F-3121.01

FHA FUJITANI HILTS & ASSOCIATES, INC.

GEOTECHNICAL CONSULTANTS

June 19, 2001

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Mr. & Mrs. T. Cummins
11100 SW Riverwood Road
Portland, Oregon 97219

FILE COPY

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PATIO DISTRESS
11100 SW RIVERWOOD ROAD
PORTLAND, OREGON

Dear Mr. & Mrs. Cummins:

In accordance with our letter dated September 18, 2000, we have installed and monitored an inclinometer casing installed on the patio at your residence at 11100 SW Riverwood Road (Vicinity Map, Figure 1). The inclinometer casing was installed to assist in our evaluation of remedial measures to mitigate movements that have been taking place in your patio since at least 1982. This letter presents the results of the inclinometer casing installation and monitoring.

BACKGROUND INFORMATION

Our September 18, 2000 letter also provided a history of the movements and previous studies made to evaluate and mitigate the movements. For continuity, the history is repeated in this letter.

We understand that your residence was constructed in 1957 and that you purchased it in 1975. In 1982, you retained the services of Shannon & Wilson, Inc. (S&W) to inspect cracks that opened up on the rear patio during the winter of 1982. Their observations were summarized in a letter dated March 29, 1982 together with several possible corrective measures. These corrective measures included a perimeter drainage trench, structural retaining wall, or horizontal drains. They also recommended making subsurface explorations and studies and provided a scope and cost in their March 29 letter.

The subsurface explorations recommended by S&W were made in May 1982, and the results of the studies were presented in a letter dated June 14, 1982. Based on their studies, S&W recommended constructing a subsurface drain along the driveway in front of your house to depress the groundwater constructing a subsurface drain along the driveway in front of you house to depress the groundwater levels near the edge of the slope and prevent groundwater levels from rising during severe rainfall. The drain was installed in late October and early November 1982, and discharges into a ravine north of your residence. We understand that there were problems during the construction of the drain, and as a result, the drain does not appear to be functioning as well as it should.

Two years after the installation of the drain, S&W made additional visits to your residence to observe movements that you noticed in the course of monitoring the patio. Based on the 1984 visits, S&W recommended in their letter dated November 30, 1984, that eight selected locations be monitored at 6-month intervals and that the groundwater measurements also be monitored during the winter season. It was also recommended that the patio repair work be postponed for at least two years. We understand that formal monitoring was not made at all of the recommended monitoring points, but that you made measurements of the movements of the patio toward the river and made visual observations of the cracks in the patio.

In May 1985, you retained Cornforth Consultants, Inc. (CCI) to review the movements affecting your patio. Using your measurements, CCI established that the movements of the patio toward the river are averaging 3/8 inch per year, and based on these measurements and their observations, CCI recommended either making no major improvements or relieving the pressure behind the retaining wall by removing all or part of the fill behind the wall. You chose to make no major improvements except sealing the cracks that slowly developed over the years.

Currently, the patio appears to have moved about 6 inches since your first measurements. This coincides with the 3/8 inch per year previously measured by CCI in 1985.

INCLINOMETER INSTALLATION

The inclinometer casing was installed in a borehole drilled at the location shown on the Site Plan, Figure 2. The location and elevation of the borehole were surveyed by Alpha Engineering, who also made a topographic map of your patio area. The boring was drilled to a depth of 35.2 feet below the patio level on October 18, 2000, by Geo-Tech Explorations of Tualatin, Oregon with a truck mounted Mobile B-59 drill rig using rotary drilling methods. After the boring was drilled about 5 feet into basalt bedrock, a 2.75 O.D. inclinometer casing was installed to near the bottom of the boring and grouted in place. A Fujitani Hilts & Associates, Inc., geologist was present throughout the drilling work to collect samples, log the borings and observe the installation of the inclinometer casing.

During the drilling of the boring, samples were obtained at 2-1/2 to 5-foot depth intervals mostly using a standard 2-inch O.D. split-spoon sampler. Standard penetration testing (SPT) was performed in conjunction with the disturbed 2-inch O.D. split-spoon sampling in accordance with ASTM D 1586 to measure in-situ relative density and consistency. The standard penetration test result (N value) is the number of blows required to drive the 2-inch sampler 12 inches with a 140 pound weight falling 30 inches. Two, relatively undisturbed 3 inch O.D. thin wall Shelby tube samples were also obtained in the boring in place of the split-spoon samples. All samples that were recovered from the borings were sealed to retain moisture and returned to our laboratory where they were visually examined in our laboratory to refine the field classifications. Water contents were also determined for all applicable samples.

The summary log of the boring is presented in Figure 3. Soil descriptions and interfaces on the log are interpretive, and actual changes may be gradual. The left-hand portion of the boring log presents groundwater information and gives our interpretation of the soils encountered during the field exploration program. The right-hand, graphic portion of the boring logs shows sample locations, the results of the SPT blow counts, and sample water contents.

INCLINOMETER READINGS

The inclinometer casing has guide grooves machined longitudinally at the four quadrants. To measure deflections of the casing, an inclinometer sensor probe is lowered into the casing along the guide grooves in two perpendicular directions, the A-axis and the B-axis. For this case, the A-axis was oriented in an east-west direction. The sensor measures the inclination (angle) of the casing, and measurements are taken at regular intervals. The inclination measurements are then converted to lateral deviations, and movement of the casing with time is determined by comparing data from the initial and subsequent surveys.

The initial survey of the inclinometer casing was made on October 26, 2000, and subsequent readings were taken on February 20, May 2, and June 6, 2001. The results of the readings are shown plotted on Figures 4 (A-axis) and 5 (B-axis). The initial reading is the vertical line through zero on the movement scale. As can be seen from Figures 4 and 5, the readings do not indicate conclusively that movements occurred over the observation period, although the data may show the beginning of slight movement at a depth of 15 feet. The lack of good data is due, most likely, to the dry winter and spring that we have experienced to date.

SUBSURFACE CONDITIONS

The site area is underlain by materials classified in geologic literature[1] as Quaternary alluvial deposits underlain by the Waverly Heights Basalt, a member of the Columbia River Basalt Formation.

The exploratory boring drilled for this study encountered ½-inch thick red tile overlying 5 inches of concrete forming the patio surface. A void space ranging from about ½- to 1-inch thick was observed immediately underneath the concrete pad. A standard split-spoon sample taken in the silty material at the base of this void at a depth of approximately 0.5 feet dropped under it's own weight to approximately 2.4 feet, and a void space measuring at least 8 inches in thickness centered on a tree root was later observed in this interval. The soil recovered from this first sample is classified as very soft, light brown, medium plasticity, clayey sandy silt. Further sampling indicated this material grades to medium dense, low- to non-plastic, stratified sandy silt to silty sand at about 3 feet, extending to a depth of approximately 9 feet. A layer of gravelly silt/silty gravel was encountered at approximately 7.5 to 8 feet overlying additional silty sand as observed above. Stiff, red, high plasticity clay was encountered at a depth of 9 feet that grades to hard, sandy clay by 9.5 feet and then to medium dense, clayey sand with slight gravel by 10.5 feet, extending to a depth of 11.5 feet. From 11.5 feet, stratified, very dense, light brown silty sand and reddish clayey sandy gravel were encountered to a depth of 15 feet. A boulder was possibly present between 17 and 20 feet (the driller reported drilling alongside this boulder, not through it). Dense, multi-colored, clayey, sandy gravel with estimated maximum clast size of 2 inches was encountered from 20 to 30.5 feet overlying very dense, relatively fresh gray basalt in which the boring was terminated.

Our interpretation of the subsurface conditions below the patio area is shown on Figure 6. The light brown sandy silt/silty sand encountered above 9 foot depth is interpreted as fill material derived from on-site materials, but no positive indications for this interpretation were observed in the samples, and a very large oak tree growing just downslope from the patio suggests this material could also be interpreted as native soil, or at least the bottom portions. The void space at approximately 1.2-foot depth mentioned above appears to have been caused by water migrating horizontally along a lateral root of a small tree standing by the northwest corner of the patio (drilling mud was observed flowing

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[1]Madin, I.P., 1990, *Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon*, State of Oregon, DOGAMI, Open-File Report 0-90-2.

up out of the ground near this tree). A certain amount of settlement and lateral movement of the patio may have also resulted from erosion of this soil.
The red clay encountered at a depth of 9 feet is interpreted as a possible topsoil zone derived from the weathering of colluvium and may possibly indicate a shear zone. The materials from 9 to 20 feet in depth appear to be a graded bed of colluvium or landslide debris (bouldery base grading upwards to sand and then clay), but also show layers of alluvial deposition of light brown sand/silt above 15 feet. The very dense, gray, relatively fresh basalt encountered from 30.5 to 35.2 feet is interpreted as Waverly Heights Basalt.

Groundwater is, most likely perched on top of the basalt and fluctuates with the time of year, being highest in late winter or early spring and lowest in late summer or early fall. Although it is believed that the drain trench installed in 1982 is not functioning well, this drain probably has some affect on the groundwater levels under the patio area. Because of the mud rotary drilling methods used to drill boring B-5, it was not possible to measure groundwater. However, appearance and moisture contents of material in the upper sand/silt unit indicate this material is above the perennial groundwater table. Measurement of the observation well in nearby boring B-4 did not reveal groundwater above the bottom of the well at 11.4 feet. The sample taken retrieved from a depth of 25 feet appeared moist rather than wet, and the 30-foot sample was of low quality, but could possibly have been below the water table based on appearance. Boring B-5 was drilled when the water levels should have been at the lowest point.

ENGINEERING ANALYSES

Stability Analysis

Several alternatives have been previously presented, and several have been discussed with you in the course of this study. These alternatives include a retaining structure design to resist the sliding and tied back into bedrock, removal of the patio fill and placing the patio on a structural deck, or possibly a combination of these two. In order to provide design criteria for these alternatives, the location of the failure (sliding) surface and the strength parameters of the sliding soil would ideally be defined. However, the inclinometer installed in boring B-5 did not positively show an area or zone of movement, and good quality samples could not be obtained for laboratory strength testing. For our analysis, we assumed a failure surface along the red clay encountered at a depth of 9 feet and back calculated an average strength along the sliding surface that is based on a factor of safety of unity (FS=1.0). The calculated strength value was then used to determine design criteria for use in the design of the alternatives so that the factor of safety of the remedial alternative is at least 1.25.

The results of the stability analysis indicates that the average shear strength required along the assumed failure surface is 220 pounds per cubic foot for a factor of safety of 1.0. The stability calculations for the existing conditions are included in the Appendix to this letter report.

Remedial Measures

General - Our stability analyses included a tied-back retaining wall, removal of the fill behind the existing retaining wall and a combination of a retaining wall and removal of the fill. Schematic sketches of these measures are presented by Figure 7. The calculations for the analysis of the remedial measures are included in the Appendix to this letter report. For the tied-back wall and combination tied-back wall, we have assumed that the wall extends below the failure surface a sufficient distance and that the sliding mass on the downhill side will slide away over time. The tie-backs extend into the basalt bedrock a sufficient distance to achieve the required pullout resistance. After they have had a chance to review this report, we will meet with your structural engineer, Miller Consulting Engineers, to discuss these alternative and evaluate others should they have other ideas.

Tied-Back Retaining Wall - A tied back retaining wall replacing the existing retaining wall would consist of steel H-beams placed in pre-drilled holes spaced at about 6- to 8-foot centers that are then filled with concrete. Timber or concrete lagging would be installed between the piles. As an alternative, the H-beams lagging could be replaced with closely spaced drilled, concrete piles with some reinforcing steel. Our analysis indicates that the wall will need to resist a shear force from the sliding mass of 4.0 kips per foot of wall plus the earth pressure at rest from the soil behind the all. For the earth pressure at rest, an equivalent fluid pressure of 60 pounds per cubic foot (pcf) may be assumed.

The H-beams or concrete piles will need to be embedded below the failure surface a sufficient distance to resist failure at the toe of the wall (kickout) assuming that the toe is at the failure surface (material on downhill side allowed to slide downhill). The resistance may be determined from the passive earth pressure against the embedded portion of the wall. The passive earth pressure may be assumed to be an equivalent fluid pressure of 225 pcf, and for the H-beams should be assumed to act against a width of beam equal to twice the width of the concrete filled hole.

The tie-back anchors should be drilled and grouted into the basalt bedrock. Resistance is derived from the bond between the rock and the grout. The required length of anchor may be determined using an allowable bond strength between the grout and basalt of 145 pounds per square inch (psi). After the anchor is installed, it should be tested. At least one of the anchors should be tested to at least 1.5 times the design load, and the remaining anchors should be proof loaded to 1.25 times the design load.

Fill Removal - Our analysis assumes that the fill material is removed to the base of the existing wall as measured on the downhill side. The analysis indicates that the removal of the fill increases the factor of safety from 1.0 to 1.15. In our opinion, the calculated improved factor of safety of 1.15 is not sufficient, and this solution should not be considered acceptable. Instead, a shorter retaining wall could be placed in the same location as the tied-back wall, and this smaller wall may not require tie backs to resist the loading.

Combination Fill Removal and Wall - Our analysis indicates that this lower wall will need to resist a shear force from the sliding mass of 1.6 kips per foot of wall plus the earth pressure at rest from the soil behind the all. Again, the wall will need to penetrate below the failure surface a sufficient distance to prevent kickout. If tie-back anchors are found to be required, they can be designed on the same basis as the tie-back retaining wall.

For the alternative consisting of fill removal and a retaining wall, the patio will need to be reconstructed as a deck. The wall could be used as one of the supports for the deck, but the other supports will need to be bear on undisturbed soil, preferably below the failure surface.

LIMITATIONS

We recommend that close quality control be exercised during the preparation and construction of building foundations. In addition, we recommend that the subgrade preparation and the footing excavations be inspected by a geotechnical engineer. If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes of construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples, or drilling test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra cost.

If you have any questions, please contact me at your convenience.

Sincerely,

FUJITANI HILTS & ASSOCIATES, INC.

By K. Frank Fujitani

K. Frank Fujitani, P.E.
President

REGISTERED PROFESSIONAL ENGINEER
5382
OREGON
JULY 9, 1965
K. FRANK FUJITANI

Expires: 12/31/01

Attachments: Figures 1 - 7
Appendix

cc: Ray Miller, Miller Consulting Engineers

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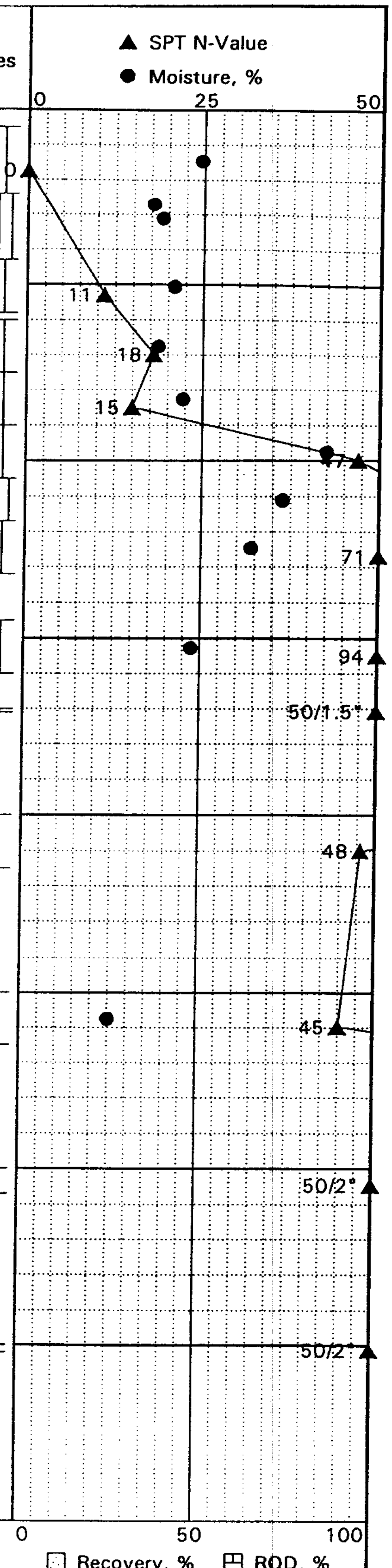
Ground Water	Remarks	Elev. / Depth Feet (100.2)	CLASSIFICATION OF MATERIAL	Depth In Feet	Samples
None Observed		99.7 / 0.5	**FILL**; red tile over 5 inches concrete. (Fill)	0	
			Clayey Sandy SILT grading to stratified Sandy SILT/Silty SAND, very soft to medium dense, medium to light brown, damp to moist, medium plasticity to non-plastic, micaceous, fine-grained, trace to rare sub-angular fine basaltic gravel, void space 0.5 to 1 inch at top of zone, 8-inch void space centered on lateral tree roots at approximately 1.2 feet, trace medium- to coarse-grained sand near top of zone. (Fill)	5	S-1, S-2, S-3, S-4, S-5
		91.2 / 9.0	**CLAY grading to Clayey SAND**, stiff grading hard, mottled red to reddish brown, moist, high plasticity, grades to underlying material. (Possible residual soil, possible shear zone)	10	S-6, S-7
		88.5 / 11.7	**Interstratified Silty SAND and Clayey Sandy GRAVEL**, very dense, light brown, gray and reddish brown, moist, medium plasticity to non-plastic, fine sand/silt of alluvial character interlayered with basaltic gravel of colluvial character, fine to coarse sub-angular gravel. (Basaltic colluvium interlayered with alluvial material)	15	S-8, S-9
		83.2 / 17.0	**3-foot BOULDER**		S-10
		80.2 / 20.0	**Clayey Sandy GRAVEL**, dense, multi-color matrix with gray basaltic gravel, medium plasticity in places, fine to coarse gravel, predominantly angular to sub-angular, trace rounded to sub-rounded. (Colluvium)	20, 25	S-11, S-12
		69.7	Trace fine rounded quartzite gravel.	30	S-13
		30.5	**BASALT**, very dense, gray, relatively fresh angular cuttings; smooth, hard drilling. (Waverly Heights Basalt)		
		65.0 / 35.2	Bottom of Boring, Completed 10/18/00	35	S-14



LEGEND

- Ⅰ = 2.0" O.D. Split Spoon Sample
- ⅠⅠ = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = Grab Sample: Drill Cuttings
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Cummins Residence Patio
Portland, Oregon

LOG OF BORING B-5

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FIG. 3

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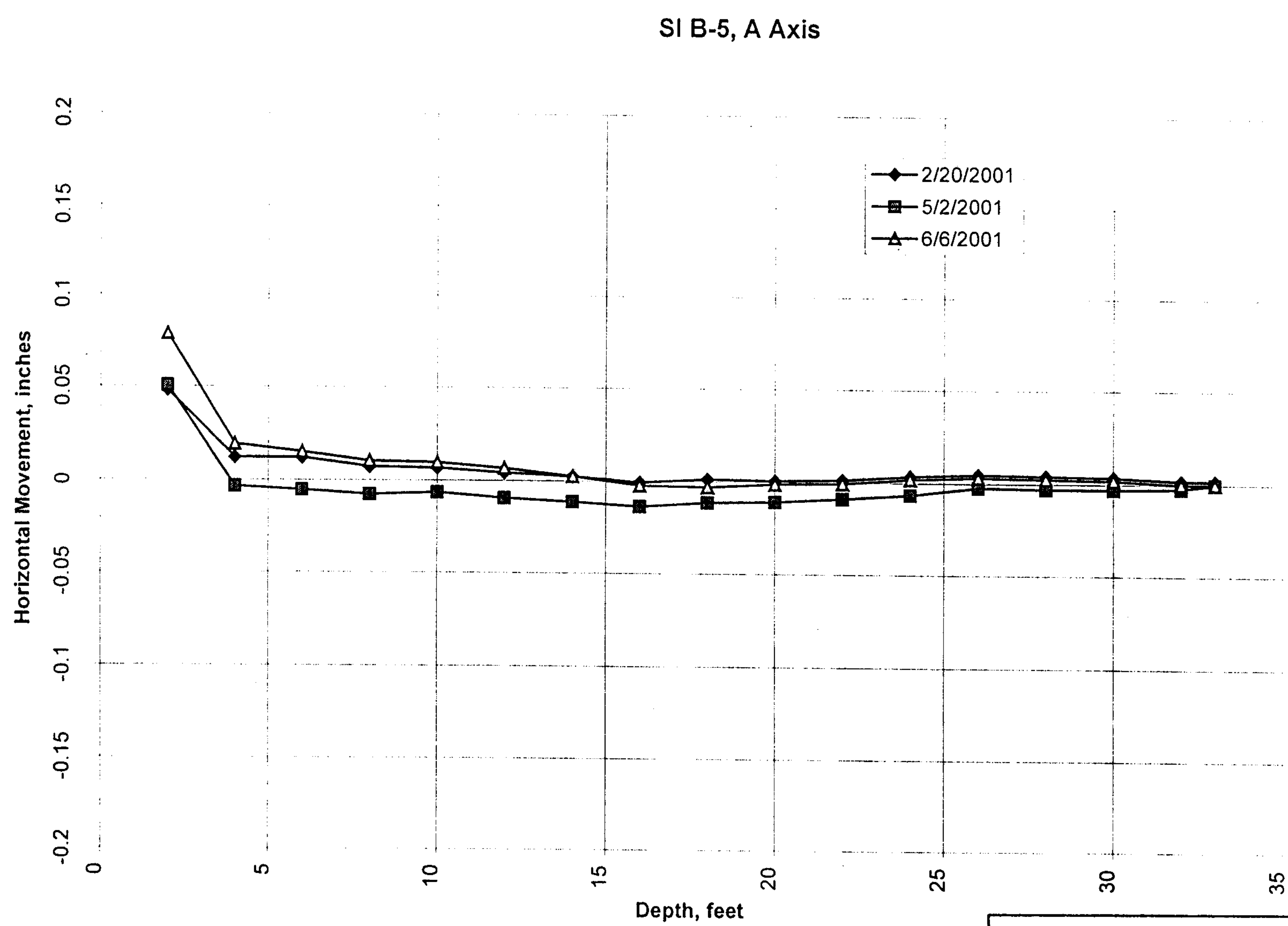
WLG CUMMS 6/19/01



Cummins Residence Patio
Portland, Oregon

INCLINOMETER READING
SI B-5, A AXIS

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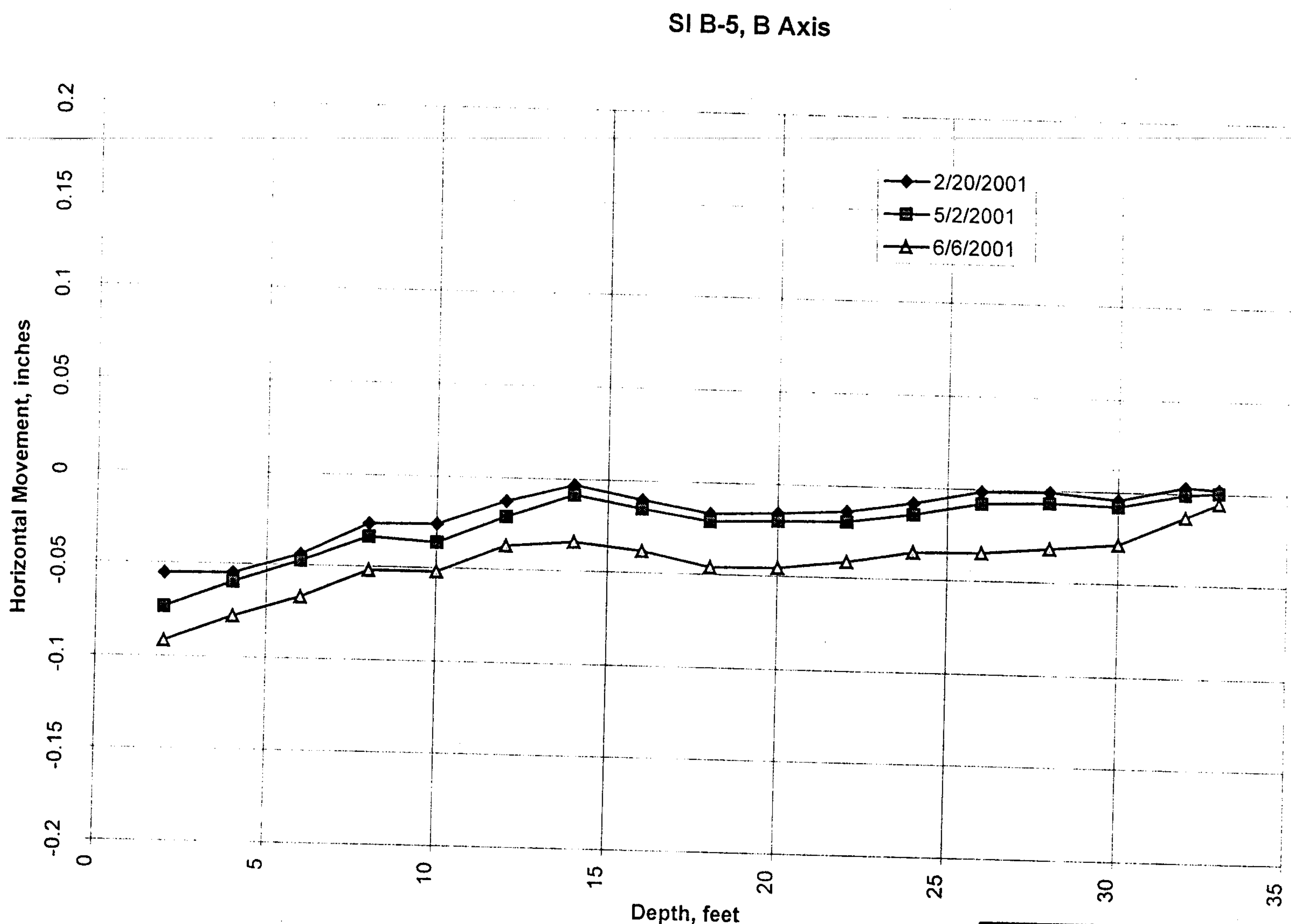
Portland, Oregon

FIG. 4



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INCLINOMETER READING
SI B-5, B AXIS

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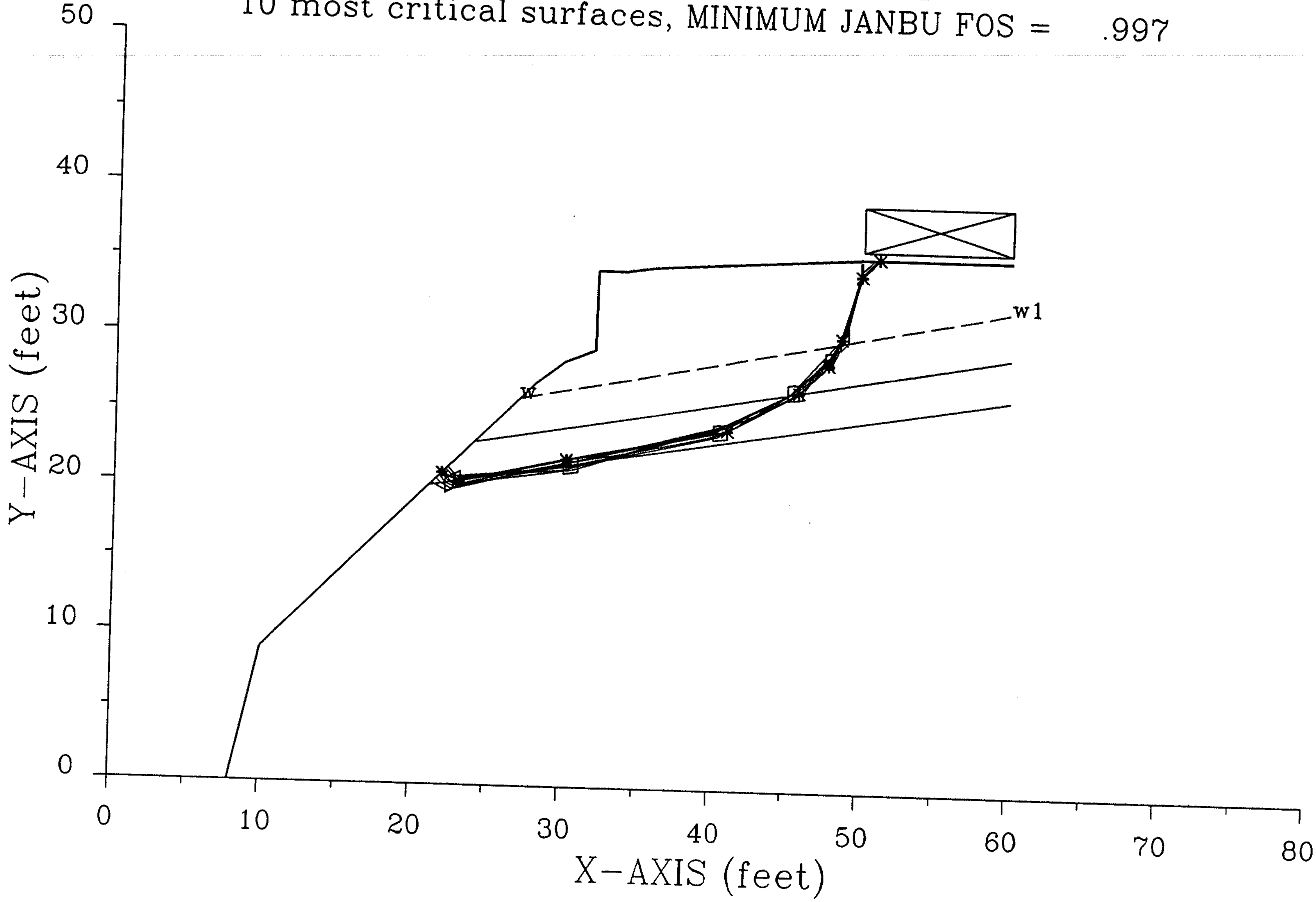
FIG. 5

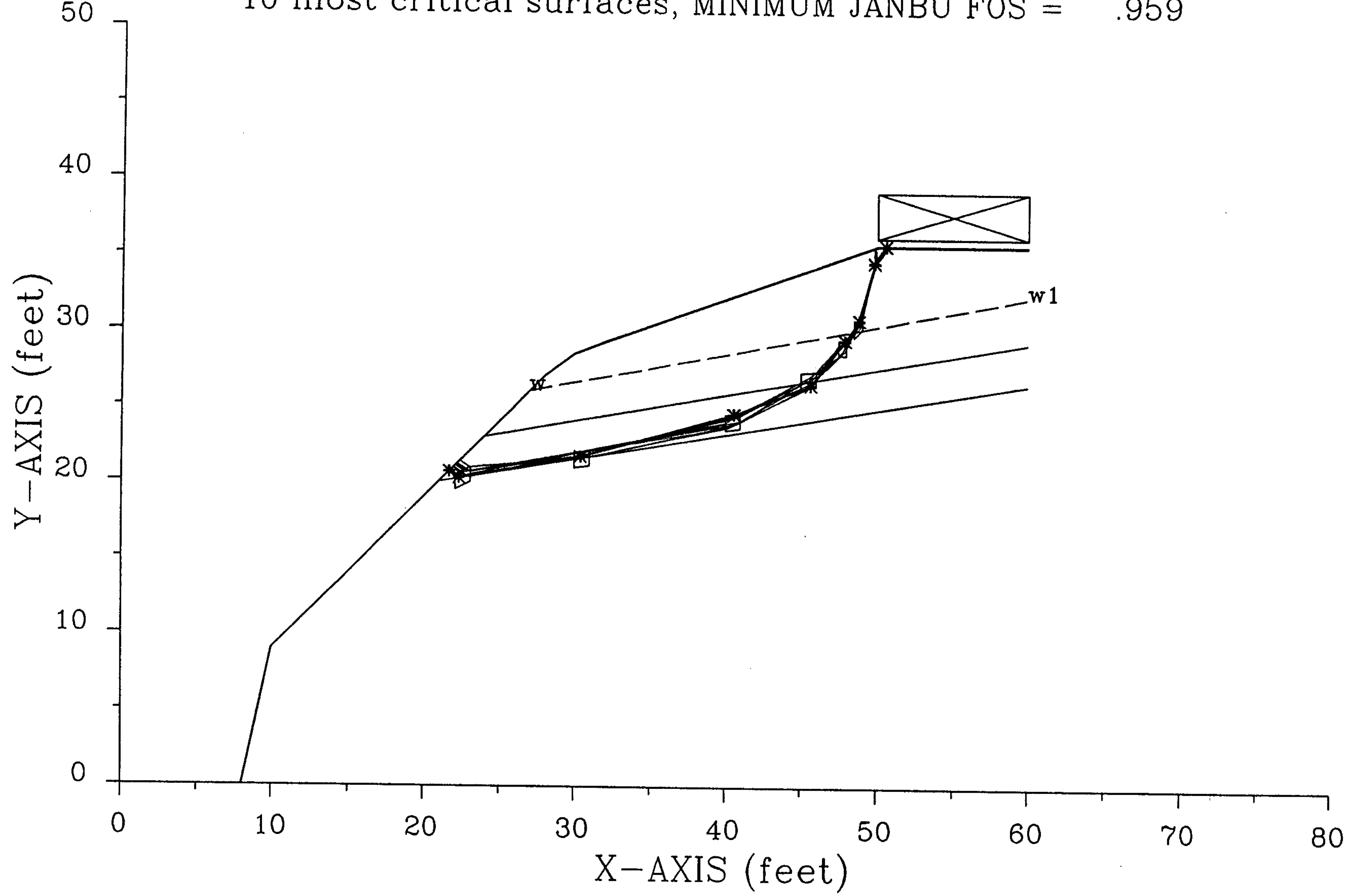
Cummins Residense C=220 psf phi=0

10 most critical surfaces, MINIMUM JANBU FOS = .997



Cummins Residense c=0 phi = 22.5

10 most critical surfaces, MINIMUM JANBU FOS = .959



Solve for FOS = 1.25

R: ARM = 15.8'

$$\frac{\Sigma M_c \circlearrowleft + R \times ARM}{\Sigma M_w \circlearrowright} = 1.25$$

$$\frac{100.76 + R(15.8)}{131.01} = 1.25$$

$R = 3.99$ Kips

$\simeq 4.0$ Kips

EARTH PRESSURES, $\phi = 22.5°$, $\gamma = 100$ pcf

$K_o = 1 - \sin\phi = 0.62$ $\qquad$ H = 12.5'

$K_a = \tan^2(45 - \phi/2) = 0.45$

$K_p = \tan^2(45 + \phi/2) = 2.24$

$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2}(0.45)(\overset{pcf}{100})(\overset{ft^2}{12.5})^2 =$ 3516 Pounds = 3.5 Kips Per Foot of WALL

$P_p = \frac{1}{2} K_p \gamma H^2 = \frac{1}{2}(2.24)(100)(12.5)^2 =$ 17,500 pounds = 17.5 Kips per foot of wall

$P_o = \frac{1}{2} K_o \gamma H^2 = \frac{1}{2}(0.62)(100)(12.5)^2 =$ 4844 Pounds = 4.8 Kips per foot of Wall

GIVEN:

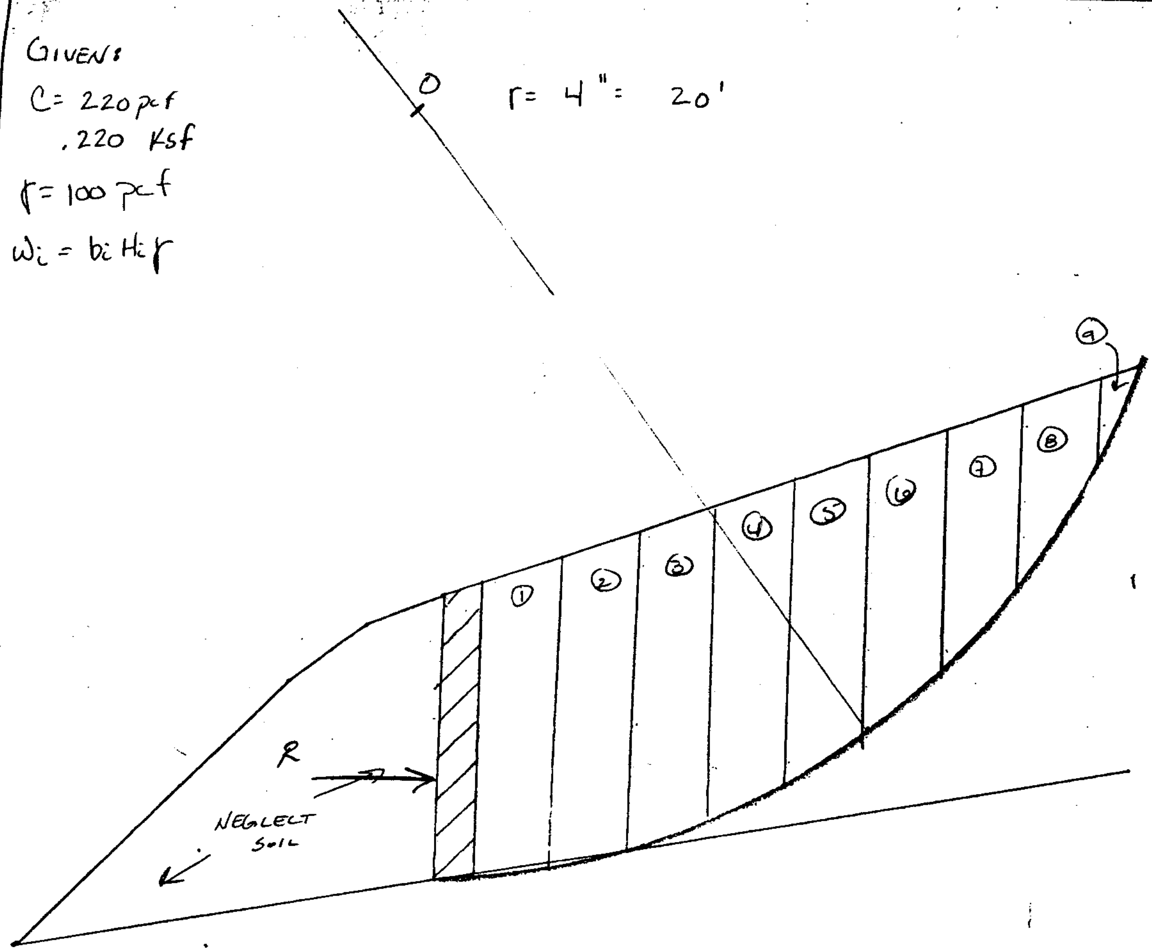
$C = 220\ pcf$

$.220\ ksf$

$\gamma = 100\ pcf$

$W_i = b_i H_i \gamma$

$r = 4" = 20'$



SLICE	b_i (ft)	H_i (ft)	W_i (Kips)	ARM (ft)	M_o↓	c_i (Kip/ft)	x_i (ft)	ARM (ft)	M_o↑
1	2	7.9	1.58	3	4.74				
2	2	8.2	1.64	5	8.2				
3	2	8.1	1.62	7	11.34				
4	2	8.0	1.6	9	14.4				
5	2	7.6	1.52	11	16.72				
6	2	6.7	1.34	13	17.42				
7	2	5.4	1.08	15	16.2				
8	2	3.4	0.68	17	11.56				
9	1	1.1	0.11	18.3	2.013				

ΣM_o↓ $= 102.6$ ΣM_o↑ $= 100.76$

Solve For FOS = 1.25

R: ARM = 17.5'

$$\frac{\Sigma M_o \uparrow + R \times ARM}{\Sigma M_o \downarrow} = 1.25$$

$$\frac{100.76 + R(17.5)}{102.6} = 1.25$$

R = 1.57

= 1.6 Kips

EARTH PRESSURES: $\phi = 22.5$, $\gamma = 100$ pcf, $H = 7.5'$

$K_o = 0.62$

$K_a = 0.45$

$K_p = 2.24$

$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2}(.45)(100)(7.5)^2 =$ 1266 pounds = 1.3 Kips

$P_p = \frac{1}{2} K_p \gamma H^2 = \frac{1}{2}(2.24)(100)(7.5)^2 =$ 6300 pounds = 6.3 Kips

$P_o = \frac{1}{2} K_o \gamma H^2 = \frac{1}{2}(0.62)(100)(7.5)^2 =$ 1744 pounds = 1.7 Kips

Cummins Residense Model E1

10 most critical surfaces, MINIMUM JANBU FOS = 1.149