Attachment IV-1
118 Pages

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.

169 KINDERKAMACK ROAD, PARK RIDGE, N.J. 07656 • Phone: (201) 391-2115

Project No. 8304
September 19, 1983
W3-HE-LP-007

Louisiana Power & Light
Waterford III
PO Box B
Killona, La. 70066

Attn: Mr. George B. Rodgers, Jr., Site Director

Subject: Waterford III SES
Analysis of Cracks and Water Seepage in Foundation
Report No. 8304-1

Enclosure: Three (3) copies of subject report.

Dear Mr. Rodgers:

Please find enclosed three copies of the subject report.

If you have any questions, please contact us.

Very truly yours,

Gunnar A. Harstead

GAH/jl

GUNNAR A. HARSTEAD
President

cc: B. W. Churchill
c/o Shaw, Pittman, Potts & Trowbridge
(w/Attachment)

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.

169 KINDERKAMACK ROAD, PARK RIDGE, N.J. 07656 • Phone: (201) 391-2115

Project No. 8304
September 21, 1983
W3-HE-LP-008

Louisiana Power & Light
Waterford III
P.O. Box B
Killona, La 70066

Attn: Mr. George B. Rodgers, Jr., Site Director

Subject: Waterford III SES
Analysis of Cracks and Water Seepage
in Foundation
Report No. 8304-1

Enclosure: Three (3) copies of Appendix M to
subject report.

Dear Mr. Rodgers:

Please find enclosed three copies of Appendix M to the subject report.

If you have any questions, please contact us.

Very truly yours,

Gunnar A Harstead

Gunnar A. Harstead
President

GAH/tm

cc: B. W. Churchill
c/o Shaw, Pittman, Potts & Trowbridge
(w/Attachment)

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.

169 KINDERKAMACK ROAD, PARK RIDGE, N.J. 07656 • Phone: (201) 391-2115

WATERFORD III SES

ANALYSIS OF CRACKS AND WATER SEEPAGE IN FOUNDATION MAT

LOUISIANA POWER & LIGHT COMPANY

REPORT NO. 8304-1

SEPTEMBER 19, 1983

Prepared by: AV du Bouchet
A. V. du Bouchet

Reviewed by: Ahmet Unsal
A. I. Unsal

Approved by: G A Harstead
G. A. Harstead

TABLE OF CONTENTS

1.0	Introduction	1
2.0	Site Inspections and Interviews	2
3.0	Foundation Mat Design Concept	3
	3.1 Site	3
	3.2 Design	3
	3.3 Construction	4
4.0	Significant Events During Construction	6
	4.1 Stop Work Order No. 1	6
	4.2 Concrete Placement of the Mat	6
	4.3 Change in Allowable Soil Bearing Pressure During Construction	7
	4.4 Site Settlements	7
	4.5 Cracks Observed in the Top of Mat in the Containment Area	8
	4.6 Cracks in Mat Outside of Containment Area	9
5.0	Analysis of Waterford III Structural Foundations	11
	5.1 Structural Concept	11
6.0	Review of Engineering Design and Construction	13
	6.1 Geologic Studies	13

TABLE OF CONTENTS (Cont'd.)

6.2	Development of Engineering Properties	14
6.3	Foundation Design Concept	15
6.4	Design of Combined Mat	17
6.5	Earth Pressure Considerations	18
6.6	Groundwater Environment	18
6.7	Excavation Sequence	19
6.8	Dewatering Systems	20
6.9	Subsurface Instrumentation Program	21
6.10	Construction of Mat	21
6.11	Summary of Movements Recorded During Construction	21
7.0	Evaluation of Cracking	24
8.0	Corrosion Potential	29
8.1	Passivation Mechanism in Reinforced Concrete	29
8.2	Job Specifications	30
8.3	Laboratory Testing	30
8.4	Steel Containment Corrosion	34
9.0	Steel Containment Stability	35
9.1	Ebasco Calculation 1352.063	35
10.0	Conclusions and Recommendations	38
10.1	Containment Vessel	38
10.2	Foundation Mat	38

TABLE OF CONTENTS (Cont'd.)

REFERENCES 40

APPENDICES

A Basemat Crack Maps

B. Properties of Subsurface Materials Design Values

C Synposis/Introduction of State-Of-The-Art of Floating Foundations by H. Q. Golder

D Generalized Site Cross Section

E Effects of Soil Modulus on Shear and Moment

F Design Envelopes of Mat Shear and Moment

G Effects of Foundation Stiffness on Dynamic Shears and Moments

H Abasco Services Letter F-16919, W3-NY-1 dated June 29, 1977

I Composite Foundation Mat Differential Settlement Contours

J Composite Foundation Mat Settlement

K Crack Width Calculation

L Steel Containment Stability Calculation

1.0 Introduction

This report summarizes a study undertaken by Harstead Engineering Associates on behalf of Louisiana Power and Light Company.

The following major evaluation items are addressed in this report:

a) The engineering criteria employed in the preparation of the site and in the design and construction of the Waterford III Nuclear Power Island Structure (NPIS) basemat.

b) Cracking and leakage in the basemat.

c) The laboratory tests performed on water and leachate samples extracted from the surface of the basemat.

d) The stability calculations performed for the Steel Containment Vessel.

As required, relevant source material is either referenced or contained as an appendix to this report.

2.0 Site Inspections and Interviews

HEA personnel have visited both the New York office of Ebasco, Inc. and the Waterford III site.

These visits are summarized in HEA Trip Reports Nos. 1-6 (References 1-5), and were conducted in order to meet with key personnel familiar with the design bases of the Waterford III NPIS basemat, to document first-hand the extent of cracking and leaking at the surface of the basemat, to gather pertinent reports and drawings, and to confirm a scope of work and corresponding schedule.

3.0 Foundation Mat Design Concept

3.1 Site

The site of the Waterford 3 plant is located next to the Mississippi River. Natural grade is at about Elevation + 15.0 feet. To a depth of about 55.0 feet from grade, the soil consists of alluvial deposits which are relatively soft. At greater depth are the Pleistocene Age soils. The upper parts of these soils are stiff.

3.2 Design

The Safety class structures are supported on a continuous mat 270 feet wide, 380 feet long and 12 feet thick. The bottom of the mat is at a depth of about 60 feet below natural grade. Support for the mat is provided on the stiff Pleistocene clays, where the natural soil pressures were about 3300 psf. After the completion of construction, the soil pressure under the foundation mat is about 3100 psf. These pressures consist of the weight of soil and construction above the mat less the buoyant pressure due to ground water. The water table is generally at an elevation of + 8.0 feet; therefore, the buoyant pressure is about 3400 psf. The weight of the soil which was excavated was about 5700 psf, while the weight of the construction now in place is about 5500 psf. The interesting feature of this is that the soil below the plant is experiencing almost the same pressures that it has in recent history. Therefore, increased consolidation of soil and the accompanying settlement that often occurs when new construction weight is added to soil does not occur in this case.

Inasmuch as the water table is at about Elevation + 8.0 feet or almost at natural grade, walls were erected

around the perimeter of the mat. These walls must resist the lateral pressure of the surrounding backfill soil and the hydrostatic pressure of the ground water. These walls extend up and provide flood protection up to Elevation + 30.0 feet, which is 22.0 feet above the normal water table.

The mat and the walls form a reinforced concrete box structure, with interior walls and concrete placements referred to as counterforts providing additional stiffening. The mat and the exterior walls are monolithic and therefore prevent water flow through joints and in the sense that ground water is prevented from collecting inside the structure, the structure has been called a floating structure.

3.3 Construction

The steps involved were:

a) Dewatering the site

b) Excavation down to Elev. - 47.0 feet

c) Construction of the mat

d) Construction of superstructure

e) Gradual release of Dewatering

f) Backfill of excavation surrounding the construction

The soil pressures existing at Elevation - 47.0 feet vary considerably during construction. After dewatering the pressure increases to the weight of the soil above due to loss of buoyancy and then after excavation the pressure, of course, reduc... zero. When the pressure is reduced, the soil heaves or rises ... to the removal of the weight of the overlying soil.

As construction proceeds, the s... ...ural weight causes the soil pressure to increase and the soil ...ns to resettle. In order to provide additional compaction, the

soil bearing pressure is allowed to increase. The pressure was allowed to increase to 4500 psf. As construction continued, the gradual release of dewatering offset the increasing structural weight.

The construction was planned to maintain a maximum differential soil bearing pressure of 2000 psf. In the final condition, a maximum differential soil pressure of 1000 psf was established by the designers (Reference 6).

During construction, settlement and water pressure readings were taken in order to ensure that control was being maintained over differential mat settlements.

4.0 Significant Events During Construction

4.1 Stop Work Order No. 1

LP&L issued SWO No. 1 on December 16, 1975 in order to correct deficiencies and nonconforming work in the inspection and control of concrete mixing, transporting and placing of concrete, and curing and finishing. This resulted from observation of Placement No. 6.

4.2 Concrete Placement of the Mat

During placement of Section 10B, a rain storm broke out. The placement was completed; however, because the concrete quality was unknown due to dilution with rain water, NCR No. W3-39 was filed.

A repair program was established which included coring, strength testing, pressure grouting of drilled holes, repair of surfaces, and waterproofing of the west face.

Discrepancy Notice C-13 dated 12-16-75 noted cracks in the west face of Placement No. 2. Cracks were chipped out and surface roughened prior to making adjacent Placement No. 4.

During placement of concrete in Placement No. 19, concrete was placed over a previous layer while it was no longer plastic. This surface was raked and fresh concrete was placed. Concrete was later shipped out in certain areas to a depth of 6 inches to 12 inches below the mat top rebar and replaced with fresh concrete. Still later, 11 cores were taken to a minimum depth of 5 feet. The cores were tested and the core holes grouted.

The "cold joints" and dilution of concrete are undesirable because of voids and weaknesses. The extensive and methodical repair program that took place as indicated in the documentation and subsequent observations of the foundation mat indicate that the repair was effective and that

there is no concern about the strength or corrosion protection of the concrete.

4.3 Change in Allowable Soil Bearing Pressure During Construction

On March 15, 1977, Ebasco requested that the maximum temporary bearing pressure during construction be increased from 4000 psf to 4500 psf. This recommendation was based on the fact that the maximum allowable bearing pressure of the soil is 15,000 psf, the desire to accelerate recompression of the soil that heaved after dewatering and excavation, and the need to permit backfilling under the Turbine Building.

Due to scheduling difficulties, the dewatering system was not in place during the initial removal of 20 feet of soil. The remaining soil heaved between 1.5 and 3.5 inches. After the dewatering was under way and about 1.0 inch of the heave was recovered, the job was shut down. The dewatering was not operating long enough to balance the total heave.

In November 1974, the dewatering was reinstated. In January 1975, the remainder of excavation was restarted and the heave increased to between 4.0 and 9.0 inches. When concrete construction proceeded, the heave reduced to between 1.0 and 6.0 inches.

The above compares to a heave projected as 2.0 inches. Rebound is a function of both load removed and time of load removal. The differences in schedule and loading were cited as reasons for the difference.

In order to ensure full recompression of the rebound, a greater soil pressure was recommended.

4.4 Site Settlements

In September 1978, a report "Review of Site Settlements" by M. Pavone and J. L. Ehasz, was issued (Reference

7). It was noticed that there was a total settlement of about 11.0 inches from the maximum heave position.

Since the maximum heave was previously noted as being between 4.0 and 9.0 inches, the overall settlements were therefore beyond the original zero position of the soil. The settlements have remained constant since early 1979.

A curvature in the North-South direction was noted, the center of the mat being 2.5 and 1.5 inches higher than the south and north edges respectively.

4.5 Cracks Observed in the Top of Mat in the Containment Area

Nonconformance Report NCR W3-535, dated August 3, 1977 reported that cracks were discovered in the top of mat, which were weeping water. The rate of weeping water was enough to show the crack and to moisten the surrounding concrete. A crack map was prepared (Reference 8) and the crack widths were noted as being between 2 to 5 mils. The piezometer level was kept below Elevation - 50.0 feet since the start of 1975, until September 1977. The concrete in this region was placed in December of 1975 and January of 1976. The lower concrete ring of the shield building was also in place. The cracks were chipped out to a 1.0 inch depth and to 12 inches on either side and repaired with epoxy grout. It was hypothesized in Ebasco Letter of July 27, 1977 (Reference 9) that the general curvature of the mat caused tension in the upper portion of the mat. Locally the lower ring wall of the shield building would have caused tension in the lower portion of the mat. This was termed as a stress reversal and the possibility of an intertie between cracks could exist providing a direct leak path. It was also stated that leakage of water through the mat was undesirable because:

a) a film of water could be presumed to develop between the mat and the fill concrete beneath the containment vessel. This could require a reanalysis of containment stability (see Section 9.0).

b) if the leakage increased and water found its way out of the fill concrete it would be collected in the mat drainage system and run through the waste treatment system (see Section 5.1).

The repair apparently stopped further leakage.

4.6 Cracks in Mat Outside of Containment Area

Ebasco Nonconformance Report W3NCR-16143 (Reference 10) noted that: "there are concrete cracks in the base mat of the Reactor Auxilliary Building. This is evidenced by the percolation of water in small amounts, up through these cracks. These cracks are located in the Gas Surge Tank Room, Waste Gas Tank Room, and Waste Gas Compressor "B" Room, all at elevation - 35.0 feet.

These are examples of where cracks were found:

G. Harstead of HEA observed the above-mentioned locations where cracks had been observed, as well as other areas, during the period of July 11-14, 1983 (Reference 1).

All accessible areas of the basemat were subsequently inspected and any cracks found were mapped during the period of August 30 - September 2, 1983 by A. du Bouchet of HEA with the assistance of LP&L and Ebasco personnel (Reference 5).

The crack maps generated during this inspection are contained in Appendix A. The reference points employed to locate these cracks accord with the geometry detailed in Ebasco Drawings LOU-1564 G-499 S01, -02 and -03.

In addition to mapping the orientation and extent of each crack, notations were made concerning any prior repairs to the crack, floor finish or lack thereof, evidence of dampness or seepage, and crack width.

As noted in Reference 5, "Crack width dimensions could not be quantified, but are designated throughout as 'hairline'. In several instances, the existence of a crack could only be inferred by the darker coloration caused by the presence of moisture. No actual gaps were noted."

The amount of moisture noted during this inspection period was minimal. In some instances dampness/moisture were present. There was, however, no evidence of seepage or migration that might have been deduced by the presence of standing water or draining along the local slope of the basemat.

5.0 Analysis of Waterford III Structural Foundations

5.1 Structural Concept

The foundation concept is an ingenious solution of the site problems in meeting the safety criteria established for the nuclear safety related structures. The most significant factor in assessing the adequacy of the design is that the final soil pressure after construction is actually less than the soil pressure which existed prior to the start of construction. The stability and safety that this implies has been demonstrated, in that, the settlement has not changed for the past several years except for changes that would be expected by changes in the water table.

As part of this concept, the mat and exterior walls were to remain watertight. If water could readily flow in and no provision was made to pump water out, conceivably the water inside would increase the effective soil pressure and result in further soil settlements, although the effective soil pressure would still be less than one half of the maximum allowable. For many reasons, flooding would be intolerable; however, there would probably be little detrimental effect upon the structure. The differential hydrostatic pressure on the exterior walls would be liminated, thereby reducing the lateral load on the building. The loading on the mat would remain approximately the same because the creased effective soil pressure would be equal to the weight of the water which leaked into the structure. There would be long term settlements in this case and perhaps some differential settlements, which is not an unusual situation in many structures.

Section 11.2 of the Waterford III FSAR (Reference 11) details the capacity of the Waste Management System (WMS).

Table 11.2-4 specifies a total expected waste flow of 1425 gpd based on the following flow sources:

Containment Building sump (40 gpd), Auxiliary Building floor drains (200 gpd), laboratory drains and waste water (400 gpd), sampling drains (35 gpd), miscellaneous (700 gpd) and blowdown (50 gpd).

Table 11.2-2 specifies a total useful internal volume for the two WMS waste removal tanks of 7200 gals.

HEA therefore concludes that the capacity of the Waste Management System as detailed above effictively eliminates the possibility of ground water accumulation within the NPIS.

6.0 Review of Engineering Design and Construction

To determine if implementation of the unique floating foundation concept resulted in excessive differential movements during construction, documents pertaining to the design, engineering, and construction were reviewed. Data included related sections of the FSAR, instrumentation reports, calculations related to the design, formulation and application of the foundation design principle, and relevant correspondence. The following specific areas were addressed:

a) Geologic studies

b) Development of engineering properties for foundation soils

c) Foundation design concept

d) Design of combined mat

e) Earth pressure considerations

f) Groundwater environment

g) Excavation sequence

h) Dewatering system

i) Construction of mat

j) Summary of movements recorded during construction.

Pertinent data related to the above will be analyzed to establish that the design concept was developed and implemented successfully.

6.1 Geologic Studies

The Waterford Nuclear Power plant is located on the west bank of the Mississippi River about 20 miles from New Orleans. The site consists of over 3,000 acres with surface elevations ranging from approximately sea level in the

southwest to about elevation plus 14 feet MSL at the base of the flood control levee.

The crest of the levee is the highest point of the site and is about elevation plus 30 feet MSL. The Mississippi River is 110 feet deep and about 2,200 feet in width adjacent to the plant.

Geologic studies conducted at the site included review and interpretation of geologic literature, subsurface borings, geophysical logs, cross-hole data and laboratory tests. The stratigraphic sequence is described as follows (FSAR Section 2.5.4.1);

Sequence	Depth (Feet)
Recent alluvium deposits	0 - 50
Pleistocene sands and clays	50 - 1,100
Plio-Pleistocene interbedded sands and clays	1,100 - 1,900
Pliocene alternating sands and clays	1,900 - 4,900
Massive sandstone interbedded with shale	4,900 - 7,500
Shale alternating with thin sandstone layers	7,500 -10,500
Marine shales	10,500 -40,000

This review will be confined to the upper 500 feet of soil strata.

6.2 Development of Engineering Properties

A total of 74 soil borings were drilled at the site to determine the detailed stratigraphy applicable to the upper 500 feet of subsurface materials. Static and dynamic engineering properties were established based on laboratory tests on selected samples and in situ geophysical measurements. A brief visual description of the principal soil strata is provided below:

Sequence	Depth (MSL)
1 Soft clay and silty clay with silt and sand	GS to -40
2 Stiff tan and grey fissured clay	-40 to -77
3 Very dense tan silty sand	-77 to -92
4 Medium stiff grey clay with silt lenses	-92 to -108
5 Stiff dark grey clay - organic	-108 to -116
6 Soft to medium stiff tan and grey clay	-116 to -127
7 Very stiff clays with silts and sands	-127 to -317
8 Very dense sands and silty sands	-317 to -500

Design values applicable to each stratum are defined in Appendix B (FSAR Table 2.5-14).

6.3 Foundation Design Concept

In reviewing the consolidation characteristics of the potential foundation bearing strata it is apparent that excessive settlement could be anticipated if the Nuclear Plant Island structures were founded directly in the stiff Pleistocene deposits (layer 2) unless the total bearing pressure from the structures were reduced during construction by buoyancy effects. Of particular concern are the clays with relatively Low Overconsolidation Ratios (OCR), such as layer 4, which has an OCR of 1.4.

By excavating a depth of soil approximately equal to the weight of the structure, the effective pressure at the base remains unchanged, thereby reducing the potential for underlying clay layers to settle. The floating foundation principle has not been used previously in nuclear power plant design; however, historically, many large structures have been constructed using this concept. See, for example, an extract from a paper entitled "State of The Art of Floating Foundations" by H. Golder, which is contained in Appendix C.

The following is a brief summary of the significant control parameters developed for the Waterford Plant incorporating the floating foundation principle.

a) All Category I structures were combined in a nuclear plant island on a common mat.

b) Base of mat foundation is in the stiff Pleistocene clays at Elevation - 47 MSL.

c) Effective bearing pressure of the Nuclear Plant structure is 3,100 psf. compared to the existing overburden pressure of 3,300 psf.

d) Dewatering systems were required to minimize potential for heave at base of excavation, control pore pressure in layer 3 and stabilize excavation slopes.

e) During construction the total pressure at base of mat may have increased to 4,000 psf resulting in an additional pressure of 700 psf. It was estimated that the heave after excavation would be in the order of 2 inches and recompression from this additional pressure would be complete by the end of construction.

f) A filter layer of compacted sand and shell (18 inches) was placed at base of excavation underlying mat to permit distribution of pore pressure in the underlying clay on application of load.

g) As soon as the total load from the Nuclear Plant Island and surrounding granular backfill reached 4,000 psf, segments of the dewatering system were released in stages to achieve buoyancy of the structures and backfill and permit construction to continue. Details of the proposed Recharge Program were developed during construction related to well efficiency and piezometric response.

h) A maximum differential loading of 1 ksf was applied

to base of structures to minimize tilting, heaving, and settlement.

i) A detailed instrumentation program was required to monitor movement of structures and groundwater levels.

j) Long term settlement was anticipated to be less than 1 inch due primarily to local pore pressure adjustments in the clays.

A generalized site cross-section showing the Nuclear Island structures and adjacent non-Category I Turbine Building is outlined in Appendix D (FSAR Fig. 2.5-80).

6.4 Design of Combined Mat

Details of the parametric and sensitivity studies conducted to establish the appropriate mat thickness and required reinforcement for static and dynamic loading conditions are outlined in Reference 12. The selection of the subgrade modulus applicable to the foundation soils and mat geometry is judgemental. The values used to estimate impact or mat thicknesses of 10, 12, and 15 feet (k_s=150 pci and 125 pci) are considered reasonable. The twelve foot thickness finally selected was based on an economic compromise between the cost of additional concrete to eliminate shear reinforcement and providing some shear reinforcement in local areas.

The influence of a constant or variable modulus on the shear and moment diagrams is shown in Appendices E and F (Figures 7 and 8 - Reference 12). The design envelope selected covers all possible support conditions.

An inherently conservative approach was also adopted in analyzing the mat for seismic loadings resulting from the SSE (0.1g) and OBE (0.05g). As shown in Appendix G (Figure 9, Reference 12) the total shear and moment increases rapidly with increasing foundation stiffness to

approximately G = 3,000 ksf where G = the Dynamic Shear Modulus.

Although the indications of soil stiffness based upon geophysical site measurements indicated that the value of G should be 1000 ksf, the seismic responses in the plant structures would be greater for increased soil stiffness. In order to be very conservative, the seismic analysis was based upon a dynamic model using a G = 3000 ksf which resulted in peak seismic response and therefore peak moments in the mat.

The seismic analysis mathematical model contains elastic springs representing the stiffness of the soil. The results of the analysis include soil spring deformations which represent soil movement with respect to some origin point of earthquake. The peak horizontal deformations were used to calculate the passive earth pressure on the perimeter walls.

6.5 Earth Pressure Considerations

The procedures outlined in section 2.5.4.10.3 of the FSAR related to determination of static and dynamic earth pressures on the structural walls were reviewed. An "At Rest" earth pressure coefficient of $K_o = 0.5$ was selected for the compacted granular fill.

This highly conservative approach adopted in determining the earth pressure for dynamic loading conditions (correlating movement of structure from dynamic analysis with strain obtained from a typical earth pressure diagram) combined with static loads results in a heavily reinforced perimeter wall.

6.6 Groundwater Environment

Evaluation of piezometric response in the Recent alluvium to fluctuations in the river level indicated that the

clays, silts, and sands were discontinuous and unresponsive. Average permeability of these deposits is estimated to be in the order of 1.5×10^{-6} cms/sec (FSAR Fig. 2.5-12). Similar conclusions were reached regarding the transmissibility of potential sand layers in the Pleistocene clays. Below the stiff clays at Elev. -77 MSL it has been stated that all strata are responsive to river level fluctuations. The major source of recharge for the granular backfill surrounding the Power Plant is expected to be from rain and run-off, possible interconnections with discontinuous sand layers extending to the river and the tan silty sand layer at Elev. -77 MSL.

Water quality has been analyzed and no corrosive elements were detected which could impact the reinforcing steel embedded in the concrete mat (see Section 8.0). The possibility of the water becoming saline at some future date was considered; however, lack of oxygen would prevent corrosion from this water source.

The greatest potential for corrosive elements to be present in the groundwater immediately adjacent to the concrete mat would be from the Mississippi River; however, the seepage path required in the assumed continuous silty sand stratum at Elev. -77 MSL, relatively low permeability, estimated gradient of 0.008 and probable filter medium would result in a groundwater environment surrounding the Plant with all corrosive elements removed or highly diluted.

6.7 Excavation Sequence

Unfortunately due to schedule and legal problems it was not possible to complete the required excavation for the Plant Island structures until October, 1975. The following is a brief summary of the major excavation phases commencing with the initial excavation in April, 1972 (FSAR Section 2.5.4.5.1).

Excavated

Stage	From	To	Started	Finished
Phase I	Grade	-5	April 72	July 72
II	-5	-22	January 75	June 75
III	-22	-40	April 75	August 75
IV	-40	-48	October 75	March 76
Turbine Bldg.	Grade	-40	January 77	March 77

Concurrent with the excavation phases, extensive dewatering systems were installed and operated.

6.8 Dewatering Systems

The dewatering systems were installed by Moortrench-American Corporation based on performance specifications prepared by Ebasco. A total of 251 dewatering wells were located around the perimeter of the excavation with 217 pumping from the Recent alluvium and the remaining 34 from the Elev. -77 Pleistocene sands. A second series of 12 deep pump relief wells were located around the combined structure mat and pumped from the Elev. -77 sands. No details were provided on the design of the well systems. It is assumed these studies were performed by Mooretrench. On evaluating the Instrumentation Reports covering the monitoring of the dewatering operation it appears that the systems generally performed as intended. It was noted in a letter from Ebasco to Boh Bros. dated June 29, 1977 (Appendix H) that significant operational and maintenance problems had developed. Corrective action was taken by the contractor and it is understood that the wells were stabilized and instrumentation readings were obtained in conformance with specifications. An extensive Recharge Program (required to achieve buoyancy of the structures) was implemented successfully in October, 1977 and completed by July, 1979 when normal groundwater pressure levels were achieved.

6.9 Subsurface Instrumentation Program

The scope of the Instrumentation Program consisted of monitoring piezometric levels, foundation soil heave, settlement, excavation slope movements and potential site subsidence due to dewatering. A total of 24 piezometers were installed to measure groundwater response in selected soil strata. Five additional piezometers were located in the filter layer underlying the combined mat. A total of nine (9) heave points, two (2) extensometers, six (6) inclinometers, and twenty-eight (28) settlement monuments were installed to measure movement of structures and excavation slopes.

6.10 Construction of Mat

The excavation for the combined mat was performed with backhoes by making an eight (8) foot vertical cut in the stiff Pleistocene clay from Elev. -40 to Elev. -48. The excavation was performed in strips. The initial strip was located under the Reactor Building area approximately 120 feet wide running to the full width of the mat. Subsequent strips (Nos. 2 and 6) were cut north and south of strip No. 1 as shown in Appendix I (Fig. 2.5-118). Concrete placements were made simultaneously in alternate strips. All exposed vertical cut faces were gunited within 8 hours of exposure to prevent dessication prior to concrete mat placement.

6.11 Summary of Movements Recorded During Construction

On completion of excavation in October, 1975 for the common mat it was noted that the clays had heaved a total of 5 to 10 inches with the maximum amount occurring at the north end closest to the river. This magnitude of heave was considerably greater than anticipated in original design (approximately 2 inches) and was due primarily

to general relaxation of the clays due to the number of excavation phases and the stop/start operation of the dewatering systems.

To compensate for the additional heave, the permissible overload of 700 psf was increased to 1,200 psf in order to accomplish most of the recompression during construction. This increase permitted the load from the structures and backfill to be increased to 4,500 psf prior to commencing the Recharge Program to achieve buoyancy. By July, 1977 recompression of the heave had occurred due to loading from structures and backfill, assisted by larger and more efficient dewatering pumps. The dewatering system was continued until October, 1977 when the average net settlement was approximately 2 inches. During the period October, 1977 to July, 1979 the rate of movement was controlled by releasing the dewatering system in stages, permitting construction of the plant to continue. The average net settlement increased to approximately 5 inches during this period. Readings have stabilized at that level for the past four years with only minor fluctuations noted due to change in river level. The composite foundation mat settlement is shown in Appendix J (FSAR Fig. 2.5-117).

Detailed review of the instrumentation records covering construction of the Plant indicated that the applied structural load was sufficiently controlled so that the permissible maximum differential loading of 1 ksf across the base of mat was not exceeded. Adherence to this criterion resulted in minimum deflections and minimum curvature (for the mat geometry) at the surface and base of mat. Maximum differential movement was recorded at 2.5 inches with the maximum settlement occurring at the north and south ends.

Although the maximum recorded heave and subsequent settlement was considerably in excess of original design estimates, careful control had been exercised in applying load from structures and equipment in a uniform sequence. By conforming to the maximum differential loading criterion of 1 ksf recompression of the heave and consequently rate of strain was controlled. This procedure minimized unusual and severe distortions of the mat.

7.0 Evaluation of Cracking

While it is not possible to precisely predict stresses in reinforcing bars, an upper bound estimate is possible by estimating the strain as the crack width divided by crack spacing. Assuming crack widths of 5 mils spaced 10 ft. on center, it may be shown (Appendix K) that the approximate stress in the top rebar is 1200 psi. The actual crack width and spacing would indicate a much lower stress. Nevertheless, if the conservative value of 1200 psi tension in the top reinforcing bars is conservatively assumed to be constant for the entire 330 feet of length of the mat, the indicated differential settlement would be somewhat less than 1.0 inch. This provides added assurance that differential soil pressures were very well controlled during construction. This also indicates that the mat is quite tolerant of such differential settlements.

Furthermore, settlement stresses are considered secondary stresses in that they do not impair the structural capacity to carry other imposed loads such as dead load and seismic loads. This is possible provided that there is no failure of the supporting soil. In the case of Waterford III the soil is loaded at about one fourth of the design load and in fact, less than the previous insitu condition. When this is compared to the reinforcing bar yield stress of 60,000 psi, it is clear that these cracks did not give any evidence at all of any structural distress.

Cracks are expected in reinforced concrete structures, and are caused by many factors, such as:

application of tensile forces,
drying shrinkage of concrete
thermal gradients,
and differential settlements.

The last three effects are the result of geometric constraints, which do not limit the ability of a properly designed reinforced concrete structure supported on competent soil to carry imposed loads. By "properly designed" it is meant that sufficient reinforcing steel is placed in the concrete to prevent large tensile cracking of the concrete, crushing of concrete, or diagonal tension shear failure.

The cracks that were reported are of little concern with respect to the structural adequacy of the mat; therefore, the precise cause of the cracks is not important. The cracks could be the result of:

shrinkage

temperature gradient,

settlement, or

a combination of the above.

However, it is concluded that the origin of the cracks detected during construction was not due to severe differential movements occurring during or immediately after application of loads from structures and equipment.

The water reported to have surfaced through the cracks is probably ground water under a pressure head. Based on records of dewatering, there does appear to have been sufficient hydrostatic head available to force water through the cracks observed in the mat. Regardless of the hydraulic process, very little water was observed. It was described as "not resulting in generally enough water to form a sheen but enough to definitely show the cracks and to moisten surrounding concrete". With the low rate of water weeping and the rather limited cracking, there is no reason for concern.

In 1983, additional cracks in the mat were reported in

areas outside of the Reactor Building (see Section 4.6 and Appendix A). These cracks probably developed several years ago. During an interview with Mr. J. Sleger, he stated that one crack was observed with evidence of seepage during the late summer of 1979. It is very probable that all of the cracks discovered in 1983 were present for some years. Indeed, several of these cracks gave indications of epoxy repairs.

All of these cracks appear to be the same; namely, a crack which is either a hairline crack or which is invisible to the naked eye. Many of the cracks are associated with "leachate", moisture and/or evidence of an epoxy repair at the top surface of the mat. Both "leachate" and moisture are observed in very small quantities. These cracks are not indicative of any high stress in the reinforcing bars. In fact, based upon the observed cracking, one could conclude that the foundation mat is virtually unloaded. If the foundation mat was actually loaded as assumed in the design calculation, one would expect considerably more cracking. This tends to confirm the statement that the calculations for the mat are indeed conservative.

Crack widths of anywhere from 10 to 80 mils, depending upon crack spacing, which would not be beyond expectation, are not cause for concern of the structural integrity of the mat.

While cracking of concrete is expected, it is, of course, important to evaluate the cracking for several reasons:

a) If the crack width becomes very large and there are corrosive chemicals and oxygen present the reinforcing steel may be subject to rusting.

b) Large and extensive cracking may be indicative of forces acting on the structure which can cause damage such that the ability of the structure to resist loads, due to service, is compromised.

c) For the case of the Waterford III mat in particular, seepage of water from cracks may invalidate the "floating mat" concept and affect the containment vessel stability.

The cracks in the mat have widths that are so small that there is no chance of intrusion of corrosive materials and that corrosive materials are not in the environment within the plant or outside. In the Commentary to ACI 318-71 Section 10.6 it is stated that "To assure protection of reinforcement against corrosion and for aesthetic reasons, many fine hair cracks are preferable to a few wide cracks." From the observation of the Waterford III mat, one would have to describe the situation as one of a few hair cracks much less than the many fine hair cracks envisaged as a preferable condition.

The observations of the cracks indicate the seepage of water up through cracks carries with it "leachate" which contains primarily calcium carbonate and magnetic iron. The leachate apparently seals the cracks because many of the cracks show leachate deposits which are now dry. This self sealing process may eventually eliminate leakage; however, seepage is still in evidence even though the process has probably been underway for several years. Nevertheless, the present seepage is minor and poses no difficulties.

Since the advent of Portland cement in construction, it has been known that steel reinforcing bars embedded in Portland cement concrete are protected from corrosion.

Quoting from the Commentary to ACI-318-71 Section 10.6 "Recent extensive laboratory work involving modern deformed bars has confirmed that crack width at service loads is proportional to steel stress." As noted above, the observed cracks indicated a very low stress in the reinforcing steel.

8.0 Corrosion Potential

8.1 Passivation Mechanism in Reinforced Concrete

In order to assess the potential for corrosion in the reinforcing steel of the NPIS basemat, several references concerning corrosion of steel in concrete were reviewed (References 14-18).

As noted in Reference 14, "the corrosion resistance of steel in Portland cement concrete has been recognized for more than a century. The protective mechanism, not described until recent years, is due to a passivating film of gamma ferric oxide which is formed and maintained in the alkaline environment produced by cement hydration".

As noted in Reference 15, "Iron and steel are not thermodynamically stable in water. Either acid or neutral water corrodes iron and forms a ferrous solution. This solution, in contact with oxygen, oxidizes to form hydrated ferric oxide -- a major constituent of rust. If the water is sufficiently alkaline, at pH 8 to 14 for example, the Fe_2O_3 and Fe_3O_4 which form are relatively insoluble and deposit a protective film on the metal surface. The metal is then said to be passivated".

The passivating mechanism, therefore, requires an alkaline environment (pH of about 12.5) and an absence of oxygen in order to form a protective film on the surface of the reinforcing steel.

The alkalinity of the water derives from the hydration of the cement, which generates calcium hydroxide.

A relatively oxygen-free environment is generally insured by careful control of the concrete mix and its subsequent placement. Depth of concrete cover is also a factor.

As noted in Reference 16, "In addition, concrete of

low water-cement ratio and well cured has a low permeability which minimizes penetration of corrosion inducing factors -- oxygen, chloride ion, carbon dioxide, and water."

8.2 Job Specifications

Section I, Paragraph 7.3 of the Ebasco Concrete Masonry specification (Reference 19) stipulates that: "The aggregate, sand and water combined in the same amounts as in the concrete mix shall not contain a total soluble chloride ion content of more than 250 ppm water when water is extracted from the combination after being thoroughly mixed, unless the Engineer allows a deviation in writing...".

Section I, Paragraph 9.7 of that specification further requires that: "No admixture containing chlorides to an extent that the requirements of Paragraph 7.3, with the admixture mixed with the water, are exceeded shall be acceptable unless the Engineer allows a deviation in writing...".

Section II, Paragraph 8.4 of that specification also stipulates that: "Calcium Chloride shall not be used for accelerating the set of the cement in any concrete containing reinforcement or embedded metal parts".

The limitation on the maximum allowable soluble chloride contained in the concrete mix defined in the Reference 19 specification is subsequently verified by the sampling and testing procedures mandated by that specification.

8.3 Laboratory Testing

In order to deduce any evidence of corrosion in the basemat reinforcing steel, several water samples and a solid (leachate) sample were subjected to laboratory analysis.

The three water samples subjected to laboratory analysis were obtained at the following locations:

a) Water rising in Conduit No. 33074, which rises near the West Temporary Electrical Pit, runs to the southeast for approximately 90 feet, and again rises above the basemat. At the south end, no water was rising, indicating a blockage to the flow of water. The conduit is located approximately 3 feet below the top of the basemat.

b) Ground water flowing through conduits which extend from the side of the mat to the East Temporary Electrical Pit.

c) Water collecting at a crack in the Waste Gas Tank Compressor B room.

The solid sample was collected along the top surface of a crack located along an east-west axis between column lines R and Q_1, and straddling column line 1_M.

The laboratory report summarizing the results of the analyses performed on these samples is contained as Appendix M.

As noted under 'Testing Methods and Results' each of the three liquid samples were subjected to analysis for pH, chloride, alkalinity, iron, calcium and sodium. The results of these analyses are subsequently tabulated on page 2 (note that samples designated '1', '2' and '3' accord with the order in which the sample locations are defined herein).

The value of the pH obtained for sample 1, 12.5, accords with the pH of concrete, as previously noted. The pH of 7.5 obtained for samples 2 and 3 is due to the carbonation process which normally occurs at the surface of concrete exposed to open air.

As noted in Reference 14, "Free carbon dioxide reduces pH by carbonation, but only to a depth of a few millimeters in sound concrete".

The report results indicate the virtual absence of iron in the three liquid samples, a clear indicator that the chemical constituents of rust are not present. The ppm of chloride are also well within the maximum allowable 250 ppm mandated in the Ebasco Concrete Masonry specification (Reference 19), as previously noted.

The solid (leachate) sample was subjected to spectrographic and X-ray diffraction techniques. Iron and Calcium are identified as the two major chemical constituents contained in the solid sample.

As noted in the appended laboratory report under 'Remarks', the calcium hydroxide liberated during the hydration of Portland cement will form calcium carbonate in the presence of carbon dioxide; the iron content contained in the solid sample is identified as magnetite.

The results of the testing of the water samples and leachate are consistent with the process of corrosion protection of the steel reinforcing bars embedded in the concrete. As a matter of interest, it should be noted that the reinforcing bars are large. In general, the top reinforcing bar diameter is 1-3/8 inches while the bottom reinforcing bar diameter is 2-1/4 inches.

These properties accord with the properties of the iron compound which (under properly controlled conditions) forms a passivating film on the surface of the reinforcing steel (see the initial extract from Reference 15).

It is interesting to note that this deposition mechanism also occurs in boilers, and is succinctly stated in Section 6, page 129 of Mark's Standard Handbook for Mechanical Engineers (Seventh Edition):

> "At saturation temperatures above moderately low pressures, a second mechanism predominates, in which iron removes oxygen from water or steam, forming iron oxide and releasing hydrogen:
>
> $$3\ Fe + 4H_2O \rightarrow Fe_3O_4 + 8H$$
>
> It is noteworthy that this mechanism does not require the intervention of dissolved gaseous oxygen in the water, which is often the rate-limiting factor in the electrochemical corrosion discussed earlier in this subsection.
>
> The stable oxide at boiler temperatures in a non-oxidizing environment is magnetite, Fe_3O_4 (ferrous ferrite). A normal protective skin of magnetite is formed from the underlying steel".

On the basis of the foregoing evaluation, it is therefore concluded that there is no evidence to infer the existence of basemat rebar corrosion in the vicinity of a crack.

8.4 Steel Containment Corrosion

As noted in HEA Trip Report No. 6 (Reference 5), an inspection of the annular area between the Containment Vessel and the Shield Building revealed some surface corrosion at the base of the Containment Vessel, which might be due to the presence of water generated by construction activity.

As soon as this area can be adequately controlled with respect to the presence of such construction-related water, it is the recommendation of HEA that a program be implemented to clean and field paint the base of the Containment Vessel to insure that the corrosion process has been eliminated in this area.

9.0 Steel Containment Stability

9.1 Ebasco Calculation 1352.063

Ebasco Calculation 1352.063 (Reference 13) was executed as a consequence of Ebasco Nonconformance Report W3NCR-16143, dated May 11, 1983 (Reference 10).

Attachment III to that NCR notes that "The effect of postulated widespread hairline cracking of the basemat has been investigated by Civil Engineering for stability of the Containment Vessel against flotation and overturning under buoyant conditions caused by postulated groundwater intrusion...".

An attached memorandum from P.-C. Liu to B. Grant dated May 24, 1983 specifically indicates that the stability of the Containment Vessel has been reviewed for a postulated hydrostatic infiltration to Elev. -1.50 feet. An examination of Ebasco Drawing No. 1564 G-817, Rev. 13, dated 02/03/83 designates El. -1.50 ft. as Top of Pier, 4.5 feet below the tangent line of the cylindrical shell and the ellipsoidal base of the Containment Vessel.

Ebasco Calculation 1352.063 (Reference 13) assumes that the base of the Containment Vessel is flat, and computes the safety factors against uplift, sliding and overturning due to the effects of E-W DBE, vertical DBE and buoyancy.

The factors of safety against uplift, sliding and overturning initially computed are 2.44, 2.51 and 6.77.

At a meeting held at Ebasco's New York office (see Section 2.0) it was agreed that Ebasco would revise the stability calculation to reflect the SRSS of the E-W and N-S DBE's, and to reduce the dead load of the Containment Vessel by the magnitude of the buoyant force.

The Revision 1 calculation, dated 07/28/83, computes revised factors of safety against sliding and overturning of 1.17 and 3.16.

In order to confirm the stability of the Containment Vessel, a simple stability model was formulated by HEA (Appendix L) which takes the curvature of the base of the Containment Vessel into account.

This stability model is formulated on the basis of two intrinsic properties of the ellipsoidal base of the Containment Vessel: that sliding and translation of the base of the Containment Vessel with respect to the mass concrete support cannot be uncoupled, and that any displaced configuration of the base of the Containment Vessel will result in "two-point" contact (points designated 'j' and 'k' on page 3 of the Appendix L calculation). The latter assumption derives from the fact that the radius of curvature of the ellipsoidal base of the Containment Vessel is not a constant.

As shown in the computation, the critical stability mode for the Containment Vessel is overturning and not sliding. The factor of safety computed against overturning is 2.34.

HEA therefore confirms the stability of the Containment Vessel under the action of the postulated earthquake and buoyancy forces.

The HEA computation also confirms the structural adequacy of the underlying mass concrete supporting the Containment Vessel as shown in Detail "B" of Ebasco Drawing LOU-1564-G-502, Rev. No. 6, dated 12/17/78.

Factors of safety against uplift, sliding and overturning were also computed for the Shield Building with respect to the top of the Mat. The respective factors of

safety calculated were 3.23, 1.35 and 1.32, which do not take into account the additional shear and axial restraint that would be generated by the reinforcing steel tieing the Shield Building and the Mat together.

HEA therefore additionally confirms the stability of the Shield Building with respect to the top of the mat.

10.0 Conclusions and Recommendations

10.1 Containment Vessel

The steel Containment Vessel is seated on a concrete dish. If it is assumed that hydrostatic pressure develops on the interface between the bottom head of the Containment Vessel and the supporting fill concrete, there would be a reduction in stability. Calculations were performed which indicate a more than adequate margin of safety. Therefore, it can be concluded that the cracking and seepage in the foundation mat could extend into the supporting fill concrete without causing any concern about the Containment Vessel stability.

Quite independent of the cracking in the foundation mat, some surface corrosion was noted on the lower cylindrical portion of the containment vessel. This surface corrosion has not affected the strength of the Containment Vessel. However, this surface corrosion should be cleaned and the steel protected to prevent future corrosion.

10.2 Foundation Mat

While certain difficulties were encountered during construction and procedural changes were made, they were resolved in a controlled manner so that there were no adverse effects upon the structural integrity of the foundation mat.

Cracks in the mat were reported in 1977 and again in 1983. However, it is likely that the cracks reported in 1983 were in existence for some time but were only noticed in 1983. In fact, if it weren't for the moisture associated with the cracks, the cracks might not have been noticed at all. The extent of cracking is minor and is certainly within expectations for a structure of this type. The specific causes of the cracks are probably a

combination of temperature effects, drying shrinkage and differential soil settlements under imposed loads.

While the cracking can be considered minor, the seepage of water through the foundation mat contrasted with statements that the foundation mat was a "watertight barrier". However, the limited amount of water seepage does not invalidate the fundamental assumption that the foundation mat can support and maintain the imposed hydrostatic pressure of the groundwater.

It was also determined that there is a self sealing of the cracks by the leachate. The leachate has two major components; calcium carbonate and magnetic iron. This magnetic iron is probably magnetite, Fe_3O_4 which is the passivating oxide which forms on and protects the steel embedded in the concrete from rusting. The water taken from a crack is not very dissimilar to water taken from the ground surrounding the foundation mat. In neither case is the water considered aggressive.

Furthermore, visual inspections of cracks reveal no evidence of rusting. If corrosion of reinforcing bars in the concrete were a problem it would be expected that the cracking would be extensive. This is because corrosion products of iron occupy a much larger volume than that of the iron. The resulting expansive forces would cause additional cracking and open up existing cracks and a rust discoloration would appear. The inspection and testing revealed no indications of such a corrosion process.

As a matter of fact, the cracking in the foundation is minor and there are no corrosive agents within the NPIS nor are any expected in the future. Therefore, there is no need to perform a program of crack repair or periodic inspection. Indeed, the leachate appears to provide

for a self sealing process.

While the laboratory test results indicated that there was iron in the leachate, the sample of pit water indicates virtually no iron. This strongly suggests that the iron is not currently waterborn and therefore is not now coming from the reinforcing bars. While the source of the iron is not known, it probably occurred over the past seven years of construction. Possible sources include pipe threading and sweeping of the floor with steel bristled brooms.

In conclusion, there is no evidence of any process which has been or could be detrimental to the structural integrity of the foundation mat.

REFERENCES

1. HEA Trip Report No. 1, W3-HE-LP-001, July 15, 1983.
2. HEA Trip Report No. 2, W3-HE-LP-002, August 1, 1983.
3. HEA Trip Report No. 3, W3-HE-LP-003, August 22, 1983.
4. HEA Trip Reports Nos. 4 & 5, W3-HE-LP-004, August 24, 1983.
5. HEA Trip Report No. 6, W3-HE-LP-006, September 6, 1983.
6. Foundation Design of the Waterford Nuclear Plant, by J. L. Ehasz and E. Radin, December, 1973.
7. Review of Site Settlements, by M. Pavone and J. L. Ehasz, September, 1978.
8. RCB Foundation Crack Map, Ebasco Drawing No. SK 1564-4.1-G-28, August 17, 1977.
9. Ebasco Letter Doc: CH-039-77, File: 6Q-R-4, July 27, 1977.
10. Ebasco Nonconformance Report W3NCR-16143, May 27, 1983.
11. WSES-FSAR-UNIT-3, Section 11.2, Liquid Waste Management System.
12. Compatibility of Large Mat Design to Foundation Conditions, by J. L. Ehasz and P.-C. Liu
13. Ebasco Calculation OFS No. 1352.063, Steel Containment Stability, Rev. 1, July 28, 1983.
14. Steel Corrosion in Concrete, by D. A. Hausmann, Materials Protection, November, 1967, pp. 19-23.
15. The Mechanism of Steel Corrosion in Concrete Structures, by C. T. Ishikawa and B. Bresler, Materials Protection, March, 1968, pp. 45-47.

16. Mechanisms of Corrosion of Steel in Concrete, by G. J. Verbeck, ACI Publication SP-49, June, 1975, pp. 21-38.

17. Criteria for Cathodic Protection of Steel in Concrete Structures, by D. A. Hansmann, Materials Protection, October, 1969, pp. 23-25.

18. Cathodic Protection of Steel in Concrete, by R. C. Robinson, ACI Publication SP-49, June, 1975, pp. 83-93.

19. Ebasco Specification Concrete Masonry, Project Identification No. LOU-1564.472, Issue Date: December 31, 1971.

APPENDIX A

Basemat Crack Maps

APPENDIX A NOTES

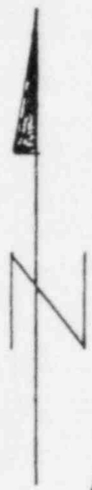
The basemat crack maps contained in Appendix A indicate the extent and orientation of cracks observed at the surface of the basemat.

"NOTES" on Basemat Crack Maps additionally indicate:

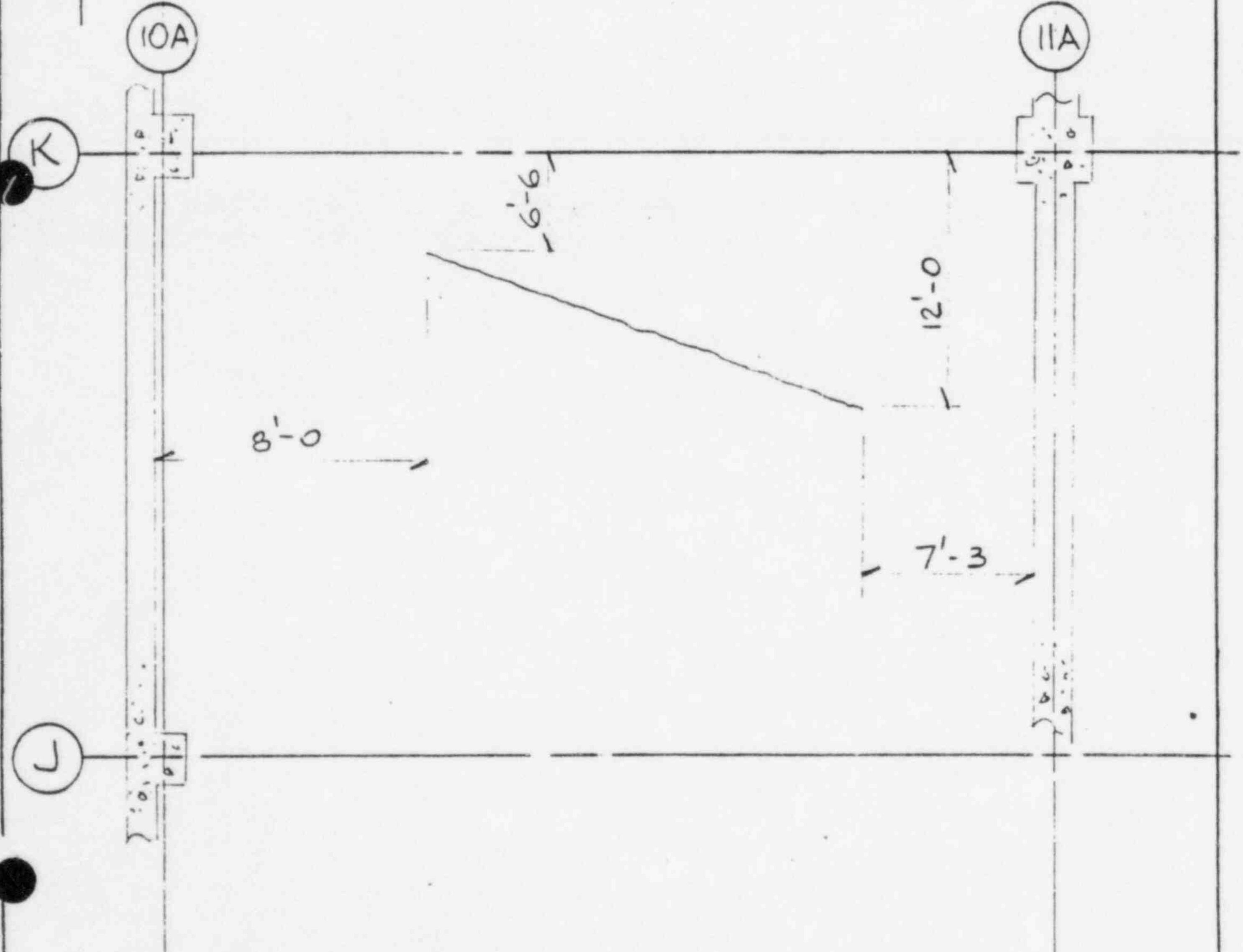
1. Any prior repair to the crack
2. Presence of floor finish
3. Evidence of dampness or seepage
4. Crack width

HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

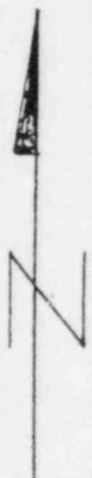
PROJECT	
CLIENT	LP&L
SUBJECT	BASEMAT CRACK MAP
PROJ. NO.	C- - -
SUBJ. SUBDIV. SHEET	
PREP. BY	DATE 8/31/83
CHCKD. BY AdB	DATE 08/31/83



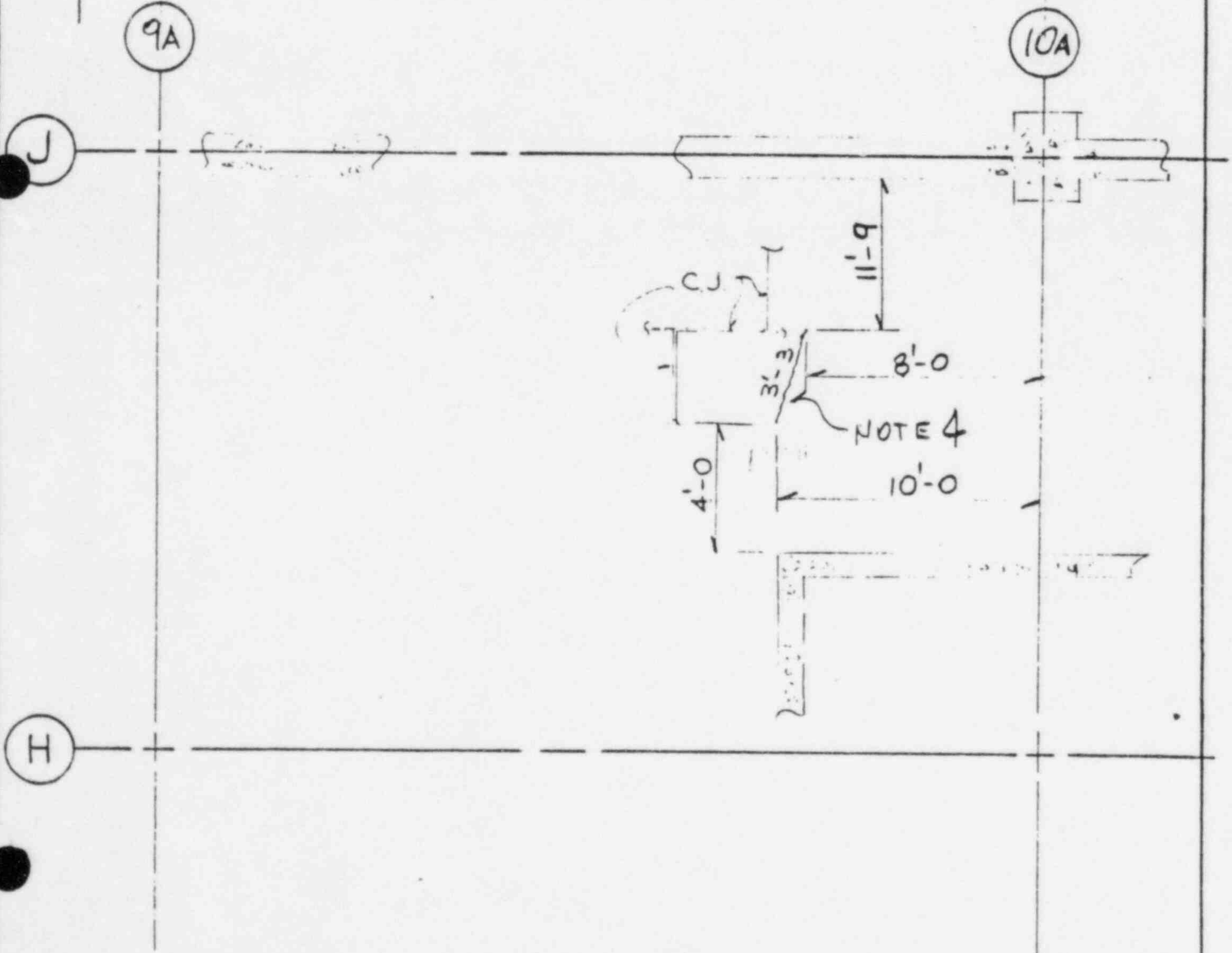
NOTES:
1. NO
2. NO
3. SLIGHT AMOUNT - 3'-0 LENGTH NEAR WEST END.
4. HAIRLINE CRACK



HEA HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. C- – – SUBJ. SUBDIV. SHEET
PROJECT CLIENT [illegible] SUBJECT BASEMAT CRACK MAP	PREP. BY [illegible] DATE 8/31/83 CHCKD. BY AdB DATE 08/31/83

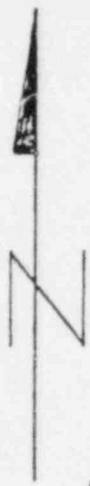


NOTES: 1. NO
2. YES FROM CRACK BEGINNING - SOUTH (SHADED)
3. NO
4. THIS AREA CLEANED FOR INSPECTION. CRACK MAY EXTEND BEYOND NOTED AREA UNDER FLOOR PREPARATION.
5. HAIRLINE CRACK

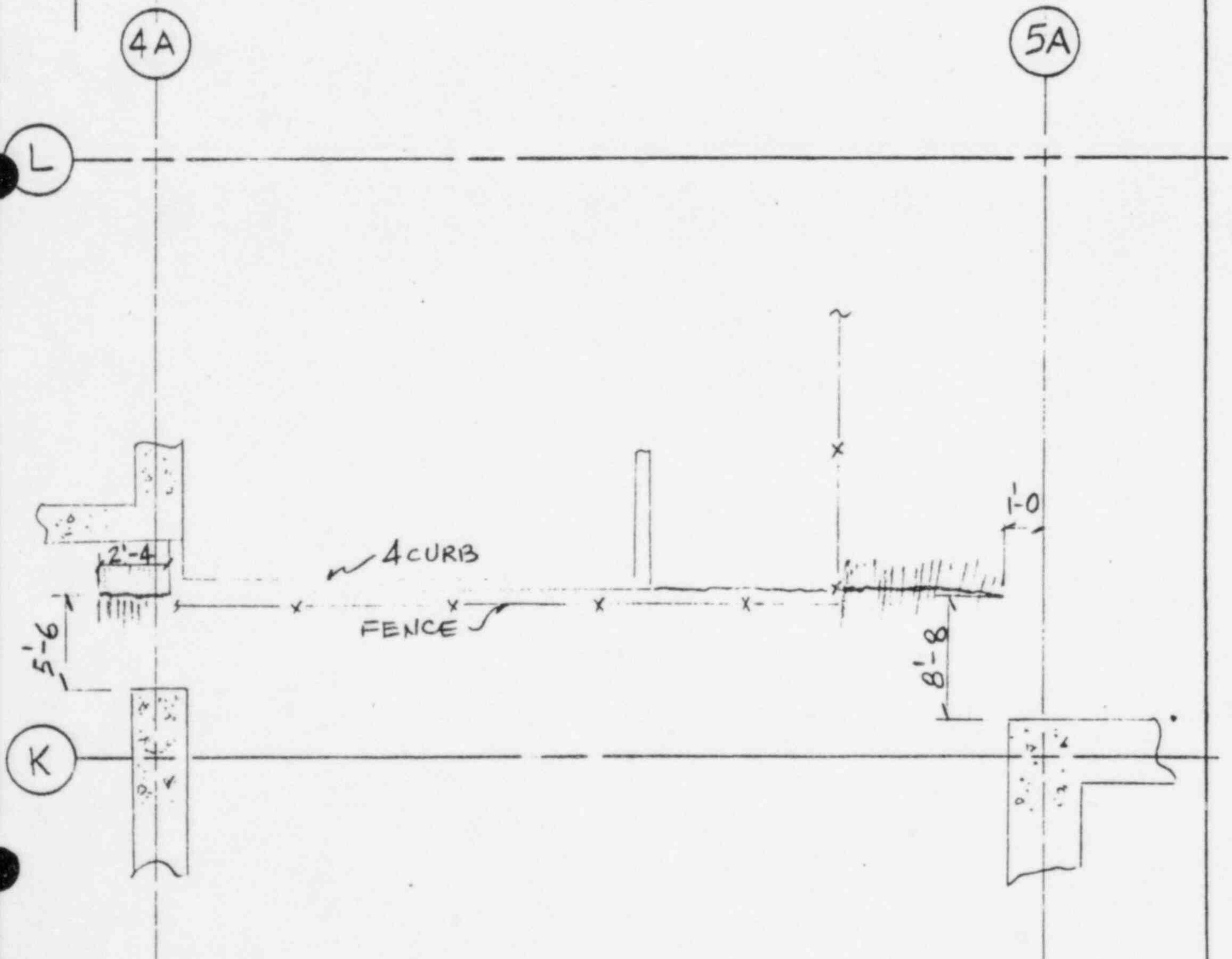


HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	C- – –
SUBJ. SUBDIV. SHEET	
PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP
PREP. BY	[illegible] DATE 8/31/83
CHCKD. BY	AdL DATE 08/31/83



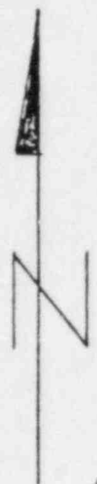
NOTES: 1. EXPOSED AREA - NO
2. YES - SHADED AREA
3. NO
4 UNABLE TO DETERMINE IF CRACK IS UNDER CURB
5. HAIRLINE CRACK



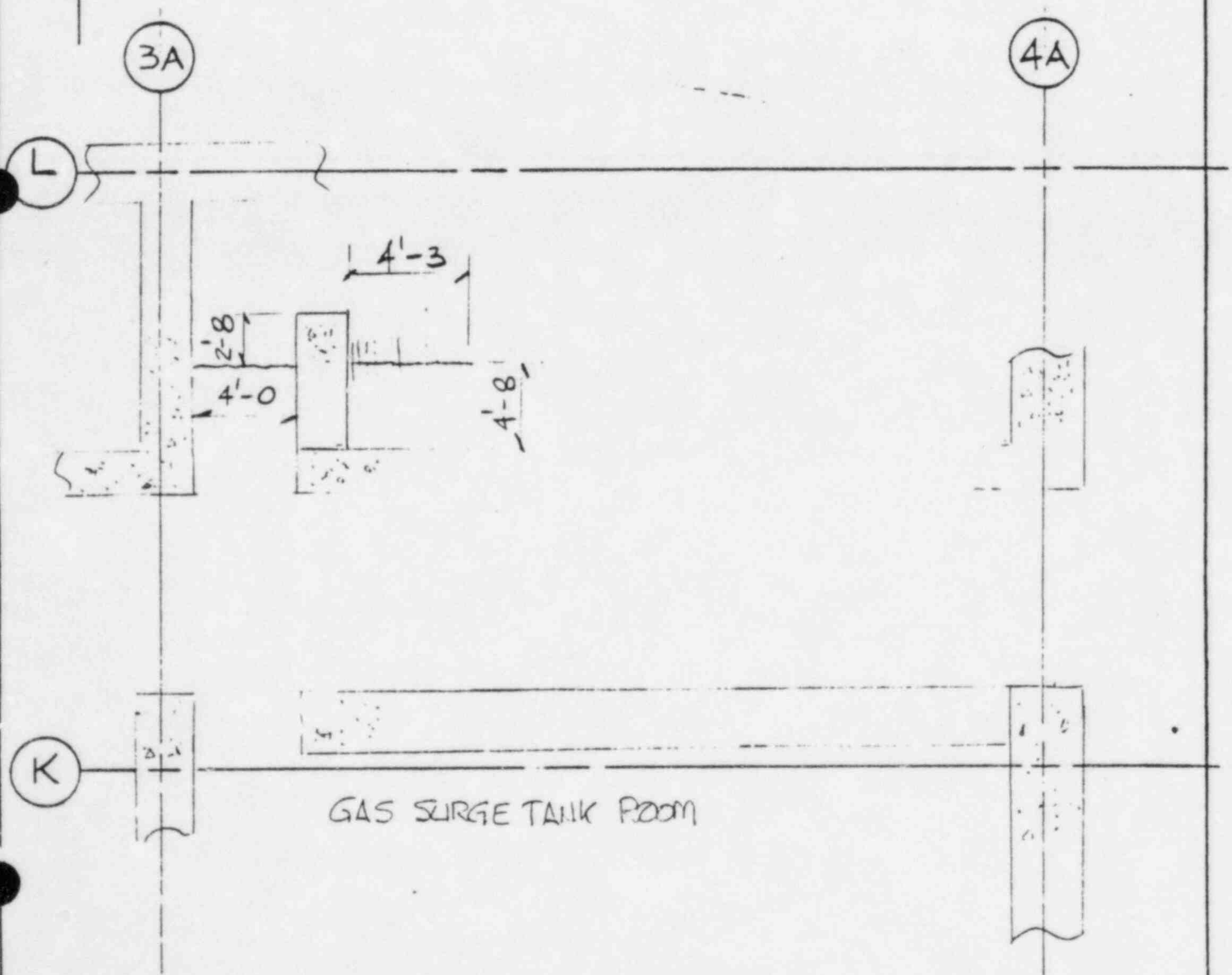
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	LP&L
SUBJECT	BASEMAT CRACK MAP

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY [signature]	DATE 8/31/83	
CHCKD. BY AdB	DATE 08/31/83	

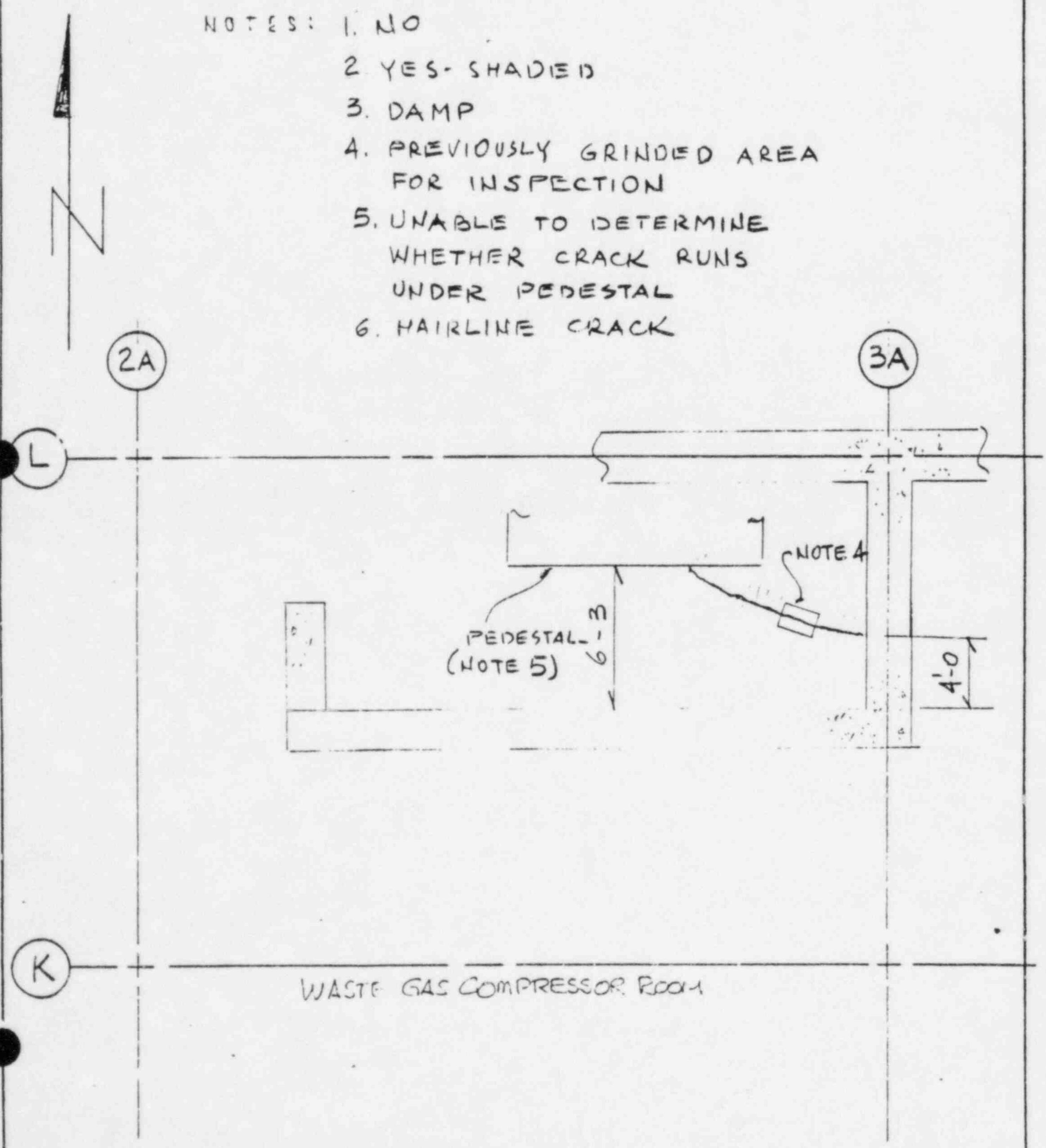


NOTES: 1. NO
2. YES - SHADED AREA
3. DRY - SHADED AREA
DAMP - UNSHADED AREA
4. HAIRLINE CRACK



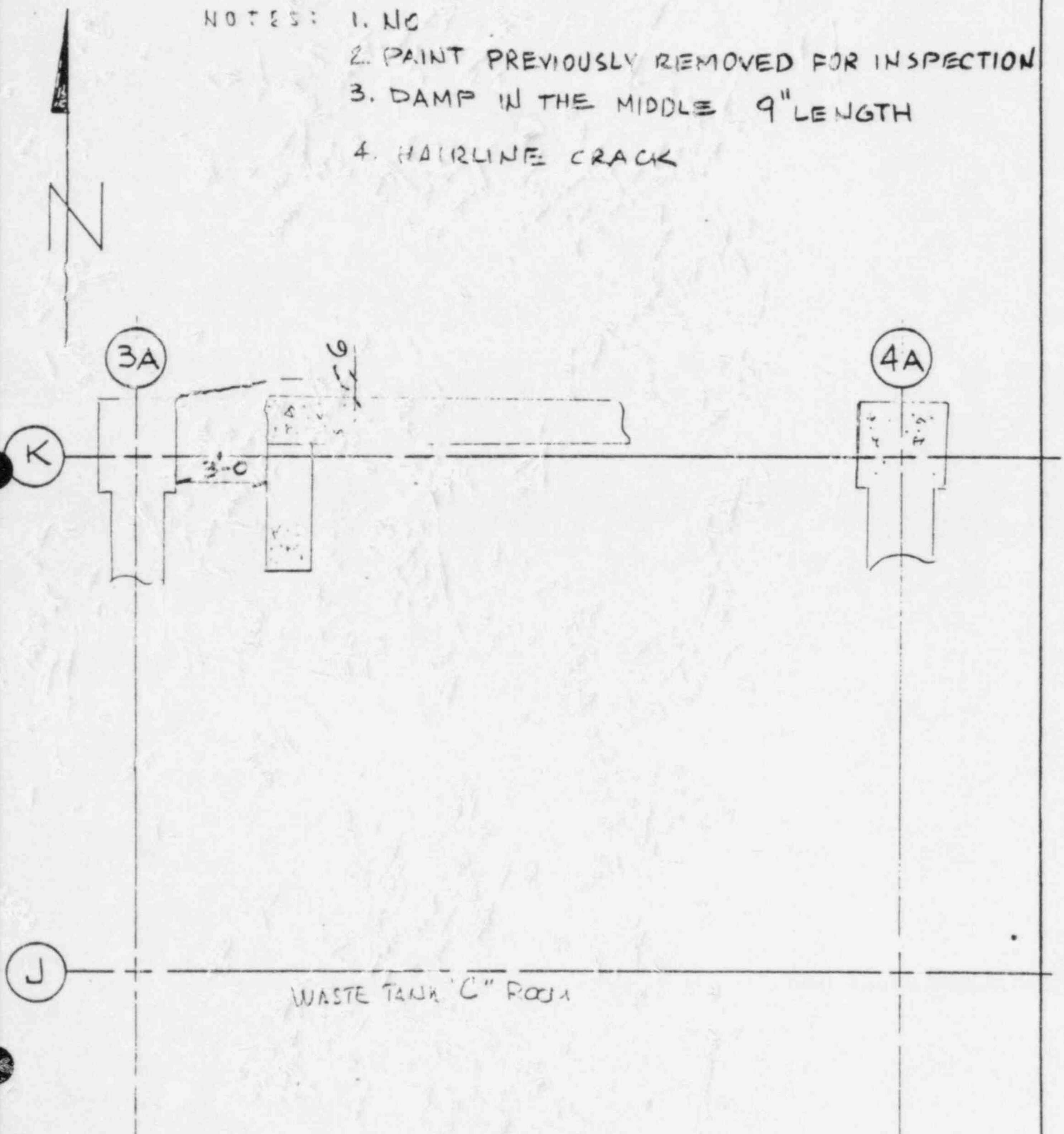
HEA	HARSTEAD ENGINEERING ASSOCIATES • INC.	PROJ. NO. C- - -
	169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	SUBJ. SUBDIV. SHEET
	PROJECT	PREP. BY [illegible] DATE 8/31/83
	CLIENT [illegible]	
	SUBJECT BASEMAT CRACK MAP	CHCKD. BY AdB DATE 08/31/83

NOTES:
1. NO
2. YES - SHADED
3. DAMP
4. PREVIOUSLY GRINDED AREA FOR INSPECTION
5. UNABLE TO DETERMINE WHETHER CRACK RUNS UNDER PEDESTAL
6. HAIRLINE CRACK



HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. C- - - SUBJ. SUBDIV. SHEET
PROJECT CLIENT [illegible] SUBJECT BASEMAT CRACK MAP	PREP. BY [signature] DATE 8/31/83 CHCKD. BY AdL DATE 08/31/83

NOTES: 1. NC
2. PAINT PREVIOUSLY REMOVED FOR INSPECTION
3. DAMP IN THE MIDDLE 9" LENGTH
4. HAIRLINE CRACK

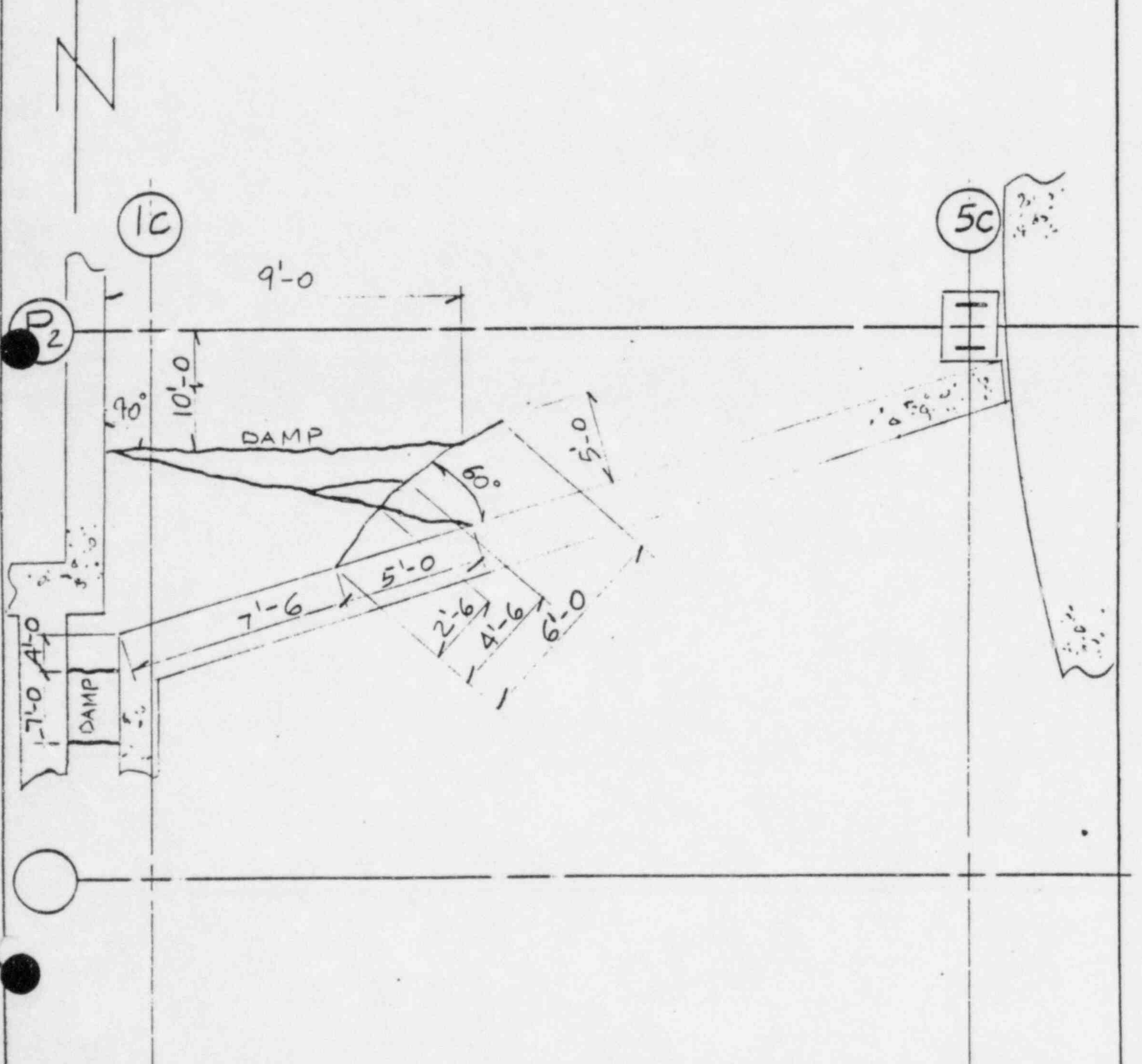


HEA	HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. C- - -
		SUBJ. SUBDIV. SHEET

PROJECT
CLIENT LP&L
SUBJECT BASEMAT CRACK MAP

PREP. BY IS/GW DATE 9/1/83
CHCKD. BY AdB DATE 09/02/83

NOTES: 1. NO
2. NO
3. YES
4. HAIRLINE CRACK



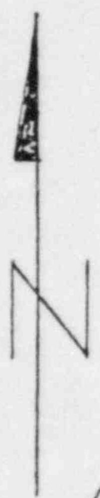
HEA

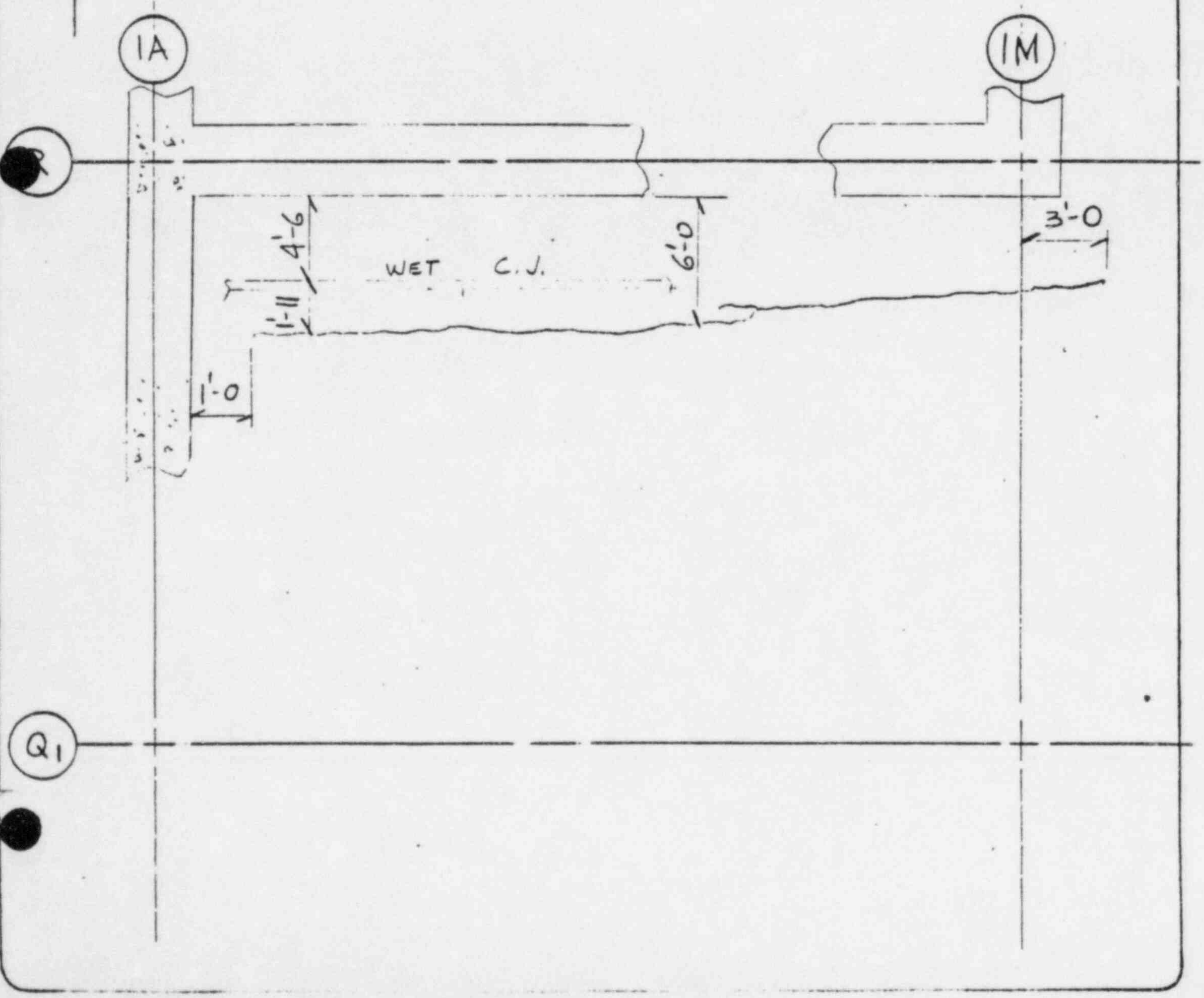
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	LPCL
SUBJECT	BASEMAT CRACK MAP

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY KR/GW	DATE 9/1/83	
CHCKD. BY AdB	DATE 09/02/83	

NOTES: 1. REPAIR ATTEMPTED
2. NO FLOOR FINISH
3. SEEPAGE EXISTS
4 HAIRLINE CRACK





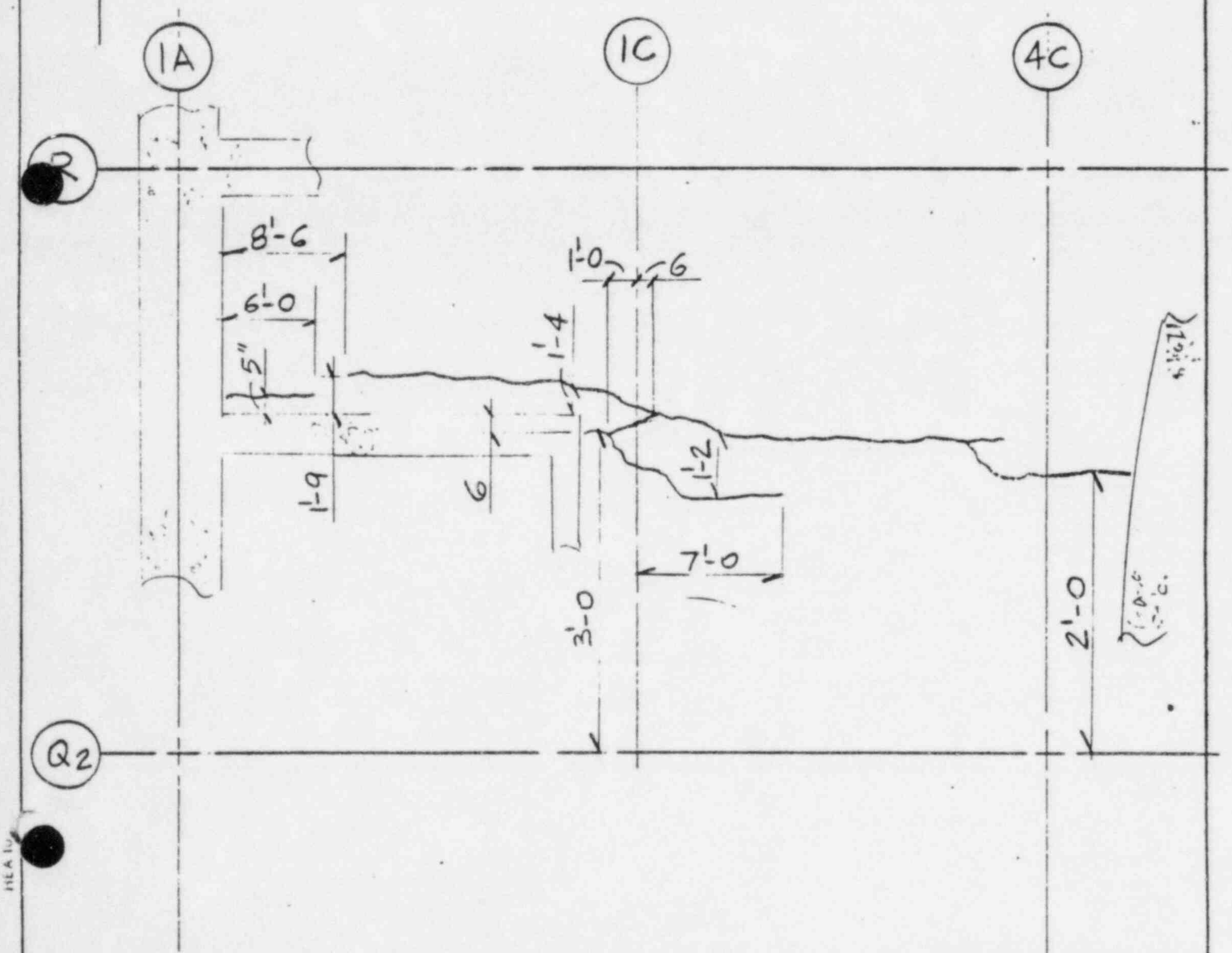
HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	LPL
SUBJECT	BASEMAT CRACK MAP

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY RR/GW	DATE 9/1/83	
CHCKD. BY AAB	DATE 09/02/83	

NOTES:
1. REPAIR ATTEMPTED
2. NO FLOOR FINISH
3. NO EVIDENCE OF SEEPAGE
4. HAIRLINE CRACK

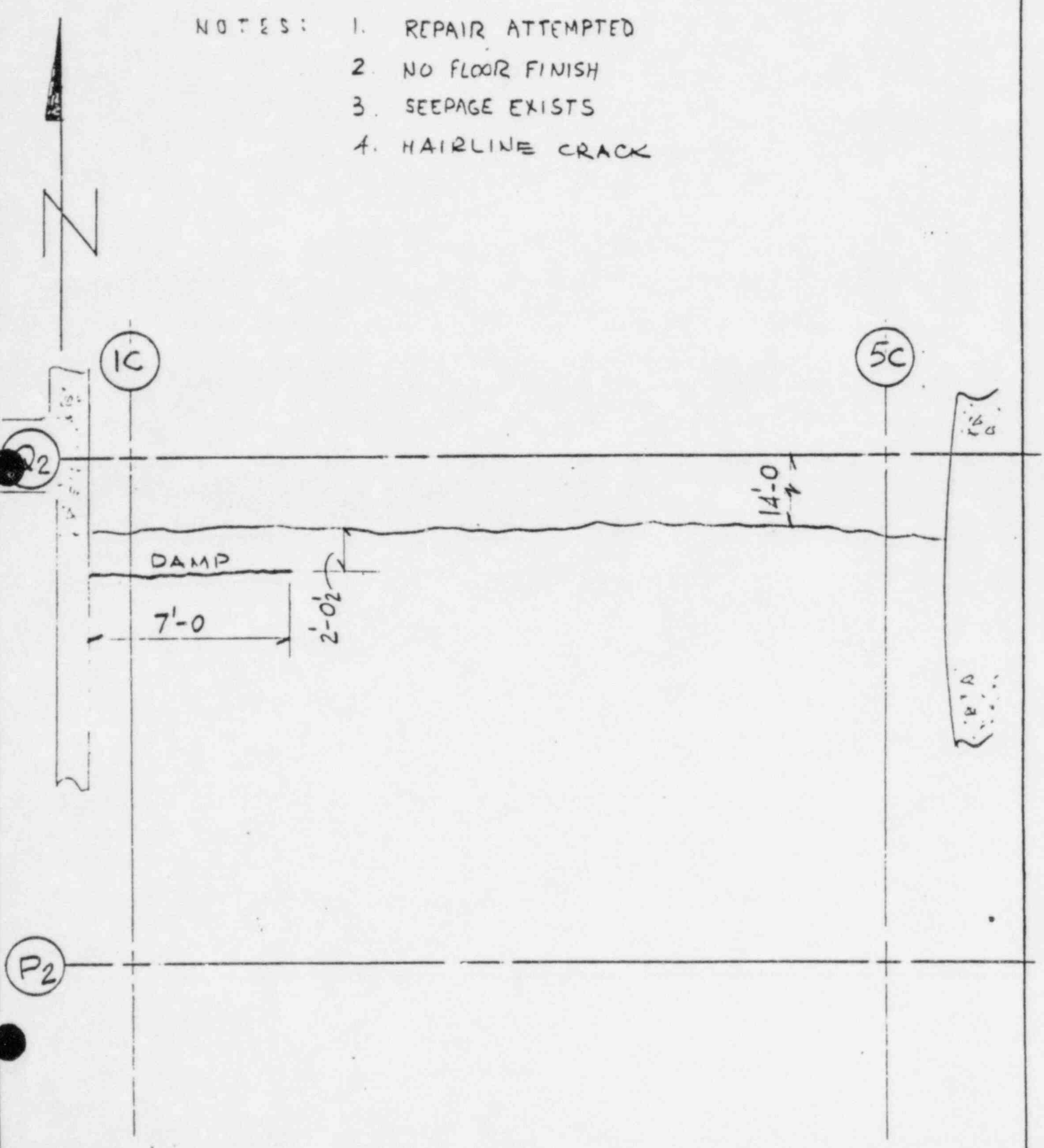
N



H E A HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	PROJ. NO. C- – –
CLIENT [illegible]	SUBJ. SUBDIV. SHEET
SUBJECT BASEMAT CRACK MAP	PREP. BY RR/GN DATE 9/1/83
	CHCKD. BY AdB DATE 09/02/83

NOTES: 1. REPAIR ATTEMPTED
2. NO FLOOR FINISH
3. SEEPAGE EXISTS
4. HAIRLINE CRACK



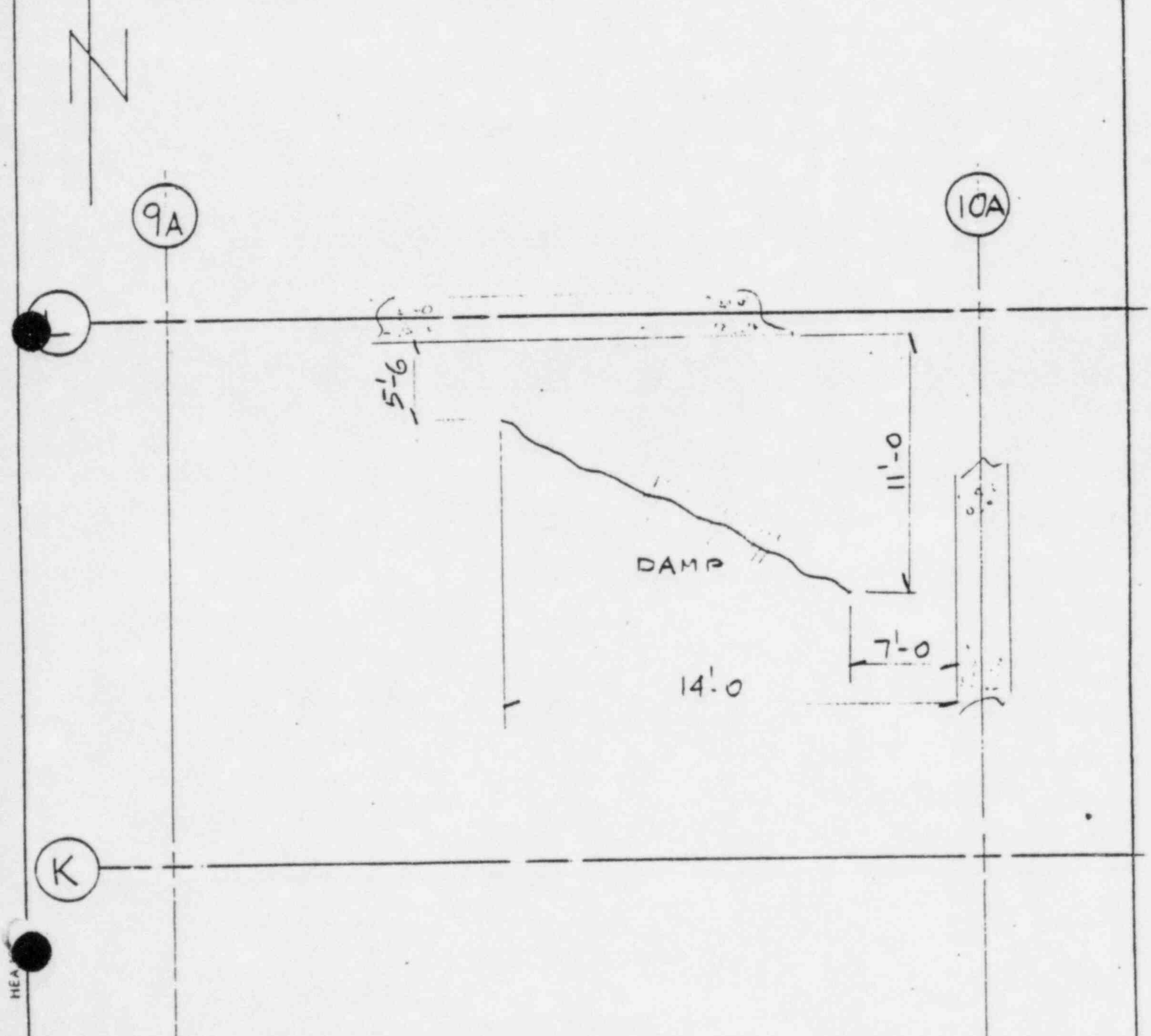
HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY I.S/GW	DATE 9/1/83	
CHCKD. BY AdB	DATE 09/02/83	

PROJECT
CLIENT [illegible]
SUBJECT BASEMAT CRACK MAP

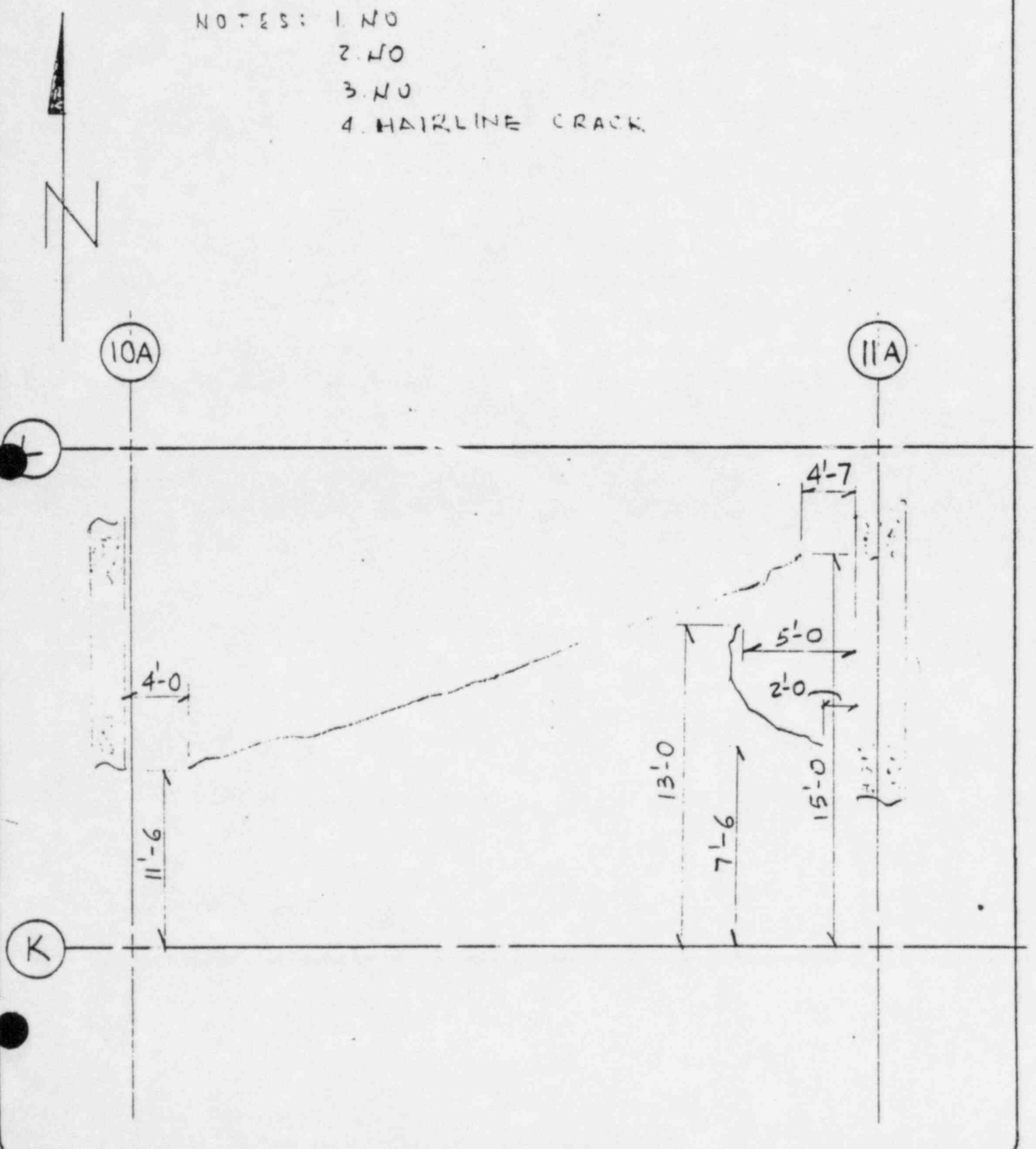
NOTES: 1. NO
2. NO
3. NO
4. HAIRLINE CRACK



HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	C- – –
SUBJ. SUBDIV. SHEET	
PROJECT	
CLIENT	GPU
SUBJECT	BASEMAT CRACK MAP
PREP. BY	GW
DATE	9/1/83
CHCKD. BY	AdB
DATE	09/02/83



HEA 102

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

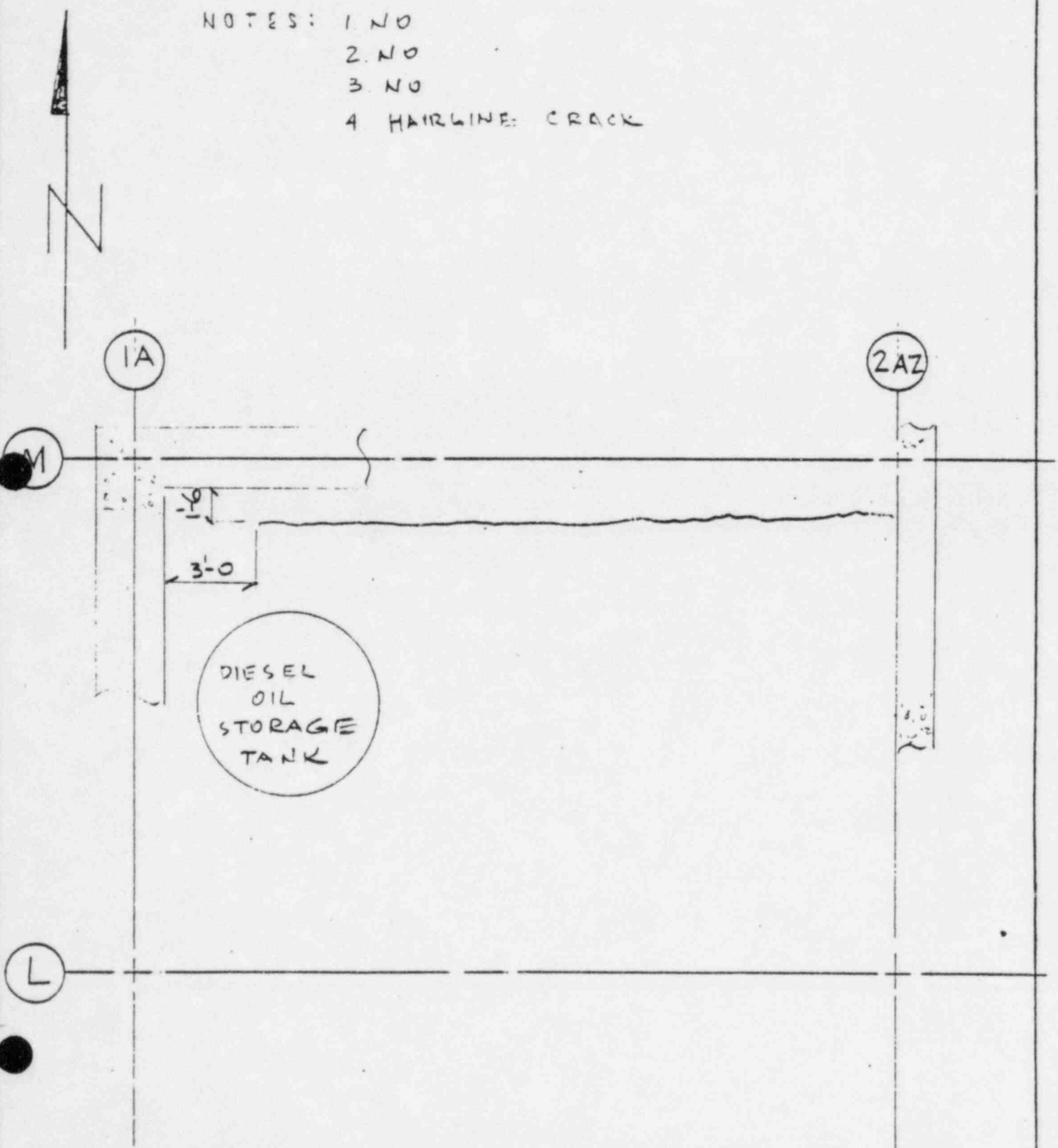
PROJ. NO.
C- – –

SUBJ.	SUBDIV.	SHEET

PROJECT
CLIENT [illegible]
SUBJECT BASEMAT CRACK MAP

PREP. BY IS/GW DATE 9/1/83
CHCKD. BY AdB DATE 09/02/83

NOTES: 1. NO
2. NO
3. NO
4. HAIRLINE CRACK

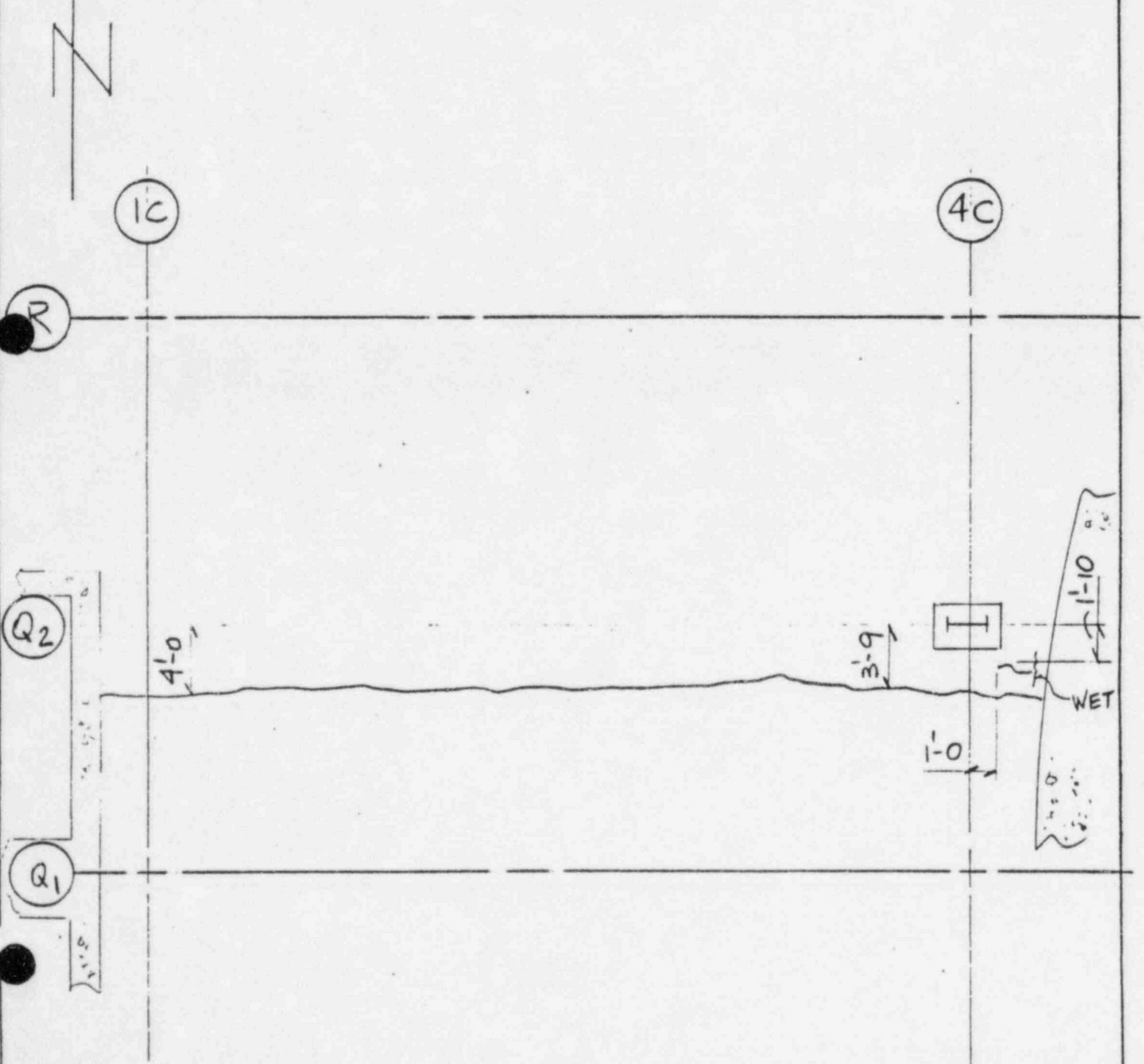


HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP

PROJ. NO. C- - -		
SUBJ.	SUBDIV.	SHEET
PREP. BY RR/GVI	DATE 9/1/83	
CHCKD. BY A.G.B.	DATE 09/02/83	

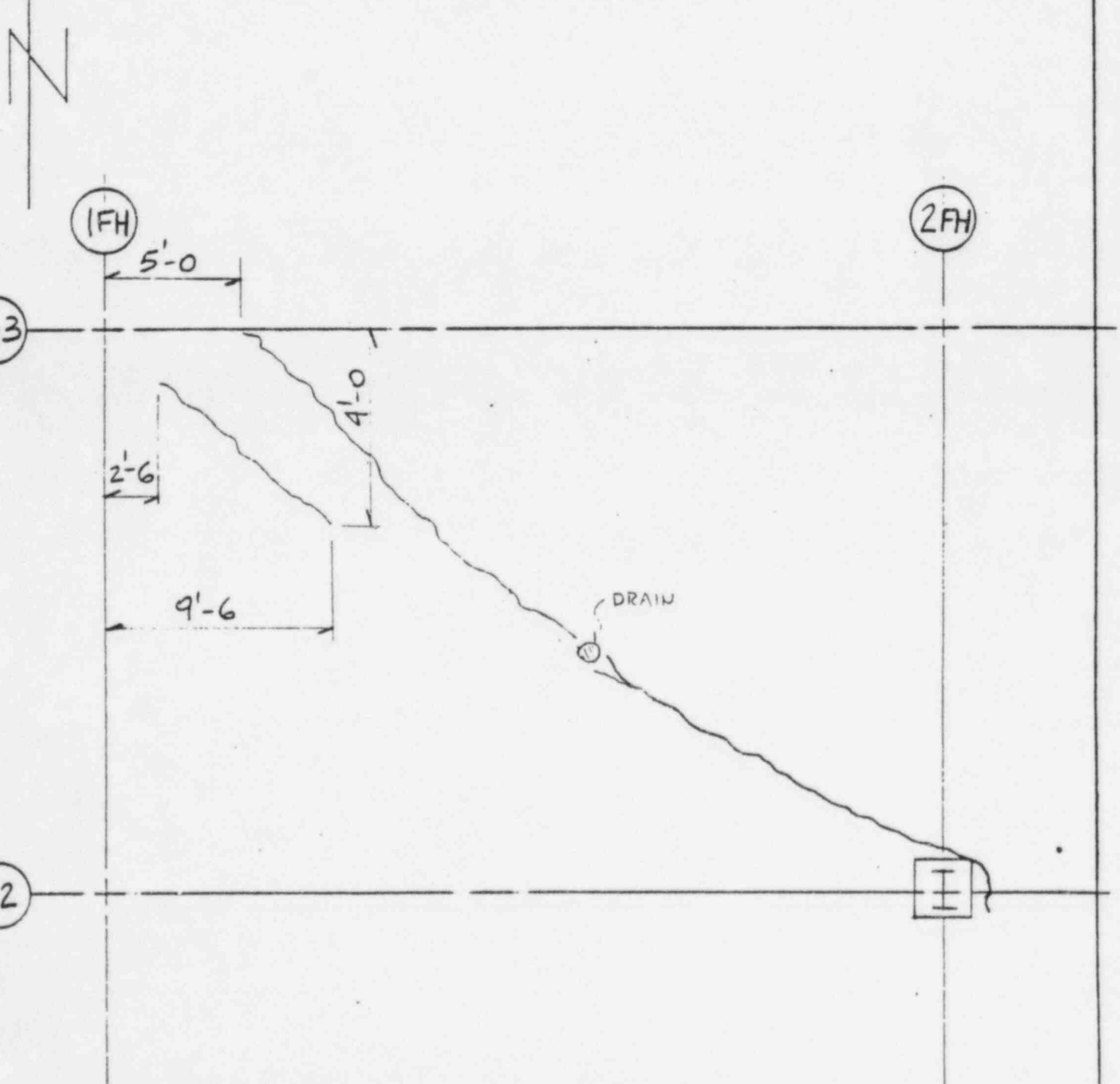
NOTES:
1. NO REPAIR
2. NO FLOOR FINISH
3. SEEPAGE AS INDICATED
4. HAIRLINE CRACK



HEA HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	PROJ. NO. C- – –
CLIENT [illegible]	SUBJ. SUBDIV. SHEET
SUBJECT BASEMAT CRACK MAP	PREP. BY RR/Gw DATE 9/1/83
	CHCKD. BY AdB DATE 09/02/83

NOTES:
1. NO REPAIR
2. NO FLOOR FINISH
3. SEEPAGE EXISTS
4. HAIRLINE CRACK

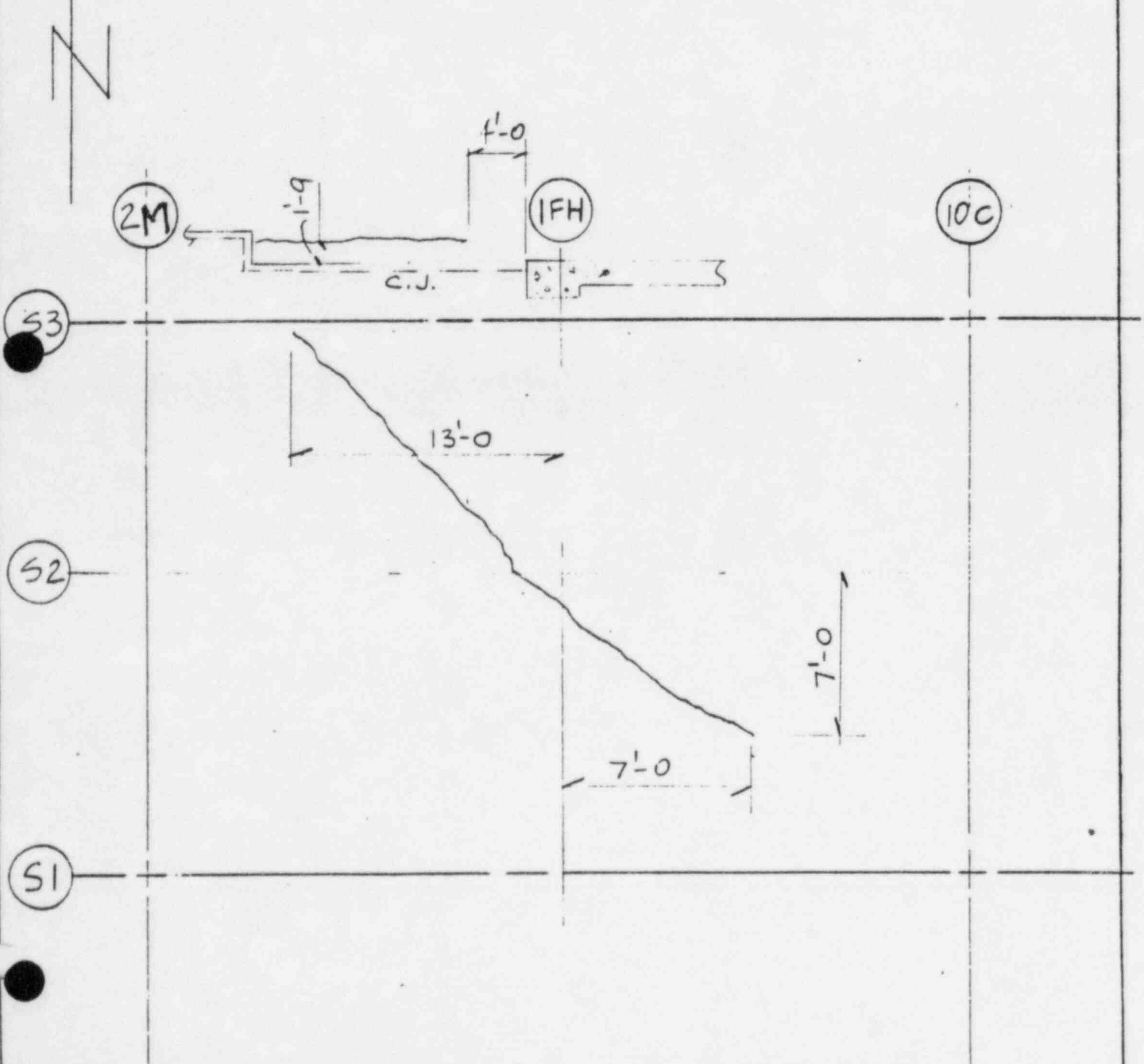


HEA

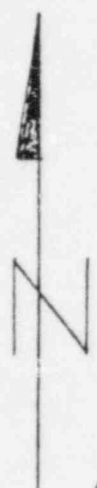
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	C- - -
SUBJ. SUBDIV. SHEET	
PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP
PREP. BY RR/GN	DATE 9/1/83
CHCKD. BY [illegible]	DATE 09/02/83

NOTES: 1. NO REPAIR
2. NO FLOOR FINISH
3. NO EVIDENCE OF SEEPAGE
4. HAIRLINE CRACK

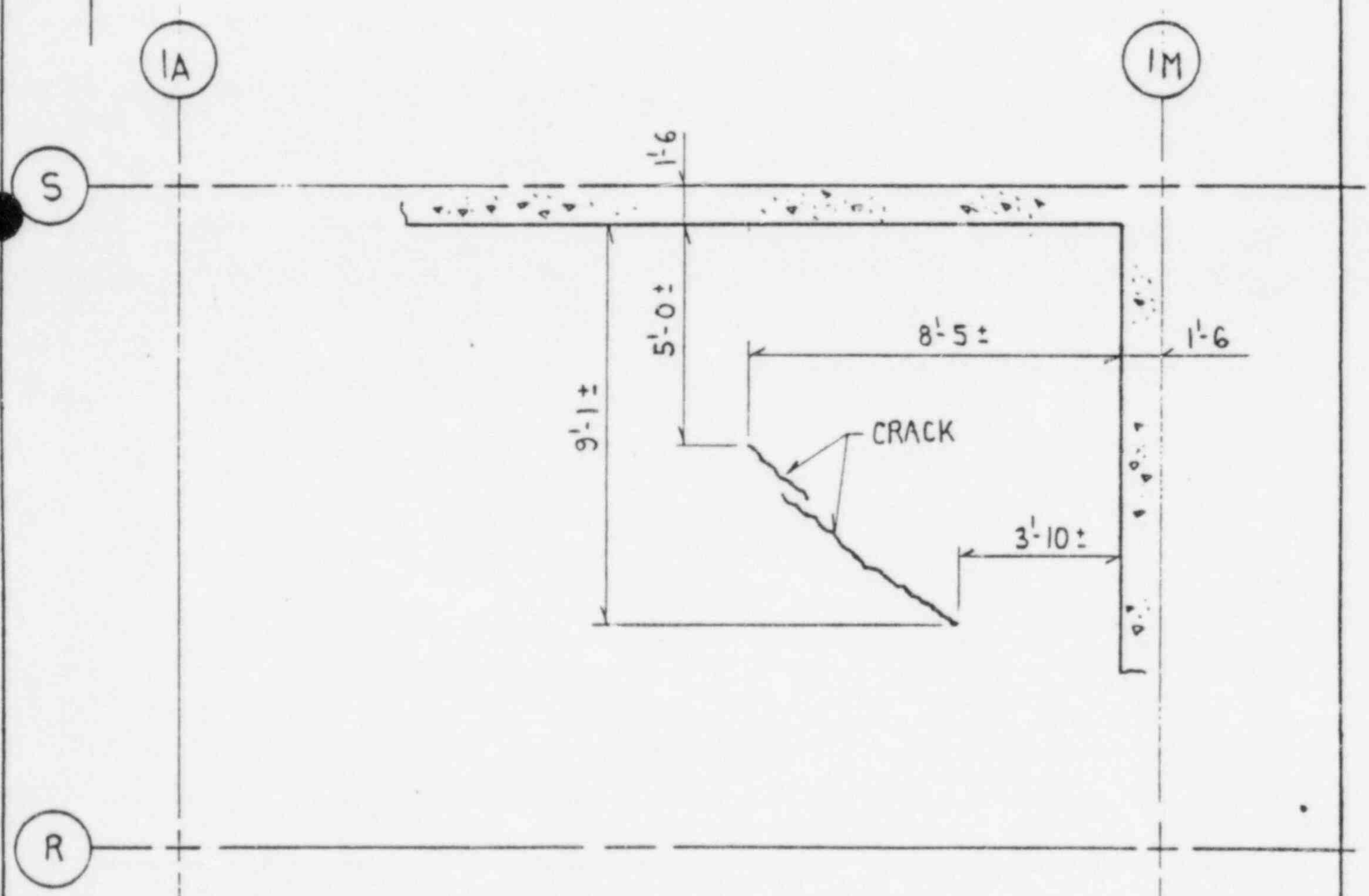


HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. C- – –
	SUBJ. SUBDIV. SHEET
PROJECT	PREP. BY RK/GW DATE 9/1/83
CLIENT LP&L	
SUBJECT BASEMAT CRACK MAP	CHCKD. BY AGB DATE 09/02/83



NOTES:

1. REPAIR ATTEMPTED
2. NO FLOOR FINISH
3. NO EVIDENCE OF SEEPAGE
4. HAIRLINE CRACK



PLAN EL - 34.75'

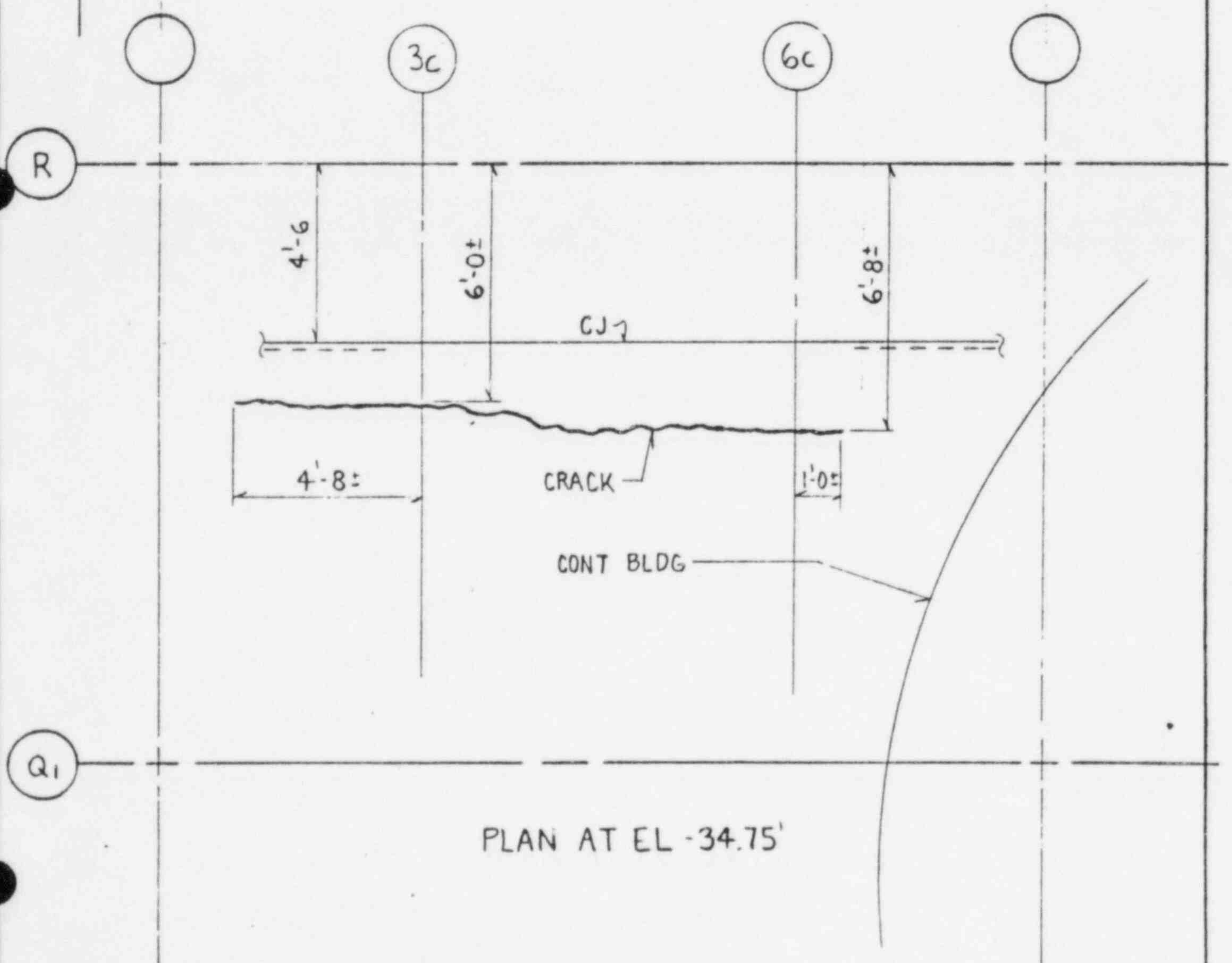
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY RR/EW	DATE 9/1/83	
CHCKD. BY AHB	DATE 09/02/83	

NOTES:

1. NO REPAIR
2. NO FLOOR FINISH
3. NO EVIDENCE OF SEEPAGE
4. HAIRLINE CRACK

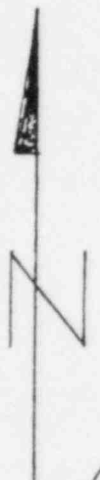


PLAN AT EL -34.75'

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

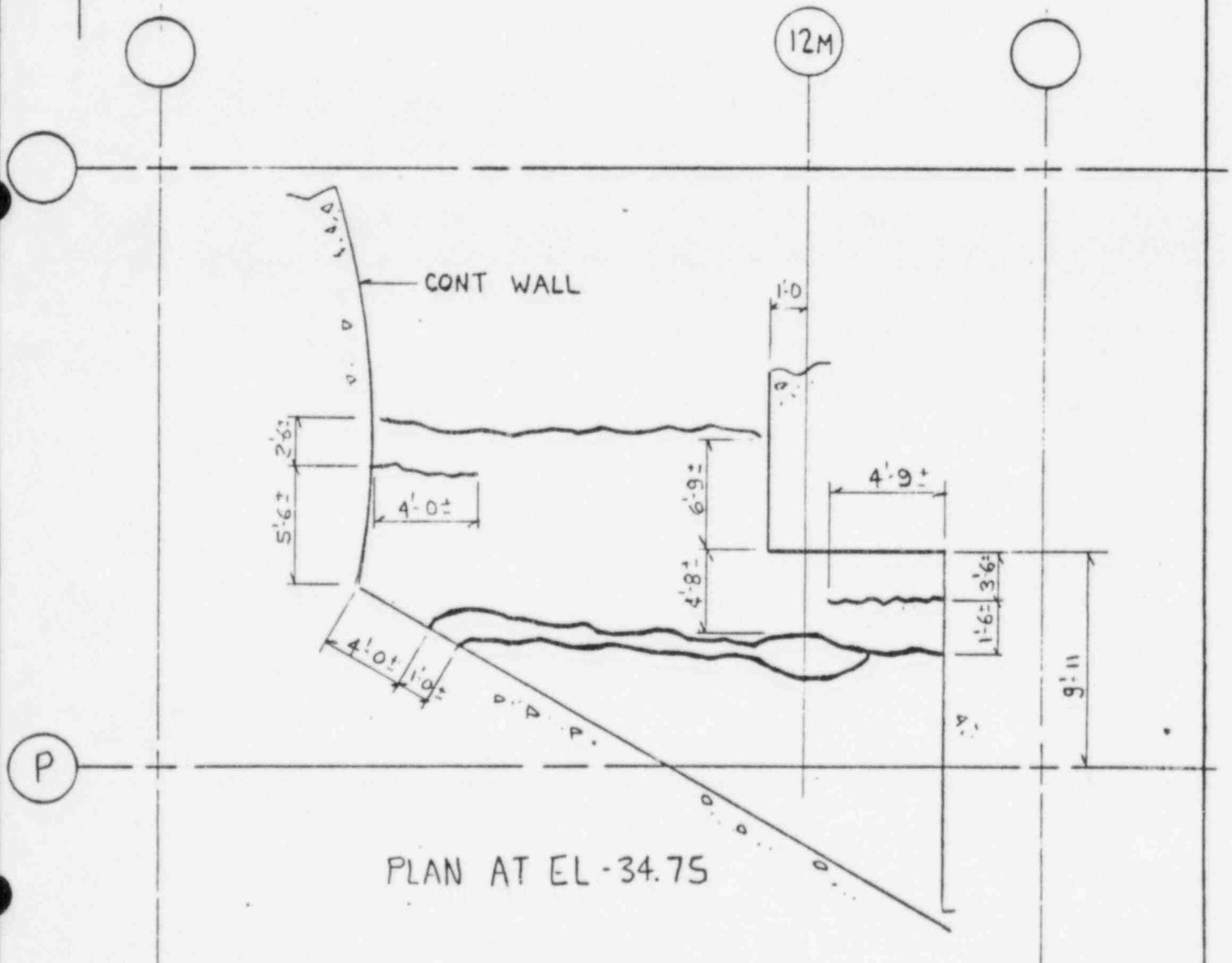
PROJECT	
CLIENT	LP&L
SUBJECT	BASEMAT CRACK MAP

PROJ. NO. C- – –		
SUBJ.	SUBDIV.	SHEET
PREP. BY RR/GW	DATE 9/1/83	
CHCKD. BY AGB	DATE 09/02/83	



NOTES:

1 NO REPAIR
2 NO FLOOR FINISH
3 SOME SEEPAGE
4 HAIRLINE CRACK



PLAN AT EL -34.75

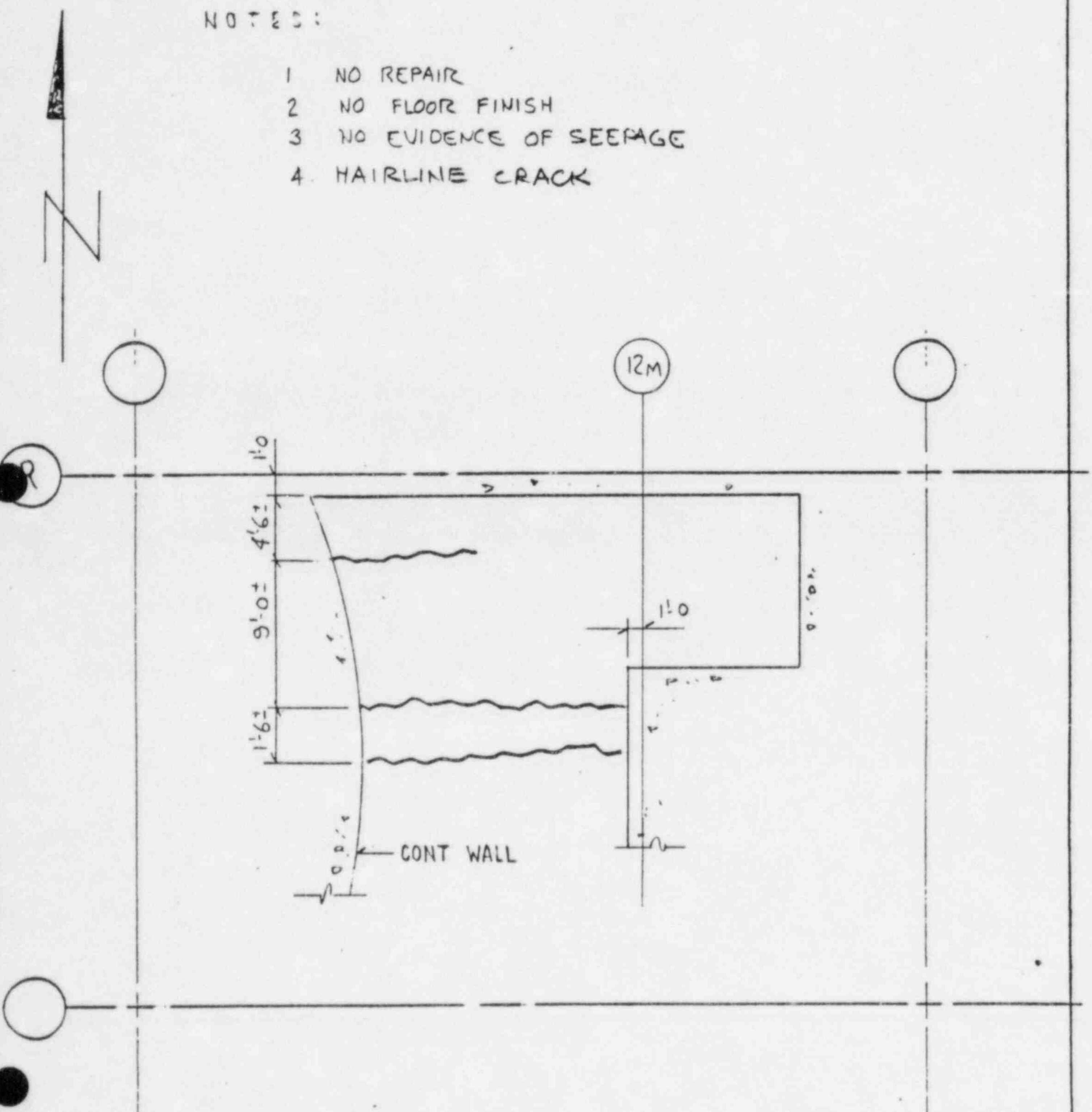
HEA **HARSTEAD ENGINEERING ASSOCIATES • INC.**
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY RR/GW	DATE 9/1/83	
CHCKD. BY AdB	DATE 09/02/83	

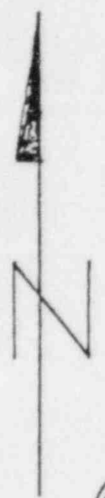
NOTES:

1 NO REPAIR
2 NO FLOOR FINISH
3 NO EVIDENCE OF SEEPAGE
4 HAIRLINE CRACK



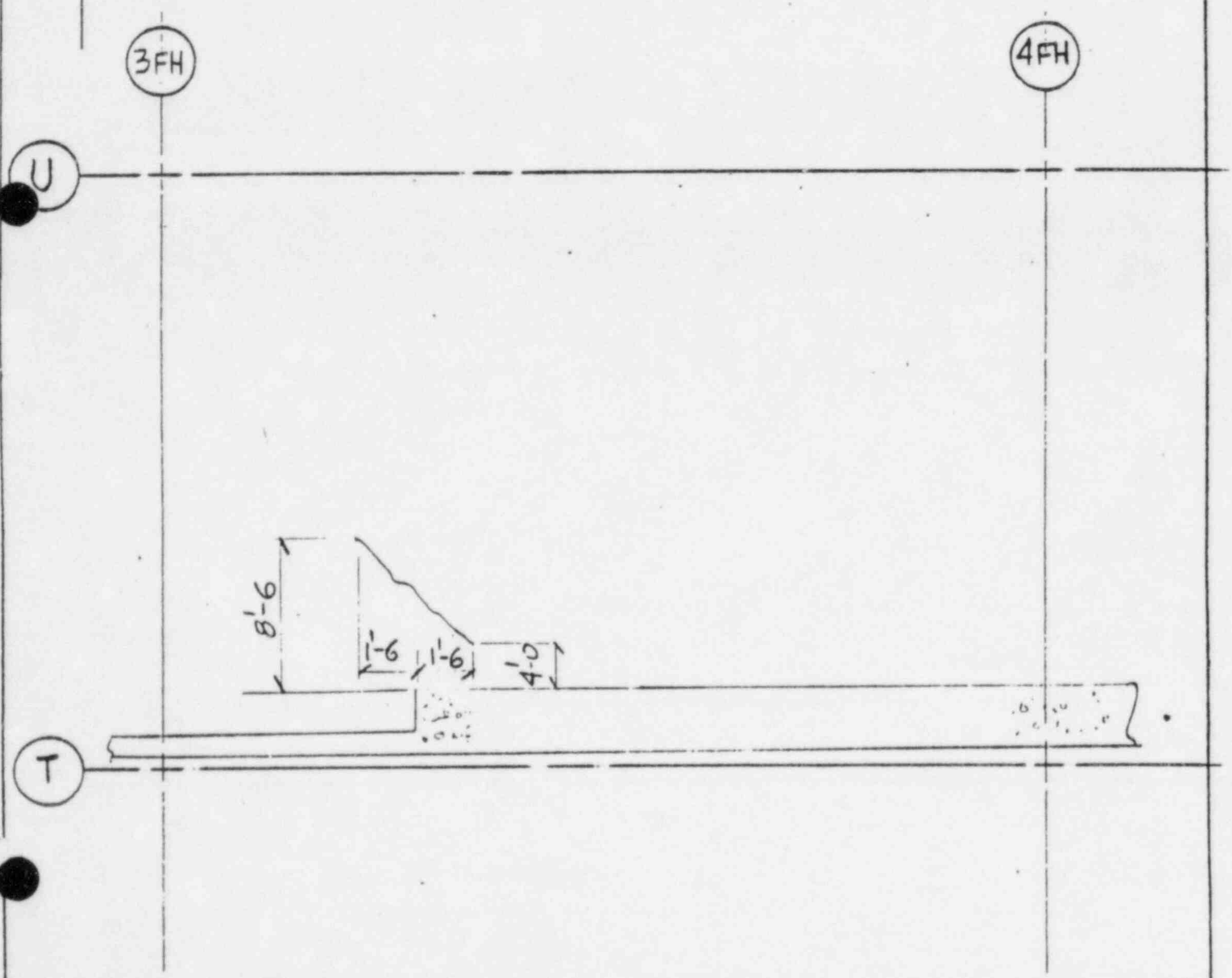
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. C- – –	SUBJ. SUBDIV. SHEET
PROJECT	
CLIENT [illegible]	PREP. BY RC/GW DATE 9/1/83
SUBJECT BASEMAT CRACK MAP	CHCKD. BY AdP DATE 09/02/83



NOTES:

1. NO REPAIR
2. FLOOR IS FINISHED
3. SEEPAGE EXISTS
4. HAIRLINE CRACK



HEA

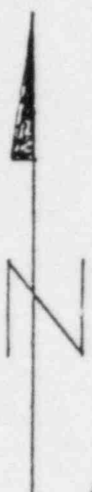
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	
C-	– –
SUBJ. SUBDIV. SHEET	
PREP. BY RR/GW	DATE 9/1/83
CHCKD. BY AAB	DATE 09/02/83

PROJECT

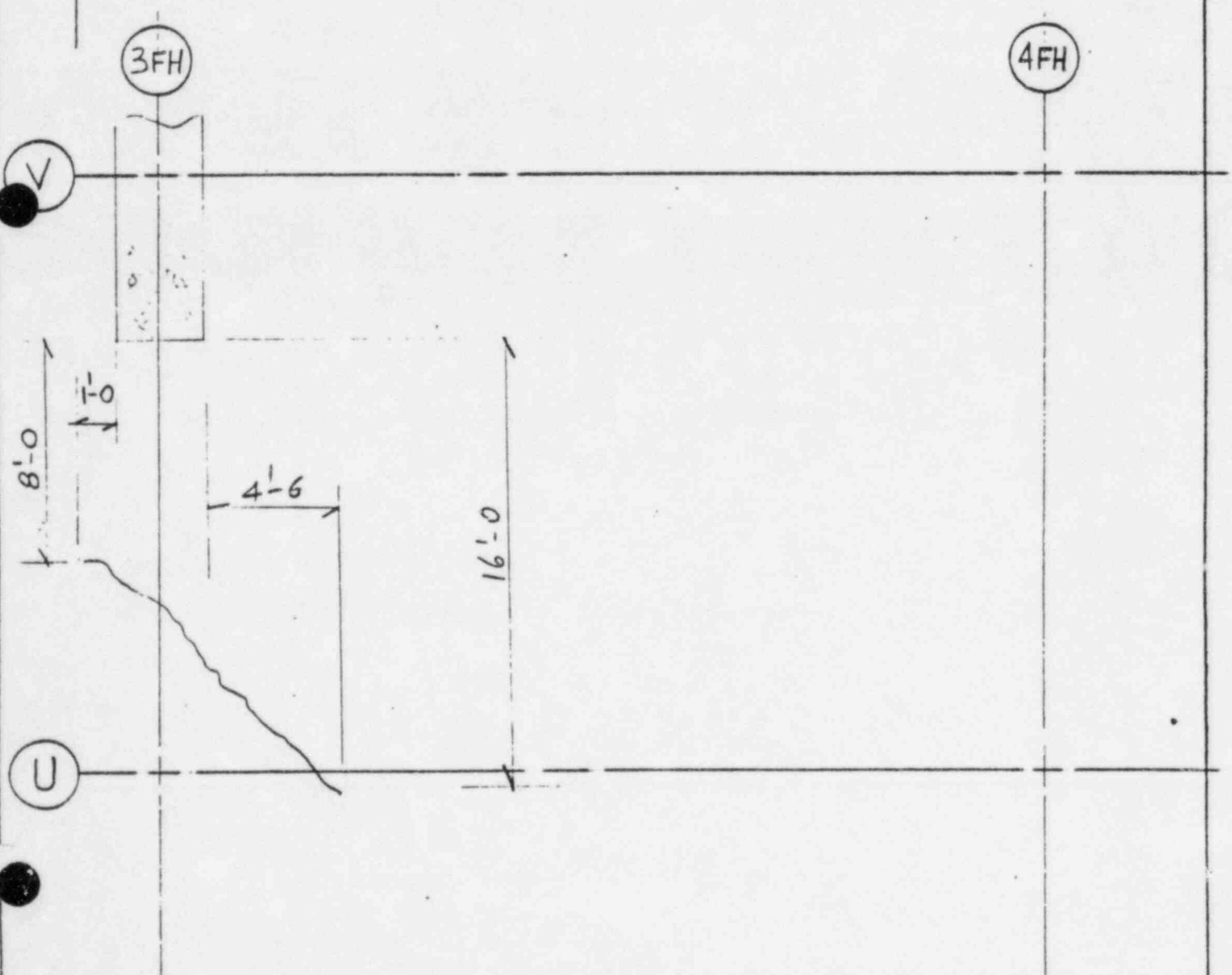
CLIENT LP&L

SUBJECT BASEMAT CRACK MAP



NOTES:

1. NO REPAIR
2. FLOOR IS FINISHED
3. SEEPAGE EXISTS
4. HAIRLINE CRACK

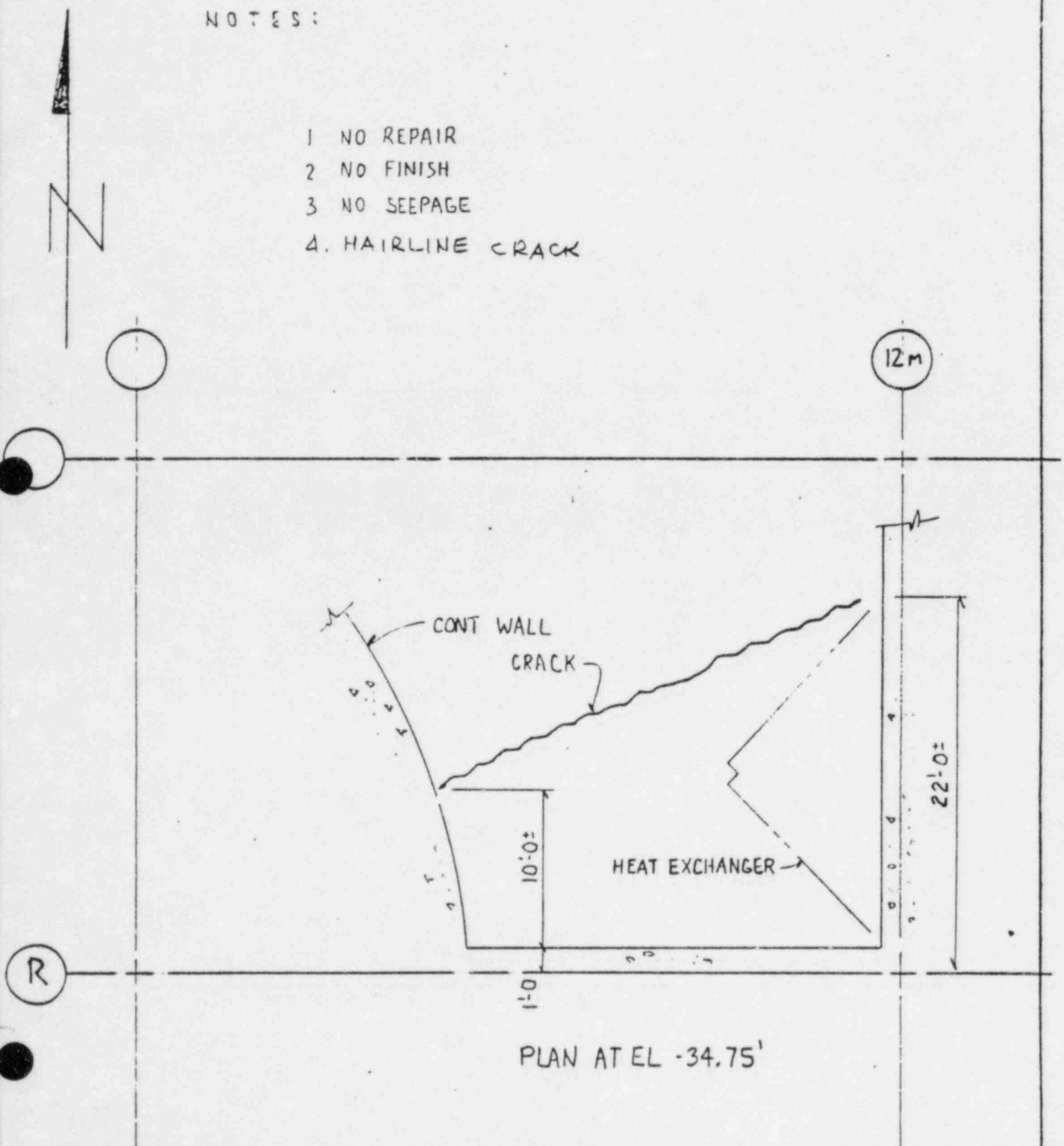


HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. C- – –	SUBJ. SUBDIV. SHEET
PROJECT	
CLIENT LP&L	
SUBJECT BASEMAT CRACK MAP	
PREP. BY RR/GW	DATE 9/1/83
CHCKD. BY [illegible]	DATE 09/02/83

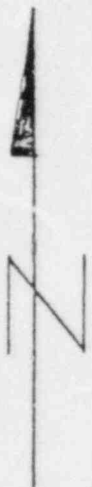
NOTES:

1 NO REPAIR
2 NO FINISH
3 NO SEEPAGE
4. HAIRLINE CRACK



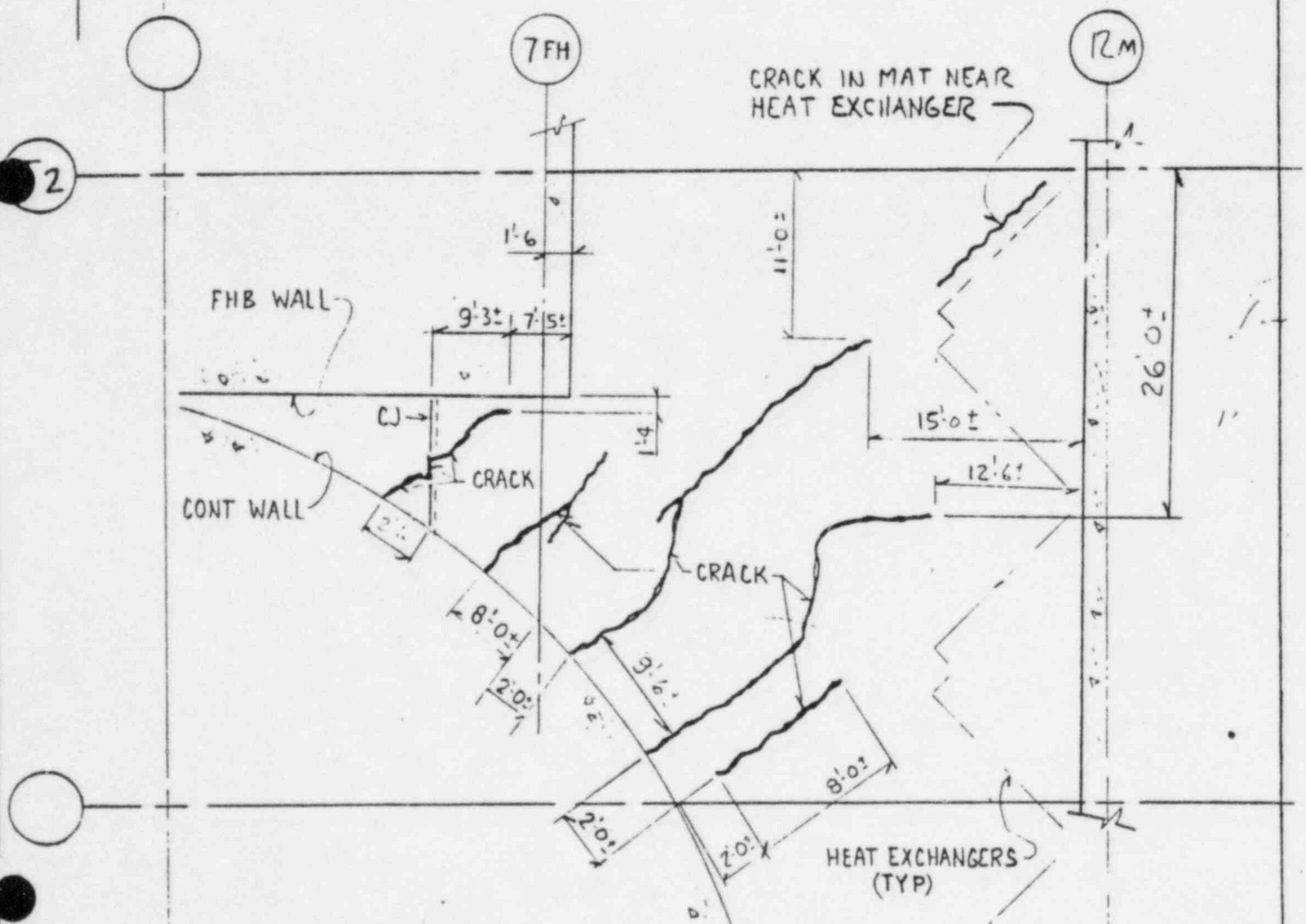
PLAN AT EL -34.75'

HEA HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD PARK RIDGE, N. J. 07656	PROJ. NO. C- - - SUBJ. SUBDIV. SHEET
PROJECT CLIENT SUBJECT BASEMAT CRACK MAP	PREP. BY RR/GW DATE 9/1/83 CHCKD. BY AAB DATE 09/02/83



NOTES:

1 NO REPAIR
2 NO FINISH
3 SOME EVIDENCE OF SEEPAGE
4. HAIRLINE CRACK



PLAN AT EL - 34.75'

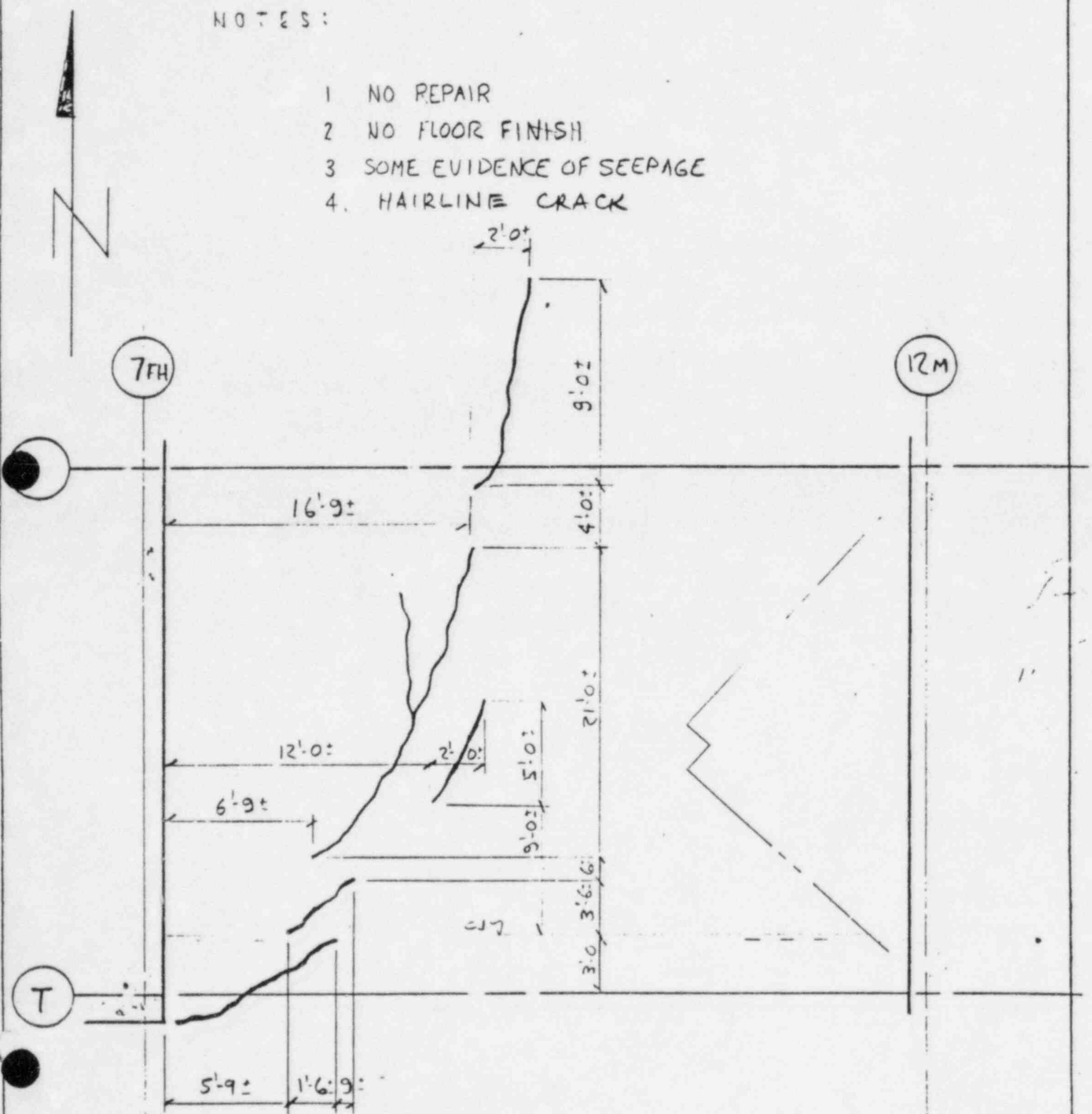
HEA **HARSTEAD ENGINEERING ASSOCIATES • INC.**
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP

PROJ. NO.		
C-	-	-
SUBJ.	SUBDIV.	SHEET
PREP. BY RK/GW	DATE 9/1/83	
CHCKD. BY AdB	DATE 09/02/83	

NOTES:

1 NO REPAIR
2 NO FLOOR FINISH
3 SOME EVIDENCE OF SEEPAGE
4. HAIRLINE CRACK



PLAN AT EL -34.75'

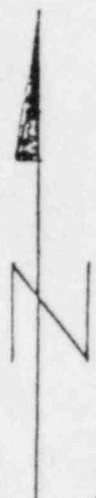
HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	C- - -
SUBJ. SUBDIV.	SHEET
PREP. BY RR/GW	DATE 9/2/83
CHCKD. BY AdB	DATE 09/02/83

PROJECT

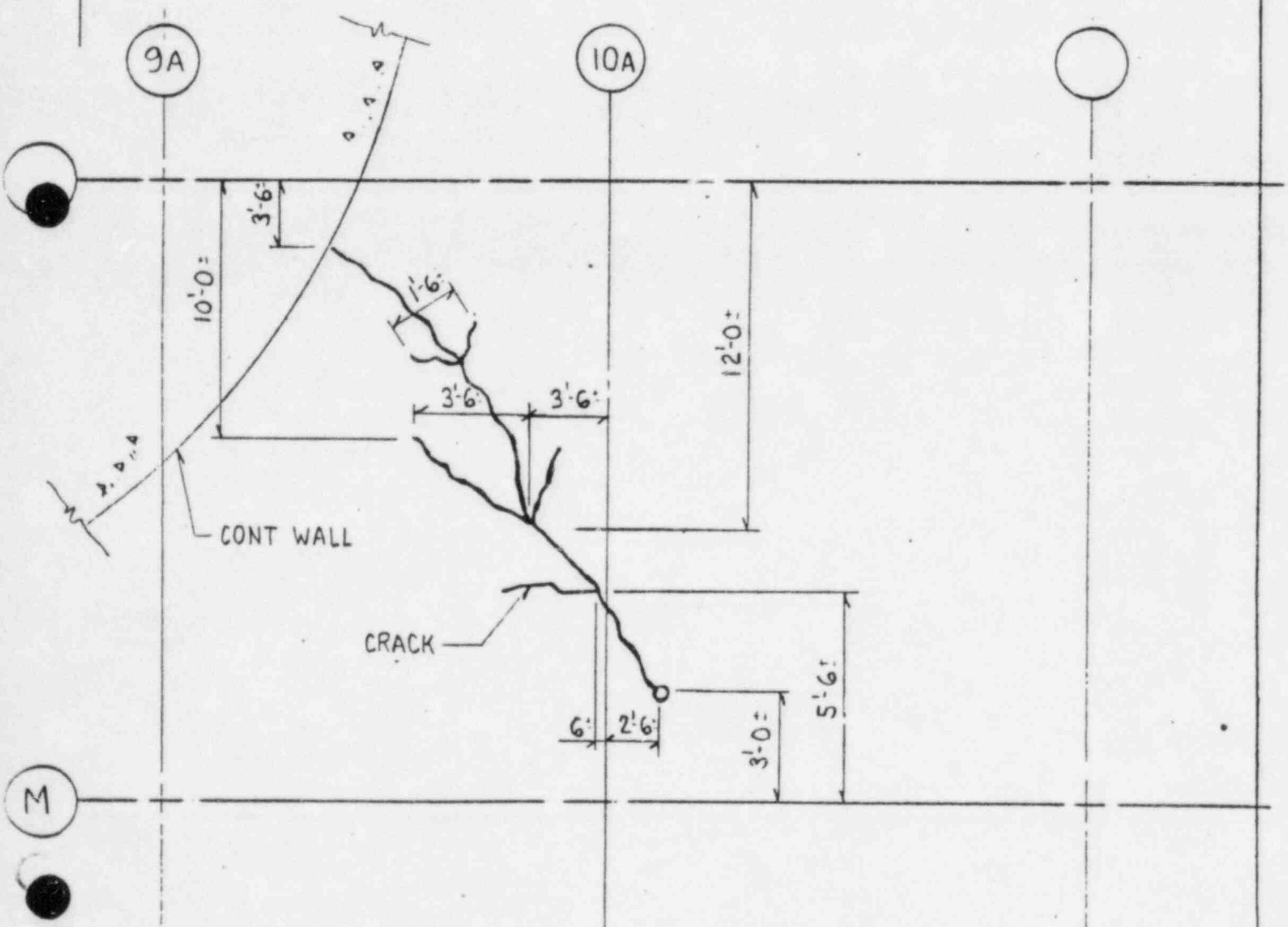
CLIENT [illegible]

SUBJECT BASEMAT CRACK MAP



NOTES:

1. NO REPAIR
2. NO FLOOR FINISH
3. CRACK IS DAMP
4. HAIRLINE CRACK



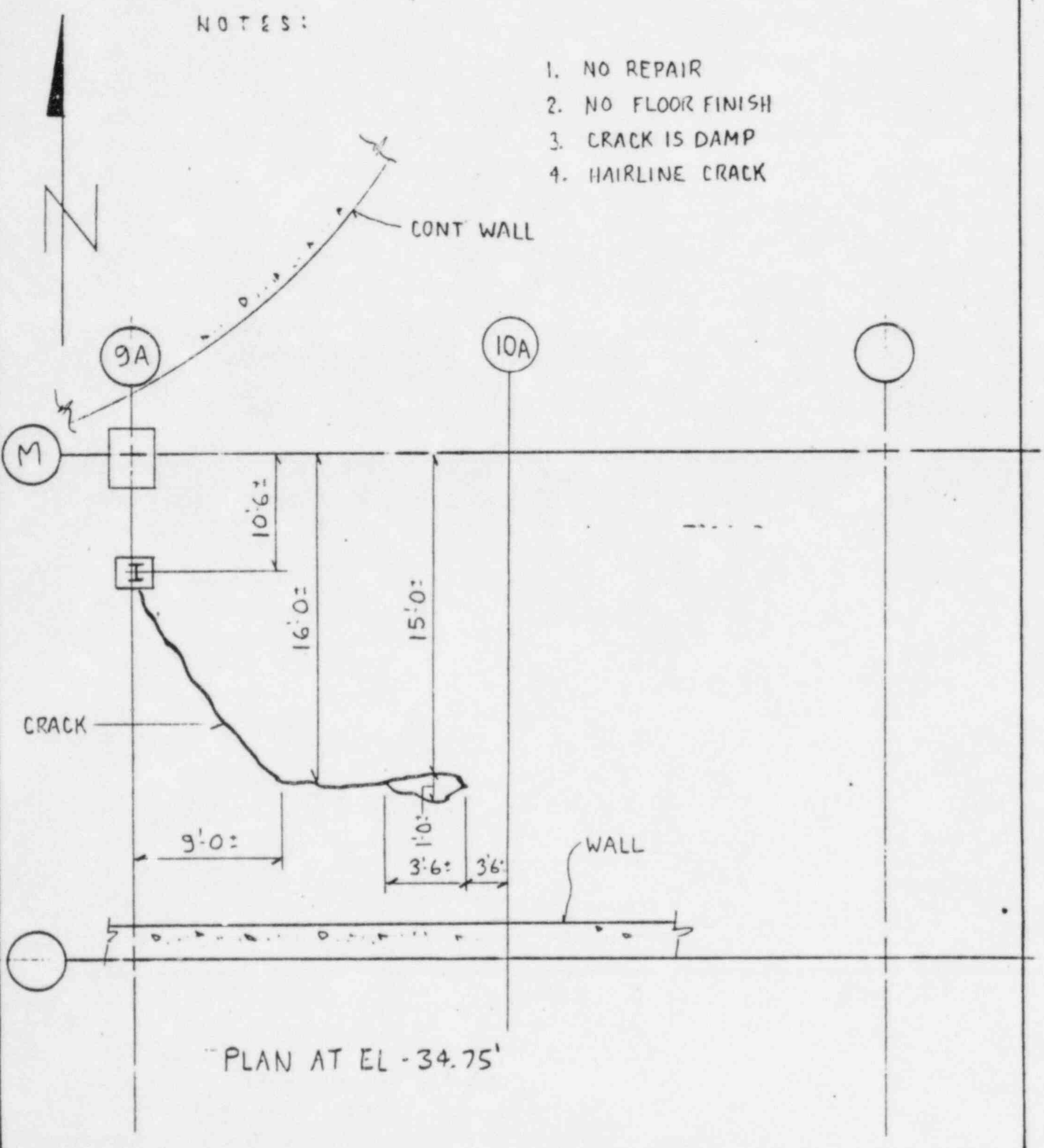
PLAN AT EL -34.75'

HEA **HARSTEAD ENGINEERING ASSOCIATES • INC.**
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT		PROJ. NO. C- - -
		SUBJ. SUBDIV. SHEET
CLIENT	[illegible]	PREP. BY RR/GW DATE 9/2/83
SUBJECT	BASEMAT CRACK MAP	CHCKD. BY AdP DATE 09/02/83

NOTES:

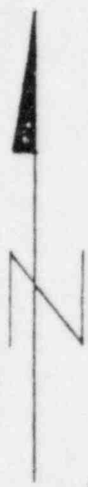
1. NO REPAIR
2. NO FLOOR FINISH
3. CRACK IS DAMP
4. HAIRLINE CRACK

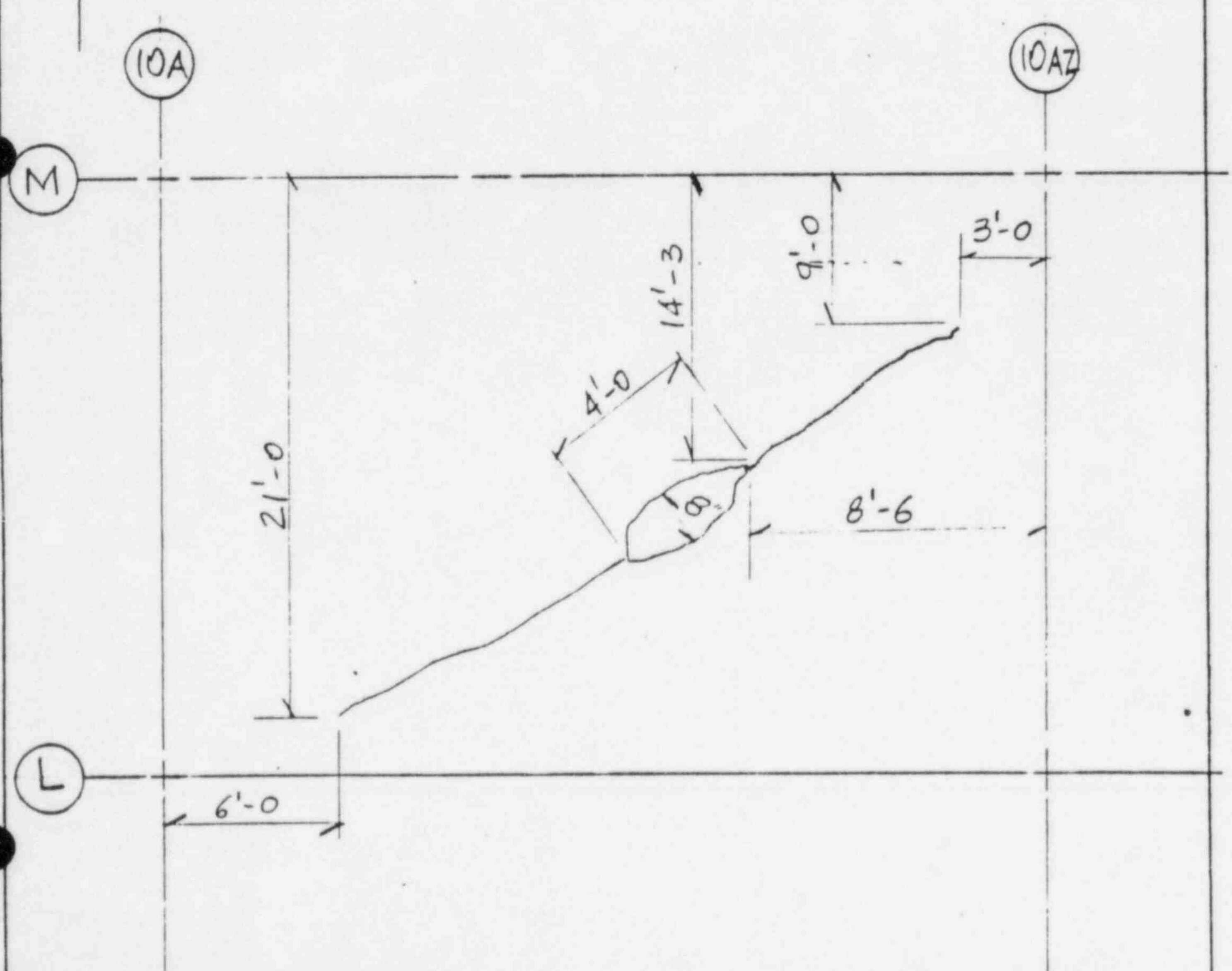


PLAN AT EL -34.75'

H E A HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. C- – – SUBJ. SUBDIV. SHEET
PROJECT CLIENT [illegible] SUBJECT BASEMAT CRACK MAP	PREP. BY IS/GW DATE 9/2/83 CHCKD. BY AdB DATE 09/02/83

NOTES: 1. NO
2. NO
3. YES - DAMP
4. HAIRLINE CRACK



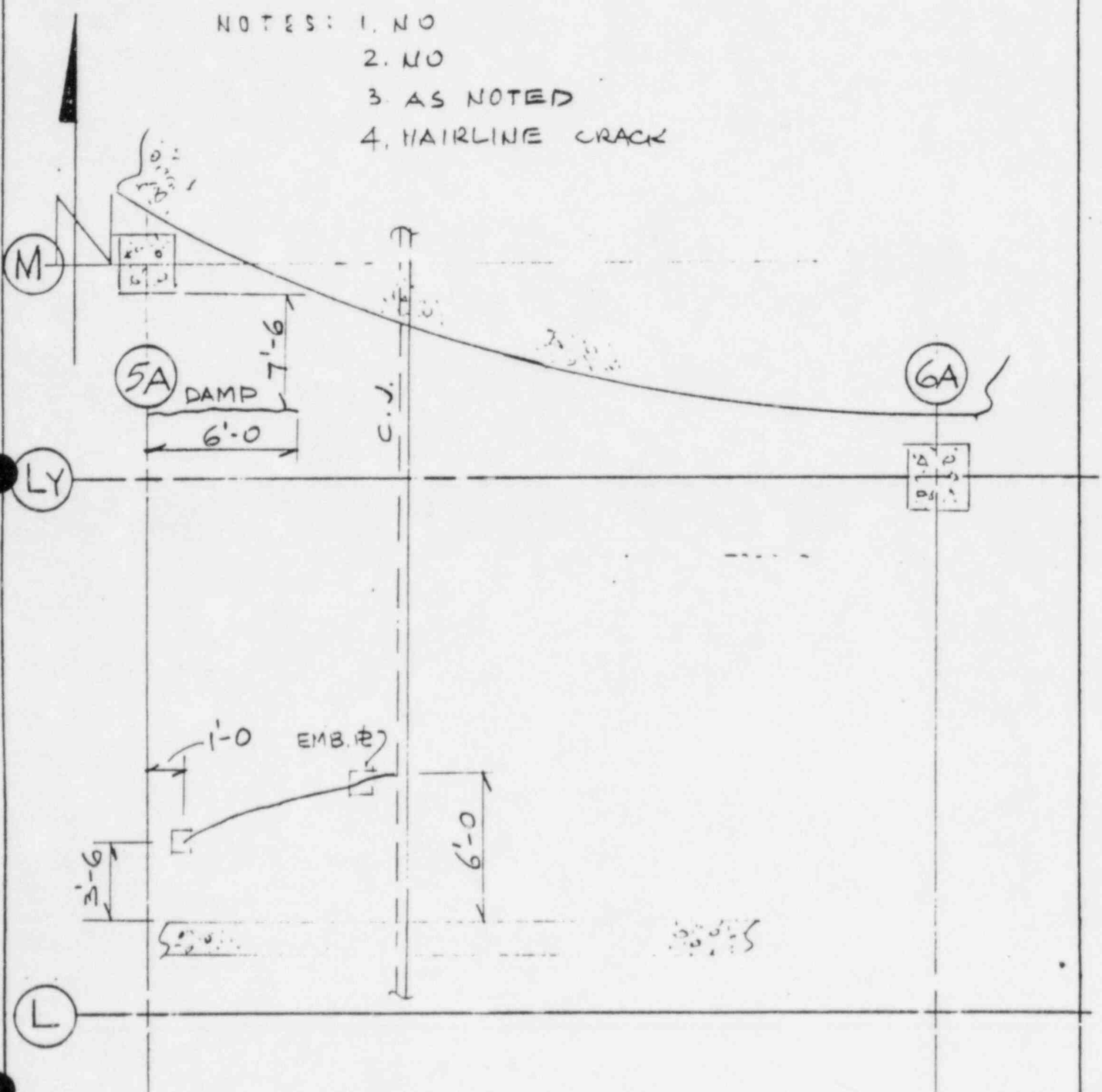


HEA 10

HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N.J. 07656

PROJ. NO. C- – –	SUBJ. SUBDIV. SHEET
PROJECT	
CLIENT [illegible]	PREP. BY ZS/GW DATE 9/2/83
SUBJECT BASEMAT CRACK MAP	CHCKD. BY AdB DATE 09/02/83



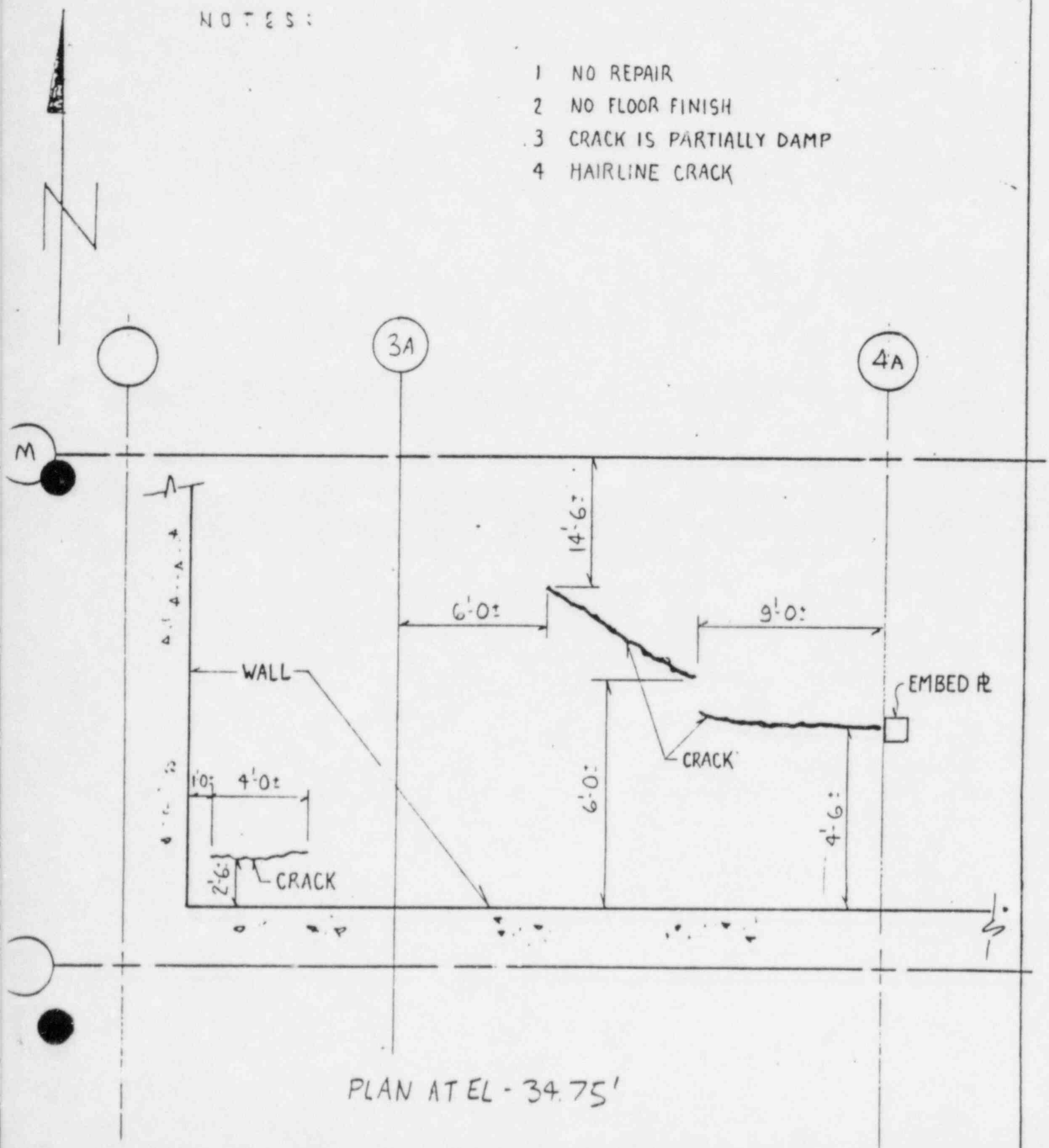
HEA

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	C- - -
SUBJ. SUBDIV. SHEET	
PROJECT	
CLIENT	[illegible]
SUBJECT	BASEMAT CRACK MAP
PREP. BY RR/GW	DATE 9/2/83
CHCKD. BY AdB	DATE 09/02/83

NOTES:

1 NO REPAIR
2 NO FLOOR FINISH
3 CRACK IS PARTIALLY DAMP
4 HAIRLINE CRACK

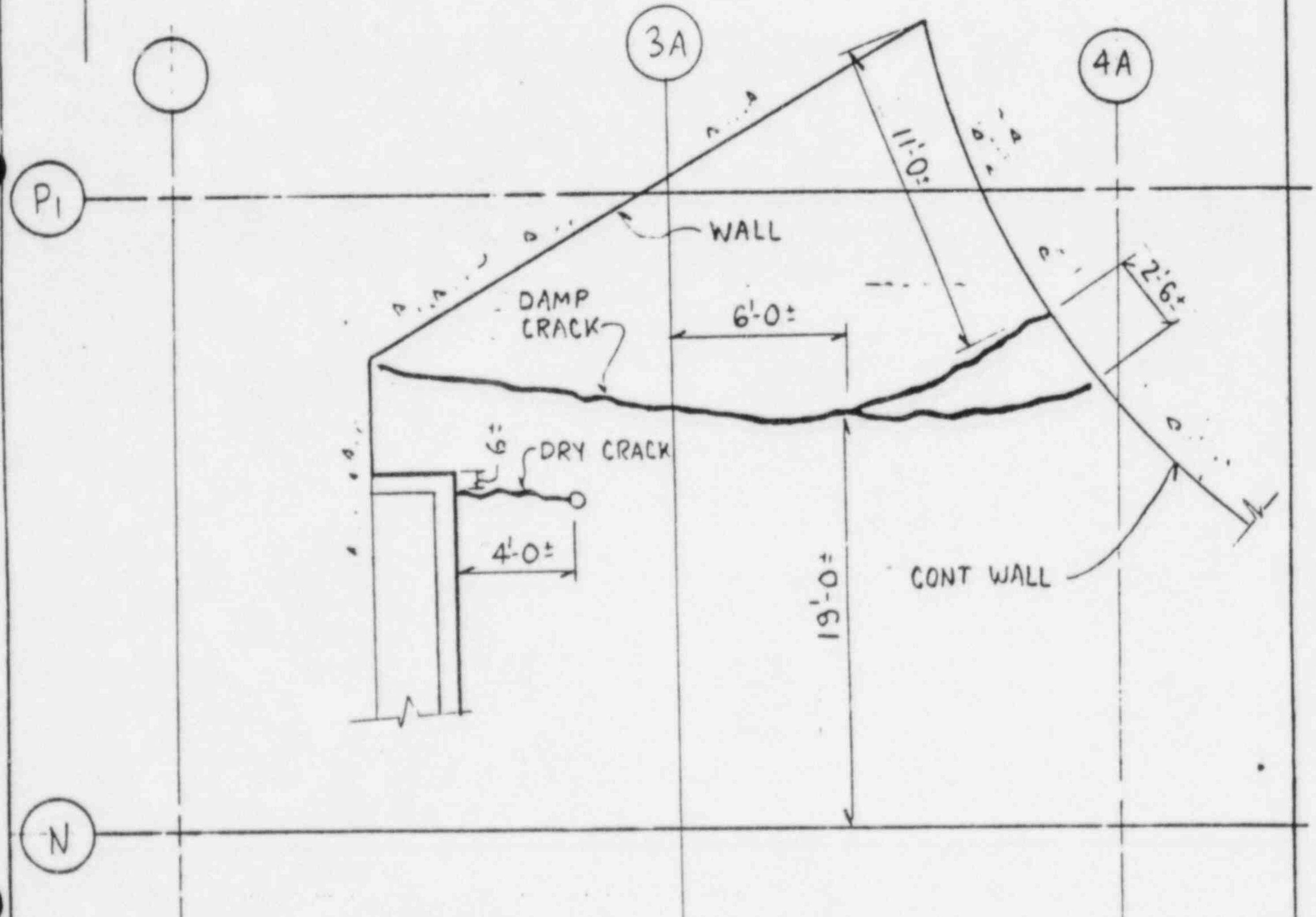


PLAN AT EL - 34.75'

HEA HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD PARK RIDGE, N. J. 07656	PROJ. NO. C- – –	SUBJ. SUBDIV. SHEET
PROJECT	PREP. BY RR/GW	DATE 9/2/83
CLIENT [illegible]	CHCKD. BY AdB	DATE 09/02/83
SUBJECT BASSMAT CRACK MAP		

NOTES:

1 NO REPAIR
2 NO FLOOR FINISH
3 DAMP AS INDICATED
4 HAIRLINE CRACK

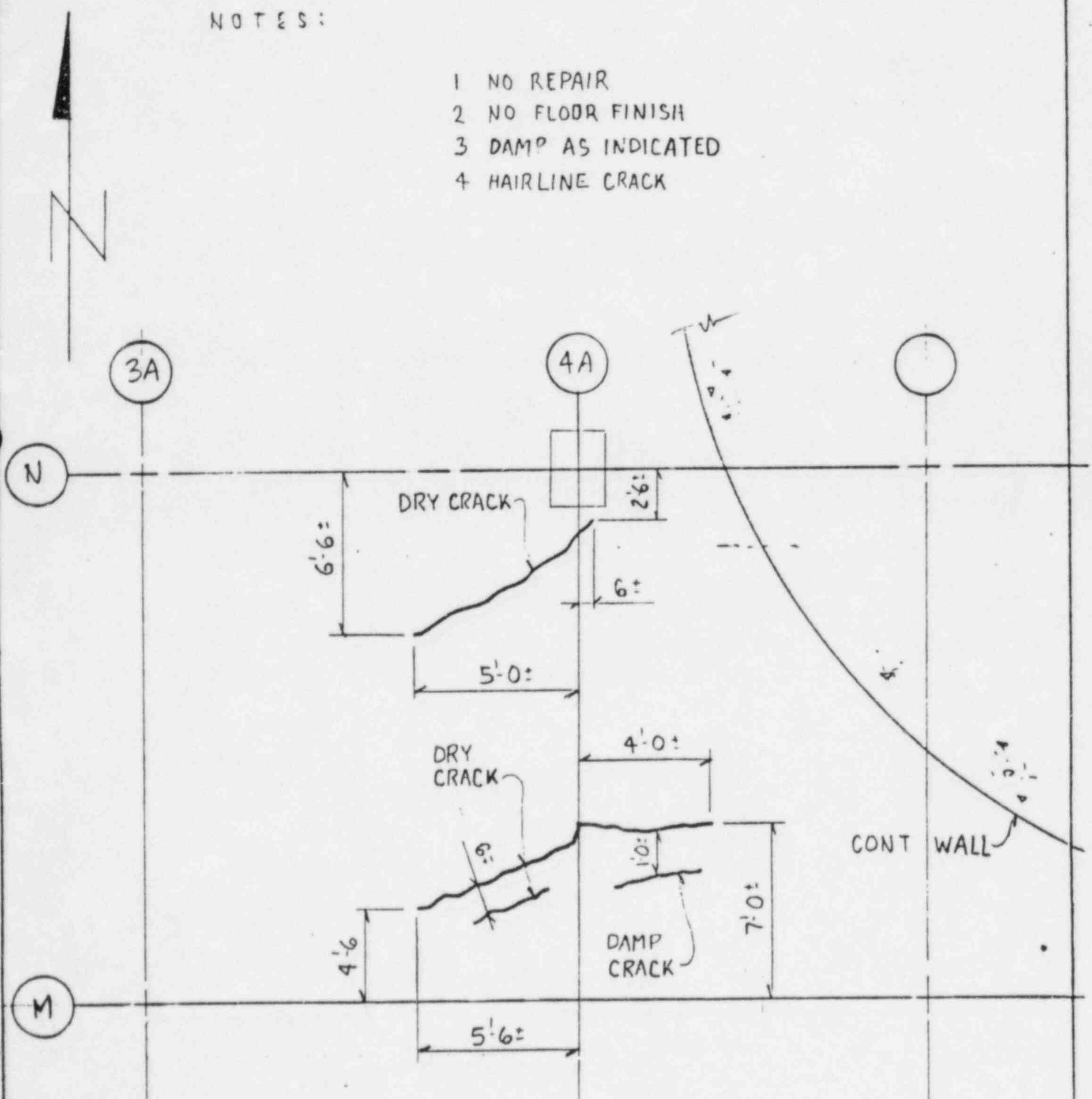


PLAN AT EL -34.75'

HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. C- – – SUBJ. SUBDIV. SHEET
PROJECT CLIENT [illegible] SUBJECT BASEMAT CRACK MAP	PREP. BY RR/GW DATE 9/2/83 CHCKD. BY AdB DATE 09/02/83

NOTES:

1 NO REPAIR
2 NO FLOOR FINISH
3 DAMP AS INDICATED
4 HAIRLINE CRACK



PLAN AT EL -34.75'

APPENDIX B

Properties of Subsurface Materials

Design Values

TABLE 2.5-14

PROPERTIES OF SUBSURFACE MATERIALS
DESIGN VALUES

	Visual Stratum Description	Elev. (MSL)	Unified Soil Description	Specific Gravity (Gs)	Natural Density (PCF)	Coefficient of Permeability (k) - cm/sec	Unconfined Compressive Strength -qu(ksf)	Undrained Shear Strength	Drained Shear Strength	Overconsolidation Ratio (OCR)	Average Shear Modulus Gmax(ksf)	Young's Modulus E (ksf)	Poisson's Ratio - μ*
1)	Clay and silty clay with silt and sand (Recent material**)	Grade to -40	CH	2.70	111	1.5×10^{-6}	1.0	c = 0.5 KSF $\phi = 0^{o}$	c' = 0 KSF $\phi = 25^{o}$	1.5 to 2.0	1200	3600	0.48
2)	Stiff tan and gray fissured clay	-40 to -77	CH	2.72	119	1×10^{-8}	3.0	c = 1.5 KSF $\phi = 0^{o}$	c' = 0.8 KSF $\phi = 12.5^{o}$	3.4	3900	11,600	0.49
3)	Very dense tan silty sand	-77 to -92	--	2.70	125	3×10^{-5}	--	--	c' = 0 KSF $\phi = 41^{o}$	--	3900	11,500	0.48
4)	Medium stiff gray clay with silt lenses	-92 to -108	CH	2.74	119	--	2.4	c = 1.2 KSF $\phi = 0^{o}$	c' = 0.8 KSF $\phi' = 12.5^{o}$	1.4	3900	11,600	0.49
5)	Stiff dark gray clay - organic	-108 to -116	MH & CH	2.68	104	--	3.6	c = 1.8 KSF $\phi = 0^{o}$	c' = 0.8 KSF $\phi' = 12.5^{o}$	1.7	3900	11,600	0.49
6)	Soft to medium stiff tan and gray clay with sand lenses	-116 to -127	ML & CL	2.69	119	--	1.4	c = 0.7 KSF $\phi = 0^{o}$	c' = 0.8 KSF $\phi' = 12.5^{o}$	2.0	3900	11,600	0.49
7)	Very stiff clays with silts and sands	-127 to -317	CH & CL	2.71	119	--	4.0	c = 2.0 KSF $\phi = 0^{o}$	c' = 0.8 KSF $\phi' = 12.5$	1.5 to 2.4	3900	11,500	0.48
8)	Very dense sands and silty sands	-317 to -500	--	2.70	119 to 125	--	--	--	--	--	10,000	29,000	0.45

*Computed from field Vp and Vs measurements
**Excavated and replaced with compacted backfill

Note: The average shear moduli values are averaged from maximum shear moduli obtained from field geophysical test results. They are representative only for low shear strains of approximately 10^{-6} in./in.

APPENDIX C

Synopsis/Introduction

of

State-Of-The-Art of Floating Foundations

by H. Q. Golder

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

STATE-OF-THE-ART OF FLOATING FOUNDATIONS[a]

By Hugh Q. Golder,[1] M. ASCE

SYNOPSIS

For a floating foundation the soil must have weight but it need not have shear strength. The foundation must be able to resist pressure on its base and sides and, if the weight or level of the soil varies, the pressures will not be uniform, and shear and bending forces will act on the foundation. In practice, most foundations are partly floating, and almost all so-called floating foundations are only partly floating because a small residual pressure is usually left on the soil. After a history of their development and the reasons for their use are given, the problems to be considered in using floating foundations are examined. Among the most important problems are excavation, bottom heave, settlement and tilting, and structural problems.

INTRODUCTION

In considering an engineering problem it is often helpful to begin with the limits between which the problem lies, in a physical sense, although these limits are not necessarily practical in an engineering sense.

When a foundation rests on the ground surface it is supported by the shear strength of the soil or rock of which the ground is composed. When the foundation is placed below the ground surface, for frost or drying protection, and the weight of the overburden is deducted from the applied pressure, the foundation is supported partly by the shear strength and partly by "buoyancy," i. e., the foundation is partly floating. The limit to a floating foundation is a ship in water.

Note.—Discussion open until August 1, 1965. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 91, No. SM2, March, 1965.

[a] This is one of the "state-of-the-art" papers presented at the ASCE Soil Mechanics and Foundations Division Conf. on "Design of Foundations for Control of Settlements," held at Northwestern Univ., Evanston, Ill., June, 1964, the compiled papers were presented in the September, 1964, division Journal.

[1] Cons. Civ. Engr., H. Q. Golder and Assocs. Ltd., Toronto, Canada.

This analogy of a ship indicates certain useful facts, namely:

1. The ship displaces a volume of water equal to its weight;
2. after the initial settlement there is no further settlement;
3. the pressure on the horizontal base is uniform;
4. if the water level ceases to be uniform, (i. e., wave action) the pressures change and the hull is subjected to bending and shear forces;
5. there are lateral pressures on the side of the hull;
6. the water has no shear strength, i. e., shear strength is not necessary for support; and
7. the hull is complete when launched into the water and launching stresses may be high.

The preceding facts indicate that, for a floating foundation, the soil must have weight but it need not have shear strength. The foundation must be able to resist pressure on its base and sides and, if the weight or level of the soil varies, the pressures will not be uniform, and shear and bending forces will act on the foundation. Once in position, the foundation will not settle further (if fully floating) unless further load is added. In soil of low shear strength construction of floating foundations may be difficult but the depth is not limited by shear strength if suitable construction procedures are used; however, "launching" stresses in the foundation may be high. In practice, most foundations are partly floating, and almost all of the so-called floating foundations are only partly floating because a small residual pressure is usually left on the soil.

A floating foundation will be considered herein as one in which the greater part of the building load is balanced against the weight of excavated material and is not supported by the shear strength of the soil.

HISTORY

The concept of using floating foundations is not new. There is some evidence that they were used in the 18th century, and it is probable that they were used intuitively before that date.

In the discussion[2] of a paper by Casagrande and Fadum K. Terzaghi refers to a German work by G. Hagen[3] dated 1870, in which there is reference to the use of floating foundations by John Rennie in London at the Albion Mills. There is no doubt that Hagen really knew what a floating foundation was. He says ". . . a heavy building can still be safely built by sinking it partly into the ground so that it actually floats. The complete weight of the building must not be greater than that of the excavated material."

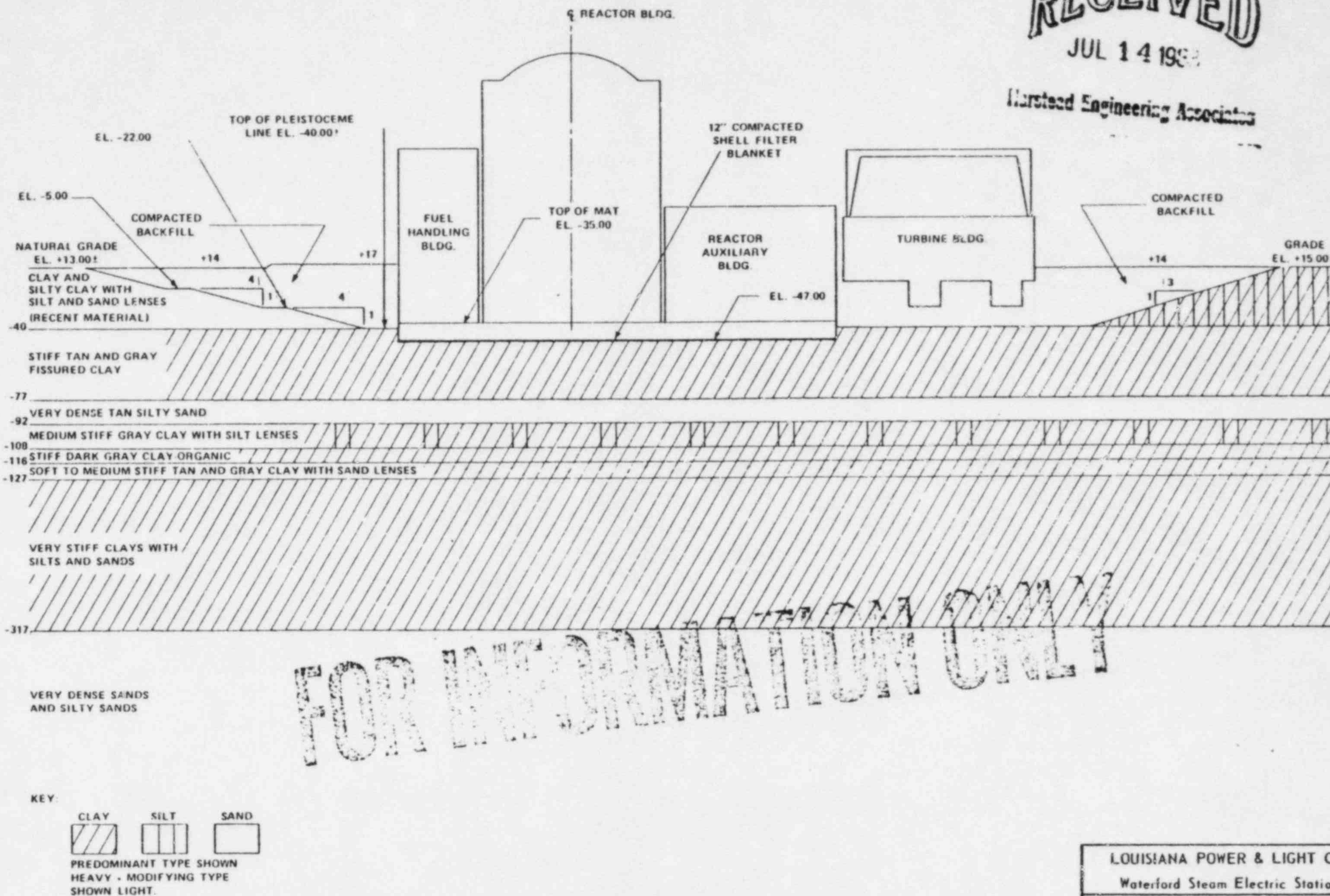
The reference to Rennie is taken from "A Treatise on the Steam Engine" by J. Farey (1827). According to Hagen, Farey says

[2] Terzaghi, Karl, discussion of "Application of Soil Mechanics in Designing Building Foundations," by A. Casagrande and R. E. Fadum, Transactions, ASCE, Vol. 109, 1944, p. 427.

[3] Hagen, G., "Handbuch der Wasserbaukunst," Ernst U. Korn, Berlin, 1870.

APPENDIX D

Generalized Site Cross Section



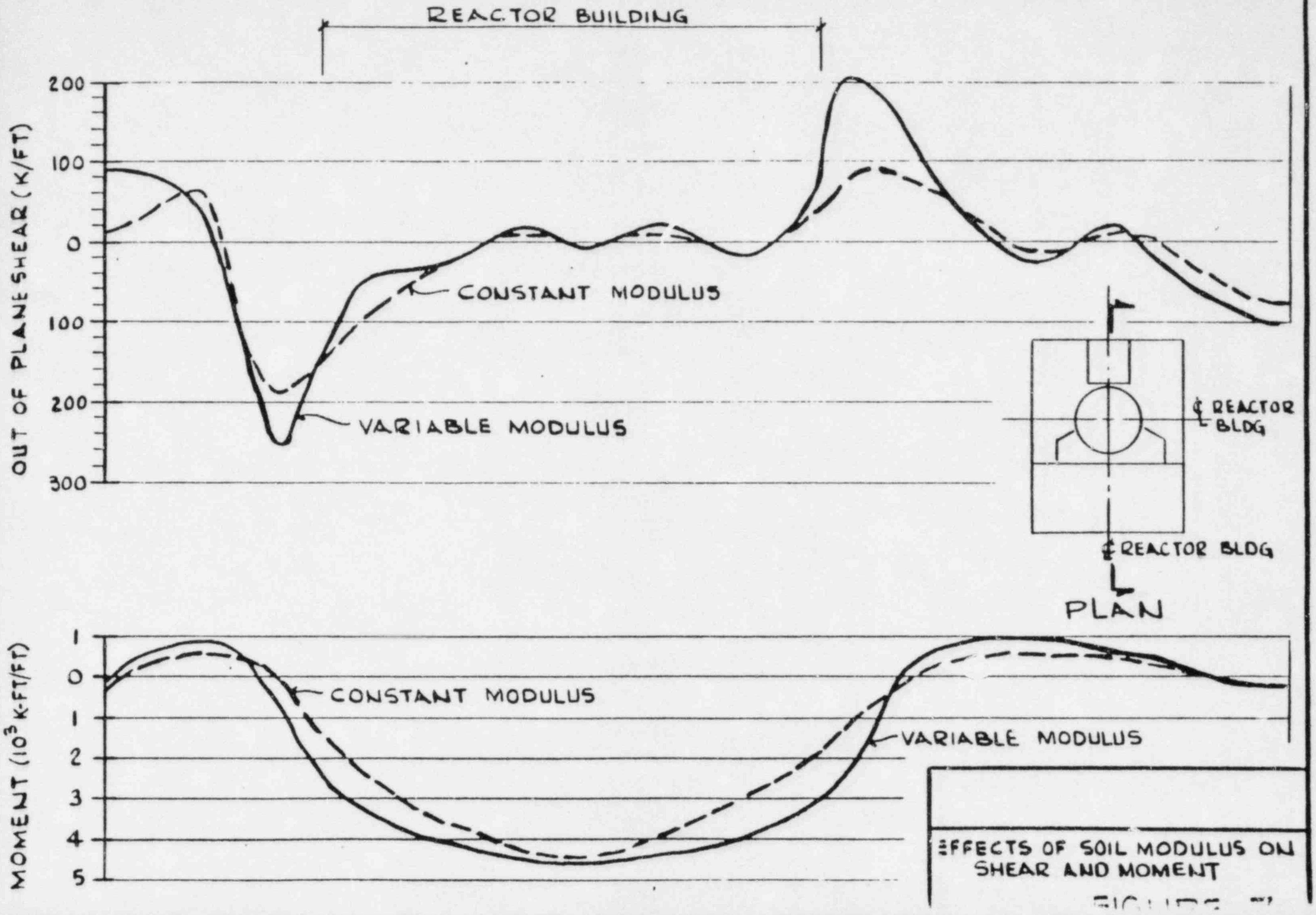
LOUISIANA POWER & LIGHT CO.
Waterford Steam Electric Station

GENERALIZED SITE CROSS SECTION
FIG 2.5-80

APPENDIX E

Effects of Soil Modulus on

Shear and Moment

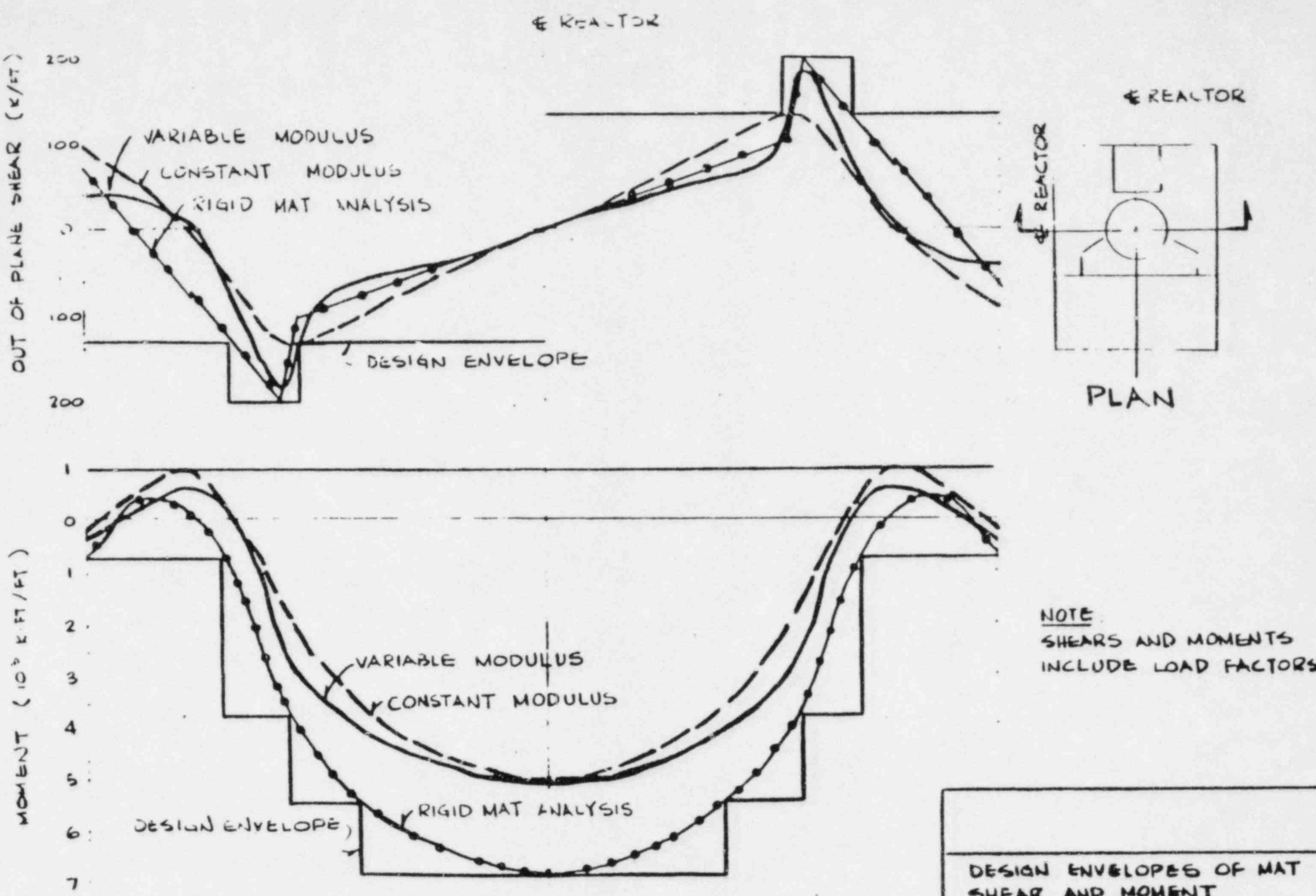


EFFECTS OF SOIL MODULUS ON SHEAR AND MOMENT

APPENDIX F

Design Envelopes of Mat

Shear and Moment



NOTE
SHEARS AND MOMENTS INCLUDE LOAD FACTORS

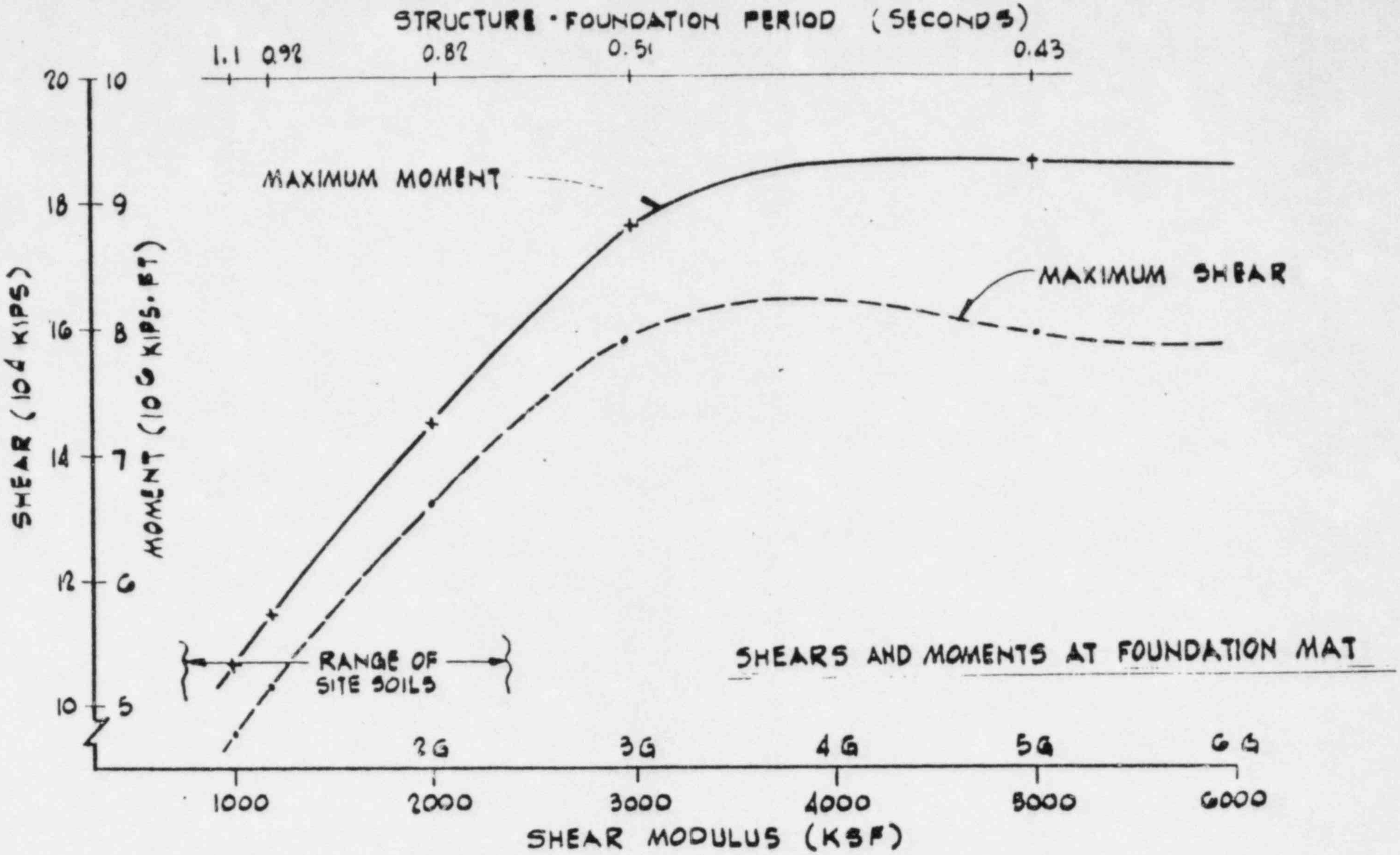
DESIGN ENVELOPES OF MAT SHEAR AND MOMENT

FIGURE 8

APPENDIX G

Effects of Foundation Stiffness

on Dynamic Shears and Moments



EFFECTS OF FOUNDATION STIFFNESS ON DYNAMIC SHEARS AND MOMENTS

FIGURE 9

APPENDIX H

Ebasco Services Letter

F-16919, W3-NY-1

dated June 29, 1977

EBASCO SERVICES
INCORPORATED

UTILITY CONSULTANTS - ENGINEERS - CONSTRUCTORS
P. O. Box 70
Killona, Louisiana 70066

June 29, 1977
F-16919
W3-NY-1

Boh Brothers Construction Company
P. O. Drawer 53266
New Orleans, Louisiana 70153

Attention: Mr. R. J. Drueding

LOUISIANA POWER AND LIGHT COMPANY
WATERFORD STEAM ELECTRIC STATION
1980 - 1165 MW INSTALLATION - UNIT NO. 3
CONTRACT W3-NY-1, EXCAVATION AND DEWATERING

Ref: (1) Ebasco letter F-7419, dated May 18, 1977
(2) Boh Brothers letter (H. G. Chapman to J. O. Booth), dated May 27, 1977

Gentlemen:

As you are aware, the efficient operation of the entire dewatering system (ejector wells and pumped relief wells) is critical to continued unhampered construction of the Waterford Nuclear Project. During the last few months, it appears that the maintenance of the dewatering system and monitoring of the site instrumentation has deteriorated to a point that the dewatering system cannot support the recharging effort. This situation is totally unacceptable and corrective action must be taken immediately.

The implementation of the Recharge Program is dependent on two primary factors. First, the effective foundation loading on the underlying Pleistocene soils must be controlled within certain limits as established by the PSAR (Preliminary Safety Analysis Report) and the job specifications. This will be accomplished by controlling the hydrostatic uplift or buoyant weight of the nuclear plant island structure. Second, the foundation soil response of the clay strata will be monitored with the site instrumentation and adjustments made to the Recharge Program as necessary to meet the design intent with respect to controlling heave and settlement of the combined structure. The Recharge Program will be implemented at the direction of and will be coordinated by the Ebasco Site Soils Engineer.

Due to the uncontrolled rise in piezometric levels beneath the nuclear plant island during the last two months, initial implementation of the Recharge Program must now be delayed. The rise in piezometric levels has been attributed by Moretrench-American to be entirely the result of water used for compaction of the backfill. Although evidence has shown that the compaction water from the

backfill has infiltrated the slotted casing of the pumped relief wells which extend through the backfill, we do not believe that this is the total cause for the rise in piezometric levels.

We note that the flow rate of the primary dewatering system (ejector wells), as reported to Ebasco by Boh Brothers, remained essentially unchanged at 180 gpm from January 9, 1977 (when the new south leg of the system went into operation) until June 9, 1977. On June 9, 1977 Boh Brothers reported the flow rate of the primary system was 82 gpm, a drop of more than one-half from the readings of the previous five months. Subsequent checks of the system by Boh Brothers and by Ebasco indicate that the primary system is presently operating at an average rate of 100 to 110 gpm, or a little less than two-thirds of the flow rate that has been reported for the last five months. On the basis of the above information, it is obvious and was admitted by Moretrench-American in a meeting on June 16, 1977 that maintenance during that time period has been minimal. We find it difficult to believe that there has been no reduction in the efficiency of the system as contended by Moretrench. The apathy indicated by such performance is intolerable.

Implementation of the Recharge Program is scheduled to begin in early August. The entire program will take from 6 to 12 months, depending on the criteria stated above. It is imperative that the entire dewatering system (both the ejector system and pumped relief wells) remain in top working order until the Recharge Program is completed and you are directed to remove the system.

In addition, as noted above, monitoring of site instrumentation and reporting of results has been very poor throughout the project. The following table indicates the frequency of readings reported by Boh Brothers from March 1977 to May 1977:

Instrument	Date			Total	
	March	April	May	Required	Actual
1. Observation Wells	15	16	16	65	47
2. Piezometers 1-10	0	3	3	13	6
3. Piezometers 11-22	4	4	5	13	13
4. Extensometers	4	3	1	13	8
5. Heave Points	4	3	2	13	9
6. Dewatering Flow Rate	15	11	9	65	35
7. River Elevation	14	12	12	65	38
8. Inclinometers (Compressions)	1	0	1	13	2
9. Inclinometers (Deflections)	5	2	4	13	11
Total	62	54	53	273	169

The above tabulation does not reflect data transmitted incorrectly or late.

A quick review of the table indicates only 62% of the required readings were taken during the three month period. This frequency of monitoring is unacceptable. This problem has been addressed before (Ebasco letter F-7419, dated May 18, 1976) and acknowledged by Boh Brothers letter dated May 27, 1976. The effectiveness of the Recharge Program depends on timely collection and compilation of the data. It is expected that all instrumentation will be read on time and the results reported within 24 hours as required by Contract and as committed to by Boh Brothers Construction Company.

In conclusion, it is apparent that the caliber of service has diminished with respect to the dewatering system and the site instrumentation. Implementation of the Recharge Program cannot start until control of the piezometric pressures has been re-established. A successful recharge program is essential to this project and all parties must adhere rigidly to the Contract requirements and responsibilities.

Yours very truly,

J. O. Booth

J. O. Booth
Project Superintendent

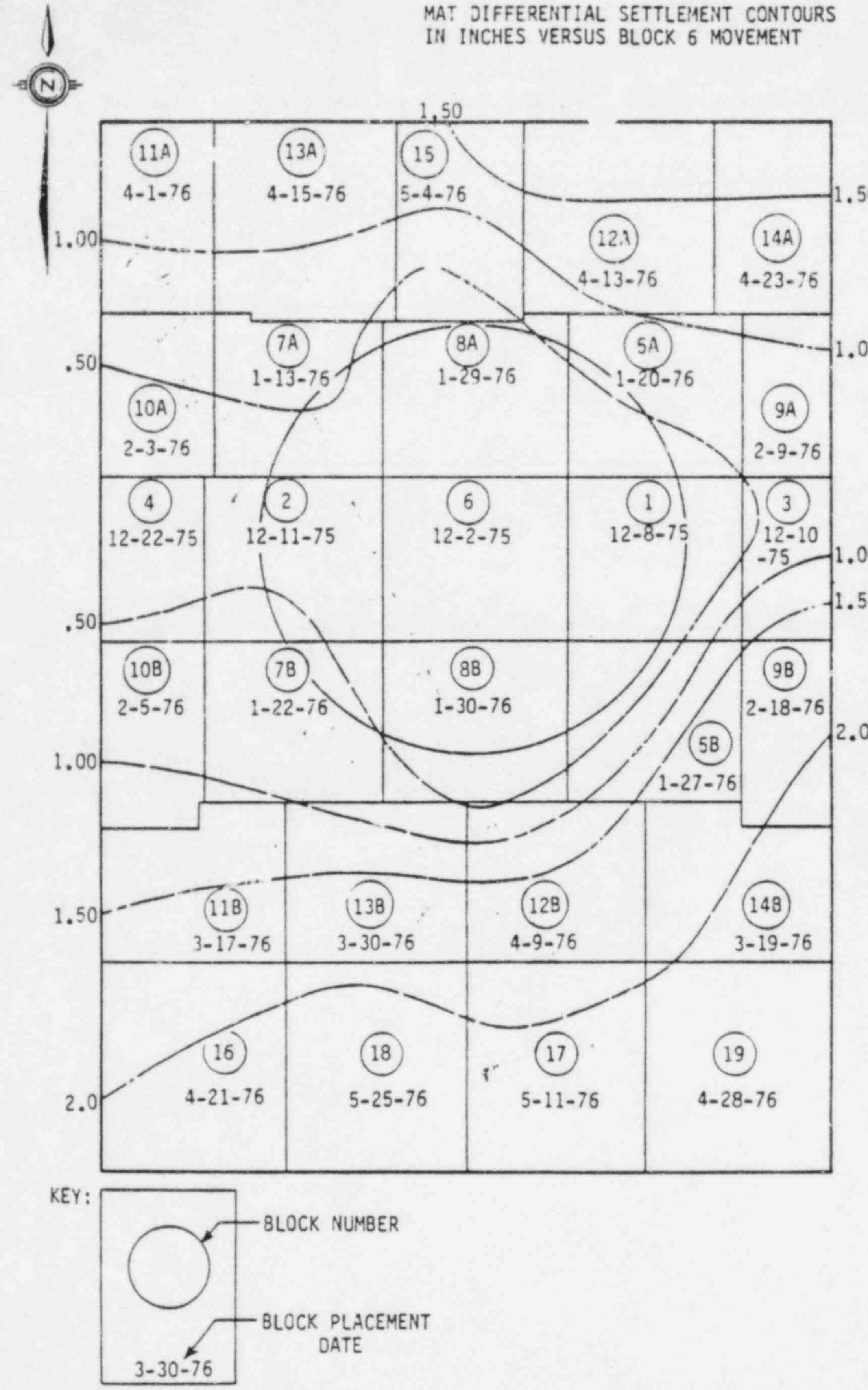
MGM
GFG/jah

cc: J. M. Brooks
L. Elliott
E. Henderson
R. Dawes
E. Boyd
S. Shallcross
R. Watt
D. Mallette
G. Goodheart
J. L. Ehasz
P. C. Liu
E. Foss

APPENDIX I

Composite Foundation Mat

Differential Settlement Contours



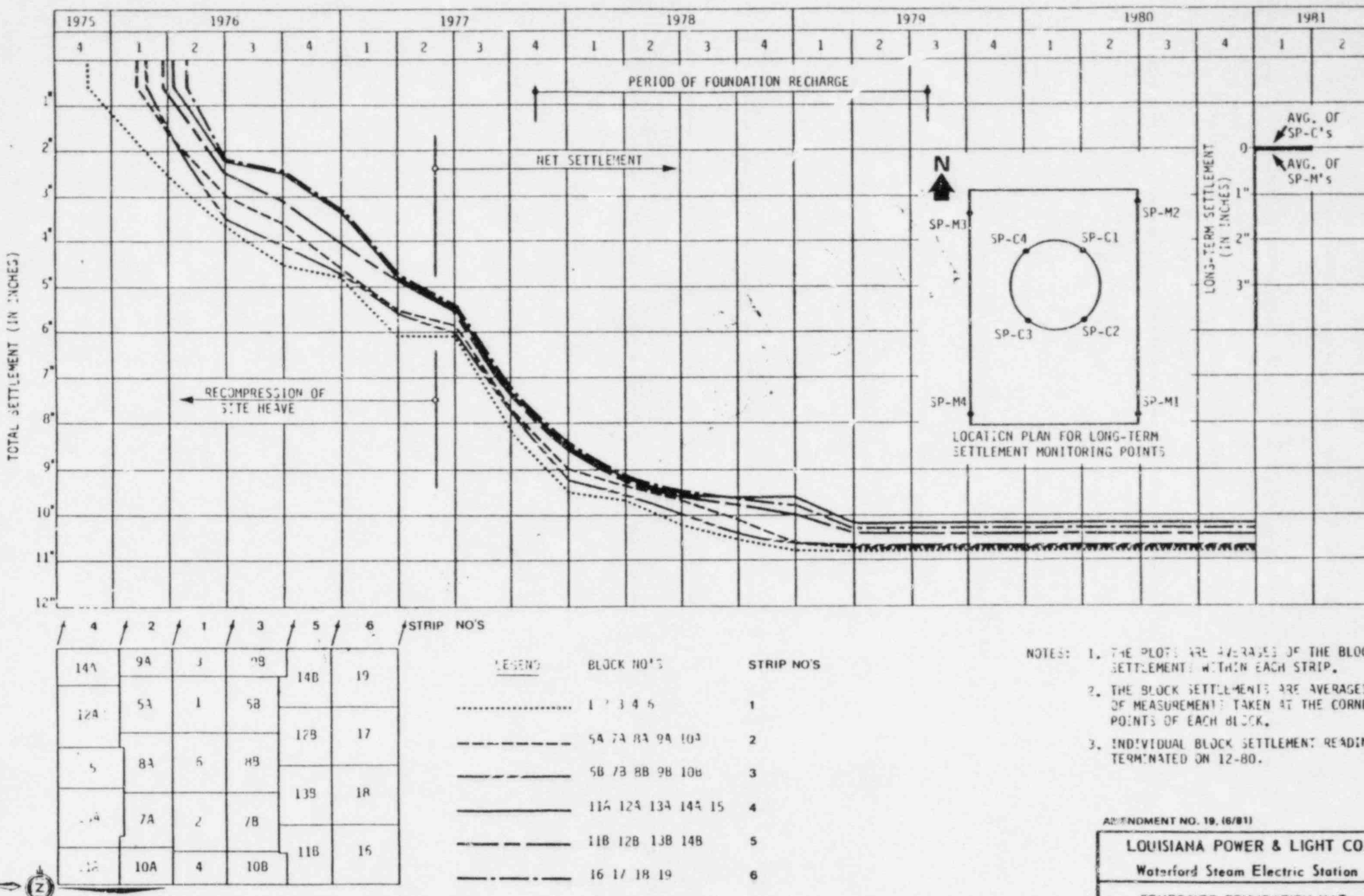
AMENDMENT NO. 19, (6/81)

LOUISIANA POWER & LIGHT CO. Waterford Steam Electric Station	COMPOSITE FOUNDATION MAT DIFFERENTIAL SETTLEMENT CONTOURS	Figure 2.5-118

APPENDIX J

Composite Foundation Mat

Settlement



NOTES:
1. THE PLOTS ARE AVERAGES OF THE BLOCK SETTLEMENTS WITHIN EACH STRIP.
2. THE BLOCK SETTLEMENTS ARE AVERAGES OF MEASUREMENTS TAKEN AT THE CORNER POINTS OF EACH BLOCK.
3. INDIVIDUAL BLOCK SETTLEMENT READINGS TERMINATED ON 12-80.

AMENDMENT NO. 19, (6/81)

LOUISIANA POWER & LIGHT CO.
Waterford Steam Electric Station

COMPOSITE FOUNDATION MAT SETTLEMENT

FIGURE 2.5-117

APPENDIX K

Crack Width Calculation

HEA **HARSTEAD ENGINEERING ASSOCIATES • INC.**
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	WATERFORD 3 SES	PROJ. NO.	8304
CLIENT	LP&L	C- 1 - 2 - 1	SUBJ. SUBDIV. SHEET
SUBJECT	CRACK WIDTH	PREP. BY GH	DATE 9-12-83
		CHCKD. BY [illegible]	DATE 09/[illegible]/83

CRACK SPACING ≥ 10 FT RCB

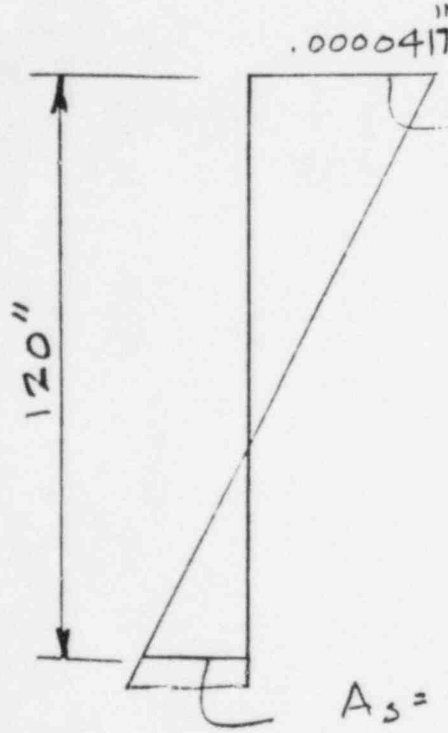
CONSERVATISM USE 10 FT

CRACKS FOR 2 TO 5 MILS IN WIDTH

CONSERVATISM USE 5 MILS

$$\epsilon = \frac{.005}{120} = 0.0000417$$

$$\sigma_{STEEL} = 29 \times 10^6 \,(40 \times 10^{-6}) = 1208 \text{ PSI}$$



#11 @ 6

$A_{s\,TOP} = 3.12\ \square''/FT$

#18 @ 6 2 LAYERS

$A_s = 16.0\ \square''/FT$

NEGLECT CONCRETE IN TENSION & COMPRESSION

REASONABLE SINCE BOTT STL >> TOP STL

BOTT $\epsilon = .000041^{7}\left(\frac{3.12}{16.0}\right) = 0.00000813$

ANGLE CHANGE PER INCH

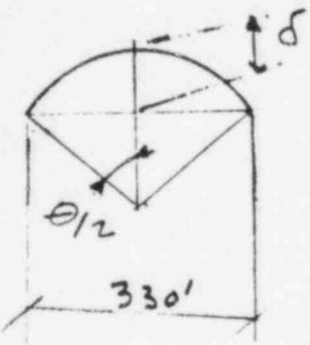
$$\phi = \frac{.000041^{7} + .00000813}{120} = \text{IN/IN/IN}$$

$$= 0.413 \times 10^{-6} = 1/\text{IN}$$

TOTAL ANGLE CHANGE IF CONSTANT OVER ENTIRE LENGTH OF MAT

$$\theta = 330(12)\,\phi = 0.001634 \text{ RAD}$$

$$= 0.0907565$$



$$\sin\frac{\theta}{2} = \frac{\frac{330}{2}}{R}$$

$$R = \frac{165(12)}{\sin(.0453783)} = 2.4242426\,9 \times 10^{6}\ ''$$

$$R\cos(.0453783^{\circ}) = 2.42424189 \times 10^{6}\ ''$$

$$\delta = R - R\cos(.0453783) = 0.81\ ''$$

APPENDIX L

Steel Containment Stability

Calculation

HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 2304
	C-1 - 1 - 1 SUBJ. SUBDIV. SHEET
PROJECT W3 - MAT CLIENT LOUISIANA POWER & LIGHT SUBJECT STEEL CONTAINMENT STABILITY	PREP. BY AdB DATE 08/05/83 CHCKD. BY GH DATE 8/08/83

INTRODUCTION

THIS CALCULATION COMPUTES THE SAFETY FACTOR AGAINST TIPPING CONTAINMENT SHOWN ON EBASCO DRAWING LOU-1564-G-817, REV. 13, DATED 02/03/83 (REFERENCE 1).

THE STABILITY OF THE STEEL CONTAINMENT/SHIELD BUILDING WITH RESPECT TO THE TOP OF THE MAT IS ALSO CALCULATED.

ADDITIONAL INFORMATION IS ACCESSED FROM EBASCO CALCULATION ENTITLED 'STEEL CONTAINMENT STABILITY', DFS NO. 1352.063, DEPT. NO. 650, DATED 05/10/83, 7 PP. (REFERENCE 2).

AT A MEETING HELD AT EBASCO'S OFFICE ON 07/27/83 IT WAS AGREED THAT THE MAGNITUDES OF THE E-W DBE ACCELERATIONS TABULATED ON PAGE 5 OF THE REFERENCE 2 CALCULATION WOULD BE MULTIPLIED BY $\sqrt{2}$ AS A CONSERVATIVE BOUND TO THE SRSS OF THE E-W AND N-S HORIZONTAL RESPONSE SPECTRA.

HARSTEAD ENGINEERING ASSOCIATES • INC. 169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656	PROJ. NO. 8304		
	C-1	-1	-2
	SUBJ.	SUBDIV.	SHEET
PROJECT	PREP. BY AdB		DATE 08/08/83
CLIENT LOUISIANA POWER & LIGHT			
SUBJECT STEEL CONTAINMENT STABILITY	CHCKD. BY GH		DATE 8/8/83

FROM P.4 OF THE REF. 2 EBASCO CALCULATION, THE SHEAR AND BENDING MOMENT DUE TO E-W DBE AT EL -1.5 IS:

$$F = 12551 \text{ k}$$

$$M = 659032 \text{ k ft}$$

MULTIPLYING BY $\sqrt{2}$ AS A CONSERVATIVE UPPER BOUND TO THE SRSS OF E-W AND N-S DBE:

$$F = \sqrt{2} \times 12551 = 17750 \text{ k}$$

$$M = \sqrt{2} \times 466009 = 659032 \text{ k ft}$$

ESTABLISHING THE ELEVATION OF POINT 'ℓ' ON THE SKETCH:

$$\text{EL. } '\ell' = 105' \times \cos 40° - 7.50'$$

$$= \text{EL. } 72.9'$$

COMPUTING THE STATICALLY EQUIVALENT MOMENT @ EL. 'ℓ':

$$M_\ell = 659032 - 17750 \times (72.9 + 1.5)$$

$$= -661568 \text{ (C'CL SENSE)}.$$

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

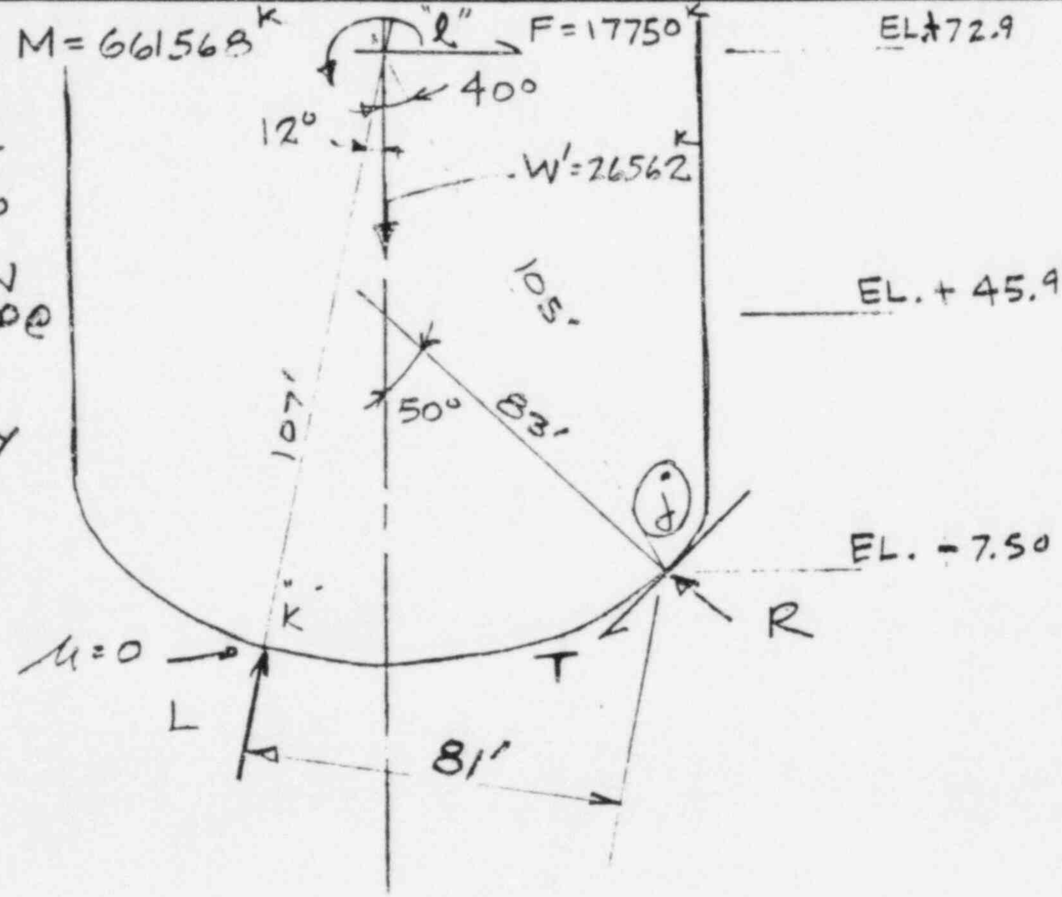
PROJ. NO. 8304
C- 1 - 1 - 3
SUBJ. SUBDIV. SHEET

PROJECT W3 - MAT
CLIENT LP&L
SUBJECT STABILITY

PREP. BY GH DATE 8/5/83
CHCKD. BY AdP DATE 08/08/83

IF STEEL VESSEL WAS MOVED HORIZ. WITHOUT TIPPING, TWO POINTS OF CONTACT CAN BE IDEALIZED @ "j" & "K".

PT K IS CONSERVATIVELY ASSUMED AS FRICTIONLESS



① $\Sigma M_j = -M + 81L + F\cos 40°(105) - W\sin 40°(105) = 0$

$-661568 + 81L + 1{,}427{,}715 - 1{,}792{,}741 = 0$

$L = 12674$

② $\Sigma V = W' - L\cos 12° - R\cos 50° + T\sin 50°$

$26562 - 12675(.9781) - R(.6428) + T(.766) = 0$

$14164 - .6428R + .766T = 0$ ②'

③ $\Sigma H = F + L\sin 12° - R\sin 50° - T\cos 50° = 0$

$17750 + (12675)(.2079) - .766R - .6428T = 0$

$20385 - .766R - .6428T = 0$ ③'

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT: W3 - MAT	PROJ. NO. 8304
CLIENT: LP&L	C- 1 - 1 - 7 (SUBJ. SUBDIV. SHEET)
SUBJECT: STABILITY	PREP. BY GH DATE 8/5/83
	CHCKD. BY AdE DATE 08/08/83

$$14164 - .6428R + .766T = 0 \qquad (2)'$$
$$20385 - .766R - .6428T = 0 \qquad (3)'$$

$(2)'\left(\frac{.6428}{.766}\right)$ $$11886 - .5394R + .6428T = 0 \qquad (2)''$$

$(2)'' + (3)'$ $$32271 - 1.3054R = 0$$
$$R = 24721^{k}$$

R INTO (2)'
$$14164 - 15890 + .766T = 0$$
$$T = +2254 \qquad \mu = \frac{2254}{24721} = .09 < 0.7$$

∴ VESSEL IS HORIZONTALLY STABLE, TIPPING OCCURS WHEN L=0

FIND K(M + 80.43F) FOR L=0 IN (1)

$$K = \frac{1,792,741}{(1427715 - 661568)} = 2.34$$

INCREASE F IN EQ (3)

(3)'' $$2.34(17750) - R(.766) - T(.6428) = 0$$
$$41534 - .766R - .6428T = 0$$
(2) $$26562 - .6428R + .766T = 0$$
(3)''' $$49494 - .9128R - .766T = 0$$
(2)+(3)''' $$76056 - 1.5556R = 0$$
$$R = 48892$$

(2) $$26562 - 31422 + .766T = 0$$
$$T = +6346$$

IF K=2.34 TIPPING STARTS
STILL NO SLIDING SINCE $\frac{T}{R} = \frac{6346}{48892} = .13 << 0.7$

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO.	8304
SUBJ. – SUBDIV. – SHEET	C-1 – 1 – 5
PROJECT	W3-MAT
CLIENT	LOUISIANA POWER & LIGHT
SUBJECT	STEEL CONT./SHIELD BLDG. STABILITY
PREP. BY	AdB DATE 08/04/83
CHCKD. BY	GH DATE 8/8/83

NEXT CALCULATE THE FACTORS OF SAFETY AGAINST UPLIFT, SLIDING AND OVERTURNING AT TOP OF MAT, AT EL -35.0 FT. CALCULATE THE ADDITIONAL SHEAR AND MOMENT DUE TO DBE ACTING ON SHIELD BUILDING. (SEE PP. 2, 5 & 6 OF REF. 2 CALC).

PT.	WEIGHT (K)	E-W DBE (G)	FORCE (K)	$\sqrt{2}$ × FORCE* (K)	ELEV (FT)	ELEV+35.0' (FT)	M @ TOP MAT (KFT)
1	7010	0.494021	3463.09	4898	200.13	235.1	1,151,520
2	4959	0.453282	2247.83	3179	172.4	207.4	659,325
3	4318	0.422525	1824.46	2580	150.7	185.7	479,106
4	4104	0.394001	1616.98	2287	131.0	166.0	379,642
5	4446	0.364952	1622.58	2295	111.0	146.0	335,070
6	6242	0.329367	2055.91	2907	86.0	121.0	351,747
7	4446	0.295310	1312.95	1857	61.0	96.0	178,272
8	4104	0.270141	1108.66	1568	41.0	76.0	119,168
9	5301	0.245203	1299.82	1838	19.0	54.0	99,252
10	2822	0.226192	638.31	903	0.0	35.0	31,605
11	10173	0.210660	2143.04	3031	-18.0	17.0	51,527
			Σ:	27343.		Σ:	3,836,234

* SRSS (CONSERVATIVE) OF E-W & N-S DBE

FROM P.4 OF THE REF. 2 CALC,

$F = \sqrt{2} \times 12551 = 17750$ K

$M @ EL{-1.5} = \sqrt{2} \times 466008.6 = 659036$ KFT

TRANSFERRING THIS MOMENT TO TOP OF MAT:

$M @ \text{TOP MAT} = 659036 + 17750 \times (35 - 1.5)$
$= 1{,}253{,}661$ KFT

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJ. NO. 8302
C-1 -1 -6
SUBJ. SUBDIV. SHEET

PROJECT W3 - MAT
CLIENT LOUISIANA POWER & LIGHT
SUBJECT STEEL CON. [illegible] BLDG. STABILITY

PREP. BY AdB DATE 08/04/83
CHCKD. BY GH DATE 8/8/83

THE TOTAL SHEAR AND MOMENT ACTING AT THE TOP OF THE MAT IS :

$$F = 17750 + 27343$$
$$= 45093\ K$$

$$M = 3,836,234 + 1,253,061$$
$$= 5089295\ K\text{-}FT$$

THE BUOYANT PRESSURE AT THE TOP OF THE MAT IS :

$$p = \gamma h = 0.0624(35.00 - 1.50)$$
$$= 2.09\ KSF$$

THE TOTAL BUOYANT FORCE IS

$$B = 2.09 \times \pi \times 77^2$$
$$= 38,929\ K$$

FROM P.3 OF THE REF. 2 CALC, THE TOTAL VOLUME OF CONCRETE IS 35218 CY. FROM P.4 OF THE REF. 2 CALC, THE TOTAL WEIGHT OF STEEL IS 9007 K.

THE TOTAL DEAD WEIGHT IS :

$$W = 35218 \times 27 \times 0.150 + 9007$$
$$= 151640\ K$$

FACTOR OF SAFETY AGAINST UPLIFT*

$$= \frac{(1.0 - 0.17) \times 151640}{38929}$$

$$= 3.23$$

* BUOYANT FORCE NOT REDUCED BY VERTICAL DBE

HARSTEAD ENGINEERING ASSOCIATES • INC.
169 KINDERKAMACK ROAD, PARK RIDGE, N. J. 07656

PROJECT	W3-MAT	PROJ. NO.	2304
CLIENT	LOUISIANA POWER & LIGHT	C-1 / -1 / -7	SUBJ. / SUBDIV. / SHEET
SUBJECT	STEEL CONT./SHIELD BLDG. STABILITY	PREP. BY AdE	DATE 08/04/83
		CHCKD. BY GH	DATE 8/8/83

FACTOR OF SAFETY AGAINST SLIDING*

$$= \frac{[0.83 \times 151640 - 38929] \times 0.7}{45093}$$

$$= 1.35$$

COMPUTE THE FACTOR OF SAFETY AGAINST OVERTURNING*:

RIGHTING MOMENT $= (0.83 \times 151640 - 38929) \times 77$

$= 6.694 \times E6$ KFT

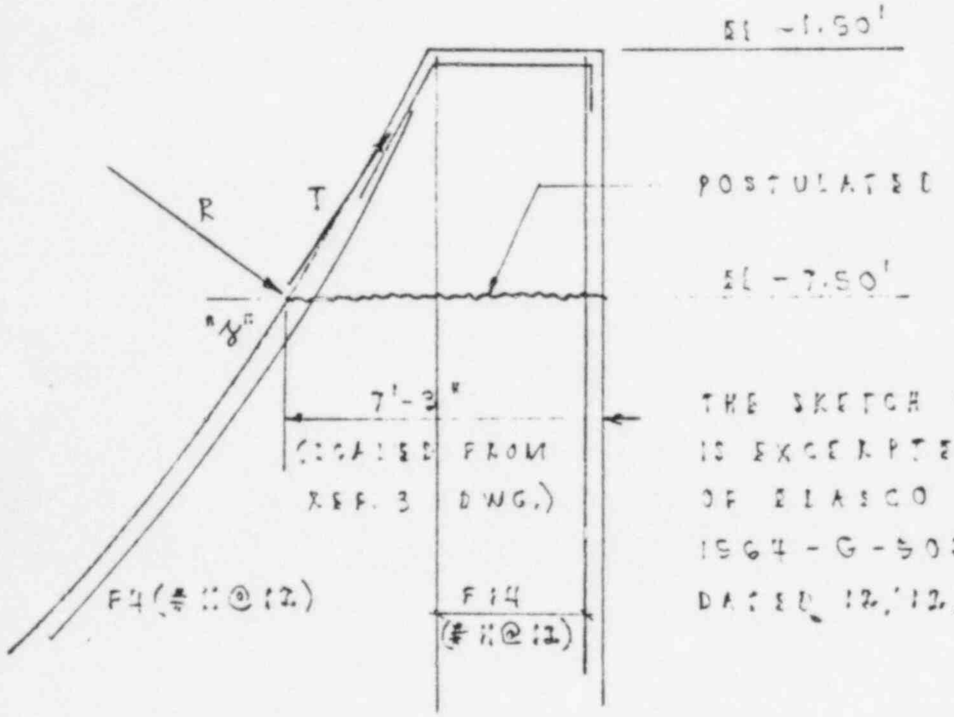
FACTOR OF SAFETY AGAINST OVERTURNING

$$= \frac{6.694\ E6}{5.090\ E6}$$

$$= 1.32$$

* BUOYANT FORCE NOT REDUCED BY VERTICAL DLE

USING THE MAGNITUDES OF THE REACTION FORCES T, R COMPUTED ON PAGE 4, DETERMINE WHETHER THE UNDERLYING REINFORCED CONCRETE IS STRUCTURALLY ADEQUATE.



THE SKETCH OF THE GEOMETRY IS EXCERPTED FROM DETAIL 'B' OF EBASCO DRAWING LOU-1564-G-502, REV. NO. 6, DATED 12/12/78 (REFERENCE 3)

SINCE T, R ARE COMPUTED AS SRSS OF N-S AND E-W DBE, AND SINCE THE RESPECTIVE BEARING POINTS ARE 90° APART, REDUCE THESE FORCES BY $\sqrt{2}$.

FROM P. 4, $T = 2254k/\sqrt{2}$

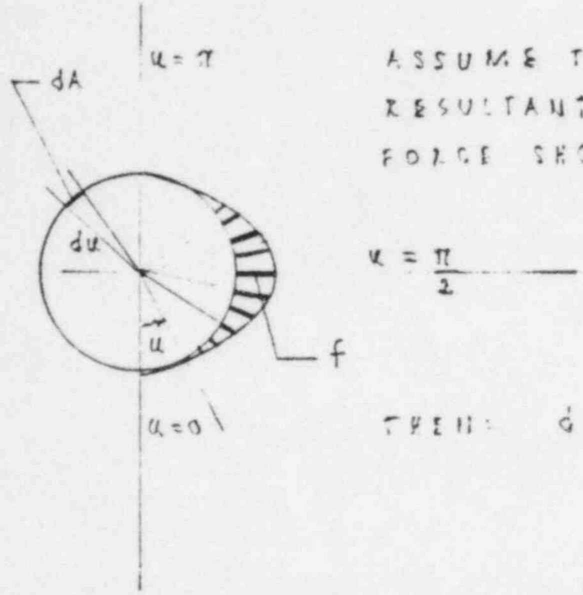
$R = 24721k/\sqrt{2}$

COMPUTING THE HORIZONTAL COMPONENT:

$$H = R \sin 50° + T \cos 50°$$

$$= \frac{24721}{\sqrt{2}} \times 0.7660 + \frac{2254}{\sqrt{2}} \times 0.6428$$

$$= \frac{20385k}{\sqrt{2}} = 14414k$$



ASSUME THAT $H = 14414\,k$ IS THE RESULTANT OF THE SINUSOIDALLY DISTRIBUTED FORCE SHOWN ON THE LEFT.

THEN: $dR' = f(u)\,dA$

$$= f \sin u \; r\,du$$

$$R' = \int_{u=0}^{\pi} f \sin u \; r\,du$$

$$R' = -fr(\cos u)\Big|_0^{\pi} = 2fr$$

SO THAT:

$$f = \frac{R'}{2r} = \frac{14414\,k}{2 \times 70.0}$$

$$f = 103\,k/ft$$

COMPUTING THE SHEAR STRESS v_u:

$$v_u = \frac{V_u}{\phi b d} = \frac{103\,k}{0.85 \times 12'' \times 7.25 \times 12}$$

$$v_u = 0.116\,ksi = 116\,psi$$

EVALUATING v_u AGAINST $2\sqrt{f_c'}$, FOR 4000 PSI CONCRETE (SEE NOTE, REF. 3):

$$2\sqrt{4000} = 126\,psi > v_u \quad OK.$$

APPENDIX M

Laboratory Report

twin city testing
and engineering laboratory, inc.
662 CROMWELL AVENUE
ST PAUL, MN 55114
PHONE 612/645-3601

REPORT OF: IDENTIFICATION OF LEACHATE

PROJECT: LOUISIANA POWER & LIGHT
PROJECT NUMBER 8304

DATE: September 9, 1983

REPORTED TO: Harstead Engineering Assoc Inc
Attn: Gunner Harstead
169 Kinderkamack Rd
Park Ridge, NJ 07656

FURNISHED BY:

COPIES TO:

LABORATORY No. 1-34799

INTRODUCTION

This report presents the results of our recent testing of samples you submitted for analysis. We received four samples; three liquid and one solid, for testing. The samples were identified as follows:

1. Liquid Conduit
2. Liquid, Pit
3. Liquid, Crack
4. Leachate

We understand the samples were taken from a reinforced concrete mat foundation. The foundation is under hydrostatic pressure from an elevated water table. The purpose of our testing is to evaluate the likelihood of corrosion in the reinforced concrete.

CONCLUSIONS

Based on the results of our testing, it is our opinion the following conclusions are appropriate:

1. The leachate consists primarily of calcium carbonate and iron. Much of the iron is magnetic, suggesting a form such as magnetite. The iron appears as fine wire-like pieces under magnification.

2. The water removed from the conduit is substantially different than the water obtained from the crack and the pit. The high pH and alkalinity of the conduit sample suggests the water has been in contact with the concrete for an extended period of time.

3. The chloride level in the water is sufficiently low to classify the fluid as not being aggressive.

TESTING METHODS AND RESULTS

On August 18, 1983, we received four samples for analysis. The samples consisted of three plastic containers of liquid and one solid leachate sample. Each of the fluid samples was tested for pH using colorphast indicator sticks. Also, each fluid sample was analyzed for chloride using the Standard Methods for Water Analysis, 407A. In addition, alkalinity, iron, calcium and sodium was determined for each of the fluid samples using EPA Method 600/4-79-020. The following results were obtained:

twin city testing
and engineering laboratory, inc.
662 CROMWELL AVENUE
ST. PAUL, MN 55114
PHONE 612/645-3601

REPORT OF: IDENTIFICATION OF LEACHATE

DATE: September 9, 1983

LABORATORY No. 1-34799

TESTING METHODS AND RESULTS (cont.)

Constituent	Sample 1	Sample 2	Sample 3
pH	12.5	7.5	7.5
Iron (ppm)	ND*	ND	1.7
Calcium (ppm)	375	71	31
Sodium (ppm)	2400	1400	5100
Chloride (ppm)	78	20	22
Alkalinity ($CaCO_3$) (ppm)	1300	-	-

*ND = Not Detected

The leachate sample was analyzed using a Jarrell Ash Emission Spectrograph. The sample was placed in carbon electrodes, and a film of the spectra was obtained with a D.C. arc. The following constituents were identified:

Concentration	Constituent(s)
Major Constituent (10% or greater)	Iron, Calcium
Minor Constituent (10% to 1%)	Sodium, Aluminum
Trace Constituent (1% or less)	Aluminum, Magnesium, Manganese, Titanium, Barium, Copper

The leachate sample was also analyzed using X-ray diffraction techniques. The diffraction analysis identifies crystalline material which is present in the sample. The sample contains a major amount of calcium carbonate.

REMARKS

Scale found on the surface of Portland cement concrete is typically comprised of calcium carbonate. During the hydration of Portland cement, calcium hydroxide is liberated. In the presence of carbon dioxide, the calcium hydroxide will form calcium carbonate. The carbonation layer is generally limited to the top 1/8" of a quality concrete.

twin city testing
and engineering laboratory, inc.
662 CROMWELL AVENUE
ST. PAUL, MN 55114
PHONE 612/645-3601

REPORT OF: IDENTIFICATION OF LEACHATE

DATE: September 9, 1983

LABORATORY No. 1-34799

REMARKS
(cont.)

The corrosion of reinforcing steel may form a magnetic residue such as magnetite. This formation requires an aqueous environment where oxygen levels are low. The very low iron content of the water samples suggests the water was not in contact with steel actively corroding. The formation of magnetite is observed frequently when steel corrodes in a chloride contaminated cementitious material and is then exposed to air. The low chloride levels found in the water suggest the presence of the iron in the leachate is not from such a condition. The test results are consistent with the iron originating from the surface of the slab.

TWIN CITY TESTING AND
ENGINEERING LABORATORY INC

Richard D. Stehly

Richard D Stehly, P.E.
Chief Engineer

RDS/st