May 1, 1981



Director of Nuclear Reactor Regulation
Attention: Mr. Dennis M. Crutchfield, Chief
Operating Reactors Branch No. 5
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: R. E. Ginna Nuclear Power Plant
Docket No. 50-244
SEP Topics III-3.A, II-3.A, II-3.B, II-3.B.1
& II-3.C

References: 1) NRC correspondence dated March 24, 1981
(Topic III-3.A)
2) NRC correspondence dated April 10, 1981
(Topics II-3.A, II-3.B, II-3.B.1 & II-3.C)

Enclosed are the Rochester Gas & Electric comments regarding the NRC assessment on the above referenced SEP Topics. Since the SEP Topics being addressed are very interrelated, and the material in reference 1 cannot be adequately addressed without prior resolution of reference 2, both NRC letters are being addressed together.

For informational purposes, and to help clarify the various pieces of material addressing the SEP Topics, a list of previous correspondence and telephone memoranda is also enclosed.

Very truly yours,

John E. Maier

J. E. Maier

JEM:mkv
Enclosure

A035
S
1/1

8105060252

Exhibit 5

28560
2

Enclosure 1

Response to NRC Correspondence on SEP Topics
III-3.A (3/24/81) and II-3.A, II-3.B, II-3.B.1 & II-3.C (4/10/81)

1. SEP Topic III-3.A - "Effects of High Water Level on Structures" (Reference 1)

   a. Page 2 - Section V.1.a

      This section is unclear as written. The first section should state: "The plant is protected from surges and wind-driven waves by a revetment with a top elevation of 261.0 feet msl." No flooding due to wave action is expected.

   b. Page 2 - Section V.1.b

      This paragraph should be clarified, consistent with the source of this information RGE letter of November 14, 1979), as follows (starting with the third sentence): "The licensee stated in a 11/14/79 letter from L. D. White, Jr. to Dennis L. Ziemann (NRC), that the maximum stillwater level for Lake Ontario considered originally in the design of Ginna was 250.78 feet. This was later revised on May 10, 1973 to a level of 253.28 feet.

   c. Page 3 - Section V.1.c

      As indicated in our correspondence of January 28, 1981, we do not believe that the PMF on Deer Creek is correct. We are currently undertaking a study to ascertain a Deer Creek PMF. Results of this study will be forwarded to the NRC in late July or early August.

      No evaluation of Category I structures (if required) should be done until the results of the ongoing study are evaluated.

   d. Page 3, Section V.2.a

      RG&E does not agree with the assumption of ground water at ground elevation except in the area of the screen house.

Original site studies conducted by Dames & Moore in 1964 - 1965 demonstrate that the assumed design ground water levels were reasonable. The following data is from Site Evaluation Study, Proposed Brookwood Nuclear Power Plant, Ontario, New York, Rochester Gas & Electric Corporation, - Dames & Moore, June 14, 1965.

1. "Borings show that the water table rises to approximately 247 ft. in the general plant area, and that it continues to rise towards the south..."

2. "Contours of the surface of the rock and the ground water table are presented on Plate IIB-3. These contours are intended to show the general position of the rock surface and the water table." A reproduction of Plate IIB-3 with the plant location superimposed on it is enclosed.

3. "The rock encountered in the borings appears to be almost impermeable. Most of the movement of groundwater, however, takes place in the more permeable soils overlying the rock. The soils have a low permeability (ranging, in general, from roughly $10^{-3}$ to $10^{-6}$ centimeters per second)..."

Based on these data, the assumption, for design purposes, of ground water level at elevation 250 msl is reasonable.

e. page 4, Section V.2.c

The information presented in item d above indicates that there is no need for any further evaluation in the Auxiliary Building as requested.

In regards to necessary analysis of groundwater effects on the Containment, we believe the chance of any groundwater accumulation around the Containment is extremely remote. Information regarding the dewatering system around the containment was given to Drew Persinko in a telephone call with Robert Mecredy on March 10, 1981 and drawings of the system were transmitted to the NRC on March 12, 1981 (White to Crutchfield).

While we acknowledge that the existing dewatering system is not a "high level" safety system, it has in fact been operating satisfactorily for approximately ten years. Detailed inspections of the area are not made on a scheduled basis, but personnel are in the area on an "occasional basis" and there has never been and indication of substantial water accumulation or any system malfunction. The operating experience of the system coupled with the original site data indicating the rock is "almost impermeable" and the overlying soils "have a low permeability" verify to us that the existing facility is sufficient to preclude a substantial accumulation of groundwater around the containment.

However, in response to NRC requests, we have attempted to provide a rough analysis of worst case effects of groundwater pressure on the containment. This information was verbally discussed during a telephone call on March 16, 1981 between George Wrobel of RG&E and Drew Persinko of the NRC. As indicated in the telephone call, we believe the containment has a large factor of safety against goundwater effects.

A spot check analysis was done to determine the membrane stresses and external collapse pressure: first assuming the containment is a cylindrical shell, and, second assuming the containment acts as a circular beam or ring. As can be seen from the attached calculations, a comparison of the collapse pressure to the pressure applied from a 22 foot head of water (times a load factor of 1.7) results in a "safety factor" of about 6 or 11 depending on the type of analysis.

Based on the above, the Containment Building is considered adequate against the effects of high water levels.

2. SEP Topics II-3.A, "Hydrologic Description"; II-3.B, "Flooding Potential and Protection Requirements"; II-3.B.1, "Capability of Operating Plants to Cope With Design Basis Flooding Conditions"; and II-3.C "Safety-Related Water Supply" (Reference 2).

   a. Page 4, Section 3.2 - Deer Creek PMF

      As stated earlier, we take exception to the PMF flow of 37,500 cfs for Deer Creek, and we have initiated a study to verify the NRC value or to justify a new value. The results of the study will be available in late July or early August.

b. Page 7, Section 3.3.0 - Lake Ontario Surge Flooding

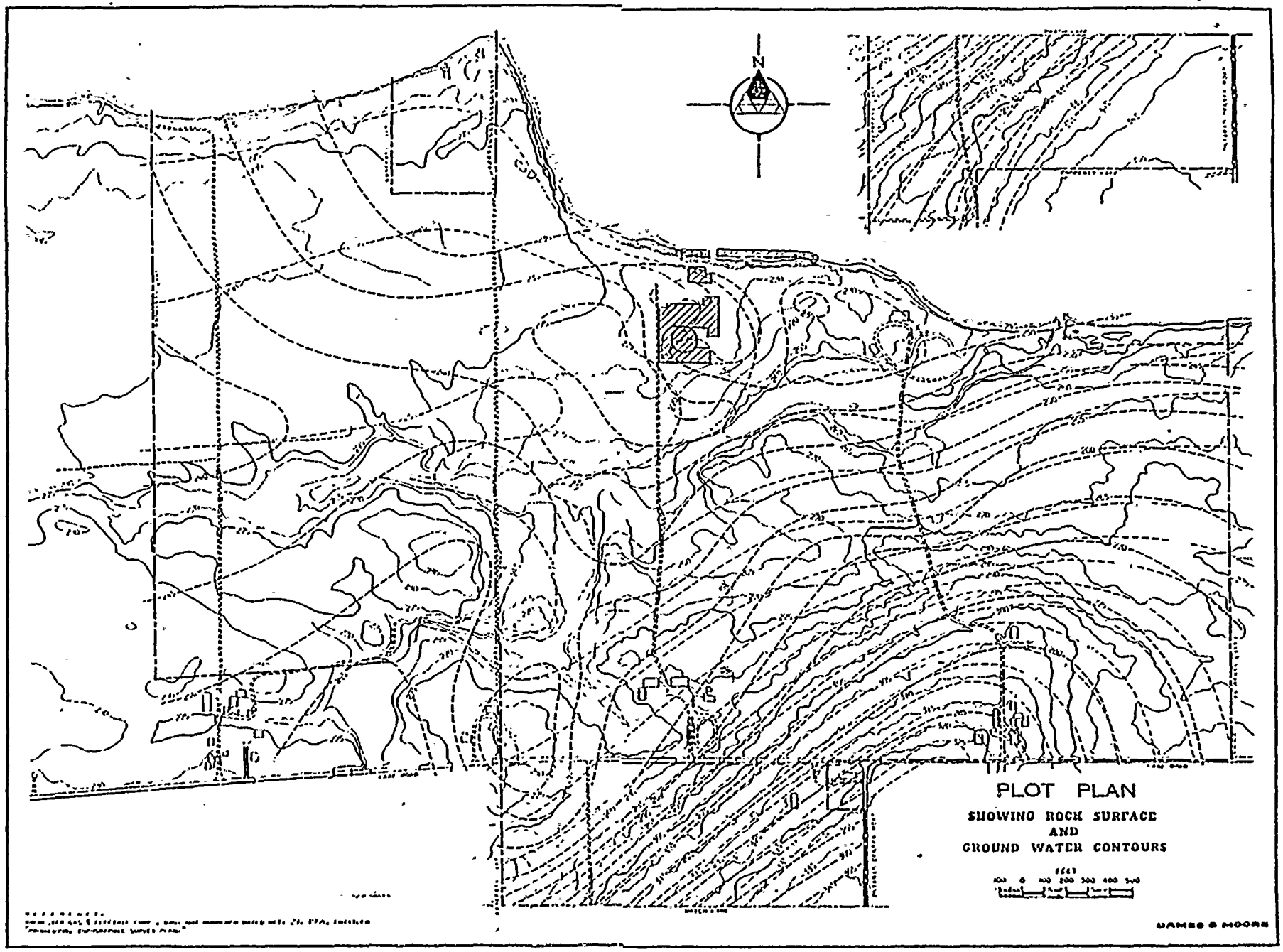
As indicated in our correspondence of January 28, 1981, we have been unable to see any visible evidence of degradation of the breakwall. We have reviewed the NRC September 6 and 7, 1978 site visit minutes, and found no reference to a concern relative to revetment degradation.

RG&E again inspected the west side of the breakwall on April 27, 1981 and was unable to find any indication of significant degradation of the breakwall. It was determined to be structurally sound and comparable to the requirements of the original specifications.

We believe that the appearance of degradation indicated by the NRC may be due to the placement of loose fill on the breakwall following the upgrading that was performed in 1973. This fill performs no function in relation to the requirements of the breakwall, but is simply excess sub-base material that was used to fill natural occurring voids in the joints between individual armor stones.

RG&E is planning to take a full series of photographs of the breakwall about May 5, 1981. Copies of these photos will be sent to the NRC as soon as they are available. If the NRC still has questions in regard to the structural integrity of the breakwall after review of the photographs, we suggest that an NRC representative visit the site so that specific problems can be resolved.

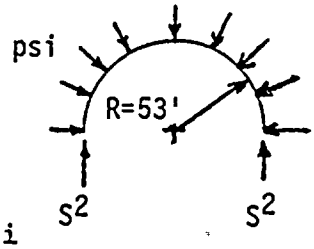
Since we believe there has been no evidence of any degradation of the breakwall since its installation, we see no reason for the development of a technical specification for maintenance of the breakwall at this time.


N
PLOT PLAN
SHOWING ROCK SURFACE
AND
GROUND WATER CONTOURS
FEET
100 0 100 200 300 400 500
DAMES & MOORE

CHECK OF CONTAINMENT CAPABILITY
TO RESIST EXTERNAL WATER PRESSURE

<u>METHOD 1</u> - ASSUME CONTAINMENT IS A CYLINDRICAL SHELL UNDER UNIFORM RADIAL WATER PRESSURE

Design Data:




h = Water level = 22' high
t = Thickness of contianment = 36"
R = Radius of containment = 53'
$f'_c$ = Compression strenght of concrete = 4000 psi
p = Max. water pressure
= 62.4 X 22 = 1370/144 = 9.51 psi
U = Design water pressure = 1.7 X 9.51 = 16.17 psi

1. Determine Membrane Stresses:

$S_1$ = Meridional Stress = $\frac{PR}{2t} = \frac{16.17 \times 53 \times 12}{2 \times 36}$ = 143 psi <4000 psi

$S_2$ = Hoop stress (comp.) = $\frac{PR}{t} = \frac{16.17 \times 53 \times 12}{36}$ = 286 psi <4000 psi

2. Determine External Collapse Pressure:

$$p' = \frac{t}{R} \quad \frac{S_y}{1 + 4 \frac{S_y}{E_c} \frac{(R)^2}{(t)^2}}$$

$S_y = f'_c$ = compressive strength of concrete, 4000 psi

$E_c$ = Modulus of elasticity of concrete
= 57.000 $f'_c = 3.6 \times 10^6$ psi

$$p' = \frac{36}{53 \times 12} \quad \frac{4000}{1 + 4 \times \frac{4000}{3.6 \times 10^6} \left(\frac{53 \times 12}{36}\right)^2}$$

= 94.8 psi

$$\text{Safety Factor} = \frac{94.8}{16.17} = 5.86$$

<u>METHOD 2</u> - ASSUME CONTAINMENT IS A UNIFORM CIRCULAR RING UNDER UNIFORM RADIAL PRESSURE

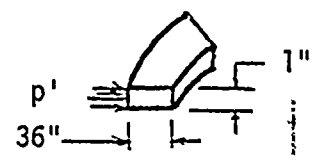
1. Determine collapse pressure of ring, p'

$$p' = \frac{3EI}{R^3}$$

$$I = \frac{1}{12} \times 1'' \times (36)^3 = 3888 \text{ in.}^4$$

$E_c$ = 57.000 $f'_c = 3.6 \times 10^6$ psi

R = Mean radius of ring = 51.5 ft.




$$p' = \frac{3 \times 3.6 \times 10^6 \times 3888}{(51.5 \times 12)^3}$$

= 178 lb. per inch per linear inch = 178 psi

$$\text{Safety Factor} = \frac{178}{16.17} = 11.00$$


<u>References</u>: 1 - "Formulas for stresses and strains". Page 293 & 343 by Roark.

2 - ACI Standard 349-76.

Enclosure 2

Correspondence Listing for SEP Topics
III-3.A, II-3.A, II-3.B, II-3.B.1 & II-C

1. November 2, 1978 - NRC Minutes of September 6 & 7, 1978 Site Visit H. M. Fontecilla to D. G. Eisenhut.

2. November 14, 1979 - RG&E letter on SEP Topics III-2, III-3.A, and III-7.B, L. D. White, Jr. to Dennis L. Ziemann.

3. December 12, 1980 - NRC correspondence on SEP Topics II-3.A, II-3.B and II-3.C, D. M. Crutchfield to L. D. White, Jr.

4. December 30, 1980 - Telephone Memo on the condition of break wall, Drew Persinko and Ted Johnson of NRC with George Wrobel of RG&E.

5. January 28, 1981 - RG&E response to item 2, L. D. White, Jr. to D. M. Crutchfield.

6. March 4, 1981 - Telephone Memo concerning the design basis groundwater level, Drew Persinko of NRC with Gary Goetz and Paul Wilkens of RG&E.

7. March 10, 1981 - Telephone Memo requesting additional information on screenhouse and containment, Drew Persinko of NRC with Robert Mecredy of RG&E.

8. March 12, 1981 - RG&E submittal of additional information on Screen House as requeted by Drew Persinko, L. D. White, Jr. to D. M. Crutchfield.

9. March 16, 1981 - Telephone Memo concerning containment flooding, Drew Persinko of NRC with George Wrobel of RG&E.

10. March 24, 1981 - NRC correspondence on SEP Topic III-3.A, D. M. Crutchfield to L. D. White, Jr.

11. April 10, 1981 - NRC correspondence on SEP Topics II-3.A, II-3.B, II-3.B.1 and II-3.C, D. M. Crutchfield to L. D. White, Jr.

NCBED-DC Re: R.E. GINNA Nuclear Generating Plant 2 November 1981

Project Officer
United States Nuclear
Regulatory Commission
Washington, DC 20555

Dear Sir:

Enclosed is a Memorandum for Record (MFR) concerning the stone shore revetment at the R.E. GINNA Nuclear Generating Plant, town of Ontario, NY. The MFR was prepared by Buffalo District, Corps of Engineers in response to Coastal Engineering Research Center Interagency Agreement No. NRC-03-81-110 Proposed Work Directive 1.

The MFR expresses our preliminary opinions and findings and will be followed by a more detailed letter report in accordance with the Statement of Work for CERC-GINNA Nuclear Power Plant - Document No. 50-244. If you have any questions concerning the content of the MFR, please contact either Messrs. Richard Gorecki (FTS 473-2230) or Jonathan Kolber (FTS 473-2169) of my Engineering staff.

Sincerely,

1 Incl
as stated

GEORGE P. JOHNSON
Colonel, Corps of Engineers
District Engineer

CF:
Mr. N. Parker
U.S. Army Coastal Engineer
Research Center
Kingman Building
Fort Belvoir, VA 22060

NRC Contracting Officer, DC
US Nuclear Regulatory Commission
Washington, DC 20555

J.P. Knight, DE
US Nuclear Regulatory Commission
Washington, DC 20555

Director, Division of Engineering
ATTN: C. Poslusny
US Nuclear Regulatory Commission
Washington, DC 20555

Mr. B. L. Grenier
US Nuclear Regulatory Commission
Washington, DC 20555

NCBED-D
NCBED-DC
NCBED-DF
NCDED-C

Exhibit 6

NCBED-DC 2 November 1981

MEMORANDUM FOR RECORD

SUBJECT: Trip Report Concerning Stone Revetment at R.E. GINNA Nuclear Generating Plant

1. In response to several FONECONS between Mr. Charles Johnson of North Central Division, Corps of Engineers, and Mr. Richard Gorecki of Buffalo District, Corps of Engineers, the District Corps office was requested to make a site inspection of a stone shore revetment at the R.E. GINNA Nuclear Generating Plant. The purpose of the site inspection was to make visual observations of the revetment in order to provide a technical opinion of the adequacy of the stone shore revetment which protects the main building complex from wave attack by Lake Ontario. The R.E. GINNA Nuclear Generating Plant is located in the town of Ontario, NY on the south shore of Lake Ontario, approximately 20 miles east of the city of Rochester, NY. The licensee of the R.E. GINNA Nuclear Generating Plant is the Rochester Gas and Electric Corporation.

2. The stone revetment was constructed in two reaches; a 420-foot long west reach and a 400-foot long east reach. The east and west stone revetment reaches are separated by the generating plant's discharge canal (see Incl A and B). The revetment was initially constructed with two layers of 5-ton minimum armor stone laid up on a 1-1/2 sideslope to a minimum elevation of 257.0 (msl). Due to high lake levels experienced on Lake Ontario during the early 1970's, the crest elevation of the revetment was raised to a minimum of 261.0 (msl) by placement of a cap stone along the top of the revetment (see Incl C and D for typical cross sections of the revetment).

3. The site inspection request was in response to a Nuclear Regulatory Commission (NRC) site visit which has resulted in questions related to the adequacy of the existing stone revetment along the lake shore. The NRC site visit indicated that portions of the revetment on the west side of the discharge canal appeared to have been degraded in terms of the number and sizes of stones at several locations. Therefore, the Corps of Engineers was requested to provide a technical opinion of the adequacy of the existing revetment and to suggest ways that the revetment may be repaired if not found adequate.

4. On 21 October 1981, Mr. Richard Gorecki of the Buffalo District Coastal Engineering Section and Mr. Jon Kolber of the Buffalo District Geotechnical Section met Mr. Gary Goetz of Rochester Gas and Electric Corporation at the R.E. GINNA plant site. Mr. Goetz, the project engineer for the revetment construction project, guided the Corps representatives on an inspection of both the east and west reaches of the stone revetment.

Incl. 1

5. The inspection team walked the length of the west revetment reach and back again without noting any significant displacement of stones in the sideslope of the structure nor any unusual depressions in the structure sideslope or crest. The fence which is adjacent to the back side of the revetment appeared to be plumb and have an alignment that runs parallel to the revetment thereby indicating that there has been no appreciable movement of the fill behind or under the revetment. The armor stones on the sideslope of the revetment are covered and obscured by stone in the 4- to 9-inch range in several areas. Therefore, it was impossible to positively determine the existence of two layers of armor stones as required by the final design at these locations, particularly at the west end of the revetment where a larger concentration of the estimated 4- to 9-inch stone had been placed. Photographs were taken of the west revetment reach to depict the condition of the wall and are attached as Inclosure E.

6. The inspection team also walked the length of the east revetment reach and back, again without noting any significant displacements of stones in the sideslope of the structure nor any depressions in the structure sideslope or crest. There were no apparent depressions or slippages evident in the backslope of the compacted earth dike, thereby indicating that there has been no appreciable movement of the fill behind or under the revetment. However, a large void, large enough for a person to crawl into, was observed roughly 100 feet east of the west end of the east revetment reach. This void extends for about 25 to 30 feet directly under the capstones which bridge the void with little rock to rock contact over the void. The two layers of 5-ton minimum armor stones on the sideslope of the east revetment are also covered and obscured by stone in the 4- to 9-inch range in a few areas. The main concentration of the 4- to 9-inch stone on the east revetment was placed in the area adjacent to the discharge canal. It was again impossible to determine the existence of two layers of armor stones in these areas where the 4- to 9-inch stones exist. Photographs were taken of the east revetment reach to depict the condition of the wall and are attached as Inclosure F.

7. The east and west ends of the revetment reaches terminate and tie into the natural high bluff away from the main (reactor) building complex. Although erosion had occurred in these bluff areas, probably during the high lake levels of the early 1970's, we observed no evidence of current active bank erosion since the bluff faces are covered with vegetative growth probably several years old. There is the possibility that the revetment ends can be flanked by erosion during high lake levels or during an extreme infrequent storm, thereby causing damage to the revetment. Since the main building complex is far enough away from the ends of the revetment, there would be no immediate danger to the facility even if the revetment is flanked and damaged.

8. Based on our visual observations, we see no evidence where the existing revetment is in any immediate or foreseeable future danger of failing. It is our opinion that the revetments are structurally sound and stable,

and, except for possibly the area in the east revetment reach where the large void exists, there is no evidence of any structure stability problem. A determination could not be made at the time of the inspection as to whether two layers of armor stones exist in the areas where the smaller stones had been placed. According to Mr. Gary Goetz, this small stone had been placed to form a roadway in the lee of the structure during the early 1970's for equipment used to place the cap stone. Upon completion of the cap stone placement, the roadway stone was dumped over the face of the revetment to fill the voids between armor stones, thereby making a more aesthetically looking revetment.

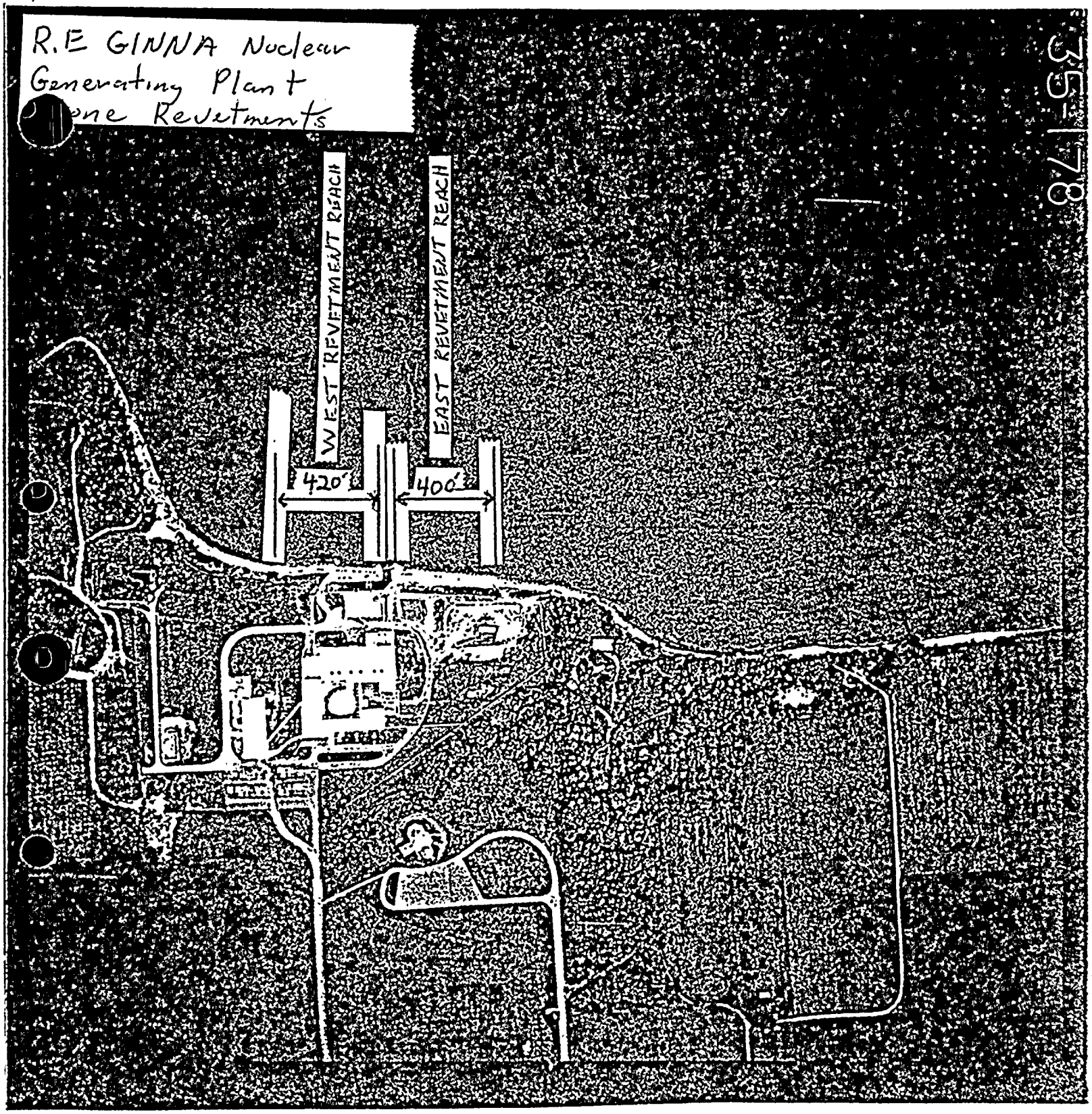
9. These preliminary findings have been already discussed between Mr. Gary Staley of NRC and Mr. Richard Gorecki of Buffalo District, Corps of Engineers. In order to verify that the revetment is of adequate design in the areas where the small stone had been placed, it will be necessary to have the licensee remove the small stone. After the small stones are removed, another site visit will be required to visually inspect the revetment and insure that the armor stones are in place. This second site visit has been tentatively set by Mr. Gary Staley for 13 November 1981.

10. The area in the vicinity of the large void in the east revetment reach will be reexamined during the 13 November 1981 inspection. Our assessment of structural adequacy and any recommendations will be presented in the future letter report.

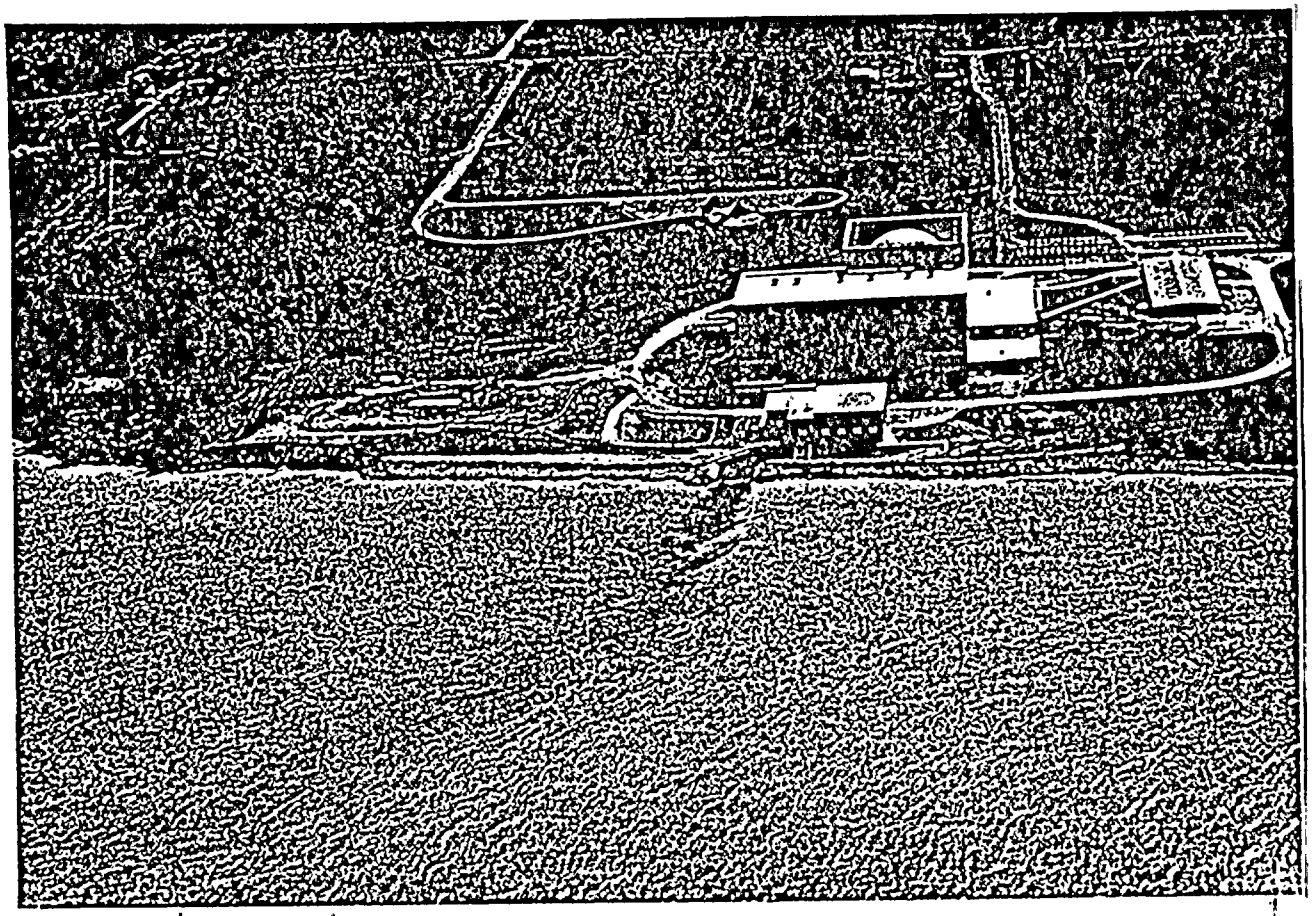
6 Incl
as

Richard J. Gorecki
RICHARD J. GORECKI
Buffalo District,
Corps of Engineers
Coastal Engineering Section

Jonathan E. Kolber
JONATHAN E. KOLBER, P.E.
Buffalo District,
Corps of Engineers
Geotechnical Section

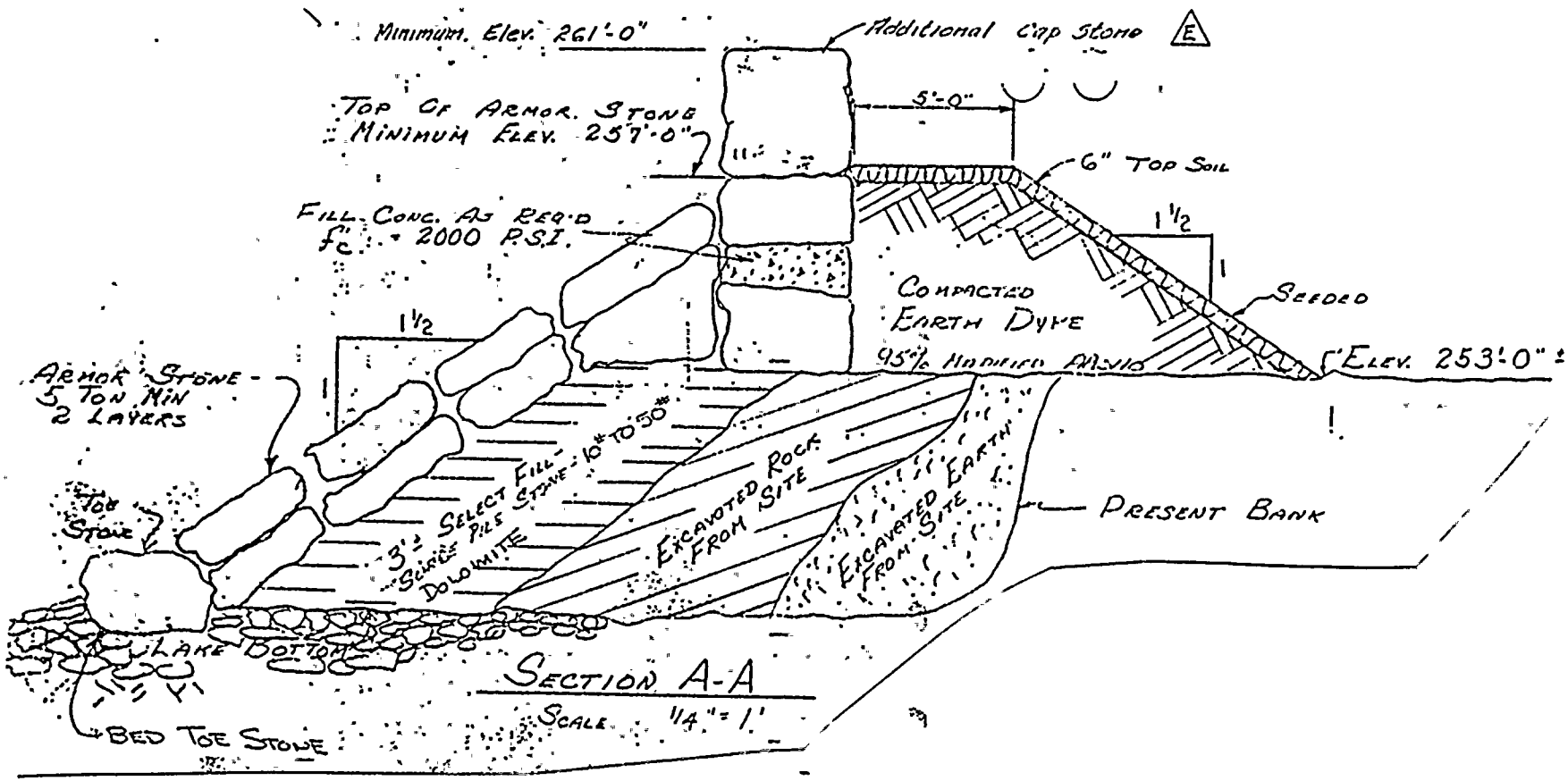

R.E GINNA Nuclear
Generating Plant
one Revetments
WEST REVETMENT REACH
EAST REVETMENT REACH
420'
400'

Inclosure A



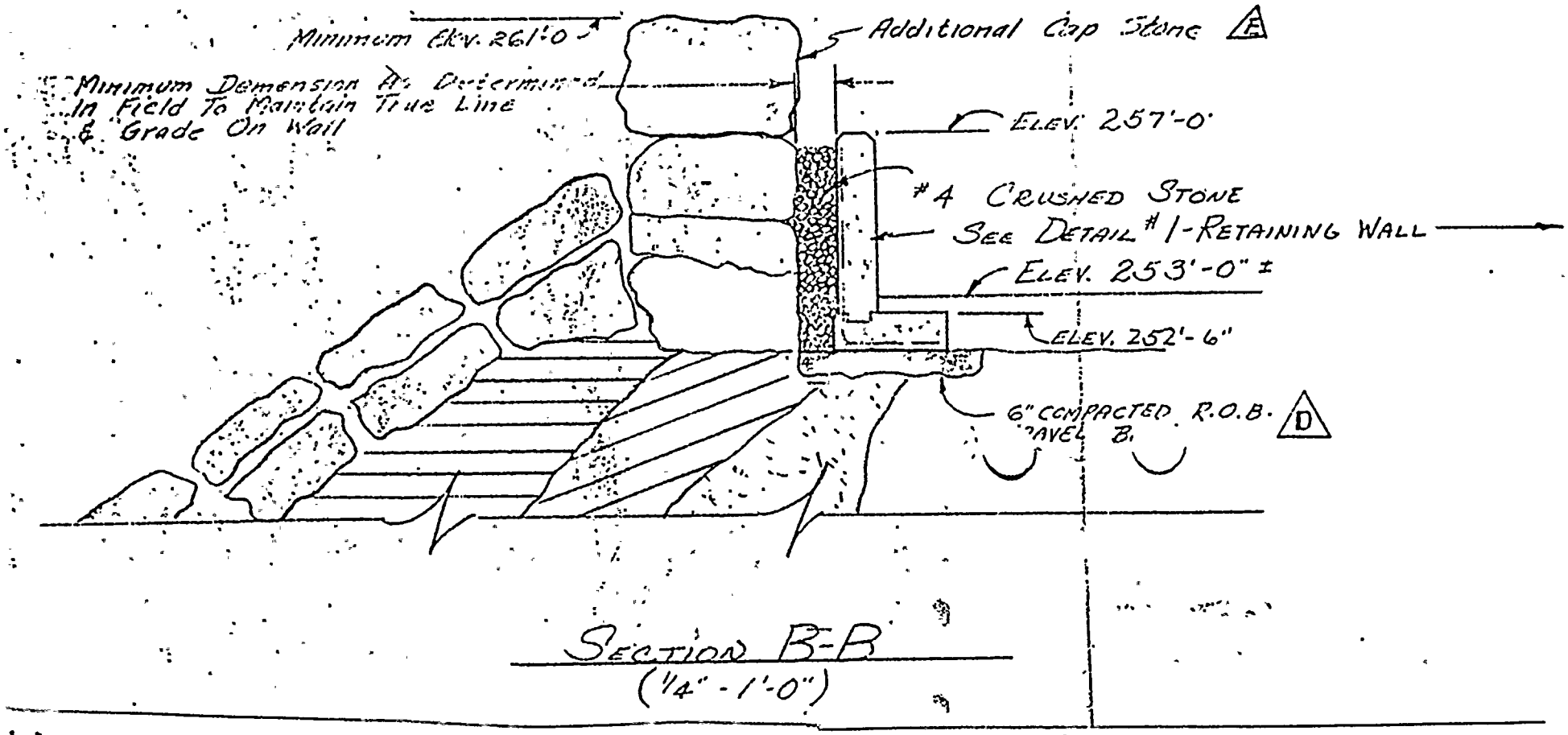
R.E. GINNA Nuclear Generating Plant

Inclosure B




TYPICAL SECTION - EAST REVETMENT REACH

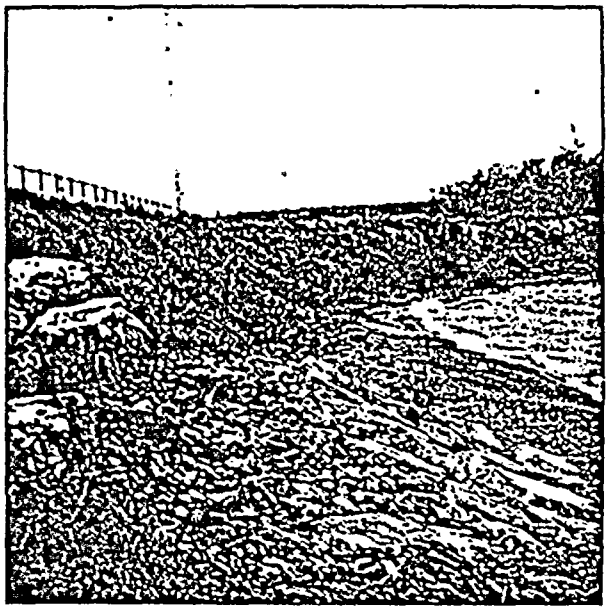
Inclosure C



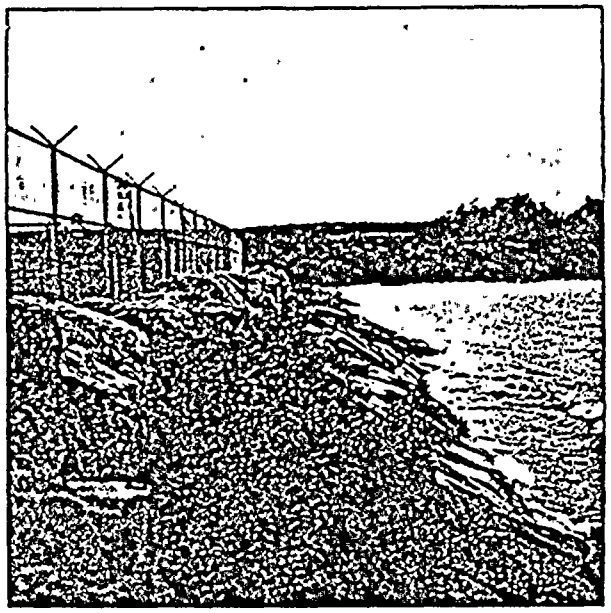

TYPICAL SECTION — WEST REVETMENT REACH

Inclosure D

D



WEST END OF WEST REVETMENT - LOOKING WEST. NOTE SMALL STONE FILL OBSCURING TOP ARMOR STONES.



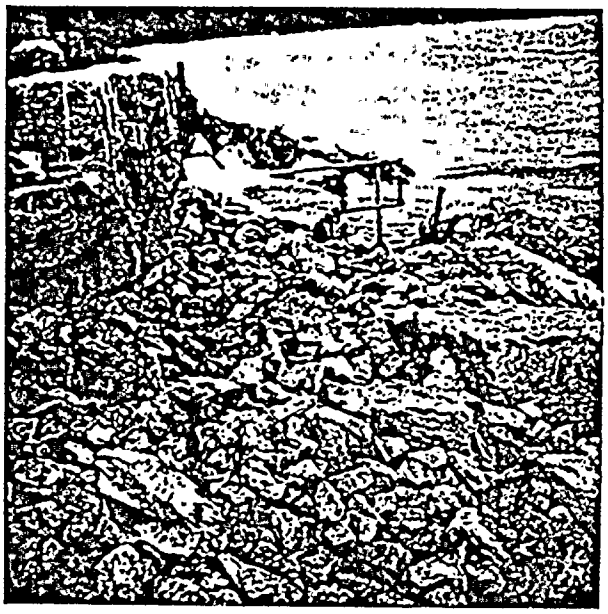
NEAR CENTER OF WEST REVETMENT - LOOKING WEST. NOTE LAID-UP REVETMENT ON RIGHT AND CAP STONES ON LEFT.

E

WEST REVETMENT NEAR EAST END - LOOKING EAST. NOTE SMALL STONE FILL OBSCURING TOP ARMOR STONES. SCREEN HOUSE IS AT RIGHT.

ENCLOSURE E

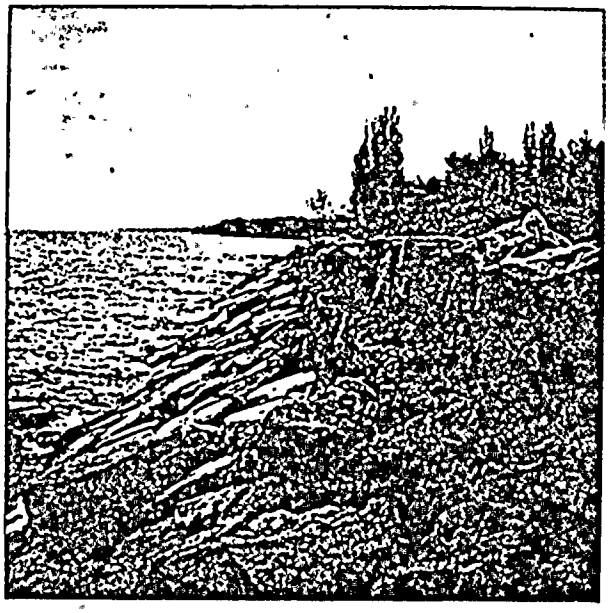
WEST REVETMENT NEAR EAST END. LOOKING EAST. NOTE LAID UP REVETMENT.



EAST REVETMENT @ WEST END. NOTE SMALL STONE FILL OBSCURING ARMOR STONES. CAP STONE @ LEFT. DISCHARGE CANAL VISIBLE IN REAR, LOOKING WEST.

EAST REVETMENT APPROX. 100 L.F. EAST OF WEST END. NOTE LARGE VOID UNDER CAP STONE (THIS VOID EXTENDS A TOTAL LENGTH OF ABOUT 25 L.F. TO RIGHT & LEFT OF PHOTO).

EAST REVETMENT NEAR EAST END - LOOKING EAST. NOTE SMALL STONE FILL OBSCURING TOP ARMOR STONES LAID UP REVETMENT @ LEFT, CAP STONES @ RT.



INCLOSURE F

EAST REVETMENT - LOOKING EAST, NEAR EAST END. NOTE LAID-UP REVETMENT, RO

R.E. GINNA NUCLEAR GENERATING PLANT
GEOTECHNICAL APPENDIX

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## 1. PROJECT DESIGN

The project consists of two revetments, the East (400 L.F. ±) and the West (420 L.F. ±) designed to resist Lake Ontario lake action (Photograph 1). These are multilayer structures with a double layer of 5 ton minimum size armor stone underlain by successive zones of select stone fill (3-foot ± thickness of 10 pound to 50 pound stone), excavated rock fill from site excavation (3-4-foot thickness), and, finally, a zone of soil (Figures 1 and 2). The East Revetment is backed by a compacted earth dike, while the West Revetment is built partly against a concrete retaining wall and partly against a compacted earth dike. A row of toe stone has been placed at the bottom of the revetment slopes. The toe has been excavated into the bottom material. Cap stones have been placed at the revetment crest to increase the maximum elevation of the revetments to Elevation 261.0 msl. The two revetments are split by a discharge canal, proceed to the east and west, and terminate by tying into high bluffs at each end (Photographs 4 and 5).

The original revetment construction was completed about 1965, to Elevation 257.0 msl. In 1973, a row of cap stones was placed to increase the elevation to offset the effects of relatively high lake levels (Photograph 6). The armor stone was placed in a "laid-up" configuration, with a side slope of 1 V on 1-1/2 H (Photograph 7).

## 2. INVESTIGATIONS

a. Site Inspections. The project site was visited by Buffalo District geotechnical personnel on 21 October 1981 and 13 November 1981 to evaluate the existing condition of the revetments and related site features. Since only surficial details were visually evident, the observations included direct visual inspection as well as a search for evidence of implied distress beneath the surface.

b. Test Pits. Two test pits were excavated near the west end of the West Revetment to verify the presence of armor stone beneath an obscuring layer of 4 to 9-inch stone.

## 3. GEOTECHNICAL EVALUATION

a. Existing Conditions.

(1) Site Conditions

(a) Erosion/Deposition Processes. Slight to moderate erosion of the medium high bluffs updrift (to the west) and downdrift (to the east) of the project revetments is occurring (Photographs 1, 2, and 4). However, the narrow beach deposit that is accumulating at the West Revetment indicates a depositional environment for this reach except during storm periods (Photographs 1, 2, and 3). The coarse sands and gravels that have been deposited on the lower one-half of the armor stone indicate the significant limits of deposition from wave run-up during storm periods (Photographs 2 and 3).

(b) Bluff Stability. Failure scarps in soil are present in the bluffs both updrift and downdrift of the West and East Revetment (Photographs 1, 2 and 4). At the terminus of the West Revetment the scarps are within several feet of the perimeter fence. Recession rates of the bluffs appear to be only slight to moderate.

(2) Structure Conditions.

(a) Armor Stone Layers. Over most of the West and East Revetments the armor stone has good rock to rock contact with only a minimum amount of voids (Photographs 5, 6, and 7). The armor face is relatively uniform; the slope has good to excellent penetration at the water line (Photographs 5 and 7). The second layer of armor stone can be seen at numerous inspection points (voids - see Photograph 8). The outer layer of armor stone contains a fairly high amount of cracked stones. Although no counts were made, about one-third or more of the stones contained fractures that penetrate the interior of the stones (Photographs 15 and 16).

Three localized anomalous conditions were present in the armor layer. The first is a layer of fines (4 to 9-inch size) that veneer portions of the upper one-half of the revetments (Photographs 2 and 7). Test pits revealed the presence of a second layer of armor (Photographs 9 and 10). There was no evidence of removal of fines from the interior select fill of the revetment. The upper limit of coarse sand/gravel and drift deposition is below the lower limit of the 4 to 9-inch fines deposition which supports the lack of wave action/run-up that might cause removal of fines from the zone in question (Photograph 2, 3, and 10). Also, the lack of any distress in the cap stone layer tends to rule out removal of fines from wave action (Photographs 5 and 6).

A second anomaly is a localized zone of slippage on the West Revetment. A few armor stones have migrated down the slope on waste concrete placed during construction (Photograph 11). The stones have displaced down the slope several feet in some cases resulting in a minor loss of rock to rock contact.

A third anomaly is located on the East Revetment. A rather long linear void (about 25 X 5 feet) is present directly under the cap stones (Photographs 12, 13, and 14). The void is located some 100 feet from the west end of the revetment. The utility stated that the void was a construction oversight; i.e., the void was present at end of construction. This appears to be the case; there was no evidence to indicate removal of fines from the interior of the revetment. Also, there was no distress in the overlying cap stone.

(b) Cap Stone Layer. A row of cap stones were placed at the original revetment crest for added protection. The stones appear fairly uniform in alignment and show no signs of distress that would be indicative of settlement or migration of armor stone down the slope (Photographs 2, 5, 6, and 7). The cap stone layer contains a very high percentage of cracked stones. Although no counts were taken, the vast majority of the stones contained fractures that penetrate the interior of the stones (Photographs 17 and 18).

(c) Toe Stone Layer. The layer of toe stones was only partially visible in some areas. At the West Revetment, the narrow beach deposit obscured the toe of the revetment. At the East Revetment, both the lake level and sand deposits tended to obscure clear examination of the toe stones although the tops of some toe stones were visible. However, the overall good to excellent penetration of the armor stone layer at the water line supports the probable presence of an intact toe. The only visible exception is at the second anomaly area previously discussed in 3a (2)(a).

(3) Foundation Conditions. Since no subsurface exploration information is available at the project site, an evaluation of existing foundation conditions must be based on observations of surficial features. Foundation distress would typically be evidenced by appreciable settlement of armor, cap and/or toe stone, significant displacement of the armor slope due to shear failure, or deviations in alignment in the security fence or retaining wall behind the revetment. Since the structure was built around 1965, it has a reasonable duration of service. None of the above occurrences were observed, nor were any other modes of failure or distress observed. Therefore, it is concluded that favorable foundation conditions exist beneath the East and West Revetments.

b. Structure Stability. Both revetments appear to have good structural stability due to visual observation of numerous positive features and the lack of evidence of any significant structural distress.

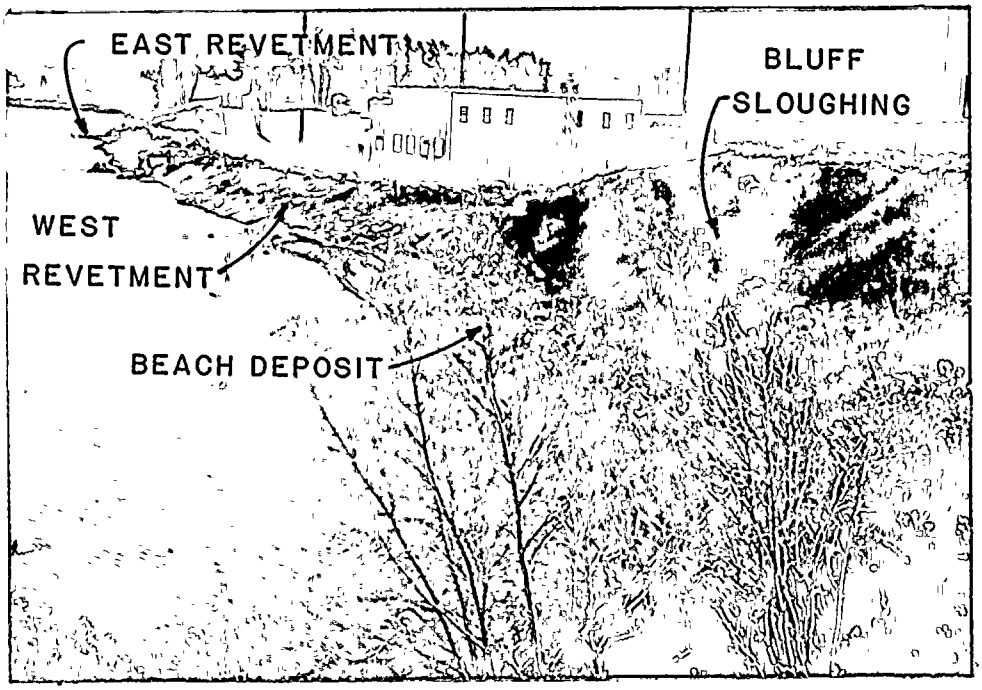
The side slopes of both the East and West Revetments are uniform to very uniform without any significant breaks. The slope armor stones cut the water sharply and continuously. The cap stones are in good alignment, both vertically and horizontally. Good to excellent rock to rock contact exists throughout, with minimal voids between adjacent rocks. The second layer of armor stone is apparent and continuous throughout. Although a significant fraction of stones in both revetments are fractured, this will not cause structural instability since the minimum 5-ton armor size is probably several times the required design size. The East and West Revetments have performed adequately over a service period of more than 15 years.

No signs of significant distress were observed. The only indication of structural instability is the slight displacement of armor stone (second anomaly). This poses no serious threat to the integrity of the West Revetment. There was no evidence of loss of fines from the underlying revetment zones due to the effects of wave action. No significant displacements of armor, toe or cap stone were evident, implying that adequate structural stability exists in the zone beneath the armor layers.

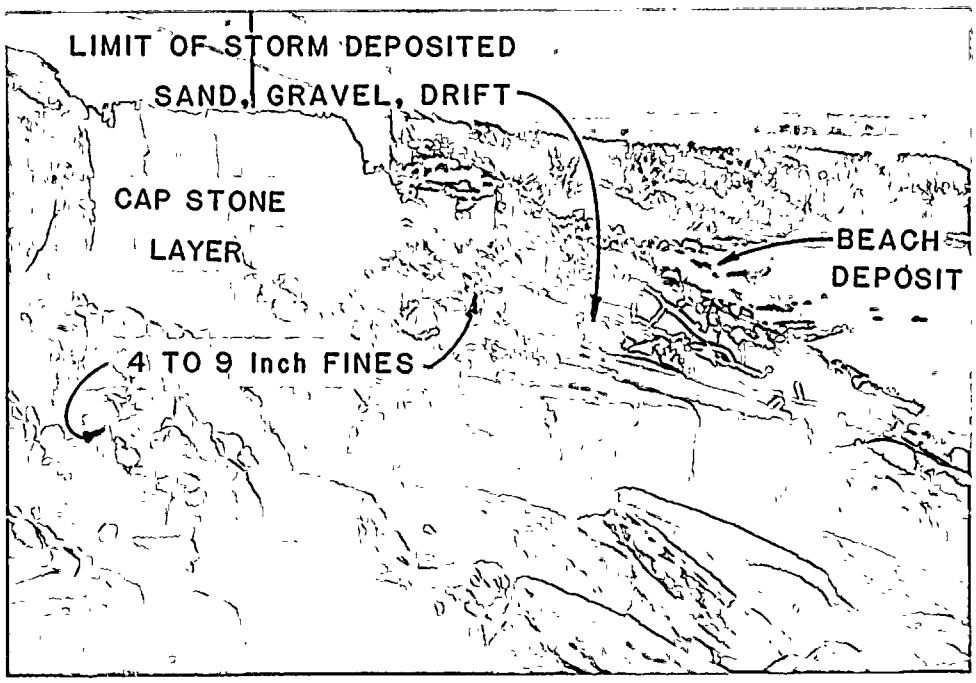
4. CONCLUSIONS.

Site investigation and analysis indicate that the initial NRC evaluation of the condition of these revetments is in error. Both the East and West Revetments have demonstrated very satisfactory performance during their 16-year period of service. Both structures appear to be providing the intended design function, i.e., protect the R.E. GINNA Station from wave attack.

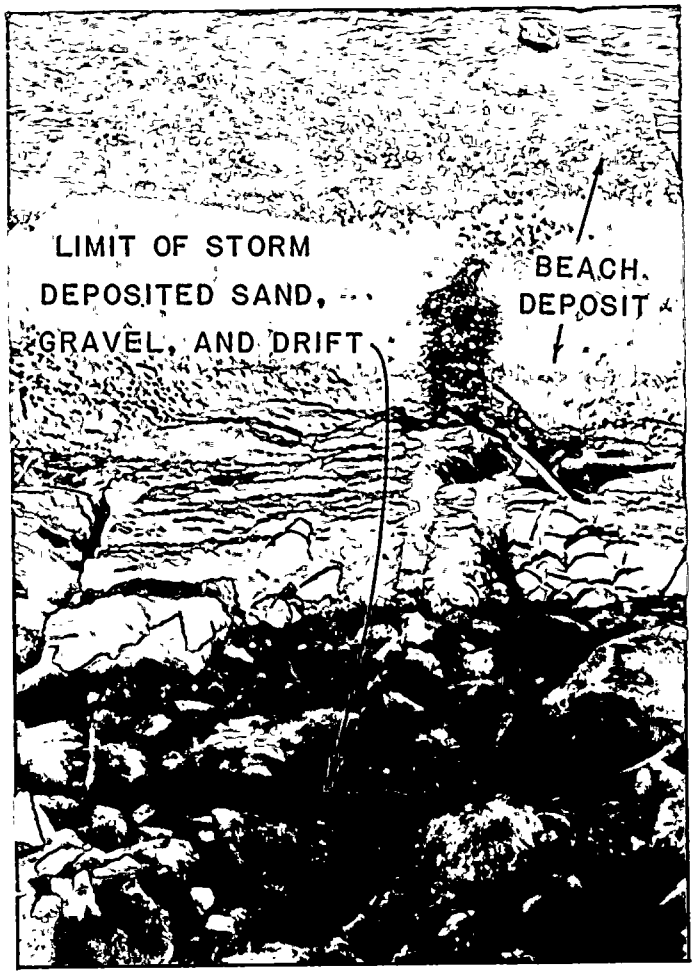
A monitoring program should be prepared and implemented to evaluate the future performance of both revetments. This can be accomplished by the establishment of permanent survey points at sufficient locations along the axes of both structures, with periodic resurvey and comparison. Also, the monitoring program should measure and evaluate erosion/deposition processes in the immediate project vicinity, including activity at the structure toe and the immediate adjacent bluffs, which might affect the performance of the project.



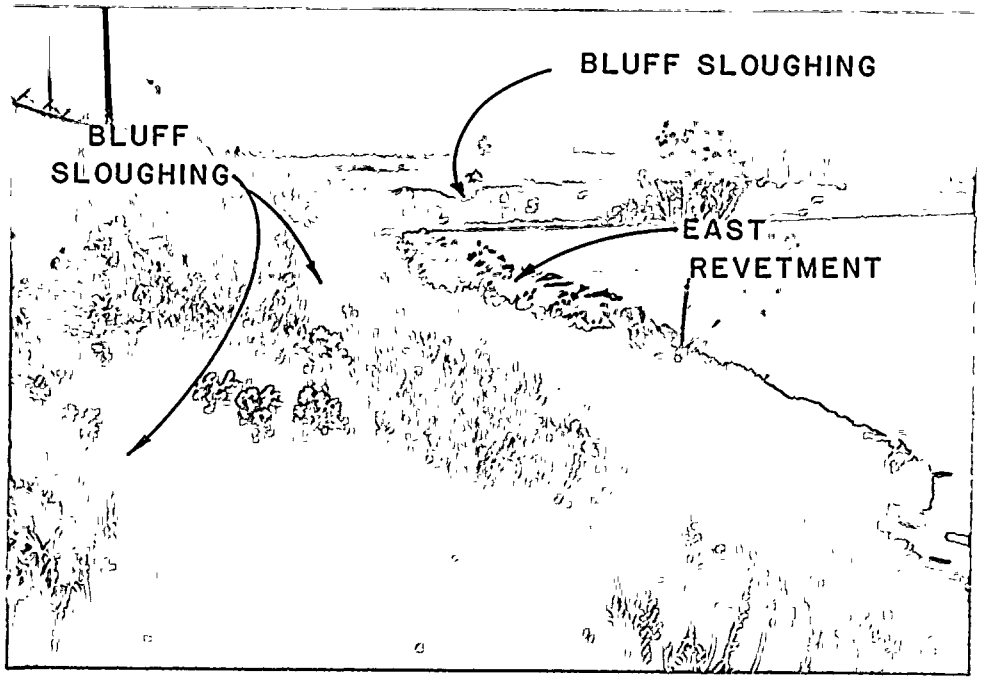

Photograph 1 - East and West Revetments with adjacent bluff at right (looking east).



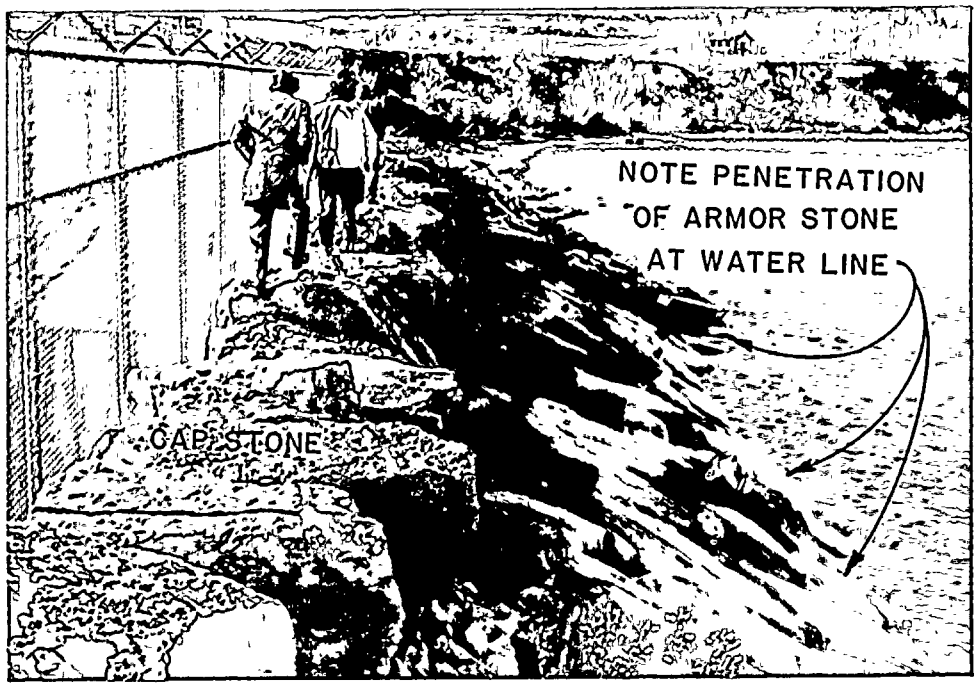

Photograph 2 - Top of West Revetment near west end, looking west.



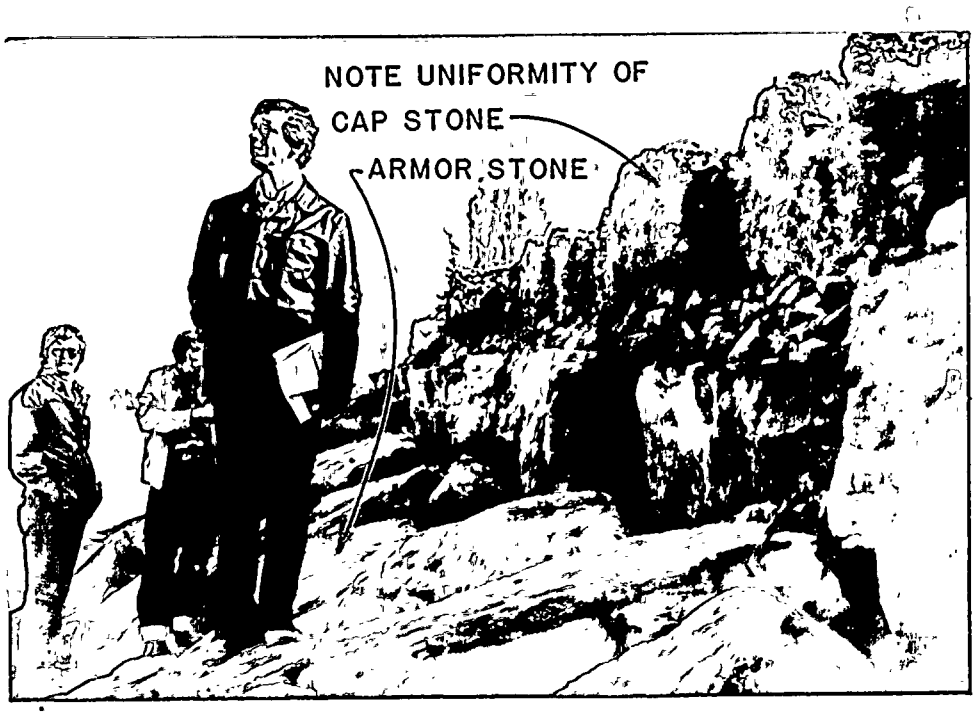

Photograph 3 - Looking at toe of West Revetment from the crest.




Photograph 4 - Bluffs adjacent to end of East Revetment (note sloughing), looking west.




Photograph 5 - Looking west at crest of West Revetment, near west end.




Photograph 6 - West Revetment, looking at top of armor stone slope and cap stones.

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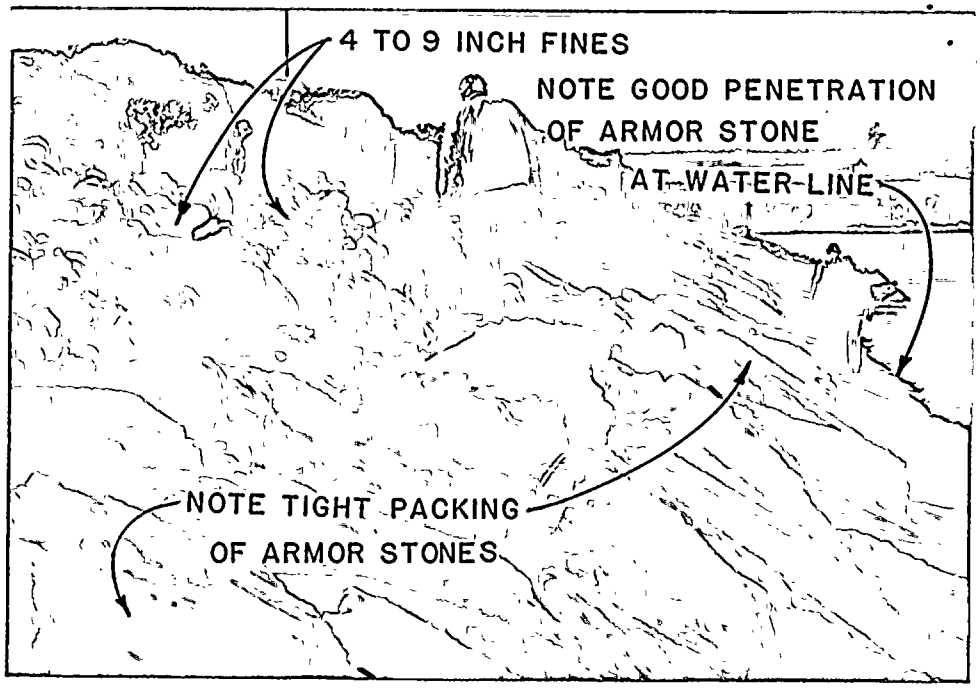
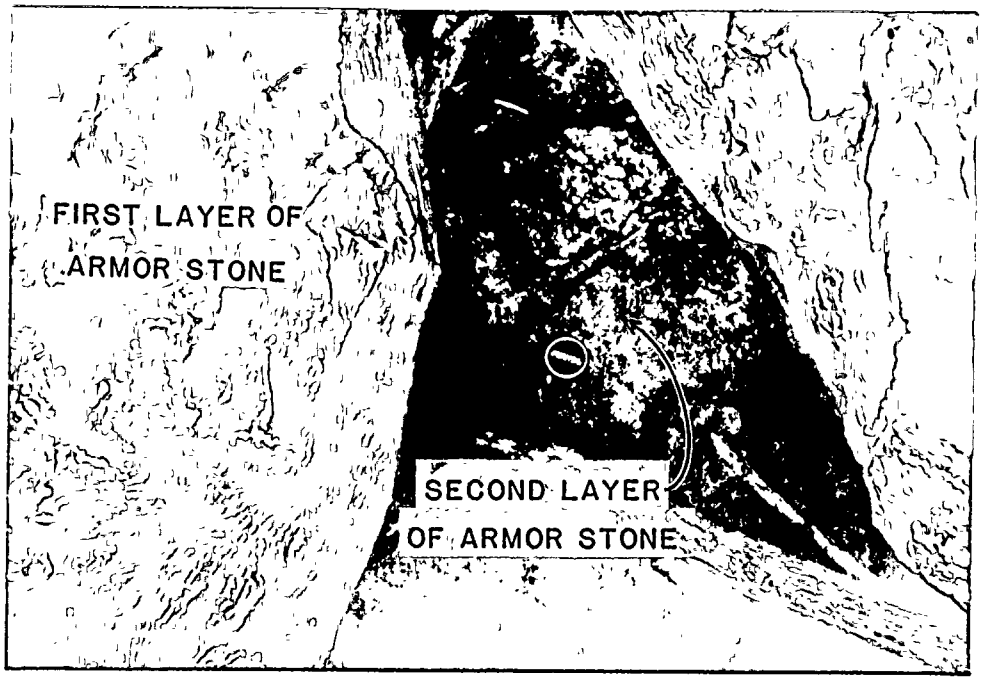
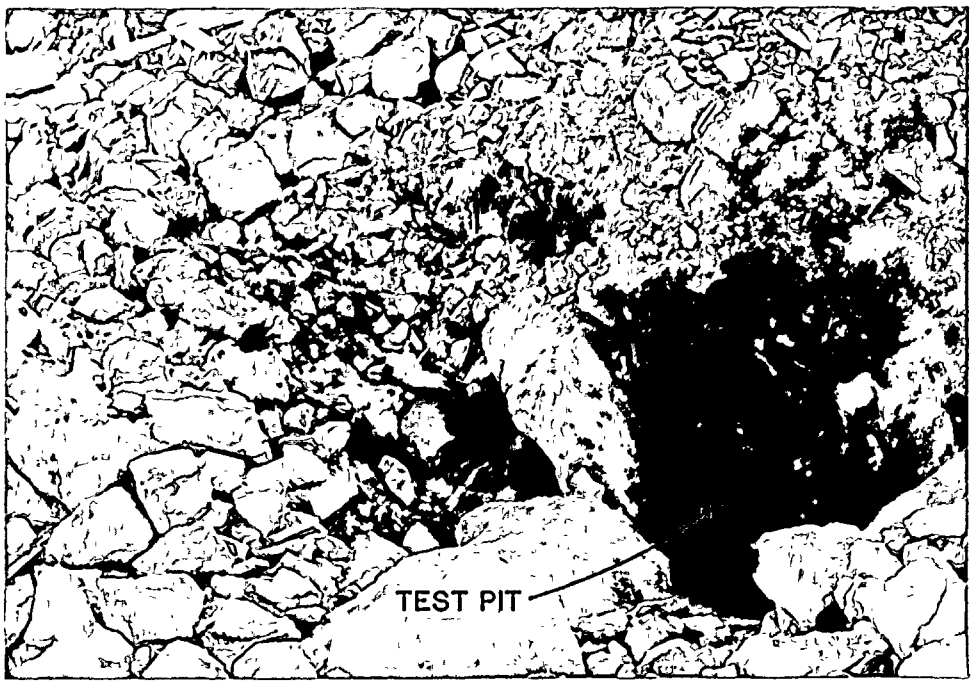


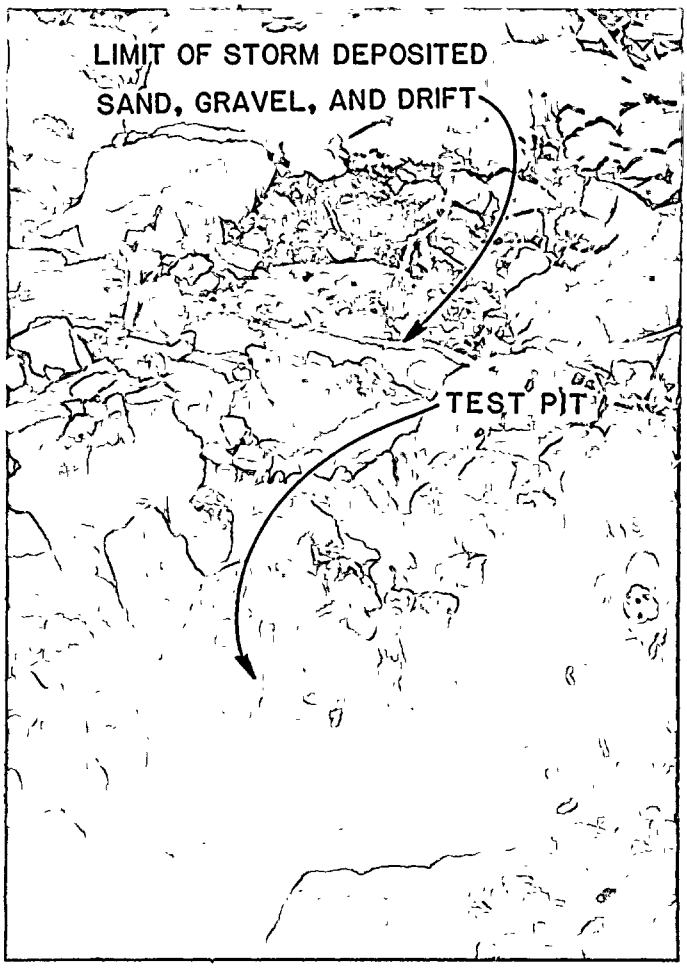

Photograph 7 - East Revetment, looking west, viewing armor stone slope and 4 to 9-inch fines on slope.




Photograph 8 - Gap between adjacent armor stones in West Revetment, showing second layer of armor stone.




Photograph 9 - Test Pit in West Revetment revealing continuity of armor stone.




Photograph 10 - Test Pit in West Revetment revealing continuity of armor stone.

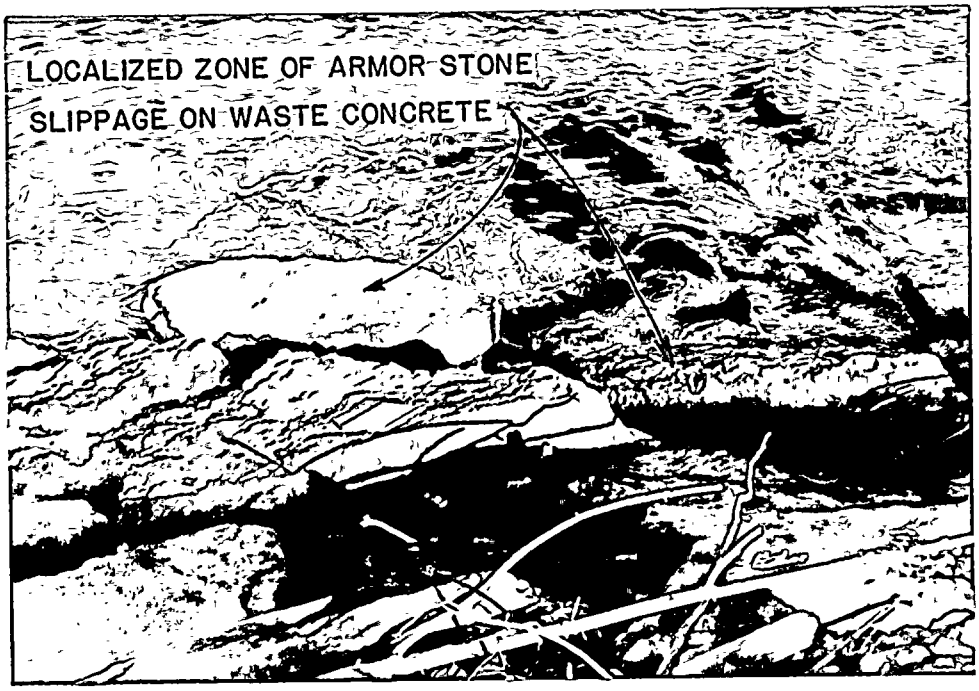
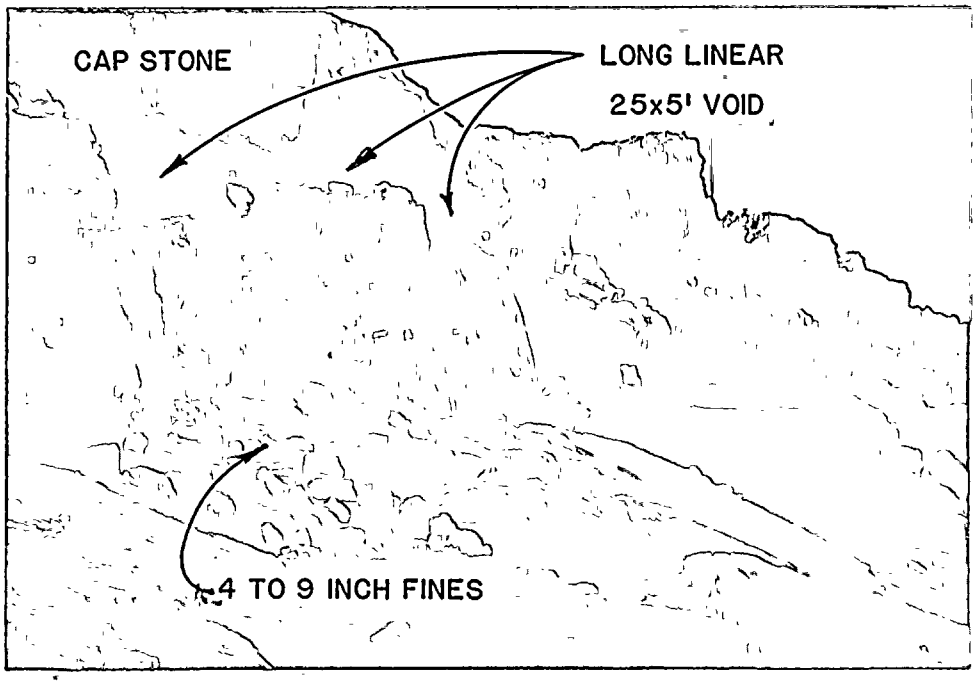
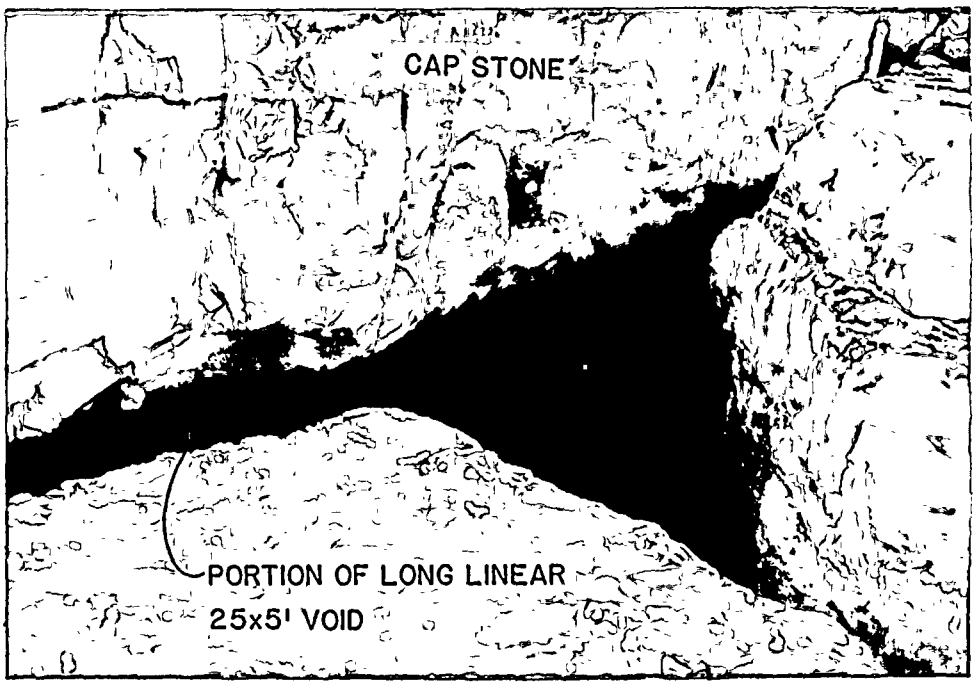


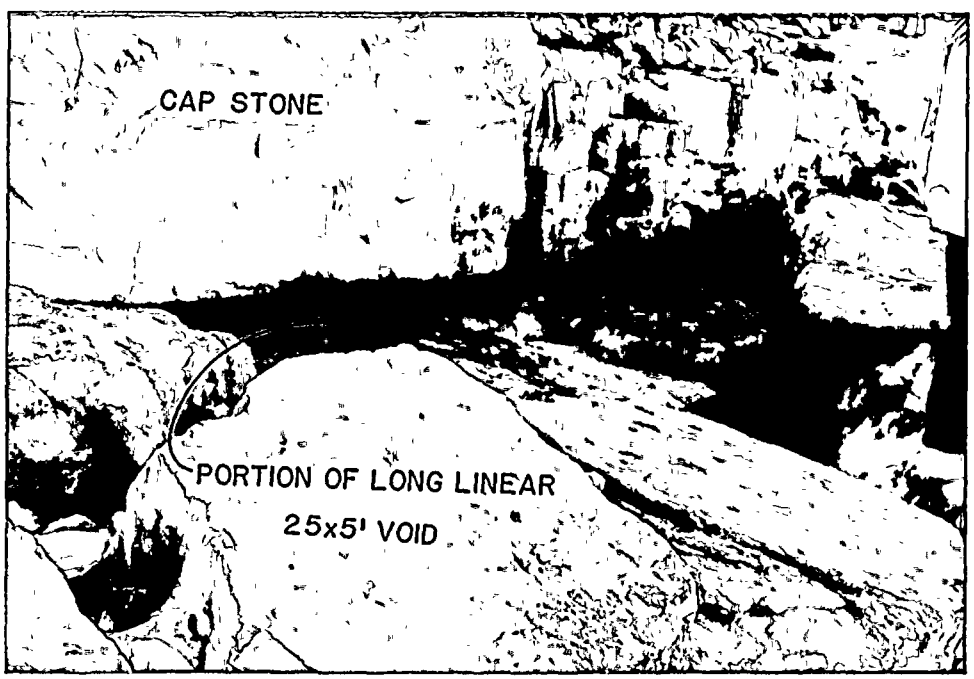

Figure 11 - Localized slippage of armor stones in West Revetment.



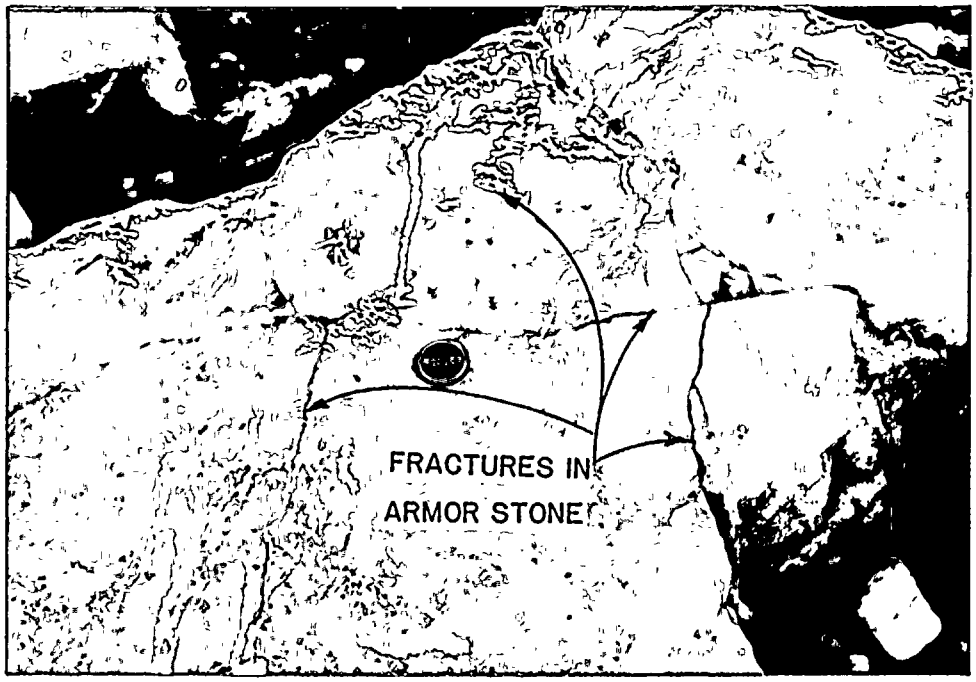

Photograph 12 - Large linear void in East Revetment beneath cap stones.



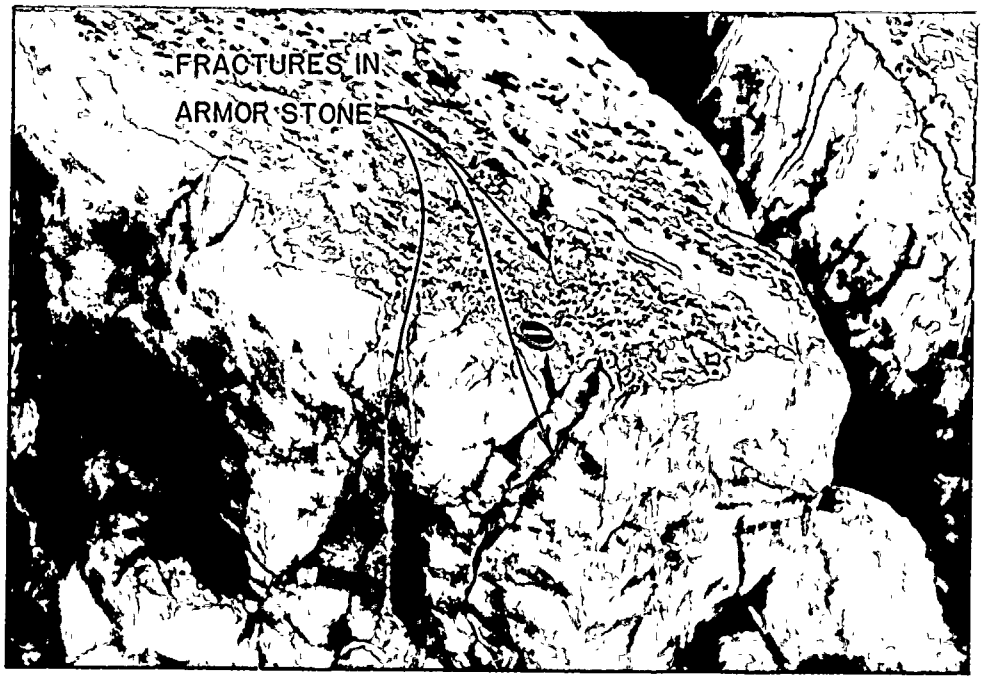

Photograph 13 - Closeup of long void in East Revetment beneath cap stones.



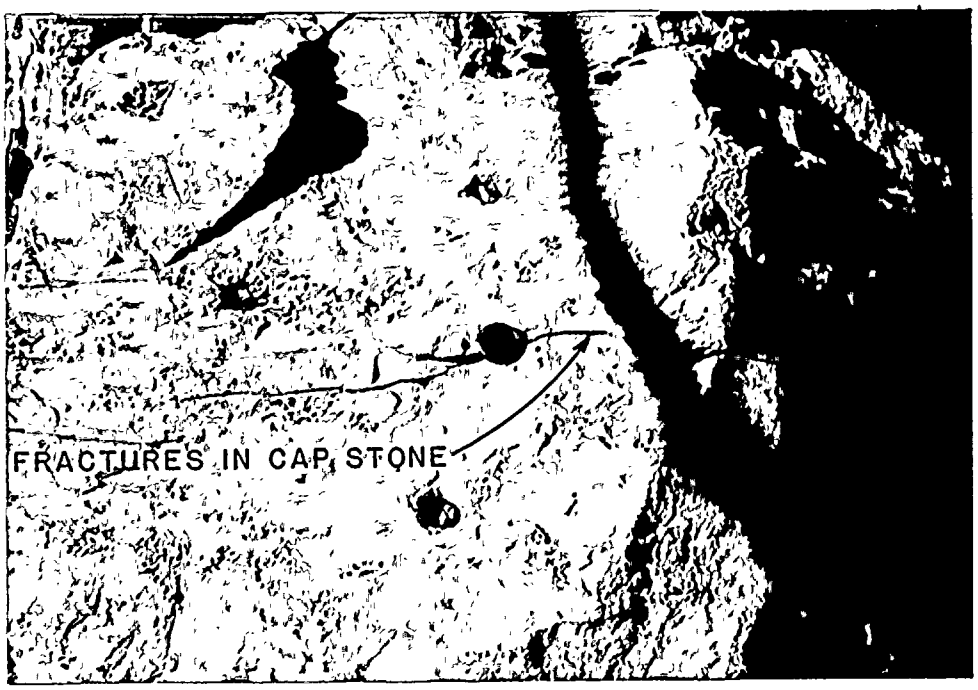

Photograph 14 - Closeup of long void in East Revetment beneath cap stones.



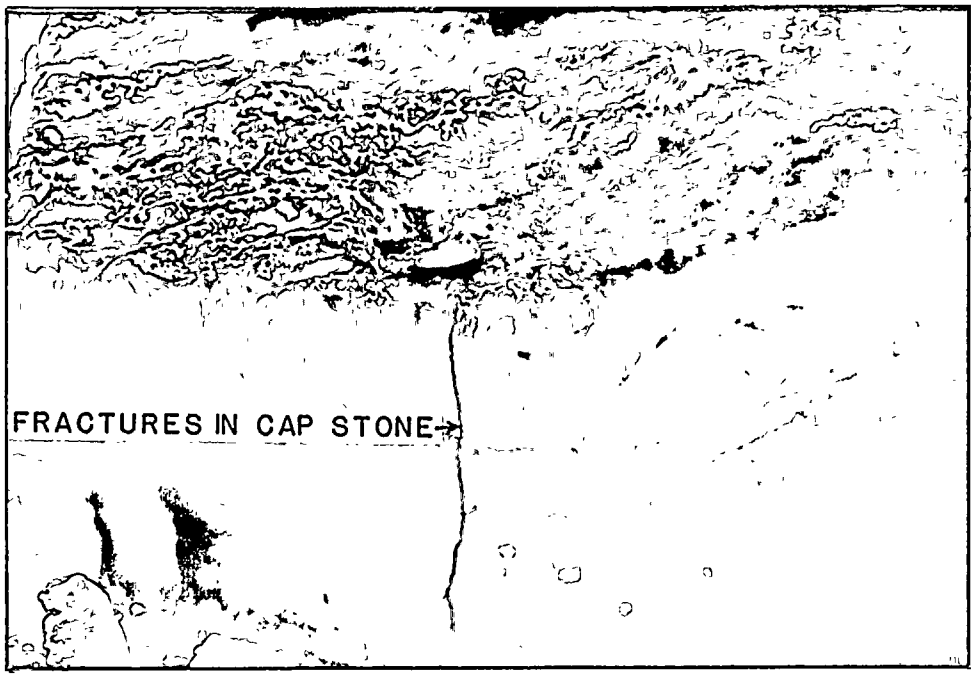

Photograph 15 - Typical fractures in armor stone.



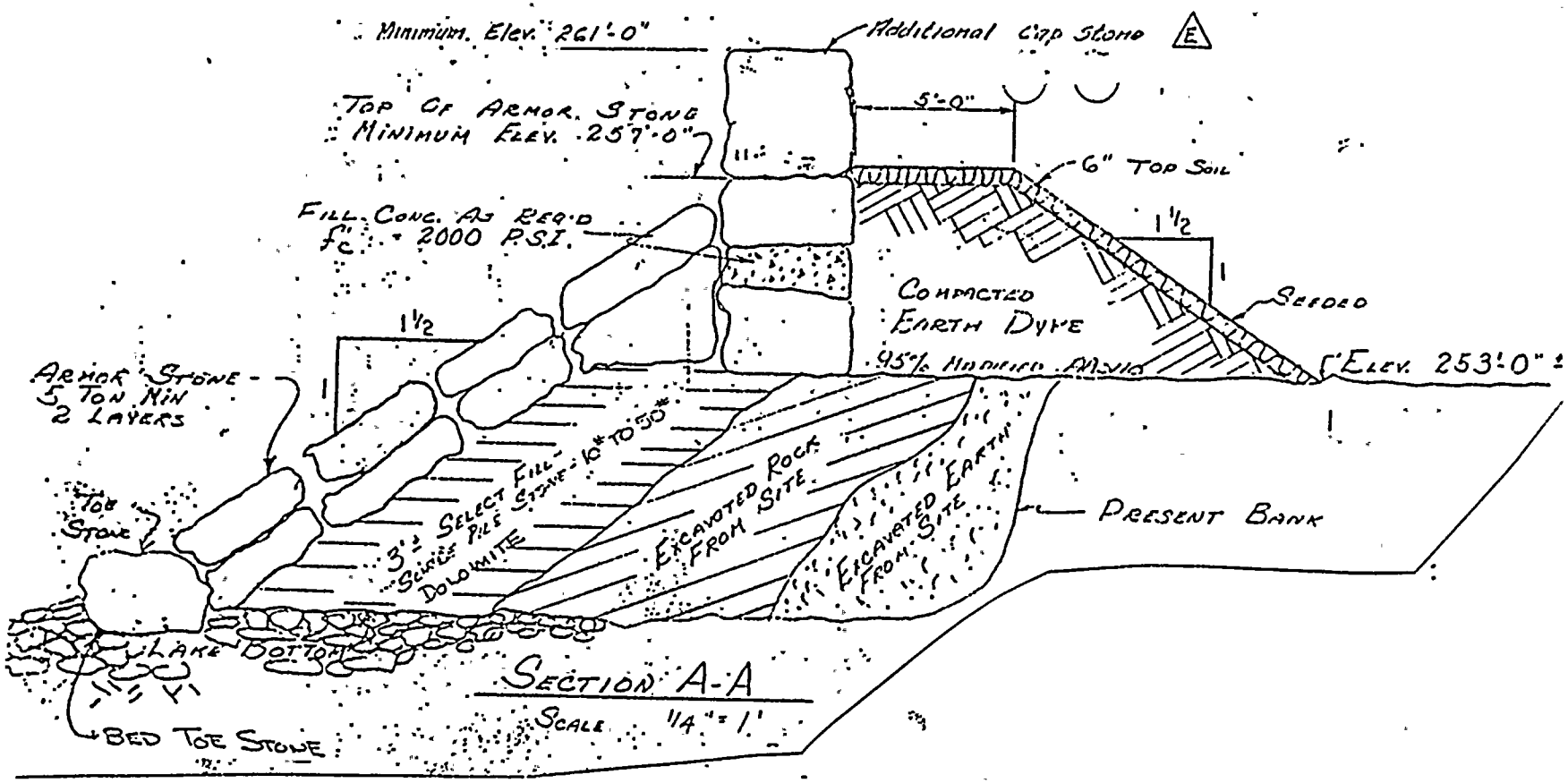

Photograph 16 - Typical fractures in armor stone.




Photograph 17 - Typical fractures in cap stone.

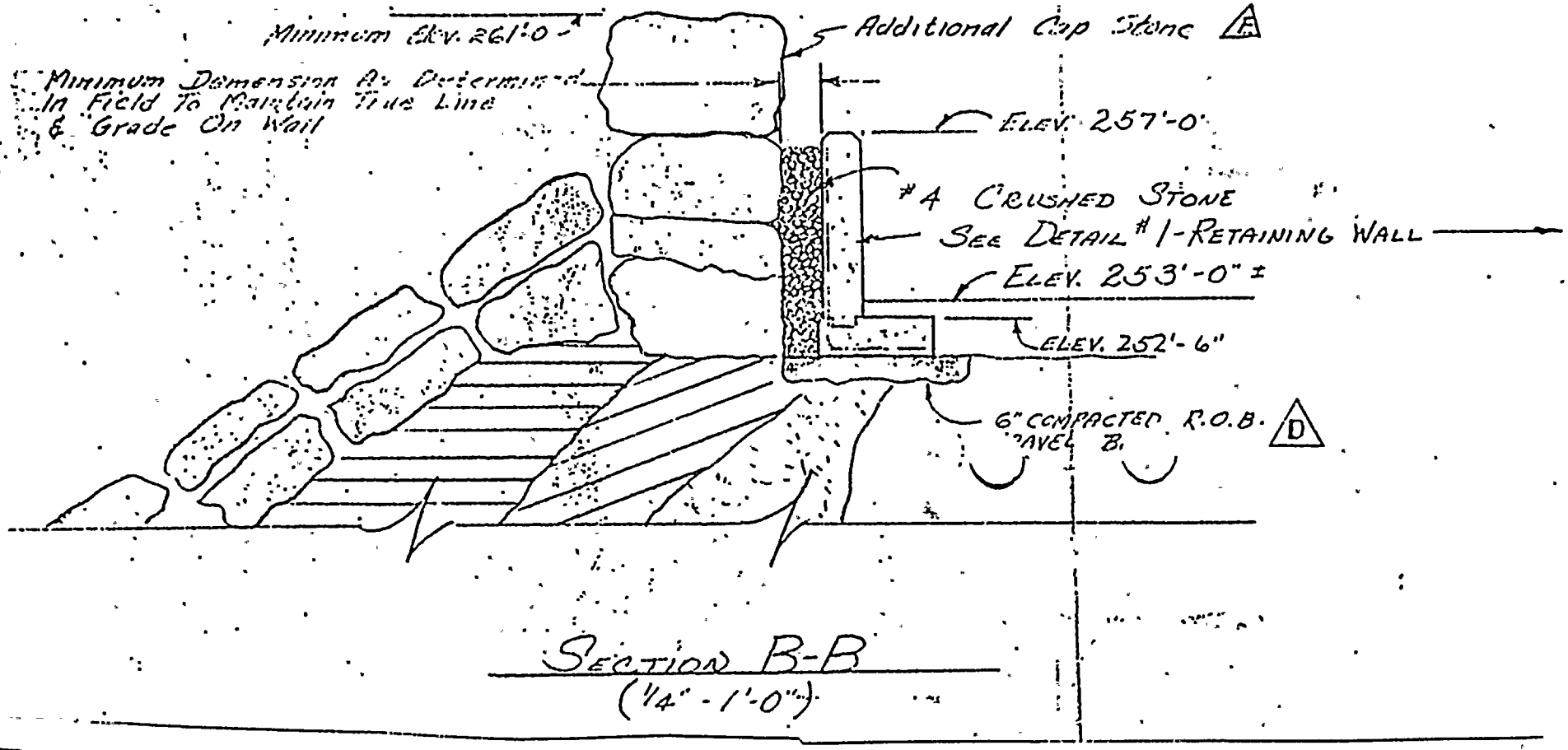



Photograph 18 - Typical fractures in cap stone.




TYPICAL SECTION – EAST REVETMENT REACH

FIGURE 1




TYPICAL SECTION — WEST REVETMENT REACH

FIGURE 2