.6	-12.2	298.5	106.9
131	89,896	177.4	-4.9	12.5	120	Jh	169.8	179.6	JbSH	12.5	160.1				1.5	222.2	78.3
PT-11	90,154	180.9	-4.7	10	120	Jh	175.6	179.6		69	160.1	JbSH	99.7	160.1	20.5	227.3	80.4
104	91,065	167.6	-4.3	101.2	120	JbSS	14.5	155.3	JbSH	8	160.1	Jh	48.2	179.6	-8	169.0	78.0
PT-10	91,324	168	-4.1	91	120	JbSS	24	155.3	JbSH	19	160.1	Jh	38.1	179.6	36.2	170.4	74.6
132	92,018	167.1	-3.8	65.6	120	JbSS	92.8	155.3	JbSH	12.5	160.1	JbSH	113	160.1	-12.2	168.6	79.3
105	92,749	184.8	-3.4	32.5	120	JbSS	90.7	155.3	JbSH	69	160.1	JbSH	105	160.1	20.5	201.6	72.7
PT-9	93,483	228.5	-3.1	40.9	120	JbSS	20	155.3	JbSIS	71	158.3	JbSH	99.7	160.1	4.6	244.6	100.4
106	94,491	258.6	-2.6	17	120	JbSS	213.6	155.3	JbSH	31	160.1	JbSH	43	160.1	36.2	279.0	97.5
PT-8	95,171	269.6	-2.2	15	120	JbSS	12	155.3	JbSIS	131.8	158.3	JbSH	113	160.1	5	296.0	117.8
107	95,974	265.3	-1.8	35.2	120	JbSS	99.9	155.3	JbSIS	27	158.3	JbSH	105	160.1	9.7	283.5	111.5
133	96,760	265.7	-1.4	27.3	120	JbSS	103.8	155.3	JbSH	136	160.1	JbSH	37	160.1	4.6	285.9	113.7
PT-7	97,285	271.9	-1.2	40	120	JbSS	33.2	155.3	JbSIS	156.9	158.3	JbSH	43	160.1	24.3	289.4	118.3
108	97,955	255.8	-0.8	24.7	120	JbSS	189.4	155.3	JbSH	14	160.1	JbSIS	28.5	158.3	5	271.7	109.0
109	98,713	239.1	-0.5	9	120	JbSS	188.6	155.3	JbSH	42	160.1	JbSH	22.5	160.1	2	257.6	103.0
134	99,315	237	-0.2	26.7	120	JbSS	176.5	155.3	JbSH	34	160.1	JbSH	25	160.1	4	250.4	101.1
PT-5	99,772	251.7	0.1	31.4	120	JbSS	70.9	155.3	JbSIS	112.6	158.3	JbSH	37	160.1	2.6	267.5	109.0
110	101,373	263.6	0.9	48	120	JbSS	178.1	155.3	JbSIS	13	158.3	JbSH	26	160.1	24.3	273.1	103.3
111	102,281	228.8	1.3	30.5	120	JbSS	160.2	155.3	JbSIS	14.5	158.3	JbSH	22.5	160.1	3.2	239.1	97.2
PT-3	103,221	210.4	1.8	18.6	120	JbSS	68.2	155.3	JbSIS	96.8	158.3	JbSH	25	160.1	13.7	223.3	90.4
112	103,725	209.5	2.1	16.3	120	JbSS	145.6	155.3	JbSH	22.5	160.1	JbSIS	20	158.3	2.6	217.6	88.7
PT-2	104,852	222.9	2.6	13.6	120	JbSS	68.4	155.3	JbSIS	104	158.3	JbSH	34.3	160.1	0	237.6	95.5
135	105,729	195.6	3.1	8	120	JbSS	144	155.3	JbSH	40.5	160.1				1	207.0	83.0
PT-1	106,109	187.3	3.2	38.5	120	JbSS	30.8	155.3	JbSIS	39	158.3	JbSH	75.5	160.1	-2	192.1	79.8
113	106,785	179.7	3.6	81.8	120	JbSS	22.3	155.3	JbSH	22	160.1	JbSIS	40	158.3	13.7	160.6	70.4
147	107,159	186.1	3.8	61.7	120	JbSS	117.6	155.3	JbSH		160.1				0	178.2	79.0
147A	107,159	186.1	3.8	61	120	JbSS	112.3	155.3	JbSH	9	160.1				3.8	182.0	77.3
114	107,571	180	4	61.5	120	JbSS	99.5	155.3	JbSH	15	160.1				-2	175.2	77.1

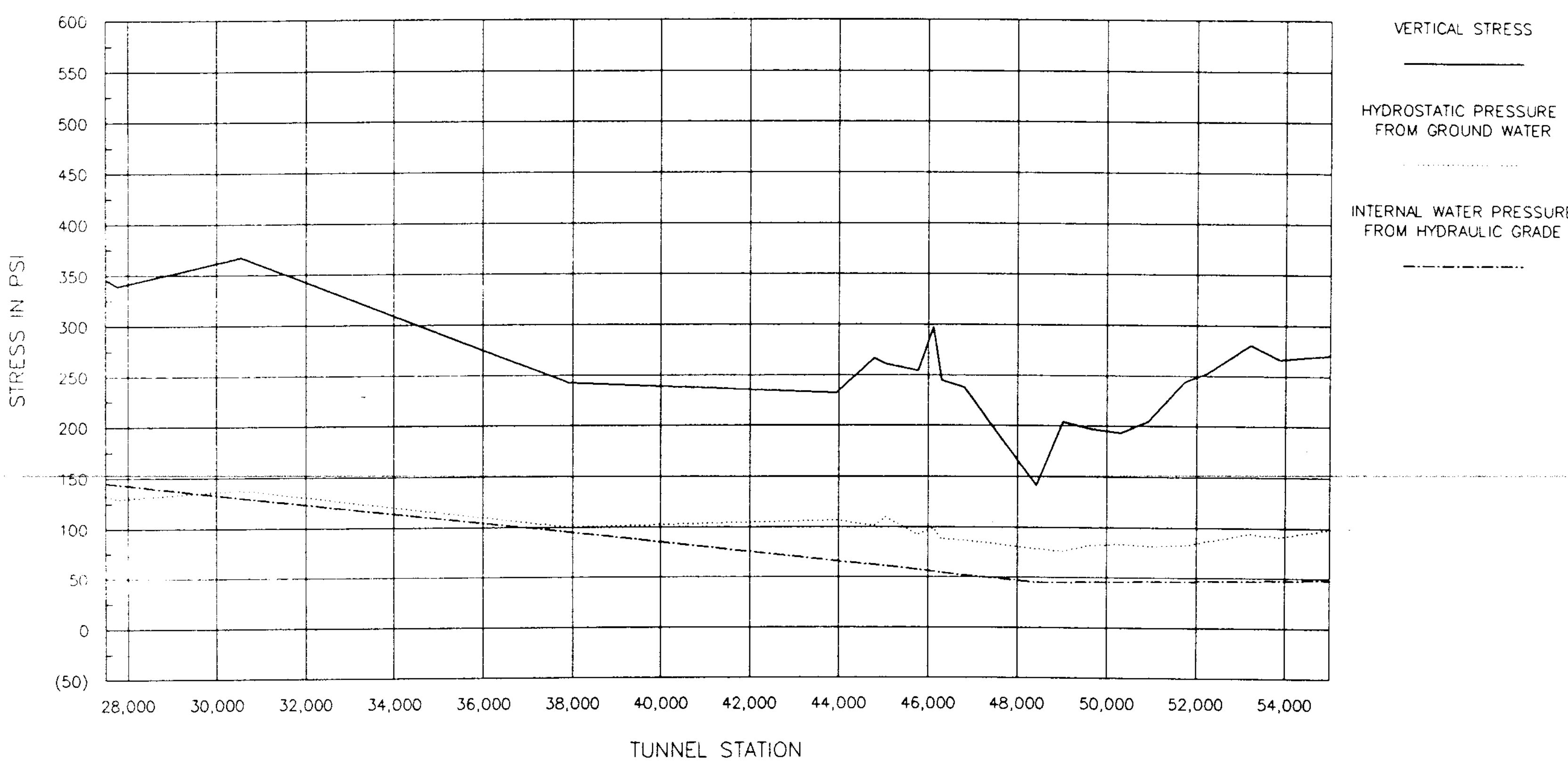
PASSAIC TUNNEL PROJECT
STRESS DISTRIBUTION AT TUNNEL INVERT



The vertical stress is based on the gravity load of the overlying material. Hydrostatic pressure is based on the water level data

SHEET 1 OF 4

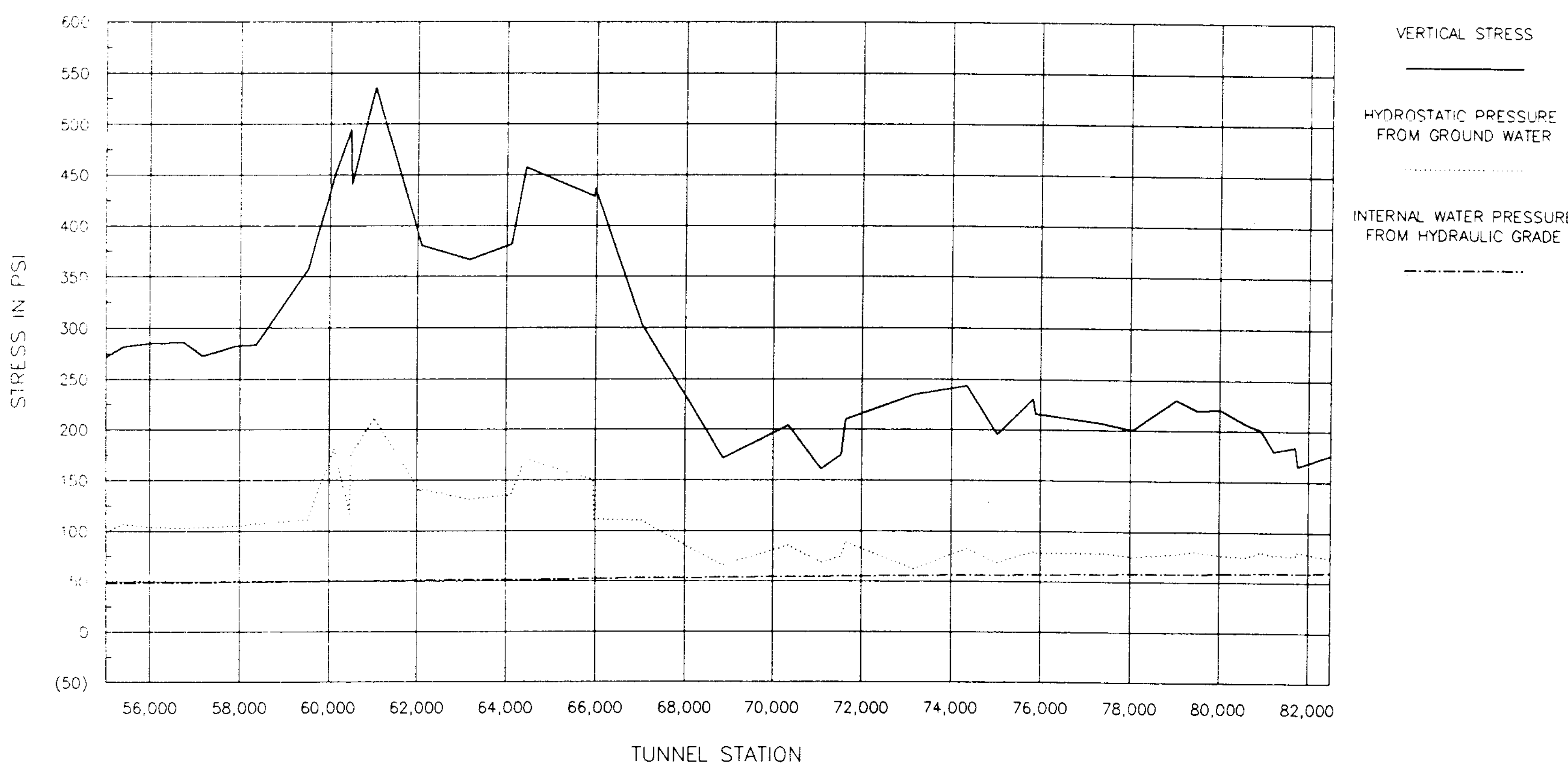
PASSAIC TUNNEL PROJECT
STRESS DISTRIBUTION AT TUNNEL INVERT



The vertical stress is based on the gravity load of the overlying material. Hydrostatic pressure is based on the water level data.

SHEET 2 OF 4

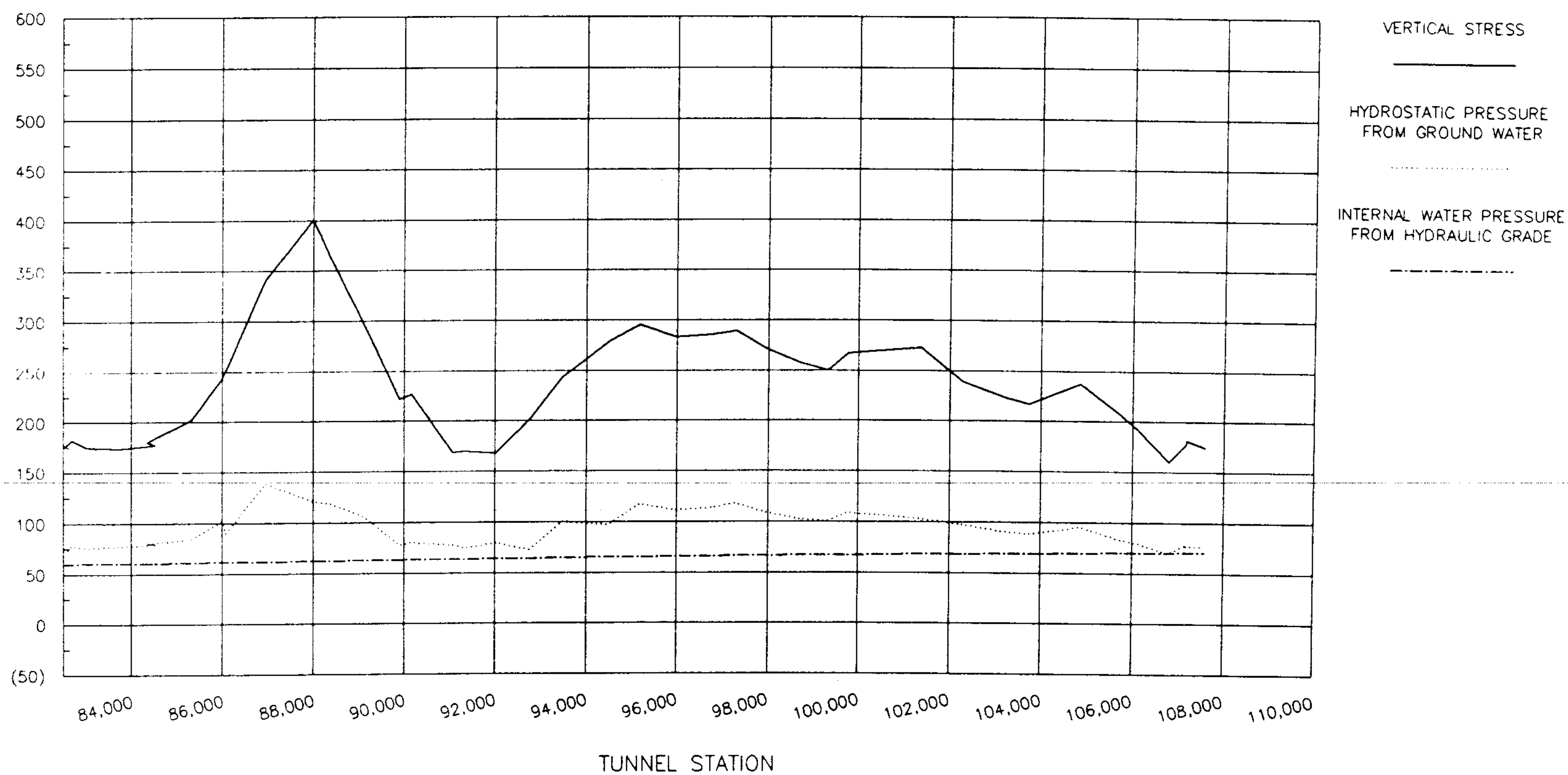
PASSAIC TUNNEL PROJECT
STRESS DISTRIBUTION AT TUNNEL INVERT



The vertical stress is based on the gravity load of the overlying material. Hydrostatic pressure is based on the water level data

SHEET 3 OF 4

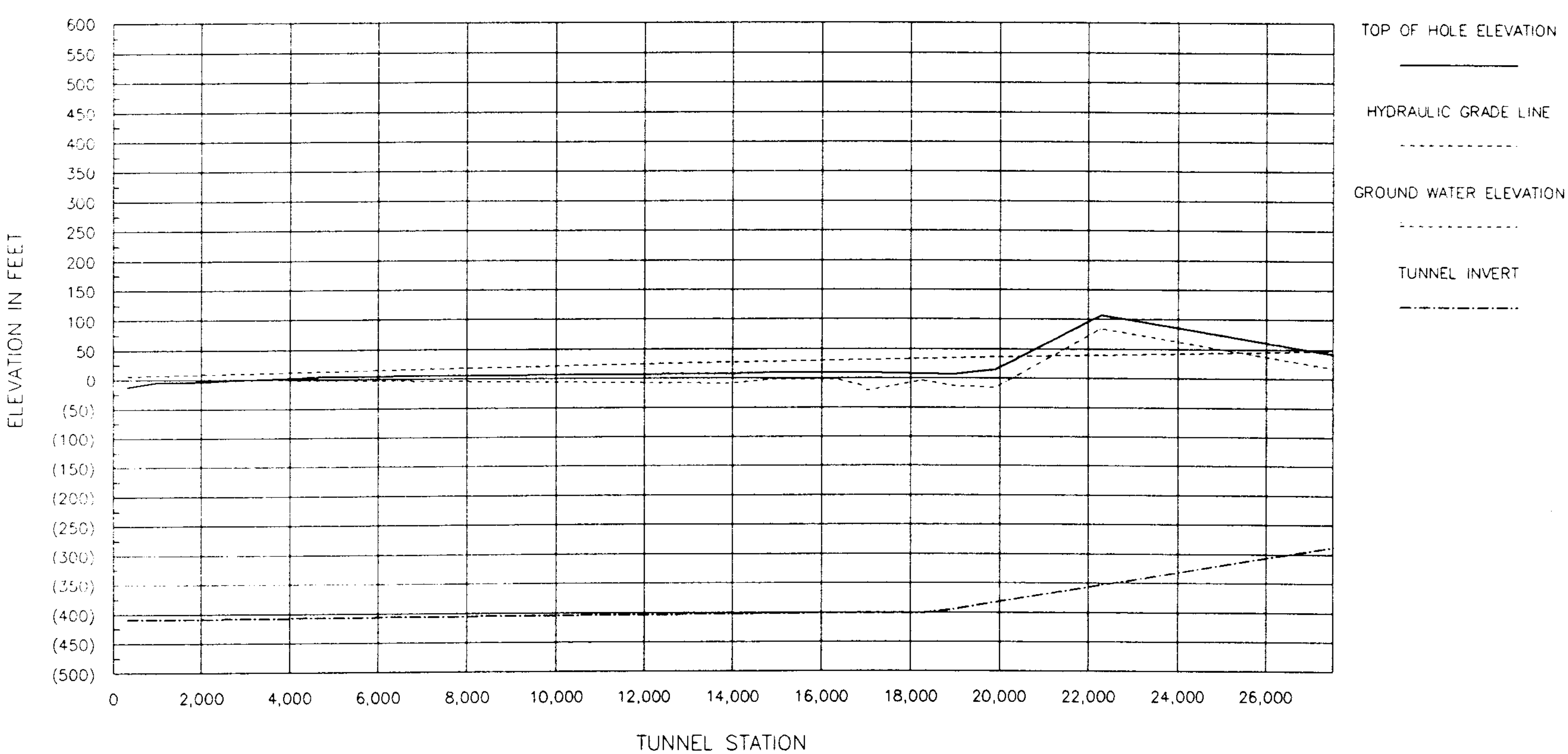
PASSAIC TUNNEL PROJECT
STRESS DISTRIBUTION AT TUNNEL INVERT



The vertical stress is based on the gravity load of the overlying material. Hydrostatic pressure is based on the water level data

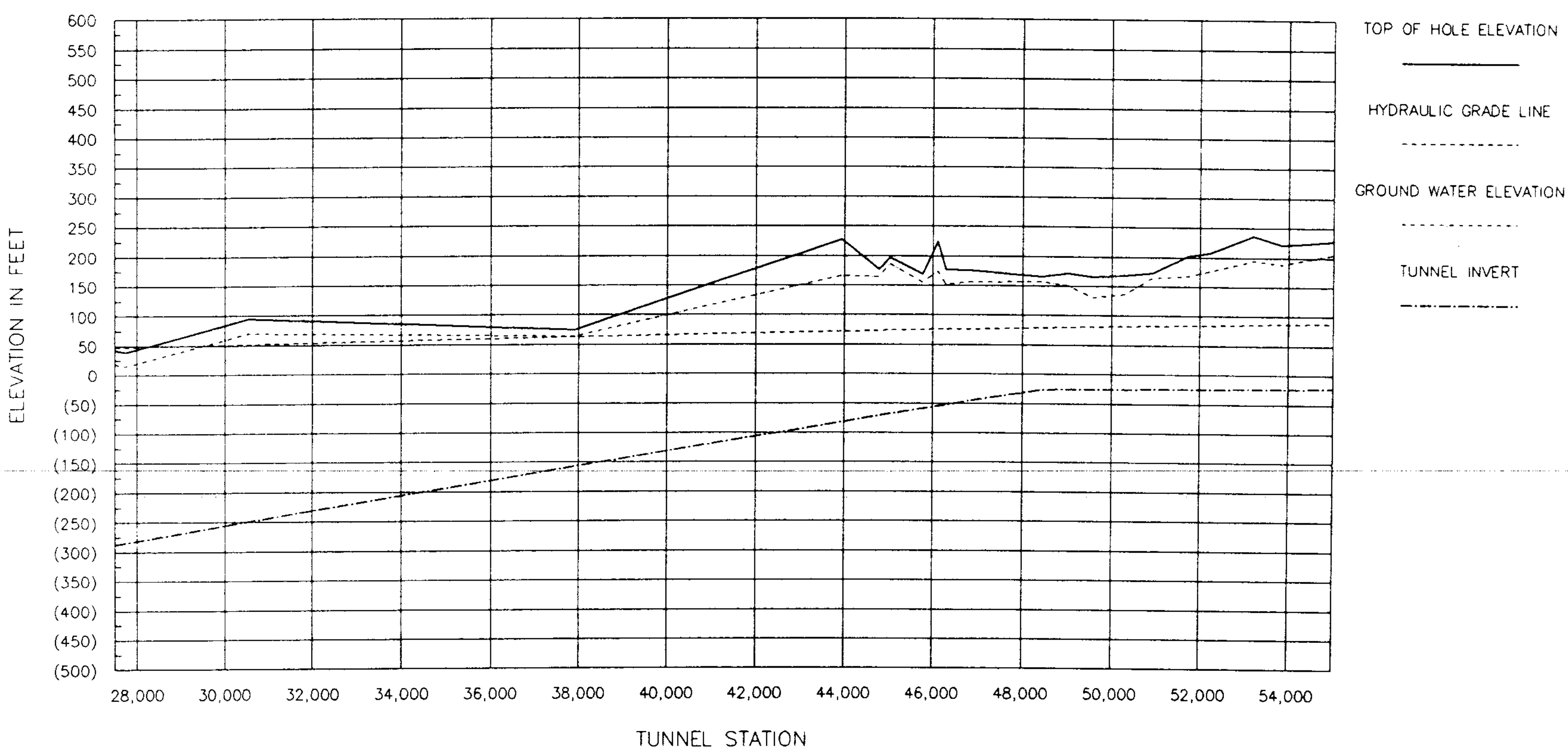
SHEET 4 OF 4

PASSAIC TUNNEL PROJECT
ELEVATIONS



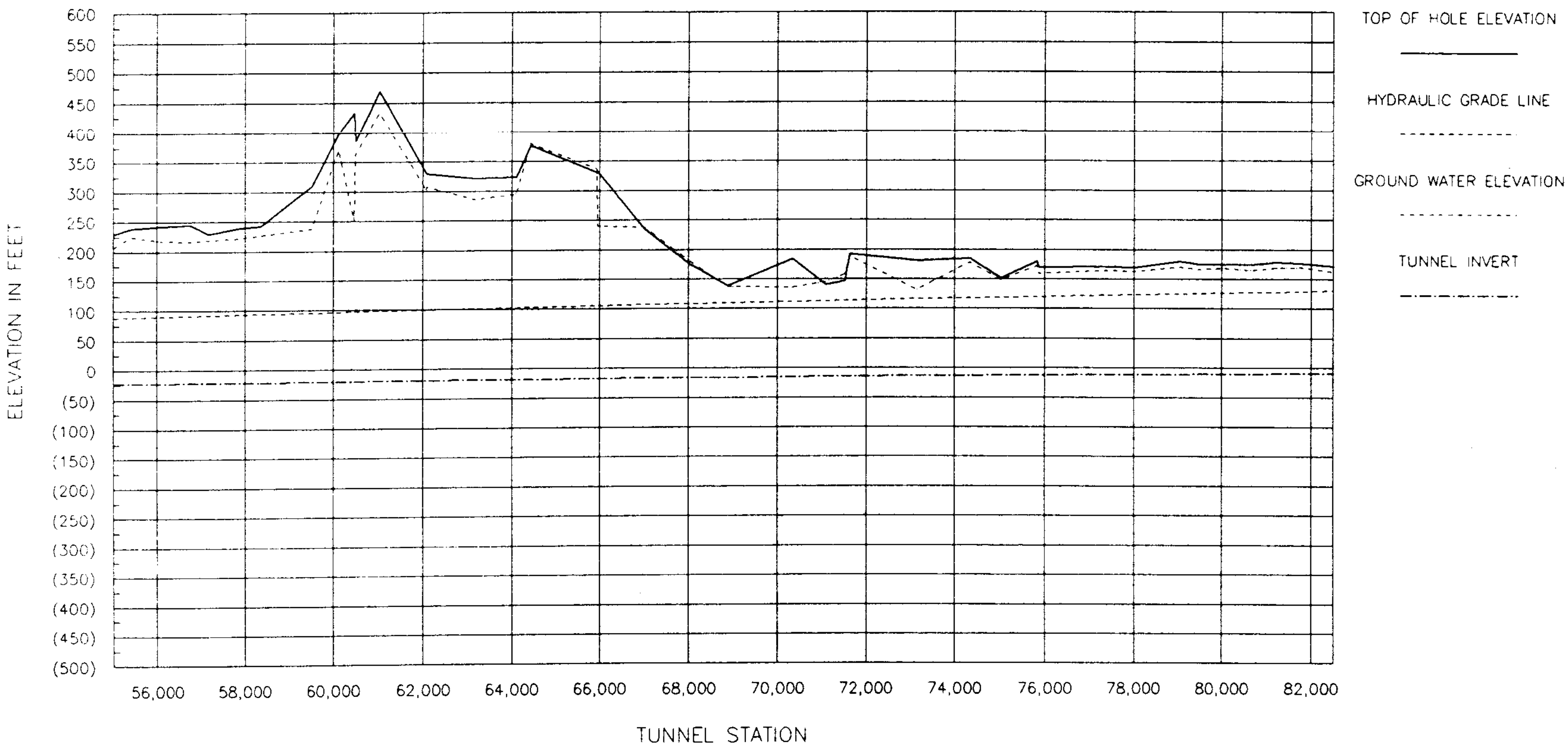
SHEET 1 OF 4

PASSAIC TUNNEL PROJECT
ELEVATIONS



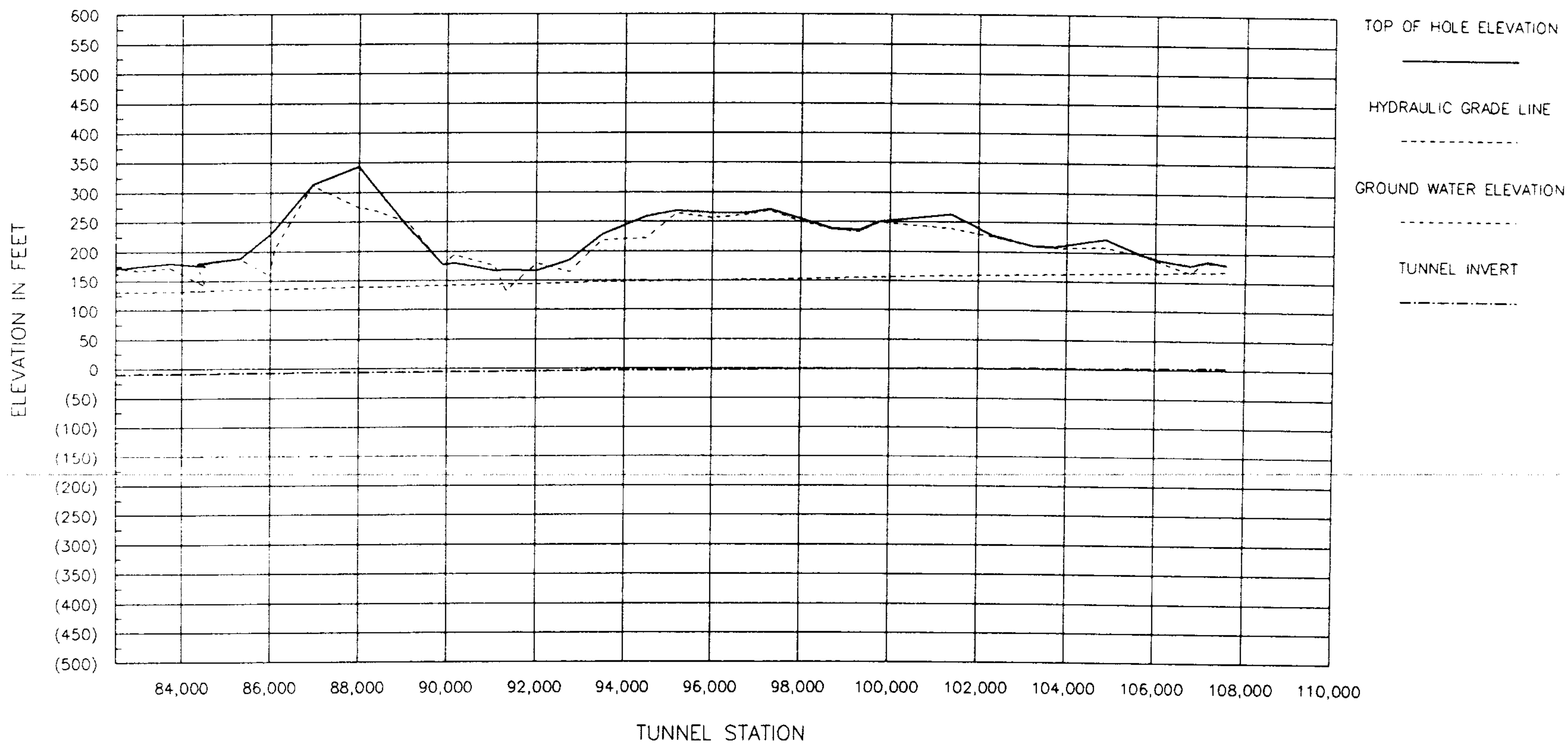
SHEET 2 OF 4

PASSAIC TUNNEL PROJECT
ELEVATIONS



SHEET 3 OF 4

PASSAIC TUNNEL PROJECT
ELEVATIONS



SHEET 4 OF 4

PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

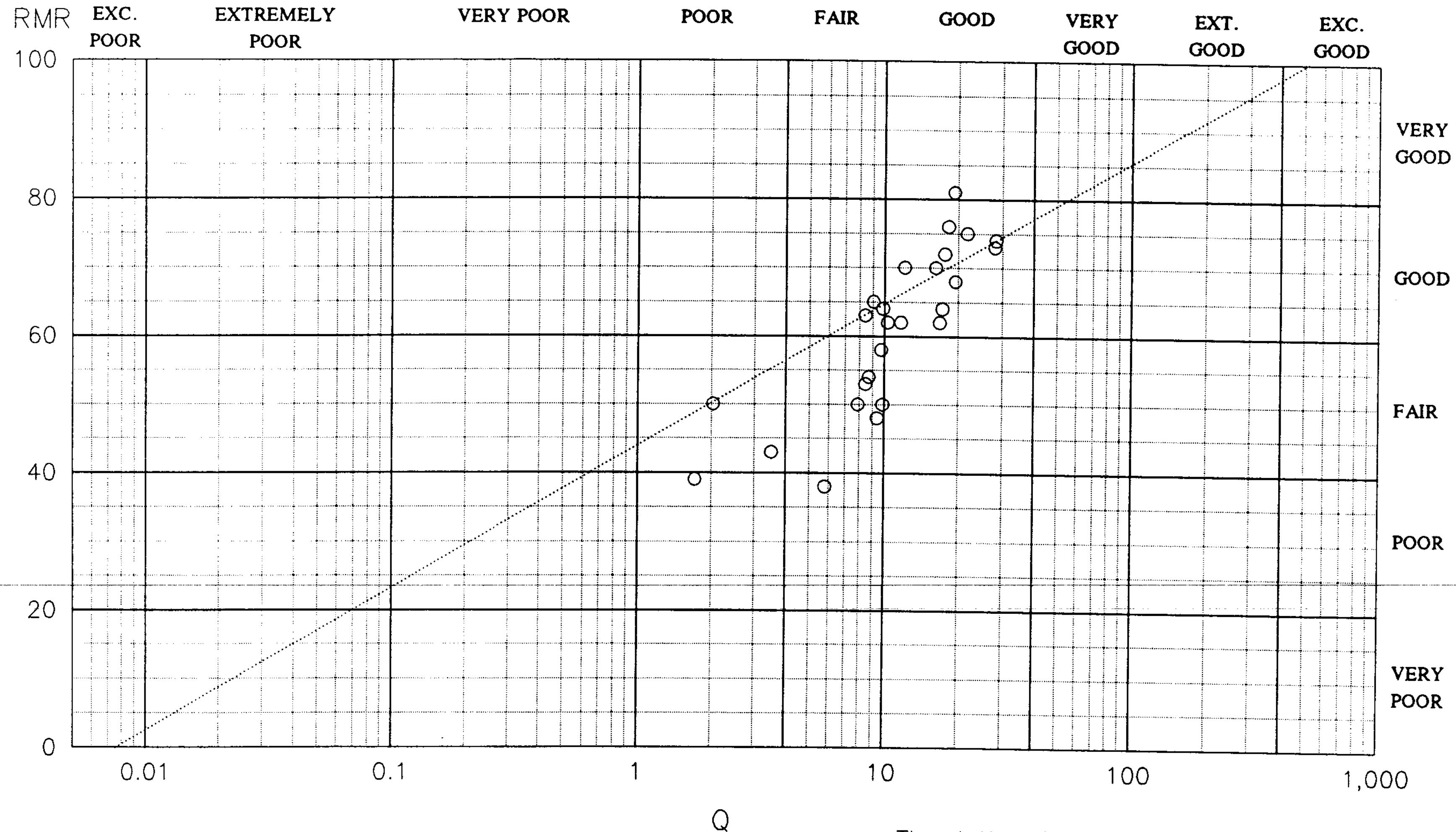
ATTACHMENT E.3.11

ROCK MASS CLASSIFICATION SYSTEM STUDIES

E.3.11

ROCK MASS CLASSIFICATION SYSTEM STUDIES

PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES SPUR TUNNEL
TOWACO FORMATION

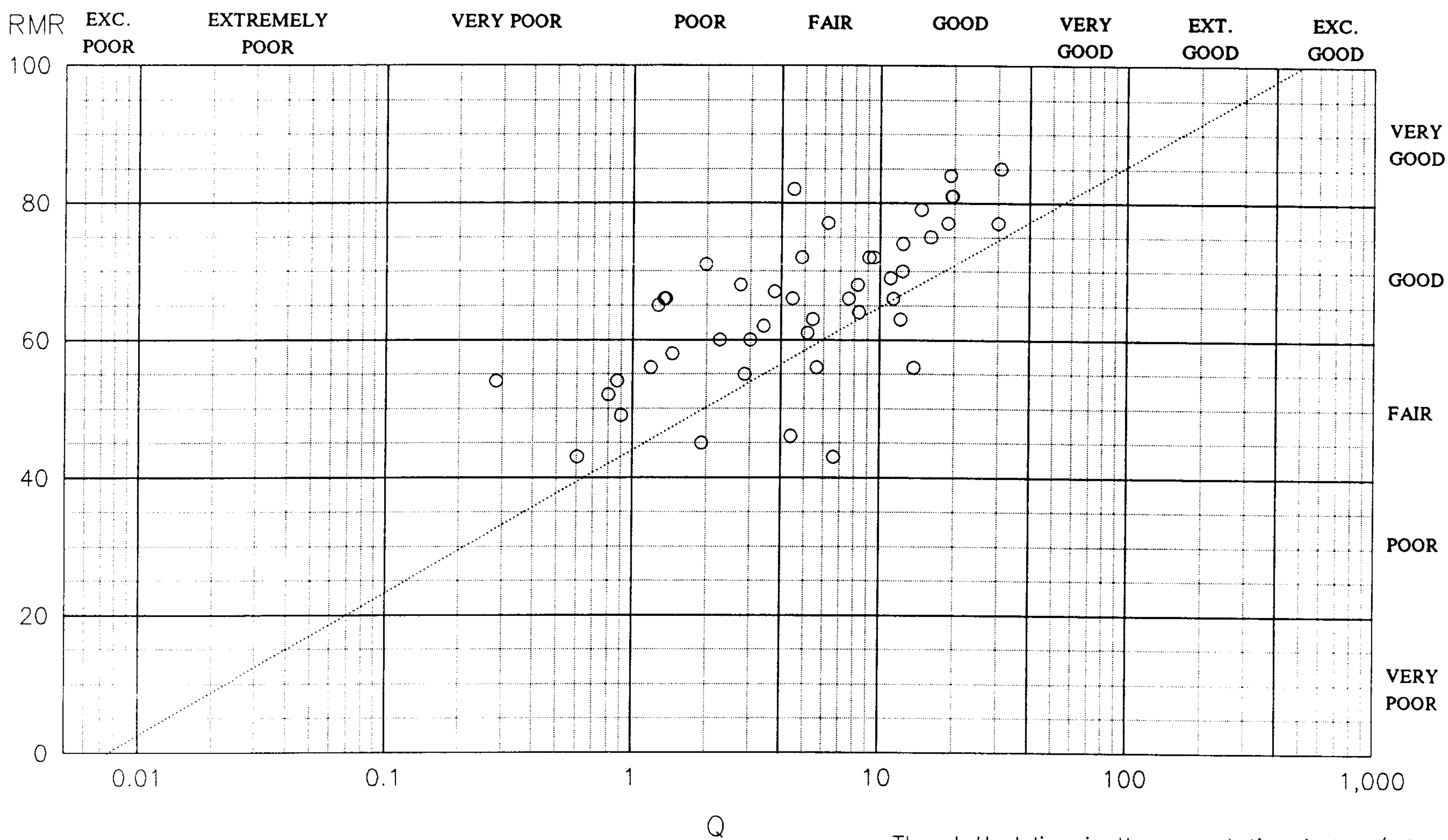


These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)

RMR = 9 ln Q + 44

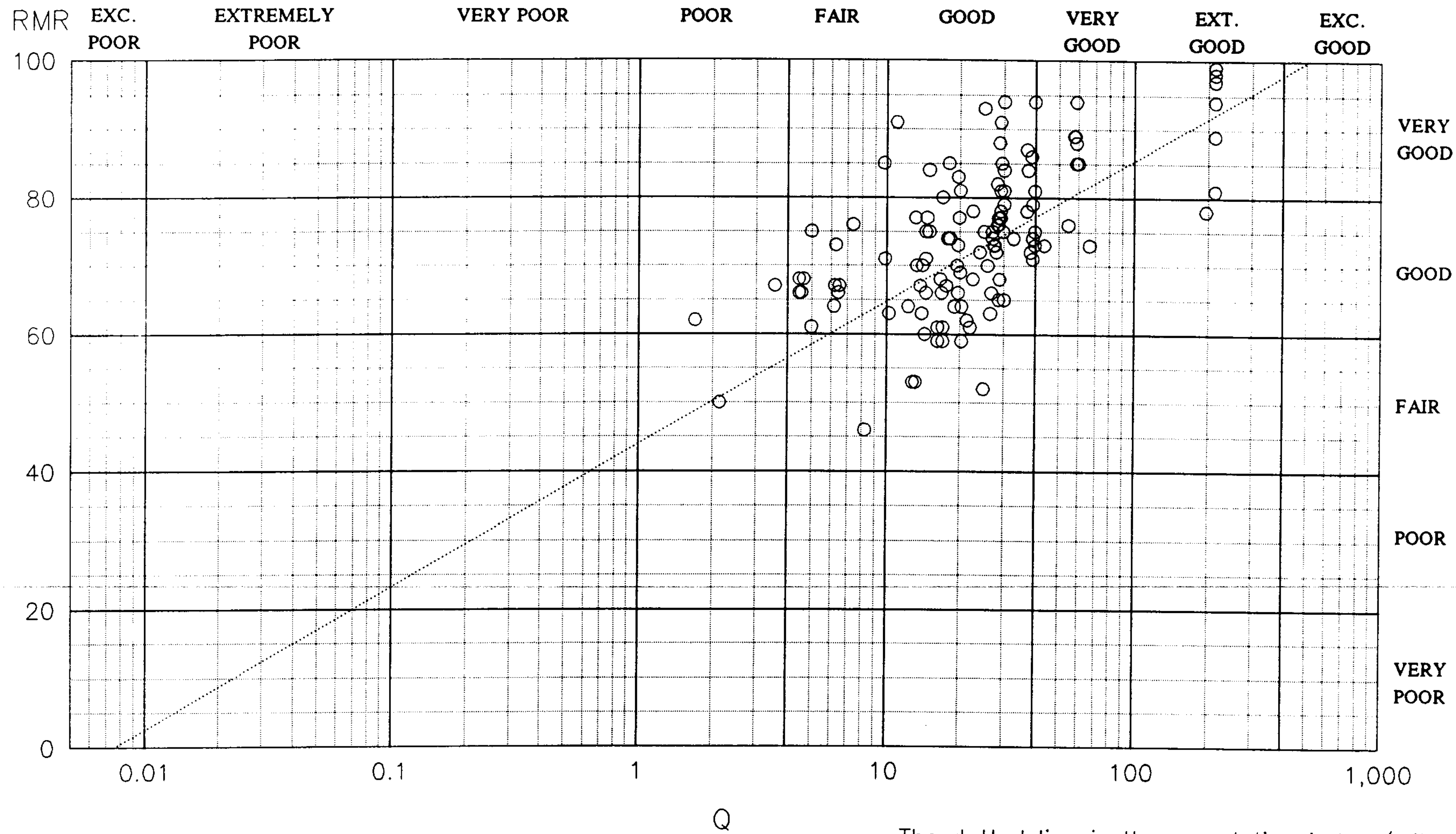
PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES SPUR TUNNEL
PREAKNESS MOUNTAIN BASALT



These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)
RMR = 9 ln Q + 44

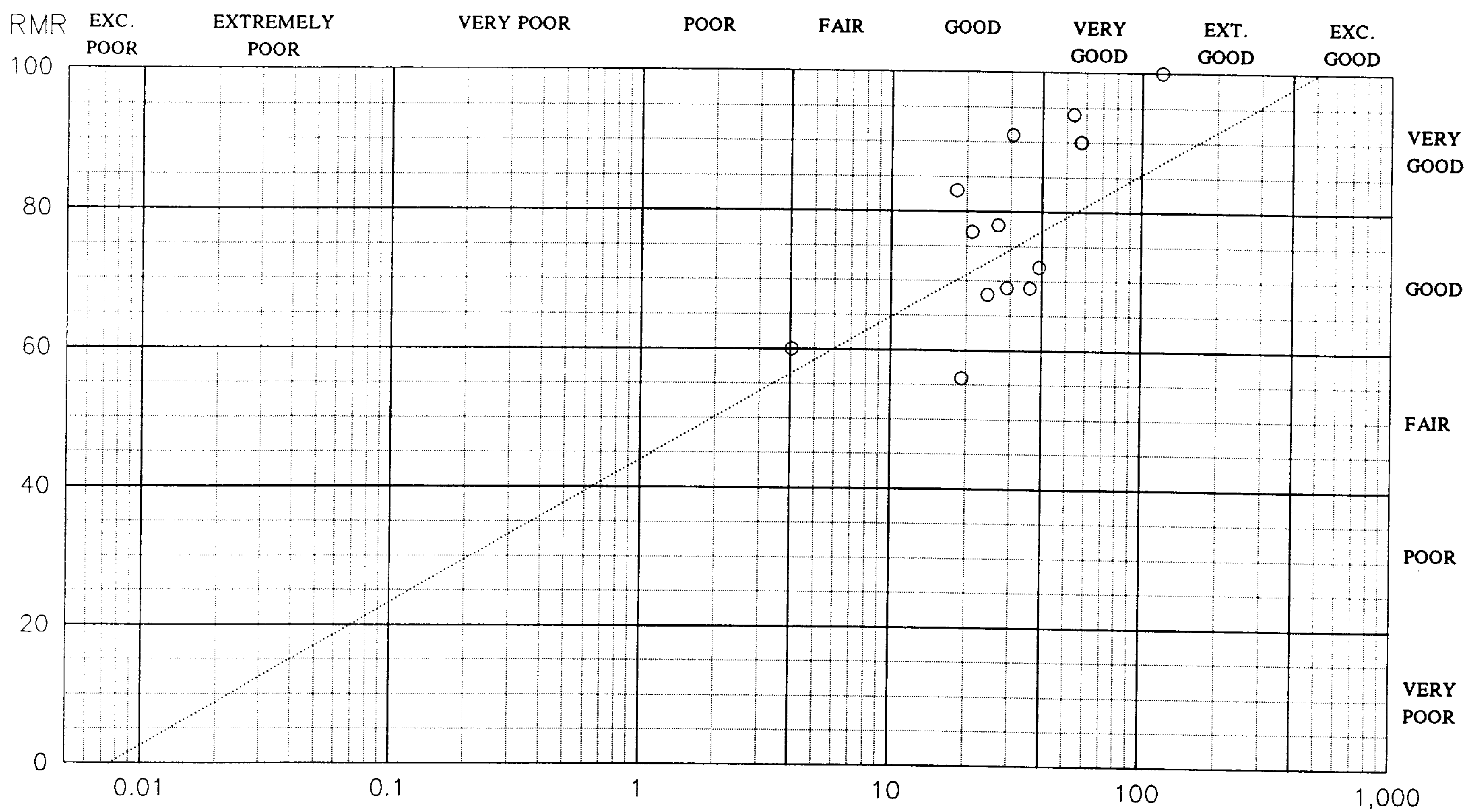
PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
BOONTON FORMATION



These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)
RMR = 9 ln Q + 44

PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
HOOK MOUNTAIN BASALT

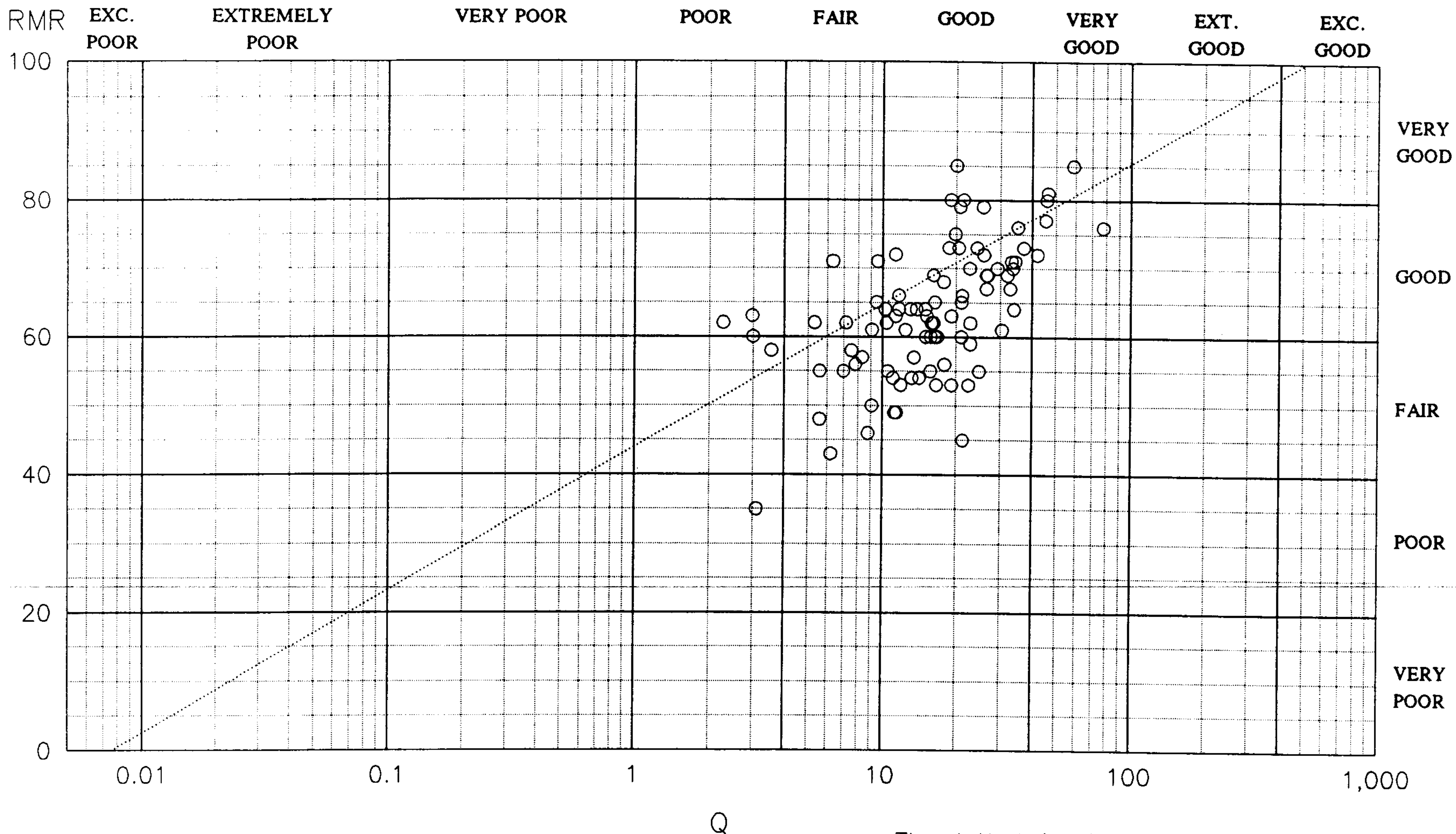


These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)

$$RMR = 9 \ln Q + 44$$

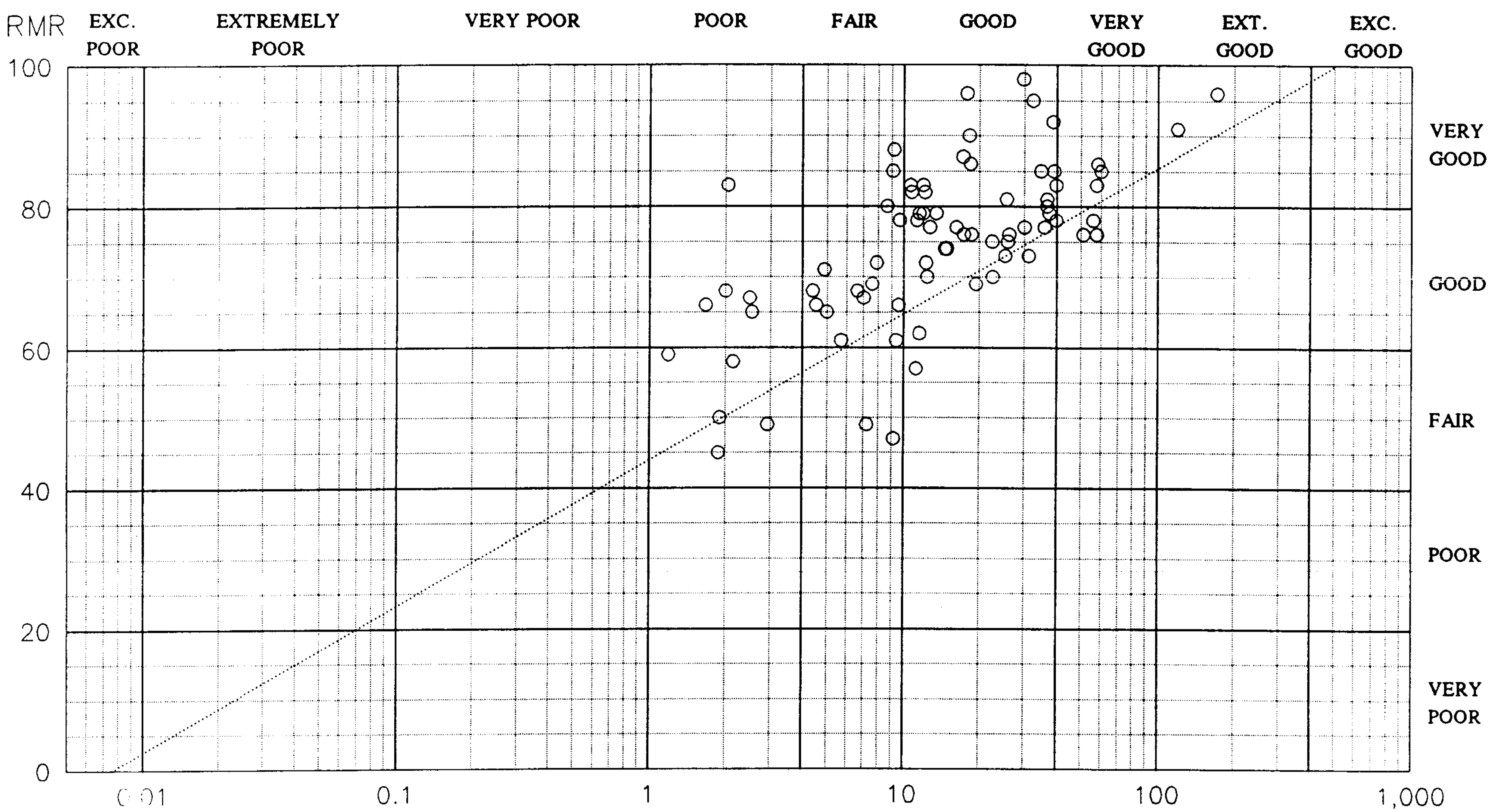
PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
TOWACO FORMATION



These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)
RMR = 9 ln Q + 44

PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
PREAKNESS MOUNTAIN BASALT

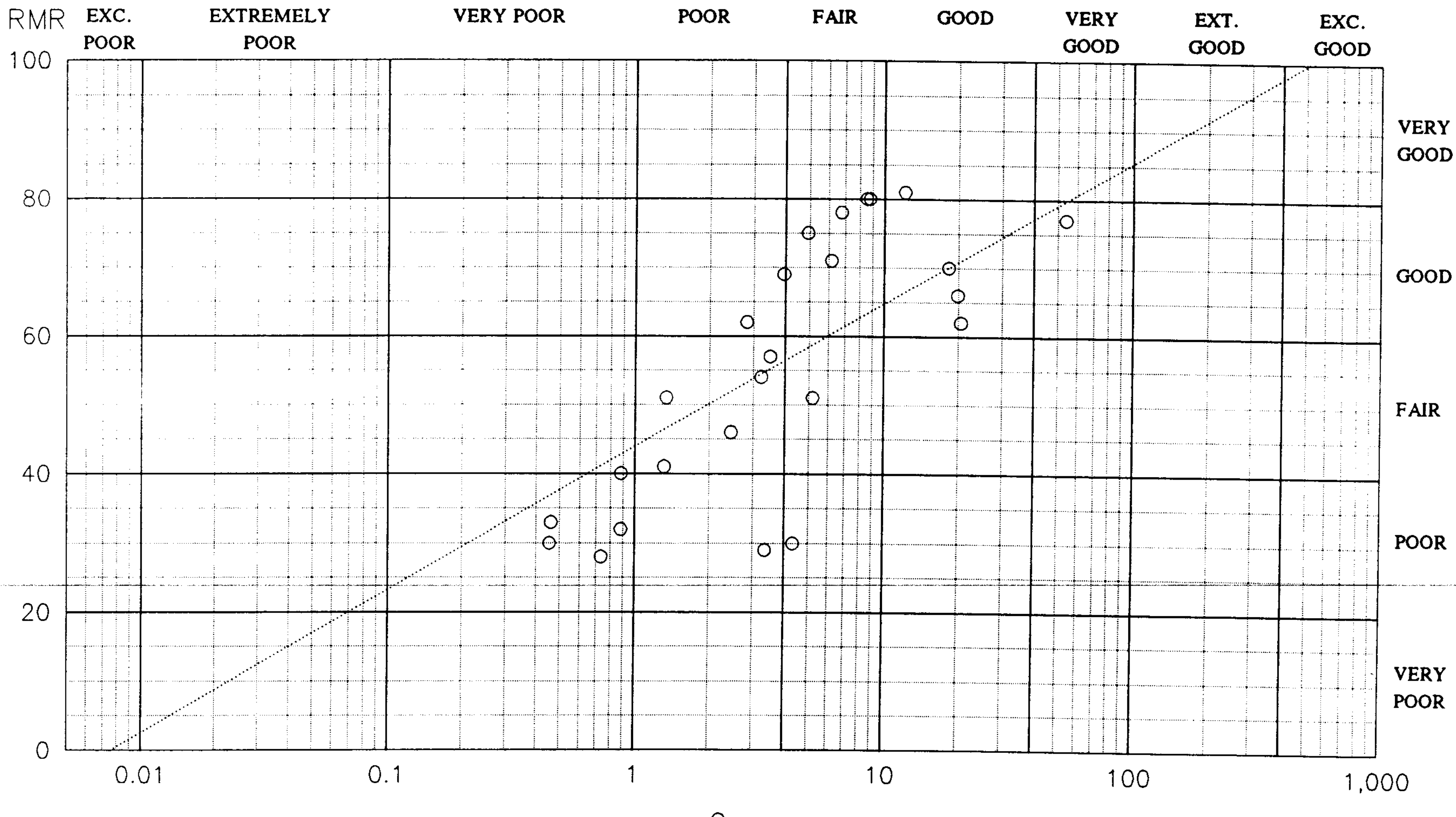


These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)

$$RMR = 9 \ln Q + 44$$

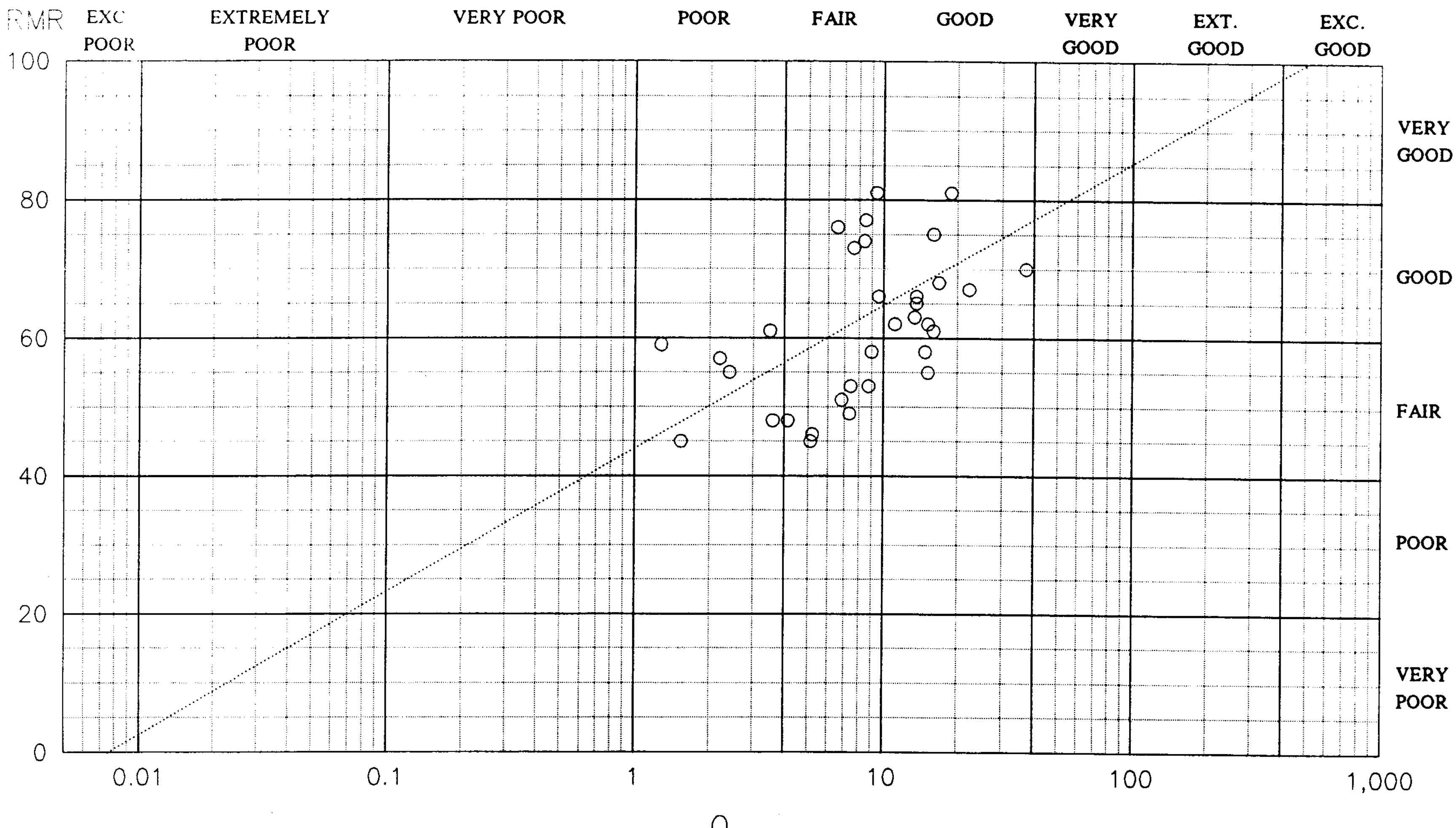
PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
FELTVILLE FORMATION



These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)
RMR = 9 ln Q + 44

PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
ORANGE MOUNTAIN BASALT

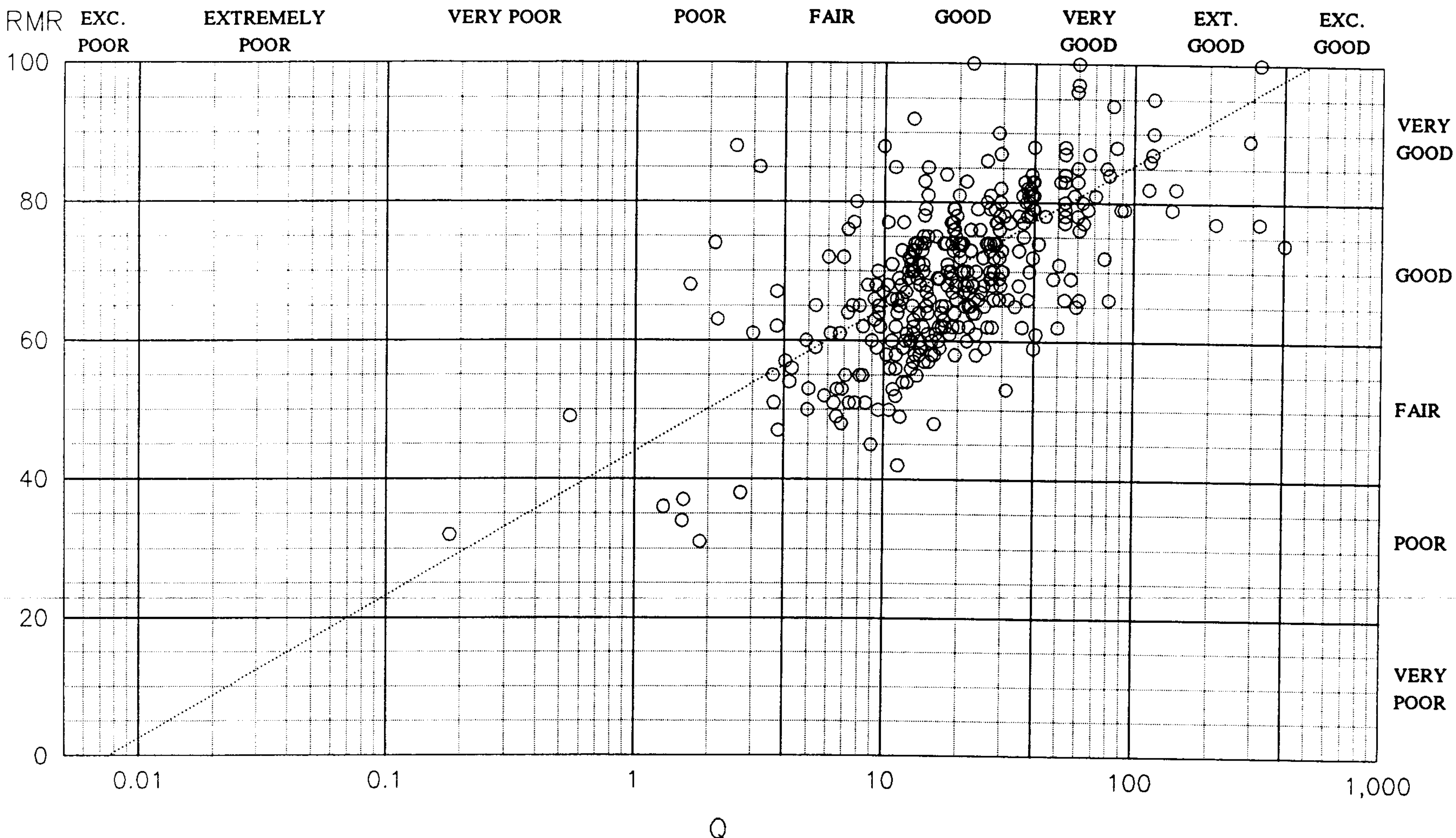


These values are based on interpretation of HQ diameter core boring logs.

The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)

$$RMR = 9 \ln Q + 44$$

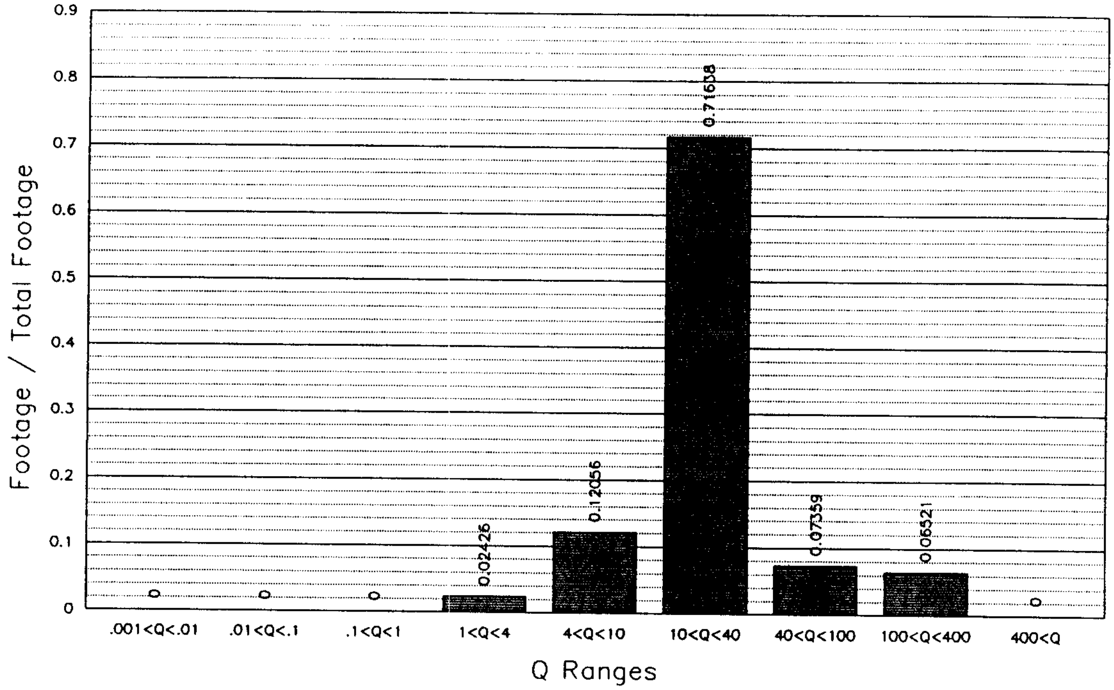
PASSAIC RIVER TUNNEL PROJECT
RMR AND Q VALUES MAIN TUNNEL
PASSAIC FORMATION



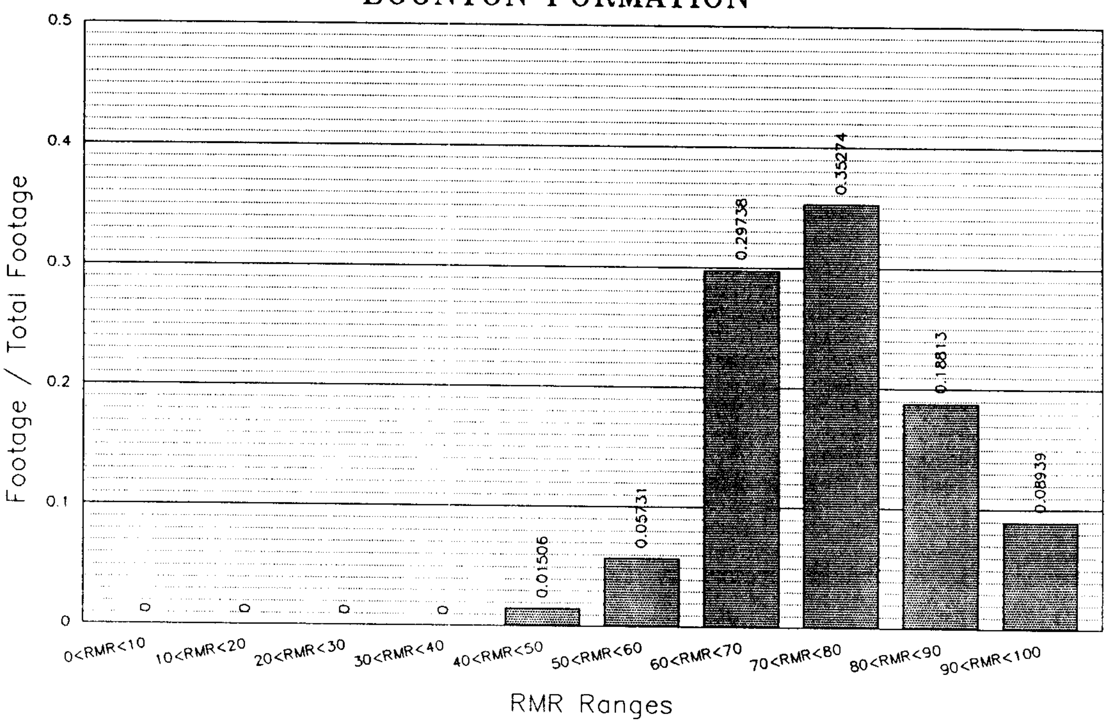
These values are based on interpretation of HQ diameter core boring logs.

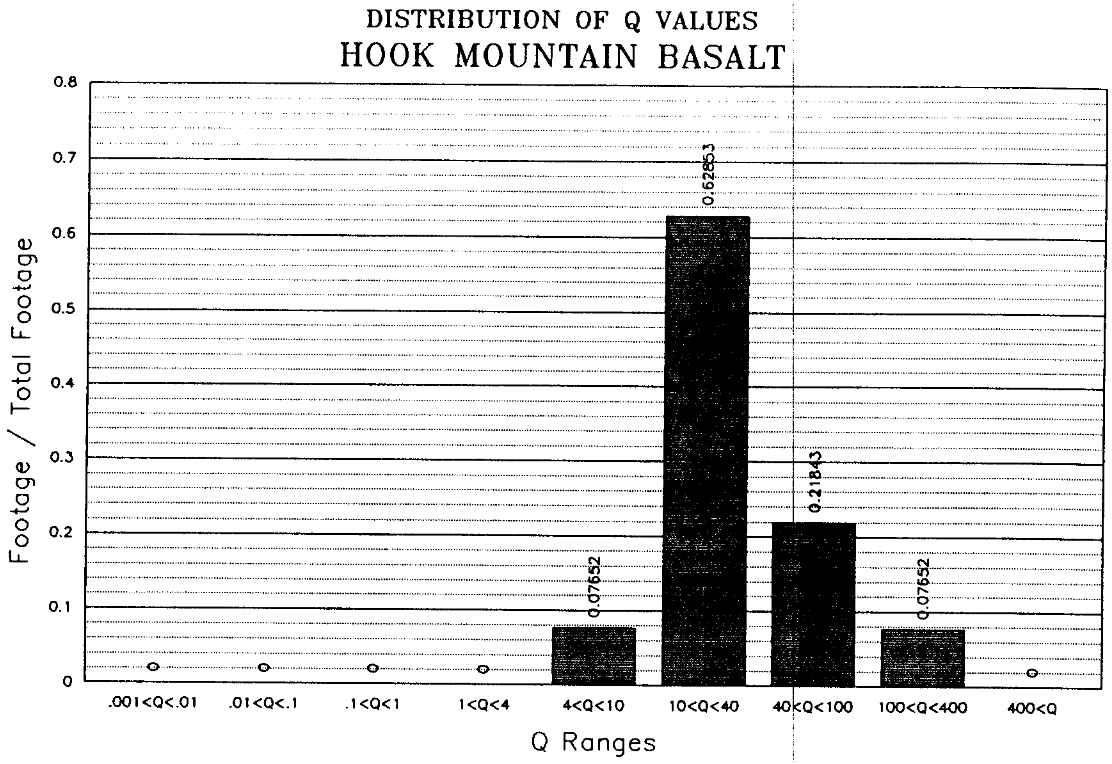
The dotted line is the correlation index (after Bieniawski, 1976 and Jethwa et al., 1982.)
RMR = 9 ln Q + 44

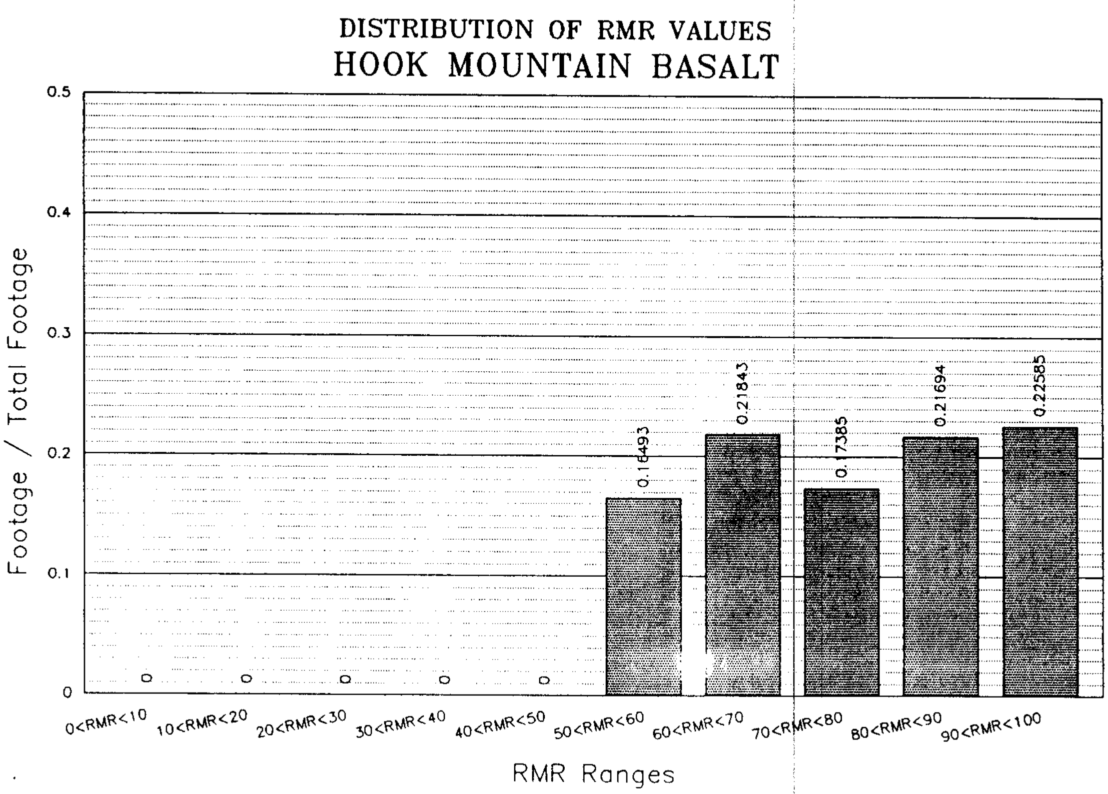
DISTRIBUTION OF Q VALUES
BOONTON FORMATION



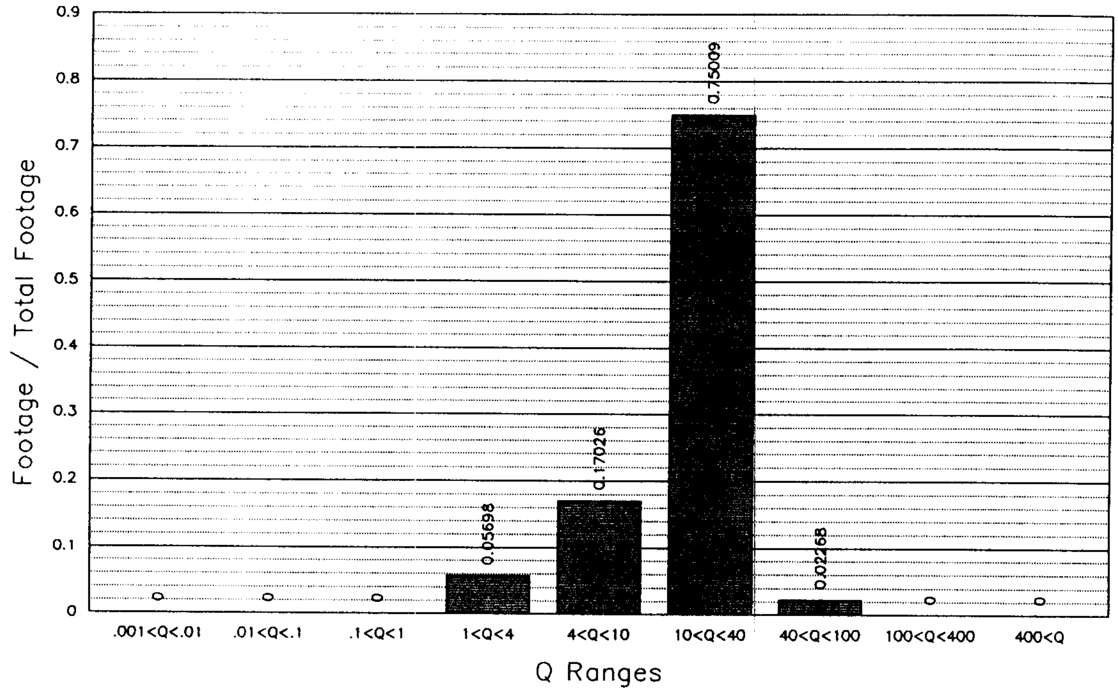
DISTRIBUTION OF RMR VALUES
BOONTON FORMATION



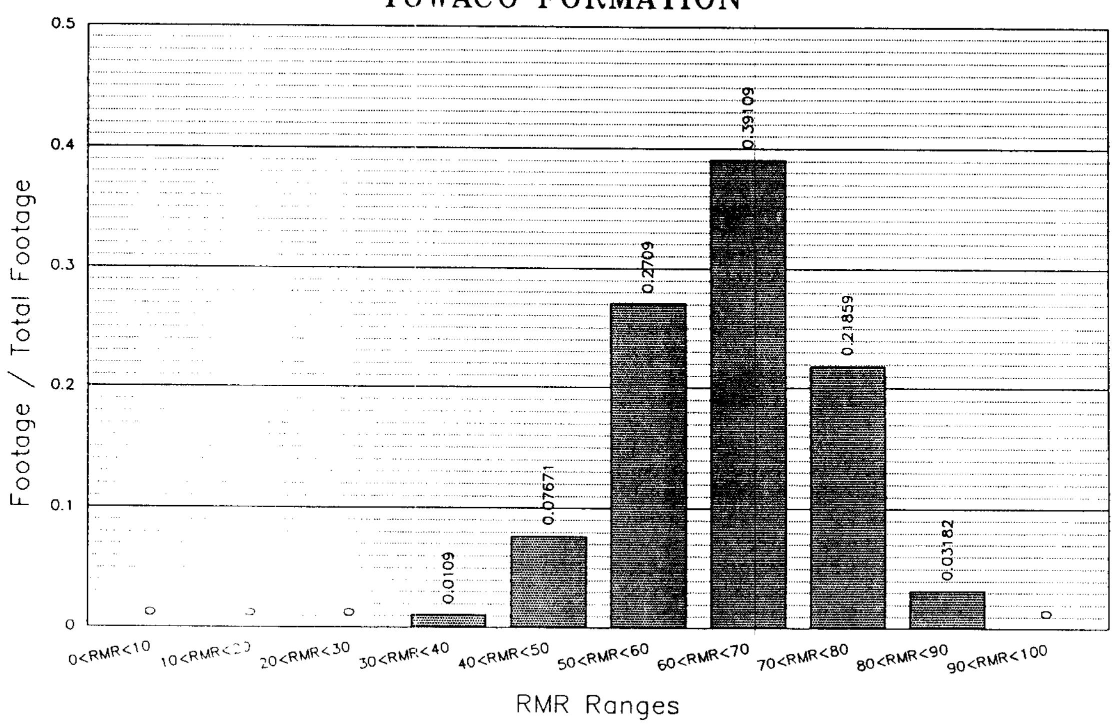

DISTRIBUTION OF Q VALUES
HOOK MOUNTAIN BASALT
Footage / Total Footage
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.07652
0.62853
0.21843
0.07652
.001<Q<.01
.01<Q<.1
.1<Q<1
1<Q<4
4<Q<10
10<Q<40
40<Q<100
100<Q<400
400<Q
Q Ranges

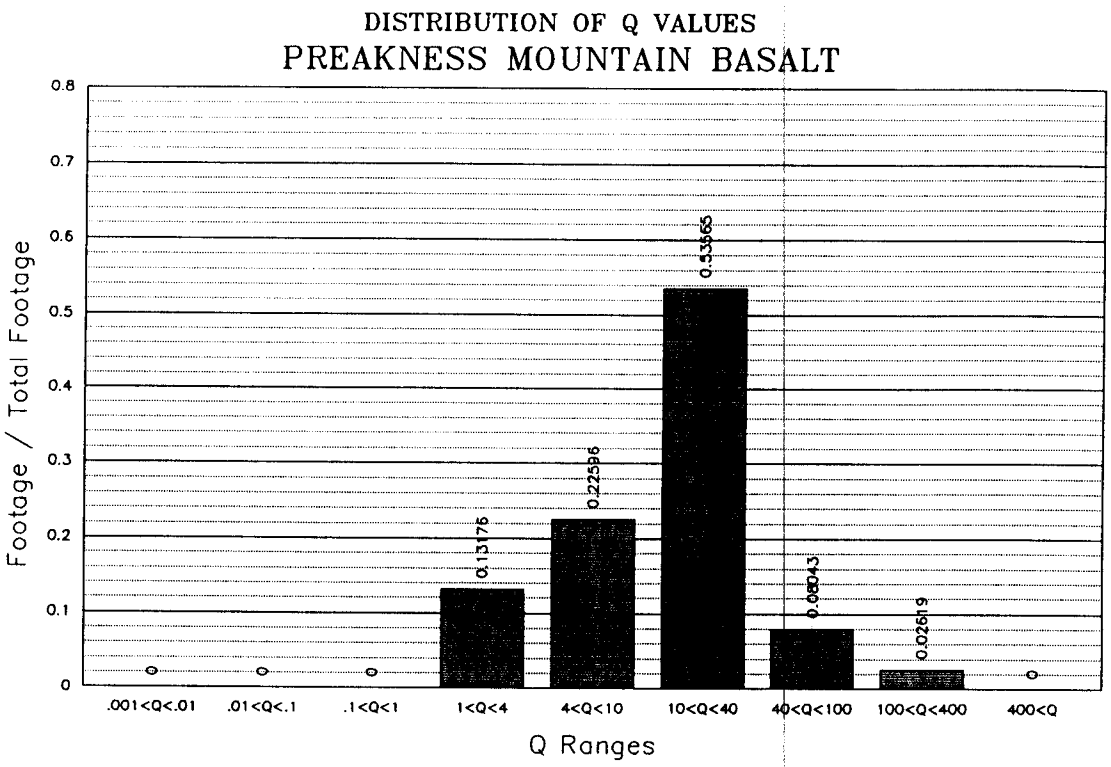

DISTRIBUTION OF RMR VALUES
HOOK MOUNTAIN BASALT
Footage / Total Footage
0
0.1
0.2
0.3
0.4
0.5
0.16493
0.21843
0.17385
0.21694
0.22585
0<RMR<10
10<RMR<20
20<RMR<30
30<RMR<40
40<RMR<50
50<RMR<60
60<RMR<70
70<RMR<80
80<RMR<90
90<RMR<100
RMR Ranges

DISTRIBUTION OF Q VALUES
TOWACO FORMATION



DISTRIBUTION OF RMR VALUES
TOWACO FORMATION

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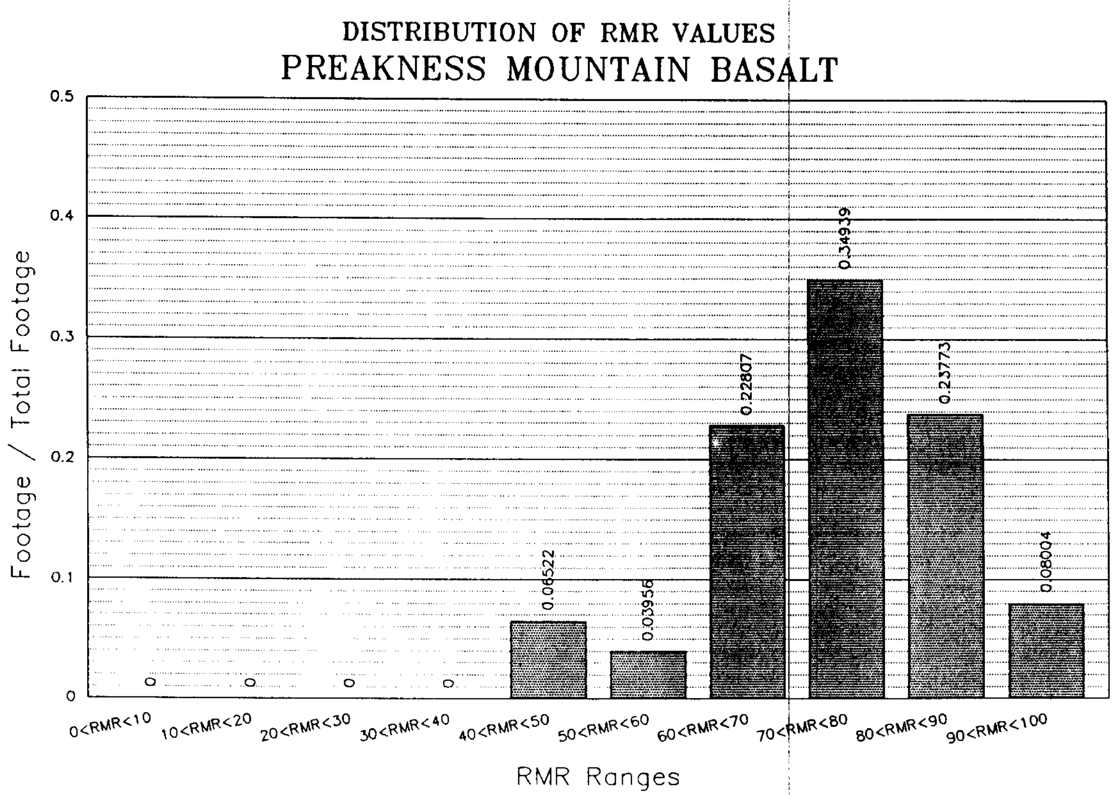
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PREAKNESS MOUNTAIN BASALT
Footage / Total Footage
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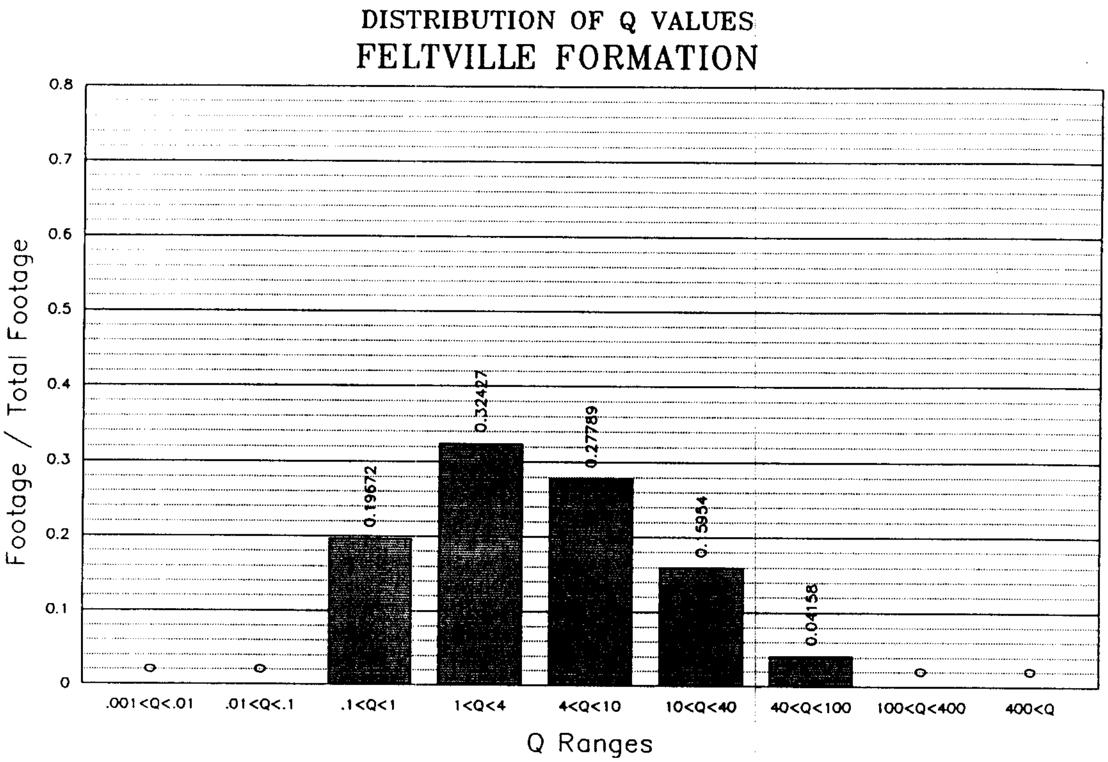
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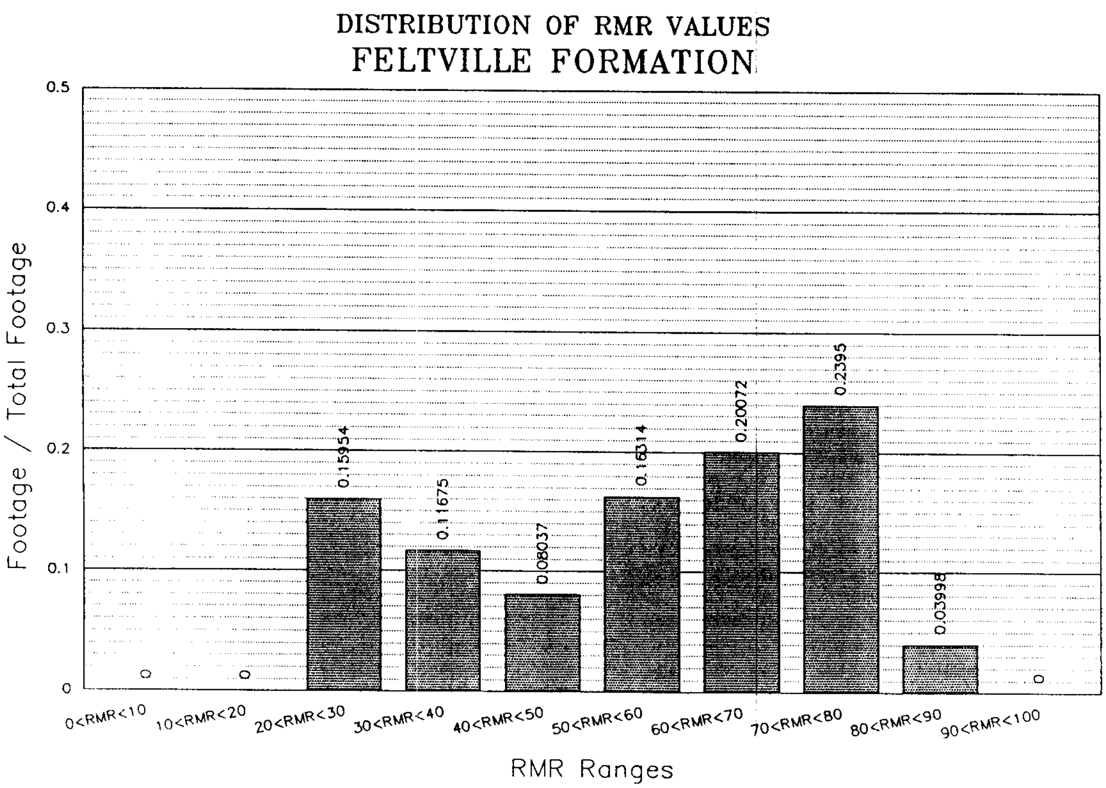
Q Ranges


DISTRIBUTION OF RMR VALUES
FELTVILLE FORMATION
Footage / Total Footage
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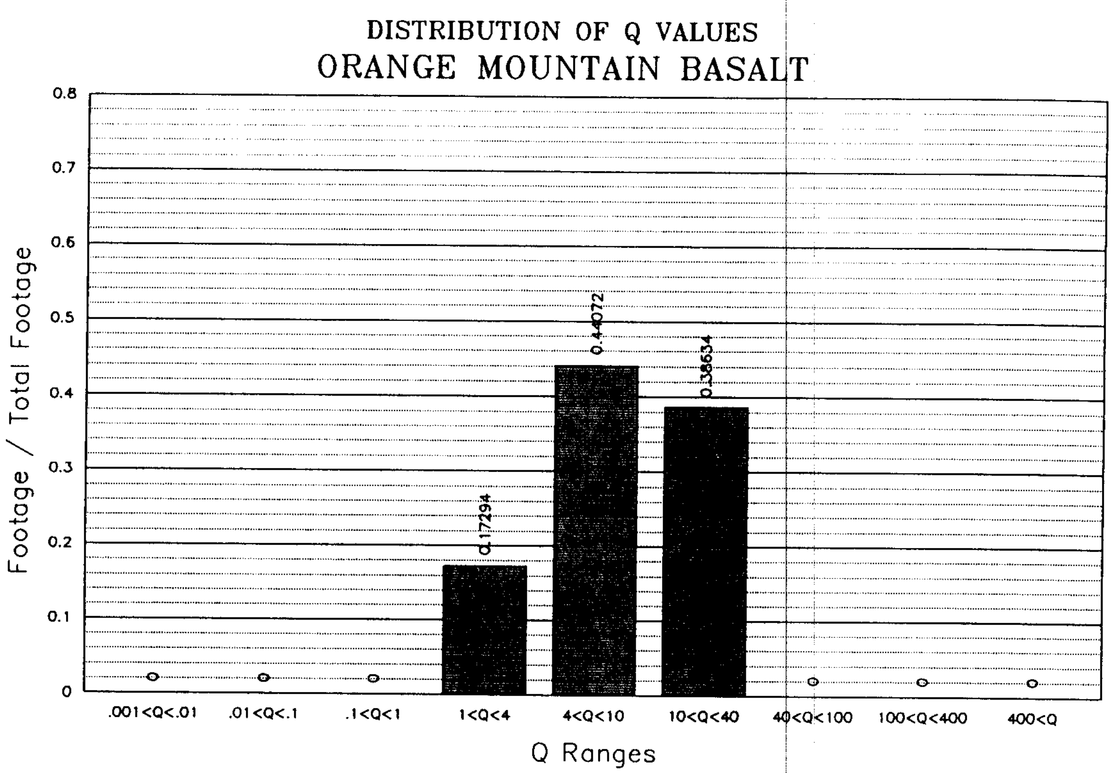
RMR Ranges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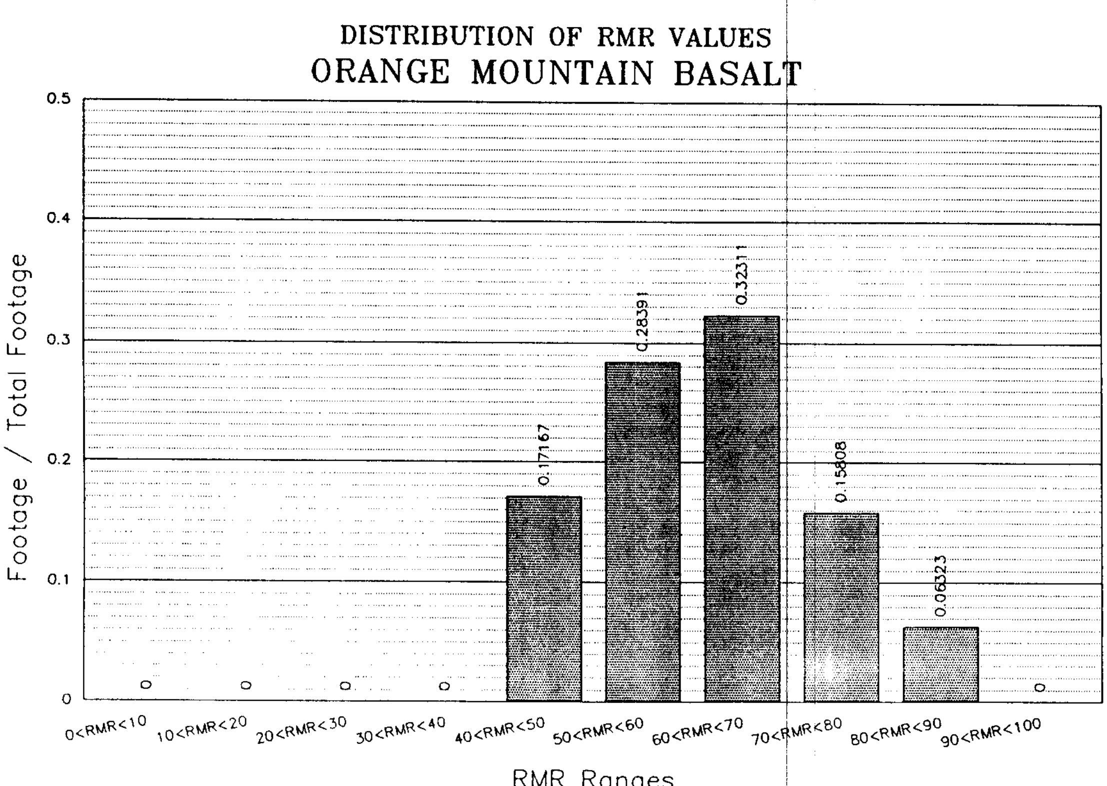
Q Ranges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ORANGE MOUNTAIN BASALT
Footage / Total Footage
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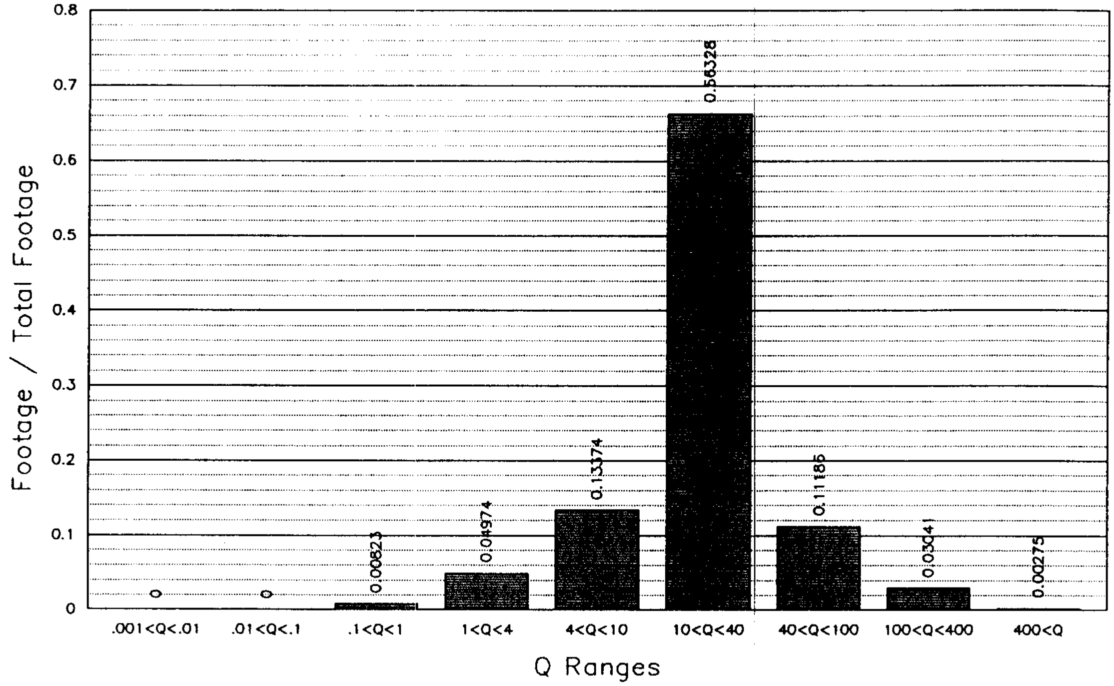
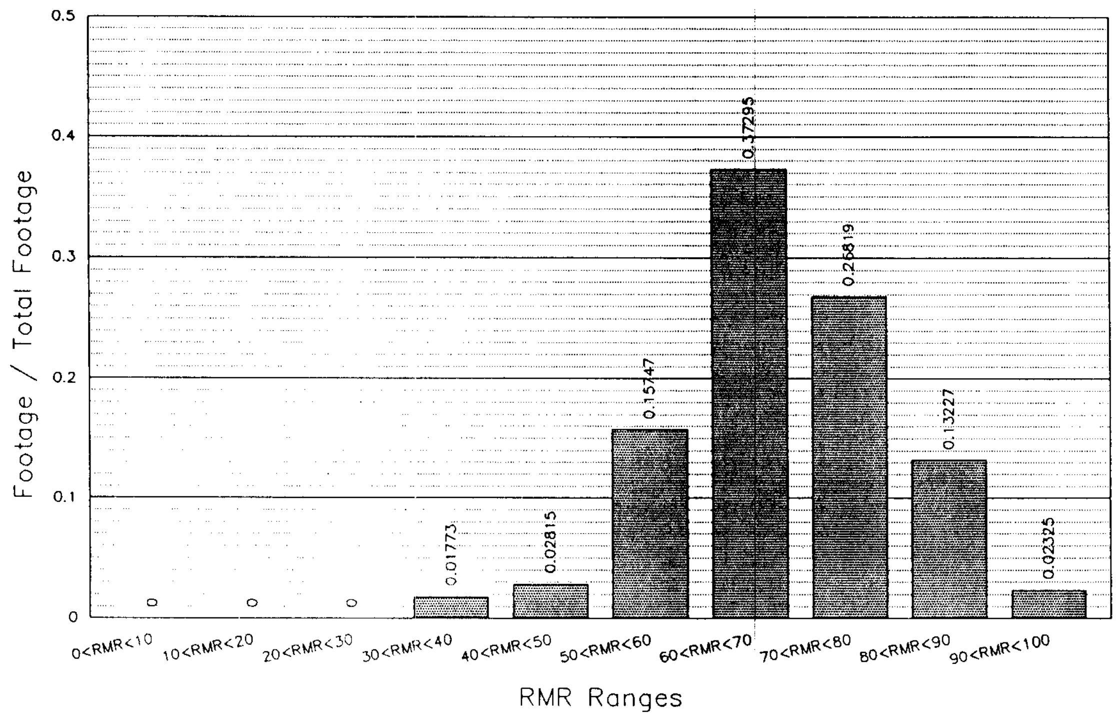
RMR Ranges

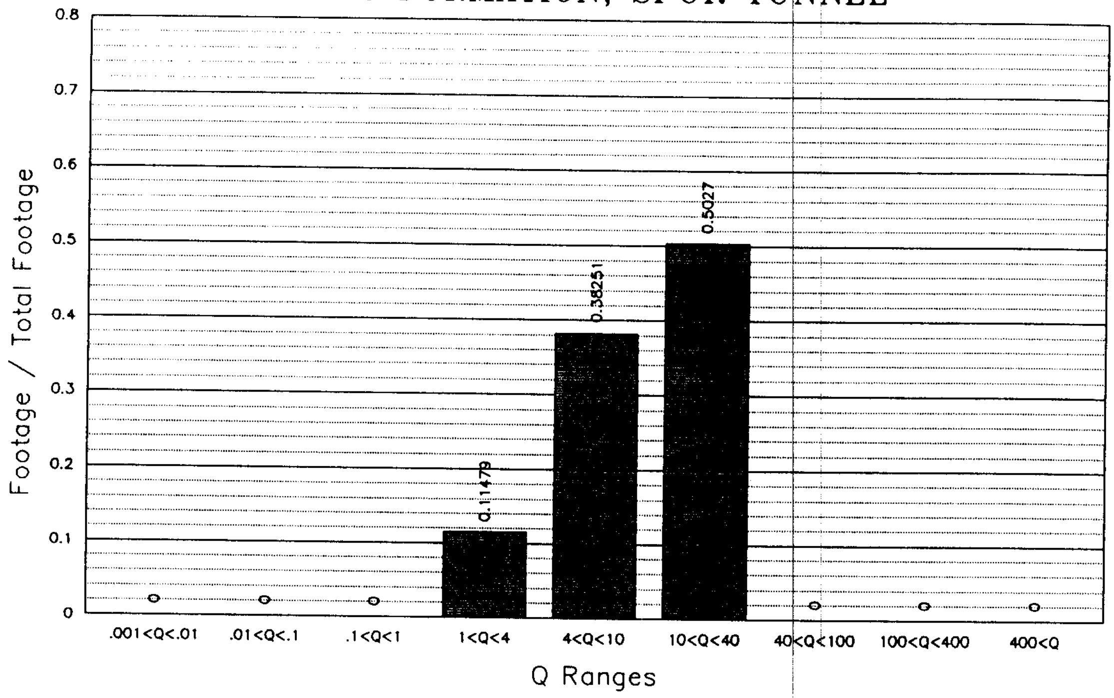
DISTRIBUTION OF Q VALUES
PASSAIC FORMATION



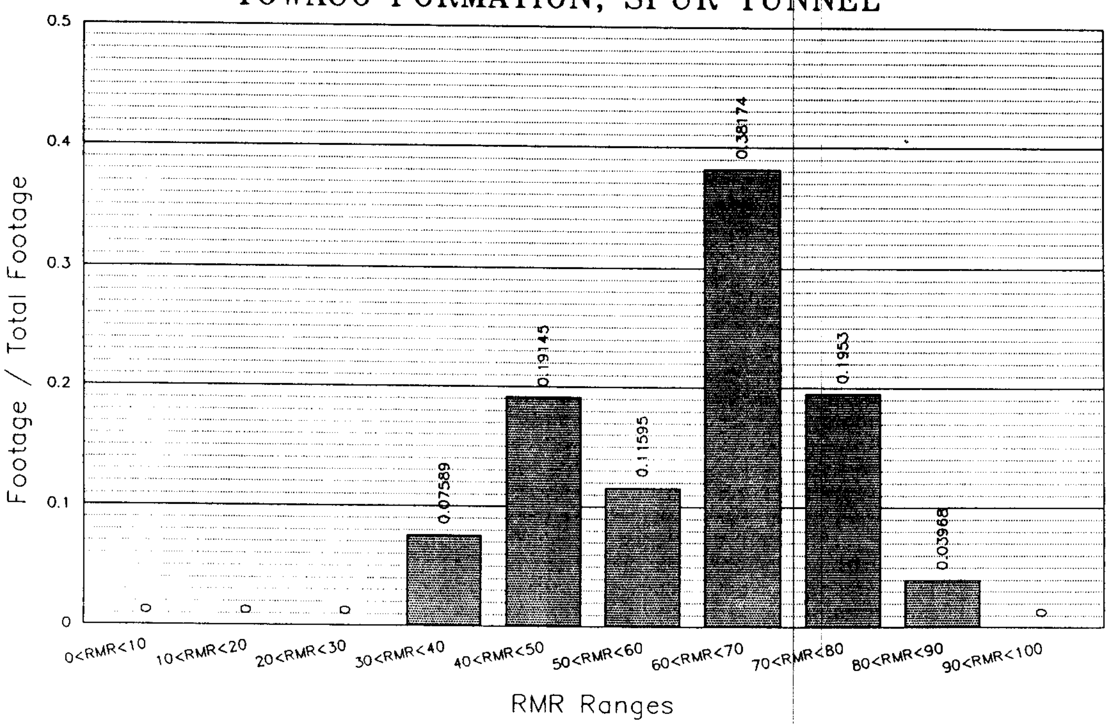
DISTRIBUTION OF RMR VALUES
PASSAIC FORMATION

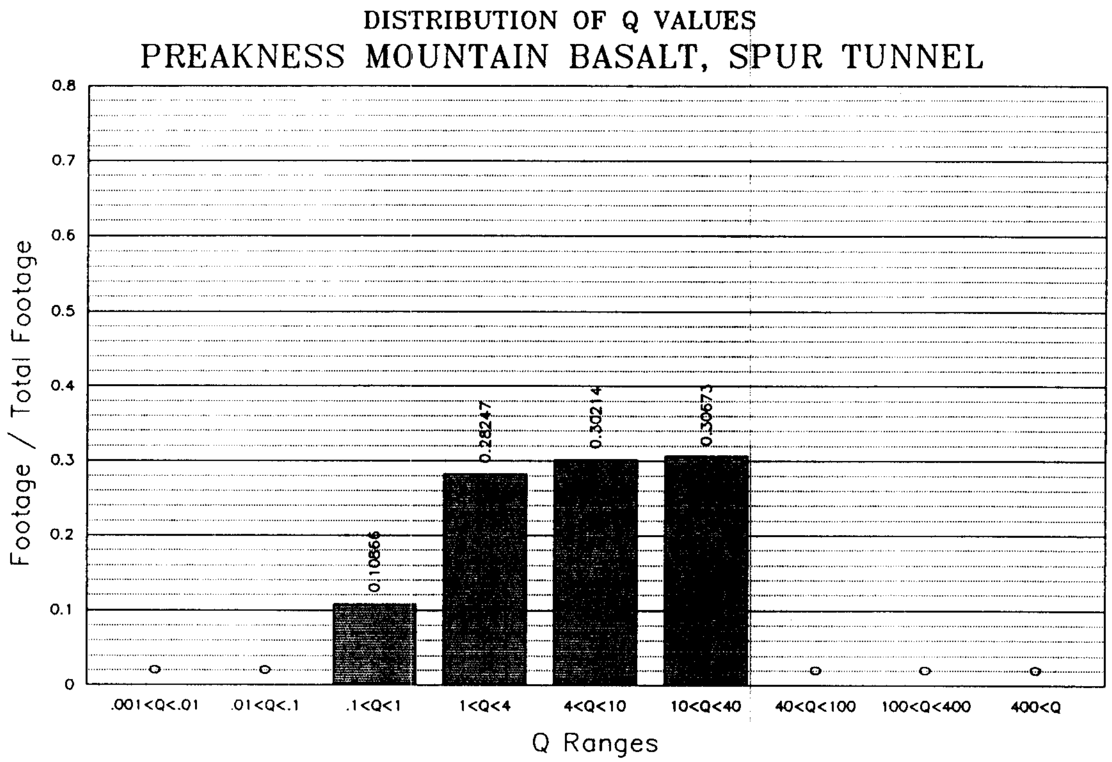


DISTRIBUTION OF Q VALUES
TOWACO FORMATION, SPUR TUNNEL



DISTRIBUTION OF RMR VALUES
TOWACO FORMATION, SPUR TUNNEL





DISTRIBUTION OF Q VALUES
PREAKNESS MOUNTAIN BASALT, SPUR TUNNEL
Footage / Total Footage
Q Ranges
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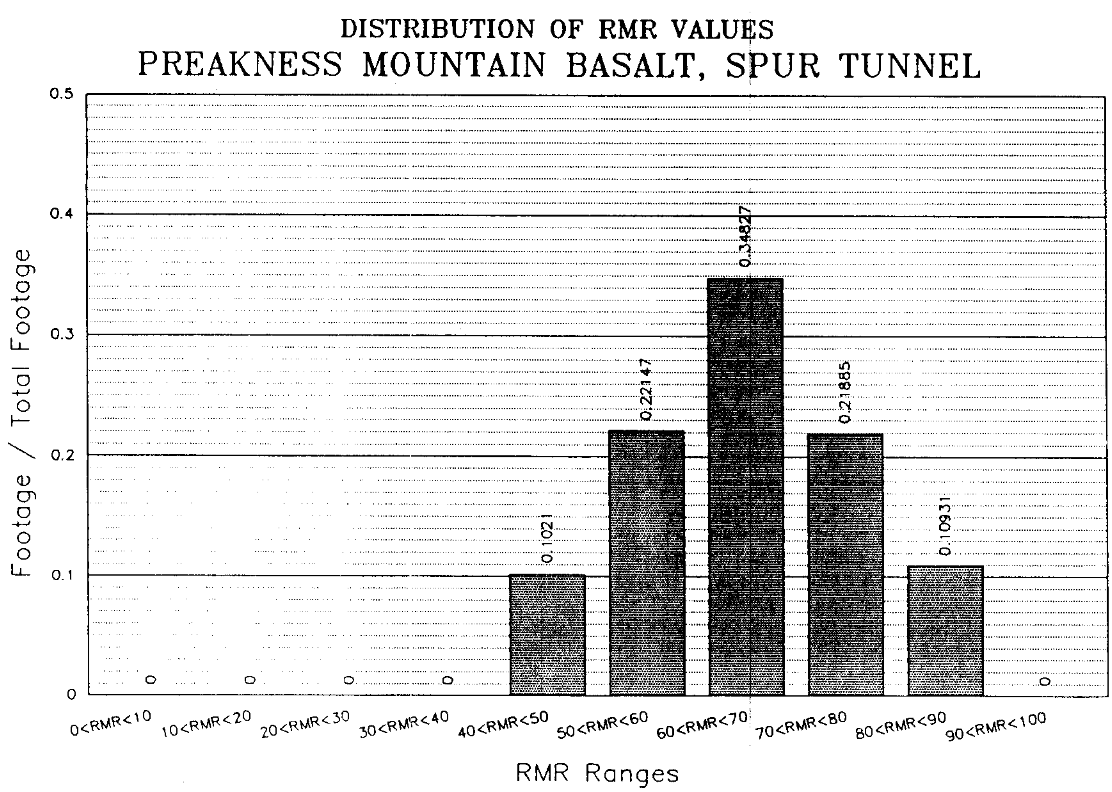

DISTRIBUTION OF RMR VALUES
PREAKNESS MOUNTAIN BASALT, SPUR TUNNEL
Footage / Total Footage
RMR Ranges
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PASSAIC RIVER FLOOD DAMAGE REDUCTION STUDY

ATTACHMENT E.3.12

QUANTITY TAKE-OFFS

E.3.12

QUANTITY TAKE-OFFS

PASSAIC TUNNEL
QUANTITY TAKE OFFS
ROCK EXCAVATION BY TYPE

CONTRACT	METHOD OF EXCAVATION *	DIRECTION OF EXCAVATION	SECTION SHEET #	ROCK TYPE	TUNNEL STATION START (feet)	TUNNEL STATION FINISH (feet)	LENGTH OF SECTION (feet) **	VOLUME OF EXCAVATION (cubic yards) ***	REMARKS
A	D&B	Down sta. from 2C	1	Tp SS/SIS	23	2280	2,257.0	135,919.0	Tunnel into outlet shaft
A	TBM	Up sta. from 2C	1	Tp SS/SIS	2280	5800	3,520.0	202,758.3	
A	TBM	Up sta. from 2C	2	Tp SS/SIS	5800	11800	6,000.0	345,610.8	
A	TBM	Up sta. from 2C	3	Tp SS/SIS	11800	16115	4,315.0	248,551.8	
TOTAL EXCAVATION FOR CONTRACT "A" IN CUBIC YARDS					D&B =	135,919.0	TBM =	796,921.0	
LINEAR FEET OF TUNNEL IN CONTRACT "A" BY CATAGORY					D&B =	2,257.0	TBM =	13,835.0	
B	D&B	Up sta. from 2C	3	Tp SS/SIS	16115	16515	400.0	24,088.4	Head & tail tunnel for 2B
B	TBM	Up sta. from 2B	3	Tp SS/SIS	16515	17800	1,285.0	74,018.3	Burried valley
B	TBM	Up sta. from 2B	4	Tp SS/SIS	17800	18506	706.1	40,670.0	Begin up slope correction 1.0000782
B	TBM	Up sta. from 2B	4	Tp SS/SIS	18506	21000	2,494.2	143,670.1	
B	TBM	Up sta. from 2B	5	Tp SS/SIS	21000	21740	740.1	42,628.7	
B	TBM	Up sta. from 2B	5	Basalt	21740	22100	360.0	20,738.3	
B	TBM	Up sta. from 2B	5	Tp SS/SIS	22100	24200	2,100.2	120,973.2	
B	TBM	Up sta. from 2B	6	Tp SS/SIS	24200	30200	6,000.5	345,637.8	
B	TBM	Up sta. from 2B	7	Tp SS/SIS	30200	36200	6,000.5	345,637.8	
B	TBM	Up sta. from 2B	8	Tp SS/SIS	36200	42200	6,000.5	345,637.8	
B	TBM	Up sta. from 2B	9	Tp SS/SIS	42200	48200	6,000.5	345,637.8	
B	TBM	Up sta. from 2B	10	Tp SS/SIS	48200	48323	123.0	7,085.6	
B	D&B	From WS 5	10	Tp SS/SIS	48323	48473	150.0	9,033.9	Work shaft 5 with 150' disassembly hall
TOTAL EXCAVATION FOR CONTRACT "B" IN CUBIC YARDS					D&B =	33,122.3	TBM =	1,832,335.3	
LINEAR FEET OF TUNNEL IN CONTRACT "B" BY CATAGORY					D&B =	550.0	TBM SOFT ROCK =	31,450.4	
							TBM HARD ROCK =	360.0	

* D&B = drill and blast, TBM = machine boring. ** measured based on assumtion that any hard rock in the face controls the penetration *** assumes no overbreak for TBM and 6" for D&B

PASSAIC TUNNEL
QUANTITY TAKE OFFS
ROCK EXCAVATION BY TYPE

CONTRACT	METHOD OF EXCAVATION *	DIRECTION OF EXCAVATION	SECTION SHEET #	ROCK TYPE	TUNNEL STATION START (feet)	TUNNEL STATION FINISH (feet)	LENGTH OF SECTION (feet) **	VOLUME OF EXCAVATION (cubic yards) ***	REMARKS
C	TBM	Down sta. from 2	10	Tp SS/SIS	48473	54200	5,727.0	329,885.5	
C	TBM	Down sta. from 2	11	Tp SS/SIS	54200	60200	6,000.0	345,610.8	
C	TBM	Down sta. from 2	12	Tp SS/SIS	60200	61776	1,576.0	90,780.4	
C	D&B	From WS 2	12	Tp SS/SIS	61776	62430	654.0	39,384.6	Work shaft 2 and excavate through faulting
C	TBM	Up sta. from 2	12	Tp SS/SIS	62430	62576	146.0	8,409.9	
C	TBM	Up sta. from 2	12	Tp SS/SIS	62576	63529	953.0	54,894.5	1,500' radius curved section–slower excavation
C	TBM	Up sta. from 2	12	Tp SS/SIS	63529	65740	2,211.0	127,357.6	
C	TBM	Up sta. from 2	12	Jo hard basalt	65740	66200	460.0	26,496.8	
C	TBM	Up sta. from 2	13	Jo hard basalt	66200	67250	1,050.0	60,481.9	
C	TBM	Up sta. from 2	13	Jo weak basalt	67250	67480	230.0	13,248.4	
C	TBM	Up sta. from 2	13	Jo hard basalt	67480	68275	795.0	45,793.4	
C	TBM	Up sta. from 2	13	Jo weak basalt	68275	68300	25.0	1,440.0	
C	TBM	Up sta. from 2	13	Jo hard basalt	68300	68670	370.0	21,312.7	
C	TBM	Up sta. from 2	13	Jo weak basalt	68670	69100	430.0	24,768.8	
C	TBM	Up sta. from 2	13	Jo hard basalt	69100	69970	870.0	50,113.6	
C	TBM	Up sta. from 2	13	Jf ss,sis,sh	69970	72200	2,230.0	128,452.0	
C	TBM	Up sta. from 2	14	Jf ss,sis,sh	72200	73150	950.0	54,721.7	
C	TBM	Up sta. from 2	14	Jp, vesicular	73150	73450	300.0	17,280.5	
C	TBM	Up sta. from 2	14	Jp, hard, jnted	73450	77250	3,800.0	218,886.9	
C	TBM	Up sta. from 2	14	Jp, vesicular	77250	77400	150.0	8,640.3	1,500' radius curved section–slower excavation
C	TBM	Up sta. from 2	14	Jp, hard	77400	78200	800.0	46,081.4	1,500' radius curved section–slower excavation
C	TBM	Up sta. from 2	15	Jp, hard	78200	78850	650.0	37,441.2	
C	TBM	Up sta. from 2	15	Jp, vesicular	78850	78950	100.0	5,760.2	
C	TBM	Up sta. from 2	15	Jp, hard, jnted	78950	80070	1,120.0	64,514.0	
C	TBM	Up sta. from 2	15	Jp, vesicular	80070	80450	380.0	21,888.7	
C	TBM	Up sta. from 2	15	Jp, hard	80450	81180	730.0	42,049.3	
C	TBM	Up sta. from 2	15	Jp, vesicular	81180	81300	120.0	6,912.2	
C	TBM	Up sta. from 2	15	Jt ss, sis, sh	81300	84200	2,900.0	167,045.2	
C	TBM	Up sta. from 2	16	Jt ss, sis, sh	84200	84286.8	86.8	4,999.8	
TOTAL EXCAVATION FOR CONTRACT "C" IN CUBIC YARDS					D&B =	39,384.6	TBM =	2,025,267.9	
LINEAR FEET OF TUNNEL IN CONTRACT "C" BY CATAGORY					D&B =	654.0	TBM SOFT ROCK -	24,514.8	
							TBM HARD ROCK -	10,645.0	

* D&B = drill and blast, TBM = machine boring. ** measured based on assumtion that any hard rock in the face controls the penetration *** assumes no overbreak for TBM and 6" for D&B

PASSAIC TUNNEL
QUANTITY TAKE OFFS
ROCK EXCAVATION BY TYPE

CONTRACT	METHOD OF EXCAVATION *	DIRECTION OF EXCAVATION	SECTION SHEET #	ROCK TYPE	TUNNEL STATION START (feet)	TUNNEL STATION FINISH (feet)	LENGTH OF SECTION (feet) **	VOLUME OF EXCAVATION (cubic yards) ***	REMARKS
D	D&B	Consruct WS3	16	Jt ss, sls, sh	84286.8	84386.8	100.0	6,022.1	
D	TBM	Down sta from inlet	16	Jt ss, sls, sh	84386.8	89060	4,673.2	269,184.7	
D	TBM	Down sta from inlet	16	Jh dense	89060	89700	640.0	36,865.2	
D	TBM	Down sta from inlet	16	Jh vesic	89700	89800	100.0	5,760.2	
D	TBM	Down sta from inlet	16	Jh dense	89800	90200	400.0	23,040.7	
D	TBM	Down sta from inlet	17	Jh dense	90200	91300	1,100.0	63,362.0	
D	TBM	Down sta from inlet	17	Jh vggy Jb	91300	91450	150.0	8,640.3	
D	TBM	Down sta from inlet	17	Jb ss,sls,sh	91450	96200	4,750.0	273,608.6	
D	TBM	Down sta from inlet	18	Jb ss,sls,sh	96200	102200	6,000.0	345,610.8	
D	TBM	Down sta from inlet	19	Jb ss,sls,sh	102200	107206	5,006.0	288,354.6	
D	D&B	Down sta from inlet	19	Jb ss,sls,sh	107206	107246	40.0	2,408.8	40' long section to start TBM
D	D&B	Down sta from inlet	19	Jb ss,sls,sh	107246	107668.4	422.4	38,532.4	62' Diam tunnel
TOTAL EXCAVATION FOR CONTRACT "D" IN CUBIC YARDS					D&B =	46,963.4	TBM =	1,314,427.1	
LINEAR FEET OF TUNNEL IN CONTRACT "D" BY CATAGORY					D&B =	562.4	TBM SOFT ROCK =	20,679.2	
							TBM HARD ROCK =	2,140.0	
SPUR	D&B	Work shaft 4		Jp, breccia	0	550	550.0	11,235	
SPUR	TBM	Up sta from WS 4		Jp, breccia	550	2170	1,620.0	30,642	
SPUR	TBM	Up sta from WS 4		Jp, hard basalt	2170	3480	1,310.0	24,779	
SPUR	TBM	Up sta from WS 4		Jp, breccia	3480	3530	50.0	946	
SPUR	TBM	Up sta from WS 4		Jt ss,sls	3530	6537.9	3,007.9	56,894	
SPUR	D&B	Up sta from WS 4		Jt ss,sls	6537.9	6900	362.1	11,821	30'diameter tunnel
TOTAL EXCAVATION FOR SPUR CONTRACT IN CUBIC YARDS					D&B =	23,055.9	TBM =	113,261.2	
LINEAR FEET OF TUNNEL IN SPUR CONTRACT BY CATAGORY					D&B =	912.1	TBM SOFT ROCK =	4,677.9	
							TBM HARD ROCK =	1,310.0	

* D&B = drill and blast, TBM = machine boring. ** measured based on assumtion that any hard rock in the face controls the penetration *** assumes no overbreak for TBM and 6" for D&B

PASSAIC TUNNEL

QUANTITY TAKE-OFFS

CONCRETE LINER QUANTITIES

THESE QUANTITIES DO NOT INCLUDE CONCRETE FOR THE INLETS, OUTLET, OR WORK SHAFTS.

CONTRACT	METHOD OF EXCAVATION	DIRECTION OF EXCAVATION	CONCRETE STRENGTH (psi)	TUNNEL STATION START (feet)	TUNNEL STATION FINISH (feet)	LENGTH OF SECTION (feet)	VOLUME OF CONCRETE (cubic yards)	REMARKS
A	D&B	Down sta. from 2C	5,000	23.0	1,800.0	1,777.0	15,830.3	Tunnel into outlet shaft
A	D&B	Down sta. from 2C	6,000	1,800.0	2,280.0	480.0	4,276.0	
A	TBM	Up sta. from 2C	6,000	2,280.0	4,600.0	2,320.0	14,590.8	
A	TBM	Up sta. from 2C	5,500	4,600.0	16,115.0	11,515.0	72,419.2	
	TOTAL CONCRETE IN LINER FOR CONTRACT "A" IN CUBIC YARDS =						107,116.3	
		CUBIC YARDS OF 6,000 psi CONCRETE =					18,866.8	
		CUBIC YARDS OF 5,500 psi CONCRETE =					72,419.2	
		CUBIC YARDS OF 5,000 psi CONCRETE =					15,830.3	

D&B = drill and blast assumed 6" average overbreak. TBM assumed no overbreak. Liner is 15" thick.

PASSAIC TUNNEL
QUANTITY TAKE-OFFS
CONCRETE LINER QUANTITIES

THESE QUANTITIES DO NOT INCLUDE CONCRETE FOR THE INLETS, OUTLET, OR WORK SHAFTS.

CONTRACT	METHOD OF EXCAVATION	DIRECTION OF EXCAVATION	CONCRETE STRENGTH (psi)	TUNNEL STATION		LENGTH OF SECTION (feet)	VOLUME OF CONCRETE (cubic yards)	REMARKS
				START (feet)	FINISH (feet)			
B	D&B	Up sta. from 2C	5,500	16,115.0	16,515.0	400.0	3,563.4	Head & tail tunnel for 2B
B	TBM	Up sta. from 2B	5,500	16,515.0	20,600.0	4,085.0	25,691.1	Burried valley
B	TBM	Up sta. from 2B	6,000	20,600.0	23,400.0	2,800.2	17,610.9	Begin up slope correction 1.0000782
B	TBM	Up sta. from 2B	6,500	23,400.0	25,400.0	2,000.2	12,579.2	
B	TBM	Up sta. from 2B	5,000	25,400.0	26,600.0	1,200.1	7,547.5	
B	TBM	Up sta. from 2B	4,500	26,600.0	36,200.0	9,600.7	60,380.3	
B	TBM	Up sta. from 2B	4,000	36,200.0	37,400.0	1,200.1	7,547.5	
B	TBM	Up sta. from 2B	3,000	37,400.0	41,800.0	4,400.3	27,674.3	
B	TBM	Up sta. from 2B	4,500	41,800.0	43,800.0	2,000.2	12,579.2	
B	TBM	Up sta. from 2B	4,000	43,800.0	45,000.0	1,200.1	7,547.5	
B	TBM	Up sta. from 2B	3,000	45,000.0	45,800.0	800.1	5,031.7	
B	TBM	Up sta. from 2B	4,000	45,800.0	46,600.0	800.1	5,031.7	
B	TBM	Up sta. from 2B	3,000	46,600.0	48,323.0	1,723.1	10,837.0	Work shaft 5 with 150' disassembly hall
B	D&B	From WS 5	3,000	48,323.0	48,473.0	150.0	1,336.4	

TOTAL CONCRETE IN LINER FOR CONTRACT "B" IN CUBIC YARDS =	204,957.7
CUBIC YARDS OF 6,500 psi CONCRETE =	12,579.2
CUBIC YARDS OF 6,000 psi CONCRETE =	17,610.9
CUBIC YARDS OF 5,500 psi CONCRETE =	29,254.4
CUBIC YARDS OF 5,000 psi CONCRETE =	7,547.5
CUBIC YARDS OF 4,500 psi CONCRETE =	72,959.5
CUBIC YARDS OF 4,000 psi CONCRETE =	20,126.8
CUBIC YARDS OF 3,000 psi CONCRETE =	44,879.4

D&B = drill and blast assumed 6" average overbreak. TBM assumed no overbreak. Liner is 15" thick.

PASSAIC TUNNEL

QUANTITY TAKE-OFFS

CONCRETE LINER QUANTITIES

THESE QUANTITIES DO NOT INCLUDE CONCRETE FOR THE INLETS, OUTLET, OR WORK SHAFTS.

CONTRACT	METHOD OF EXCAVATION	DIRECTION OF EXCAVATION	CONCRETE STRENGTH (psi)	TUNNEL STATION START (feet)	TUNNEL STATION FINISH (feet)	LENGTH OF SECTION (feet)	VOLUME OF CONCRETE (cubic yards)	REMARKS
C	TBM	Down sta. from 2	3,000	48,473.0	52,200.0	3,727.0	23,439.6	
C	TBM	Down sta. from 2	4,000	52,200.0	59,000.0	6,800.0	42,766.0	
C	TBM	Down sta. from 2	4,500	59,000.0	59,800.0	800.0	5,031.3	
C	TBM	Down sta. from 2	6,500	59,800.0	61,400.0	1,600.0	10,062.6	
C	TBM	Down sta. from 2	5,500	61,400.0	61,776.0	376.0	2,364.7	
C	D&B	From WS 2	5,500	61,776.0	62,430.0	654.0	5,826.1	Work shaft 2 and excavate through faulting
C	TBM	Up sta. from 2	5,500	62,430.0	63,000.0	570.0	3,584.8	
C	TBM	Up sta. from 2	4,500	63,000.0	64,200.0	1,200.0	7,546.9	
C	TBM	Up sta. from 2	5,000	64,200.0	65,800.0	1,600.0	10,062.6	
C	TBM	Up sta. from 2	4,500	65,800.0	67,000.0	1,200.0	7,546.9	
C	TBM	Up sta. from 2	3,000	67,000.0	84,286.0	17,286.0	108,713.8	

TOTAL CONCRETE IN LINER FOR CONTRACT "C" IN CUBIC YARDS =	226,945.4
CUBIC YARDS OF 6,500 psi CONCRETE =	10,062.6
CUBIC YARDS OF 6,000 psi CONCRETE =	0.0
CUBIC YARDS OF 5,500 psi CONCRETE =	11,775.6
CUBIC YARDS OF 5,000 psi CONCRETE =	10,062.6
CUBIC YARDS OF 4,500 psi CONCRETE =	20,125.2
~~CUBIC YARDS OF 4,000 psi CONCRETE =~~	~~42,766.0~~
CUBIC YARDS OF 3,000 psi CONCRETE =	132,153.3

D&B = drill and blast assumed 6" average overbreak. TBM assumed no overbreak. Liner is 15" thick.

PASSAIC TUNNEL
QUANTITY TAKE-OFFS
CONCRETE LINER QUANTITIES

THESE QUANTITIES DO NOT INCLUDE CONCRETE FOR THE INLETS, OUTLET, OR WORK SHAFTS.

CONTRACT	METHOD OF EXCAVATION	DIRECTION OF EXCAVATION	CONCRETE STRENGTH (psi)	TUNNEL STATION		LENGTH OF SECTION (feet)	VOLUME OF CONCRETE (cubic yards)	REMARKS
				START (feet)	FINISH (feet)			
D	D&B	Consruct WS3	3,000	84286.8	84386.8	100.0	890.8	
D	TBM	Down sta from inlet	3,000	84386.8	85800	1,413.2	8,887.8	
D	TBM	Down sta from inlet	4,000	85800	87000	1,200.0	7,546.9	
D	TBM	Down sta from inlet	4,500	87000	88600	1,600.0	10,062.6	
D	TBM	Down sta from inlet	4,000	88600	89400	800.0	5,031.3	
D	TBM	Down sta from inlet	3,000	89400	93800	4,400.0	27,672.1	
D	TBM	Down sta from inlet	4,000	93800	102200	8,400.0	52,828.6	
D	TBM	Down sta from inlet	3,000	102200	107206	5,006.0	31,483.3	
D	D&B	Down sta from inlet	3,000	107206	107246	40.0	356.3	40' long section to start TBM
D	D&B	Down sta from inlet	3,000	107246	107668.4	422.4	5,308.1	52' Diam tunnel
	TOTAL CONCRETE IN LINER FOR CONTRA D&B =						150,068.0	
		CUBIC YARDS OF 6,500 psi CONCRETE =					0.0	
		CUBIC YARDS OF 6,000 psi CONCRETE =					0.0	
		CUBIC YARDS OF 5,500 psi CONCRETE =					0.0	
		CUBIC YARDS OF 5,000 psi CONCRETE =					0.0	
		CUBIC YARDS OF 4,500 psi CONCRETE =					10,062.6	
		CUBIC YARDS OF 4,000 psi CONCRETE =					65,406.9	
		CUBIC YARDS OF 3,000 psi CONCRETE =					74,598.5	

SPUR	D&B	Work shaft 4	3,000	0	550	550.0	2,772.4	
SPUR	TBM	Up sta from work s	3,000	550	6450	5,900.0	20,809.4	
SPUR	TBM	Up sta from work s	3,000	6450	6537.9	87.9	443.1	
SPUR	D&B	Up sta from work s	3,000	6537.9	6810	272.1	1,759.1	30' diameter tunnel
	TOTAL CONCRETE IN LINER FOR CONTRACT "C" IN CUBIC YARDS = ALL 3,000 psi =						25,784	

D&B = drill and blast assumed 6" average overbreak. TBM assumed no overbreak. Liner is 15" thick.

PASSAIC RIVER FLOOD REDUCTION STUDY
TUNNEL INITIAL SUPPORT, QUANTITY TAKE-OFFS
SUMMARY TABLE

CONTRACT	CLASSIFICATION SYSTEM	ROCK BOLTS (Feet)	WIRE MESH (Square feet)	STRAPPING (Feet)	STEEL RIBS (8x48) (Pounds)
A	Q	473,376	119,121	44,329	0
A	RMR	440,101	65,208	21,169	0
A	Q x RMR	460,017	97,808	35,525	0
A	**AVERAGE**	**420,489**	**94,046**	**33,674**	**0**
B	Q	920,194	206,282	72,269	0
B	RMR	862,325	110,604	31,097	0
B	Q x RMR	892,330	164,405	54,634	0
B	**AVERAGE**	**808,803**	**160,430**	**52,667**	**0**
C	Q	1,033,624	298,181	111,793	1,946,014
C	RMR	943,599	134,021	41,240	1,732,818
C	Q x RMR	999,091	239,935	86,396	1,714,468
C	**AVERAGE**	**909,116**	**224,046**	**79,810**	**1,797,767**
D	Q	656,452	130,986	45,111	0
D	RMR	617,847	58,663	15,147	0
D	Q x RMR	648,192	112,890	37,740	15,908
D	**AVERAGE**	**561,084**	**100,846**	**32,666**	**5,303**
SPUR	Q	118,129	49,597	14,788	1,975,681
SPUR	RMR	108,697	23,560	8,164	102,998
SPUR	Q x RMR	117,649	46,343	14,118	1,245,711
SPUR	**AVERAGE**	**111,867**	**39,833**	**12,357**	**1,108,130**

Because of variability in the classifications systems the average of three systems was used to predict the initial support requirements for the tunnels. The values shown in boldface above are these average values and should be used in the project cost estimate.

APPENDIX E

SECTION 4

SHAFTS

ERRATA SHEET

14 September 1995

Please note that Workshaft 2A (Emergency Access Shaft 2A) has been deleted from the flood reduction project and will not be included in the proposed construction. Subsequent discussion and design drawings regarding Workshaft 2A in the following section have been left in place for descriptive purposes in the event other report sections in this General Design Memorandum make reference to Workshaft 2A. All cost estimates have been revised to reflect this change.

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

GEOTECHNICAL DESIGN
APPENDIX E

SECTION 4
SHAFTS

FIGURES

ATTACHMENTS

APPENDIX E

SECTION 4

SHAFTS

4.1 SCOPE

In this section of the appendix the geotechnical design for the 12 shafts currently required under the recommended plan is summarized. Detailed information presented in the Section 2 of the Appendix on exploration and testing is not repeated here. This report is intended to summarize the most important information gained through the field investigations and present the methods of analysis which were used and the results obtained. Detailed computation sheets are provided as examples only. The complete set of computations is available from the PRD.

4.2 FEATURE DESCRIPTION

4.2.1 Work/Dewatering Pump Station Shaft 2C Two shafts will be constructed at this site, a 42 foot inside diameter combination work shaft and dewatering pump station shaft, and a 15 foot inside diameter vent shaft. The 42 foot diameter work shaft will be offset from the main tunnel about 100 ft at tunnel station 21+60.05 on Kearny Metro Water Authority property. The shaft will be connected to the tunnel with a 42 foot diameter horizontal bore which will ultimately house the pumps. During tunnel construction the 42 foot diameter shaft and bore will be utilized for TBM egress and muck removal. A general plan view of the shaft is shown on Figure E.4.1. The estimated total depth of the shaft will be about 420 ft. Shaft excavation through the overburden will be performed using a freezewall. This requires "freezing" the soil surrounding the work shaft area before construction begins. The frozen soil has adequate strength to remain stable during excavation and construction of the work shaft. The portion of the shaft that passes through the overburden will be constructed as a conventional structural concrete wall that will be built within a freezewall. This 15 foot diameter vent shaft will be located on the tunnel centerline at station 20+80.05.

4.2.2 Work Shaft 2B This is a major work shaft located at tunnel station 163+15.03 at the end of Bergan Avenue in Kearney. The ground surface elevation at this location is approximately 6. The estimated total depth of this shaft to tunnel invert is 418 feet. Prior to construction, a freezewall will be advanced through the overburden as described for work shaft 2C. During construction of the tunnel this 42-foot inside diameter, concrete lined shaft will be used for TBM access and removal, muck removal, general construction support, and concrete placement. This shaft is adjacent to Conrail track so it is planned to locate a switch-yard/tipple to facilitate muck transportation by rail. A general plan view of this location is shown on Figure E.4.2. Upon completion of the lower portion of the tunnel a extension will be added to the top of the shaft to raise it to elevation 55 so that it will function as a vent.

4.2.3 <u>Emergency Access Shaft 2A</u> Is located on Hedricks Field Golf Course property adjacent to Joralemon Road in Belleville at tunnel station 346+89.5. The ground surface elevation is estimated at around 130 and the total estimated depth of the shaft to tunnel invert is 326.6 feet. A general plan view of the shaft is shown on Figure E.4.3. This 15 foot inside diameter shaft is to be an emergency access shaft constructed to satisfy OSHA safety requirements. The shaft will be expanded to 20 feet in diameter through the overburden so that a structural slurry wall can be utilized. The thickness of the structural slurry wall will be 24 inches. The wall will be placed in a series of six panels, each having a length of twelve feet. The slurry wall will be keyed into rock for a depth of thirteen feet due to the poor quality of the top layer of rock. The rock face will be protected with a twelve inch concrete liner that will extend up into the structural slurry wall until a five foot overlap is established between the slurry wall and the concrete liner. At this point the concrete liner will be discontinued. Upon completion of the tunnels it is to be retained as an access shaft.

4.2.4 <u>Work Shaft 5</u> Work shaft 5 is otherwise known as the "hook hole". It is located adjacent to the Garden State Parkway near West Passaic Avenue at tunnel station 483+98. Top of ground elevation 165.9 and invert elevation is -25.6 making the total depth of the shaft 191.5. The location is adjacent to the return ramp for the service center and is very constricted. Expansion of the work area to accommodate operations other than disassembly of the TBMs using a 75 ton ± crane will be difficult. However it would be an advantage to be able to deliver concrete for the cast in place liner through this shaft. A general plan view of this 15-foot inside diameter shaft is shown on Figure E.4.4. The work shaft diameter will be expanded to 20 foot through the overburden for incorporation of a structural slurry wall. The slurry wall will be twenty-four inches thick and will be placed in a series of six panels, each having a length of eleven and one-half feet. The rock face will be protected with a twelve inch thick concrete liner that will extend up into the structural slurry wall until a five foot overlap is established. At this point the concrete liner will be discontinued. Upon completion of the construction, this shaft is to be kept open as a vent shaft.

4.2.5 <u>Work Shaft 2</u> Work Shaft 2 is located on the floor of an abandoned quarry on the property of Montclaire State Teachers College. This is a major work shaft located at tunnel station 623+76. The ground surface elevation at this location is approximately 330 feet. The estimated total depth of this shaft to tunnel invert is 349 feet. During construction of the tunnel this 42-foot inside diameter, concrete lined shaft will be used for TBM access and removal, muck removal, general construction support, and concrete placement. This shaft is adjacent to a commuter Conrail track so it is planned to locate a switch-yard/tipple to facilitate muck transportation by rail. Upon completion of the tunnel this shaft is to be used as a maintenance access shaft. Additional surface works at this location include a master control center for the entire tunnel system, maintenance facilities, and a visitor's center. A general plan view of the shaft is shown on Figure E.4.5.

4.2.6 <u>Work Shaft 6</u> This 15 foot inside diameter vent shaft is to be constructed near the intersection of the Spur and the Main Tunnels. It is located at tunnel station 782+10 on the same piece of wooded property as Work Shaft 4. Top of ground elevation is approximately 180 and tunnel invert is approximately -12 making the shaft 192 total depth. The diameter will be expanded to 20 feet through the overburden so that a structural slurry wall may be incorporated into the design. The slurry wall will be keyed into rock for a depth of two feet. The thickness of the structural slurry wall will be twenty-four inches. The wall will be placed in a series of six panels, each having a length of eleven and one-half feet. A concrete liner will be placed over the rock face and extend up into the structural slurry wall until a five feet overlap is established between the slurry wall and the concrete liner. At this point the concrete liner will be discontinued. Since it is not needed for the construction it may be constructed at any time prior to operation of the tunnel. A general plan view of the shaft is shown on Figure E.4.6.

4.2.7 <u>Work Shaft 3</u> This site is in a heavily wooded area near an industrial park, and golf course. It is immediately adjacent to a narrow access road to the Wayne Municipal Yard. It is located on the Main Tunnel center line at tunnel station 843+36.8. The top of ground elevation is approximately 174 and the tunnel invert is -7.73. This 182 foot deep shaft is primarily to be used for removal of the TBMs and after construction as a vent shaft. The shaft has a 42 foot inside diameter and depth to rock is 87 feet. A structural slurry wall with a thickness of 30 inches will be used in the overburden. The wall will be placed in a series of twelve panels, each having a length of twelve feet. The structural slurry wall will be keyed into rock for a depth of three feet. An eighteen inch thick concrete liner will be placed over the rock face and extended up into the structural slurry wall until a five feet overlap is established between the slurry wall and the concrete liner. At this point the concrete liner will be discontinued. A general plan view of the shaft is shown on Figure E.4.7.

4.2.8 Work Shaft 4 Work shaft 4 is located in the curved section of the Spur Tunnel which connects with the main tunnel. The shaft has an inside diameter of 23 feet. Top of ground elevation is approximately 180 and tunnel invert is approximately -12 making the shaft 192 feet in total depth. A structural slurry wall will be incorporated through the overburden. The slurry wall will have a thickness of thirty inches and extend into the rock for a depth of two feet. The wall will be placed in a series of seven panels, each having a length of twelve feet. The rock face will be protected with a twelve inch concrete liner that will extend up into the structural slurry wall until an overlap of five feet is established between the slurry wall and the concrete liner. At this point the concrete liner will be discontinued. It will be used for TBM access, muck removal, concrete placement, and general construction support for the Spur. A general plan view of the shaft is shown on Figure E.4.6.

4.3. SUBSURFACE INVESTIGATIONS

The following table shows the exploratory holes which have been used for the shaft design. Additional information on the exploration is provided in paragraph 3. The SPT was intermittent, normally on five foot centers. The footage shown includes the unsampled intervals between the SPT drive intervals. See Attachment E.4.2 for assumed soil profiles at the shafts and Attachment E.4.5 for detailed boring logs of the overburden materials.

Table 4.3.1
EXPLORATION FOR WORK SHAFTS

WORK SHAFT #	HOLE #	FEET OF SPT	FEET OF SHELBY TUBE	FEET OF UN-SAMPLED	FEET OF ROCK CORE
2C	DC-146	103	42	0	396
2B	DC-21	22.5	0	138.5	240.9
2A	C-48	45.5	0	0	356.1
5	C-67	0	0	27	221
2	C-85	0	0	1.7	444.3
6	DC-99	12.5	0	.5	214.7
3	DC-102	60.5	9	26.2	266.5
4	C-98	0	0	33.4	189.9

4.4 WORK SHAFT SITE GEOLOGY

4.4.1 Overburden

Work shaft 2C has a soil profile that includes: a layer of heterogeneous fill about 11 ft. thick; organic silt (OH) layer that is 12 feet in thickness; and 13 feet of silty sands. The bulk of material to be drilled through is the next layer which is silty clay (CL) that is 62 feet in thickness. Underlying the silty clay is 16 feet of silty gravel (GM) that rests on rock.

Work shaft 2B has a soil profile 155 feet in depth. Preliminary layers of only a few feet consist of miscellaneous fill (GC), Organic silt (OL) and sand silt (ML). After the first 12 feet of these materials, a layer of silty sand (SM) 20 feet deep is encountered. Underlying the silty sand is 18 feet of sandy silt (ML) and 50 feet of varved clay and silt (ML-CL). The varved clay is underlain by 40 feet of silty sand (SM) which is underlain by 15 feet of glacial till.

The soil profile for work shaft 2A consists of a clayey silt overlaying a sandy silt which in turn overlays gravel. The clayey silt (ML) is approximately 4 feet deep. The sandy silt (ML) is approximately 10 feet deep and the gravel (GW) is 6 feet in depth.

Overburden at work shafts 5 and 6 are composed of one layer of gravel, 27 and 33 feet, respectively. Work shaft 2 has no overburden and drilling will begin at ground surface, which is top of rock.

Work shaft 4 has a soil profile consisting of approximately 33 feet of sand, gravel and cobbles with variable amounts of silt and clay (GW-GH).

4.4.2 Bedrock

Work shafts 2C and the Dewatering Pump Station Shaft will be excavated in red-brown siltstone, shale, and sandstone of the Passaic Formation as encountered in boring DC-146. Top of rock was hit in this hole at elevation -113 ± at a depth of 103. Siltstone and shale makes up most of the section. The sandstone is very fine to fine with occasional beds of coarse, porous rock. There is a 15 foot thick very fractured zone at a depth of 208.1. High angle fractures, which were infrequent, were usually healed with calcite.

The geology of Work Shaft 2B was investigated with boring PR-7 which encountered rock around elevation -151 at a depth of 161 feet. Coring was started at 171.3 in red-brown shale of the Passaic Formation. This material is vuggy, calcareous and contains abundant dense, sparry gypsum as both fracture filling and horizontal seams up to 4 inches thick. Green staining is associated with the gypsum.

Work shaft 2A is to be excavated in red-grey to red-brown sandstone, siltstone, and shale of the Passaic Formation as sampled in boring DC-48. Top of rock was encountered at elevation 85.0, 45.5 feet below the ground surface. The top 10.8 feet of rock was not sampled and casing was set at 56.3 feet. The first 6.8 feet of core had a fair RQD of 59 because of high angle fractures but below this depth the rock is sound and of good quality. The sandstone is mainly fine to very fine grained and micaceous with occasional beds of medium grained rock and pebble conglomerate. Siltstone and shale interbeds up to 12 feet thick were encountered through the entire depth of the hole. A three-foot thick, brecciated, slickensided section at 216 feet is probably a shear zone. It is made up of sub-parallel, soft green-clay coated fractures dipping between 45 and 70 degrees with numerous calcite healed hairline breaks. Thin, soft to stiff red clay seams were encountered at depths of 232, 251.5, 258.5, 287.6, 300.5, 317.8, 349.7, 363.5, and 390.4. A dual piezometer with ten foot screen sections was installed in hole DC-48. The mid-tip of the upper screen was at a depth of 98 and the lower screen at 289.

Boring C-67 is close to Work Shaft 5 which will be excavated in material very similar to that in Work Shaft 2A except that the shale beds are thinner and spaced much further apart. The rock is fresh with only a few high angle joints. Shaft construction should present very little problem at this site.

Boring C-85 is located in the area proposed for Work Shaft 2 on the floor of an abandoned quarry at elevation 330. Assumed top of rock is overlain by less than 2 feet of broken rock left behind by the quarry operation. The upper 30 feet of rock has been very disturbed by quarrying so it is not certain if this material is in place or muck. Eight feet of Orange Mountain Basalt was cored at the top of rock. From there to tunnel invert elevation the rock is red-brown Passaic Formation sandstone of various textures. A few thin shale beds are located in the upper portion of the hole. Most of the rock in this hole is brecciated or fractured by high angle irregular shears. Mineralization consisting primarily of calcite is widespread in the brecciated zones. Slickensides and clay fillings are also common. This area is obviously in the Montclaire Fault Zone and will present a lot of difficulties in sinking the shaft. Several holes should be drilled in this area in the future to try and find a place that is less broken up for locating this shaft.

Hole DC-99 is the closest boring to Work Shaft 6. Based on it the work shaft will be excavated in very fractured rock of the Preakness Mountain Basalt. Very low RQDs extend to a depth of 140 feet after which they improve considerably. A two foot core loss occurred at 70 feet which probably indicates a very soft or broken zone that may be a shear or fault plane. The highly broken nature of the rock indicates that it has been disturbed in some way. Staining and mineral coatings are present on most of the joint surfaces and many fragmented zones are reflected in the core recovery. A 3.4 foot thick shale seam was encountered at a depth of 180 feet. This was overlain and underlain by layers of vesicular basalt indicating that it probably marks the boundary between two flows. A loss of 0.6 feet occurred in the reddish-brown shale. The basalt below the shale layer is of much better quality than that above.

Hole DC-102 was drilled directly over the proposed location of Work Shaft 3. The Towaco Formation at this location is mainly shale and mudstone with interbedded, thin layers of fine grained sandstone. Most of these rocks are of good quality and vary from dark reddish-brown to dark grey. Some of the sandstones are cross-bedded and most contain abundant mica. Carbonaceous material is present in all of the rock types. High angle jointing was very infrequent in this boring.

Work Shaft 4 is close to boring C-98 and Work Shaft 6. It will be excavated in Preakness Mountain Basalt that is of good quality. A fine crystalline, vesicular zone extends from the top of rock at 33 feet to a depth of 90 feet. It contains some thin layers of dense basalt and amygdaloidal layers. Below 135 feet the basalt shows signs of significant disturbance. A loss of 0.8 feet occurred at 141 feet and mineralized seams, chemically altered rock, and mineralized breccia are mixed with dense fractured basalt from there to the bottom of the hole. A possible fault plane containing waxy, green, slickensided, gouge material, 0.1 to 0.4 feet thick was cored at 218 feet. Additional exploration is required to provide more information on this shaft location.

4.5 LABORATORY ROCK AND SOILS TESTING

4.5.1 General The laboratory test program for the soils has been aimed at providing for the shaft and surface structure design. The rock mechanics laboratory data has focussed on the tunnel design but much of the data is equally applicable to the shafts and inlets.

4.5.2 Laboratory Testing, Rock This information is covered in paragraph 3.5 above. The testing of most concern for the work shaft construction and stability is the unconfined strength, the unit weight, and the shear strength testing.

4.5.3 Laboratory Testing, Soils Testing to date for the work shafts consists of: triaxial (R-Bar) test and consolidation tests for work shaft 2B and triaxial (R-Bar) test for work shaft 3. (See Attachment E.4.1 for laboratory soil test data. Soil shear strengths were typically determined from the results of the triaxial shear tests. Where test data was unavailable, soil shear strengths, unit weights and other design parameters were determined from the soil boring data and engineering correlations. Shafts settlement estimates were based on the results of the consolidation and index property tests.

4.5.4 Soil and Rock Design Parameters The workshaft and ventshaft liners were designed based on the soil and rock strengths determined from the laboratory testing program and engineering correlations. Earth pressure coefficients, soil unit weights and cummulative earth and hydrostatic pressures are tabulated for each work shaft within Attachment E.4.2. Also included in these tables are the cummulative total design pressures (soil, rock and hydrostatic) and design details for each shaft liner.

4.6 DESIGN CONSIDERATIONS

4.6.1 Overburden The design conditions for the overburden include water control, stability of the excavation, positive contact at the rock interface, and limited work areas for construction. Water control is very critical, especially at the rock interface. The design of the work shafts must incorporate these items and insure the necessary precautions have been taken into consideration.

4.7 DESIGN ALTERNATIVES

4.7.1 Overburden Several alternatives for the work shaft design have been considered and two alternatives have been chosen for this project. Work shafts 2C and 2B will utilize ground freezing to create a freeze wall through the overburden soils and extending several feet into rock. This alternative, though costly, will provide a stable excavation with efficient groundwater control to significant depths. This method also reduces excavation quantities by freezing the insitu soils to create a structural excavation wall and thus minimizes site disturbance. The location of work shaft 2C is in an area where contamination is expected and a process that will minimize disturbance at the work site is critical. Work shaft 2B is located in an area with a "buried valley" where overburden is very deep thereby eliminating other, more conventional methods. The remaining work shafts will be constructed using a structural slurry wall. This process greatly reduces the required work area and is very applicable to depths up to 100 feet.

4.8 STABILITY ANALYSES

4.8.1 Wedge Failure Analysis For the shafts the critical mode of failure is the wedge failure illustrated by Figure E.4.10. The wedge of rock that is formed by the intersection of two joints is only free to move into the shaft if the following conditions are met: The orientation of the joints form a wedge of rock whose line of intersection is sloping at an angle greater than the Phi angle of the joint planes. The top surface of the wedge must be a separated bedding plane that has no tensile strength. The limits of the wedge do not extend beyond the perimeter of the shaft. For this study the joint sets which were derived by the discontinuity study described in paragraph 3.4.3.2 were used to generate the wedges. Some of these wedges were overhanging, that is both planes were dipping in roughly the same direction. The largest wedge that was possible given the shaft dimensions and the joint geometry was computed. The stability of the wedge was then evaluated using the sub-routine "COMPWEDGE" in the computer program "ROCKPACK" by C.F. Watts. This program follows the wedge failure procedure described in Hoek and Bray. The wedge stability was computed for both a wet case and a dry case. In cases where the wedges were found to be unstable they were evaluated with the rock bolting installed. Most were found to be stable with the additional support provided by the pattern bolting which is described in paragraph 4.9.2.4 and shown within the figures included in Attachment E.4.4. A few of the larger wedges required spot bolting to provide an adequate safety factor for the wet case. This procedure provides a good first approximation of the wedge failure analysis however in the next phase of design additional exploration will be needed at the shafts to get more site specific information on the discontinuities which are present at each site. These orientations can be compared to those used in this phase of design and the analysis updated if needed. The results of the wedge failure analysis are included as Attachment E.4.4.

4.9 CONSTRUCTION CONSIDERATIONS

4.9.1 Site Development Considerations

4.9.1.1 Shaft 2C and Dewatering Pump Station Shaft This site is largely covered with fragmities vegetation and will require little clearing. The site has been used for disposal of a chipper collection service. This very compressible material will have to be removed in some areas to provide firm foundations for construction operations and for any permanent structures associated with the dewatering pump station. This area may require a cofferdam or built up area to get the collar of the work shafts above a set storm surge elevation.

4.9.1.2 Shaft 2B This is a wooded site containing abandoned waste dumps of uncertain content. It would be desirable to minimize the tree clearing, grubbing, and other ground disturbing operations however the rail yard for muck handling will have to be constructed on engineered fill. The site is adjacent to Frank Creek which is heavily polluted but will require protection from run off from the construction site. Generally security fencing will be required at all of the sites.

4.9.1.3 Shaft 2A This site is conspicuous to a major thoroughfare, a large residential population, and a golf course. It has been located to minimize impacts on these but it will be disruptive until the shaft is constructed. For this reason setting up a batch plant at this location should be carefully considered. It may be necessary to clear a few trees to gain access to this site. This should be kept to a minimum. An access ramp will have to be constructed to Joralemon Road.

4.9.1.4 Shaft 5 This site is in a wooded area adjacent to the Garden State Parkway and a rest stop. Clearing and grubbing will be required and will adversely affect the property. Security fencing will be required also.

4.9.1.5 Shaft 2 This site is located in an abandoned quarry on Montclair State College property. The site has scattered trees which will have to be cleared. During heavy rainfall the site partially fills with water up to a depth of 5 feet. Backfill of the site and drainage will have to be provided. The most logical approach to developing this site would be to use it as a spoil area. This has two advantages. One, it minimizes the haul distance and placement costs for approximately 500,000 cubic yards of excavated material and two, it results in several acres of developable land upon completion of construction. There is a small ponded area adjacent to the abandoned quarry wall. This surface water area will be lost as a result of backfilling the quarry.

4.9.1.6 Shaft 6 and 4 This site is in a heavily wooded area near a commercial area between Interstate 80 and Route 46. Numerous trees will have to be cleared. In addition, a small unnamed drainage course will require protection from runoff from these sites. The access corridor to Highway 46 should be located to minimize clearing. It is important to keep the sites clear of the Wanaque Aqueduct which cuts through the property north of the work shafts.

4.9.1.7 Shaft 3 Numerous trees will have to be cleared. In addition two small unnamed drainage courses will require protection from runoff from this site. The access road to the Wayne Municipal Yard will probably have to be relocated slightly. The entire site will have to be surrounded by a security fence.

4.9.2 Shaft Construction Methods

4.9.2.1 In Soil

The freezewall process used for work shafts 2C and 2B involves encircling the work shaft area with freon-filled pipes. This will, over a period of 6 to 9 months, freeze the soil for a thickness of approximately 3-4 feet, acting as a water control device for the work shaft. The frozen soil strength is normally 500-600 psi in sand and 300-400 psi in clay. The freon pipes must be maintained throughout the construction phase. Excavation of the work shaft will occur after adequate time for freezing has been allocated. After excavation, conventional reinforced concrete walls will be constructed within the freezewall. Upon construction completion, the piping will be removed and the soil allowed to thaw, usually taking 6 to 9 months.

The remaining work shafts will incorporate a structural slurry/concrete wall. The structural slurry/concrete wall will reduce excavation and provide an impermeable barrier as a form of water control through the overburden material to top of rock. The slurry/concrete wall will be a structural wall and provide support and bracing against the water and soil pressures. The wall will encircle the work shaft areas and be keyed into rock. To construct the wall, a slurry trench will be excavated to the required depth. The slurry will consist of a 5 percent bentonite/95 percent water mixture. The exact mixture will be defined later, at which time chemical analysis of the soil will be required to finalize the mix ratio. The necessary reinforcing steel will then be placed in the trench and the slurry will then be displaced by structural concrete, thereby constructing the wall without excessive excavation of the area. The slurry will be recovered as the concrete displaces it.

4.9.2.2 <u>In Rock</u> For the cost estimate it was assumed that all of the shaft sinking in rock would be accomplished by drill and blast with muck removal by skip pan and crane, using loaders at the bottom of the shaft. The rock would be shot in five foot lifts with approximately one foot of sub-drilling. This will be a very time consuming process but it is dependable and commonly used. Other alternatives include mechanical shaft sinking, raise boring, and blasting, are possible. Shaft 2A may be constructed by raise bore technique in which case the excavated material could be hauled out at work shaft 2B.

4.9.2.3 Vibration Control All of the shafts are located in developed areas that will be affected by blasting vibrations. Control of blasting and monitoring of nearby structures will be necessary. A pre-blasting inventory of all adjacent structures should be a part of the construction activities on these sites. Work Shaft 2 has very minimal cover over the rock so blasting and vibration control are likely to be more problematic here.

4.9.2.4 Rock Support

A vertical shaft through near horizontally bedded rock is an inherently stable configuration. Rock support for the current plan is to be provided by resin encapsulated bolts. The cost estimate is based on using #8, grade 60 deformed bars having an ultimate capacity of 60 kips and a recommended design load of 30 kips. These bolts should be tensioned to about 22.5 kips. Attachment E.4.4 indicates the proposed configuration of the bolting around the shafts. In general it was assumed that an approximate 8 foot by 8 foot spacing on the rock bolts would be adequate to ensure stability of the shafts. The 8 foot spacing dictated how many bolts would fit around the perimeter of the shaft. For the 15-foot inside diameter shafts the actual excavated diameter would be 17 feet to accommodate a 1 foot concrete liner. This results in 7 bolts per row with 7' 7½" spacing between the bolts. Each bolt is 8 feet long. For the 42-foot inside diameter shafts the actual excavated diameter would be 45 feet to accommodate a 1½ foot concrete liner. This results in 18 bolts per row with 7' 10" spacing between the bolts. Each bolt is 12 feet long. For work shaft 4 the actual excavated diameter would be 25 feet to accommodate a 1 foot concrete liner. This results in 10 bolts per row with 7' 10" spacing between the bolts. Each bolt is 10 feet long. The bolt spacing for all shafts was checked through the wedge failure analysis, described above, and, in general, found to be adequate. Longer, higher strength, 150 KSI steel bolts may be used for spot bolting of critical wedge blocks. This spot bolting quantity is appropriately covered by contingency in the cost estimate. Additional bolts are also required in the crown of the tunnel where the shaft intersects it. These would be angled into the crown to provide additional support at this critical location.

This bolt pattern will require future study but is also covered under the contingency in the cost estimate.

For added protection from rock falls, welded wire mesh is to be used between the bolts. Full perimeter coverage with mesh is planned in the large shafts. In the smaller shafts, which are more stable because of their size, only 20% of the perimeter area was assumed to require wire mesh. Ultimately the shafts will be lined with unreinforced concrete as described in Appendix G. The wire mesh will be incorporated in this concrete liner.

4.9.3 <u>Ground Water Control</u> Groundwater seepage into the shaft excavations will be minimized by slurry walls and freeze walls in the overburden soils and by cement grouting in rock. Ground water considerations are discussed in Section 2 above. It is anticipated that ground water inflows through the rock will be handled through normal sumping. The depth of the shafts will require specialized pumps.

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 4
SHAFTS

FIGURES

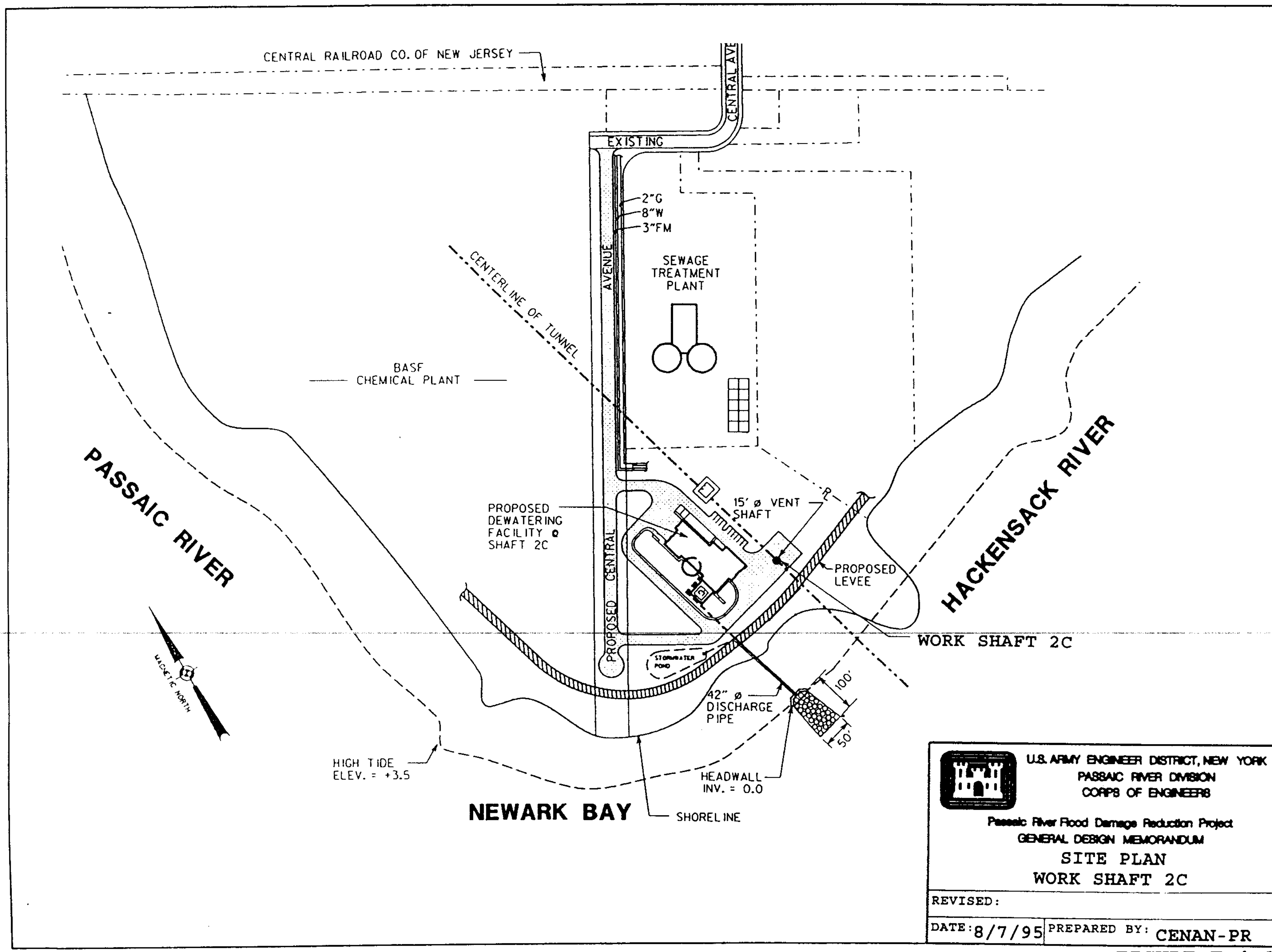
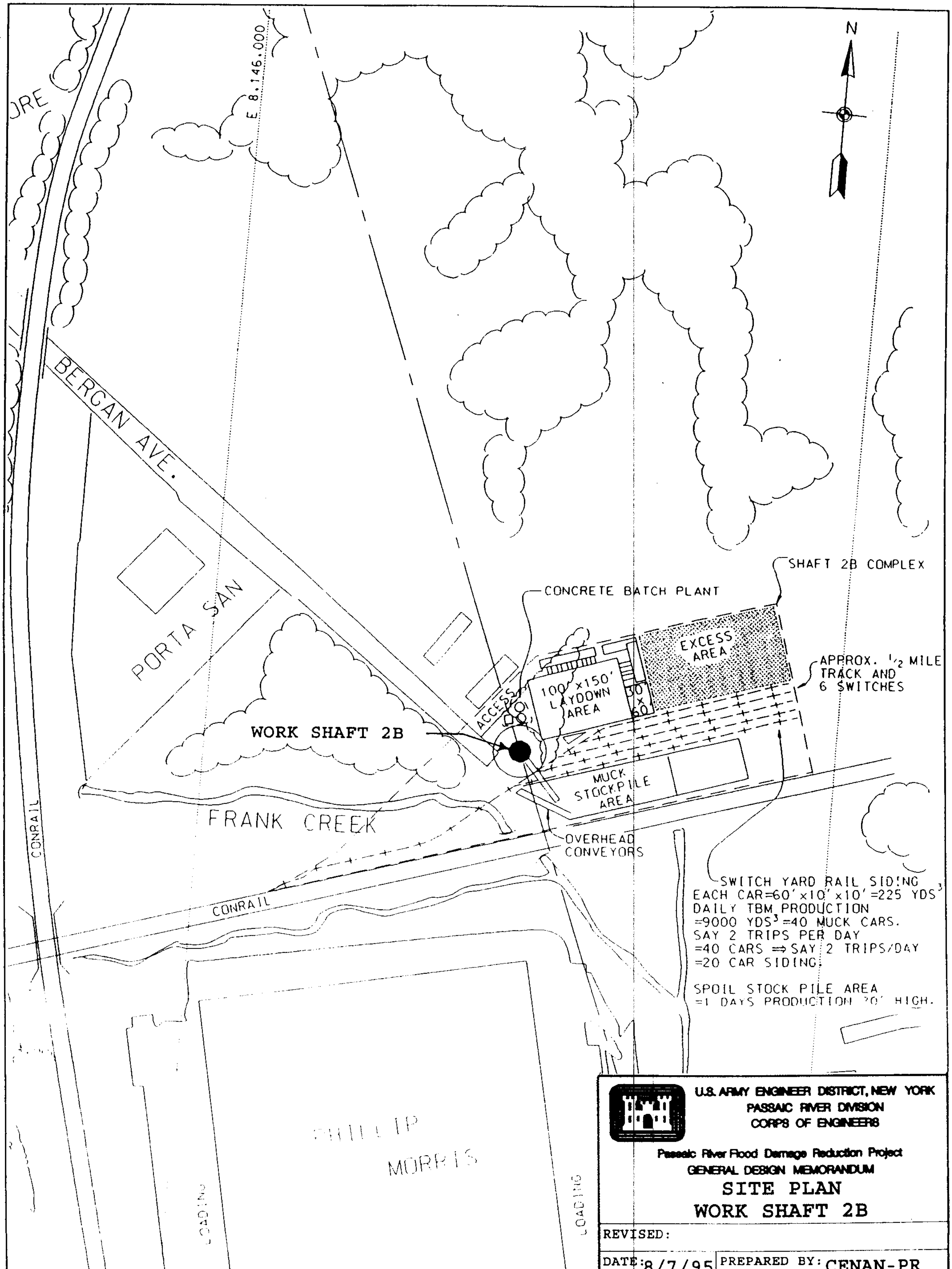

CENTRAL RAILROAD CO. OF NEW JERSEY
CENTRAL AVE
EXISTING
2"G
8"W
3"FM
AVENUE
SEWAGE TREATMENT PLANT
CENTERLINE OF TUNNEL
BASF CHEMICAL PLANT
PASSAIC RIVER
HACKENSACK RIVER
15' ø VENT SHAFT
PROPOSED DEWATERING FACILITY @ SHAFT 2C
PROPOSED CENTRAL
PROPOSED LEVEE
WORK SHAFT 2C
STORMWATER POND
42" ø DISCHARGE PIPE
100'
50'
MAGNETIC NORTH
HIGH TIDE ELEV. = +3.5
HEADWALL INV. = 0.0
NEWARK BAY
SHORELINE
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
SITE PLAN
WORK SHAFT 2C
REVISED:
DATE: 8/7/95
PREPARED BY: CENAN-PR

FIGURE E.4.1



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

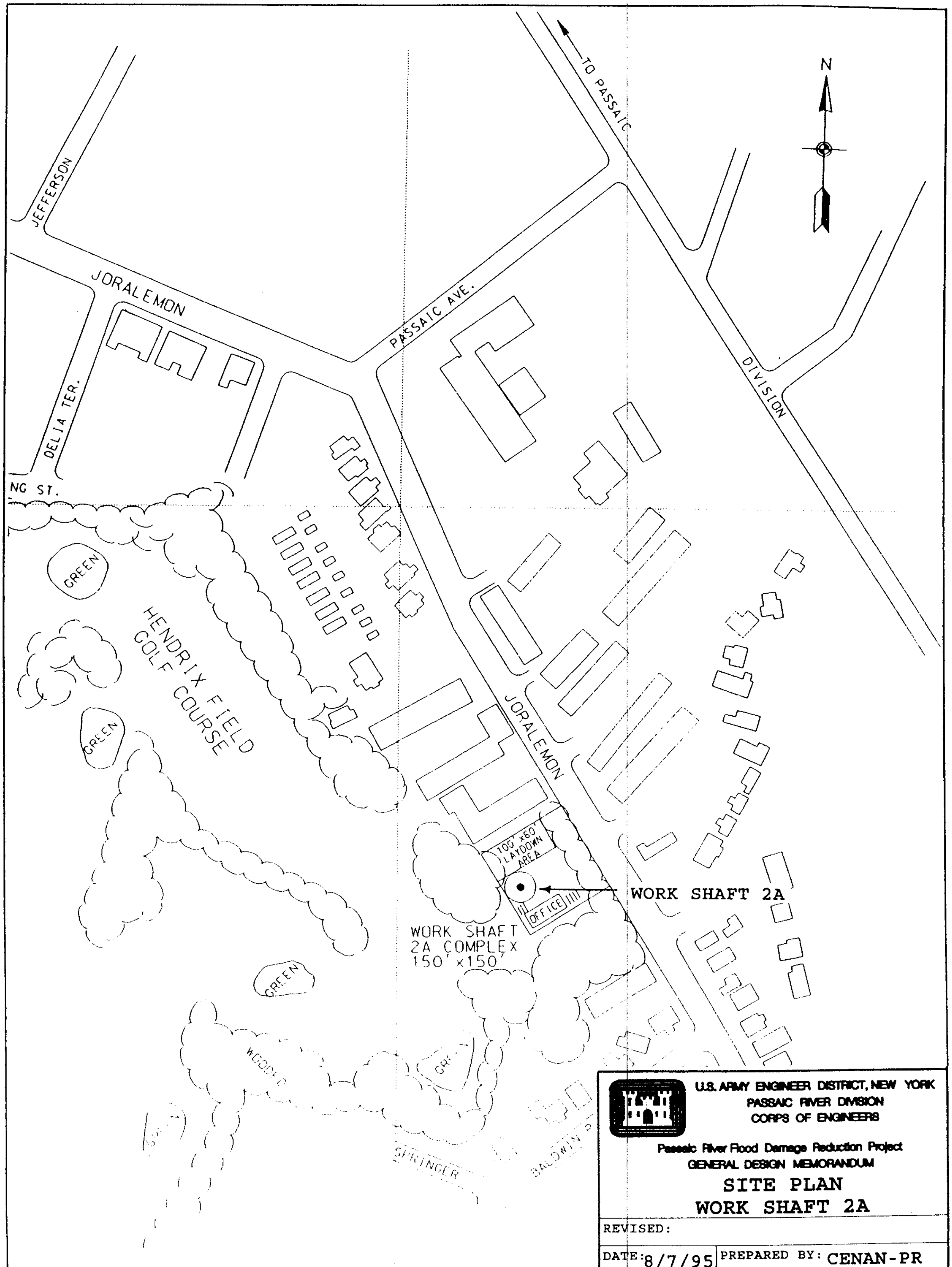
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

SITE PLAN
WORK SHAFT 2B

REVISED:

DATE: 8/7/95 PREPARED BY: CENAN-PR

FIGURE E.4.2



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

SITE PLAN
WORK SHAFT 2A

REVISED:

DATE: 8/7/95 | PREPARED BY: CENAN-PR

FIGURE E.4.3

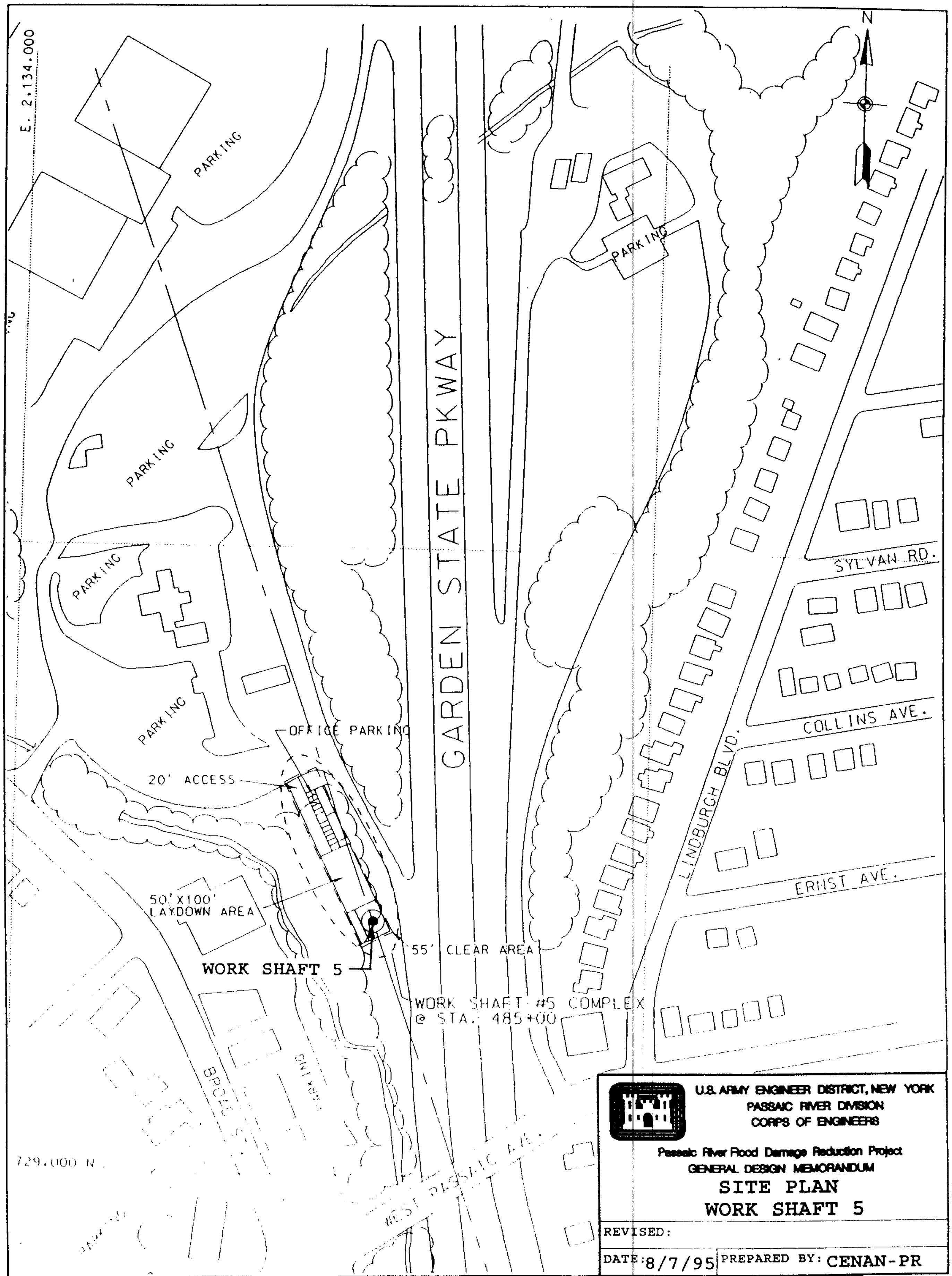
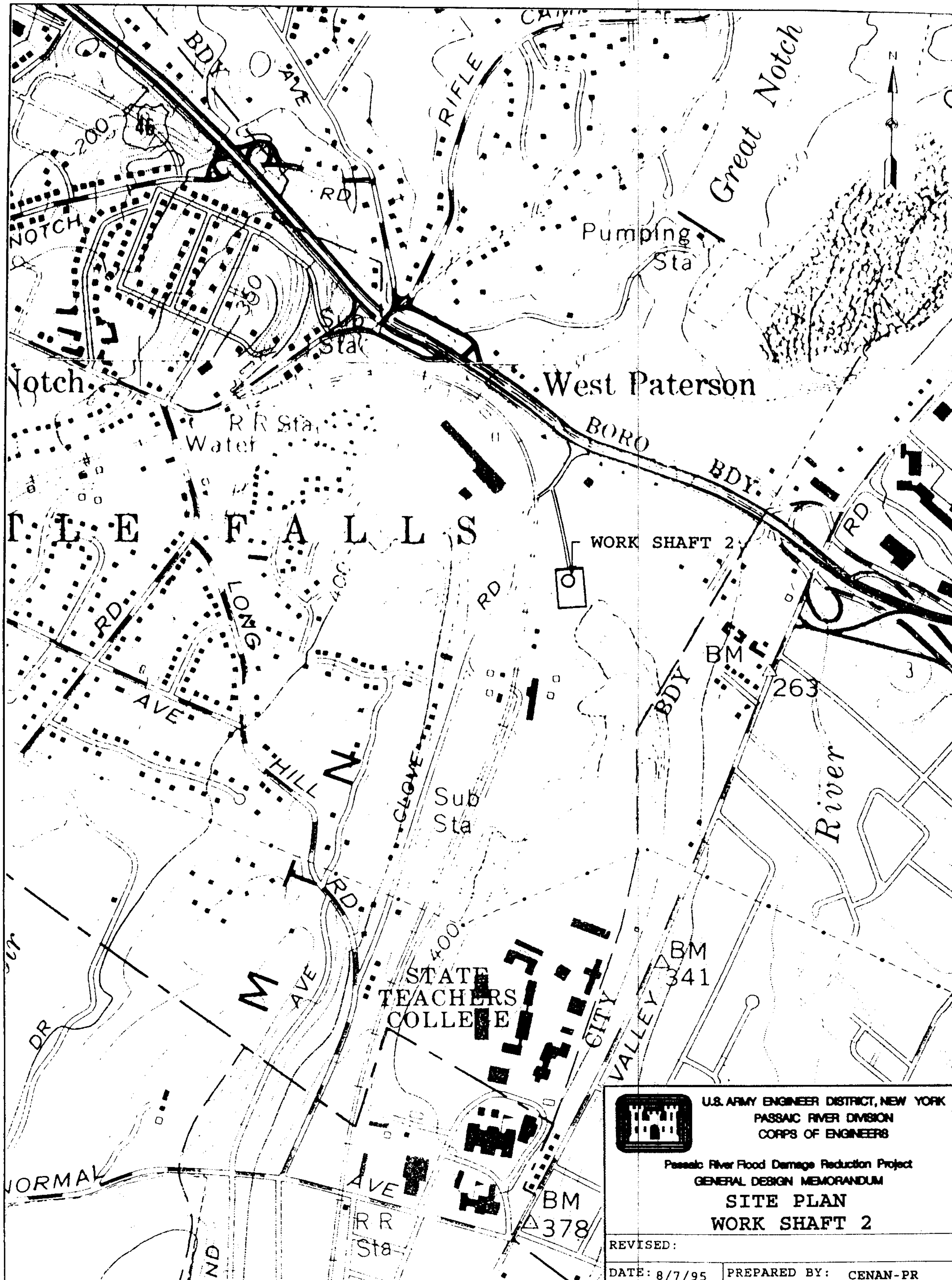

E. 2,134,000
PARKING
PARKING
PARKING
PARKING
PARKING
GARDEN STATE PKWY
N
SYLVAN RD.
COLLINS AVE.
ERNST AVE.
LINDBURGH BLVD.
OFFICE PARKING
20' ACCESS
50'X100' LAYDOWN AREA
55' CLEAR AREA
WORK SHAFT 5
WORK SHAFT #5 COMPLEX @ STA. 485+00
729,000 N
WEST PASSAIC AVE.
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
SITE PLAN
WORK SHAFT 5
REVISED:
DATE: 8/7/95
PREPARED BY: CENAN-PR

FIGURE E.4.4



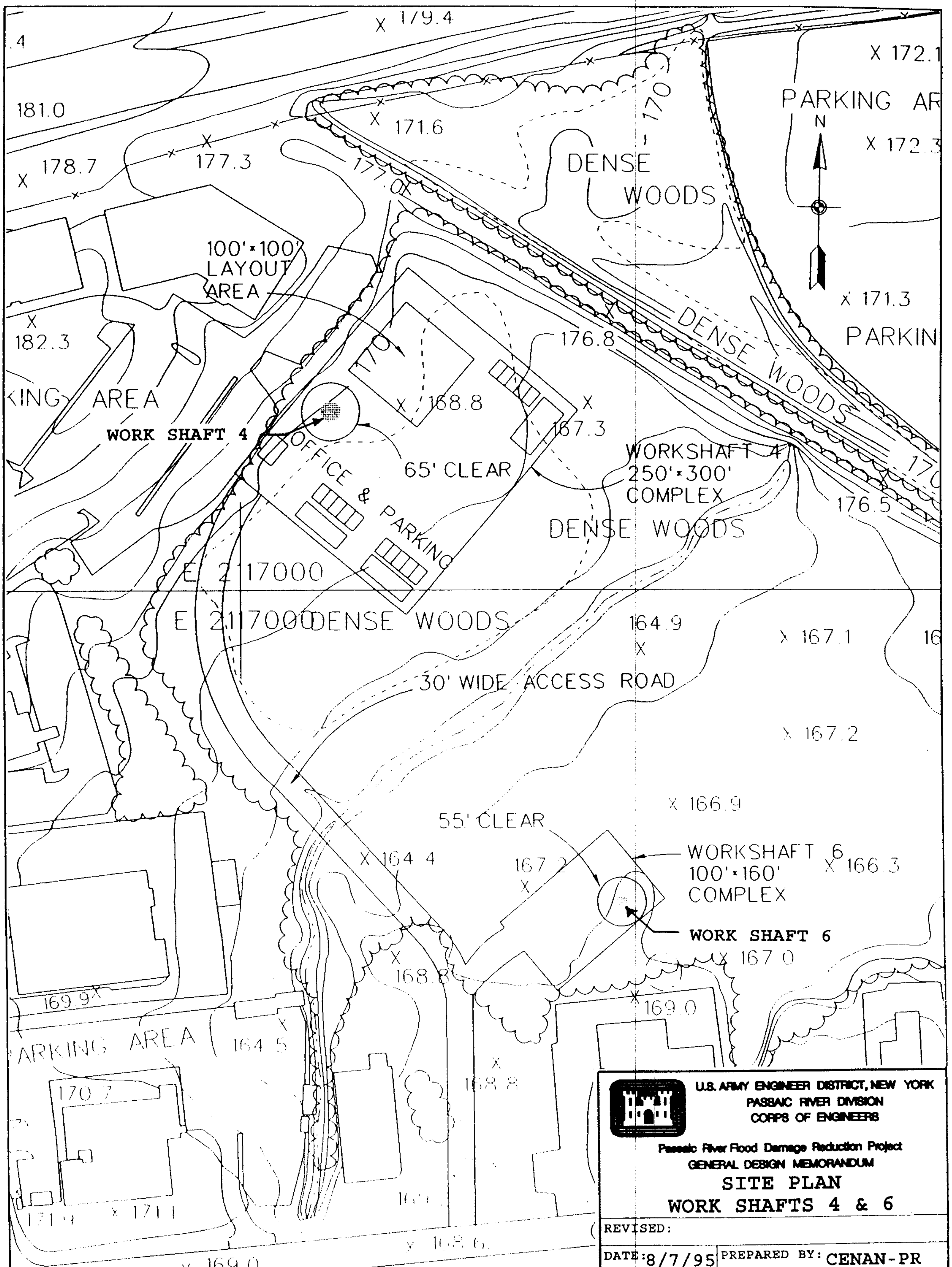
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

SITE PLAN
WORK SHAFT 2

REVISED:	
DATE: 8/7/95	PREPARED BY: CENAN-PR

FIGURE E.4.5



U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS

Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM

SITE PLAN
WORK SHAFTS 4 & 6

REVISED:

DATE: 8/7/95 PREPARED BY: CENAN-PR

FIGURE E.4.6

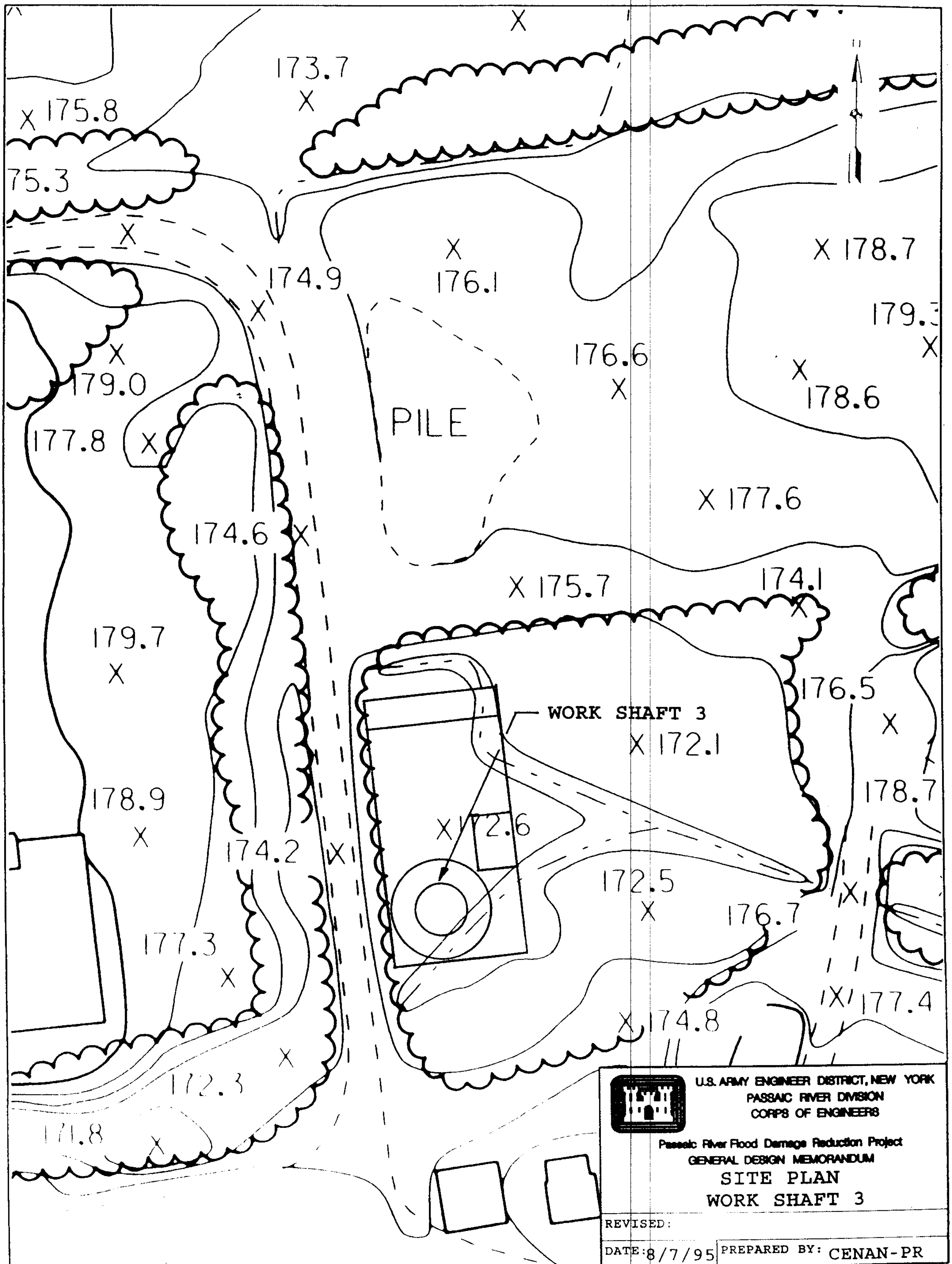

173.7
175.8
75.3
174.9
176.1
X 178.7
179.3
176.6
178.6
179.0
PILE
177.8
174.6
X 177.6
X 175.7
174.1
179.7
176.5
WORK SHAFT 3
X 172.1
178.9
178.7
X172.6
174.2
172.5
176.7
177.3
177.4
X 174.8
172.3
171.8
U.S. ARMY ENGINEER DISTRICT, NEW YORK
PASSAIC RIVER DIVISION
CORPS OF ENGINEERS
Passaic River Flood Damage Reduction Project
GENERAL DESIGN MEMORANDUM
SITE PLAN
WORK SHAFT 3
REVISED:
DATE: 8/7/95
PREPARED BY: CENAN-PR

FIGURE E.4.7

PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT

SECTION 4
SHAFTS

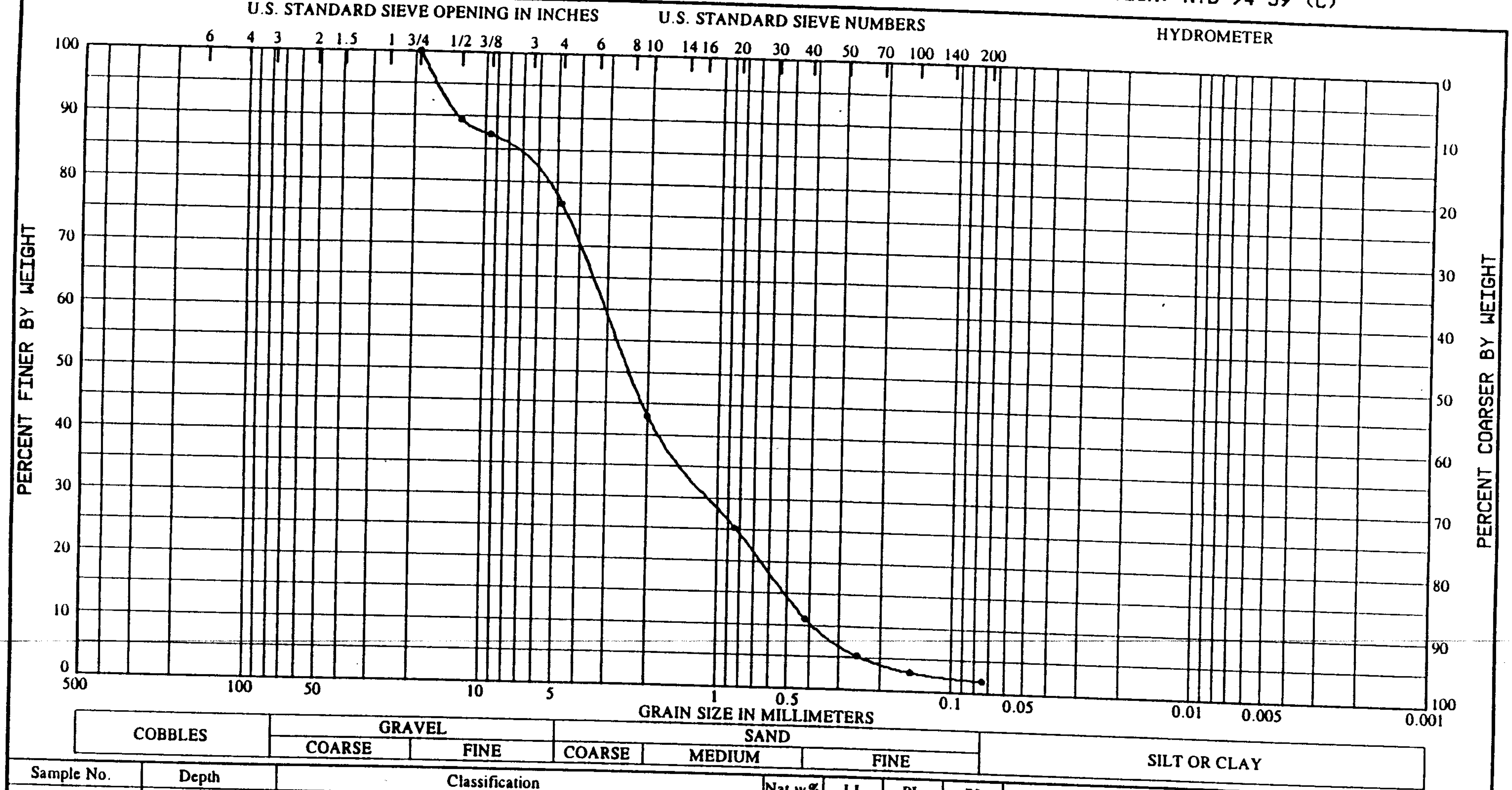
ATTACHMENT E.4.1
LABORATORY SOIL TEST DATA

E.4.1

LABORATORY SOIL TEST DATA

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7319
REQUISITION: NYD-94-59 (C)



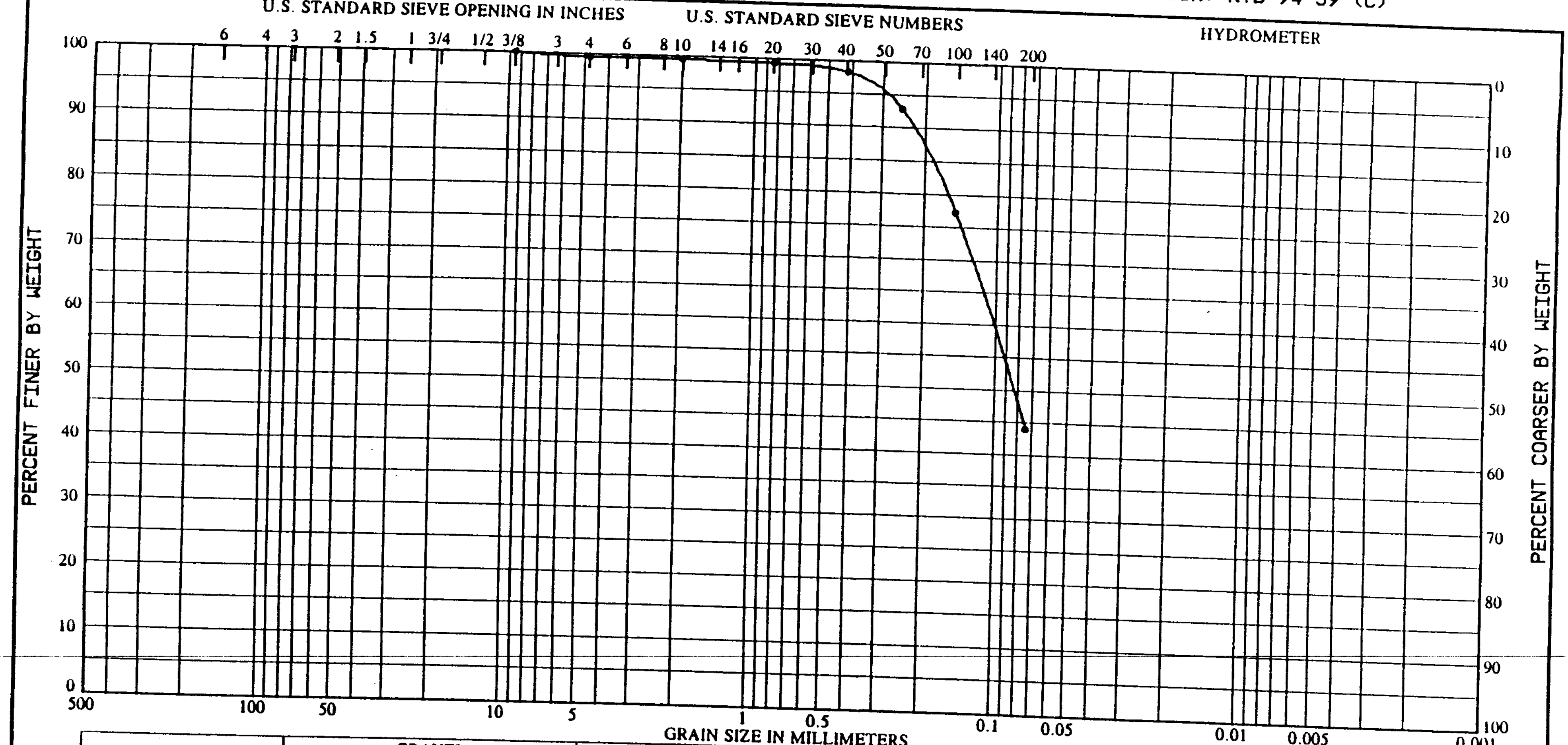
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
40	58.5-59.0'	CLASSIFICATION NOT REQUESTED.				

Project PASSAIC RIVER
WORK SHAFT 3
Lab No. 264/635
Boring No. C-102
Date 07/14/94

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7319
REQUISITION: NYD-94-59 (C)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

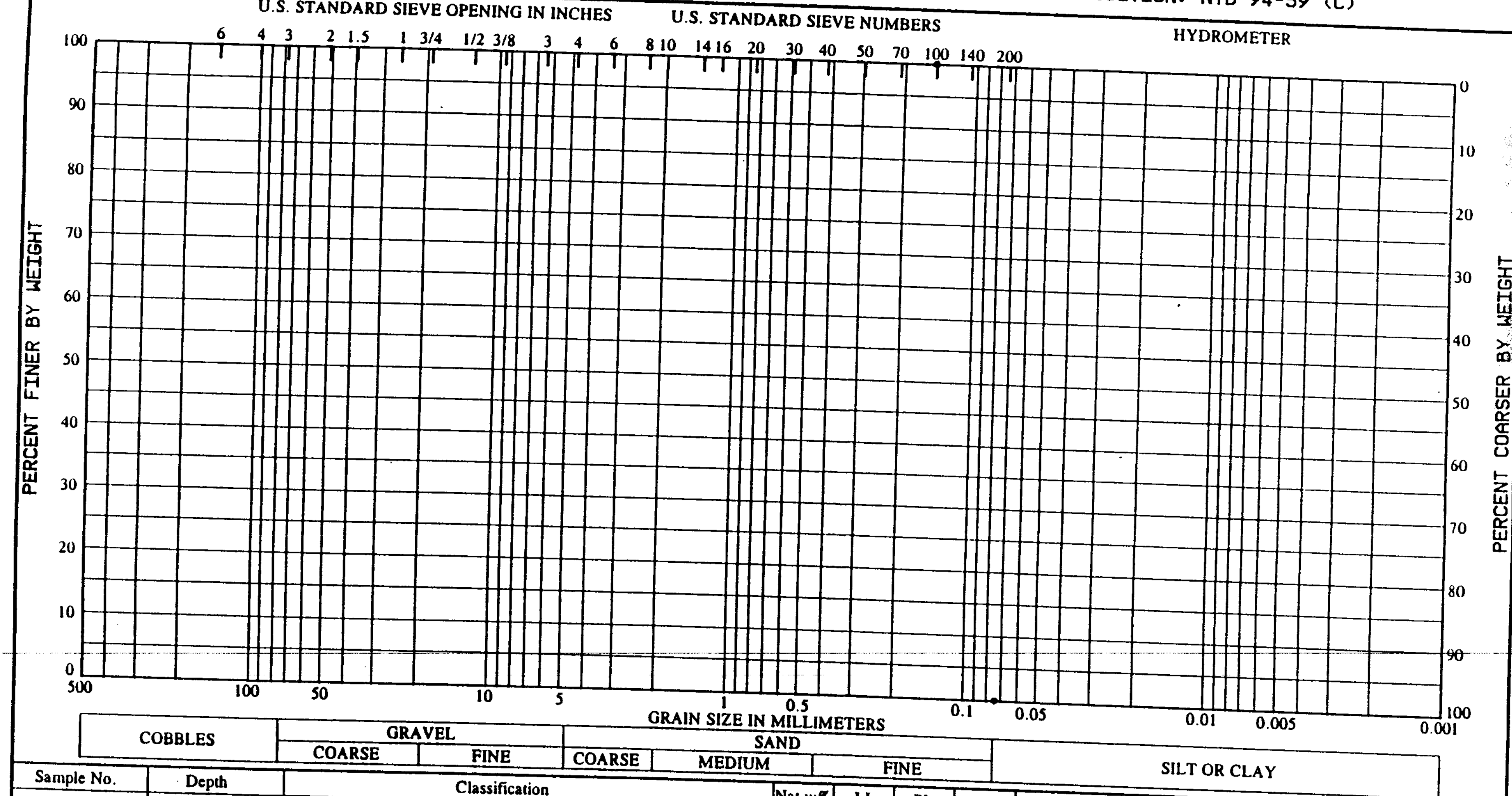
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
36	53.5-54.0'	CLASSIFICATION NOT REQUESTED.				

Project PASSAIC RIVER
WORK SHAFT 3
Lab No. 264/634
Boring No. C-102
Date 07/14/94

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7319
REQUISITION: NYD-94-59 (C)



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
30	44.5-45.0'	CLASSIFICATION NOT REQUESTED.	38.1	42	22	20

Project PASSAIC RIVER
WORK SHAFT 3

Lab No. 264/633

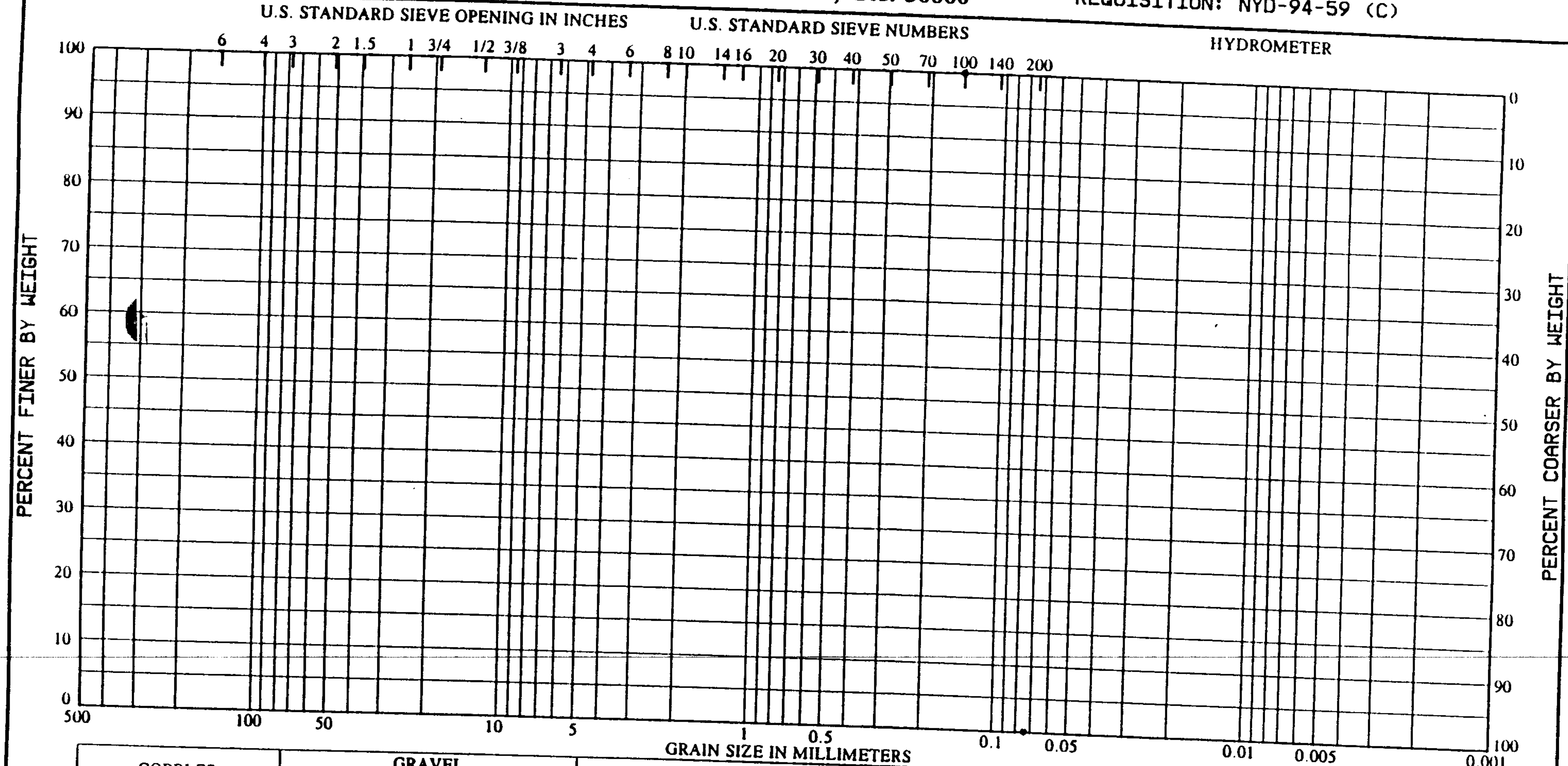
Boring No. C-102

Date 07/14/94

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7319
REQUISITION: NYD-94-59 (C)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

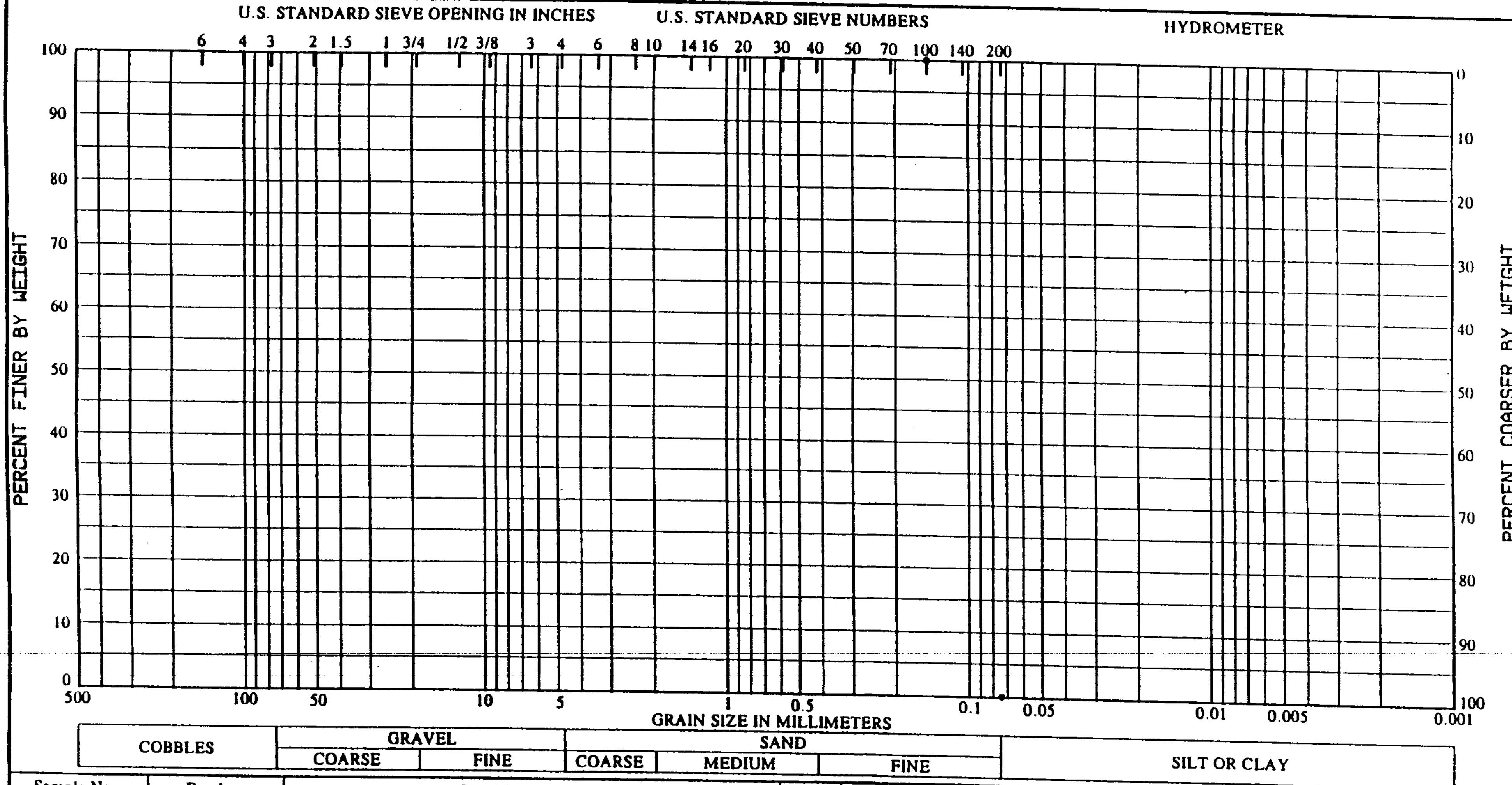
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
22	32.5-33.0'	CLASSIFICATION NOT REQUESTED.		41	21	20

Project PASSAIC RIVER
WORK SHAFT 3
Lab No. 264/632
Boring No. C-102
Date 07/14/94

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7319
REQUISITION: NYD-94-59 (C)



Sample No.	Depth	Classification	Nat w%	LL	PL	PI
17	25.0-25.5	CLASSIFICATION NOT REQUESTED.		27	19	8

Project PASSAIC RIVER
WORK SHAFT 3

Lab No. 264/631

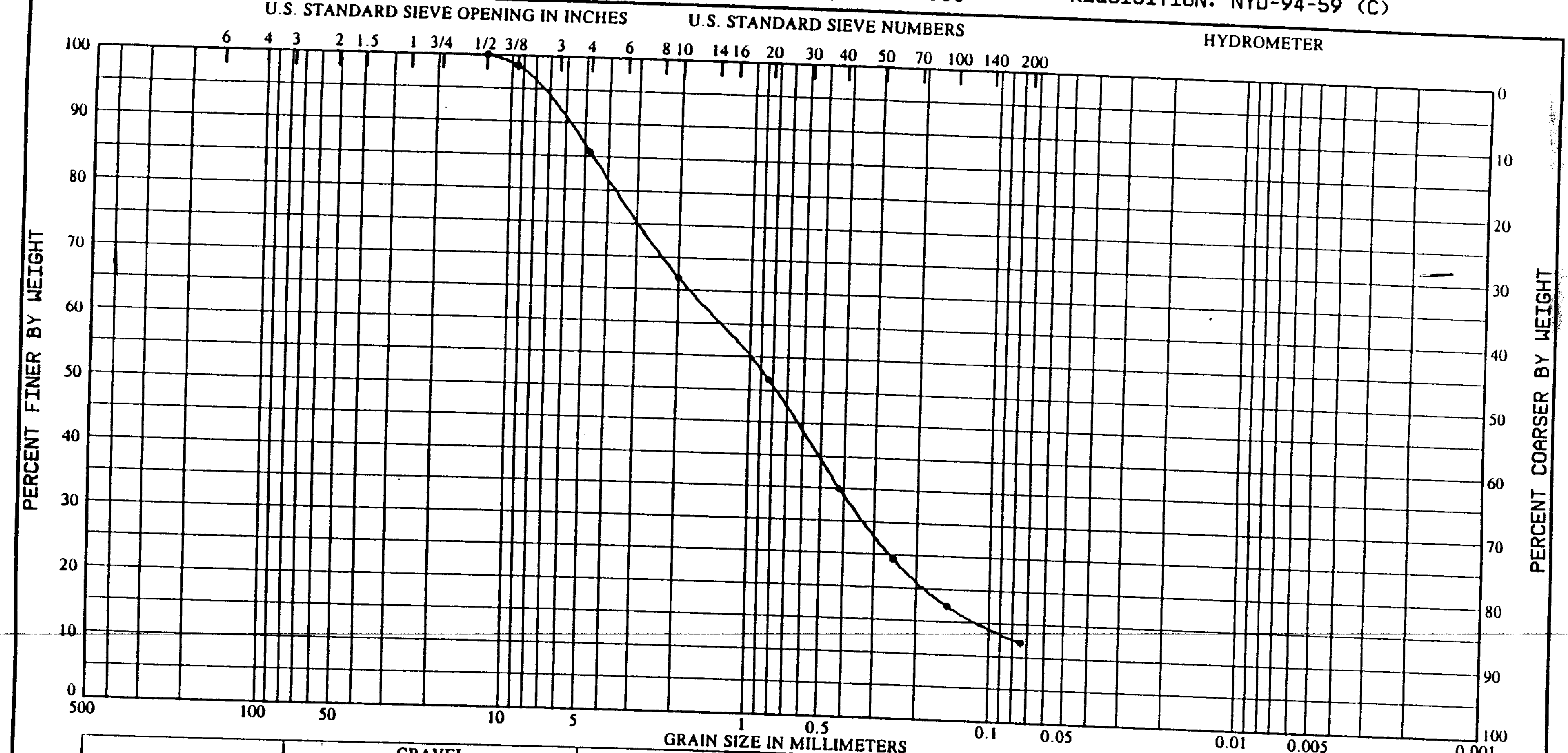
Boring No. C-102

Date 07/14/94

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE , MARIETTA, GA. 30060

WORK ORDER: 7319
REQUISITION: NYD-94-59 (C)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

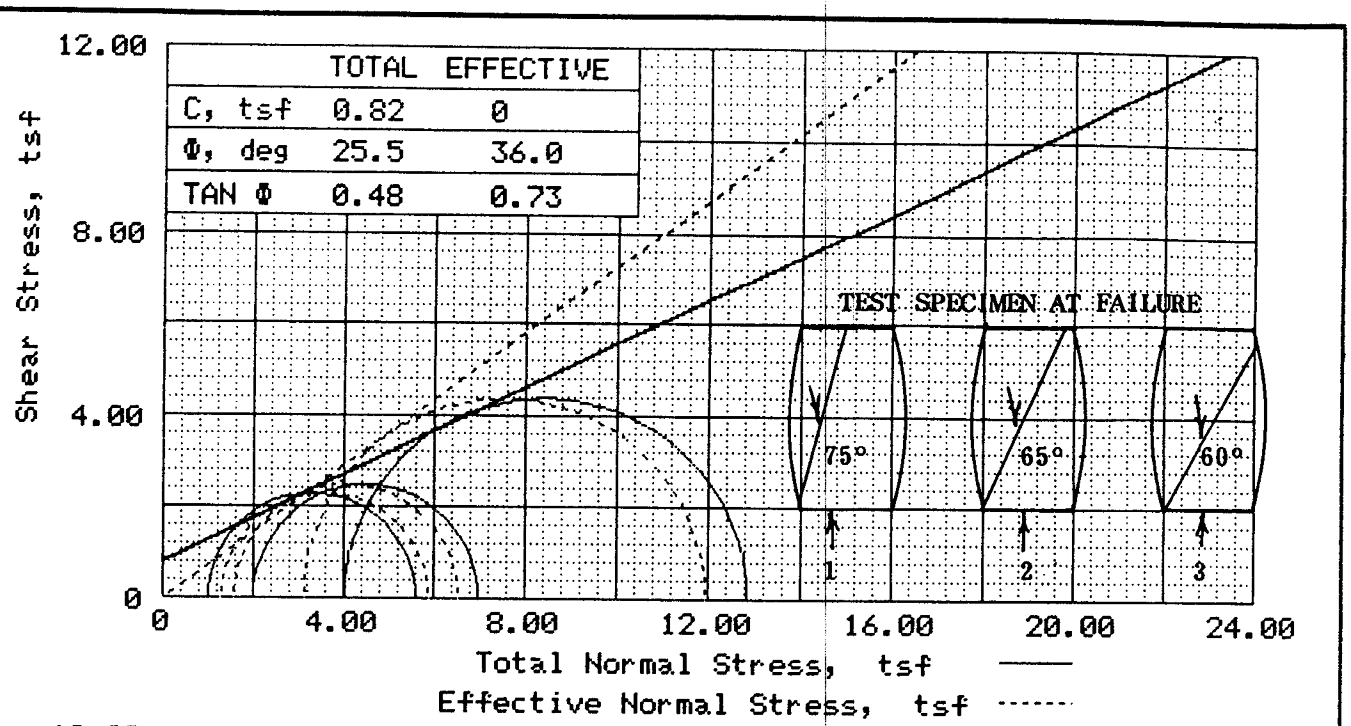
Sample No.	Depth	Classification	Nat w%	LL	PL	PI
3	4.0-4.5'	CLASSIFICATION NOT REQUESTED.				

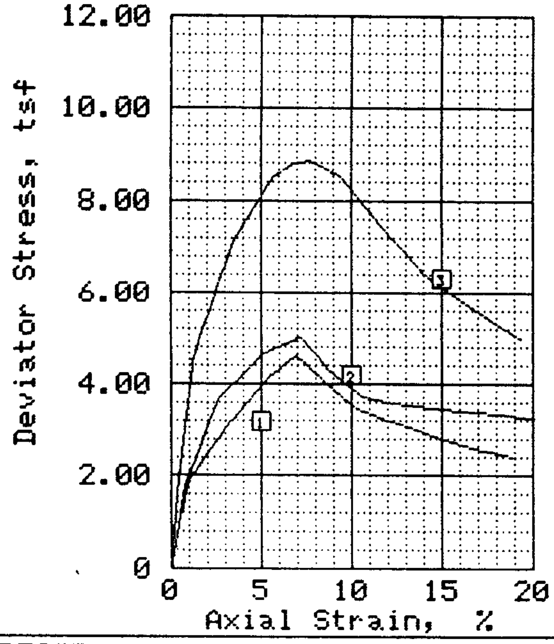
Project PASSAIC RIVER
WORK SHAFT 3
Lab No. 264/630
Boring No. C-102
Date 07/14/94

GRADATION CURVES

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Work Order No. 7319
Requisition No. NYD-94-59(C)

	TOTAL	EFFECTIVE
C, tsf	0.82	0
Φ, deg	25.5	36.0
TAN Φ	0.48	0.73





SAMPLE NO.		1	2	3
INITIAL	WATER CONTENT, %	26.8	27.9	27.7
	DRY DENSITY, pcf	96.9	94.7	95.2
	SATURATION, %	97.9	96.7	97.1
	VOID RATIO	0.740	0.779	0.770
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.08	3.08	3.08
@ TESTING	WATER CONTENT, %	25.1	26.0	24.5
	DRY DENSITY, pcf	100.5	99.0	101.5
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.678	0.703	0.661
	DIAMETER, in	1.36	1.36	1.35
	HEIGHT, in	3.04	3.03	3.02
BACK PRESSURE, tsf		5.04	5.04	5.04
INIT. EFF. STR., tsf		1.01	2.02	4.03
MAX. DEV. STRESS, tsf		4.60	4.98	8.82
PORE PRESSURE, tsf		-0.29	0.43	0.90
TIME TO FAILURE, min.		60	50	65
RATE, %/min.		0.12	0.14	0.12
ULT. DEV. STRESS, tsf		2.78	3.44	6.12

DESCRIPTION (VISUAL) Brownish Gray Lean Clay (CL)

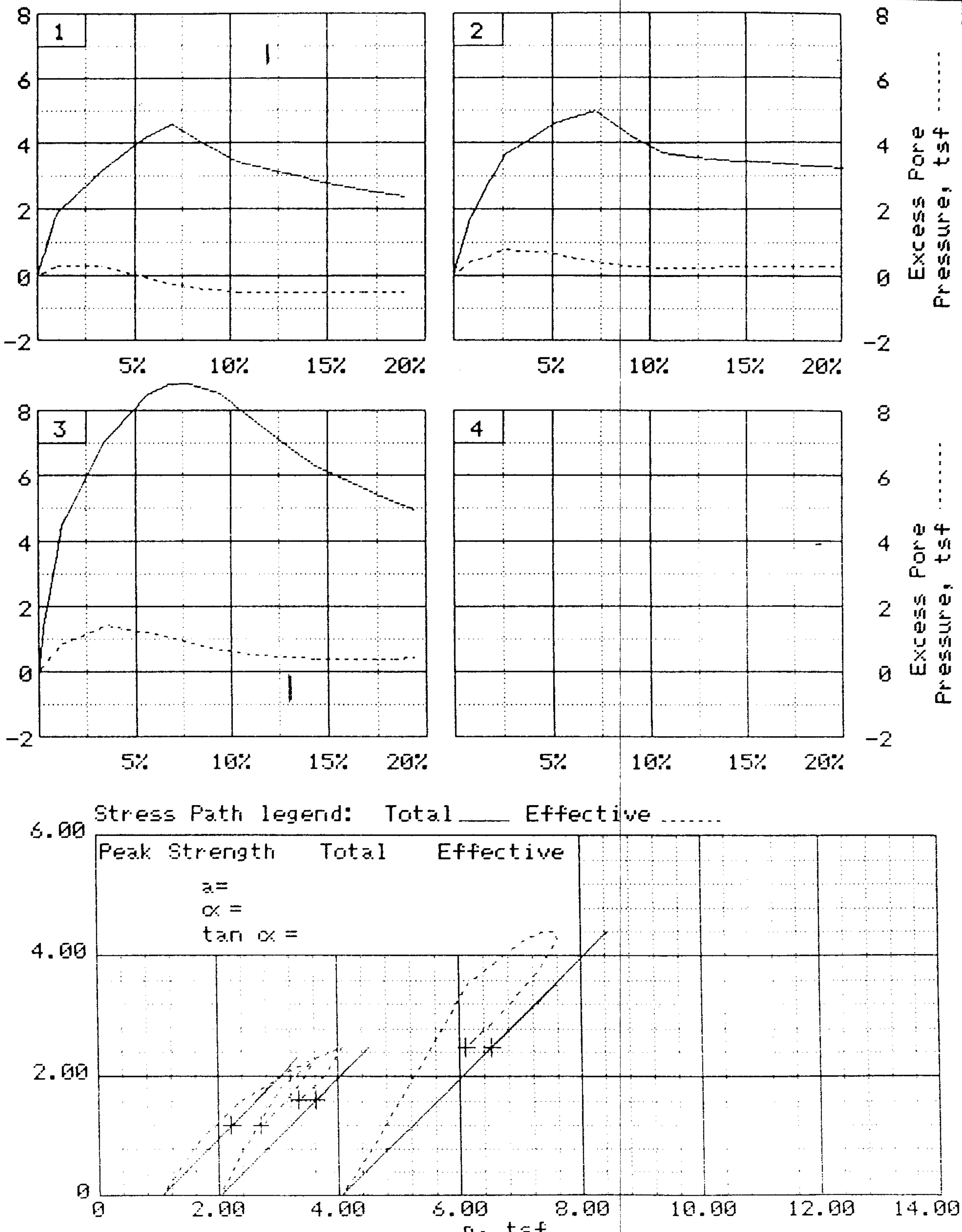
LL= 24	PL= 26	PI= 8	Gs= 2.70	CONTROLLED STRAIN TEST
TYPE OF SPECIMEN UNDISTUEBED				TYPE OF TEST R w/pore pressures

REMARKS

PROJECT PASSAIC RIVER PROTECTION
FLORE, KEARNY
AREA
LAB NO. 184/698
BORING NO. SHAFT 2BFP0
SAMPLE NO. ST-1
DEPTH/~~ELEV~~ 60.0 - 62.0'
LABORATORY CESAD-EN-FL DATE 24 OCT. 1994
Pg 1 of 2 TRIAXIAL COMPRESSION TEST

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA

Requisition No. Work Order No.



PROJECT PASSAIC RIVER PROTECTION FLORE, KEARNY

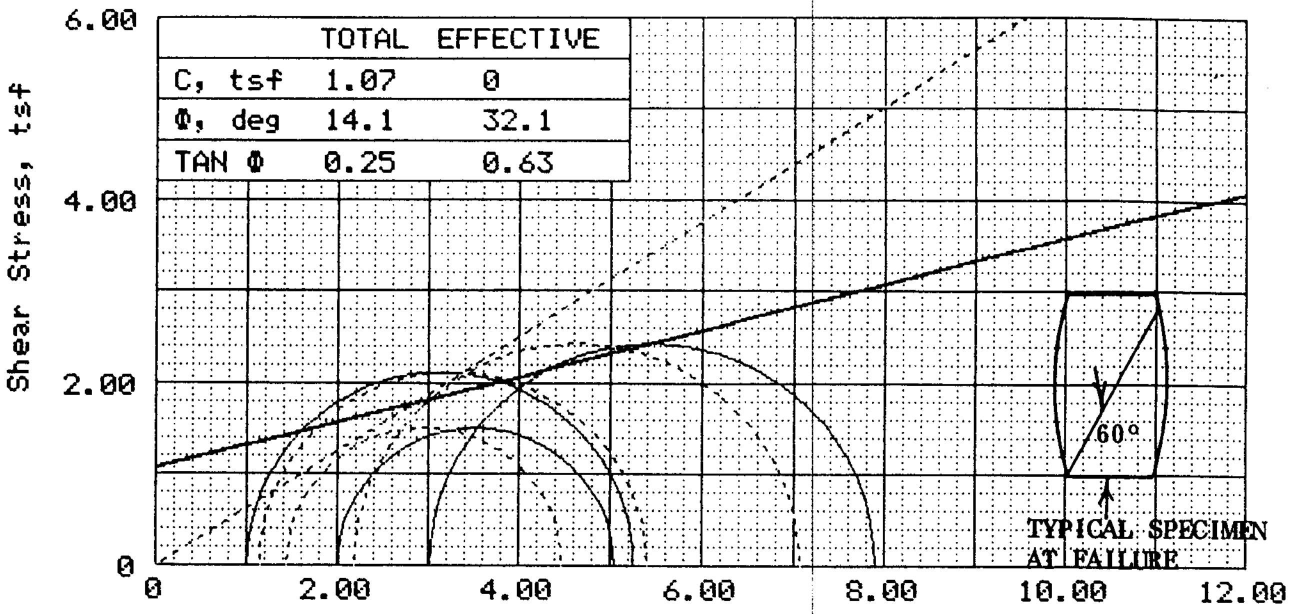
BORING SHAFT 2BFP0 SAMPLE ST-1 DEPTH/~~ELEV~~ 60.0 - 62.0'

TYPE OF TEST R w/pore pressures LAB NO. 184/698

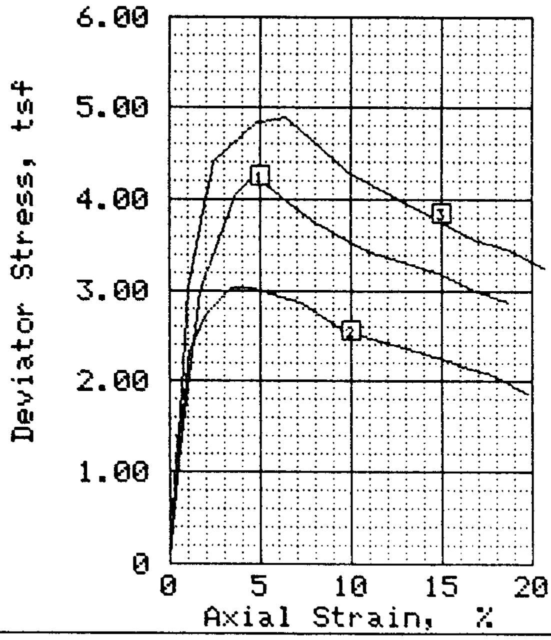
LABORATORY CESAD-EN-FL PAGE 2/2 DATE 24 OCT. 1994

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Requisition No. NYD-94-59 (C) Work Order No. 7319

	TOTAL	EFFECTIVE
C, tsf	1.07	0
Φ, deg	14.1	32.1
TAN Φ	0.25	0.63



Total Normal Stress, tsf ———
Effective Normal Stress, tsf - - - - -



	SAMPLE NO.	1	2	3
INITIAL	WATER CONTENT, %	25.1	26.1	22.6
	DRY DENSITY, pcf	100.4	94.9	99.8
	SATURATION, %	99.8	90.8	88.7
	VOID RATIO	0.679	0.777	0.689
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.08	3.08	3.08
@ TESTING	WATER CONTENT, %	24.6	25.9	20.2
	DRY DENSITY, pcf	101.3	99.1	109.1
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.663	0.700	0.545
	DIAMETER, in	1.37	1.36	1.34
	HEIGHT, in	3.07	3.03	2.99
	BACK PRESSURE, tsf	5.04	5.04	5.04
	INIT. EFF. STR., tsf	1.01	2.02	3.02
	MAX. DEV. STRESS, tsf	4.25	3.03	4.88
	PORE PRESSURE, tsf	-0.14	0.57	0.83
	TIME TO FAILURE, min.	65	35	65
	RATE, %/min.	0.07	0.13	0.10
	ULT. DEV. STRESS, tsf	3.17	2.23	3.74

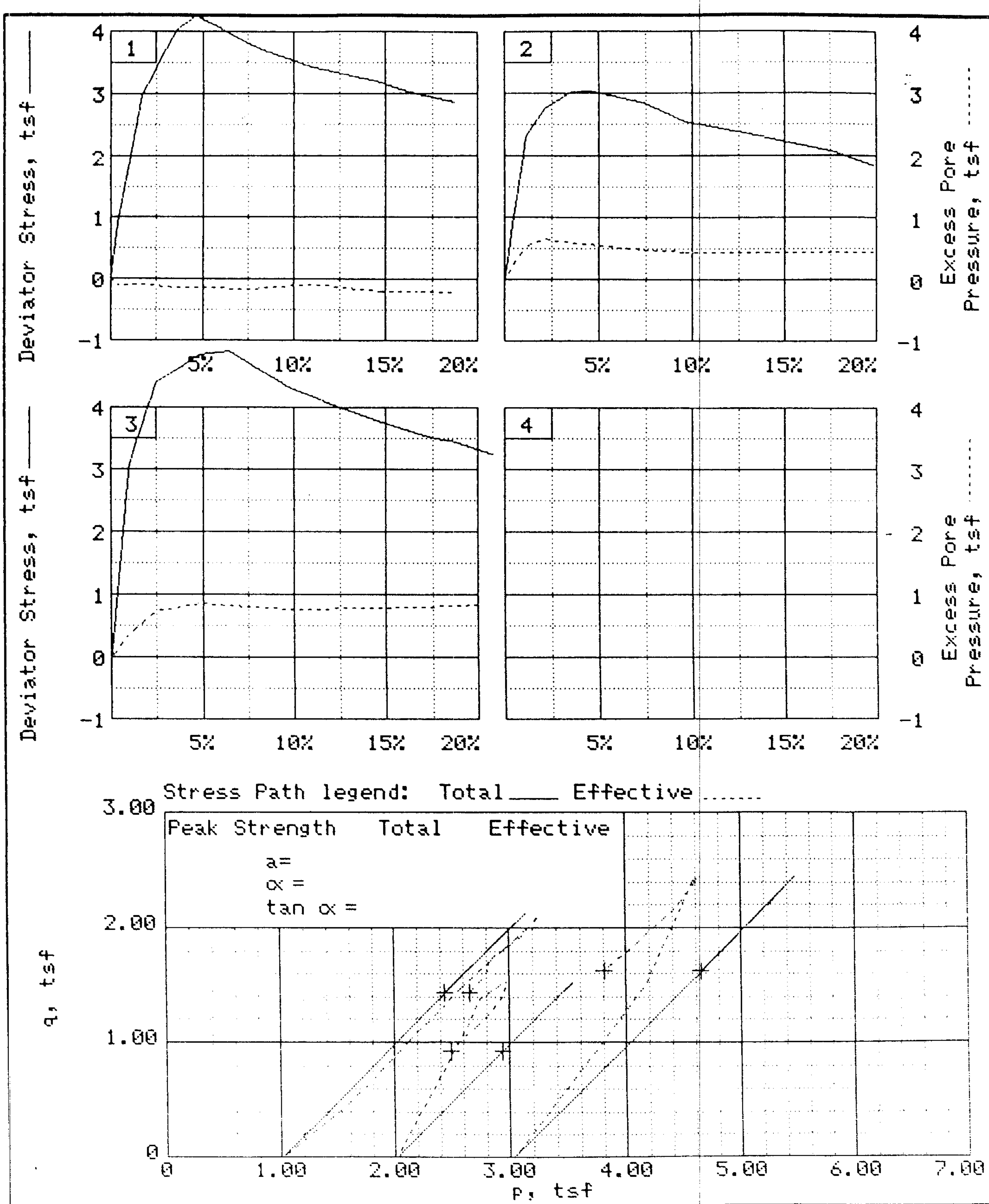
DESCRIPTION (VISURAL) GRAY LEAN CLAY (CL)

LL= 30	PL= 20	PI= 10	Gs= 2.70	CONTROLLED STRAIN TEST
TYPE OF SPECIMEN UNDISTUEBED				TYPE OF TEST R w/pore pressures

REMARKS

PROJECT PASSAIC RIVER
TUNNEL
AREA
LAB NO. 264/689
BORING NO. C-102U
SAMPLE NO. 1
DEPTH/~~ELEV~~ 21.0 - 23.5'
LABORATORY CESAD-EN-FL DATE 24 OCT, 1994
Pg 1 of 2 TRIAXIAL COMPRESSION TEST

Requisition No. NYD-94-59 (C)   Work Order No. 7319



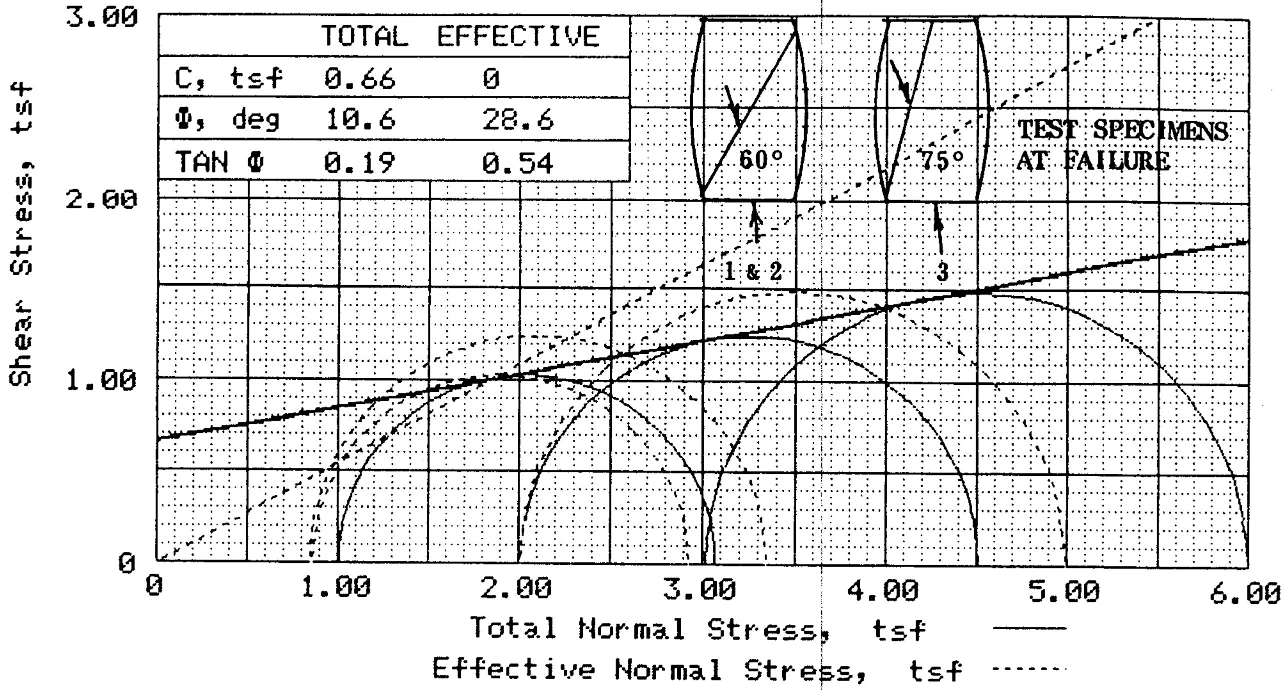
PROJECT PASSAIC RIVER TUNNEL

BORING C-102U   SAMPLE 1   DEPTH/~~ELEV~~ 21.0 - 23.5'

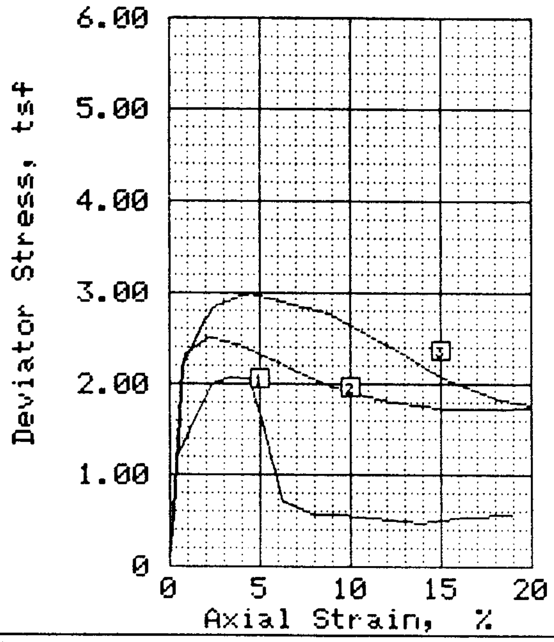
TYPE OF TEST R w/pore pressures   LAB NO. 264/689

LABORATORY CESAD-EN-FL   PAGE 2/2   DATE 24 OCT, 1994

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Work Order No. 7319
Requisition No. NYD-94-59 (C)



	TOTAL	EFFECTIVE
C, tsf	0.66	0
Φ, deg	10.6	28.6
TAN Φ	0.19	0.54



SAMPLE NO.		1	2	3
INITIAL	WATER CONTENT, %	37.0	32.3	37.8
	DRY DENSITY, pcf	84.3	84.6	83.1
	SATURATION, %	99.9	87.9	99.1
	VOID RATIO	1.000	0.993	1.029
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.08	3.08	3.08
@ TESTING	WATER CONTENT, %	35.1	33.5	33.9
	DRY DENSITY, pcf	86.6	88.5	88.0
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.947	0.905	0.915
	DIAMETER, in	1.37	1.36	1.35
	HEIGHT, in	3.05	3.03	3.02
BACK PRESSURE, tsf		5.04	5.04	5.04
INIT. EFF. STR., tsf		1.01	2.02	3.02
MAX. DEV. STRESS, tsf		2.06	2.50	2.98
PORE PRESSURE, tsf		0.14	1.17	1.01
TIME TO FAILURE, min.		45	15	45
RATE, %/min.		0.08	0.15	0.10
ULT. DEV. STRESS, tsf		0.49	1.72	2.08

DESCRIPTION (VISUAL) GRAY LEAN CLAY (CL)

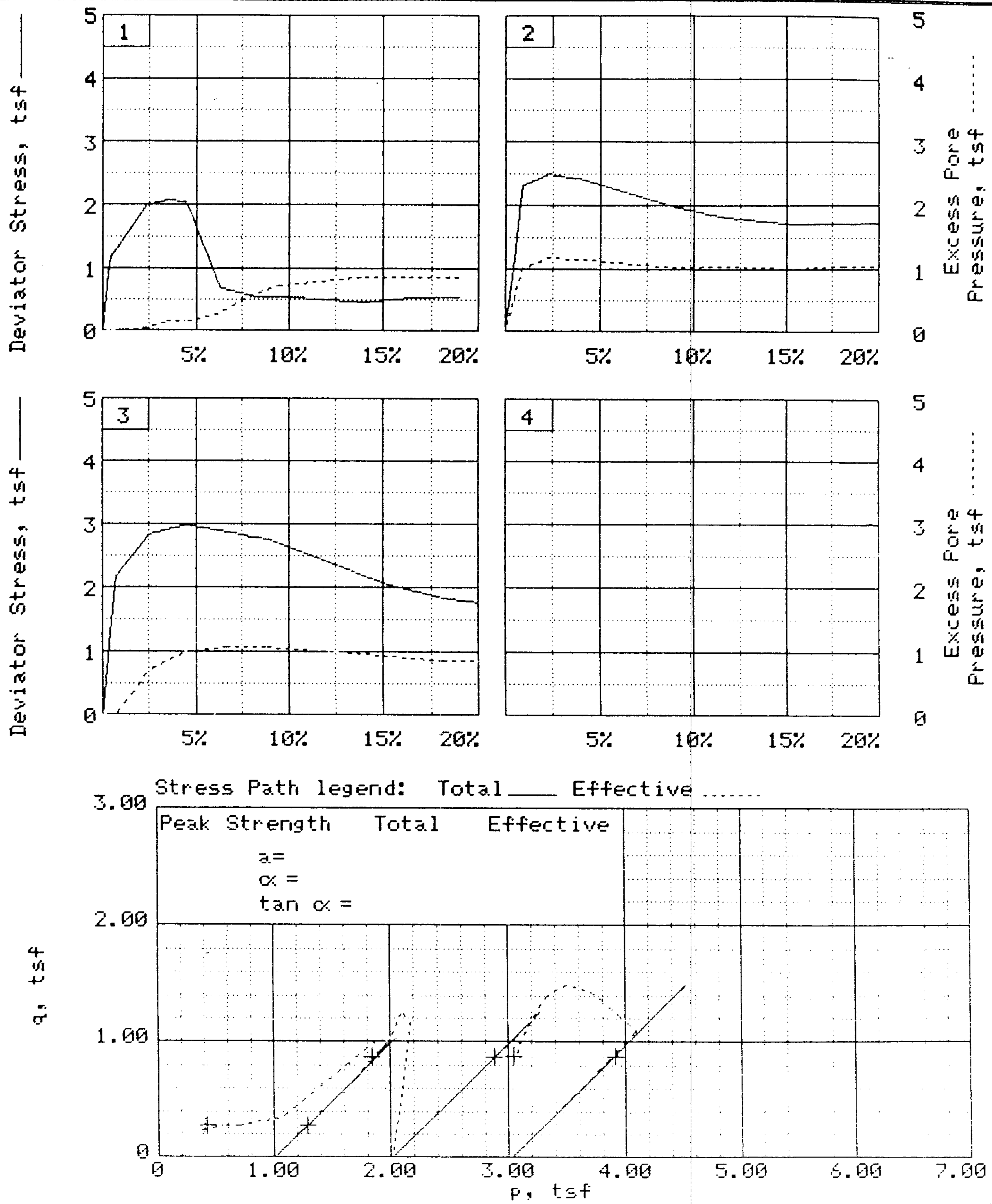
LL= 47	PL= 24	PI= 23	Gs= 2.70	CONTROLLED STRAIN TEST
TYPE OF SPECIMEN UNDISTURBED				TYPE OF TEST R w/pore pressures

REMARKS

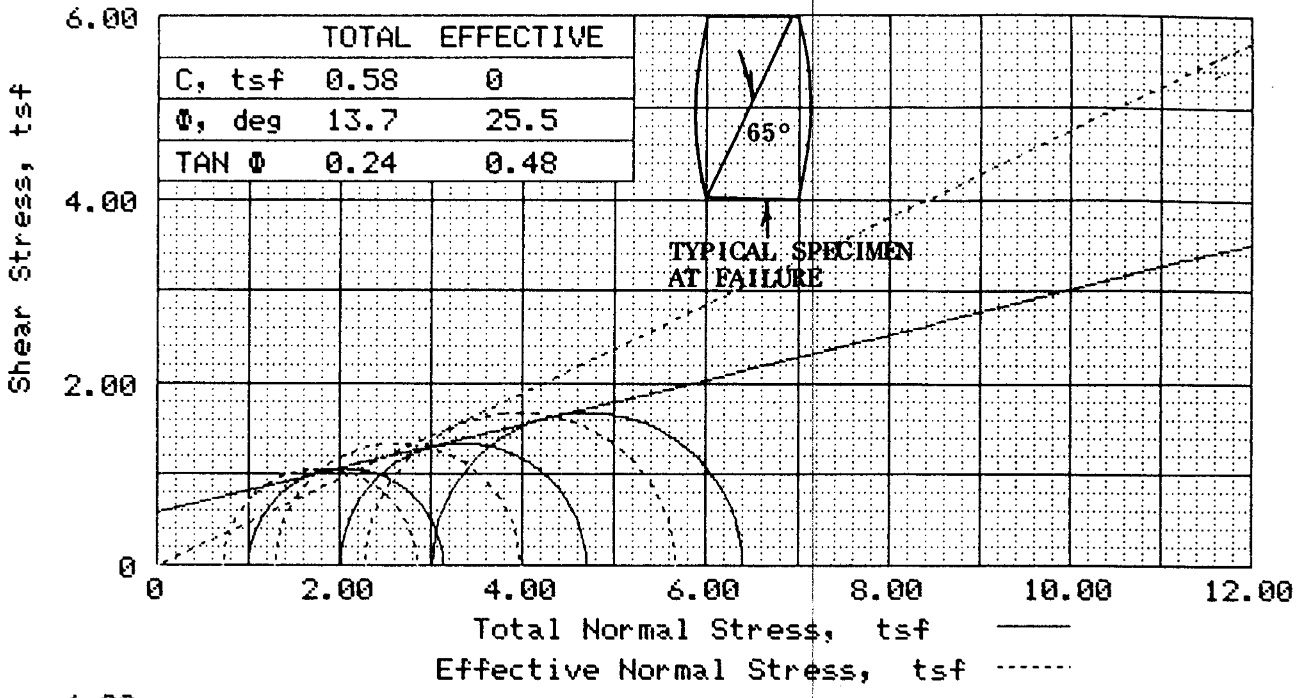
PROJECT PASSAIC RIVER TUNNEL
AREA
LAB NO. 264/690
BORING NO. C-102U
SAMPLE NO. 2
DEPTH/~~ELEV~~ 32.0 - 34.5'
LABORATORY CESAD-EN-FL DATE 24 OCT, 1994

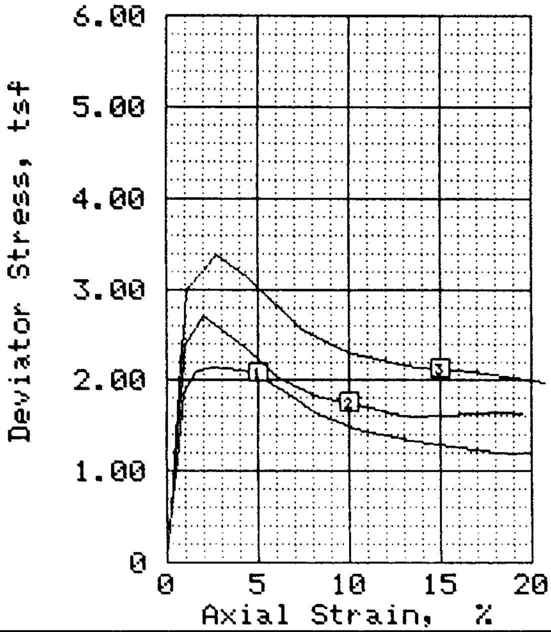
TRIAXIAL COMPRESSION TEST

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Requisition No. NYD-94-59 (C) Work Order No. 7319



PROJECT PASSAIC RIVER TUNNEL		
BORING C-102U	SAMPLE 2	DEPTH/~~ELEV~~ 32.0 - 34.5'
TYPE OF TEST R w/pore pressures	LAB NO. 264/690	
LABORATORY CESAD-EN-FL	PAGE 2/2	DATE 24 OCT, 1994





SAMPLE NO.		1	2	3
INITIAL	WATER CONTENT, %	37.2	36.4	37.1
	DRY DENSITY, pcf	83.4	85.0	84.2
	SATURATION, %	98.4	99.9	99.9
	VOID RATIO	1.021	0.983	1.002
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.08	3.08	3.08
@ TESTING	WATER CONTENT, %	35.6	31.5	31.0
	DRY DENSITY, pcf	86.0	91.1	91.8
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.961	0.850	0.836
	DIAMETER, in	1.36	1.35	1.34
	HEIGHT, in	3.05	3.01	2.99
BACK PRESSURE, tsf		5.04	5.04	5.04
INIT. EFF. STR., tsf		1.01	2.02	3.02
MAX. DEV. STRESS, tsf		2.13	2.69	3.39
PORE PRESSURE, tsf		0.28	0.72	0.73
TIME TO FAILURE, min.		25	10	15
RATE, %/min.		0.10	0.21	0.19
ULT. DEV. STRESS, tsf		1.29	1.57	2.11

DESCRIPTION (VISUAL) GRAY, LEAN CLAY (CL).

LL= 45	PL= 23	PI= 22	Gs= 2.70	CONTROLLED STRAIN TEST

TYPE OF SPECIMEN UNDISTURBED — TYPE OF TEST R w/pore pressures

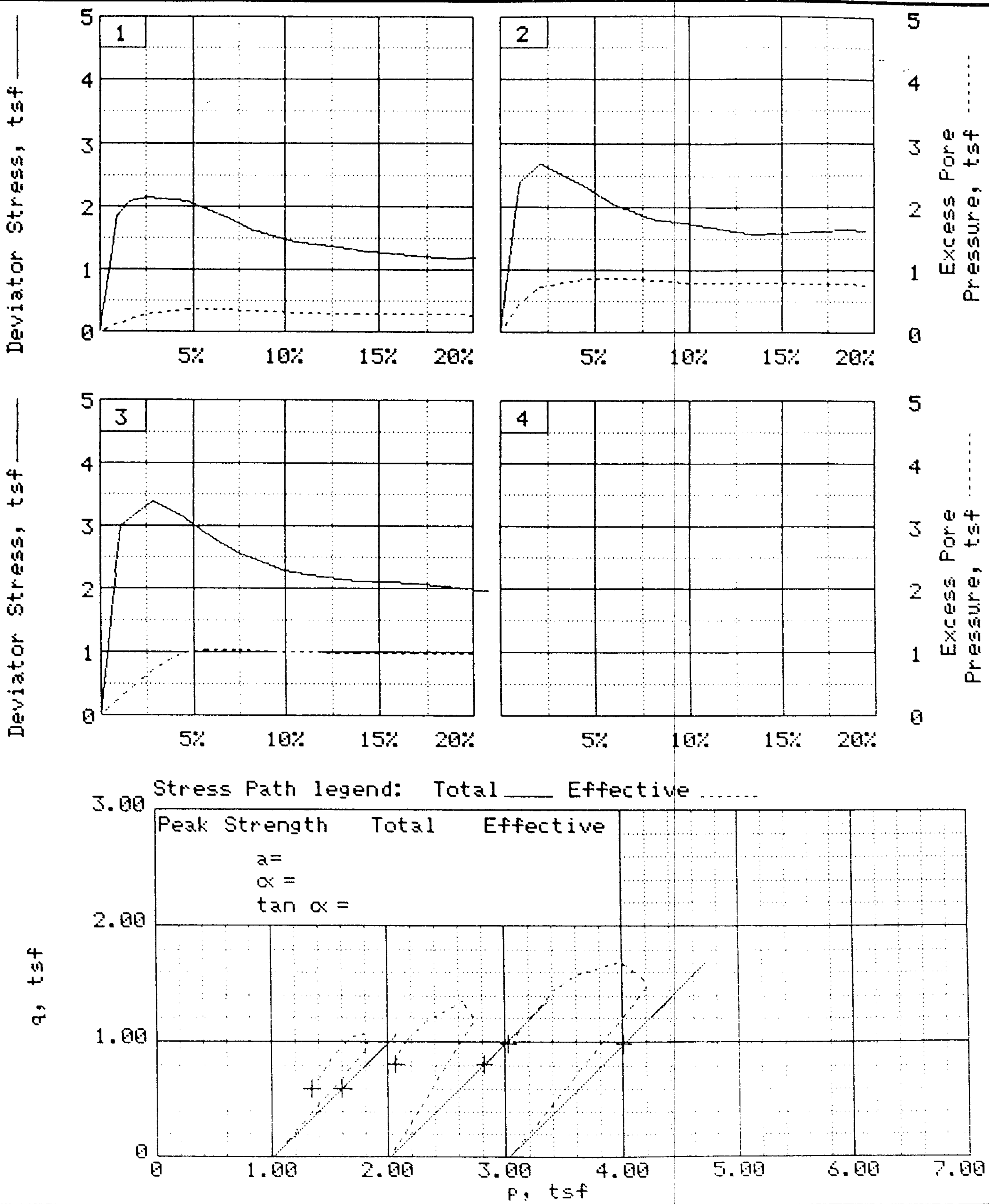
REMARKS

PROJECT PASSAIC RIVER
TUNNEL
AREA
LAB NO. 264/691
BORING NO. C-102U
SAMPLE NO. 3
DEPTH/~~ELEV~~ 43.5 - 45.5'
LABORATORY CESAD-EN-FL DATE 24 OCT, 1994

TRIAXIAL COMPRESSION TEST

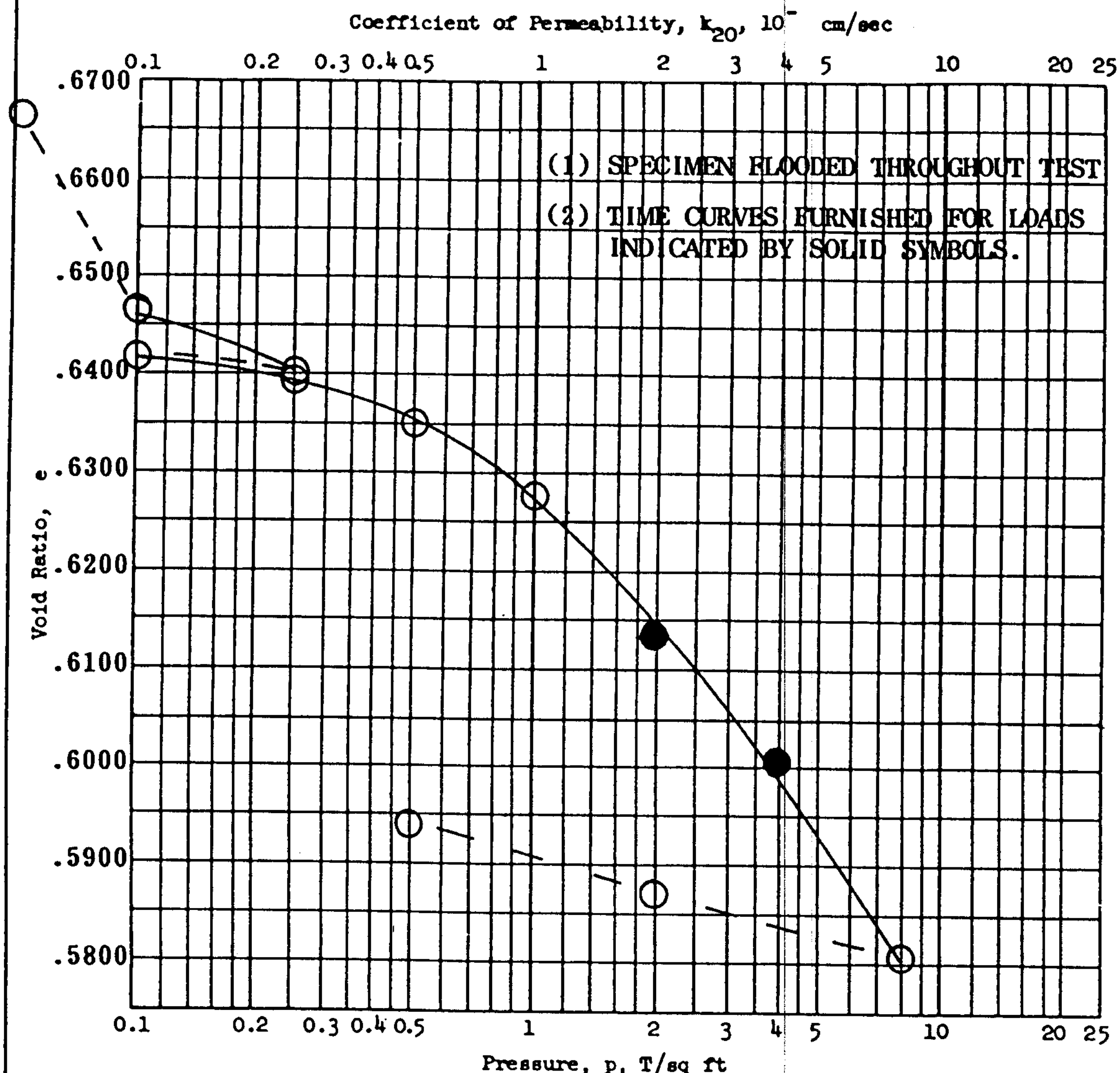
SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Requisition No. NYD-94-59 (C) Work Order No. 7319

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Requisition No. NYD-94-59 (C) Work Order No. 7319



PROJECT PASSAIC RIVER TUNNEL
BORING C-102U SAMPLE 3 DEPTH/~~ELEV~~ 43.5 - 45.5'
TYPE OF TEST R w/pore pressures LAB NO. 264/691
LABORATORY CESAD-EN-FL PAGE 2/2 DATE 24 OCT, 1994

WORK ORDER NO. 7319
Req. No. NYD-94-59(C)



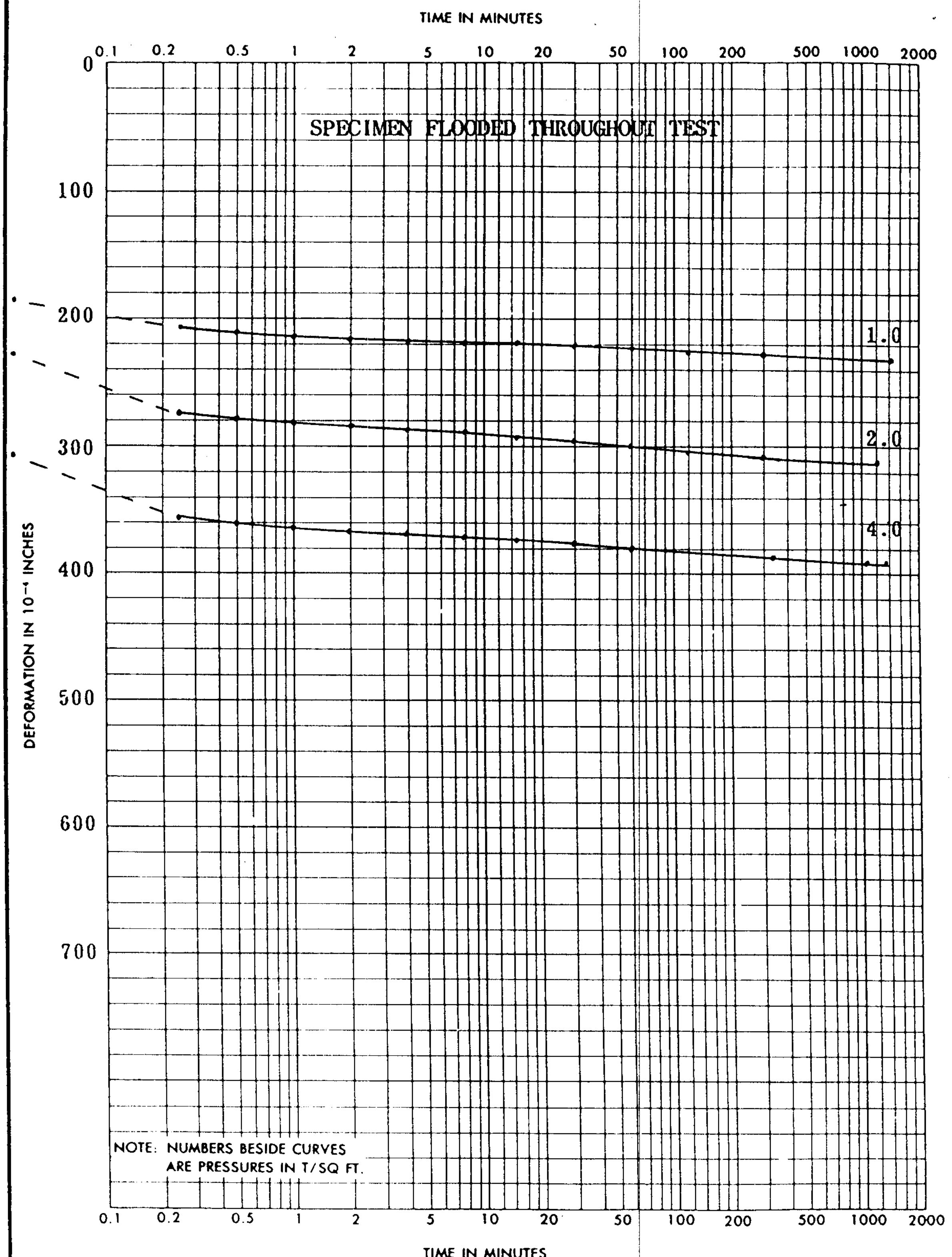
Type of Specimen Undisturbed		Before Test		After Test	
Diam 2.48 in.	Ht 1.00 in.	Water Content, w_o	24.7 %	w_f	22.0 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.666	e_f	0.594
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	100.0 %	S_f	100.0 %
Compression Index, C_c		Dry Density, γ_d	101.1 lb/ft^3		
Classification (Visual) Brownish Gray Lean Clay (CL)		k_{20} at e_o = × 10^{-} cm/sec			
LL 24	G_s Assumed 2.70	Project Passaic River			
PL 16	D_{10} –	Lab No. 264-698			
Remarks		Area			
		Boring No. SHAFT 3 (DC-22)		Sample No. ST-1	
		Depth ~~Ek~~ 60.0-62.0'		Date 12 Dec 94	
		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE.

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30061

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER NO. 7319
Req. No. NYD-94-59(C)



PROJECT Passaic River

AREA SHAFT 3 Lab No. 264/698

BORING NO. (DC-22) SAMPLE NO. ST-1 DEPTH XK 60.0-62.0' DATE 12 Dec 94

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

* GPO : 1964 OF—715-965

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 4
SHAFTS

ATTACHMENT E.4.2
SOIL STRENGTHS AND DESIGN DATA

E.4.2

SOIL STRENGTHS AND DESIGN DATA

Passaic River Workshaft / Ventshaft Liner
Spur Inlet Vent Shaft (Sta. 68+40)

19-Jul-95

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	178.0								0	
Sand	168.0	0.33	3.00	0.50	0	30	11.0	125	686.4	344.3
Clay	157.0	0.44	2.30	0.61	750	23	11.0	120	1372.8	730.8
Glacial Till	146.0	0.22	4.59	0.36	0	40	42.0	125	3993.6	1677.3
Rock	104.0						Top of rock			
T. Rock to C.C.	1.0						103.0		10420.8	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	3993.6	1677.3	20	126	12	120	132	39.4	413.5		3000	1785
ROCK LINER	10420.8	0.0	12	78	12	72	84	72.4	505.2	545.5	3000	1785

Terms:
h= head
~~P1= water pressure h(62.4)~~
P2= soil pressure (Sat. Soil – 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

18-Jul-95

Passaic River Workshaft / Ventshaft Liner
Pompton Inlet Vent Shaft (Sta. 1075+46)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	196.0								0	
Gravel	186.0	0.33	3.00	0.50	0	30	16.0	125	998.4	500.8
Sand	170.0	0.33	3.00	0.50	0	30	10.0	120	1622.4	788.8
Clay	160.0	0.61	1.60	0.76	500	14	40.0	120	4118.4	2539.8
Rock	120.0						Top of rock			
T. Rock to C.C.	31.0						89.0		9672.0	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	4118.4	2539.8	20	126	12	120	132	46.2	485.5		3000	1785
ROCK LINER	9672.0	0.0	15	96	12	90	102	67.2	569.9	606.6	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil – 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Mar 1, 95

PASSAIC RIVER WORKSHAFT/VENT SHAFT LINER
WORKSHAFT 2 (STA 623+76, I.D.=42')

SOIL TYPE	GROUND EL. ft	K_a	K_P	K_O	C	Φ	SOIL LAYER/h ft	SAT. SOIL pcf	CUMM. P_1 psf	CUMM. P_2 psf
Safety Barrier	10									
Ground Elev.	390.00						387.8		24198.72	0.00
T.Rock to C.C.	2.20									

WKSHAFT NAME/No.	10 P_1 psf	P_2 psf	I.D. ft	R in	t in	$P_1 + P_2$ psi	fc psi	CONC. USE psi	WORKING STRESS psi
SOIL LINER	0.00	0.00	45	282.00	24	0.00	0.00	4500	2677.50
ROCK LINER	24198.72	0.00	42	261.00	18	168.05	-2436.68	4500	2677.50

Passaic River Workshaft / Ventshaft Liner 18-Jul-95
Workshaft 2A (Sta. 364+89)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	155.0								0	
Clay	145.0	0.61	1.60	0.76	500	14	4.0	105	249.6	129.5
Silt material	141.0	0.33	3.00	0.50	1000	30	11.0	115	936.0	418.8
Gravel	130.0	0.22	4.60	0.43	0	35	5.0	130	1248.0	564.1
Silty Sand	125.0	0.27	3.70	0.43	0	35	23.0	130	2683.2	1232.7
Rock	102.0						Top of rock			
T. Rock to C.C.	-177.0						279.0		20092.8	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	2683.2	1232.7	20	126	12	120	132	27.2	285.5		3000	1785
ROCK LINER	20092.8	0.0	15	96	12	90	102	139.5	1183.9	1260.2	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil - 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Passaic River Workshaft/Ventshaft Liner Design
Workshaft 2B (Sta. 163+15)

18-Jul-95

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Abv Grnd	55.0						48.8		0	
GC Fill	6.2	0.36	2.8	0.53	0	28	5.0	115	312.0	139.4
Orgnc Silt	1.2	0.70	1.4	0.83	250	10	3.0	105	499.2	245.5
Sandy Silt	−1.8	0.70	1.4	0.83	500	10	3.0	115	686.4	376.4
Silty Sand	−4.8	0.27	3.7	0.43	0	35	20.0	125	1934.4	914.8
Sandy Silt	−24.8	0.70	1.4	0.83	2000	10	18.0	130	3057.6	1924.7
Varved Clay	−42.8	0.70	1.4	0.36	0	40	50.0	125	6177.6	3051.5
Silty Sand	−92.8	0.22	4.6	0.36	0	40	40.0	130	8673.6	4025.0
Glacl Til	−132.8	0.22	4.6	0.36	0	40	15.0	130	9609.6	4390.0
Rock	−147.8						Top of rock			
T.Rck to C.C.	−380.0						232.2		24098.9	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	t in	r in	P1+P2 psi	fc psi	Conc. Use psi	Working Stress psi
SOIL LINER	9609.6	4390.0	45	18	279	97.2	1506.9	3000	1785
ROCK LINER	24098.9	0.0	42	18	261	167.4	2426.6	4500	2678

Passaic River Workshaft / Ventshaft Liner 07-Aug-95 wksft2c.wk1

Workshaft 2C (Sta. 20+80)

Pump Station

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Fill Material	8.0	0.36	2.77	0.53	0	28	11.0	115	686.4	306.7
Org. Silt/Clay	-3.0	1.00	1.00	1.00	300	0	12.0	105	1435.2	817.9
Alluvial Sand	-15.0	0.31	3.25	0.47	0	32	13.0	115	2246.4	1139.2
Var Silts/Clays	-28.0	1.00	1.00	1.00	2000	0	62.0	130	6115.2	5330.4
Glacial Till	-90.0	0.22	4.60	0.36	0	40	17.0	130	7176.0	5744.2
Rock	-107.0						Top of rock			
T. Rock to C.C.	-388.0						281.0		24710.4	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	7176.0	5744.2	42	264	24	252	276	89.7	987.0		3000	1785
ROCK LINER	24710.4	0.0	42	264	24	252	276	171.6	1887.6		4000	2380

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil - 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Mar 1, 95

PASSAIC RIVER WORKSHAFT/VENT SHAFT LINER
WORKSHAFT 3 (STA 843+47, I.D.=42')

SOIL TYPE	GROUND EL. ft	K_a	K_P	K_O	C	Φ	SOIL LAYER/h ft	SAT. SOIL pcf	CUMM. P_1 psf	CUMM. P_2 psf
Safety Barrier	10									
Clay Material	180.00	0.61	1.60	0.76	500	14	2.00	120	124.8	87.33
Sand Material	178.00	0.33	3.00	0.50	0	30	10.00	125	748.8	400.33
Clay Material	168.00	0.61	1.60	0.76	2000	14	24.00	120	2246.4	1448.30
Clay Material	144.00	0.66	1.50	0.79	1240	12	16.00	120	3244.8	2178.29
Glacial Till	128.00	0.22	4.60	0.36	0	40	35.00	130	5428.8	3023.45
Rock	93.00						Top of rock			
T.Rock to C.C.	13.6						79.40		10383.36	0.00

WKSHAFT NAME/No.	P_1 psf	P_2 psf	I.D. ft	R in	t in	$P_1 + P_2$ psi	PRESS ON LINER psi	CONC. USE psi	WORKING STRESS psi
SOIL LINER	5428.80	3023.45	45	282.00	24	58.70	-689.68	3000	1785.00
ROCK LINER	10383.36	0.00	42	261.00	18	72.11	-1045.55	3000	1785.00

Passaic River Workshaft / Ventshaft Liner 19-Jul-95
Workshaft 4 (Sta. 3+27)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	182.0								0	
Sandy Clay	172.0	0.61	1.60	0.76	500	14	8.0	115	499.2	319.8
Glacial Till	162.0	0.22	4.60	0.36	0	40	5.0	130	811.2	441.5
Rock	157.0						Top of rock			
T. Rock to C.C.	14.8						142.3		9687.6	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	811.2	441.5	25	156	12	150	162	8.7	113.1		3000	1785
ROCK LINER	9687.6	0.0	23	144	12	138	150	67.3	807.3		3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil – 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Passaic River Workshaft / Ventshaft Liner 18-Jul-95

Workshaft 5 (Sta. 483+98)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	175.0								0	
Gravel	165.0	0.3	3.40	0.46	0	33	27.0	130	1684.8	839.6
Rock	138.0						Top of Rock			
T. Rock to C.C.	-4.6						142.6		10583.0	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	1684.8	839.6	20	126	12	120	132	17.5	184.1		3000	1785
ROCK LINER	10583.0	0.0	15	96	12	90	102	73.5	623.5	663.7	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil – 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

Passaic River Workshaft / Ventshaft Liner 18-Jul-95

Workshaft 6 (Sta. 783+00)

Soil Type	Ground El. ft	Ka	Kp	Ko	C	phi	Soil Layer/h ft	Sat. Soil pcf	Cumm. P1 psf	Cumm. P2 psf
Safety Barrier	180.0								0	
Gravel	170.0	0.29	3.40	0.46	0	33	15.0	130	936.0	466.4
Rock	155.0						Top of Rock			
T. Rock to C.C.	10.3						144.7		9965.3	0.0

WKSHFT NAME/No.	P1 psf	P2 psf	I.D. ft	r in	t in	R1 in	R2 in	P1+P2 psi	fc psi	fc(max) psi	Conc. Use psi	Working Stress psi
SOIL LINER	936.0	466.4	20	126	12	120	132	9.7	102.3		3000	1785
ROCK LINER	9965.3	0.0	15	96	12	90	102	69.2	587.2	625.0	3000	1785

Terms:

h= head
P1= water pressure h(62.4)
P2= soil pressure (Sat. Soil - 62.4)(Ko)(h)
r= radius to point of lining analyzed
R1= inside radius of liner
R2= outside radius of liner
t= liner thickness
I.D.= inside diameter of liner
Concrete Working Stress= (0.7)(0.85)f'c

If t <or= 1/10(R1) then treat as thin shell where:

fc= P(r)/t Eqn. 3-7 EM 1110-2-2901

If t > 1/10(R1) then treat as thick shell where:

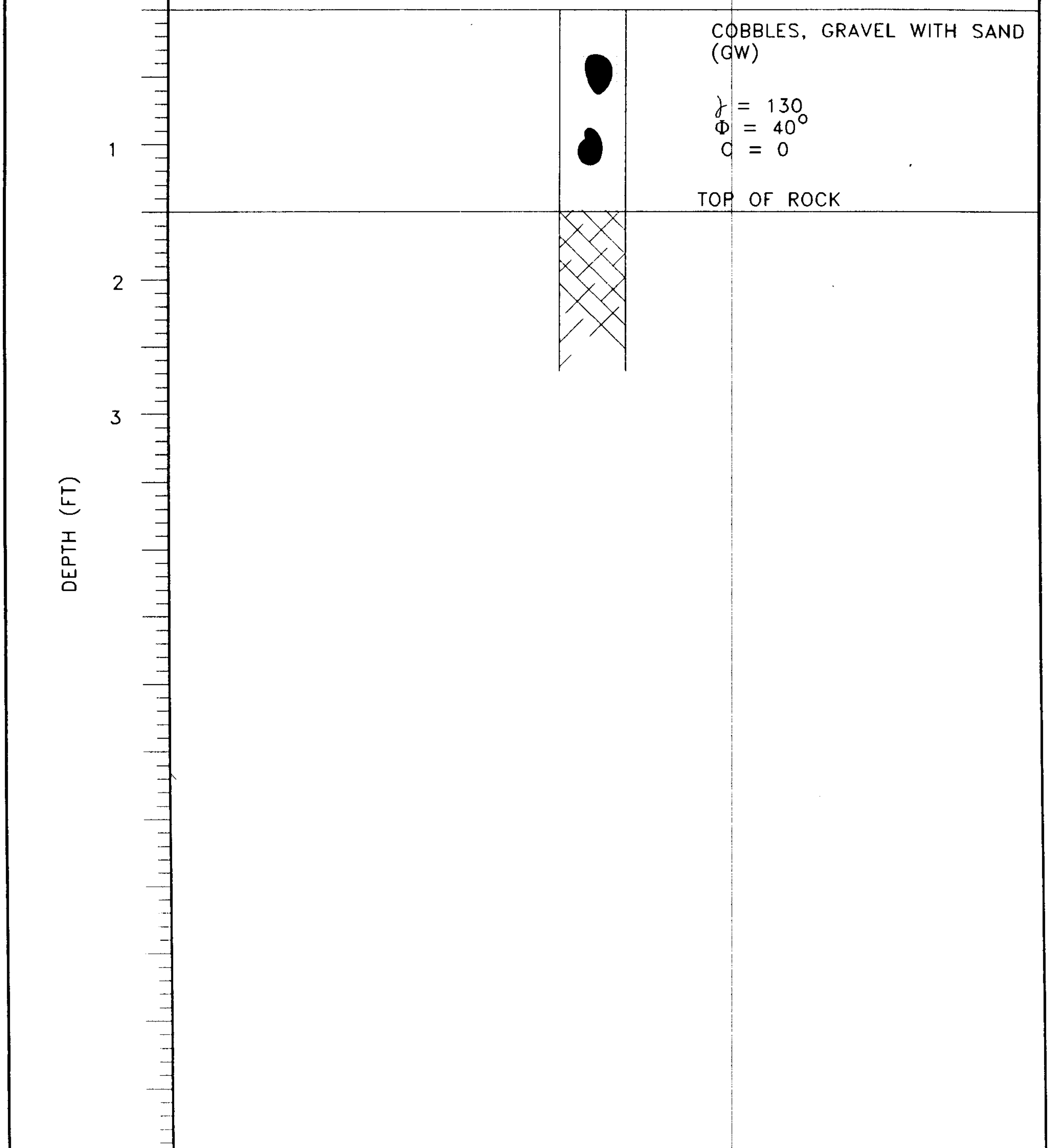
fc= Eqn. 3-8 in EM 1110-2-2901

fc(max)= Eqn. 3-9 in EM 1110-2-2901

WORKSHAFT NO. 2
SOILS DESIGN DATA

DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT — SHEET 1 OF 1 SHEETS
PROJECT PASSAIC TUNNEL	SUBJECT WORKSHAFT 2 BORING DC-85

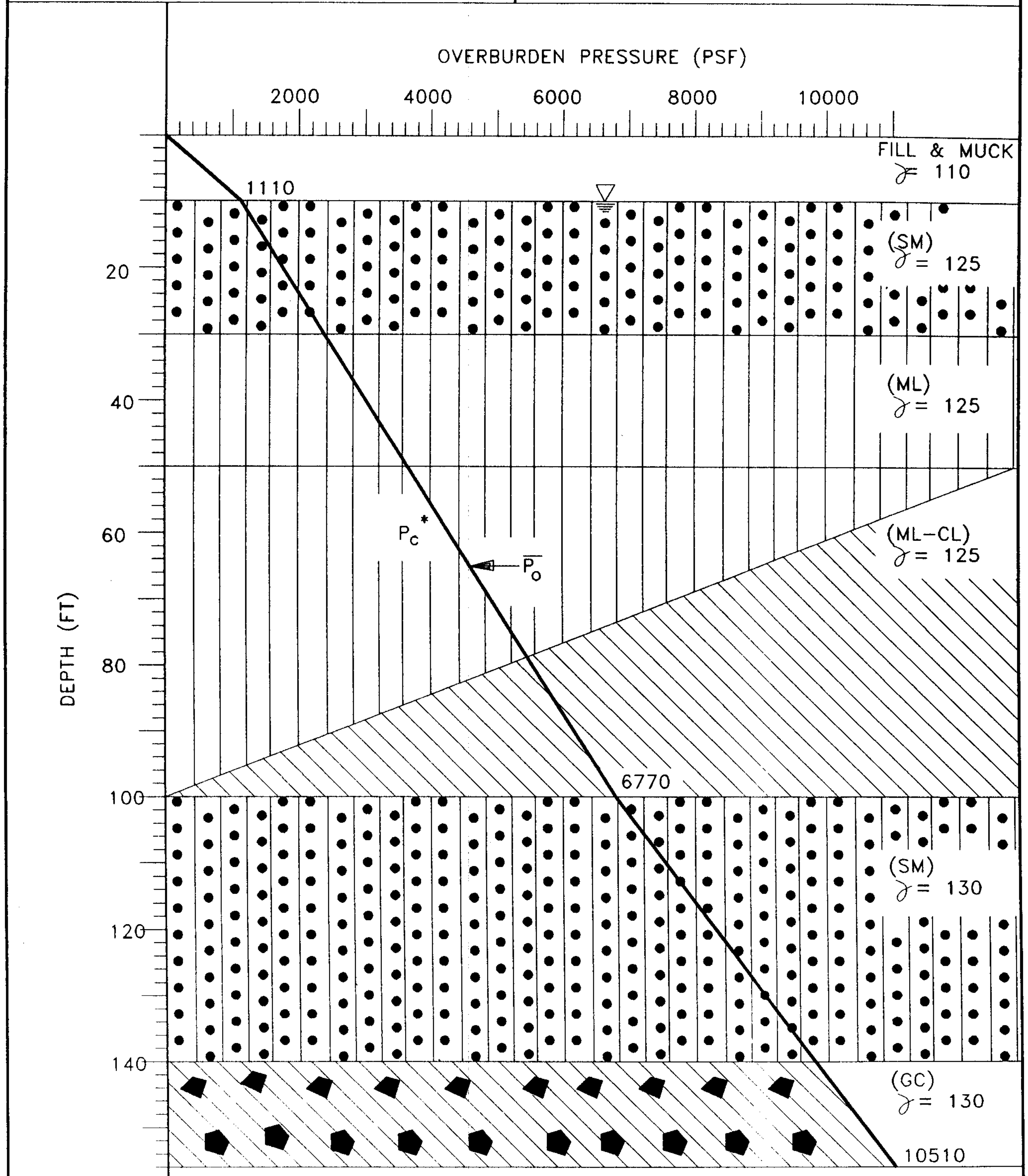
BORING LOG DC-85



WORKSHAFT 2A
SOILS DESIGN DATA

WORKSHAFT 2B
SOILS DESIGN DATA

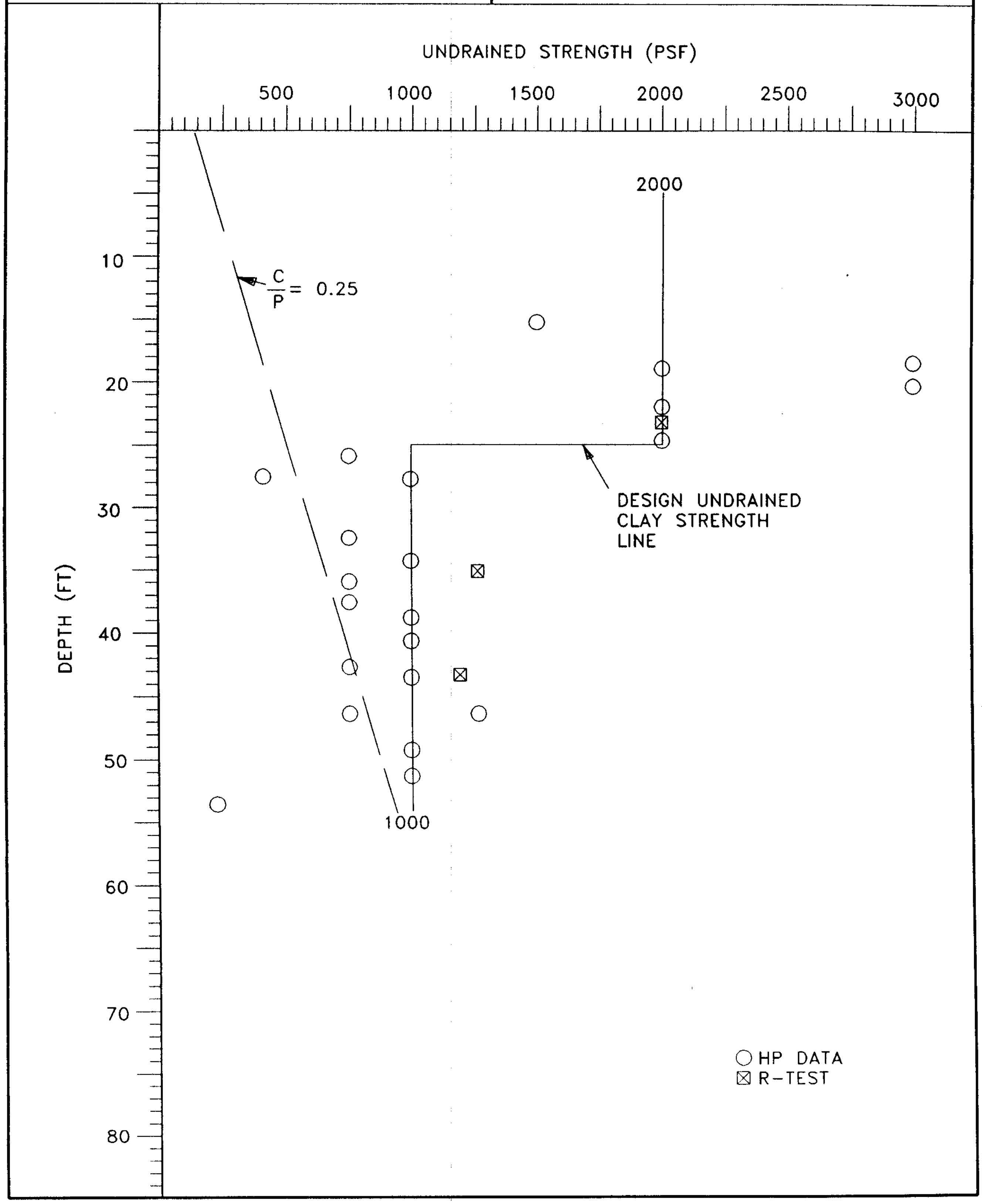
DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 1 SHEETS
PROJECT PASSAIC TUNNEL	SUBJECT WORK SHAFT 2B (KEEGAN)	



* ASSUME VARVED CLAYS
NORMALLY CONSOLIDATED

WORKSHAFT 3
SOILS DESIGN DATA

DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 1 SHEETS
PROJECT PASSAIC TUNNEL	SUBJECT WORKSHAFT 3 STRENGTHS	



WORKSHAFT NO. 4
SOILS DESIGN DATA

HOOK HOLE NO. 5
SOILS DESIGN DATA

WORKSHAFT NO. 6
SOILS DESIGN DATA

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 4
SHAFTS

ATTACHMENT E.4.3
SLURRY WALL DOCUMENTATION

E.4.3

SLURRY WALL DOCUMENTATION

CEORN-EP-G

MEMORANDUM FOR RECORD

SUBJECT: Workshafts incorporating structural slurry walls on the Passaic Flood Control Project

1. Reference: Nino Catalano, ICOS Boston.

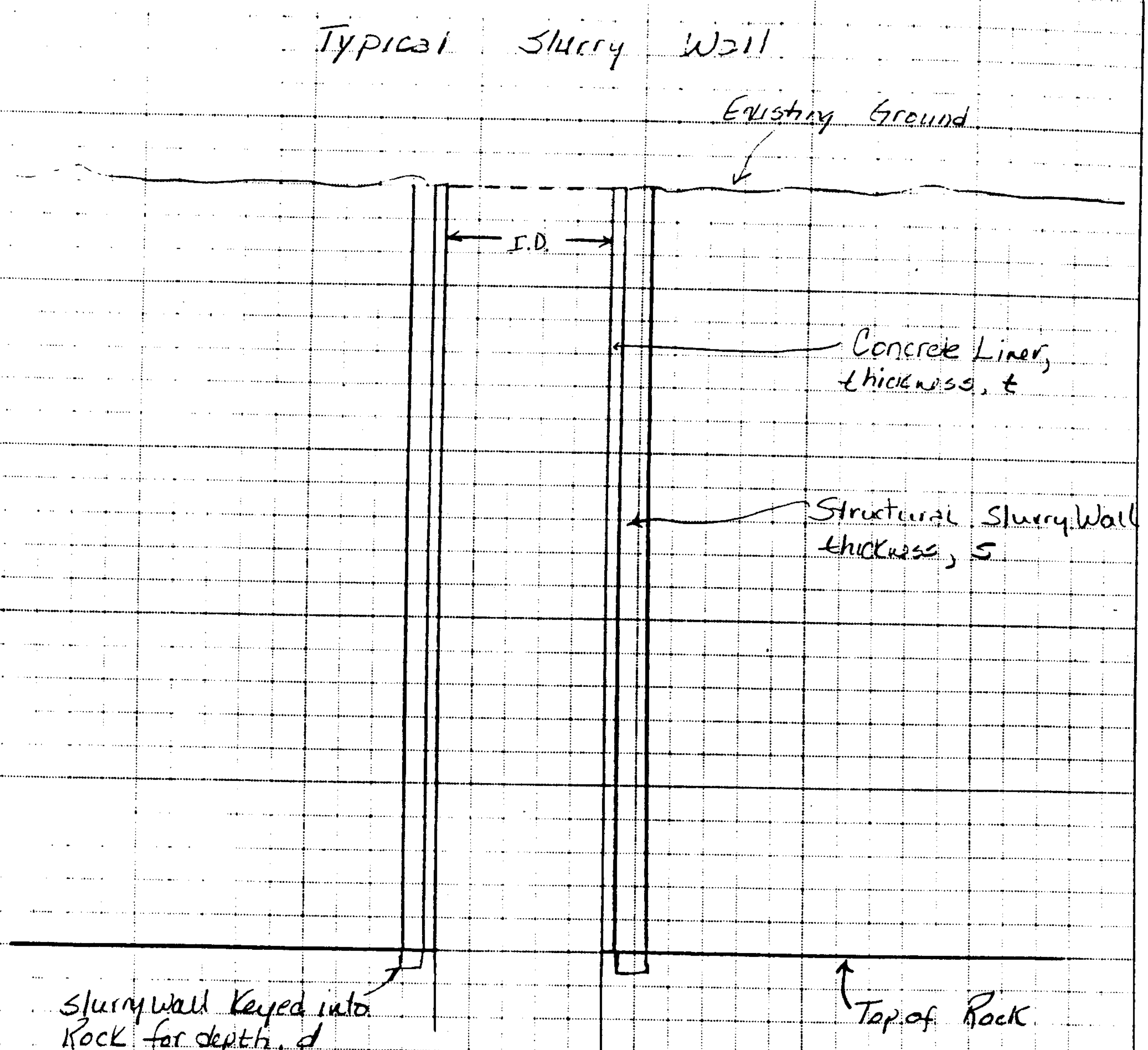
2. Structural slurry walls will be 24 inches thick for 20 feet diameter workshafts up to 50 feet in depth.

3. Structural slurry walls will be 30 inches thick for diameters in excess of 20 feet and up to 50 feet.

4. The length of the individual panels for the slurry wall shall be within a range of 10-12 feet.

5. Use of structural slurry walls is not recommended for depths in excess of 100 feet due to problems achieving the necessary verticality.

CORPS OF ENGINEERS, U.S.ARMY OHIO RIVER DIVISION	COMPUTATION SHEET	PAGE OF PAGES	DATE 3-24-95
CEORN-EP-H NASHVILLE DISTRICT	SUBJECT Passaic Flood Control Project		
COMPUTED BY: MM	COMPUTATION Drawing for Typical Slurry Worksheet	NUMBER	
CHECKED BY:			



ORD FORM 427

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 4
SHAFTS

ATTACHMENT E.4.4
WORK SHAFT WEDGE FAILURE ANALYSIS

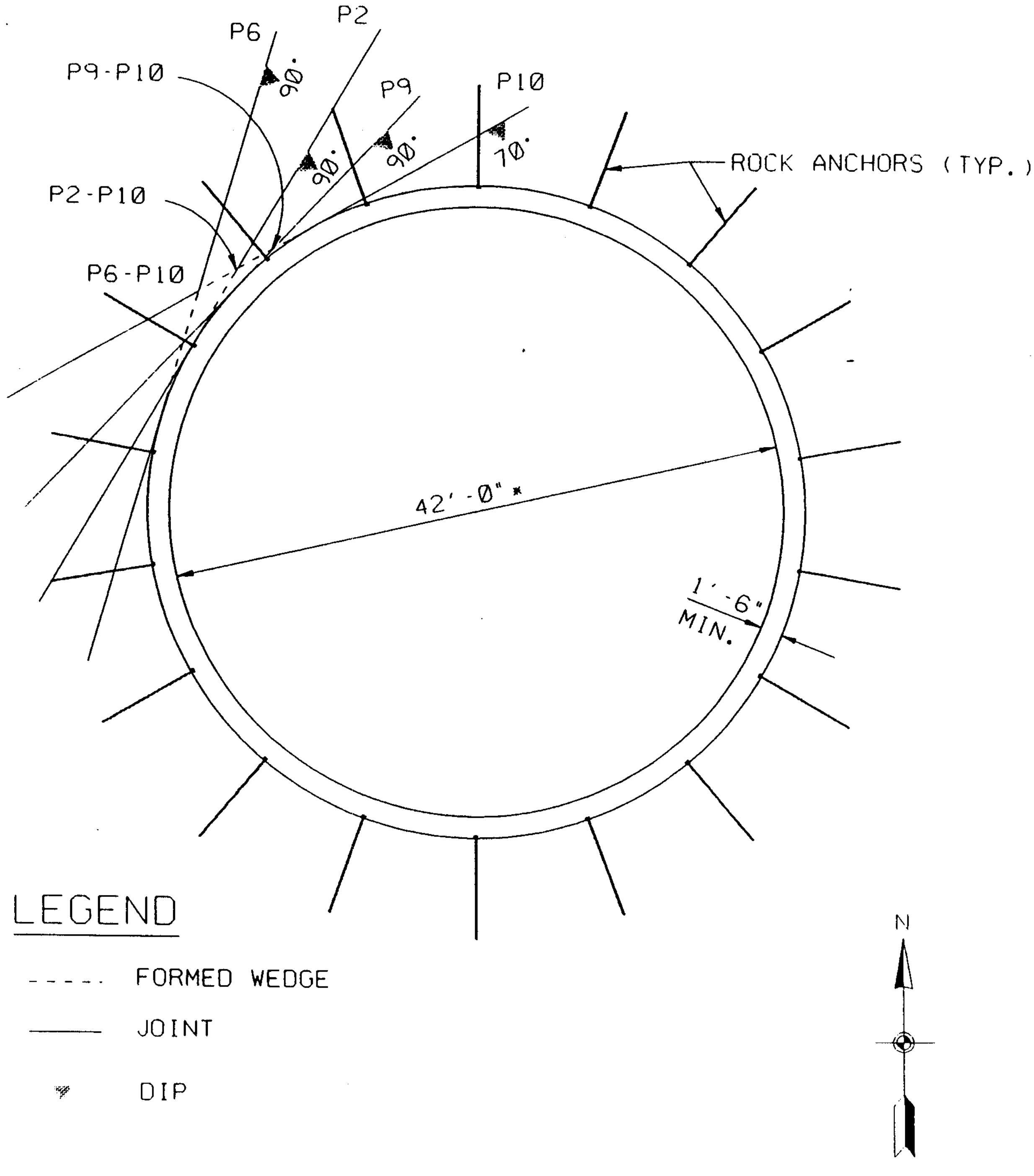
E.4.4

WORKSHAFT WEDGE FAILURE ANALYSIS

TITLE

SHAFTS 2C, 2B, 2A, 5, AND 2

POTENTIALLY UNSTABLE WEDGES



PLAN VIEW

SCALE: 1" = 10'

E.4.4-1

PASSAIC TUNNEL PROJECT
POTENTIALLY UNSTABLE WEDGES
SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

Note 3: The upper slope surface is assumed to be formed by a bedding plane in the rock. This is a conservative assumption since no bedding plane may exist at the optimum location.

Note 4: The wet slope analysis assumes the joints to be free draining where they intercept the shaft.

Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

Note 6: The height of the wedge is approximated as shown on the attached sheet. This height is the maximum height possible for the joint combination shown and for the largest work shaft. This is a very conservative approach.

JOINT NUMBER	DIP	DIP DIRECTION
p2	90	120
p6	90	106
p9	90	133
p10	70	150

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS

JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY	
					WET SLOPE	DRY SLOPE
p2-p10	40	210	135	5.06	0 f	0.3 x
p6-p10	53	196	128	12.06	0 f	0.3 x
p9-p10	62	223	141.5	6.32	0 f	0.3 x

Joints p1, p3, p4, p5, p7, and p8 do not form wedges that are free to move.

f signifies the wedge is floating.
x signifies contact on only one plane

PASSAIC FORMATION **C = 0, O = 39.5, Unit Weight + 158 PCF**

RELEVANT FOR THE OUTLET, WORKSHAFTS 2C, 2B, 2A, 5, and 2.

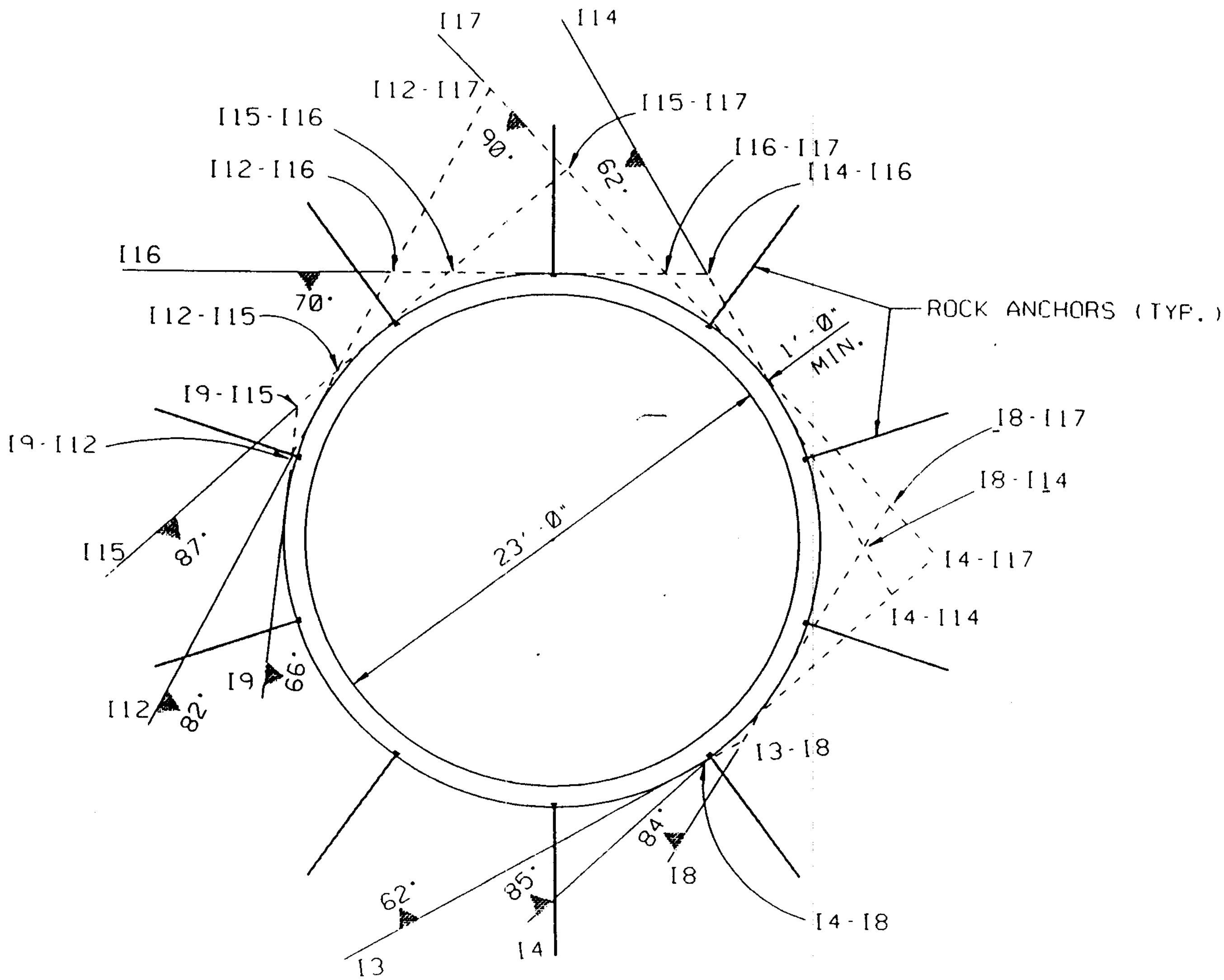
SHAFTS 4 AND 6

POTENTIALLY UNSTABLE WEDGES

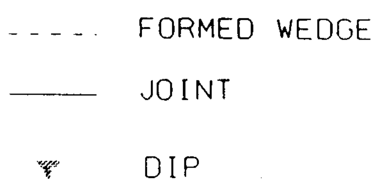
JOINT COMBINATION	SHEET NO.
I1-I3	1
I1-I4	1
I1-I8	1
I1-I9	1
I1-I12	1
I1-I14	1
I1-I15	1
I3-I8	2
I4-I8	2
I4-I14	2
I4-I17	2
I8-I14	2
I8-I17	2
I9-I12	2
I9-I15	2
I12-I15	2
I12-I16	2
I12-I17	2
I14-I16	2
I15-I16	2
I15-I17	2
I16-I17	2

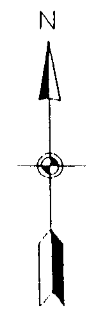
SHAFTS 4 AND 6

POTENTIALLY UNSTABLE WEDGES



LEGEND





PLAN VIEW

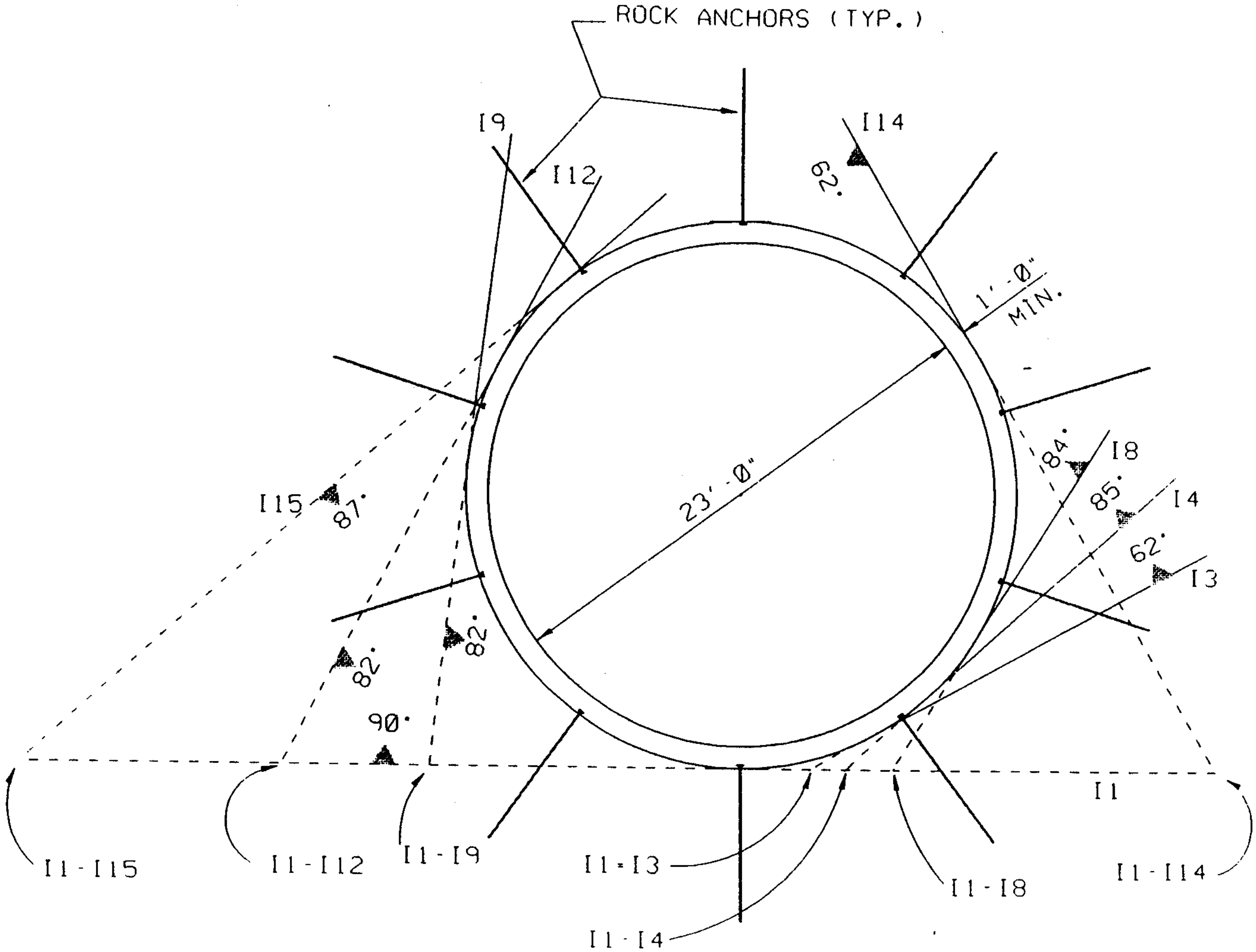
SCALE: 1"=7'-6"

E.4.4-4

SHEET

SHAFTS 4 AND 6

POTENTIALLY UNSTABLE WEDGES



LEGEND

- - - - - FORMED WEDGE

——— JOINT

▼ DIP

PLAN VIEW

SCALE: 1"=7'-6"

E.4.4-5

SHEET 1

PASSAIC TUNNEL PROJECT

POTENTIALLY UNSTABLE WEDGES

SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

Note 3: The upper slope surface is assumed to be formed by a bedding plane in the rock. This is a conservative assumption since no bedding plane may exist at the optimum location.

Note 4: The wet slope analysis assumes the joints to be free draining where they intercept the shaft.

Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

Note 6: The height of the wedge is approximated as shown on the attached sheet. This height is the maximum height possible for the joint combination shown and for the largest work shaft. This is a very conservative approach.

JOINT NUMBER	DIP	DIP DIRECTION
i1	90	180
i2	55	6
i3	61	331
i4	85	318
i7	48	318
i8	84	302
i9	66	97
i12	82	118
i13	50	190
i14	62	240
i15	87	138
i16	70	180
i17	90	226

Joints i5, i6, i10, and i11 do not form wedges that are free to move.

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS

JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY	
					WET SLOPE	DRY SLOPE
i1-i3	37	270	345.5	2.63	0 f	0.8218 x
i1-i4	85	270	339	59.23	0 f	0.1297 x
i1-i7	34	270	339	3.50	0.515 x	1.335 x
i1-i8	83	270	331	60.95	0 f	0.1558 x
i1-i9	65	90	138.5	25.61	0 f	0.66 x
i1-i12	80	90	149	46.00	0 f	0.2084 x
i1-i14	58	270	210	12.47	0 f	0.7883 x
i1-i15	85	90	159	59.23	0 f	0.078 x
i2-i3	56	10	348.5	2.04	0.4694 x	1.038 x
i2-i4	50	44	342	6.21	0.269 x	1.038 x
i2-i7	48	322	342	2.89	0.828	1.391 x
i2-i8	55	24	334	9.93	0.359	1.038
i2-i9	51	41	51.5	12.31	0.86	1.55 x
i2-i12	52	40	62	22.91	0.861	1.765
i2-i14	37	305	303	17.80	2.24	3.092

f signifies the wedge is floating.

x signifies contact on only one plane

PREAKNESS MOUNTAIN BASALT **C = 0, O = 56.1, Unit Weight + 176 PCF**

RELEVANT FOR WORKSHAFTS 4 and 6.

PASSAIC TUNNEL PROJECT

POTENTIALLY UNSTABLE WEDGES

SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

Note 3: The upper slope surface is assumed to be formed by a bedding plane in the rock. This is a conservative assumption since no bedding plane may exist at the optimum location.

Note 4: The wet slope analysis assumes the joints to be free draining where they intercept the shaft.

Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

Note 6: The height of the wedge is approximated as shown on the attached sheet. This height is the maximum height possible for the joint combination shown and for the largest work shaft. This is a very conservative approach.

JOINT NUMBER	DIP	DIP DIRECTION
i1	90	180
i2	55	6
i3	61	331
i4	85	318
i7	48	318
i8	84	302
i9	66	97
i12	82	118
i13	50	190
i14	62	240
i15	87	138
i16	70	180
i17	90	226

Joints i5, i6, i10, and i11 do not form wedges that are free to move.

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS						
JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY WET SLOPE	FACTORS OF SAFETY DRY SLOPE
i2-i15	46	53	72	30.34	1.62	2.717
i2-i17	44	316	296	35.82	2.151	3.389
i3-i8	48	26	316.5	2.77	0 f	0.8249 x
i3-i9	44	33	34	22.80	1.835	2.714
i3-i12	41	34	44.5	38.63	3.038	4.341
i3-i14	52	284	285.5	12.55	0.7803	1.48
i3-i17	61	316	278.5	31.74	0.3203 x	1.075
i4-i8	81	256	310	2.84	0 f	0.1564
i4-i9	52	44	27.5	45.15	1.359	2.679
i4-i12	57	43	38	116.55	1.947	4.735
i4-i13	40	230	254	22.85	1.81	2.805
i4-i14	61	235	279	17.25	0.129 x	0.7913 x
i4-i16	57	234	249	52.34	0.8373	2.28
i4-i17	86	316	272	199.92	0 f	0.1349
i7-i14	46	284	279	7.15	0.9898	1.625

f signifies the wedge is floating.

x signifies contact on only one plane

PREAKNESS MOUNTAIN BASALT

C = 0, O = 56.1, Unit Weight + 176 PCF

RELEVANT FOR WORKSHAFTS 4 and 6.

PASSAIC TUNNEL PROJECT

POTENTIALLY UNSTABLE WEDGES

SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

Note 3: The upper slope surface is assumed to be formed by a bedding plane in the rock. This is a conservative assumption since no bedding plane may exist at the optimum location.

Note 4: The wet slope analysis assumes the joints to be free draining where they intercept the shaft.

Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

Note 6: The height of the wedge is approximated as shown on the attached sheet. This height is the maximum height possible for the joint combination shown and for the largest work shaft. This is a very conservative approach.

JOINT NUMBER	DIP	DIP DIRECTION
i1	90	180
i2	55	6
i3	61	331
i4	85	318
i7	48	318
i8	84	302
i9	66	97
i12	82	118
i13	50	190
i14	62	240
i15	87	138
i16	70	180
i17	90	226

Joints i5, i6, i10, and i11 do not form wedges that are free to move.

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS						
JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY	
					WET SLOPE	DRY SLOPE
i7-i17	48	316	272	15.53	0.797	1.376
i8-i9	39	28	19.5	48.68	4.012	5.65
i8-i13	45	218	246	18.79	1.073	1.965
i8-i14	60	223	271	10.81	0.039 x	0.791
i8-i15	58	222	220	152.32	1.42	5.28
i8-i16	62	224	241	41.89	0.1907 x	1.378
i8-i17	84	316	264	100.35	0 f	0.1564 x
i9-i12	49	38	107.5	1.50	0 f	0.6626 x
i9-i13	45	161	143.5	10.82	1.091	1.778
i9-i14	35	170	168.5	26.80	3.275	4.379
i9-i15	58	55	117.5	6.13	0 f	0.6626 x
i9-i16	60	133	138.5	13.77	0.219	1.02
i9-i17	59	136	161.5	47.10	0.3423 x	1.64
i12-i13	48	198	154	8.90	0.645 x	1.249 x
i12-i14	53	196	179	29.56	0.821	1.861

f signifies the wedge is floating.

x signifies contact on only one plane

PREAKNESS MOUNTAIN BASALT

C = 0, O = 56.1, Unit Weight + 176 PCF

RELEVANT FOR WORKSHAFTS 4 and 6.

PASSAIC TUNNEL PROJECT
POTENTIALLY UNSTABLE WEDGES
SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

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Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

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JOINT NUMBER	DIP	DIP DIRECTION
i1	90	180
i2	55	6
i3	61	331
i4	85	318
i7	48	318
i8	84	302
i9	66	97
i12	82	118
i13	50	190
i14	62	240
i15	87	138
i16	70	180
i17	90	226

Joints i5, i6, i10, and i11 do not form wedges that are free to move.

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS						
JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY	
					WET SLOPE	DRY SLOPE
i12-i15	74	62	128	3.54	0 f	0.209 x
i12-i16	68	184	149	12.62	0 f	0.542 x
i12-i17	80	136	172	105.38	0 f	0.302
i13-i14	48	186	215	3.38	0.705 x	1.249 x
i13-i15	43	224	164	5.38	0.494 x	1.249 x
i13-i17	35	136	208	3.07	0.397 x	1.249 x
i14-i15	60	221	189	26.46	0.291 x	1.057
i14-i16	60	226	210	7.02	0.209 x	0.872
i15-i16	63	220	159	7.52	0 f	0.542 x
i15-i17	86	136	182	186.43	0 f	0.078 x
i16-i17	62	136	203	10.78	0 f	0.542 x

f signifies the wedge is floating.
x signifies contact on only one plane

PREAKNESS MOUNTAIN BASALT **C = 0, O = 56.1, Unit Weight + 176 PCF**

RELEVANT FOR WORKSHAFTS 4 and 6.

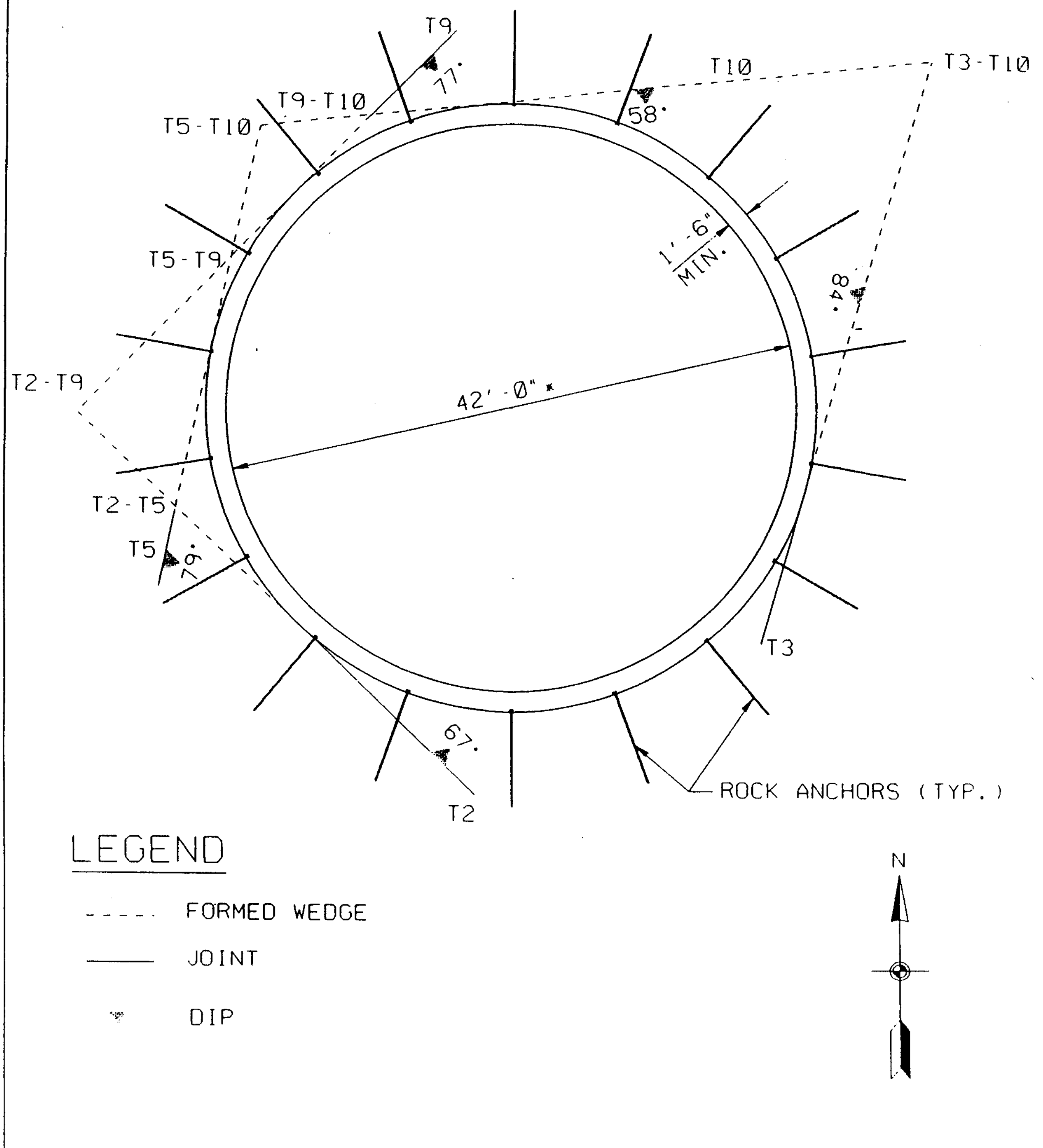
SHAFT 3

POTENTIALLY UNSTABLE WEDGES

JOINT COMBINATION	SHEET NO.
T2-T9	1
T3-T1Ø	1
T5-T9	1
T5-T1Ø	1
T6-T1Ø	2
T8-T1Ø	2
T9-T1Ø	1
T1-T2	3
T1-T5	3
T1-T6	4
T1-T1Ø	3
T2-T3	3
T2-T5	3
T2-T6	4
T2-T8	4

SHAFT 3

POTENTIALLY UNSTABLE WEDGES

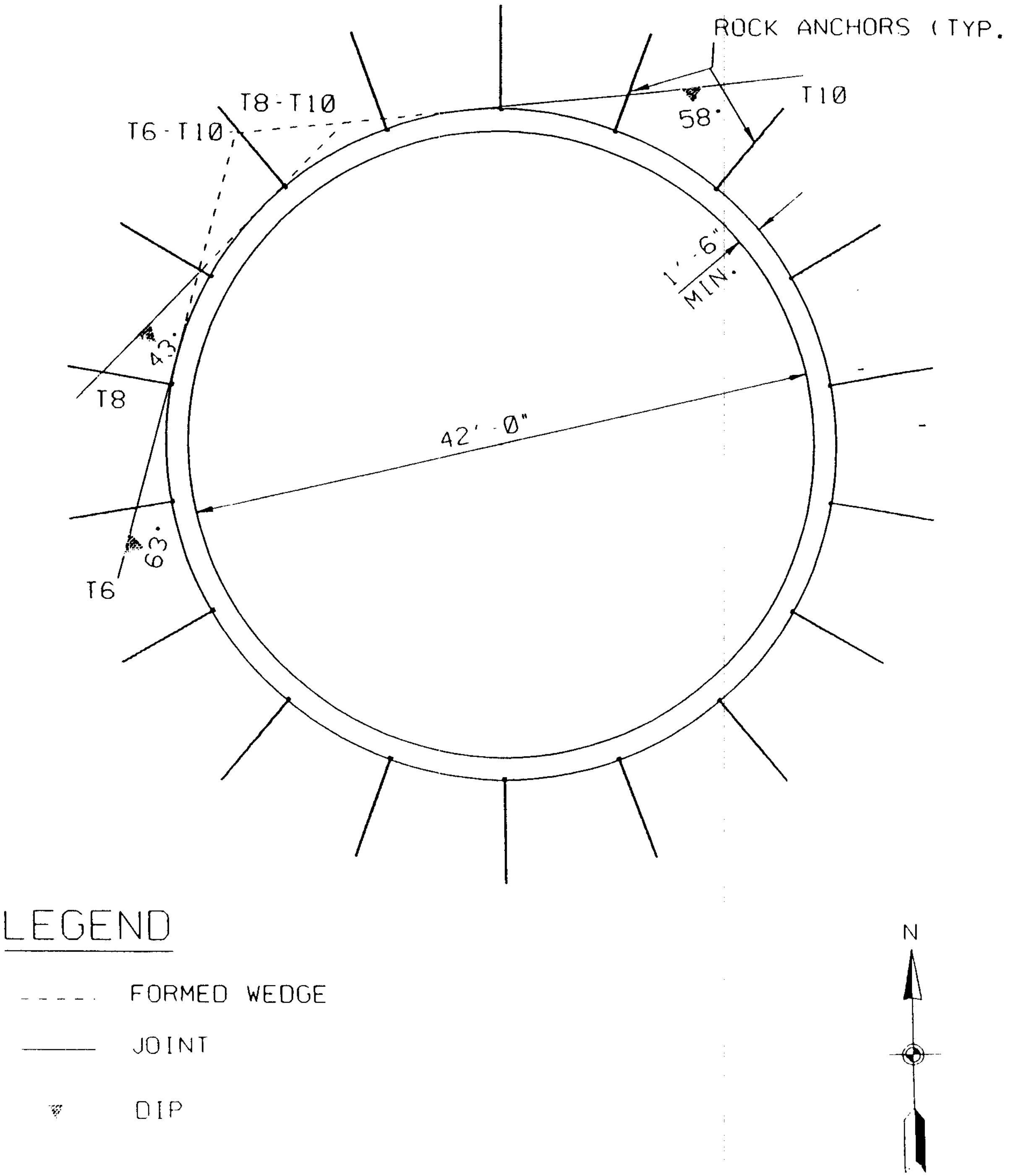


PLAN VIEW

SCALE: 1" = 10'

SHEET 1

SHAFT 3
POTENTIALLY UNSTABLE WEDGES



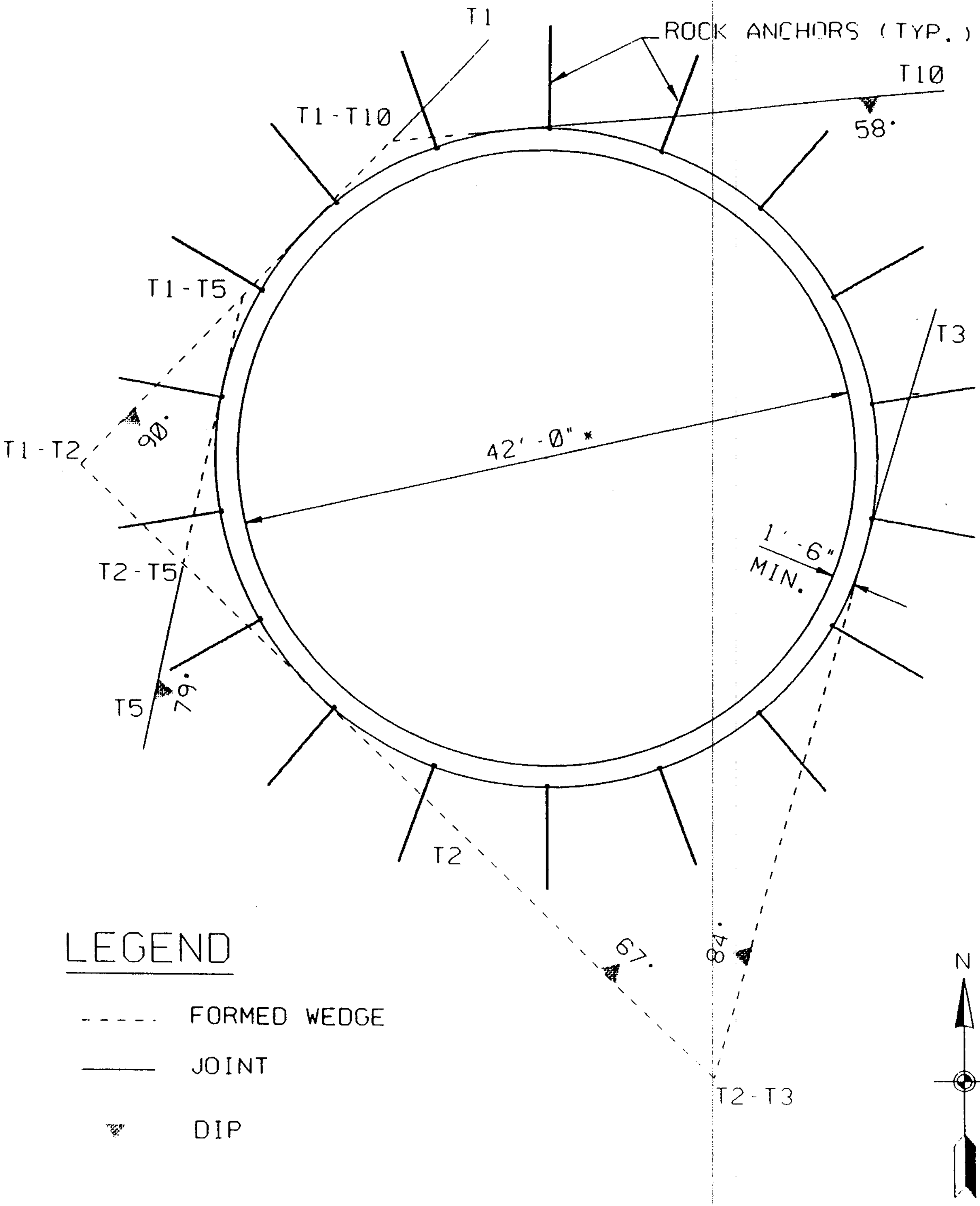
PLAN VIEW

SCALE: 1" = 10'

E.4.4-12

SHAFT 3

POTENTIALLY UNSTABLE WEDGES



PLAN VIEW

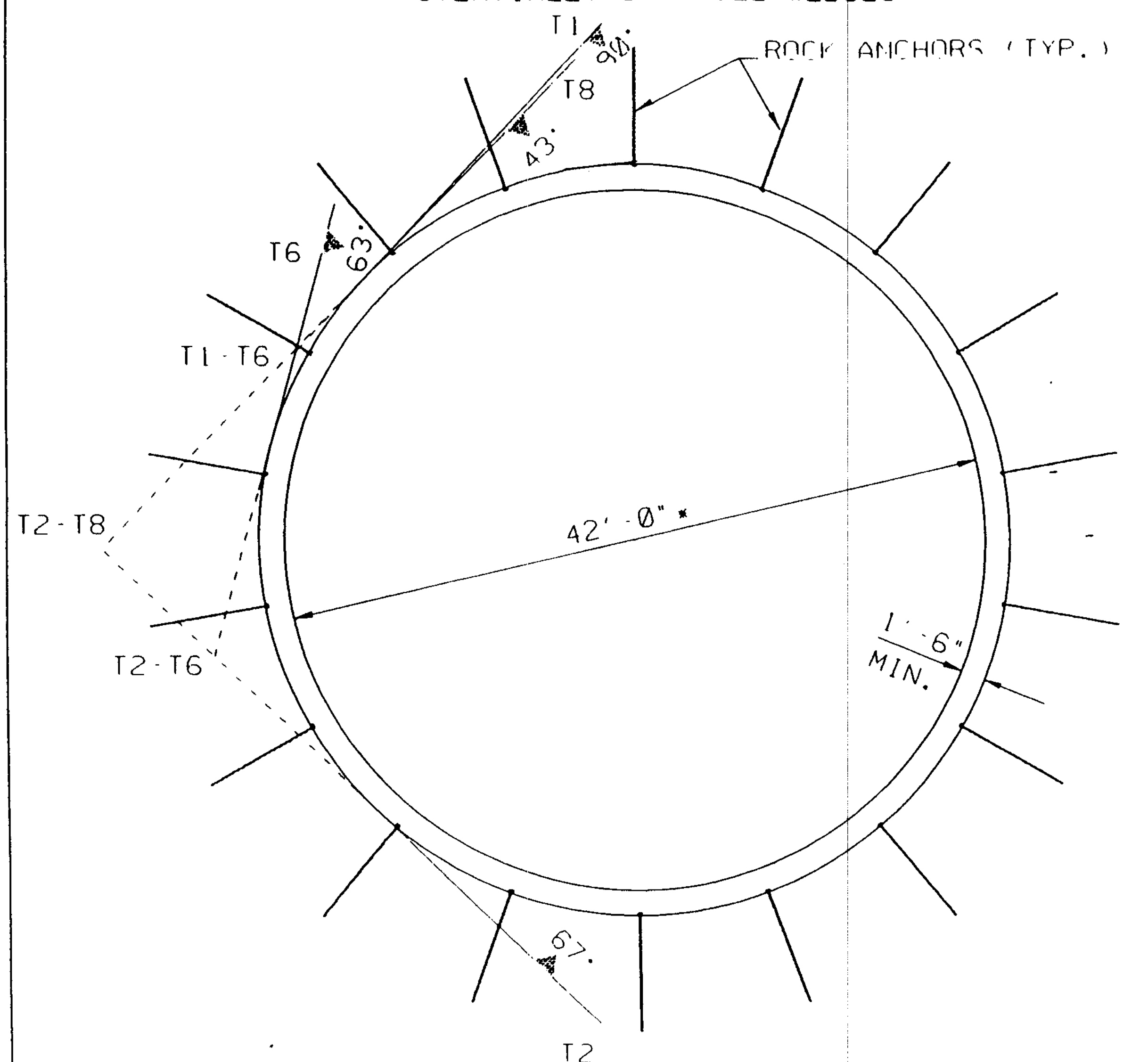
SCALE: 1" = 10'

E.4.4-13

SHEET3

SHAFT 3

POTENTIALLY UNSTABLE WEDGES



LEGEND

- - - - FORMED WEDGE

——— JOINT

DIP

PLAN VIEW

SCALE: 1" = 10'

PASSAIC TUNNEL PROJECT
POTENTIALLY UNSTABLE WEDGES
SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

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Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

Note 6: The height of the wedge is approximated as shown on the attached sheet. This height is the maximum height possible for the joint combination shown and for the largest work shaft. This is a very conservative approach.

JOINT NUMBER	DIP	DIP DIRECTION
t1	90	133
t2	67	44
t3	84	286
t5	79	102
t6	63	105
t8	43	134
t9	77	134
t10	58	174

Joints t4 and t7 do not form wedges that are free to move.

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS							
JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY		
					WET SLOPE	DRY SLOPE	
t1-t2	67	43	88.5	52.09	0 f	0.3352	x
t1-t3	77	223	209.5	405.94	0 f	0.744	
t1-t5	68	43	117.5	15.44	0 f	0.1535	x
t1-t6	44	43	119	5.42	0 f	0.4024	
t1-t10	46	223	153.5	8.71	0 f	0.4935	
t2-t3	63	5	345	67.04	0.1134 x	0.7295	
t2-t5	68	42	73	17.46	0 f	0.3352	x
t2-t6	61	86	74.5	12.38	0.0697	0.4831	
t2-t8	43	114	89	16.37	0.6382	1.052	

f signifies the wedge is floating.
x signifies contact on only one plane

TOWACO FORMATION

C = 0, O = 38.3, Unit Weight + 155 PCF

RELEVANT FOR THE SPUR INLET AND WORK SHAFT 3.

PASSAIC TUNNEL PROJECT

POTENTIALLY UNSTABLE WEDGES

SHAFTS AND INLETS

Note 1: For wedge stability in the shafts assume the cut slope orientation to be perpendicular to the line of intersection of the two joints.

Note 2: The conventional wedge analysis is slightly less conservative in this application because it assumes a flat slope face instead of a curved one. For this reason the weight of the wedge is less than calculated and therefore the normal forces on the planes are less. This is compensated for slightly by the fact that the wedges formed are not as free to move into the opening because they are constrained by the curved sides of the shaft.

Note 3: The upper slope surface is assumed to be formed by a bedding plane in the rock. This is a conservative assumption since no bedding plane may exist at the optimum location.

Note 4: The wet slope analysis assumes the joints to be free draining where they intercept the shaft.

Note 5: The factors of safety were computed using the "ROCKPACK" computer program developed after the proceedures in Hoek and Bray by C.F. Watts. Rock reinforcement is not accounted for in these figures, it is figured separately for those wedges which are unstable.

Note 6: The height of the wedge is approximated as shown on the attached sheet. This height is the maximum height possible for the joint combination shown and for the largest work shaft. This is a very conservative approach.

JOINT NUMBER	DIP	DIP DIRECTION
t1	90	133
t2	67	44
t3	84	286
t5	79	102
t6	63	105
t8	43	134
t9	77	134
t10	58	174

Joints t4 and t7 do not form wedges that are free to move.

POSSIBLE UNSTABLE JOINT COMBINATIONS IN SHAFTS

JOINT COMBINATION	INTERSECTION LINE SLOPE	INTERSECTION LINE BEARING	FACE ORIENTATION	APPROXIMATE WEDGE HEIGHT	FACTORS OF SAFETY	
					WET SLOPE	DRY SLOPE
t2-t9	65	74	89	35.32	0.025 x	0.4963
t2-t10	40	116	109	36.97	1.099	1.647
t3-t9	55	205	210	125.53	0.7172	1.889
t3-t10	54	204	230	42.35	0.27	0.8366
t5-t9	76	137	118	7.54	0 f	0.1823 x
~~t5-t10~~	~~57~~	~~175~~	~~138~~	~~18.53~~	~~0.1351~~	0.4941
t6-t10	54	149	139.5	12.22	0.2334	0.6263
t8-t10	44	120	154	3.26	0.4964	0.8469
t9-t10	53	205	154	5.91	0.062 x	0.4935

f signifies the wedge is floating.
x signifies contact on only one plane

TOWACO FORMATION

C = 0, O = 38.3, Unit Weight + 155 PCF

RELEVANT FOR THE SPUR INLET AND WORK SHAFT 3.

PASSAIC RIVER FLOOD DAMAGE
REDUCTION PROJECT

SECTION 4
SHAFTS

ATTACHMENT E.4.5
DETAILED SOIL BORING LOGS

ATTACHMENT E.4.5

DETAILED SOIL BORING LOGS

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 9 SHEETS

1. PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL
2. LOCATION *(Coordinates or Station)* SITE 2BK, BERGEN AVE., KEARNY , NJ
3. DRILLING AGENCY SUMMIT DRILLING
4. HOLE NO. *(As shown on drawing title and file number)* DC-21 (IT-2BK-PB01)
5. NAME OF DRILLER DJ GRAHAMER
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ________ DEGREES FROM VERTICAL
7. THICKNESS OF OVERBURDEN 155'
8. DEPTH DRILLED INTO ROCK 355'
9. TOTAL DEPTH OF HOLE 510'
10. SIZE AND TYPE OF BIT MUD-ROTARY
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*
12. MANUFACTURER'S DESIGNATION OF DRILL
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN | DISTURBED | UNDISTURBED
14. TOTAL NUMBER CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE STARTED 5-16-94 COMPLETED 5-17-94
17. ELEVATION TOP OF HOLE
18. TOTAL CORE RECOVERY FOR BORING
19. SIGNATURE OF INSPECTOR GERRY GILLILAND

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS *(Description)* d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS *(Drilling Time, water loss, etc.)* g.
	0'–6'		VARIOUS SOILS (ORGANIC SOIL, YELLOW SANDY CLAY, SAND) MIXED WITH CUT GRAVEL, GLASS, AND BRICKS -FILL (GC)	2, 7, 18, 16		10" RECOVERY 3" STAINLESS-STEEL SPOON, LOW RECOVERY
				3, 2, 1, 1		8" RECOVERY SAME AS ABOVE SAMPLE (LOW RECOVERY)
				3, 18, 17, 10		8" RECOVERY SAME AS ABOVE SAMPLE (VERY LITTLE RECOVERY)
	6'–9'		ORGANIC SOIL WITH SOME FILL SETTLED IN (OL)	2, 1, 1, 1		1" RECOVERY 2" SPOON, LOW RECOVERY
				6, 12		18" RECOVERY 2X3" SS SPOONS FOR HTRW
	9'–12.5'		GRAY, SATURATED SANDY SILT (SM)	20, 33		
	12'–14'		GRADES INTO GRAY, SATURATED SILTY SAND (SM)	12, 26, 32, 29		16" RECOVERY 2X3" SS SPOONS HTRW

ENG FORM 1836 (CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. DC-21

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 2 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			SAME			
	15'					
	16'			5		18" RECOVERY 2" SPOON
				6		
	17'			12		
				18		
	18'					
	19'					
	20'			5		15" RECOVERY 2" SPOON
			GRAY, SATURATED, CLEAN M-C SAND AND FINE GRAVEL (SP)	10		
	21'			17		
				22		
	22'					
	23'					
	24'			9		18" RECOVERY 2" SPOON
				15		
	25'		BROWNISH/GRAY, SILTY VERY FINE SAND (SM)	18		
				27		
	26'					
	27'					
	28'		SAME	11		12" RECOVERY 2" SPOON
				18		
	29'			18		
				19		
	30'					
	31'					
	32'		GRADATION TO SANDY SILT WITH TRACE CLAY (ML)	10		16" RECOVERY 2" SPOON
	33'			16		

ENG FORM 1836A (CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PREVENTION, MAIN TUNNEL | HOLE NO. DC-21

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 3 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			SAME	16		
				17		
	34'					
	35'					
	36'		SAME	12		18" RECOVERY 2" SPOON
				15		
	37'			15		
				12		
	38'					
	39'					
	40'		SAME	7		20" RECOVERY 2" SPOON
				8		
	41'			10		
				10		
	42'					
	43'					
	44'		BROWNISH/GRAY TO BROWN, VARVED SILTY VERY FINE SAND AND SILTY CLAY (SM/CL)	12		22" RECOVERY 2" SPOON
				14		
	45'			14		
				11		
	46'					
	47'					
	48'			10		22" RECOVERY 2" SPOON
				14		
	49'			18		
				20		
	50'		GRAY, VARVED CLAY AND CLAYEY SILT WITH LENSES OF DARK RED/BROWN VERY FINE SAND (CL)	10		20" RECOVERY 2X3" STAINLESS SPOONS FOR HTRW SAMPLE
	51'			12		
				14		
	52'			15		

ENG FORM 1836A
(CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. DC-21

DRILLING LOG | DIVISION NORTH ATLANTIC | INSTALLATION NEW YORK DISTRICT | SHEET 4 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			SAME			
	53'					
	54'					
	55'			3		22" RECOVERY 2" SPOON
				3		
	56'			4		
				4		
	57'					
	58'					
	59'					
	60'		MOD-BROWN TO MOD-REDDISH/BROWN, VARVED, CLAY AND SILT WITH TRACE VERY FINE SAND SEAMS (NOTE COLOR CHANGE WITH THIS SPOON)	7		22" RECOVERY 2" SPOON
				9		
	61'			11		
				14		
	62'					
	63'					
	64'					
	65'			5		20" RECOVERY 2" SPOON
	66'		SAME, VARVES ARE 0.25" THICK	7		
				9		
				11		
	67'					
	68'					
	69'					
	70'		SAME, VARVES ARE 0.5"-1" THICK WITH THIN VERY FINE SAND SEAMS	5		24" RECOVERY 2" SPOON
	71'			6		

ENG FORM 1836A (CADD Facsimilie) | PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. DC-21

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 5 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			SAME	7		
				12		
	72'					
	73'					
	74'					
	75'		SAME	2		24" RECOVERY 2" SPOON
				4		
	76'			7		
				7		
	77'					
	78'					
	79'					
	80'		MOD-BROWN TO MOD-RED/ BROWN, VARVED SILT AND CLAY (SILT CONTENT HAS INCREASED, CLAY LENSES ARE THIN (CL/ML)	4		22" RECOVERY 2" SPOON
				4		
	81'			5		
				7		
	82'					
	83'					
	84'					
	85'		SAME	WGT		21" RECOVERY 2" SPOON
				OF		
	86'			RODS		
				6		
	87'					
	88'					
	89'					
	90'					

ENG FORM 1836A
(CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. DC-21

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 6 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	90'–95'		SAME	WGT OF RODS "		24" RECOVERY 2" SPOON
	95'–100'		SAME	WGT OF RODS "		22" RECOVERY 2" SPOON
	100'–102'		SAME	20 30 24 39		24" RECOVERY 3" STAINLESS-STEEL SPOON FOR HTRW SAMPLE, COMPOSITE
	102'–104'			14 35 35 35		18" RECOVERY (100' – 104') 3" SS SPOON FOR COMPOSITE HTRW SAMPLE LISTED ABOVE
	103'		MOD-RED/BROWN, SILTY VERY FINE SAND WITH TRACE CLAY (SM)			
	105'–107'			65 45 37 46		24" RECOVERY 2" SPOON
	109'					

ENG FORM 1836A
(CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION MAIN TUNNEL | HOLE NO. DC-21

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 7 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	110'			35		24" RECOVERY 2" SPOON
				36		
	111'		TRACE M-C SAND	35		
				45		
	112'					
	113'					
	114'					
	115'			28		18" RECOVERY 2" SPOON
				45		
	116'		TRACE F GRAVEL	70		
				100/4"		
	117'					
	118'					
	119'					
	120'		MOD-RED/BROWN, M-C SAND WITH SOME SILT AND TRACE FINE GRAVEL (SM)	45		10" RECOVERY 2" SPOON
				58		
	121'			63		
				65		
	122'					
	123'					
	124'					
	125'		MOD-RED/BROWN FINE SAND INTERBEDDED WITH LITTLE CLAYEY SILT (SM)	47		10" RECOVERY 2" SPOON
	126'			92		
				100/2"		
	127'					
	128'					

ENG FORM 1836A
(CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION MAIN TUNNEL | HOLE NO. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 8 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	129'					
	130'		MOD-RED/BROWN CLAYEY SILT AND GRAY, SUBANGULAR TO ROUNDED, FINE GRAVEL (ML)	71 96 100/3"		12" RECOVERY 2" SPOON
	131'					
	132'					
	133'					
	134'					
	135'		MOD-RED/BROWN SILTY FINE SAND WITH TRACE CLAY LENSES (ML)	50 55 72 100/3"		12" RECOVERY 2" SPOON
	136'					
	137'					
	138'					
	139'					
	140'		MOD-RED/BROWN, FINE SAND AND CLAY MIXED WITH WEATHERED ROCK FRAGMENTS (GLACIAL TILL) (SC/CL)	48 100/1"		7" RECOVERY 2" SPOON USED DOWN PRESSURE FOE FIRST TIME AT 141'
	141'					
	142'					
	143'					
	144'		BOULDER/LEDGE FROM APROX. 141' TO 148'			
	145'			100/0"		0" RECOVERY 2" SPOON
	146'			100/0"		
	147'					

ENG FORM 1836A
(CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION MAIN TUNNEL | HOLE NO. DC-21

Hole No. DC-21

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 9 OF 9 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			GLACIAL TILL			
	148					
	149					
	150					
	151					
	152					
	153					
	154					
	155		COMPETENT SILTSTONE BEDROCK (PASSAIC FORMATION)			
	156					
			ROCK DRILLING TO 510'			
	157					
	158					
	159					
	160					
	161					
	162					
	163					
	164					
	165					
	166					

ENG FORM 1836A (CADD Facsimilie)

PROJECT PASSAIC RIVER FLOOD PROTECTION MAIN TUNNEL | HOLE NO. DC-21

HOLE NO: C-146

DRILLING LOG

DIVISION: N. Atlantic	INSTALLATION: NY District — SHEET: 1 OF 63
PROJECT: Passaic River Flood Protection Tunnel	DRILL BIT: Pq (4.827"), diamond surface set
LOCATION: Newark Bay (Kearney Point)	EL. DATUM: Bay bottom
DRILLING CONT: Warren George, Inc.	DRILL RIG: Ackler Soilmax
HOLE NO (DRAWING): C-145	TOTAL SOIL SAMPLES: 44 — DIST: 30 — UNDIST: 14
DRILLER: Greg Marney	TOTAL NUMBER OF CORE BOXES: 29
	GROUNDWATER ELEVATION: 0 MSL
HOLE DIRECTION: Vertical	DATE STARTED: 9/16/93 — COMPLETED: 10/15/93
OVERBURDEN THICKNESS (FEET): 103	TOP OF HOLE ELEVATION: ?
DEPTH INTO ROCK (FEET): 396	TOTAL CORE RECOVERY: 390.6 feet
TOTAL DEPTH (FEET): 499	INSPECTOR'S SIGNATURE:

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
0.0	0.0		SILT (ML), homogeneous Munsell N2 (grayish black)				6" ID Casing 2" OD Split-Spoon 140 LB. Hammer, 30" Drop	
-10	10			33%	S-1			WOR
-20	20							
-30	30			100%	S-2			WOR
-40	40							
				100%	S-3		Strata continues to 8.5'	WOR
	50							

DRILLING LOG	HOLE NO: C-146	SHEET: 2 OF 63					PROJECT: NEWARK BAY	
ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-5.0	5.0		SILT: (ML), homogenous. Munsell N2 (grayish black).		S-3			
-6.0	6.0							
-7.0	7.0			100%	T-1			PUSH
-8.0	8.0							
-9.0	9.0		SAND: w/ little silt, medium (SM) Munsell 5Y 6/1 (lt olive gray)					
-10.0	10.0			100%	T-2			PUSH
-11.0	11.0		SILT (ML) Munsell N2 (grayish black).					
-12.0	12.0			100%	S-4			WOR
			SAND: Continued on Sheet 3					

DRILLING LOG	HOLE NO: C-146	SHEET: 3 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-13.0	13.0		SAND: medium to course w/ little silt & trace fine gravel & shells (SM). Munsell 5Y 4/1 (olive gray).				Attempted 3" Shelby 13' to 15.5', no recovery.	
-14.0	14.0			50%	S-5			5-3-4-7
-15.0	15.0							
-16.0	16.0			50%	S-6			5-12-13-14
-17.0	17.0							
-18.0	18.0			50%	S-7			7-11-13-15
-19.0	19.0							
-20.0	20.0			50%	S-8		6" ID casing to 20'	12-15-16-19
	21.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 4 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-21.0	21.0		SAND: medium to course, loose w/ trace course gravel & silt (SM). Munsell 10R 4/2 (grayish red).					
-22.0	22.0			33%	S-9			7-9-14-15
-23.0	23.0		SAND: medium to course, loose w/ trace silt & clay (SC). Munsell 10R 4/2 (grayish red).					
-24.0	24.0			63%	S-10			9-10-11-14
-25.0	25.0		SILTY CLAY: firm w/ medium plasticity (CL). Munsell 10R 4/2 (grayish red).					
-26.0	26.0			67%	T-3			PUSH
-27.0	27.0							
-28.0	28.0			100%	T-4			PUSH
	29.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 5 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-29.0	29.0		SILTY CLAY: firm w/ medium plasticity (CL). Munsell 10R 4\2 (grayish red)		T-4			
-30.0	30.0		SILT: medium hard (ML). Munsell 10R 3\4 (dk reddish brown).					
-31.0	31.0			50%	S-11			15-22-34-40
-32.0	32.0							
-33.0	33.0			33%	T-5			PUSH
-34.0	34.0							
-35.0	35.0		SILTY CLAY: firm (CL). Munsell 10R 4\2 (grayish red)	66%	S-12			5-8-12-20
-36.0	36.0							
	37.0				S-13			

DRILLING LOG	HOLE NO: C-146	SHEET: 6 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-37.0	37.0		SILTY CLAY: firm (CL). Munsell 10R 4\2 (grayish red).	100%	S-13			7-8-11-13
-38.0	38.0							
-39.0	39.0			100%	T-6			PUSH
-40.0	40.0		SILT: medium hard (ML). Munsell 10R 3\4 (dk reddish brown).					
-41.0	41.0							
-42.0	42.0		SILTY CLAY: firm (CL). Munsell 10R 4\2 (grayish red)	100%	S-14		9/17/93	7-8-11-14
-43.0	43.0							
-44.0	44.0			50%	S-15			6-7-12-14
	45.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 7 OF 63	PROJECT: NEWARK BAY				
ELEV. (FT)	DEPTH (FT)	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-45.0	45.0	SILT: (ML). Munsell 10R 3\4 (dk reddish brown).					
-46.0	46.0		100%	T-7			PUSH
-47.0	47.0						
-48.0	48.0	SILTY CLAY: Firm (CL). Munsell 10R 3\4 (dk reddish brown).					
-49.0	49.0		100%	T-8			PUSH
-50.0	50.0						
-51.0	51.0		66%	S-16			10-8-10-12
-52.0	52.0	CLAYEY SILT: (ML). Munsell 10R 3\4 (dk reddish brown).	100%	T-9			PUSH
	53.0						

DRILLING LOG | HOLE NO: C-146 | SHEET: 8 OF 63 | PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-53.0	53.0		CLAYEY SILT: (ML). Munsell 10R 3\4 (dk reddish brown).	100%	T-9			PUSH
-54.0	54.0							
-55.0	55.0		SILTY CLAY: Firm (CL). Munsell 10R 3\4 (dk reddish brown)	100%	T-10			PUSH
-56.0	56.0							
-57.0	57.0			63%	S-17		Trace 1/4" to 1/2" Fragmented siltstone to 70' 9/20/93	9-12-13-14
-58.0	58.0							
-59.0	59.0			100%	T-11			PUSH
-60.0	60.0							
	61.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 9 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-61.0	61.0		SILTY CLAY: firm (CL). Munsell 10R 3\4 (dk reddish brown). Trace 1/4" to 1/2" fragmented siltstone to 70'		T-11			
-62.0	62.0							
				66%	S-18			9-11-12-18
-63.0	63.0							
-64.0	64.0							
				66%	S-19			8-11-14-19
-65.0	65.0							
-66.0	66.0							
				100%	T-12			PUSH
-67.0	67.0							
-68.0	68.0							
				33%	S-20		Attempted Shelby from 68' to 70.5', no recovery	4-7-10-9
	69.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 10 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-69.0	69.0		SILTY CLAY: Firm (CL). Munsell 10R 3\4 (dk reddish brown). Trace 1/4" to 1/2" fragmented siltstone to 70'.		S-20			
-70.0	70.0		Trace 1" siltstone & basalt pebbles to 78.5'.					
-71.0	71.0			100%	S-21		-	11-14-18-22
-72.0	72.0							
-73.0	73.0			100%	T-13			PUSH
-74.0	74.0							
-75.0	75.0			67%	S-22			17-22-29-30
-76.0	76.0							
	77.0			63%	S-23			14-17-23-24

DRILLING LOG	HOLE NO: C-146	SHEET: 11 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-77.0	77.0		SILTY CLAY: firm (CL). Munsell 10R 3\4 (dk reddish brown). Trace 1" siltstone & basalt pebbles to 78.5'.	63%	S-23			14-17-23-24
-78.0	78.0							
-79.0	79.0		SILTY CLAY: firm (CL). Munsell 10R 3\4 (dk reddish brown)	100%	S-24			7-9-12-14
-80.0	80.0							
-81.0	81.0		CLAYEY SILT: medium hard (CL). Munsell 10R 3\4 (dk reddish brown) & 10R 4\2 (grayish red).	67%	S-25			10-12-14-15
-82.0	82.0							
-83.0	83.0			67%	T-14			PUSH
-84.0	84.0							
	85.0							

DRILLING LOG | HOLE NO: C-146 | SHEET: 12 OF 63 | PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-85.0	85.0		SILTY CLAY: firm (CL). Munsell 10R 3\4 (dk reddish brown).					
-86.0	86.0			67%	S-26			10-11-14-17
-87.0	87.0		CLAYEY SILT: medium hard (ML) Munsell 10R 3\4 (dk reddish brown). Little 1/4" siltstone pebbles					
-88.0	88.0			67%	S-27			14-17-27-34
-89.0	89.0							
-90.0	90.0			0			Attempted split-spoon, no recovery	24-38-47-100
							9/21/93	
-91.0	91.0		Hard, very broken				Did not sample, anticipated rock. Drilled out.	
-92.0	92.0							
	93.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 13 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	SPT BLOWS (PER 6")
-93.0	93.0		CLAYEY SILT: hard (ML). Munsell 10R 3\4 (dk reddish brown).				Drilled out, not sampled to 98'.	
-94.0	94.0							
-95.0	95.0							
-96.0	96.0							
-97.0	97.0							
-98.0	98.0		CLAYEY SILT: hard, dry (ML), w/ some 1\4" to 1\2", sub-angular, unweathered siltstone. Munsell 10R 3\4 (dk reddish brown)					
-99.0	99.0			63%	S-28			75-60-65-120
-100.0	100.0						Drilled out, not sampled to 107'.	
	101.0							

DRILLING LOG	HOLE NO: C-146	SHEET: 14 OF 63	PROJECT: NEWARK BAY

ELEV. (FT)	DEPTH (FT)	LEGEND	DESCRIPTION	REC. (%)	SAMPLE NO.	SAMPLE TYPE	REMARKS	DRILL TIME
-101.0 to -103.0	101.0 to 103.0		CLAYEY SILT: hard, dry (ML), w/ some 1/4" to 1/2", sub-angular, unweathered siltstone. Munsell 10R 3/4 (dk reddish brown).					
-103.0 to -107.0	103.0 to 107.0		Top of rock				Drilled out and set PW (5" ID) casing to 10[illegible] 4" into rock 9/22/93	
-107.0 to -109.0	107.0 to 109.0		SANDSTONE: Fine, medium soft, permeable w/ few small vugs. Munsell 10R 4/6 (moderate reddish brown)				15° - 30° jointing, ∅)α Is = 222 PSI @ 107.5' Core w/ Pq (4.827" hole) barrel Bit = 3.345" diamond. Pressure = ? 130 RPM Drilling Fluid: Water 45° joint @ 108.9', ∅)α	6:40 5:40

Hole No. C-67

DRILLING LOG | DIVISION: NORTH ATLANTIC | INSTALLATION: NEW YORK DISTRICT | SHEET 1 OF 1 SHEETS

1. PROJECT: PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL
2. LOCATION (Coordinates or Station): WORKSHAFT 5 (HOOK HOLE)
3. DRILLING AGENCY: MOBILE DISTRICT
4. HOLE NO. (As shown on drawing title and file number): C-67
5. NAME OF DRILLER: CARL MOON
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ________ DEGREES FROM VERTICAL
7. THICKNESS OF OVERBURDEN: 27.0'
8. DEPTH DRILLED INTO ROCK: 221.0'
9. TOTAL DEPTH OF HOLE: 248.0'
10. SIZE AND TYPE OF BIT: HQ
11. DATUM FOR ELEVATION SHOWN (TBM or MSL):
12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 1500
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED | UNDISTURBED
14. TOTAL NUMBER CORE BOXES: 21
15. ELEVATION GROUND WATER: 8.8'
16. DATE HOLE: STARTED 11-9-91 | COMPLETED 11-18-91
17. ELEVATION TOP OF HOLE:
18. TOTAL CORE RECOVERY FOR BORING: 99.8%
19. SIGNATURE OF INSPECTOR: ROBERT R. BIMAN

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS e.	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	0		ROLLER BIT DRILLING TO 27.0'			
	2'		NO SAMPLING			
	4'		BROWN/RED BROWN SAND AND GRAVEL NOTED IN WASH			
	6'					
	8'					
	10'					
	12'					
	14'					
	16'					
	18'					
	20'					
	22'					
	24'					
	26'		TOP OF ROCK = 27.0'			
	28'		RED BRN. SANDSTONE CORE DRILLING TO 248.0'			

ENG FORM 1836 (CADD Facsimilie) 1C-67.dwg | PROJECT: PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. C-67

Hole No. DC-99

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 1 SHEETS

1. PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL	10. SIZE AND TYPE OF BIT SPT/HQ DIAMOND
2. LOCATION (Coordinates or Station) 30 GALESI DRIVE, WAYNE, NJ	11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
3. DRILLING AGENCY MOBILE	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 314
4. HOLE NO. (As shown on drawing title and file number) DC-99	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED 10 — UNDISTURBED 0
5. NAME OF DRILLER JAMES KNOX	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED _________ DEGREES FROM VERTICAL	15. ELEVATION GROUND WATER
	16. DATE HOLE — STARTED 6-6-91 — COMPLETED 6-15-91
7. THICKNESS OF OVERBURDEN 16.5'	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK 211.2'	18. TOTAL CORE RECOVERY FOR BORING 97.8%
9. TOTAL DEPTH OF HOLE 227.7'	19. SIGNATURE OF INSPECTOR JAMES S. BEAUJO

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS e.	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	0'–1'		BROWN CLAYEY SAND WITH BRICK AND ROCK FRAGS. (SM-SC) -FILL	5 / 27 / 14	1	0.9' RECOVERY
	1'–3'		SAME -FILL	15 / 11 / 10	2	0.9' RECOVERY
	3'–4.5'		SAME -FILL	5 / 4 / 5	3	0.6' RECOVERY
	4.5'–6'		SAME -FILL	3 / 3 / 12	4	0.8' RECOVERY
	6'–7'		BROWN SANDY CLAY, TRACE FINE GRAVEL (CL)	5 / 18	5	1.1' RECOVERY
	7'			50	6	BOULDER/COBBLE
	7.5'–9'		SAME	30 / 16 / 21	7	0.5' RECOVERY
	9'–10.5'		BROWN CLAY LITTLE FINE SAND, TRACE FINE GRAVEL (CL)	15 / 34	8	0.8' RECOVERY BOULDERS 10-11.5'
	10.5'–12'		SAME	50+	9	0.5' RECOVERY
	12'–12.5'		BROWN SANDY CLAY WITH ROCK FRAGMENTS (CL)	50+	10	0.5' RECOVERY
	12.5'–14'		TOP OF ROCK = 12.5' BROWN FRACTURED BASALT ROCK CORE DRILLING TO 227.7'			

ENG FORM 1836 (CADD Facsimilie) 1DC-99.dwg

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. DC-99

Hole No. C-98

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 1 SHEETS
1. PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL		10. SIZE AND TYPE OF BIT HQ DIAMOND	
2. LOCATION (Coordinates or Station) 555 RTE 46, WAYNE, NJ		11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	
3. DRILLING AGENCY MOBILE DISTRICT		12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 1500	
4. HOLE NO. (As shown on drawing title and file number) C-98		13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED / UNDISTURBED
5. NAME OF DRILLER CARL MOON		14. TOTAL NUMBER CORE BOXES	
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED _________ DEGREES FROM VERTICAL		15. ELEVATION GROUND WATER	
7. THICKNESS OF OVERBURDEN 33.4'		16. DATE HOLE STARTED 8-17-91	COMPLETED
8. DEPTH DRILLED INTO ROCK 189.9'		17. ELEVATION TOP OF HOLE	
9. TOTAL DEPTH OF HOLE 223.3'		18. TOTAL CORE RECOVERY FOR BORING 99%	
		19. SIGNATURE OF INSPECTOR ROBERT B. BOMAN	

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS e.	SAMPLE/BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	0		ROLLER BIT DRILLING TO 33.4'			
	3'		NO SAMPLING			
	6'		BROWN GRAVEL NOTED IN WASH			
	9'					
	12'					
	15'					
	18'					
	21'					
	24'					
	27'					
	30'					
	33'		TOP OR ROCK = 33.4'			
			BROWN FRACTURED BASALT			
	36'		ROCK CORE DRILLING TO 223.3'			
	39'					
	42'					

ENG FORM 1836 (CADD Facsimile)

1C-98.dwg

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. C-98

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 3 SHEETS

1. PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL	10. SIZE AND TYPE OF BIT 2" SPT
2. LOCATION (Coordinates or Station) WORKSHAFT 2A	11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
3. DRILLING AGENCY USACE – MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 314
4. HOLE NO. (As shown on drawing title and file number) DC-48	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 30; UNDISTURBED
5. NAME OF DRILLER JAMES KNOX	14. TOTAL NUMBER CORE BOXES 32
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ________ DEGREES FROM VERTICAL	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 9/19/94; COMPLETED 11/10/94
7. THICKNESS OF OVERBURDEN 45.5'	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK 356.1'	18. TOTAL CORE RECOVERY FOR BORING 99%
9. TOTAL DEPTH OF HOLE 401.1'	19. SIGNATURE OF INSPECTOR ROBERT BROWN

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS e.	SAMPLE/BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	0'–1'		BROWN CLAYEY SILT, TRACE FINE TO MEDIUM SAND WITH ROOTS (ML)	1, 4, 1	1	0.3' RECOVERY
	1'–2'		SAME	1, 1, 3	2	0.5' RECOVERY
	3'–4'		DARK BROWN CLAYEY SILT (ML)	4, 5, 16	3	1.0' RECOVERY
	4'–5'		RED/BROWN CLAYEY SILT (ML)	22, 20, 19	4	1.2' RECOVERY
	6'–7'		SAME	25, 19, 21	5	1.1' RECOVERY
	7'–8'		SAME WITH TRACE SAND AND GRAVEL	15, 15, 18	6	0.5' RECOVERY
	9'–10'		SAME	12, 11, 11	7	1.3' RECOVERY
	10'–12'		RED BROWN CLAYEY SILT, LITTLE FINE SAND, TRACE FINE GRAVEL (ML)	10, 14, 18	8	1.5' RECOVERY
	12'–13'		RED BROWN GRAVELLY SAND, LITTLE SILT (SW)	9, 10, 20	9	1.0' RECOVERY
	14'		SAME	18	10	1.2' RECOVERY

ENG FORM 1836 (CADD Facsimilie) 1DC-48.dwg | PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. DC-48

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 2 OF 3 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			RED/BROWN SANDY GRAVEL, LITTLE SILT (SW)	27	10	
				38		
	15'		SAME	12	11	0.8' RECOVERY
				28		
	16'			38		
			SAME	18	12	0.7' RECOVERY
	17'			12		
				12		
	18'		SAME	20	13	1.0' RECOVERY
				14		
	19'			5		
			BROWN SILTY FINE SAND (SM)	11	14	1.1' RECOVERY
	20'			11		
				11		
	21		SAME			
	22'					
			SAME	12	15	0.0' RECOVERY
	23'			14		
				25		
	24'		BROWN SITLY FINE SAND, LITTLE FINE GRAVEL (SM)	32	16	1.5' RECOVERY
				22		
	25'			21		
			SAME	6	17	1.5' RECOVERY
	26'			14		
				16		
	27'		SAME	7	18	1.5' RECOVERY
				12		
	28'			14		
			SAME	9	19	1.5' RECOVERY
	29'			11		
				11		
	30'		SAME	6	20	1.5' RECOVERY
				8		
	31'			9		
			BROWN SILT AND SAND, LITTLE FINE GRAVEL (ML-SM)	10	21	1.5' RECOVERY
	32'			12		
	33'			14		

ENG FORM 1836A
(CADD Facsimilie)

2DC-48.dwg

PROJECT PASSAIC RIVER FLOOD PREVENTION, MAIN TUNNEL | HOLE NO. DC-48

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 3 OF 3 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	34'		SAME	15 15 15	22	1.5' RECOVERY
	35'		SAME	13 5 18	23	0.8' RECOVERY
	36' 37'		SAME	25 21 11	24	1.0' RECOVERY
	38'		SAME	12 14 36	25	1.3' RECOVERY
	39' 40'		SAME	29 12 16	26	0.8' RECOVERY
	41'		BROWN SANDY SILT, LITTLE FINE TO MEDIUM GRAVEL (ML)	23 44 23	27	0.7' RECOVERY
	42' 43'		SAME	25 14 12	28	1.2' RECOVERY
	44'		SAME	17 24 20	29	1.0' RECOVERY
	45'		SAME TOP OF ROCK = 45.5'	60	30	0.5' RECOVERY
	46' 47' 48' 49' 50' 51' 52'		RED/BROWN WEATHERED SANDSTONE, FRACTURED ROCK CORE DRILLING TO 401.1'			

ENG FORM 1836A (CADD Facsimilie)

3DC-48.dwg

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL

HOLE NO. DC-48

Hole No. C-102

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 1 OF 5 SHEETS

1. PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL	10. SIZE AND TYPE OF BIT 2" SPT
2. LOCATION (Coordinates or Station) DEY ROAD, WAYNE, NJ	11. DATUM FOR ELEVATION SHOWN (TBM or MSL)
3. DRILLING AGENCY USACE – MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 314
4. HOLE NO. (As shown on drawing title and file number) C-102	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED: — UNDISTURBED:
5. NAME OF DRILLER JAMES KNOX	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ________ DEGREES FROM VERTICAL	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 86.7	16. DATE HOLE — STARTED 3-2-94 — COMPLETED 3-17-94
8. DEPTH DRILLED INTO ROCK 266.4	17. ELEVATION TOP OF HOLE
9. TOTAL DEPTH OF HOLE 353.1	18. TOTAL CORE RECOVERY FOR BORING 99%
	19. SIGNATURE OF INSPECTOR ROBERT BROWN

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS e.	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
	0'–1.5'		BROWN SANDY GRAVEL WITH ROOTS –FILL (GM)	4, 1, 2	1	0.3' RECOVERY
	1.5'–3'		BROWN SILTY SAND, LITTLE FINE GRAVEL (SM)	1, 1, 5	2	0.8' RECOVERY
	3'–4.5'		BROWN SAND, SOME FINE TO COARSE GRAVEL, TRACE SILT (SW)	29, 18, 15	3	1.5' RECOVERY
	4.5'–6'		SAME	7, 11, 9	4	1.5' RECOVERY
	6'–7.5'		SAME	10, 8, 8	5	0.3' RECOVERY
	7.5'–9'		SAME	5, 5, 8	6	0.5' RECOVERY
	9'–10.5'		SAME	4, 4, 6	7	0.6' RECOVERY
	10.5'–12'		SAME	5, 7, 6	8	1.5' RECOVERY
	12'–13.5'		GRAY CLAYEY SILT, TRACE OF FINE SAND (ML)	11, 11, 11	9	0.7' RECOVERY
	13.5'–14'		SAME	14	10	1.5' RECOVERY

ENG FORM 1836 (CADD Facsimilie) 1C-102.dwg

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL — HOLE NO. C-102

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 2 OF 5 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			SAME	15, 21	10	
	15'		SAME	10, 14, 18	11	1.5' RECOVERY
	16'					
	17'		SAME	19, 18, 21	12	1.5' RECOVERY
	18'		SAME	24, 17, 17	13	1.5' RECOVERY
	19'					
	20'		SAME	10, 10, 16	14	0' RECOVERY
	21'		SAME	5, 5, 7	15	1.5' RECOVERY
	22'					
	23'		SAME	8, 11, 10	16	1.5' RECOVERY
	24'		SAME	3, 8, 10	17	1.5' RECOVERY
	25'					
	26'		SAME	8, 10, 10	18	1.5' RECOVERY
	27'		SAME	8, 5, 8	19	1.5' RECOVERY
	28'					
	29'		SAME	6, 6, 5	20	1.5' RECOVERY
	30'		GRAY BROWN SILTY CLAY (CL)	4, 5, 4	21	0.2' RECOVERY
	31'		SAME	7, 4, 5	22	1.5' RECOVERY
	32'					
	33'					

ENG FORM 1836A (CADD Facsimile)

2C-102.dwg

PROJECT PASSAIC RIVER FLOOD PREVENTION, MAIN TUNNEL | HOLE NO. C-102

Hole No. C-102

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 3 OF 5 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			SAME	2		1.5' RECOVERY
	34'			4	23	
				5		
	35'		SAME	4		1.5' RECOVERY
				3	24	
				6		
	36'		SAME	4		1.5' RECOVERY
	37'			4	25	
				5		
	38'		SAME	3		1.5' RECOVERY
				5	26	
				6		
	39'		SAME	2		1.5' RECOVERY
	40'			2	27	
				3		
	41'		SAME	8		1.5' RECOVERY
				7	28	
	42'			6		
			SAME	3		1.5' RECOVERY
	43'			4	29	
				4		
	44'		SAME	3		1.5' RECOVERY
				5	30	
	45'			7		
			SAME	3		1.5' RECOVERY
	46'			5	31	
				6		
	47'		SAME	5		1.5' RECOVERY
				5	32	
	48'			5		
			SAME	3		1.5' RECOVERY
	49'			4	33	
				5		
	50'		SAME WITH GRAVEL	6		1.5' RECOVERY
				6	34	
	51'			12		
			SAME	5		1.5' RECOVERY
	52'			7	35	

ENG FORM 1836A (CADD Facsimilie) 3C-102.dwg

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. C-102

Hole No. C-102

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 4 OF 5 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			BROWN SILTY SAND (SM)	7	35	
	53'		SAME	4, 9, 9	36	1.5' RECOVERY
	54'		SAME	4, 20	37	0.8' RECOVERY
	55'		BROWN SILTY SAND, TRACE GRAVEL (SM)	20		
	56'		SAME	12, 12, 29	38	0.5' RECOVERY
	57'		SAME	25, 21, 28	39	0.5' RECOVERY
	58', 59'		SAME	11, 15, 33	40	0.5' RECOVERY
	60', 61'		SAME	11, 28, 26	41	0.5' RECOVERY
	62'		RED/BROWN SILTY FINE TO COARSE SAND, SOME FINE TO COARSE GRAVEL (SM) (TILL)			
	63'–71'		ROLLER BIT DRILLING 62.5' TO 86.7' NO SAMPLING			

ENG FORM 1836A (CADD Facsimilie) 4C-102.dwg

PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. C-102

Hole No. C-102

DRILLING LOG	DIVISION NORTH ATLANTIC	INSTALLATION NEW YORK DISTRICT	SHEET 5 OF 5 SHEETS

ELEVATION a.	DEPTH b.	LEGEND c.	CLASSIFICATION OF MATERIALS (Description) d.	SPT BLOWS	SAMPLE/ BOX NO. f.	REMARKS (Drilling Time, water loss, etc.) g.
			ROLLER BIT DRILLING TO 86.7'			
	72'		NO SAMPLING			
	73'					
	74'					
	75'					
	76'					
	77'					
	78'					
	79'					
	80'					
	81'					
	82'					
	83'					
	84'					
	85'					
	86'		TOP OF ROCK = 86.7'			
	87'		ROCK CORE DRILLING TO 353.1'			
	88'		RED/BROWN WEATHERED SANDSTONE.			
	89'					
	90'					

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(CADD Facsimilie)

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PROJECT PASSAIC RIVER FLOOD PROTECTION, MAIN TUNNEL | HOLE NO. C-102