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RHODE,G.K. Niagara Mohawk Power Corp.
RECIP.NAME RECIPIENT AFFILIATION
RUBENSTEIN,L.S. Light Water Reactors Branch 4

SUBJECT: Forwards geological info for util as per 791025 ltr to NRC.
Response to Request 361.5 will be provided by 800229.
Response to Request 361.24 will be submitted w/responses to
remaining four requests by 800601.

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NIAGARA MOHAWK POWER CORPORATION/300 ERIE BOULEVARD WEST, SYRACUSE, N.Y. 13202/TELEPHONE (315) 474-1511

January 30, 1980

Mr. L. S. Rubenstein, Acting Chief
Light Water Reactors Branch No. 4
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Mr. Rubenstein:

Re: Nine Mile Point Unit 2
Docket No. 50-410

Our letter of October 25, 1979 provided a schedule for providing geological information for Nine Mile Point Unit 2. Attached is this information except for Requests 361.5 and 361.24.

The response to Request 361.5 will be provided by February 29, 1980. The response to Request 361.24 will be included with the responses to remaining four requests. These are still scheduled to be submitted by June 1, 1980.

Very truly yours,

NIAGARA MOHAWK POWER CORPORATION

Gerald K. Rhode
Vice President
System Project Management

PEF/kmb
Attachment

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Q361.3 During the telephone conversation referenced above, the licensee stated that the structures are, in effect, isolated from the bedrock by the presence of a slot cut between the structure walls and foundation and rock. The slot below the foundation slabs is filled with vermiculite concrete. What material is to be used in the slots around the walls? What is the design width of the slots, and what is the basis for the selection of that slot width?

RESPONSE 1. There are two types of "slots" at Nine Mile Point - Unit 2:

a. Rock Slot Behind 12-Line Wall

This 8- to 12-inch wide drilled rock slot is located approximately 35 feet west of the 12-line wall. The slot was drilled parallel to the 12-line wall and is approximately 35 feet deep and 260 feet long. The slot was filled with loosely placed expanded vermiculite.

b. Slots Between Buildings and Rock

These are actually spaces between the excavated rock walls and the building walls and foundations.

2. Filler materials in the slots

Based on postulated rock movements, all backfill materials were selected to minimize the stresses that would be transmitted to the building walls. The materials selected are:

a. Vermiculite concrete is being used around the foundation mats.

b. A one foot width of cross-linked polyetheylene foam (Nufoam, as manufactured by Republic Systems Corporation), is being provided from the top of the Reactor Building mat to the top of the rock at the Reactor Building ares.

c. Loosely placed expanded vermiculite backfill or Nufoam are being used to fill the slots around other safety-related building walls.

The width of the slots between rock and building walls varies from 6 inches to 6 feet depending upon construction conditions. The nominal design width required was 6 inches based on the criteria of rock movement.

Q361.4 Was there evidence of heave in the bedrock floor or differential movements along faults, joints or bedding planes during the excavation of the Unit 2 site, or at any time following the completion of those excavations? Describe the amount, direction and rate of movements of any such adjustments.

RESPONSE All excavated rock surfaces were routinely checked by geologists during the progress of the excavation. Several lines of evidence were observed that can be attributed to heave of the bedrock floor, differential movements along bedding planes, and separation along joints. These observations include small buckle folds in the floors of excavations, a displaced heave monument borehole, offset boreholes and inclinometer readings that indicate bedding plane slippage. Descriptions of the various observations are provided.

A. Buckle Folds:

A buckle fold in the Control Building was noticed on August 12, 1976, following removal of muck which had been left in place since February 3, 1976 when a 25-foot rock column was blasted. Therefore, a period of six months, including the last part of winter, was available for heave development. This buckle appeared in a 1 to 2 inch thick shale layer overlying silty sandstone-siltstone, was 35 to 40 feet long and 2 to 4 feet wide with a N35°W trend, a 2-inch maximum amplitude and cracks on the crest (Figure 361.4-1). The floor of the Control Building excavation is 38 feet below the original bedrock surface.

The floor in the Circulating Water Encasement had not shown any anomalous features for four months from May 19, 1976, when the rock was removed immediately after blasting to September 20, 1976, when the floor was cleaned and geologically inspected. The buckle was discovered six months later at the beginning of April 1977, when the floor was again inspected in preparation for pouring mat concrete. The buckle was approximately 14 feet long and the crest-line was oriented N08°W to N30°E. The wave length of the buckle ranged from approximately 3 feet to 9.5 to 10 feet and the amplitude 1.75 inches. Dilation was apparent between a folded siltstone bed and an underlying siltstone bed creating a 1.75 inch void. The thickness of the arched layer ranged from a minimum of 3.5 inches to a maximum of 7 inches (Figure 361.4-1). The floor of the Circulating Water Encasement excavation is 35 feet below the original bedrock surface. Each of these buckle folds were confined to the floors of the excavations and did not affect the bedrock walls.

Once the rock loosened by the buckles was removed, no further buckling was noted, and the area was subsequently covered with fill concrete mud mats.

B. Open Joints:

During the excavation of Unit 2, numerous open joints were noted. In most instances the age of the opening of the joints was not determinable, but in several areas, especially in the Cooling Water Piping Trench, the joints appear to have opened up after the spray painted grid lines were applied to the rock surface. The joints are open from 2 to 20 millimeters and inspection of the area indicates that there are no fractures or open faces towards which the rock could have moved to produce the openings.

These open joints could be interpreted to be a result of minor arching or heaving of the excavation floors as a result of the horizontal stresses and the removal of the vertical confining pressure.

C. Offset Drill Holes:

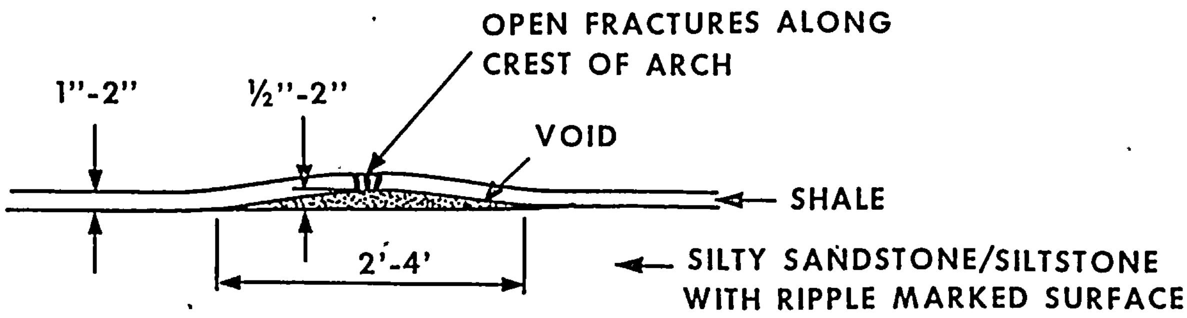
Many of the boreholes drilled for blasting for the Unit 2 excavation are offset along bedding planes in the bedrock. Most of the offsets indicate movements on the order of 1/4 to 1/2 inch, but displacements of up to 7 inches were observed. The offset boreholes appeared to be located throughout the site, generally near the surface, but several offsets were noted near the bottom of the reactor excavation. Displacements were generally directed towards the free face created by the excavation, but several offsets were noted parallel to the free face. Conclusive determination of the cause of the offsets is difficult because of the possibility of gas pressures being built up during the blasting (especially in shale layers), resulting in a lifting of the overlying column of rock and a possible shift along the bedding planes towards the free face. However, in the excavation for the exploratory trenches along the Cooling Tower Fault, offset drill holes were noted. No explosives were used in these excavations. The offsets generally were about 1/4 to 1/2 inches with displacements of the overlying beds into the small (5 feet x 12 feet) rock excavations. These resultant displacements were typically observed approximately one month after the slots were excavated. Minor offsets were detected within one to two weeks following excavation. The movements could be interpreted to be bedding plane slip adjustments with the removal of the lateral confining pressure.

D. Displaced Heave Monument:

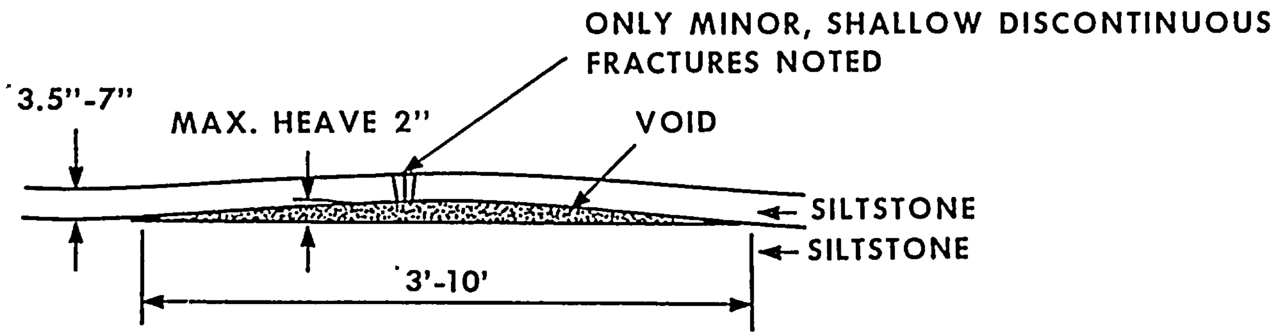
After the reactor excavation was completed to final grade, the upper portion (approximately 4 inches) of a borehole, originally drilled October 1975 for installation of a heave monument, in the bottom of the reactor excavation, was displaced 1.8 inches. The elevation of the base of the excavation was irregular. The displacement of the hole is attributed to a 4 inch thick bed of sandstone that slid towards the east over a shale layer and partially closed off the top western part of the borehole on the floor of the excavation.

E. Instrumentation:

Because of the previously mentioned observations that suggest differential movement along bedding planes, various types of instrumentation were established at the Unit 2 site. Inclinometers were installed in the bedrock in the northwestern portion of the site near the intake and discharge shaft and along the Circulating Water Piping Trench. This part of the site was chosen because the piping trenches were generally the most recently excavated. Inclinometers were also installed in the perimeter of the trench 5 excavation to monitor displacements during and after excavation. Monitoring of the inclinometer installations has been regular over the last two years and monthly readings are continuing. Generally, most of the inclinometers demonstrate some apparent relative displacement of the rock mass at some time during the monitoring period. Maximum displacements of approximately 5 millimeters are observed at the installations closest to the major excavations. Displacements are generally non-linear with time and apparent reversals of established displacements are commonplace. Details regarding the location of the inclinometers and results of the monitoring are presented in the response to Question 361.5.



TYPICAL HEAVE CROSS-SECTION IN CONTROL BLDG. AREA



TYPICAL HEAVE CROSS-SECTION IN CIRCULATING WATER ENCASEMENT AREA

FIG 361.4-1

TYPICAL HEAVE CROSS-SECTION

NINE MILE POINT NUCLEAR STATION UNIT 2

NIAGARA MOHAWK POWER CORP

Q361.6 The thorough investigation and analysis of the Cooling Tower fault has allowed for a reasonable interpretation with respect to its history of movements since Late Paleozoic, including a determination of the causes of those respective movements. Evidence acquired in a smaller study is also presented which leads to a conclusion that the Drainage Ditch fault is similar to the Cooling Tower fault with respect to origin, history of displacements, senses of movement, and orientation. A postulated projection of the Drainage Ditch fault would intersect the tunnels beneath Lake Ontario. Evaluate the potential effects on an engineered structure, including intake and discharge tunnels, overlying or cutting across such a feature, considering the types of adjustments (rates and magnitudes of movements) that you consider possible in the future.

RESPONSE Based on previous field investigations, it is logical to assume that the Drainage Ditch Fault is similar in character, origin and history of development to the Cooling Tower Fault. Extrapolation of the Drainage Ditch Fault trend indicates that it would likely intersect the lake water tunnels. However, examination of the rock cores from the widely-spaced borings drilled for the lake water tunnels disclosed no direct evidence of the fault. Nevertheless, for the purposes of preliminary design considerations; we have assumed that the character of rock deformation along the Drainage Ditch Fault will be similar to that of the Cooling Tower Fault at similar depths and stratigraphic position. The final design for the tunnel crossing of the Drainage Ditch Fault has not been completed at this time. The tunnel bore will be excavated, and if the fault is encountered, it will be geologically investigated and analyzed. Based on these investigations a final design for the intake "pipe" (see Response to Question 361.18 for details on the proposed tunnel design) will be prepared. At the present time, the conceptual alternatives for design of the pipe consist of bridging the fault with the pipe or designing a pipe with flexibility capable of accommodating the predicted movements. Additional design alternatives may be forthcoming based on results of the geological investigation and analysis of the fault.

Q361.7 Evidence was presented in the report that demonstrated that the Cooling Tower Fault zone was of limited extent, at least to the west-northwest, and that the most recent movements on the fault were confined to the upper 200 feet of rock. Can a similar conclusion be made regarding the thrust faults and the Drainage Ditch Fault? Provide the basis for your conclusion.

RESPONSE No direct evidence was collected during the investigation at Nine Mile Point that was relevant to the precise determinations of the lateral extent and the depth of development of the Drainage Ditch Fault. However, some inferences can be made with regard to its possible dimensions using information obtained during previous geologic investigations.

The Drainage Ditch Fault and the "Teepee Fold" observed in the Turbine Room excavation of the James A. FitzPatrick facility resemble each other in attitude and appearance and they trend toward each other. On this basis, it is logical to assume that they are the same structure. Trenching conducted during the investigation of the "Teepee Fold" indicated that it extended approximately 2000 feet east of the James A. FitzPatrick facility. Considering this together with the location of the exposure at the drainage ditch, the minimum lateral extent of this fault is approximately 3000 feet (Niagara Mohawk Power Corporation, Geologic Investigation, Volume I, Geology, 1978, Plate 1-1).

During the investigation of the Teepee Fold, an air photo lineament was noted to extend 6000 feet east of the James A. FitzPatrick facility, in a location that could indicate that it may be a surficial expression of the structure. If this lineament is assumed to correspond to the Drainage Ditch structure, then its lateral extent is greater than one mile.

From the results of investigation of the Cooling Tower Fault it is known that the depth of development of the reverse-slip displacements and the rotated bedrock sliver must be of limited extent. The depth to which buckling develops is controlled by four factors. These are:

1. The wavelength of the initial deflection of the strata;
2. The inclination of the pre-existing discontinuity;
3. The magnitude of the acting stress; and
4. The thickness of bedding.

In the case of the Drainage Ditch Fault only the last factor is known not to be significantly different from the same factor for the Cooling Tower Fault. Therefore, it is not possible to directly extrapolate the depth of development of buckling on the Drainage Ditch Fault by comparison to the Cooling Tower Fault. However, it is not anticipated that the differences in the depth of development between the two structures are very large. Thus, it seems that the depth of buckling along the Drainage Ditch feature is not great, and possibly on the order of 200 feet.

With reference to the thrust faults noted in the intake shaft and heater bay excavations, it is reasonable to anticipate that the depth of development of these structures is likewise limited. The structures are thought to be developed as a result of bedding plane instability. Their development results from the existence of shear stress acting along bedding planes coupled with relatively minor shear strength for the majority of bedding planes. This shear strength can be estimated using the Coulomb-Navier criterion of shear failure:

$$\tau = C + \sigma_n \cdot \tan \phi$$

where tan ϕ is (on the average) approximately 22°, C is approximately 10 psi, and σ_n is variable depending upon the burial depth of a specific bedding plane. Assuming that the shear stress in the bedrock at Nine Mile Point has a certain finite value, then there must be a depth below which the shear stress on bedding planes is smaller than the strength of the bedding planes. It is anticipated that the development of the referenced thrust faults is limited to that depth. With the data presently available, it is not possible to ascertain whether or not a depth of 200 feet represents the maximum depth of development of these thrust faults.

Q361.8 On page II-C-1 and Table II-D-3 in Volume II, it is not clear where the MP sample series came from. Provide profiles and/or plans showing the precise location of these samples.

RESPONSE The MP series are sample locations for compositional analysis. Appendix II-C (Niagara Mohawk Power Corporation, Geologic Investigations, Volume II, Geomorphology, Vertical Crustal Movements, Seismicity, 1978) contains the results of the grain size analysis illustrated on gradation curves. Appendix II-D (NMPC, Volume II, 1978) contains the information on the mineralogical analyses. The attached Table (Figure 361.8-1) provides information on the MP series giving the sample number in Column 1, the location of the sample in terms of the cross section number, wall, and the trench number in Column 2 and Column 3 provides the plate number from the report where the sample location is presented graphically. All plate numbers for the MP series appear in Niagara Mohawk Power Corporation, Geologic Investigation, Volume I, Geology, 1978.

TABLE 361.8-1

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Q361.8

SAMPLE NO.	LOCATION	REFERENCE IN REPORT Volume I
MP-1	XS 5-1 East wall, Trench 5	Plate 4-48
MP-2	XS 4-1A West wall, Trench 4	Plate 4-35
MP-3	XS 4-1A West wall, Trench 4	Plate 4-35
MP-4	XS 4-1A West wall, Trench 4	Plate 4-35
MP-5	XS 4-1A West wall, Trench 4	Plate 4-35
MP-6	XS 4-1A West wall, Trench 4	Plate 4-35
MP-7	XS 4-1A West wall, Trench 4	Plate 4-35
MP-8	XS 4-1A West wall, Trench 4	Plate 4-35
MP-9	XS 4-1A West wall, Trench 4	Plate 4-35
MP-10	XS 4-1A West wall, Trench 4	Plate 4-35
MP-11	XS 4-1A West wall, Trench 4	Plate 4-35
MP-12	XS 4-1B West wall, Trench 4	Plate 4-36
MP-13	XS 4-1B West wall, Trench 4	Plate 4-36
MP-14	XS 4-1B West wall, Trench 4	Plate 4-36
MP-15	XS 4-1B West wall, Trench 4	Plate 4-36
MP-16	XS 4-1B West wall, Trench 4	Plate 4-36

Q361.8

SAMPLE NO.	LOCATION	REFERENCE IN REPORT Volume I
MP-17	XS 4-3A East wall, Trench 4	Plate 4-41
MP-18	XS 4-3A East wall, Trench 4	Plate 4-41
MP-19	XS 4-3A East wall, Trench 4	Plate 4-41
MP-20	XS 4-3C East wall, Trench 4	Plate 4-43
MP-21	XS 4-3C East wall, Trench 4	Plate 4-43
MP-22	XS 4-3C East wall, Trench 4	Plate 4-43
MP-23	XS 5-2 West wall, Trench 5	Plate 4-49
MP-24	XS 5-2 West wall, Trench 5	Plate 4-49

[illegible]

Q361.9 In Volume III, pages 3-14 through 3-18, low stress values measured in Boring OC-3 are attributed to the close proximity of this boring to a projection of the Cooling Tower Fault to the west. An inconsistency is apparent in that mostly high stress measurements were obtained in Boring OC-2, which is drilled adjacent to the Cooling Tower Fault. Please discuss how nearness to the fault can explain low in situ stress values in one area but does not seem to effect the magnitude of stress in another location near the fault.

RESPONSE Low stress differences in Borehole OC-3, not low magnitudes of maximum horizontal stress, are attributed to proximity to the fault. Below 45 feet depth, maximum stress in OC-3 is interpreted to be 950-1100 pounds per square inch (with the exception of local variation attributed to bending); maximum stress in Borehold OC-2 is stated to be typically 1200 - 1400 pounds per square inch.

Additional factors which would relate to differences of stress between the two locations are:

(i) The boreholes are interpreted to be located on different sides of the fault and therefore may have experienced different stress histories.

(ii) Borehole OC-2 was drilled very close to the trace of the fault and although the exact location of the fault is unknown in the vicinity of OC-3, it is improbable that Borehole OC-3 was drilled an identical distance from the fault (Niagara Mohawk Power Corporation, Geologic Investigation, Volume III, Rock Stresses, 1978, Plate 1-1).

Q361.10 Reference is made to Volume III, last paragraph on page 3-23 and item 3 on page 3-24. The former states that the maximum principal stress outside of the fault block plunges toward the southeast. The latter indicates that areas unaffected by faulting show a southwest plunge of the stress trajectory. Please clarify this apparent inconsistency.

RESPONSE The statement on page 3-23 to the effect that the stress trajectory trends northeast and plunges south east is in error. These should read that the stress trajectory trends northeast and plunges southwest.

Q361.11 Please define "unacceptable lag times" as used in the next-to-last paragraph on page 5-2, Volume III.

RESPONSE The term "lag time" is used to denote the time for a change of pore pressure in the rock mass to be registered as a change of water level in the standpipe. This depends on rock mass hydraulic characteristics, magnitude and rate of pressure change, and standpipe construction.

Sensitivity of standpipes to changes of pore pressure is discussed in Appendix Section III-J-5 (Niagara Mohawk Power Corporation, Geologic Investigation, Volume III, Rock Stresses, 1978). These standpipes were not designed to give maximum possible accuracy and response to pore pressure changes, but for simplicity and reliability to give an approximate representation of pore pressure and pore pressure changes. The observed changes and fluctuations of water levels indicated that the lag time was such that readings were representative and the standpipes were sensitive to significant changes of pore pressure.

The term "unacceptable lag times" was used in discussion of lag times associated with unrepresentative readings caused by insensitivity to changes of pore pressure.

Q362.12 Provide an analysis to support your hypothesis in Volume III Section 5.0 that the lower piezometric head in the vicinity of the fault zone is the result of an increase in storage space within the Whetstone formation due to a continuation of buckling along the fault zone.

RESPONSE Appendix Section III-J-6.2 (Niagara Mohawk Power Corporation, Geologic Investigation, . Volume III, Rock Stresses, 1978) states that computations "suggest that the low water level measurements in Unit A of the Whetstone Gulf Formation could be related to dilation caused by buckling and an associated increase of storage capacity in the structure". This conclusion is stated to be tentative and to be based on various assumptions and poorly defined hydrologic parameters. The explanation for the low water levels related to dilation associated with buckling is not therefore presented as a firm conclusion and a simple analysis to support the explanation is presented in the 1978 report (Appendix III-J-6).

More detailed analysis would require numerical values for parameters which are not available. These parameters include:

- history of rates of buckling and associated creation of storage space.
- permeability and storage around the fault zone.
- history of groundwater head gradient.

Q361.15 On the Surficial Geology map, Plate 1-11, Volume 11, a circular peat deposit, about 1/4 mile in diameter is mapped immediately south of the site. While all other deposits of peat and rock shown on the map are related in some way to mapped drainage systems, this one is not. Discuss the origin of this feature and provide the data supporting your interpretation.

RESPONSE The circular peat deposit reference on Plate 1-11 (Niagara Mohawk Power Corporation, Geologic Investigation, Volume II, Geomorphology, Vertical Crustal Movements, Seismicity, 1978) is related to drainage. Examinations of aerial photographs taken in 1938 and 1961, prior to any construction at the site, illustrate roughly south to north drainage into and from the peat deposit.

Q361.16 The west-northwest termination of the Cooling Tower Fault is apparently somewhere between Trench 2 and Pit 1. What is your estimate of the east-southeast extent of the Cooling Tower Fault? Provide the basis for your answer.

RESPONSE The minimum lateral extent of the Cooling Tower Fault as determined by our investigation is approximately 3,000 feet (Niagara Mohawk Power Corporation, Geologic Investigation, Volume I, Geology, 1978, Section 4, Page 27 and Plate 1-1). With the data presently available, it is difficult to provide an accurate estimate of its extent to the east-southeast. However, it may be possible to provide a crude estimate of size of the fault based on the magnitude of the displacements observed along it.

The episodes of faulting which generated this structure were both strike-slip and normal-slip. The stratigraphic displacements incurred during these episodes of faulting were only on the order of several feet. The largest amount of lateral displacement on the fault was observed in the Cooling Tower Trench and was equal to approximately three feet (Volume 1, Section 4, Page 6). At a depth of approximately 200 feet a normal displacement of 0.5 feet was inferred on the basis of stratigraphic correlations. The total amount of normal displacement above this depth is not known because the reverse-slip movements along the fault have eradicated the older stratigraphic displacement.

Based on these observations, it is difficult to envision this fault to be a major tectonic element. Its lateral extent may exceed one mile, but if so probably not by a significant amount.

Q361.17 To what geologic event do you attribute the development of the zone of breccia along the Cooling Tower Fault?

RESPONSE The zone of breccia along the Cooling Tower Fault was initiated and further developed owing to three types of movement which occurred along the fault. These movements occurred at different geologic times in relation to different geologic events. It is not clear in your question from the term "zone of breccia" whether you mean this in the sense of a "zone of a discontinuity" or in the sense of a "zone of cataclasis". For this reason we will answer the question considering both aspects.

In the sense of a zone of discontinuity, the Cooling Tower Fault was developed as a result of two different types of shear failure. These movements consisted of strike-slip and normal-slip. Initially, the fracture was developed only in the stiffest strata of the sedimentary section at the site, namely the Oswego Sandstone. Minerals, whose crystallization occurred, synchronously with the strike-slip faulting, provided evidence which can be interpreted such that the faulting occurred in late Paleozoic time. This time of development could conceivably suggest that strike-slip faulting occurred as the result of orogenic shortening within the Appalachian geosyncline; however, the characteristic feature of the strike-slip faults and related fractures is that they are developed only in the stiffest members of the stratigraphic section. This characteristic indicates that the amount of shortening was very small, and that the rates of straining were likewise small. Fractures bearing the foregoing characteristic have been postulated to be a result of the response of sediments within an elliptical basin to vertical differential movements affecting the basin (Price, N.J., 1974, The Development of Stress Systems and Fracture Patterns in Undeformed Sediments in Advances in Rock Mechanics, Volume 1, Part A, Washington, D.C., National Academy of Sciences, Pages 487-496). It should be reiterated that the initial development of the discontinuity occurred in this manner in Late Paleozoic time during regional epeirogenesis.

Subsequent to the strike-slip fracturing, the discontinuity was further developed and propagated downward into the Pulaski and the Whetstone Gulf Formations during the episode of normal faulting. This faulting was synchronous with the crystallization

of minerals, the age of which can be interpreted to be Mesozoic on the basis of their homogenization temperatures. This normal fault is probably an element of a pervasive system of fractures and faults expressed in the bedrock of the region. The time of development of this system may be correlative with the time of emplacement of the Mesozoic igneous intrusives in the Finger Lakes region. Such a correlation would imply that the final development of the discontinuity may have occurred during, and in relation to, a tectonic episode during the Mesozoic Era which may be associated with tectonism that resulted in the opening of the Atlantic Ocean.

The third type of movement on the discontinuity, namely reverse-slip and associated bedding plane slip related to buckling, occurred utilizing the pre-existing, fully developed discontinuity. It is unlikely that these movements contributed to further propagation in space of the original discontinuity. In other words, the localization of this third episode of movement is restricted to the volume of rock in which the discontinuity already existed.

The zone of breccia, as considered in the alternative sense of cataclasis, was clearly developed during all three episodes of strike-slip, normal-slip, and reverse-slip movement.

Q361.18 At the meeting on April 24, 1979, several alternative designs of the cooling water tunnel were presented to accommodate rock squeeze. It was also stated that should it be determined that the Drainage Ditch fault intersects the funnel it may be decided to excavate beneath the distrubed area. How do you plan to determine in advance of excavation whether or not the fault intersects the tunnel route? It is our position that you keep the staff informed as to your plans and the progress of the design and construction of the tunnel. We also require that the tunnel be geologically mapped in detail and that all anomalous conditions be reported immediately to the NRC staff.

RESPONSE Extrapolation of the Drainage Ditch Fault trend indicates that it would cross the tunnel route if it persists in direction and character. Recent reexamination of the rock cores from the borings drilled for the lake tunnels disclosed no evidence of the fault. However, the borings are widely spaced and this does not conclusively rule out the presence of the fault. Beyond this, additional investigations are not planned to determine in advance if the fault intersects the tunnel route. A short feeler hole will be drilled into the tunnel heading. This is standard tunneling practice.

If the fault is encountered in the tunnel, geologic investigations of the fault will be conducted at that time.

The present tunnel design consists of two tunnels with separate chambers for the intake flow (safety-related) and the discharge and fish return flows (nonsafety-related). The safety-related intake will be a "pipe" founded on the floor of the tunnel. In this way, the intake pipe will be separated from the tunnel walls and will not be affected by any rock stresses or movements of the rock walls. Discharge water will flow through the open space remaining between the intake pipe and the tunnel boundaries. The fish-return pipe will be attached to the roof of the tunnel or to the intake pipe. Tunnel dimensions have been selected, and tunneling commenced in January 1980. The intake pipe will be a cast-in-place section. Final design of the pipe is not complete.

The tunnels will be inspected by engineering geologists and will be geologically mapped. This has always

been planned as a part of existing programs for geologic inspection and mapping of site excavations in accordance with the Preliminary Safety Analysis Report commitment. All anomalous geologic conditions discovered in the tunnels will be reported to the Nuclear Regulatory Commission, as has been our practice in the past.

It is not planned to lower the tunnel invert to avoid the fault or to excavate the fault zone to any great depth if encountered. Niagara Mohawk Power Corporation will keep the Nuclear Regulatory Commission infomred as the design and construction of the tunnel progresses.

Q361.19 It is our position that all monitoring of in-situ stress, rock measurements, and engineered structures on the site, be continued until it is determined by the NRC and its consultants that such operations are no longer necessary. In regard to the bi-monthly monitoring of the twelve line wall, it is our position that this procedure should be incorporated into the technical specifications.

RESPONSE All inclinometers, extensometers, stressmeters and piezometers are currently being monitored on a monthly basis. The 12 strain gauges installed within the 12-line wall will be read on bi-monthly intervals during construction.

Q361.20 As of the time of the April 24 meeting, it had not been decided what type of deformable material would be used to backfill the 6 inch slot between the structures walls and bedrock. We request that you keep us informed as to the decisions made and the reasons for those decisions.

RESPONSE It has been decided to use vermiculite to backfill most of the slots between the structure walls and bedrock. Placement of vermiculite or Nufoam filler materials in the slabs will maintain the slot spaces free of accumulations of rock chips or other relatively incompressible foreign materials. The vermiculite is to be loosely placed. This material will minimize load transfers to building walls by absorbing the horizontal rock displacements predicted. The vermiculite materials used are Zonolite Brand, Expanded Vermiculite, Grade No. 1 or Grade No. 2, as manufactured by W.R. Grace and Company.

Vermiculite backfill was selected because it is widely used as a construction material. It is relatively easy to install and highly compressible.

A series of one-dimensional incremental consolidation tests using soil testing oedometer equipment performed on the vermiculite. The backfill pressures due to horizontal rock movements were based on results of this type of testing. A typical compression curve is presented in Figure 316.20-1. The discontinuity shown in the curve is a result of unloading and reloading during the test.

Nufoam (reference to the Response to Question 361.3). is also being used as a backfill material in some areas, principally for construction convenience.. As demonstrated by the compressive stress-strain curve (Figure 361.20-2), the Nufoam is also highly compressible.

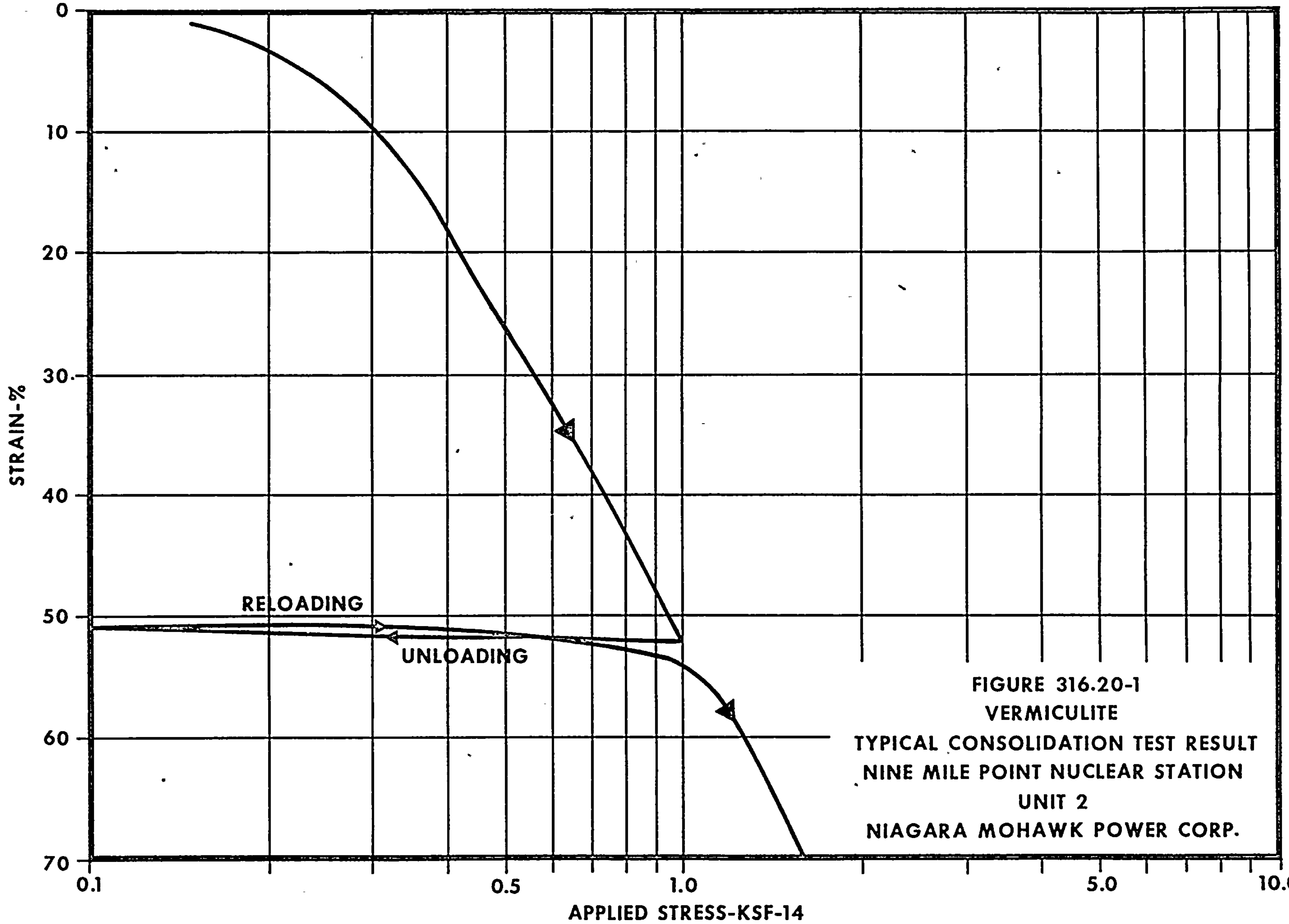


FIGURE 316.20-1
VERMICULITE
TYPICAL CONSOLIDATION TEST RESULT
NINE MILE POINT NUCLEAR STATION
UNIT 2
NIAGARA MOHAWK POWER CORP.

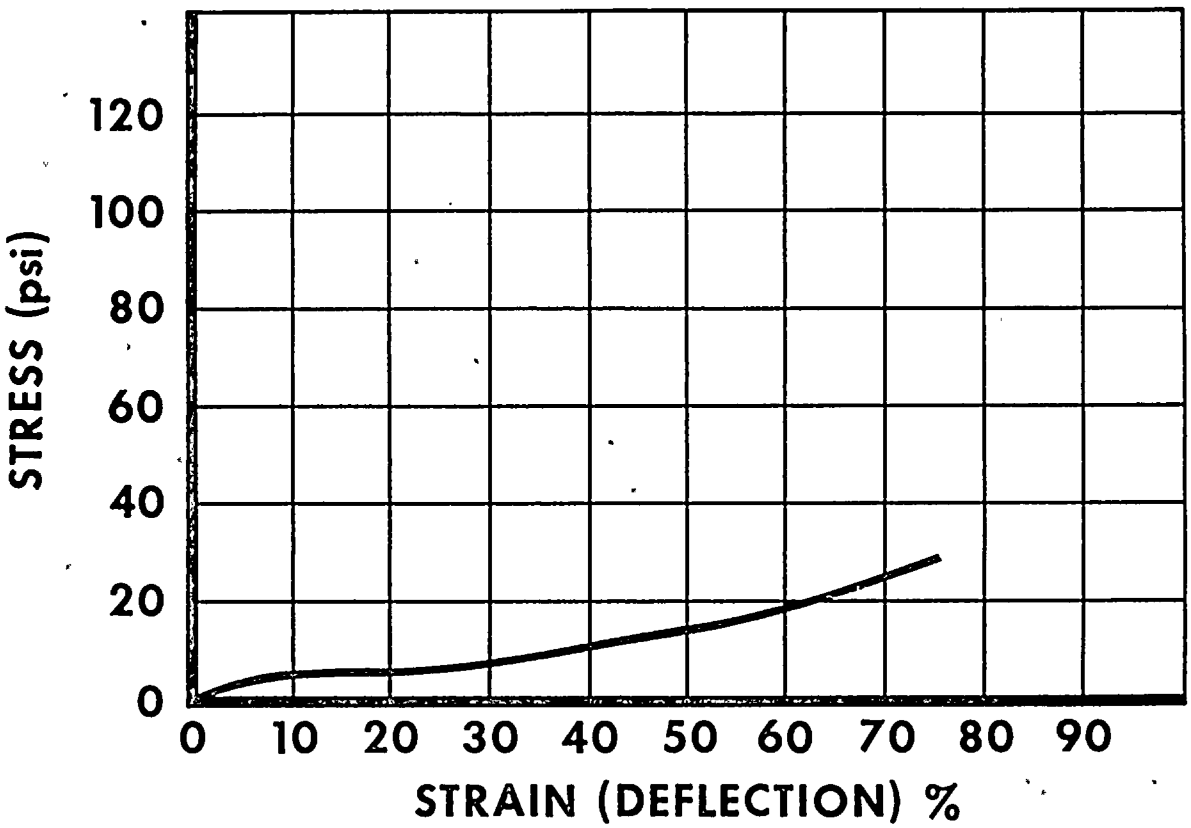


FIG. 361.20-2
COMPREHENSIVE STRESS-STRAIN
FOR NUFOAM TYPE I
NINE MILE POINT NUCLEAR STATION UNIT 2
NIAGARA MOHAWK POWER CORP.

Q361.21 As stated earlier vermiculite concrete has been used between the foundation slabs and rock. Describe the mix used and provide the laboratory tests on which you base the design of the slot backfill. Include the data for both the foundation and wall slot backfill.

RESPONSE A vermiculite concrete mix in the proportion of 1 to 8 by volume of cement to vermiculite was used as backfill around the foundation mats and/or slabs. The material will absorb the rock displacements predicted with minimal load transfer.

The material requirements (per cubic yard) for 1:8 mix per cubic yard are;

Cement:	329 pounds
Vermiculite:	28 cubic feet
Water:	84-91 gallons

The vermiculite used in the mix is manufactured by W.R. Grace and Company as "Zonolite Stabilized Concrete Aggregate Number 4." All the vermiculite used in the mix had a dry unit weight of between 6 and 10 pounds per cubic foot.

Trial mixes were prepared, and unconfined compression and one-dimensional compression tests were done before selection of the final mix to be used.

One-dimensional compression tests were used in predicting lateral loads transferred to structures as a result of rock movements. A series of one-dimensional confined compression tests performed on 13 samples taken from different placement sections at the site is presented in Figure 361.21-1 to illustrate the compressibility properties of this material.

The data for the loose vermiculite backfill between the building walls and rock are presented in the response to Question 361.20.

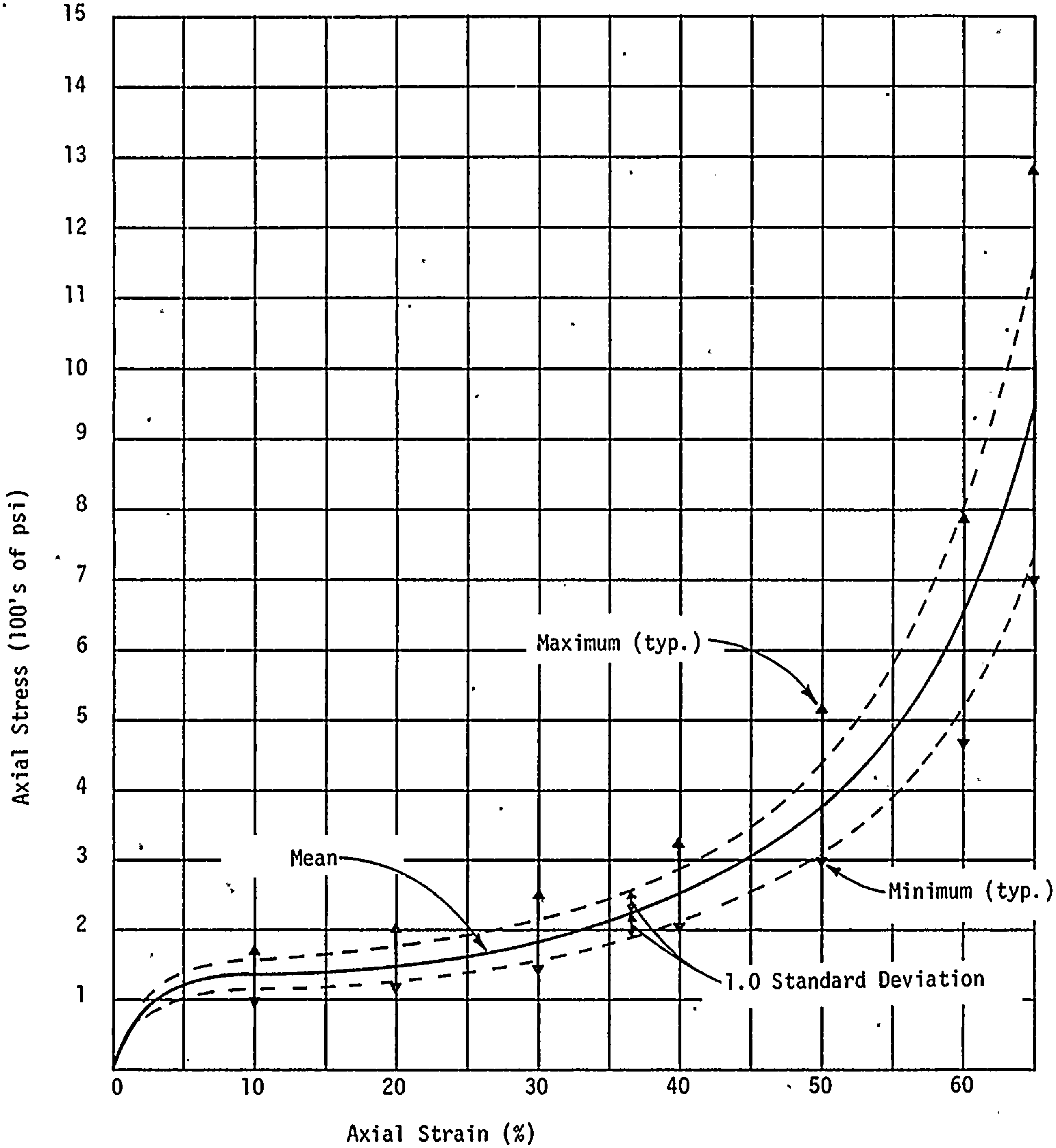


FIGURE 361.21-1

VERMICULITE CONCRETE COMPOSITE CURVE
CONFINED COMPRESSION TESTS
NINE MILE POINT STATION - UNIT 2
NIAGARA MOHAWK POWER CORPORATION

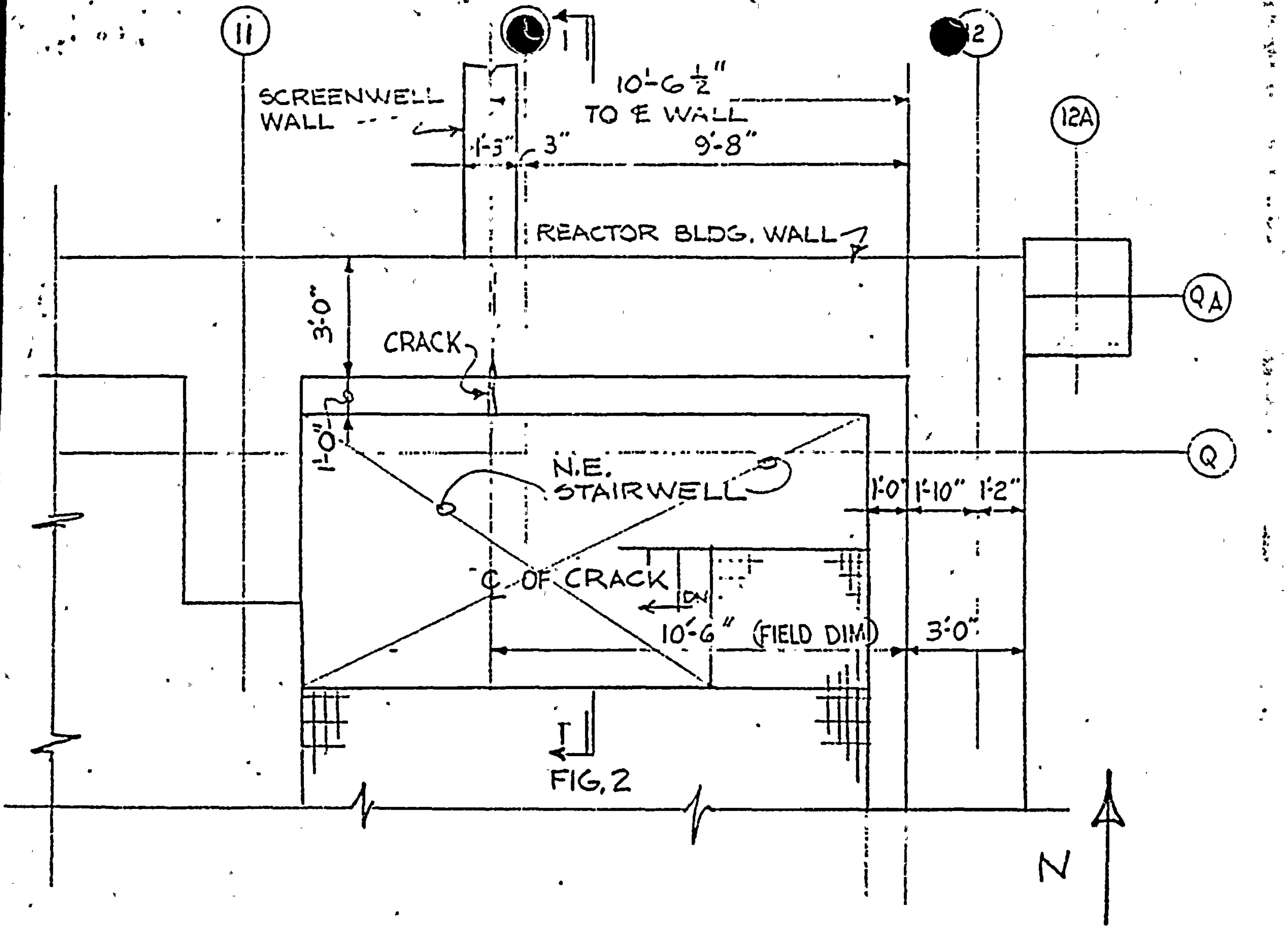
Q361.22 During the meeting on April 24, 1979, Niagara Mohawk indicated that a crack in the northeast corner of the secondary containment was suspected to have been caused by high bedrock stresses. It was further stated that monitoring of the wall was currently underway. Describe the crack and discuss the reasons why you attribute the cause of the crack to rock squeeze. Describe the monitoring program and provide the results obtained to date. It was learned at the meeting that a space at least three feet wide had been constructed between the rock and the structure's wall. Is this indicative of rock movement on the order of several feet into the slot? Discuss.

RESPONSE The location and dimensions of the crack were originally documented in March 1977 (see Figures 1 and 2). It is located in the exterior north wall of the Nine Mile Point Unit 1 Reactor Building approximately 10.5 feet to the west of the east wall. At its maximum, it is one-eighth of an inch wide at a point approximately eighteen inches below the one foot ledge at Elevation 237 feet. At a point six feet below the ledge, it is approximately 1/32 inches wide. The crack extends down from the ledge at Elevation 237 feet approximately eight feet and extends above the ledge about the same distance. The upper and lower ends of the crack disappear in a random hairline pattern.

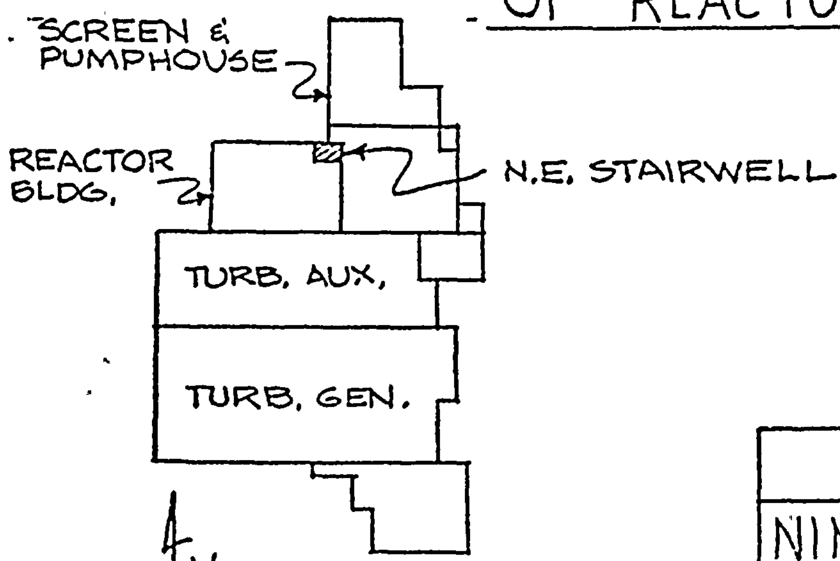
The cause of the crack has not been determined. As shown in Figures 1 and 2 the west wall of the Screenwell Building is in contact with (but not attached to) the north wall of the Reactor Building. With this structural configuration, there are several ways that the west wall of the Screenwell Building could contribute to the cause of the crack. Cracking of concrete can be attributed to relatively low temperature changes causing elongation in the intersecting Screenwell Building wall during original construction. From the attached figures, it can also be seen that small movements of the rocks supporting the Screenwell Building wall could produce sufficient load to cause the cracking in the Reactor Building wall.

When the crack was originally found in March 1977, a measurement was taken perpendicular to the wall to a reference point in the stairwell. This measurement was checked in May 1979 and negligible change was recorded. The present monitoring system consists of two parallel bars anchored side-by-side, one to each side of the crack line at elevations 236 feet and 232 feet (see Figure 3). This system will detect horizontal movement of the crack (widening) by separation of the two parallel bars anchored side-by-side. Vertical elongation of the crack is being monitored by means of measurement with a tape. Measurements are taken semiannually.

The three feet wide space between the rock and the substructure walls is not indicative of rock movement on the order of several feet. Since the potential for minor rock movements was recognized in the original structural design of the plant, the concrete walls were not poured directly against vertical rock faces. However, the reason for a three foot wide slot was to allow space for formwork bracing and working platforms during construction. This slot was later backfilled with non-compacted, free draining, graded material.

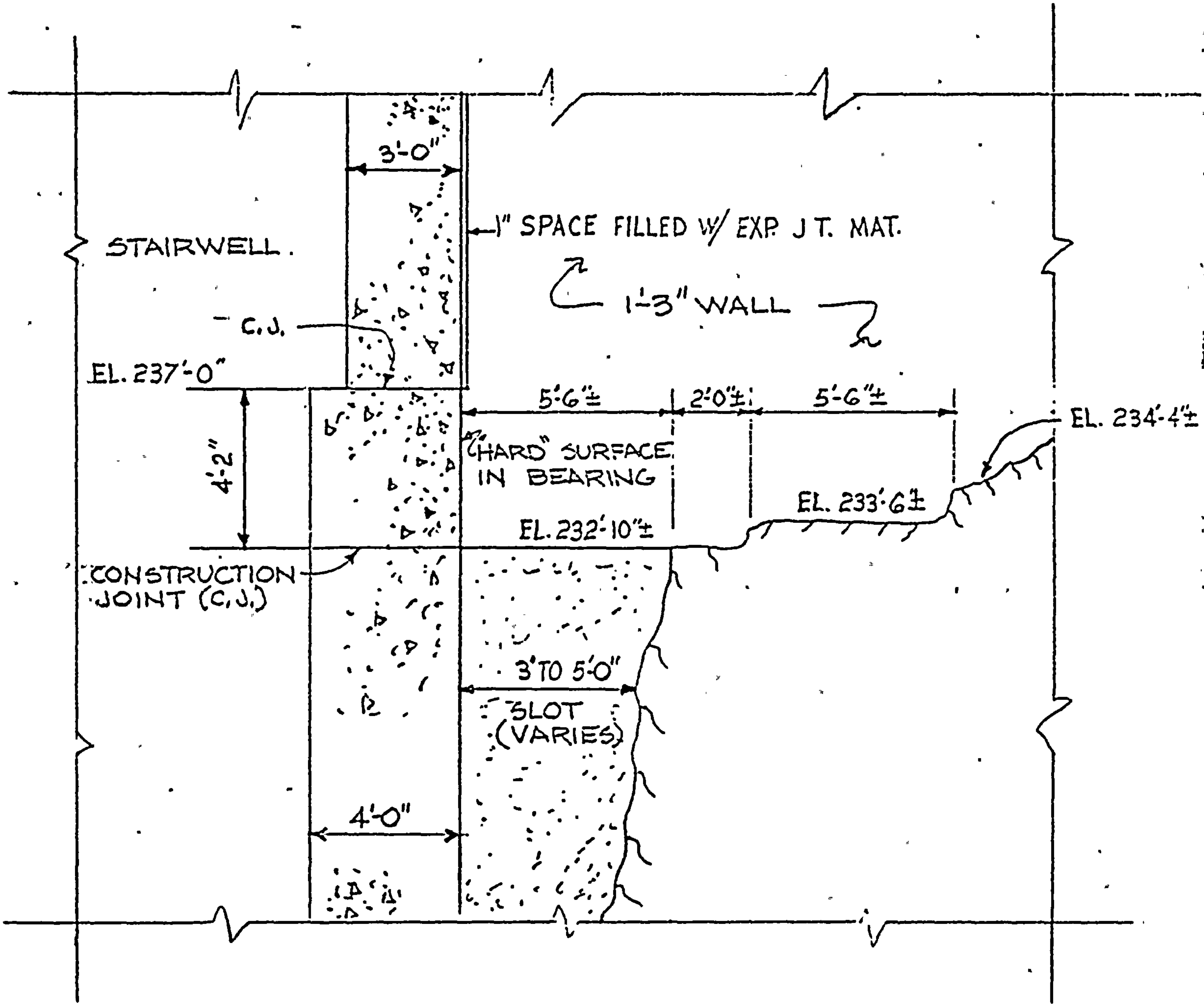


FLOOR-PLAN EL. 237'-0
OF REACTOR BUILDING



KEY PLAN

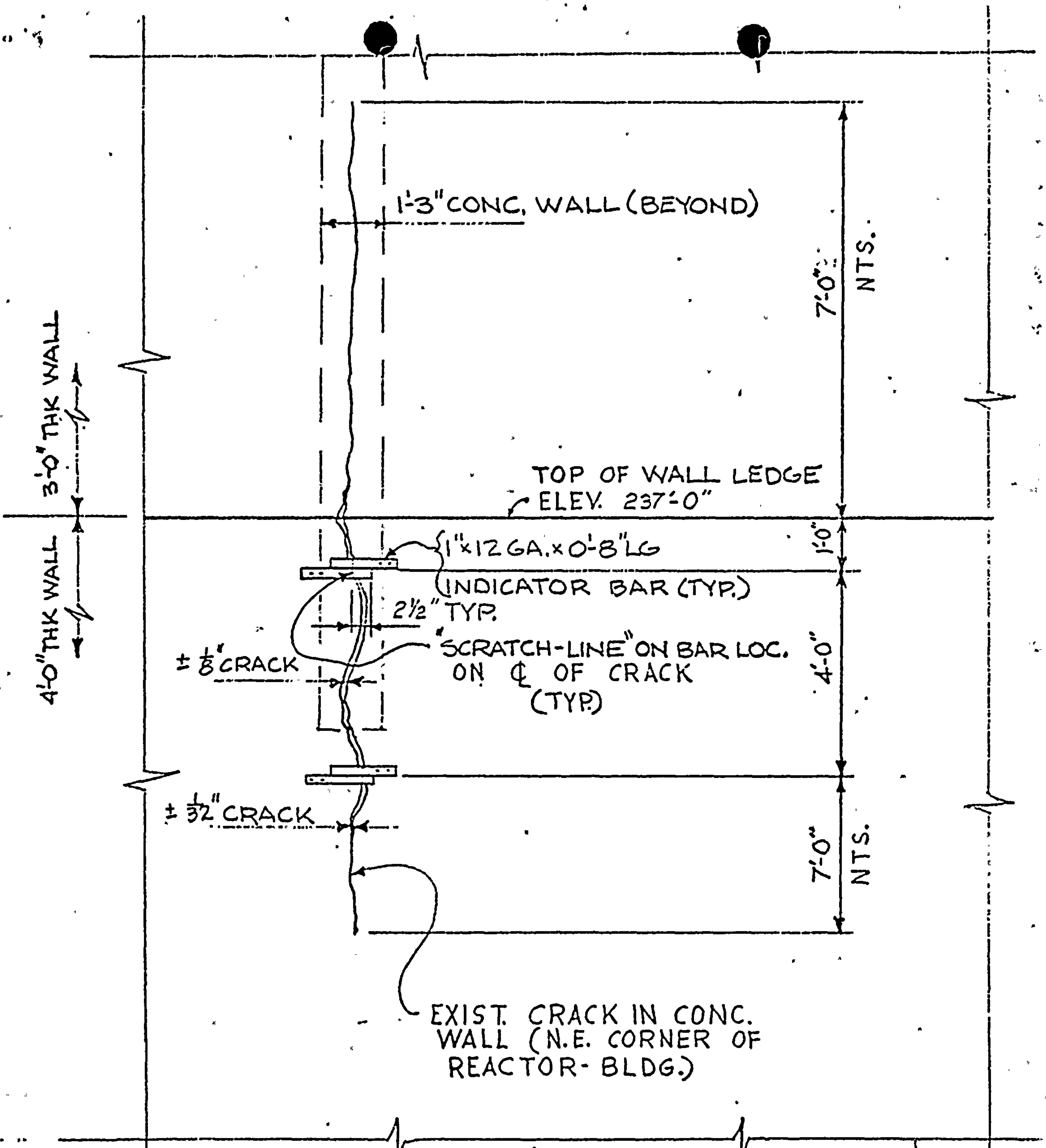
NIAGARA MOHAWK P.C.
NINE MILE PT. NUCLEAR STA #1 REACTOR BLDG. WALL CRACK
[illegible] DATE 1-30-8? SCALE [illegible]
FIGURE #1



SEC. 1-1 LOOKING WEST

- REINFORCING NOT SHOWN FOR CLARITY.

NIAGARA MOHAWK P.C.
NINE MILE PT. NUCLEAR STA. #1 REACTOR BLDG WALL CRACK
FIGURE #2



ELEVATION - LOOKING NORTH

NIAGARA MOHAWK P. C.
NINE MILE PT. NUCLEAR STA. # 1 REACTOR BLDG. WALL CRACK
DES. DR. [illegible] DATE [illegible] SCALE [illegible]
FIGURE # 3

Q361.23 Describe the standpipe installations for measuring groundwater pressure.

RESPONSE Procedures utilized for installing standpipes in the area of the Cooling Tower Fault are provided in Appendix III-J (Niagara Mohawk Power Corporation, Geologic Investigation, Volume III, Rock Stresses, 1978). As-built construction details are illustrated on Plate III-J-1 and Plate III-J-2 of Appendix III-J (NMPC, Geologic Investigation, Volume III, Rock Stresses, 1978). Readings are no longer taken in these standpipes.

In addition, four standpipe type piezometers were installed to measure the groundwater levels in the vicinity of the reactor excavation. The piezometers were installed during April 1979. Water level measurements have been taken at regular intervals (mostly weekly or bi-weekly) since installation. A typical piezometer installation is shown in Figure 361.23-1.

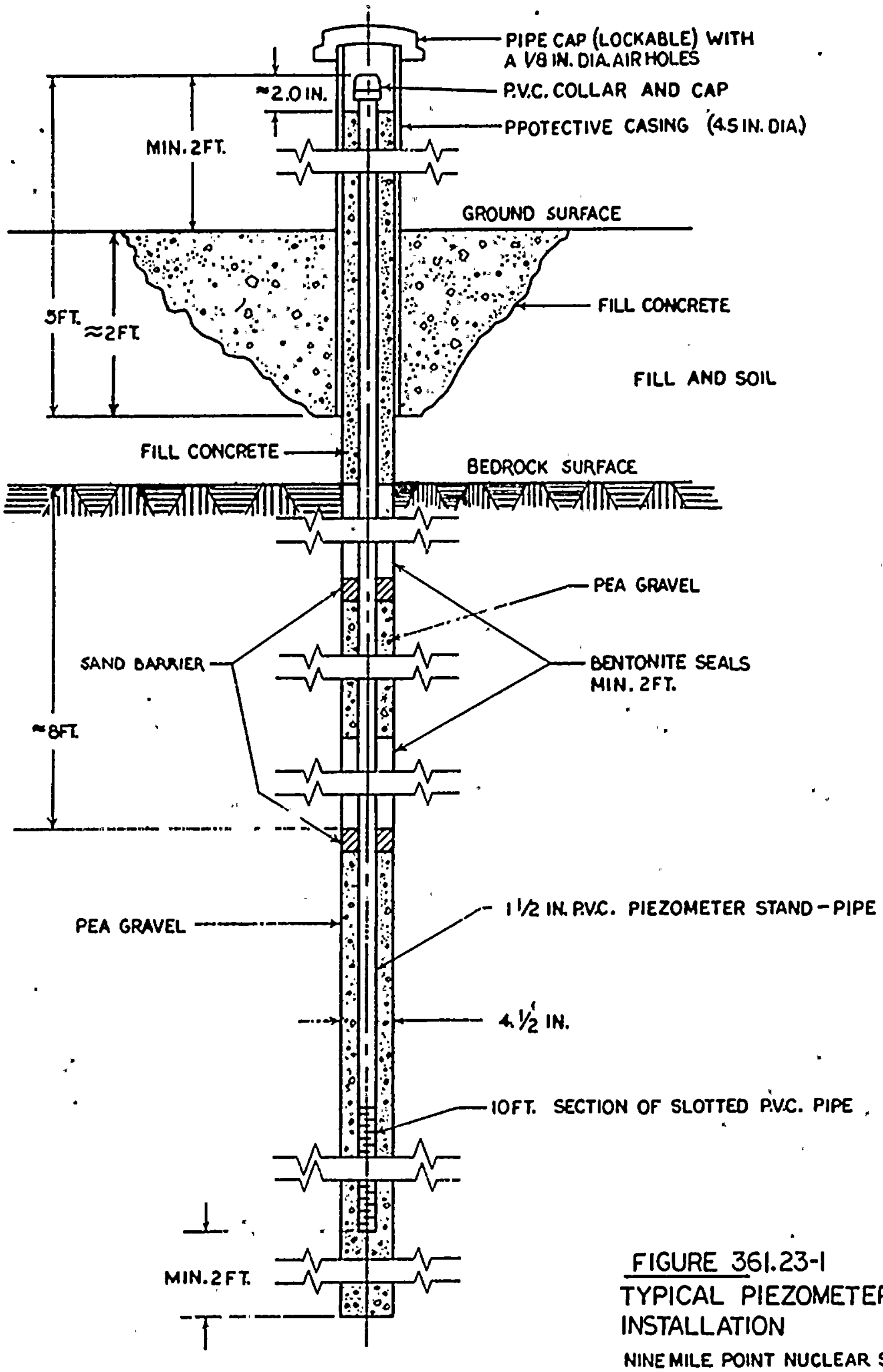


FIGURE 361.23-1
TYPICAL PIEZOMETER INSTALLATION

NINE MILE POINT NUCLEAR STATION
UNIT 2
NIAGARA MOHOWK POWER CORPORATION

Q361.25 What is the consequence of the possible fact that the most recent displacement along the Heater Bay fault and the other thrust faults in the vicinity are of the same age and origin as those along the Cooling Tower buckle?

RESPONSE If investigations of the Heater Bay fault indicate that recent displacements are of the same age as those along the Cooling Tower Fault, the amount and rate of such displacements will have to be investigated. Additionally, the affect of any such displacements on the Unit 2 structures will have to be evaluated.