SOILS STUDIES & OPINION REPORTS

Rollins, Brown and Gunnell	**June 8, 1977**
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ROLLINS, BROWN AND GUNNELL

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SOILS INVESTIGATION

NORFOLK AVENUE
PARK CITY, UTAH

JUNE 1977

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JUNE 8, 1977

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GENTLEMEN:

IN ACCORDANCE WITH YOUR REQUEST, A SOILS INVESTIGATION HAS BEEN COMPLETED AT THE SITE OF THE PROPOSED DEVELOPMENT KNOWN AS NORFOLK AVENUE. THIS INVESTIGATION WAS PERFORMED FOR THE PURPOSE OF DEFINING THE SUBSURFACE SOIL AND ROCK CONDITIONS THROUGHOUT THE PROJECT DEVELOPMENT SO THAT SATISFACTORY SUBSTRUCTURES AND SLOPE PROTECTION COULD BE DESIGNED FOR THE PROPOSED FACILITIES IN THIS AREA.

ACCESS TO THE HILLSIDE ABOVE THE ACTUAL DEVELOPMENT AREA WAS LIMITED BECAUSE OF THE DISTURBING EFFECTS WHICH THE SUBSURFACE INVESTIGATION WOULD HAVE ON THE NATURAL VEGETATIVE GROWTH ON THE HILLSIDE.

THE CONCLUSIONS AND RECOMMENDATIONS PRESENTED IN THE REPORT ARE NECESSARILY BASED UPON THE SUBSURFACE CONDITIONS THROUGHOUT THE DEVELOPMENT SITE. THE RESULTS OF THE INVESTIGATION ALONG WITH PERTINENT RECOMMENDATIONS RELATIVE TO SLOPE STABILITY, FOUNDATION DESIGN AND LATERAL EARTH PRESSURES ARE DISCUSSED IN THE FOLLOWING SECTIONS OF THIS REPORT.

1. SITE GEOLOGY AND THE SUBSURFACE SOIL CONDITIONS

THE CHARACTERISTICS OF THE SUBSURFACE SOILS THROUGHOUT THE DEVELOPMENT AREA WERE INVESTIGATED BY EXCAVATING 11 TEST PITS VARYING IN DEPTH FROM APPROXIMATELY 6 FEET TO 15 FEET BELOW THE EXISTING GROUND SURFACE. THE LOCATION OF THE TEST PITS IS PRESENTED IN FIGURE NO. 1 WHILE THE LOGS ARE PRESENTED IN FIGURES NO. 2 THROUGH NO. 7.

IN MOST OF THE TEST PITS, BEDROCK WAS ENCOUNTERED AT A DEPTH OF BETWEEN 7 AND 8 FEET BELOW GROUND SURFACE. HOWEVER, SOME EXCEPTIONS TO THIS GENERAL PATTERN ARE NOTED. IN TEST BORING NO. 2, THE OVERBURDEN EXTENDED TO A DEPTH OF NEARLY 15 FEET WHILE IN TEST BORING NO. 6, BEDROCK WAS ENCOUNTERED AT A DEPTH OF 2 FEET BELOW THE GROUND SURFACE.

THE SUBSURFACE PROFILE THROUGHOUT THE DEVELOPMENT SITE CAN GENERALLY BE DESCRIBED IN TERMS OF 4 ZONES. ZONE NO. 1 CONSISTS OF A BLACK SILTY TOPSOIL WHICH

extends to a depth of between 1.5 and 3 feet below the existing ground surface. The surface zone is underlain by a granular zone varying in depth from 2 to 7 feet. The granular zone is composed of angular fragments in a matrix of silt. The angular fragments vary all the way from sand-size particles through gravels and cobbles. Zone No. 2 is underlain by a medium plastic clay of variable thickness which extends to the bedrock surface.

At all locations encountered during this investigation, the clay was in a medium to stiff condition. It should be noted that the clay zone is absent in test holes No. 1, 3, 4, 6, 8 and 9. However, the clay exists in a sufficient number of test holes throughout the site that its presence in the area cannot be neglected.

The bedrock underlying the overburden material is known as the Weber Quartzite Formation. In general, the Weber Quartzite Formation is a pale gray to tan quartzite and limey sandstone with some interbedded gray and white limestone and dolemite layers. At the proposed development, the Weber Quartzite Formation exposed in the trenches all showed the bedrock to be a light gray quartzite. From a structural standpoint, the development site is located on the northwest limb of the Park City anticline. The Park City anticline plunges to the northeast with the northwest limb having a stratigraphic dip to the northwest. The strike of the Weber Quartzite varies between North 30° West to North 45° West with a dip between 5° and 20° to the northwest. All joint sets observed throughout the development area essentially have high dip angles into the mountain. One joint set, however, was observed which has a low angle dip toward the mountain.

The results of this investigation indicate that there is no apparent joint set which will cause slippage down the slope of the mountain. However, the high angle joints will cause some fallout on any vertical wall cut perpendicular to the face of the mountain. The joint pattern exposed in the adit above Norfolk Avenue is presented in Figure No. 8. The symbols designating the strike and the dip of the joint sets are separated on the diagram for illustration purposes.

During the excavation of the test pits throughout the development area, in-place density tests were performed at three-foot intervals and miniature vane shear tests were performed in the clay materials. The results of the in-place density tests are presented on the boring logs, and it will be observed that the in-place dry density of the granular material varied from 112 pounds per cubic foot to 119 pounds per cubic foot, while the clay material varied from 93 to 95 pounds per cubic foot.

The miniature vane shear tests provide an indication of the undrained shearing strength of the clay materials. The miniature vane shear tests are designated as the torvane value on the test pit logs and are specified in terms of tons per square foot. The results of the miniature vane shear tests indicate that the subsurface clays are in a medium to stiff condition.

Each sample obtained in the field was subsequently classified in the laboratory according to the Unified Soil Classification System. The symbol designating the soil type according to this system is presented on the boring logs. A description of the Unified Soil Classification System is presented in Figure No. 9 and the full meaning of the various soil symbols can be obtained from this figure.

2. Slope Stability Considerations

Based upon the topographic map furnished our organization, the average slope throughout the development area is approximately 2 horizontal to 1 vertical. In order to perform a rigorous stability analysis at this site, it would be necessary to determine the soil profile of the entire hillside above the development area. Since it was not possible to excavate test pits up the hillside due to environmental considerations, the statements made in this section of the report are of necessity based upon the conditions which exist in the development area.

The results of the geological investigation indicate that there is no tendency for any slide to occur within the rock mass along the face of the slope in this area and that any failure that may occur will take place in the overburden material.

In attempting to obtain an estimate of the stability conditions for the overburden material at the site, two cases have been considered. Case No. 1 assumes an infinite slope with a depth of cohesionless soil equal to approximately 8 feet. Soil parameters obtained during the field and laboratory investigation have been used in the stability analysis for this case. A friction angle of 34° and a saturated unit weight of 133 pounds per cubic foot have been used in the analysis. If the entire mass of granular material above the bedrock is assumed to be saturated with seepage occurring parallel to the slope, the results of our analysis indicate that a factor of safety of 0.70 would occur for this site. Failure conditions would obviously occur under the stipulated conditions.

If the overburden material is less than saturated with no seepage occurring parallel to the slope, the results of the stability analysis indicate a factor of safety of 1.40. It is also apparent that the hillside would be stable under these conditions.

Case No. 2 considers the overburden material to consist of clay having the characteristics of the clay material observed in the lower portion of the soil profile at the site. The results of a stability analysis performed for this condition assuming the clay to be near saturated, but with no seepage parallel to the hillside, indicates a factor of safety of greater than 2. It is apparent from the above considerations that the stability of the overburden material at this location is a sensitive function of seepage parallel to the slope. If seepage conditions parallel to the slope can be restricted, the calculations indicate that the overburden material throughout the area

would be stable. Our study of the entire area throughout the development site indicates that no slides or slumps exist throughout the overburden material and that the hillside is stable under its existing conditions.

In order to insure stability throughout the development area, we recommend that adequate subsurface and surface drainage be provided throughout the development area and that all disturbance of existing shrubs and overburden material be minimized as much as possible. We recommend that an interceptor drain be constructed uphill from the proposed development area to restrict downhill seepage. Such a facility will not only prevent water from flowing into the development area, but it will provide a means whereby water uphill from the proposed development can be readily intercepted and removed from the site.

In constructing roads and houses throughout the area, care should be taken to minimize the disturbance of the existing vegetative cover. We also recommend that adequate lateral support be provided in all areas where the overburden material is under cut.

If the above precautions are taken, it is our opinion that the slopes at this location will remain stable during the development of this site.

3. Foundation Considerations

In accordance with our recommendations outlined above, to minimize the disturbance of the existing materials throughout the development area, we recommend that the structures erected at this site be stepped up the hillside in such a way that the maximum cut at any location does not exceed 10 feet and that all foundations supporting the structures be located on bedrock. The normal cut for the proposed facility would expose bedrock over a portion of the building area; however, piers extending to bedrock may be required at other locations. Allowable soil bearing pressures of 3 to 4 tons would be very conservative for the rock existing at this location.

It is recognized that there may be some areas in which minor structural foundations would be located on the overburden material above the bedrock. In order to provide basic information in which foundations in these areas can be proportioned, bearing capacity recommendations are provided in Table No. 1. In providing the bearing capacity recommendations, it has been assumed that the foundations would be located on the existing slopes and that the depth below the existing ground surface may vary considerably. It is apparent from Table No. 1 that the allowable soil bearing pressures for footings placed on the slope is a function of the width of the footing and the depth at which the footing is placed below the actual ground surface.

In preparing Table No. 1, consideration has also been given to differential settlement. If the proposed facilities are designed in accordance with Table No. 1,

the maximum settlement of any footing will not exceed one inch and differential settlements will not likely exceed one-half inch which should be tolerable for the proposed facility.

4. Excavation Procedures and Lateral Earth Pressures

It is our understanding that Norfolk Avenue will be constructed by widening an existing trail throughout the proposed development area. We recommend that the depth of the cut into the hillside along the roadway alignment be minimized by filling downhill for a portion of the roadway. The natural angle of repose for the quartzite rock to be excavated along the roadway alignment will be approximately 1.5 horizontal to 1 vertical. If the rock excavation is performed in such a manner that the cut and fill is balanced, the dump rock should provide a stable roadway on the downhill side of the cross-section.

Prior to the placement of any rock along the alignment, we recommend that all of the topsoil existing throughout the area be removed to eliminate the possibility of slippage along this plane of weakness. It is also recommended in placement of the rock fill that it be densified by rolling the material with at least 4 passes of a D8 cat or with 5 to 6 passes with a vibratory roller having a 10-ton vibratory force.

We also recommend that lateral restraint be provided for the overburden material located above the bedrock surface.

As indicated earlier in the report, the bedrock throughout the site is competent rock and will stand at a near vertical slope. It is our understanding that the retaining facility to be used in providing the lateral restraint for the overburden materials will consist of 4 by 6 woodpiles imbedded into the rock on the innerside of the roadway alignment and that lagging will be placed between the wood piles to provide the necessary lateral support.

In designing the proposed earth retaining facility, we recommend that an earth pressure coefficient of 0.4 be used to determine the lateral earth pressures. We also recommend that the bedrock surface be inclined slightly in order to provide a more aesthetical and efficient design for the pile sections. It may be necessary to anchor the pile sections at the top of the pile in order to resist the applied moment. This could be performed in a relatively simple manner by extending a cable from the pile supports to the bedrock in the hillside.

5. The Results of Field and Laboratory Tests

A number of field and laboratory tests have been performed during this investigation to define the characteristics of the subsurface material throughout the

area. These tests include: in-place unit weight, natural moisture content, Atterberg limits, mechanical analysis, and unconfined compressive strength. A summary of all test data performed during the investigation is presented in Table No. 2, Summary of Test Data. It will be observed that the unconfined compressive strength of the clay layer underlying the granular material in the soil profile varies from 2738 pounds per square foot to 3435 pounds per square foot.

As indicated earlier in the report, the clay is in a relatively stiff condition and is capable of supporting moderate load intensities.

Atterberg limits performed on the clay material indicate that it classified generally as a CL-2 material according to the Unified Soil Classification System. This means that the material has medium plastic characteristics and may possess some slight swell potential if it is permitted to assorb moisture. This material is sufficiently plastic that it should not be used for any kind of backfilling operations behind retaining facilities.

In areas where the natural material will exist adjacent to earth retaining structures, we recommend that it be excavated and replaced with granular material.

The in-place density of the natural granular material is relatively high, and the strength characteristics of this materialare reasonably good.

The conclusions and recommendations presented in this report are based upon the results of the field and laboratory tests which, in our opinion, define the characteristics of the subsurface material in the development area in a reasonable manner. The characteristics of the overburden material, however, uphill from the development area are unknown.

Please advise us if there are any questions relative to the information contained herein.

Yours truly,

ROLLINS, BROWN AND GUNNELL, INC.

Ralph L Rollins

Ralph L. Rollins

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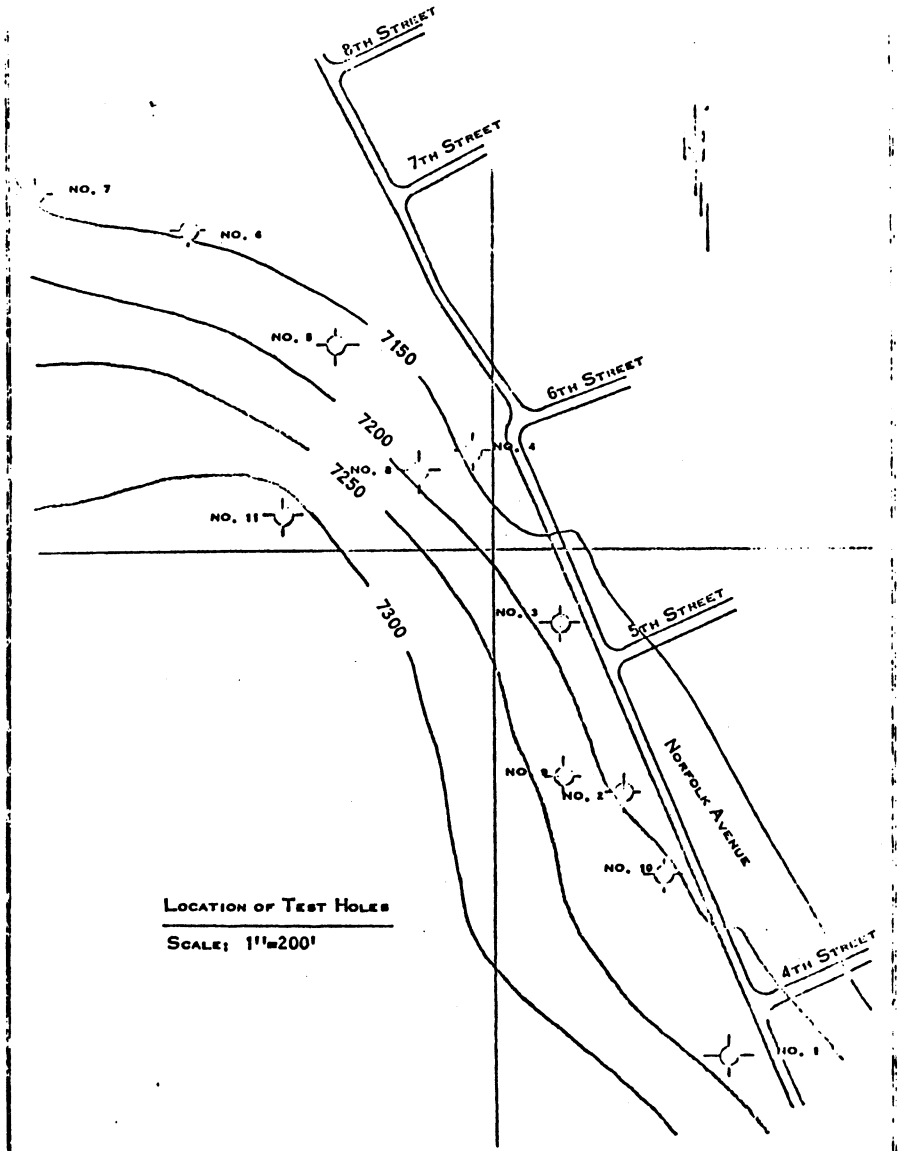
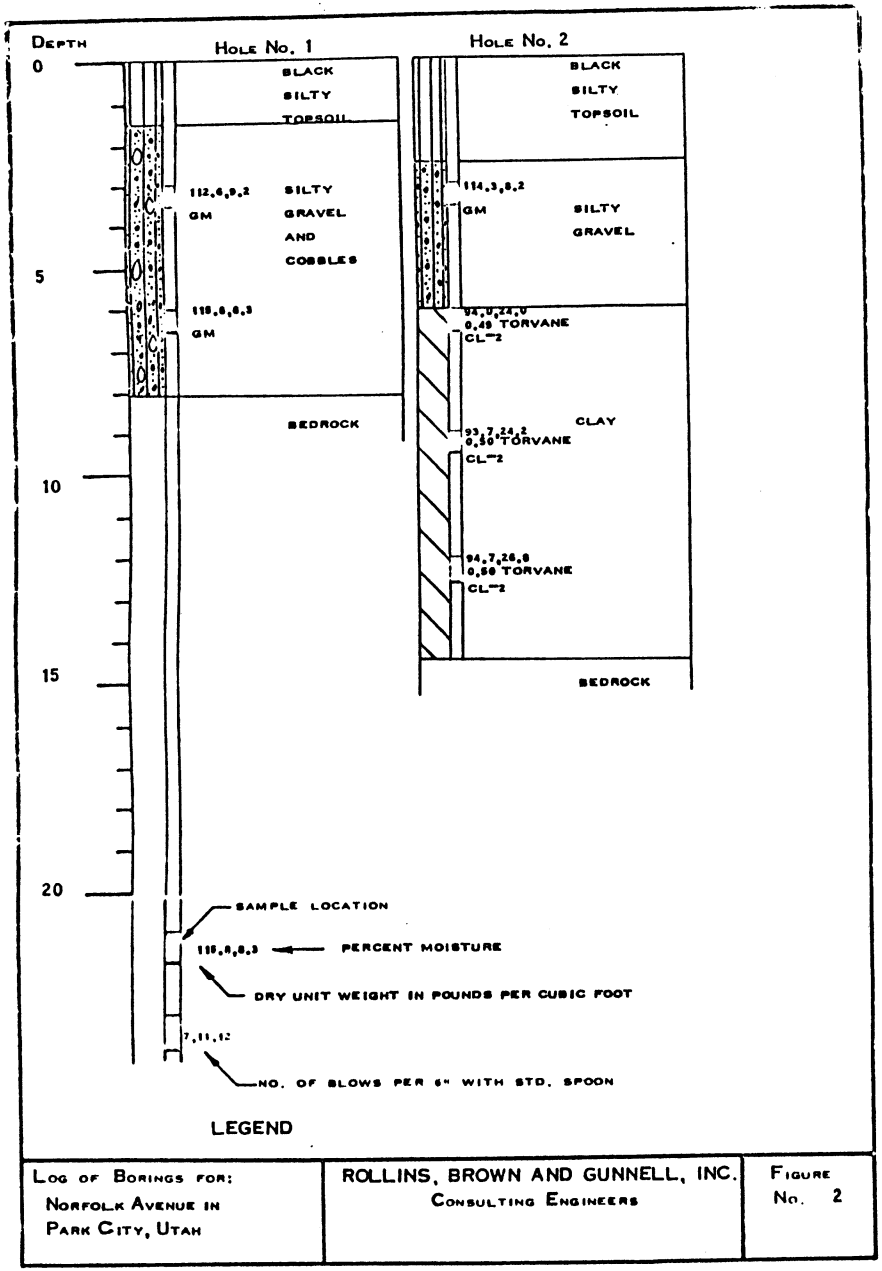
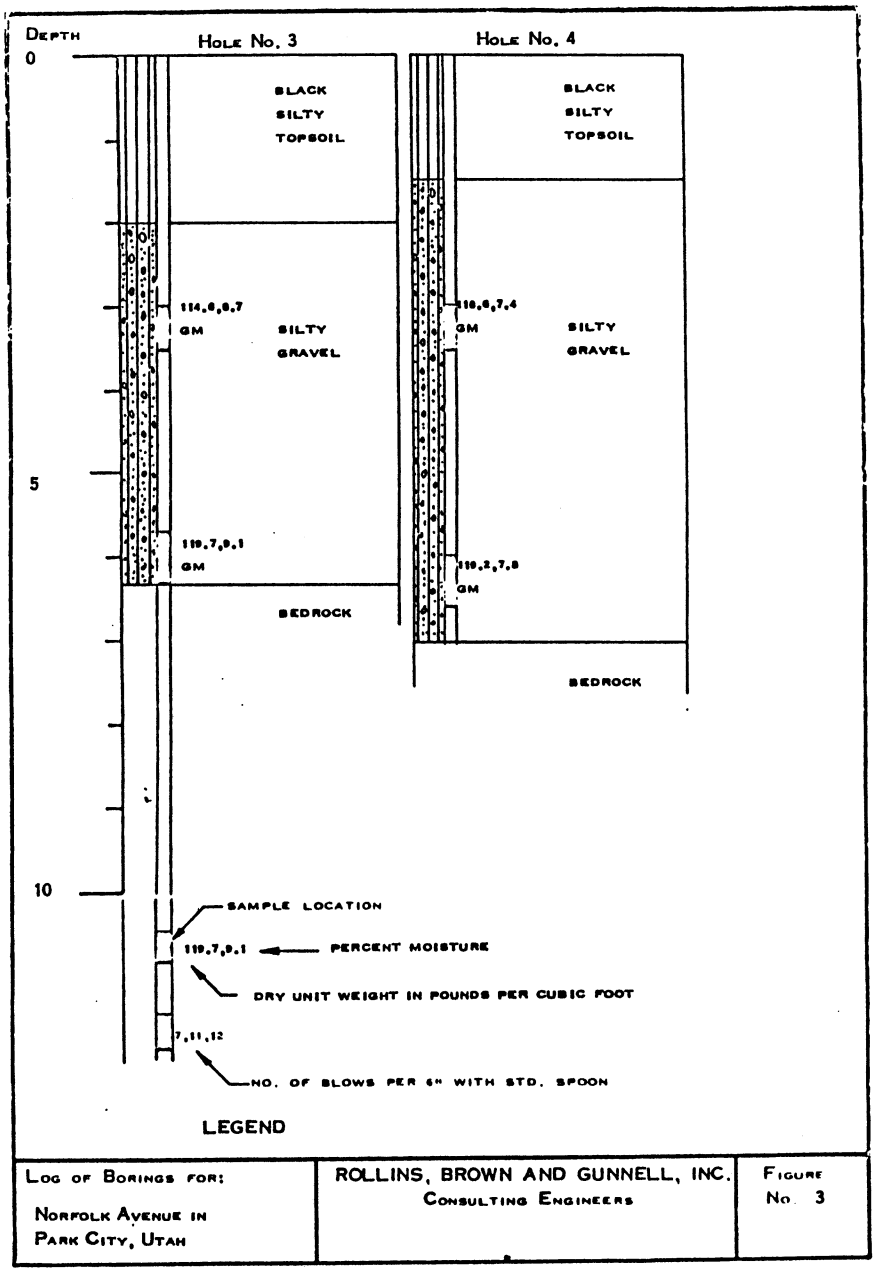


ROLLINS, BROWN & GUNNELL, INC. CONSULTING ENGINEERS	Location of Test Holes For Norfolk Avenue Located In Park City, Utah	Figure No. 1

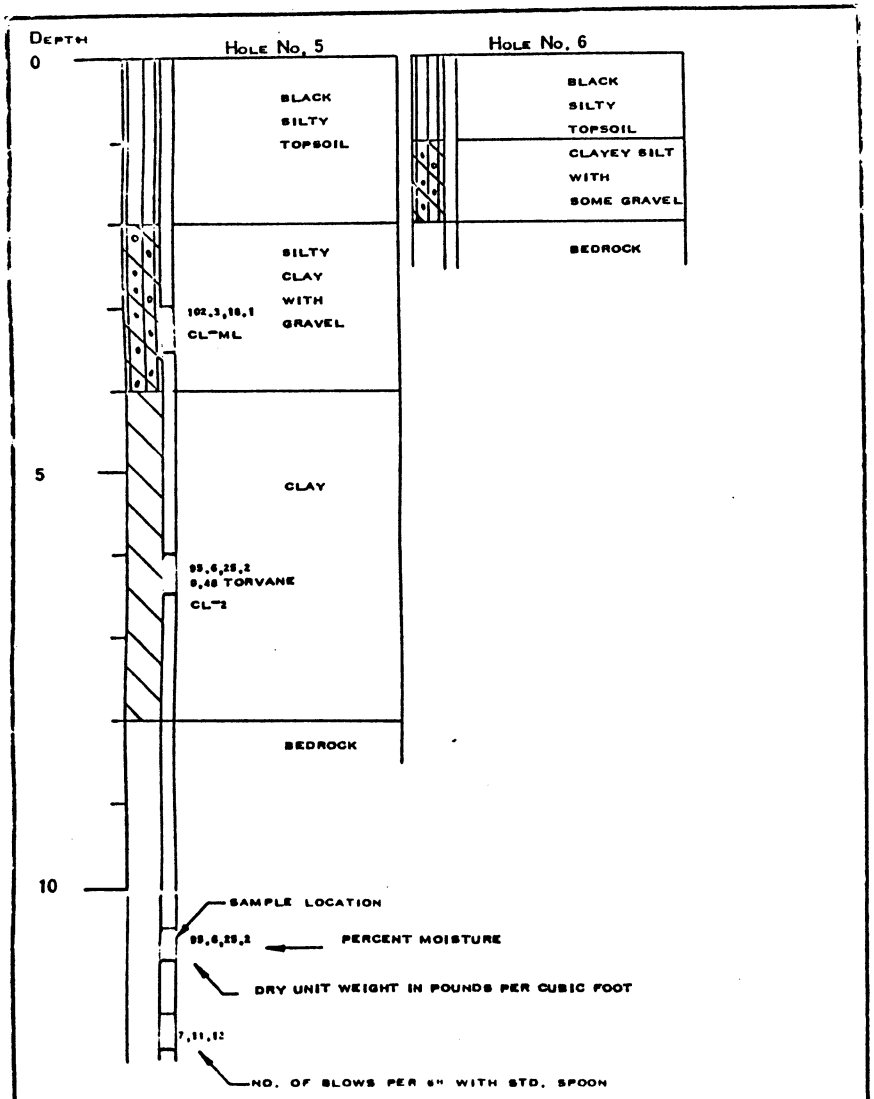
Drawing "500"



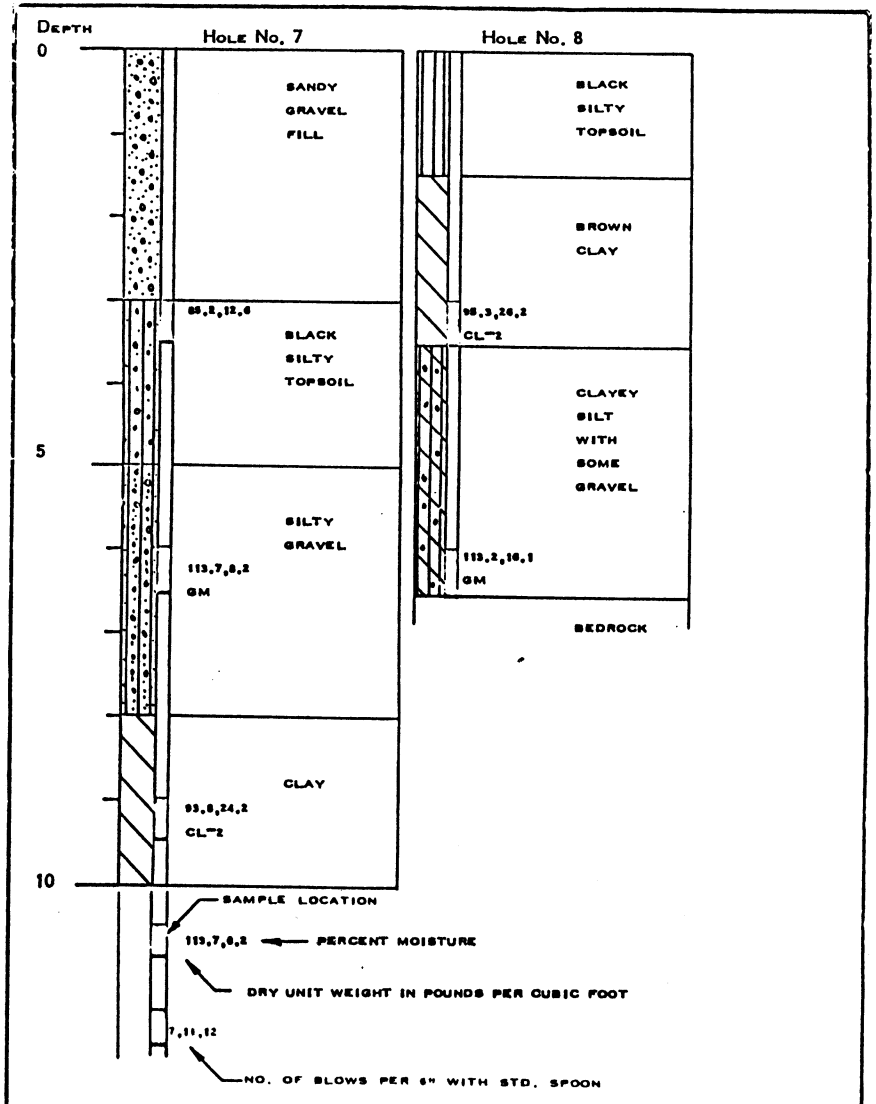
LOG OF BORINGS FOR: NORFOLK AVENUE IN PARK CITY, UTAH	ROLLINS, BROWN AND GUNNELL, INC. CONSULTING ENGINEERS	FIGURE No. 2



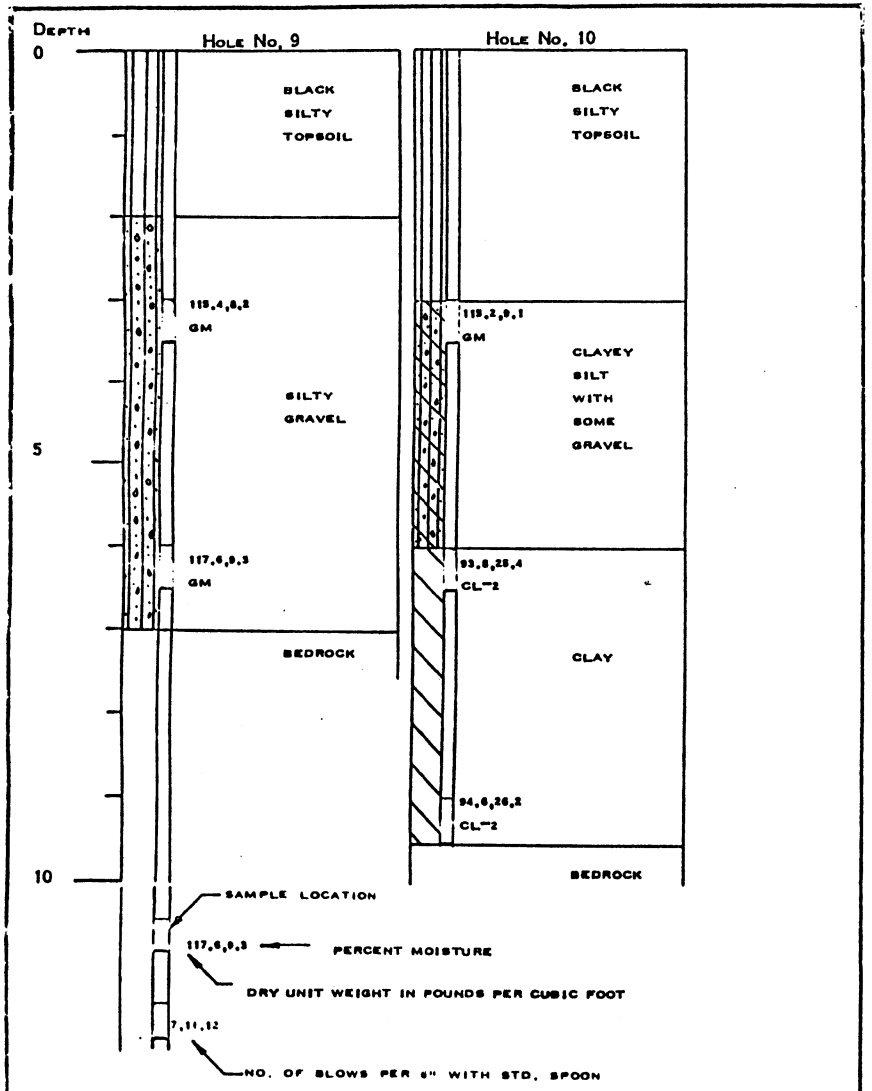
LOG OF BORINGS FOR: NORFOLK AVENUE IN PARK CITY, UTAH	ROLLINS, BROWN AND GUNNELL, INC. CONSULTING ENGINEERS	FIGURE No. 3



LOG OF BORINGS FOR: NORFOLK AVENUE IN PARK CITY, UTAH	ROLLINS, BROWN AND GUNNELL, INC. CONSULTING ENGINEERS	FIGURE NO. 4

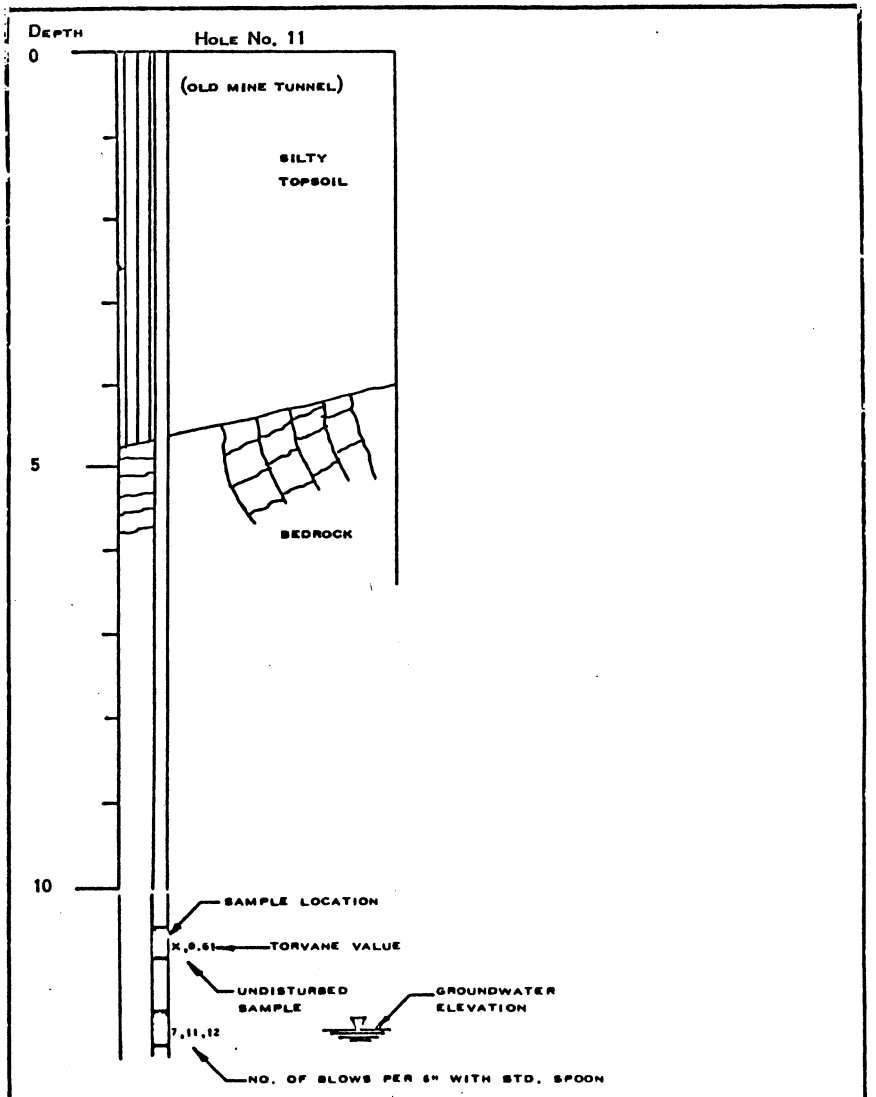


LOG OF BORINGS FOR: NORFOLK AVENUE IN PARK CITY, UTAH	ROLLINS, BROWN AND GUNNELL, INC. CONSULTING ENGINEERS	FIGURE NO. 5



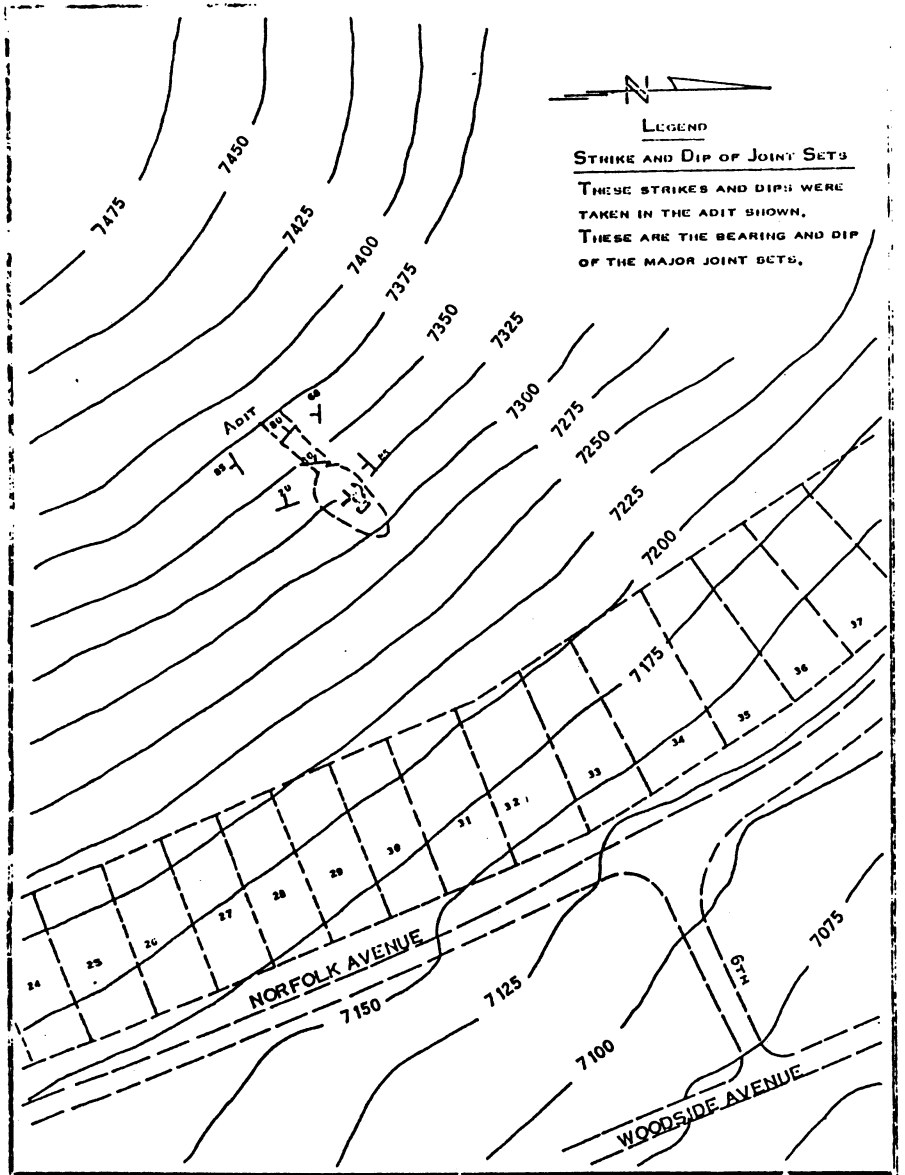
LEGEND

LOG OF BORINGS FOR: NORFOLK AVENUE IN PARK CITY, UTAH	ROLLINS, BROWN AND GUNNELL, INC. CONSULTING ENGINEERS	FIGURE No. 6



LEGEND

LOG OF BORINGS FOR: NORFOLK AVENUE IN PARK CITY, UTAH	ROLLINS, BROWN AND GUNNELL, INC. CONSULTING ENGINEERS	FIGURE No. 7

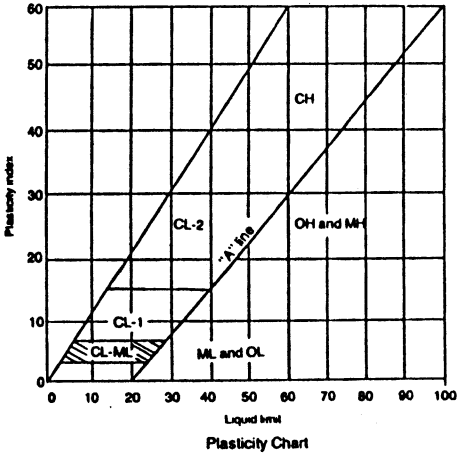


ROLLINS, BROWN & GUNNELL, INC. CONSULTING ENGINEERS	BEARING AND DIP OF JOINT SETS IN WEBER QUARTZIATE ABOVE PROPOSED DEVELOPMENT	FIGURE No. 8

FIGURE NO. 9
Unified Soil Classification System

Major divisions			Group symbols	Typical names	Laboratory classification criteria
Coarse-grained soils (More than half of material is larger than No. 200 sieve size.)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size.)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	$Cu = \frac{D_{60}}{D_{10}}$ greater than 4, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW
		Gravels with fines (Appreciable amount of fines)	GM* (d / u)	Silty gravels, gravel-sand-silt mixtures	Atterberg limits below "A" line or P.I. less than 4 / Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits above "A" line with P.I. greater than 7
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size.)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	$Cu = \frac{D_{60}}{D_{10}}$ greater than 6, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			SP	Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW
		Sands with fines (Appreciable amount of fines)	SM* (d / u)	Silty sands, sand-silt mixtures	Atterberg limits below "A" line or P.I. less than 4 / Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
			SC	Clayey sands, sand-clay mixtures	Atterberg limits above "A" line with P.I. less than 7
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	
			CL (1 / 2)	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
			OL	Organic silts and organic silty clays of low plasticity	
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
			CH	Inorganic clays of high plasticity, fat clays	
			OH	Organic clays of medium to high plasticity, organic silts	
	Highly organic soils		Pt	Peat and other highly organic soils	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:
Less than 5 percent ..... GW, GP, SW, SP
More than 12 percent ..... GM, GC, SM, SC
5 to 12 percent ..... Borderline cases requiring dual symbols**



Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L. L. is 28 or less and the P.I. is 6 or less, the suffix u used when L. L. is greater than 28.
** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.

TABLE NO. 1

RECOMMENDED ALLOWABLE SOIL BEARING PRESSURES FOR SPREAD FOOTINGS ON GRANULAR SLOPES (2 HORIZONTAL TO 1 VERTICAL)

B	D	D/B	Q(PSF)
2	0	0	544
2	2	1	1824
3	0	0	820
3	3	1	2736
4	0	0	1094
4	4	1	3648
5	0	0	1368
5	5	1	4460

B = Width of Footing. Applicable for rectangular or strip footings.

D = Depth of Footing Below Ground Surface

Q = Allowable Soil Bearing Pressure

Note: For D/B ratios between 0 and 1 interpolate allowable soil bearing pressures linearly between 0 and 1.

TABLE 2 SUMMARY OF TEST DATA

PROJECT NORFOLK AVENUE FEATURE FOUNDATIONS LOCATION PARK CITY, UTAH

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRA. BLOWS PER FT.	IN-PLACE UNIT WEIGHT LB./FT³	IN-PLACE MOISTURE PERCENT	IN-PLACE VOID RATIO	UNCONFINED COMPRESSIVE STRENGTH LB/FT²	FRICTION ANGLE φ	CONSISTENCY LIMITS L.L. %	CONSISTENCY LIMITS P.L. %	CONSISTENCY LIMITS P.I. %	MECHANICAL ANALYSIS % GRAVEL	MECHANICAL ANALYSIS % SAND	MECHANICAL ANALYSIS % SILT & CLAY	SOIL CLASSIFICATION UNIFIED SYSTEM
1	3		112.6	9.2							65.0	8.7	26.3	GM
	6		115.8	8.3							58.8	7.0	34.2	GM
2	3		114.3	8.2							82.3	7.0	10.7	GM
	6		94.0	24.0		3435		40.1	19.8	20.3				CL-2
	9		93.7	24.2		3153		40.4	19.3	21.1				CL-2
	12		94.7	26.8		2987		38.5	17.8	20.7				CL-2
3	3		114.6	8.7							64.0	9.5	26.5	GM
	5.5		119.7	9.1							60.0	10.5	29.5	GM
4	3		118.6	7.4							66.2	7.8	26.0	GM
	6		119.2	7.8							62.2	10.8	27.0	GM
5	3		102.3	18.1				23.2	16.8	6.4				CL-ML
	6		95.6	25.2		2997		38.7	17.8	20.9				CL-2

TABLE 2 SUMMARY OF TEST DATA

PROJECT NORFOLK AVENUE FEATURE FOUNDATIONS LOCATION PARK CITY, UTAH

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRA. BLOWS PER FT.	IN-PLACE			UNCONFINED COMPRESSIVE STRENGTH LB/FT²	FRICTION ANGLE Φ	CONSISTENCY LIMITS			MECHANICAL ANALYSIS			SOIL CLASSIFICATION UNIFIED SYSTEM
			UNIT WEIGHT LB./FT³	MOISTURE PERCENT	VOID RATIO			L.L. %	P.L. %	P.I. %	% GRAVEL	% SAND	% SILT & CLAY	
7	3		85.2	12.6				20.3	15.7	4.6				ML
	6		113.7	8.2							58.6	13.9	27.5	GM
	9		93.8	24.2		2919		37.9	17.6	20.3				CL-2
8	3		95.3	26.2		3299		35.6	19.2	16.4				CL-2
	6		113.2	16.1							54.6	10.4	35.0	GM
9	3		115.4	8.2							62.1	11.1	26.8	GM
	6		117.6	9.3							46.2	31.4	22.4	GM
10	3		115.2	9.1							44.8	31.2	24.0	GM
	6		93.8	25.4		2637		37.5	19.4	18.1				CL-2
	9		94.6	26.2		2738		39.6	19.0	20.6				CL-2

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