Bradley-Noble Geotechnical Services
A Division of The Bradley Group, Inc.
2401 Bristol Court SW, Olympia, WA 98502 PO Box 12267, Olympia, WA 98508-2267
Phone 360-357-7883 FAX 360-754-4240

SOILS INVESTIGATION REPORT
FOR THE EXPANSION OF THE NATURAL BIOPOLYMER FACILITY
PORT OF WILLAPA HARBOR
RAYMOND, WASHINGTON

This report presents the results of our subsurface investigation for the proposed new plant for Natural Biopolymer in the Port of Willapa Harbor facility, 1725 Ocean Avenue, Raymond, Washington. We understand that the new plant will utilize a pre-engineered metal building with a free span width of 65 feet, a planned length of 250 feet, and a wall height of 30 feet. We expect a concrete slab on grade will be used for the floor system. We also understand that processing tanks of various diameters and height will be placed in the building as part of the manufacturing process.

Our purposes in exploring the subsurface soil conditions were to evaluate bearing capacity of the site soils and to address other geotechnical considerations for this project. Work was authorized on behalf of the owner by Mr. Alan Gozart, Principal of Andring & Gozart Architects & Planners, the project's architectural consultants.

SITE CONDITIONS

Surface Conditions

The area of proposed construction is of low relief and nearly level. The surface is comprised of a crushed rock surfacing. At the time of our field work, the area was being used by the port for storage of construction material.

Subsurface Conditions

We explored the subsurface conditions by three test borings at locations indicated on the enclosed site plan. These explorations were made using a continuous-flight, hollow-stem auger to advance the borings and to provide borehole support between sampling intervals. Samples were obtained at standard intervals using a two-inch outside diameter, split-spoon sampler driven by a pin-guided, 140-pound

weight free falling 30 inches. The blows per six-inch interval were recorded. The first six-inch drive interval is allowed for seating the sampler. The blow counts for two six-inch intervals, when combined, yield the Standard Penetration Resistance (N-value) of the soils encountered in the sample interval. The number of blows required to drive the sampler the last 12 inches provides a measure of the relative density of granular soils or the consistency of cohesive soils. The results obtained from the Standard Penetration Test, along with other tests and engineering judgments, were used to develop the recommendations of this report.

The general area of the port has been subjected to fill placement as part of the commercial use of the property. From a discussion with a former employee of the timber company that had used a portion of the site for log storage, we understand that there may be areas of hog fuel fill. We understand that this hog fuel was capped with crushed rock. While we did not encounter pockets of highly organic soil or wood waste, there is a high probability that they do exist under the footprint of the proposed building.

In our explorations, we found the first 20 to 25 feet to be fill soils of varying density and composition. The cap of dense shot rock imported material that forms the surface varied from 5 to 8 feet in thickness. This crushed rock fill was not uniform, and layers of other soils and organic material was incorporated in this fill section. The fill soils under the crushed rock cap also varied in composition and apparent density. These fills were placed over the river bottom soils as part of construction of the dock facility.

Underlying the fill soils, we found soils associated with fluvial deposition. These soils varied in density and composition. All explorations were terminated in the fluvial deposits. Underlying the fluvial deposits in this area is the bedrock unit of dense siltstone and sandstones, the McIntosh Formation. We expect that the bedrock unit would have been encountered with additional drilling of perhaps 10 to 20 feet.

During the last continental ice sheet that penetrated present Washington State, about 14,000 years ago, sea levels were lower. With the increased precipitation, we expect that the volume of water carried by the Willapa river was significantly higher. The channel was cut into the bedrock unit. As the ice sheet melted and sea level rose, the volumes of flowing water decreased and the ability of the river to carry large sediments was reduced. As the river became tidal, deposition of fine grained soils into the former channel occurred. This deposition has continued to today, with fine grained soils being deposited into the river channels.

Ground water levels under the site are influenced by the tidal levels of the adjacent Willapa River. In order to control soils under hydrostatic pressures, we had to maintain a water column in the auger to help keep these soils from flowing into the auger. Even maintaining a water column, drilling below 50 feet encountered soils under hydrostatic pressure. The N-values may have been influenced by the loosening of the soils in the sample interval by material migrating up the auger.

DISCUSSION AND RECOMMENDATIONS

Site Preparation

We expect that little additional grading or filling will be required to develop the building pad. Any additional structural fill soils should be placed as specified in the Earthwork Criteria section of this report.

Foundation Support

We expect that the column footings supporting roof and wall loads will be high. Also, these columns will have to resist the wind loading against the 30-foot high walls. In our opinion, based on the data from the drilled explorations, there is a high risk of long-term settlements for highly loaded foundations. Therefore, we recommend that the columns and processing tanks be supported on piles.

Had soil conditions in all test borings been similar, the use of shallow lightly loaded standard spread footings may have been feasible to use for foundation support. Soils found in boring 3 indicate that excessive total and differential settlements of shallow foundations between soils found in borings 1 & 2 and those of boring 3 would occur. Based on the data obtained from the borings, we can not accurately predict the settlements between elements, but estimate that long-term differential settlements may exceed four inches. Under seismic loadings, the subgrade response of the soils in boring 3 would be significantly different than those of 1 & 2, and differential settlement of the structure if probable, with these settlements having the potential of being in excess of 6 inches.

We recommend the use of driven end-bearing class B pressure treated wood piles as the most economical means of providing foundation support. Predrilling of the pile locations to protect the pile tip from damage from the existing shot rock fill section will be required. All piles are to be banded with a minimum of two bands at the tip and one band at the butt.

Wood pile capacity in end-bearing is determined by cross-sectional area of the pile tip. We recommend that the minimum pile tip specified reflect the load-carring need of the pile in the design. The following is the capacity of the piles:

7-inch tip = 19 tons

8-inch tip = 25 tons

9-inch tip = 31 tons

A minium pile length of 45 should be specified, as we expect development of design capacity based on driving resistance to develop about 40-42 feet below the ground surface. The pile will lose about 1 to 1.5 feet during the "fresh heading". An additional 2 to 3 feet will be lost in the pile helmet of the hammer. The installation of the piles should avoid overdriving once the driving resistance to capacity has developed. Overdriving can damage the pile tip and reduce the capacity of the pile.

If higher capacity piles would be beneficial to the project, we recommend the use of closed-end driven steel pipe piles. We recommend a minimum pile diameter of 12 inches. After driving, the pipe pile is filled with concrete. Pile capacity of 60 to 80 tons each is expected to be developed depending on wall thickness, especially if the piles are driven to the bedrock unit, the McIntosh Formation of sandstones and siltstones.

During the pile installation, debris in the existing fill sections may cause deflection of the pile from the plan location. We recommend that all piles be installed under our supervision to ensure that the piles are driven to the specified bearing soils and so that, if deflection of the piles from plan locations is observed during the driving, we can consult with the structural engineer to determine if the pile is acceptable or would need to be pulled and redriven.

All piles are to be installed by an experienced pile installation contractor and driven using fixed leads. We recommend that the pile hammer be single-action and such that it will develop a minimum of 15,000 foot pounds of energy per blow. Use of the modified Engineering News Formula is acceptable for control and evaluation of the pile capacity.

Uplift capacity is expected to be one-third of the end-bearing capacity. The uplift capacity of the pile is determined in part by the connection between the pile and the pile cap.

Liquefaction of the fill soils and fluvial deposits is not likely other than during a major seismic event. The confining pressure of the crushed rock fill section would limit this to all but the larger seismic events. We expect that the piles will become fixed about 20 feet below the surface. We do not expect liquefaction to occur in the soils below 15 to 20 feet from the present top of the fill surface. Liquefaction is typically a near-surface phenomenon during major seismic events in soils that are fine grained, loose, and saturated.

Resistance to lateral loads can be achieved by either bending moments in the piles or the use of battered

piles. Batters should be limited to a maximum of 3:12 (horizontal to vertical). Piles placed in groups will experience reduction in capacity because of group action. Typical two- to six pile groups usually incur a 12% reduction. Piles in groups should be placed no closer than four pile diameters center to center. The project's structural engineer should verify that, if battered piles are used, the tips of the piles do not approach within four pile diameters of other pile tips to prevent overstressing the bearing soils.

Floor Slabs

If the owner is unwilling to tolerate any settlement or deflection of the floor slabs, then the use of a pile-supported structural slab is recommended. If the owner is willing to tolerate some settlement and deflection, then the existing structural fill section of shot rock or additional structural fill material placed and compacted as specified in the Earthwork Criteria section may be used for support. The loss of the slab during a major seismic event is possible. We recommend that the slab be designed for the anticipated floor loads to the design recommendation of the current edition of the of the Reinforcing Steel Institute's Design Manual.

Seismic Site Period

The project area is in Seismic Zone 3 as shown by Figure 18-2, Seismic Zone Map of the United States, in the 1994 edition of the Uniform Building Code. Actual ground motion during a seismic event is influenced by the distance from the epicenter, the depth to bedrock under the site, and the condition (degree of consolidation and saturation) of the soils supporting the structure. We believe that the underlying soils are prone to spontaneous liquefaction under major seismic loadings. Available geologic information shows a sediment thickness of about 80 feet under the project area. We believe that these native soils are fine grained, loose, and saturated. We recommend that a soil coefficient type S_3 be used for seismic design.

Earthwork Criteria

In areas under structures, paving sections, and

sidewalks, strip all topsoil and organic material. For structural fill in areas under footings and slabs-on-grade, we recommend that all soils be compacted to a minimum density of 90 percent of ASTM D-1557. This includes proofrolling native soils exposed in the bottom of the excavation before placing fill. Materials under the paving section should also be compacted to the minimum density by proofrolling before placing the paving section. This includes proofrolling in-place soils, soils that have been disturbed during construction, and all structural fill materials.

For imported structural fill, we recommend that a clean, six-inch minus, well graded gravel or gravelly sand (classifying as GW or SW as determined by ANSI/ASTM test method D-2487), conforming to APWA specification 9-03.14 for gravel borrow, be used. We also recommend that no more than 7% by weight pass the number 200 screen as tested by ANSI/ASTM D-1140 test procedure. Other material may be used after the review and written approval by the soils engineer or engineering geologist.

All fill should be placed in uniform horizontal lifts of six- to eight- inch loose thickness. Each lift should be conditioned to the optimum moisture content and compacted to the specified minimum density before placing the next lift. We further recommend that all utility trench backfill be compacted as specified above. Earthwork should be performed under the continuous supervision and testing of Bradley-Noble Geotechnical Services to ensure compliance with the compaction requirements.

Placement of fill sections on slopes greater that 4:1 (horizontal to vertical) will be benched as directed into the native soils. Height and width of the bench will be determined in the field by the soils engineer or engineering geologist.

Unrestricted slopes shall not exceed 2:1 (horizontal to vertical) for fill embankments and cuts that expose native soils. All fill slopes will be rolled. The project's civil engineer is responsible for the protection of the constructed fill slopes from uncollected runoff. We recommend that all

cut-and-fill slopes be seeded as soon as possible after construction, so that vegetation can protect the slopes from sheet washing.

No fill is to be placed during periods of unfavorable weather or while the fill is frozen or thawing. When work is stopped by rain, placement of fill will not resume until the soils engineer or engineering geologist determines that the moisture content is suitable for compactive effort and that the previously placed fill has not been loosened. The contractor will take appropriate measures during unfavorable weather to protect the fill already placed. Measures that may be required include limiting wheeled traffic and grading to provide temporary drainage of the fill. At the direction of the soils engineer or engineering geologist, the contractor will be responsible for the removal and reworking of fill that has softened or has less than the required compaction.

LIMITS OF LIABILITY

BRADLEY-NOBLE GEOTECHNICAL SERVICES is responsible for the opinions and conclusions contained in this report. These are based on the data relating only to the specific project and locations discussed herein. This report was prepared within the standard and accepted practices of our industry. In the event conclusions and recommendations based on these data are made by others, such conclusions and recommendations are not the responsibility of the soils engineer or engineering geologist unless he has been given an opportunity to review them and concurs in such conclusions or recommendations in writing.

The analysis and recommendations submitted in this report are based upon the data obtained in the explorations at the locations indicated on the attached plan. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations between explorations may not become evident until construction is underway.

We recommend that Bradley-Noble be given the opportunity to review the final plans and

specifications for soils work. This is to verify that our geotechnical engineering recommendations have been correctly interpreted and implemented in the final design and specifications.

We also recommend that we be retained to provide geotechnical services during the foundation construction and trenching. These services would include review of backfill operations, excavations, and other geotechnical considerations that may arise during construction. We would observe compliance with the design concept and project specifications. If the subsurface conditions differ from those anticipated in our explorations, we would also evaluate changes in construction specifications.

BRADLEY-NOBLE GEOTECHNICAL SERVICES

Report prepared by:

David C. Strong
Engineering Geologist

Please see attached soil engineer's review letter.

10 August 1997

MC SQUARED
INCORPORATED

1235 EAST 4TH AVE
SUITE 101
OLYMPIA, WA 98506
(360) 754-9339
FAX (360) 352-2044

August 11, 1997

Mr. David Strong
Bradley Noble Geotechnical Services
P O Box 12267
Olympia, WA 98508-2267

RE: SOILS INVESTIGATION REPORT
EXPANSION OF THE NATURAL BIOPOLYMER FACILITY
PORT OF WILLAPA HARBOR
RAYMOND, WASHINGTON

Dear David:

We have reviewed the attached nine page soils report entitled "Soils Investigation Report for the Expansion of the Natural Biopolymer Facility, Port of Willapa Harbor, Raymond, Washington."

This report was prepared by you based on field work performed by you and your firm. MC Squared, Incorporated has reviewed this report and the conclusions contained therein. It is our opinion that the report and the conclusions meet the standards of good geotechnical practice for this place and time.

MC Squared's recommendations and opinion are based on our review of the field information and soils logs presented to us. MC Squared has made recommendations to you regarding allowing bearing stresses, lateral resistance and loads, site period, and other items of an engineering nature. These recommendations and conclusions have been included in the attached report.

If you have any questions, or I may be of further help, please call me at (360) 754-9339.

Sincerely yours,
MC SQUARED, INC.

VINCE MCCLURE, PE
Principal

JOSEPH V. McCLURE
STATE OF WASHINGTON
15227
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 08/17/ 1999

GENERAL NOTES
FOR SOIL INVESTIGATIONS CONDUCTED BY AUGERED BORINGS

SAMPLE IDENTIFICATION

All sample classifcations are reviewed by a soils engineer in accordance with the Unified Soil Classification System ASTM D-2487. Field soil classifications are in accordance with ASTM D-2488.

SOIL PROPERTY SYMBOLS

Dd: Dry density, pcf
PL: Plastic limit
q_p: Penetrometer value, tsf
q_u: Unconfined compressive strength, tsf
LL: Liquid limit
W : Moisture content
q_s: Vane-shear strength, tsf
PI: Plasticity index

N: Penetration resistance per foot or fraction thereof, of standard 2-inch O.D., 1.3-inch I.D., split-spoon sampler driven with a 140-pound weight free falling 30 inches, in accordance with Standard Penetration Test Specifications ASTM D-1586
__: Apparent ground water level at the time noted after completion

SOIL STRENGTH CHARACTERISTICS

COHESIVE SOILS

Comparative Consistency	Blows/Foot	Unconfined Compressive Strength (tsf)
Very soft	0 - 2	0 - 0.25
Soft	2 - 4	0.25 - 0.50
Medium	4 - 8	0.50 - 1.00
Stiff	8 - 15	1.00 - 2.00
Very stiff	15 - 30	2.00 - 4.00
Hard	30+	4.00+

NON-COHESIVE (GRANULAR) SOILS

Relative Density	Blows/Foot (N-Value)
Very loose	0 - 4
Loose	4 - 10
Firm	10 - 30
Dense	30 - 50
Very dense	50+

DRILLING AND SAMPLING SYMBOLS

SS: Split spoon
AU: Auger sample
WS: Washed sample
TC: Tri-core drilling
ST: Shelby tube - 3" O.D. (except where noted otherwise)
DB: Diamond bit core
CB: Carbide bit core
RL: Ring-lined sampler

Natural Biopolymer Expansion

JOB #: 97.07-02
DATE: 7/7/97
TESTING BY: D. Strong

Boring 1

0 — Parking lot surface of crushed rock

Blows/6"drive using a 2.0" O.D. split-spoon sampler

Rubble and crushed rock

11-15-15 N=30	Firm to dense mixture of gravel and hog fuel
10-9-7 N-16	Firm dark brown organic-rich gravel over gray medium sand
3-4-7 N=11	Firm gray-green gravelly sand over dark brown organic silt
3-4-10 N=14	Firm gray gravelly sand over dark brown silt with wood fragments
1-8-12 N=20	Firm dark brown organic silt over olive green micaceous silt
2-4-4 N=8	Loose dark brown and light green silts
4-10-15 N=25	Firm gray-green silts, dark brown silts with some granules, massively to finely bedded
8-12-14 N=26	Firm gray-green silty fine sands

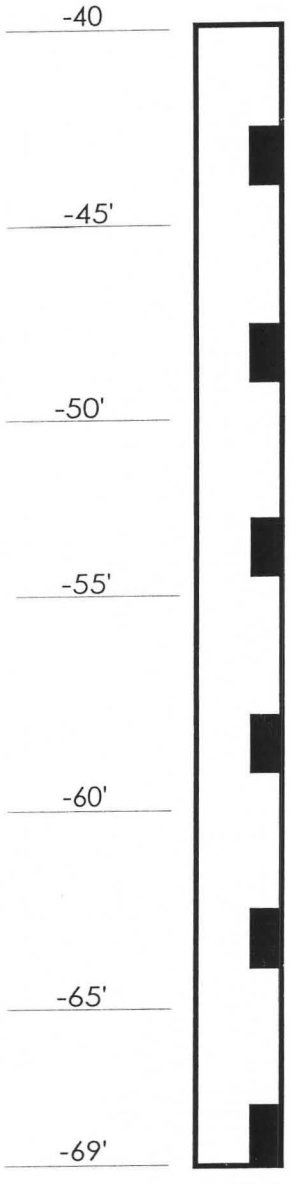
-5' -10' -15' -20' -25' -30' -35' -40'

continued

Natural Biopolymer Expansion

JOB #: 97.07-02
DATE: 7/7/97
TESTING BY: D. Strong

Boring 1, continued



Blows/6"drive using a 2.0" O.D. split-spoon sampler

5-9-9
N=18
Firm gray-green silty fine sands

5-12-16
N=28
Firm dark gray silty sands

Sands moving up auger between sample intervals.
Added water to hold the sands.

4-6-16
N=22
Firm dark gray and gray-green sands

About 5' of sands moving up auger

No sample because hydraulic sands filled lower 3' of auger; gray sands and small gravels recorded from this interval

6-12-18
N=30
Dense gray sands with small gravels

4-12-18
N=30
Dense gray sands with small gravels

Vertical Scale
1.0":5.0'

Bradley-Noble Geotechnical Services

Natural Biopolymer Expansion

JOB #: 97.07-02
DATE: 7/7/97
TESTING BY: D. Strong

Boring 2

0 — Parking lot surface of crushed rock

Blows/6"drive using a 2.0" O.D. split-spoon sampler

Rubble and crushed rock

3-6-12 N=18	Firm 6" minus crushed rock fill
11-12-12 N=24	Firm crushed rock fill over gray silty sand
5-6-6 N=12	Firm gray and green silty sands with small pebbles
8-10-12 N=22	Firm interbedded gray-green silty sands, fine sands and silts
6-12-21 N=33	Dense gray medium sands over dark brown silt with fibrous organics

Filled auger with water to control hydraulic sands

3-4-5 N=9	Loose olive green sands and dark gray micaceous silts
3-10-12 N=22	Firm interbedded silts and silty fine sands
5-12-15 N=27	Firm gray fine sands

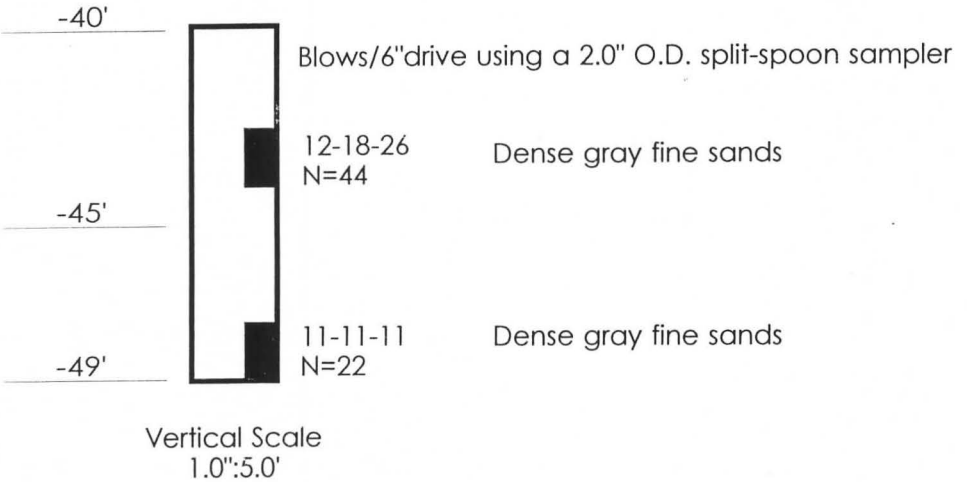
-5' -10' -15' -20' -25' -30' -35' -40'

continued

Natural Biopolymer Expansion

JOB #: 97.07-02
DATE: 7/7/97
TESTING BY: D. Strong

Boring 2, continued



Bradley-Noble Geotechnical Services

Natural Biopolymer Expansion

JOB #: 97.07-02
DATE: 7/7/97
TESTING BY: D. Strong

Boring 3

0		Parking lot surface of crushed rock Blows/6"drive using a 2.0" O.D. split-spoon sampler
	16-50 for 6" N=100+	Very dense 6" minus crushed rock
-5'		
		-7' drilling is through crushed rock
	10-13-15 N=28	Firm gray gravelly silts with organics over reddish brown gravelly sands
-10'		
		Soft layers Filled auger with water to control hydraulic sands
	5-3-8 N=11	Firm saturated reddish brown sand over gray gravelly sand
-15'		
	5-3-3 N=6	Loose gray medium sand with interbeds of dark gray silty fine sands
-20'		
	4-3-2 N=5	Loose reddish brown fine sands over dark brown organic silts
-25'		
	2-1-5 N=6	Loose gray fine to medium sands with layers of gray silts
-30'		
	2-4-8 N=12	Firm light gray fine to medium sand and dark gray silty fine sands
-35'		
	5-10-11 N=21	Firm gray sands and dark gray slightly silty fine sands
-40'		

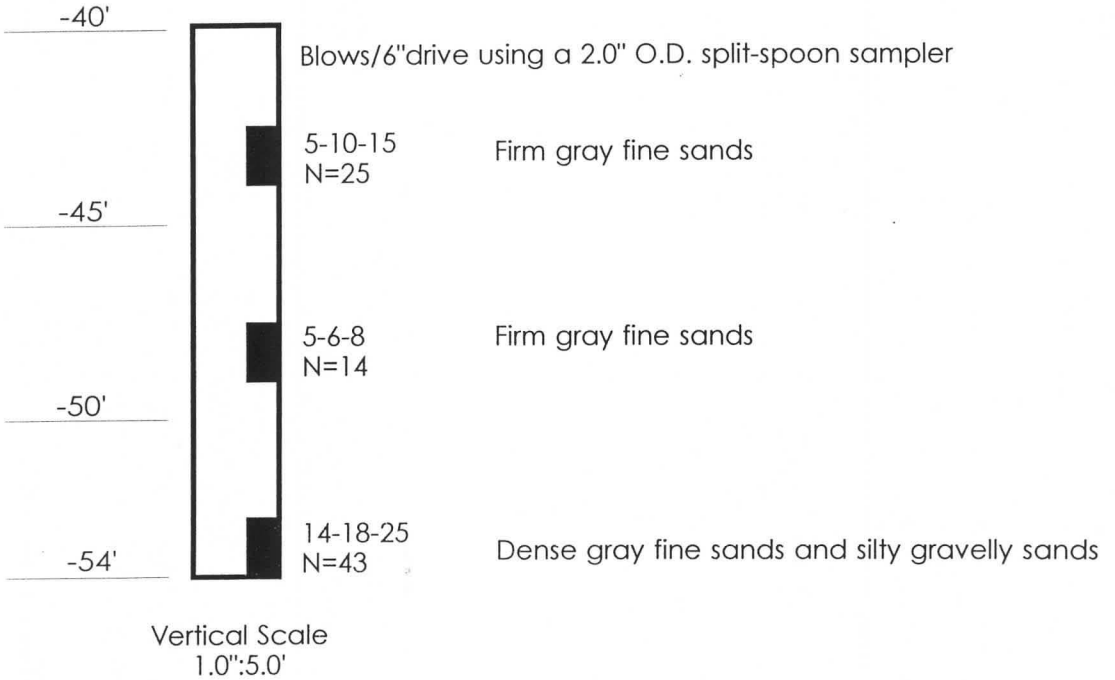
continued

Bradley-Noble Geotechnical Services

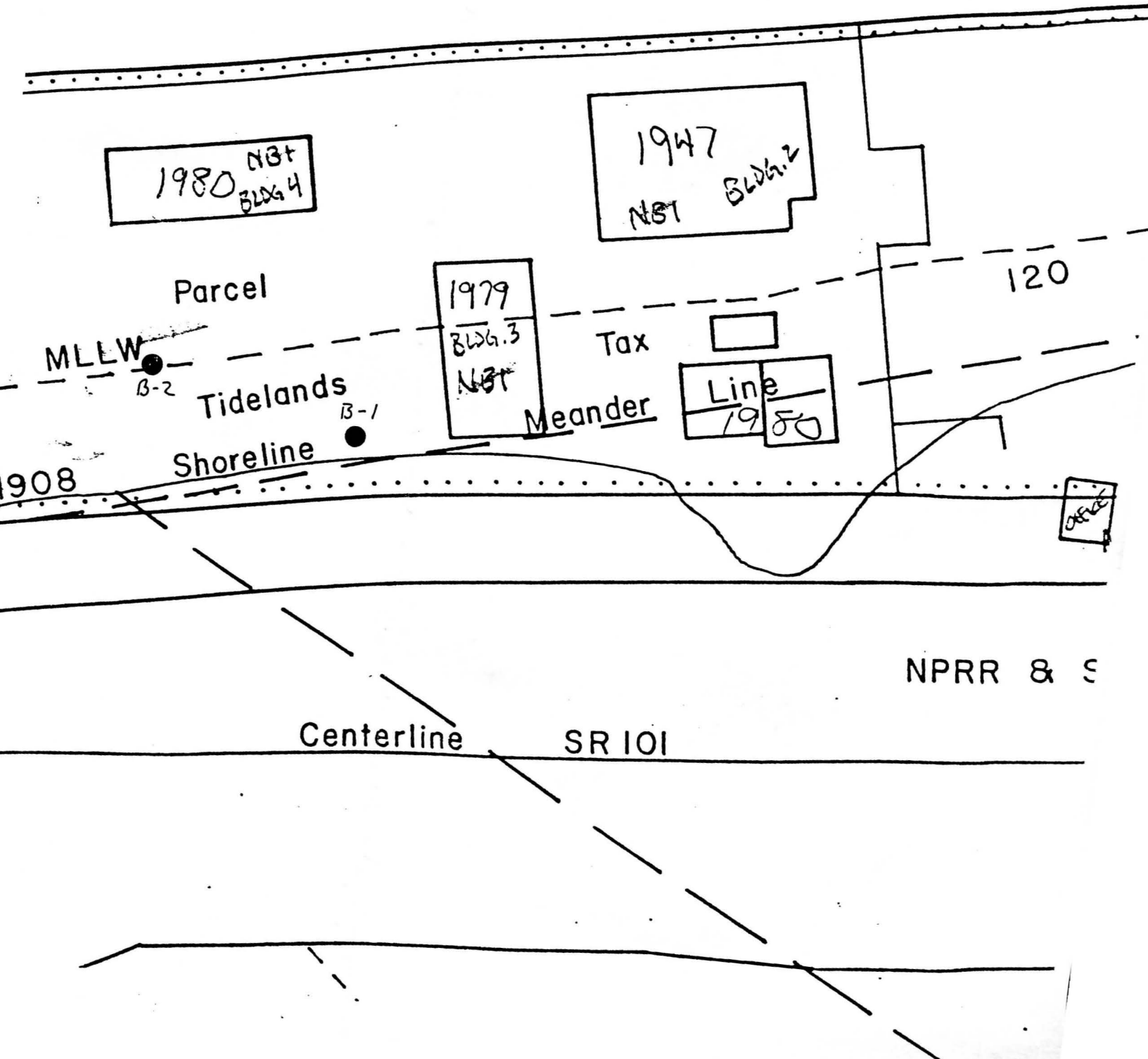
Natural Biopolymer Expansion

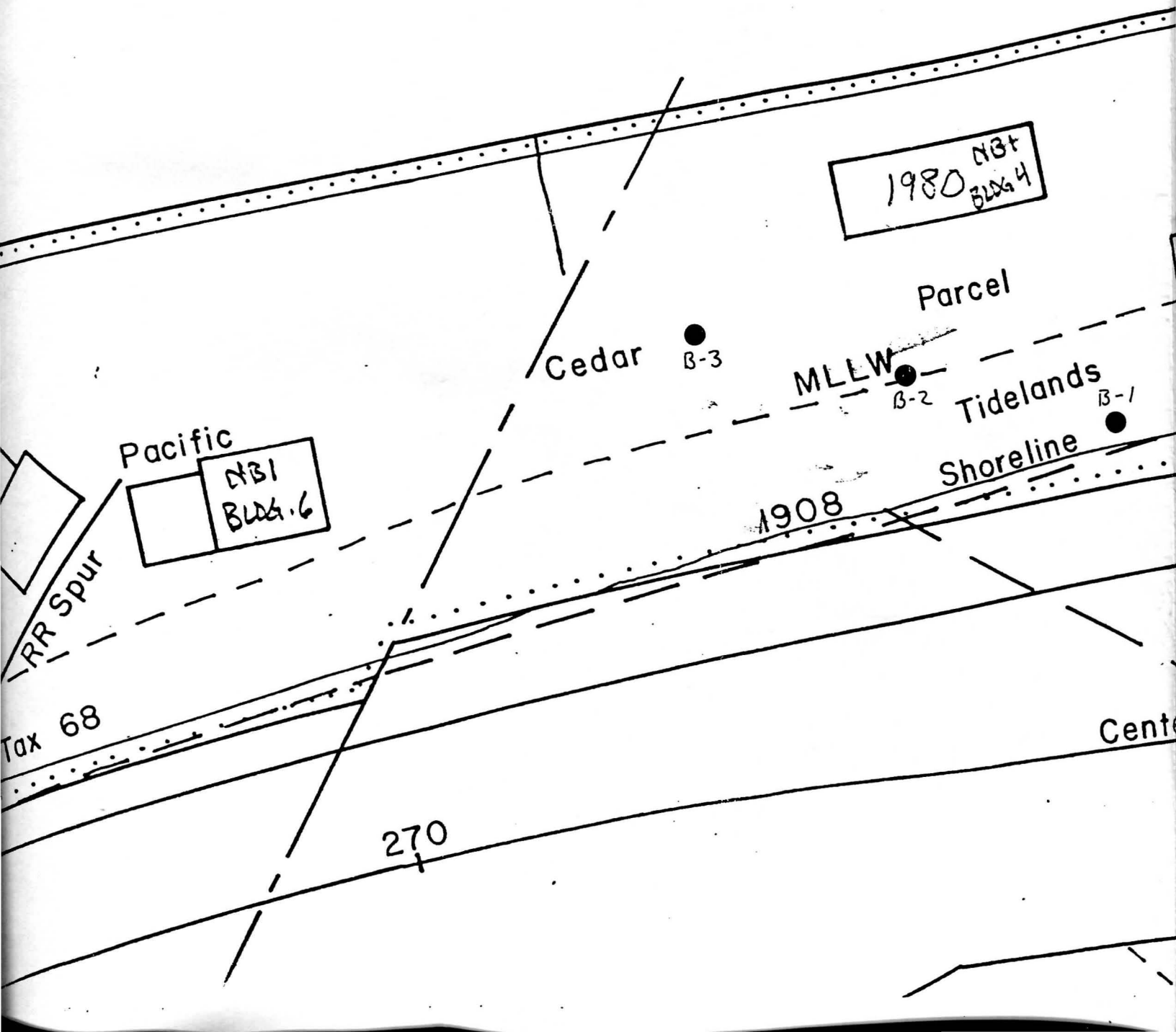
JOB #: 97.07-02
DATE: 7/7/97
TESTING BY: D. Strong

Boring 3, continued

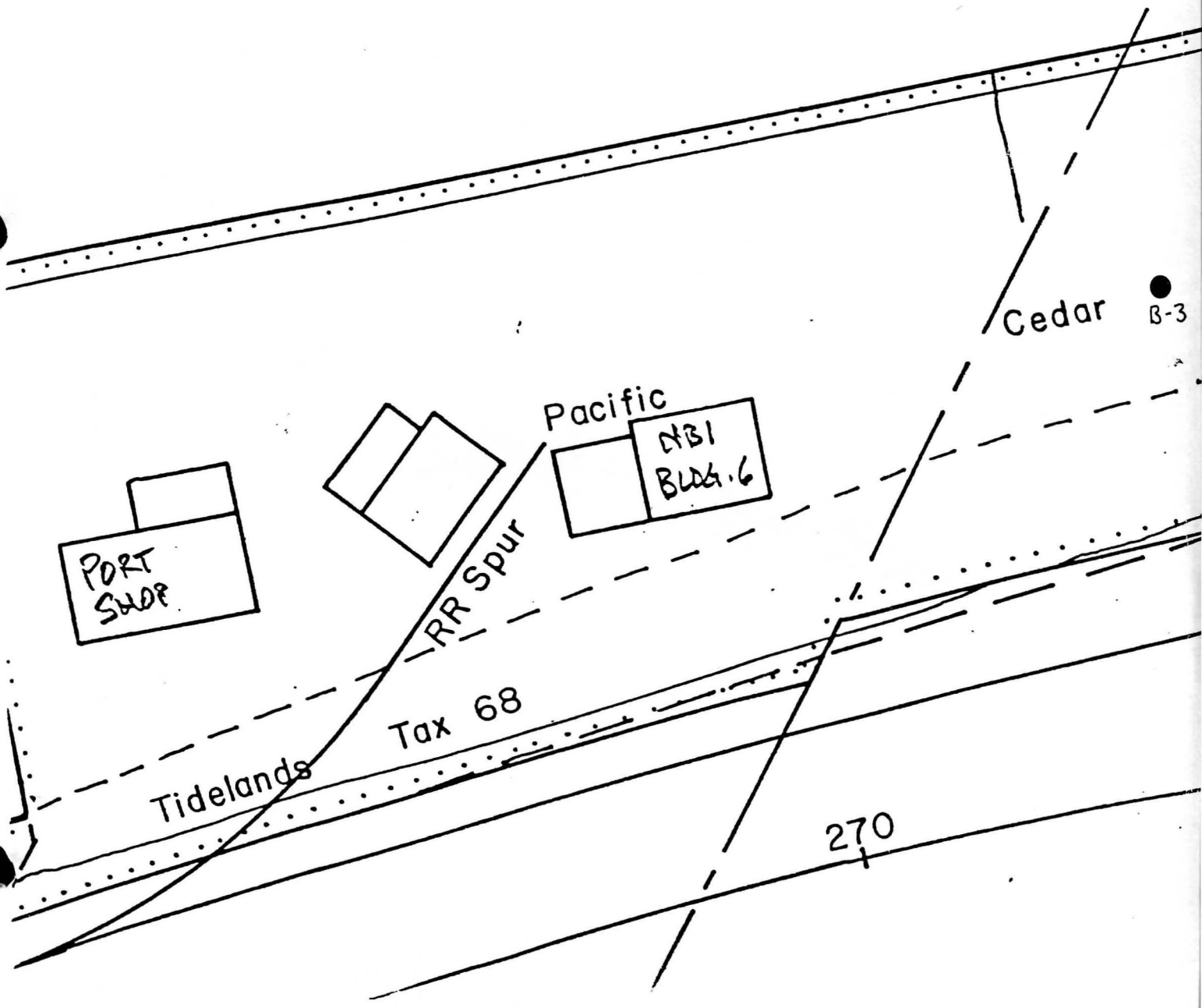


continued


1980 NBt BLDG 4
1947 NBt BLDG 2
Parcel
1979 BLDG 3 NBt
Tax
120
MLLW
B-2
Tidelands
B-1
Meander
Line
1950
Shoreline
1908
NPRR & S
Centerline
SR 101


1980 NBI BLDG 4
Parcel
Cedar
B-3
MLLW
B-2
Tidelands
B-1
Shoreline
Pacific
NBI BLDG. 6
1908
RR Spur
Tax 68
Cente
270

epa


Cedar
B-3
Pacific
NBI
BLDG. 6
PORT
SHOP
RR Spur
Tax 68
Tidelands
270