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Failure Investigation and Restoration of Two Cellular Sheetpile Structures

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FAILURE INVESTIGATION AND RESTORATION OF TWO CELLULAR SHEETPILE STRUCTURES

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ABSTRACT

The Trainer - Delco Tap - Mickleton 220-38 kilo-volt transmission lines are carried across the Delaware River by two 332-foot high steel latticed towers each founded on a man-made foundation island structure. Each island structure is comprised of four interconnected cellular sheetpile structures. One island, suffered a severe partial failure due to long-term scour in the Delaware River, near Chester, Pennsylvania. The other island exhibited early symptoms of potential failure, also due to scour. The client was the Philadelphia Electric Company, now known as PECO, acting on behalf of the owner, the Atlantic Electric Company (AECO).

The author served as project manager and principal investigator for AECOM (formerly Earth Tech, formerly TAMS). The paper describes the failure investigation, including the structures before and after failure, the original installation (1959-1960), the condition survey of each island, condition of the failed sheetpiles, divers' findings of an underwater survey, hydrographic studies, scour and loss of sheetpile embedment. Also described are the subsurface investigation, soils laboratory testing, the soil/rock profile, the probable cause(s) of failure, the sequence comprising the failure mechanism, metallurgical findings, circumferential stress analysis and brittle failure of the sheetpile panels outside the interlocks.

Remedial measures are described and the design and construction of the selected restoration/stabilization solution via a crushed stone buttress is presented. The author established the construction sequence and provided technical liaison to PECO during the underwater staged construction, which included geo-instrumentation and hydrographic monitoring of an 80,000 cubic yard crushed stone and riprap protected circumferential stabilizing buttress, over 50 ft high, placed around the failed island in the Delaware River. The failure investigation, the design and the restorative construction occurred during 1991-1994, yet the lessons learned from this case history are as aptly important today.

INTRODUCTION

On May 7, 1991, PECO authorized AECOM to proceed with a failure investigation of two manmade foundation islands located in the Delaware River. The investigation included a condition survey, an underwater inspection, a determination of the probable cause(s) of the partial collapse of the cellular structure of one of the islands, and to propose appropriate conceptual design alternatives for restoration of both islands to a long-term safe condition. In order to provide an appropriate level of quality assurance to the project, the author recommended the engagement of Edwin Paul Swatek, Jr., P.E. to review the findings and recommendations of AECOM's draft report and to visit the site with the author. The author was aware of Mr. Swatek's expertise from a paper (1970) by Mr. Swatek.

Description of Structures

The Trainer - Delco Tap - Mickleton 230 kV transmission lines cross the Delaware River in the vicinity of Chester, Pennsylvania. The aerial crossing is supported by four lattice-type towers. Two 150 ft high anchor towers are founded on land and located near the shoreline in Chester, PA and Bridgetown, NJ. Two crossing towers, 332 ft in height, are located in the Delaware River. One tower called the New Jersey Tower is approximately 1000 ft from the New Jersey shoreline. The other tower, called the Pennsylvania tower is located approximately 400 ft from the Pennsylvania shoreline. The foundation islands for both of these towers were erected in 1959-60 to protect the tower foundations from ship and ice impact, provide lateral support to the tower foundation H-piles, create a platform for driving the H-piles and for subsequent inspection and maintenance of the tower structures. The islands are constructed of steel sheetpile cells filled largely with granular material. Each island has four main circular cells, approximately 66-ft in diameter, with connecting arcs joined to the cells with riveted 90° tee sections. A project location map is shown in Fig. 1 and the original layout of the cells is shown in Fig. 2. Both Cell A and Cell B of the New Jersey Island experienced partial collapse as outlined in Fig. 3. On the Pennsylvania Island, Cells A and B exhibited early signs of potential failure that if not corrected could lead to a collapse similar to the one that had already occurred on the New Jersey Island.

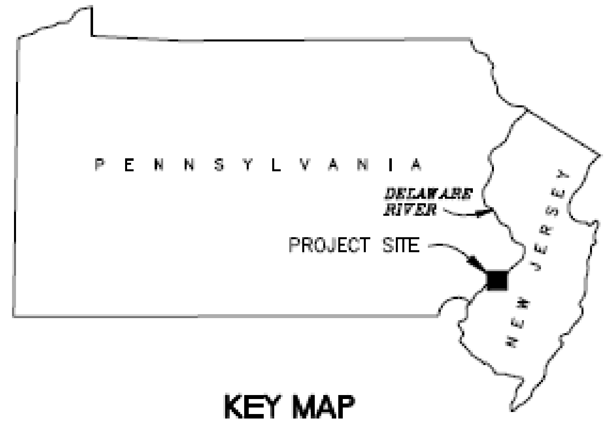

Fig. 1: Project Location

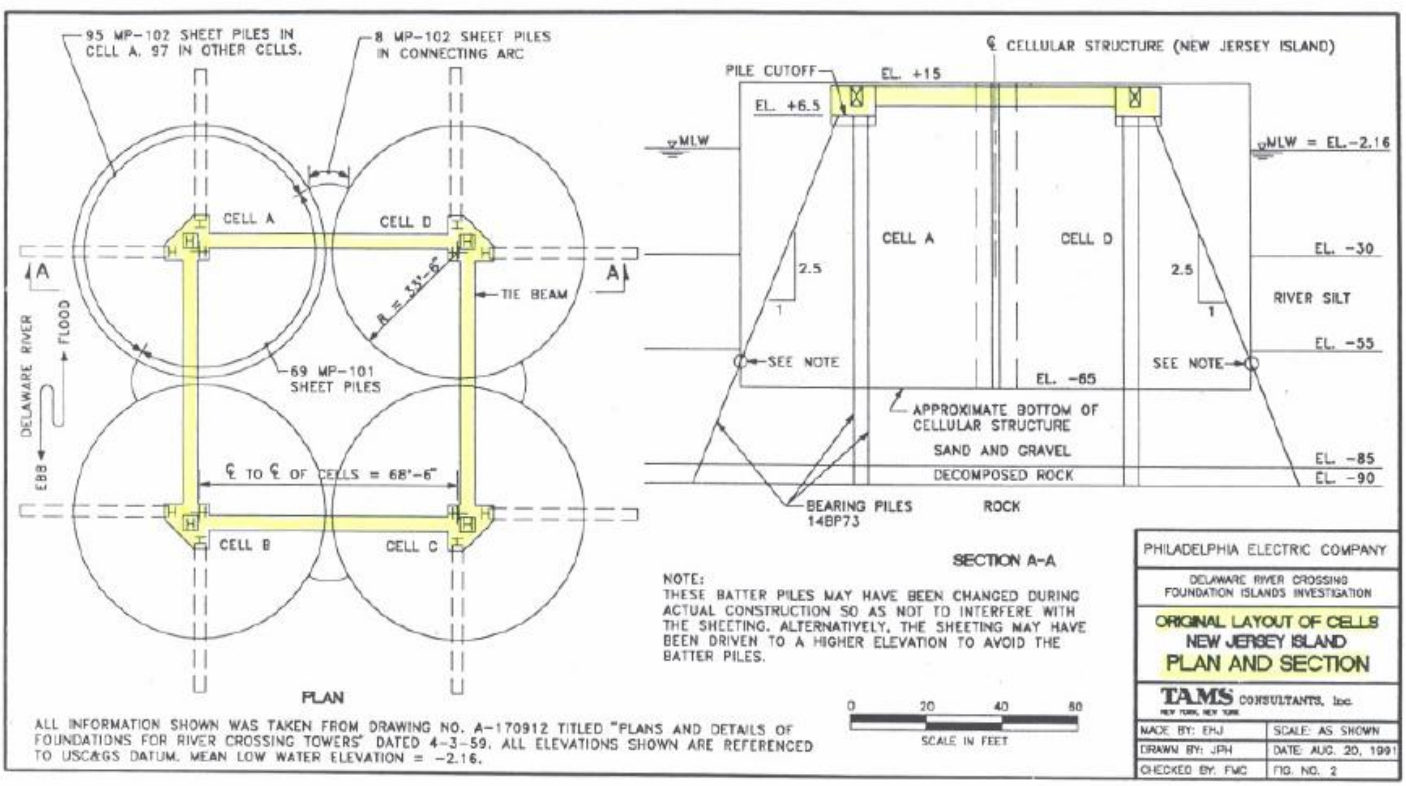

Fig. 2: Plan and Elevation of New Jersey Island

The exterior sheetpiles, which have one face exposed to the river, are US Steel MP-102 sections with ½-inch web thickness. The interior sheetpiles, which are backfilled on both sides, are MP-101 sections with 3/8- inch web thickness. US Steel T2A riveted 90° tee connections were used. These sections conform to ASTM Specification A-328 and are made of A36 structural steel with a minimum interlock strength of 16 k/in (separation of interlocks in direct tension), minimum ultimate strength of 70 ksi and a minimum yield point of 39 ksi.

The crossing towers are supported on 14BP73 steel H-piles designed for bearing on rock. The H-piles were driven through the cell fill. Each tower leg pile group has two vertical and two batter piles embedded in a concrete pile cap. The pile caps are tied together with concrete grade beams. The cutoff elevation of the interior sheetpiles was stepped down to permit construction of the grade beams.

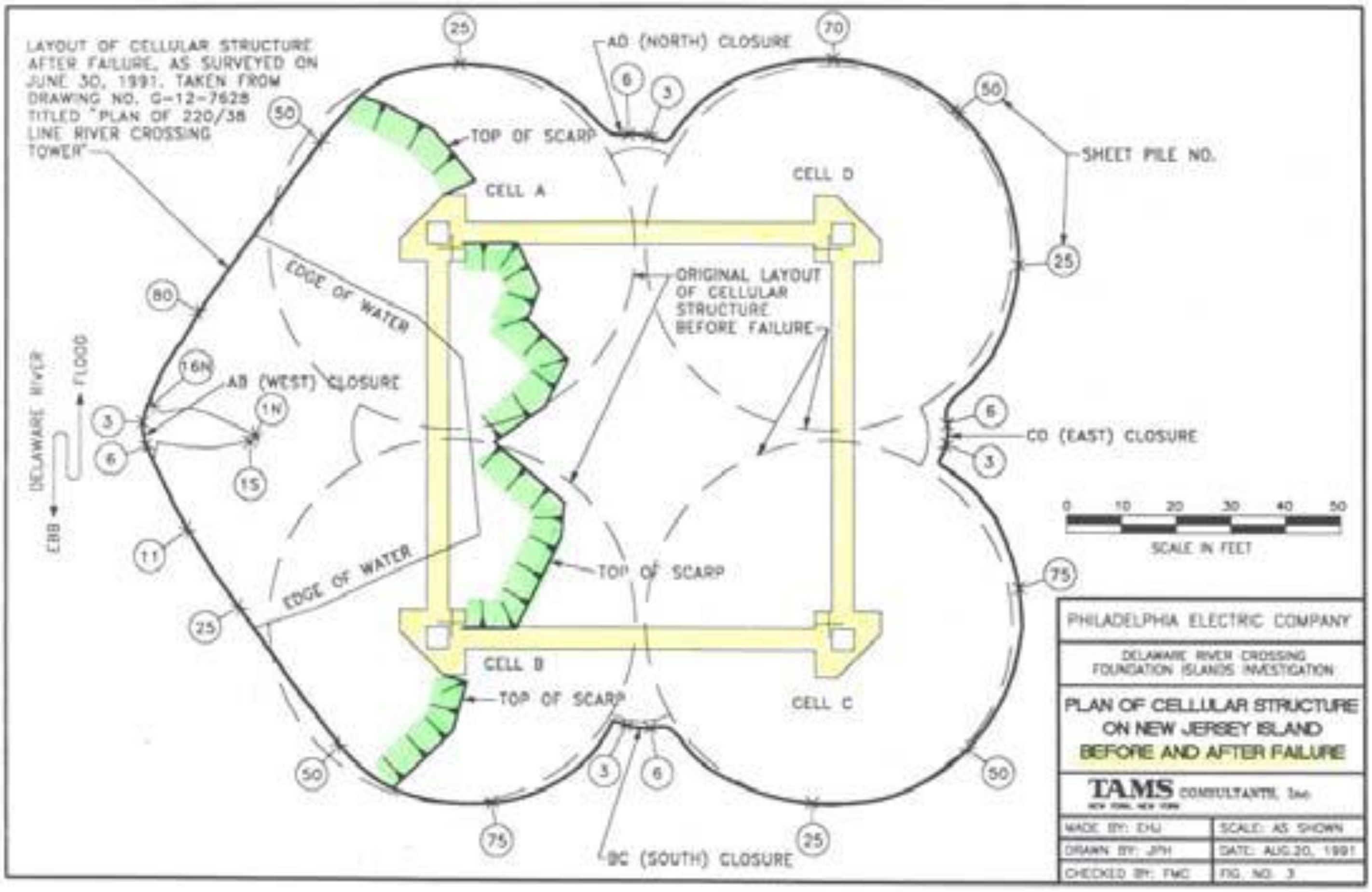

Fig. 3: Before and After Failure

Existing Borings and Soil Profile

Subsurface conditions in the area were determined from a series of deep borings drilled to rock. In the vicinity of the New Jersey Island the channel depth is about 30 ft. The top stratum is composed of river silt and sand seams extending to a depth of about 25 ft. This is underlain by a sand and gravel layer approximately 30-ft thick above a 5 to 15-ft layer of decomposed mica schist. Bedrock is mica schist. A subsurface profile in the vicinity of the New Jersey Island is shown in Fig. 4.

The subsurface conditions on both islands are similar, except the thickness of the silt layer and the depth to the sand and gravel layer under the Pennsylvania Island is less than under the New Jersey Island.

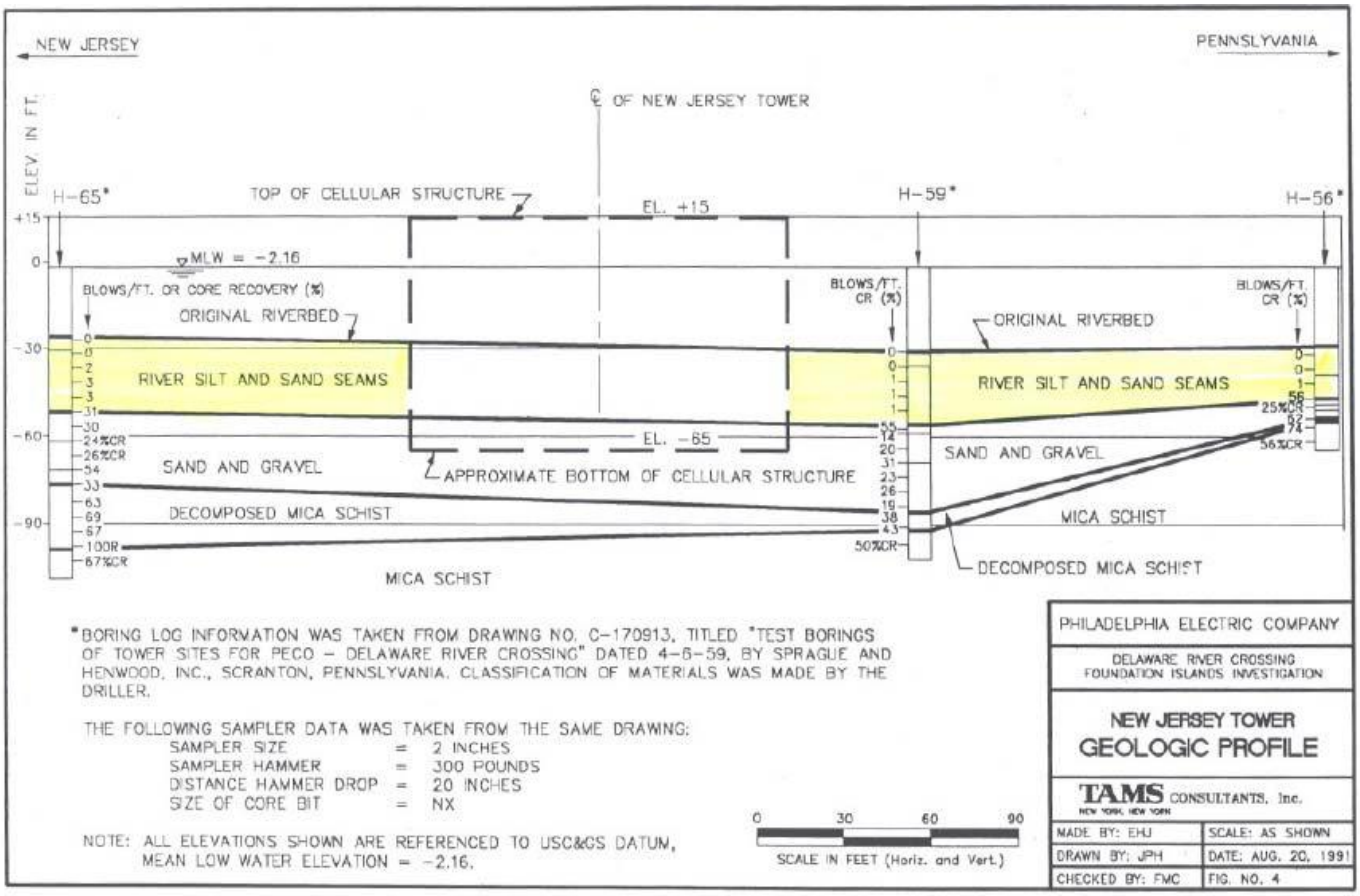

Fig. 4: Idealized Soil and Rock Profile

C. Installation

According to the original design construction drawings, the sheetpiles were to be driven to the sand and gravel layer. The design called for 80-ft long sheetpiles for the New Jersey cells, and 60-ft long sheetpiles for the Pennsylvania cells. Sheetpiles were to be driven to either the cut-off elevation or to refusal (0.1 inch penetration under a minimum of 7000 ft-lb of driving energy), if that occurred first. Because the sheetpile driving records are no longer available, the as-built condition of the sheetpiles was not known.

The foundation piles for each tower were driven after the cellular structure was completed. These piles were driven to a bearing capacity of 90 tons as determined by the Engineering News Formula or to refusal on hard rock.

The H-piles were provided with corrosion protection by coating them with Tarset, a coal tar epoxy, and with an impressed current cathodic protection system, whereas the steel sheetpiles did not have corrosion protection.

Reports of Failure

In early 1991 PECO performed a routine aerial survey in the vicinity of the transmission line crossing. The surveyors did not note any gross changes in the geometry of the New Jersey Island. A pre-existing sinkhole was observed, and it did not appear to have enlarged since the previous aerial survey. Nevertheless, on Sunday, April 14, 1991, a boater telephoned PECO to report that the New Jersey Island appeared damaged. PECO personnel visited the island on April 16, 1991 and corroborated the damage report. The two cells facing the channel had partially collapsed and leaned out toward the river. Sheetpiles were ruptured and severely distorted. This allowed the fill inside the cells to displace laterally downward, exposing two pile caps and the grade beam that connected them. Fig. 5 and Fig. 6 are photographs of the failure.

Fig. 5: Partial Collapse Exposing Pile Caps and Tie Beams

Fig. 6: Partial Collapse Exposing Pile Caps and Tie Beams

FAILURE INVESTIGATION

Review of Existing Information

Existing information on the design, construction and performance of the foundation islands was reviewed in the preliminary stages of the failure investigation. Available documents relating to the design and construction of the islands were received from PECO's archives. Additional survey information for the Delaware River channel was provided by the US Army Corps of Engineers and the National Oceanographic and Atmospheric Administration. A literature search was conducted to provide information on design and construction practices in use at the time the islands were built and the performance of similar structures. Telephone interviews were conducted with PECO personnel who were involved with the construction and maintenance of the islands.

Surface Investigations

A land survey was performed on the New Jersey Island by PECO personnel to define the post-failure configuration of the fill and exposed portions of the sheetpiling. The top of fill was marked on the inside of the exposed sheetpiles to provide a rapid means of evaluating new movements of the fill. The failure surface was established for future monitoring. A plan of the surveyed area is presented in Fig. 3. A visual inspection was made of the sheetpiles, exposed pile caps, grade beam and exposed portions of the H-piles. Micrometer readings were taken on the exposed portions of the H-piles. Soil samples were retrieved from the upper 8 ft of the failure surface for sieve analysis and determination of Atterberg limits. Ultrasonic thickness measurements of the webs of the sheetpiles were taken and the cathodic protection systems on both islands were evaluated. On the Pennsylvania Island a sinkhole was measured and the top of fill was marked on the inside of the exposed sheetpiles to track further subsidence.

Underwater Investigations

Divers performed visual inspections of every sheetpile from the low water line to the mudline. The inspection also included the underwater area accessible inside the failed cells. The divers noted the general condition of the sheetpiles and interlocks and examined areas of potential weakness or apparent defects, such as severe pitting, missing or deteriorated welds, and lack of embedment of the sheetpiles. Samples of soil were retrieved underwater from the mound of soil displaced in front of the failure zone for soil mechanics analysis. Ultrasonic thickness measurements were taken at selected vertical and horizontal intervals along the cells to determine the extent of corrosion of the sheetpiles. A hydrographic survey of the area extending 200 ft beyond each island was conducted with the survey data presented in 2-ft intervals. The depths of scour at selected sections around the New Jersey Island are shown in Fig. 7. Scour adjacent to the cells was measured by a diver using a pneumo-fathometer.

Analysis of Materials

Representatives from PECO's Metallurgy Laboratory and AECOM jointly selected coupons from the sheetpiles in the failed area for testing and analysis. A chemical analysis of a water sample taken from the ponded water inside the failed cells was also performed.

Stress Analysis

Simple calculations in accordance with accepted procedures were made to estimate earth pressures and corresponding tensile stresses in the cellular sheetpile structure as a guide to establishing the cause of failure.

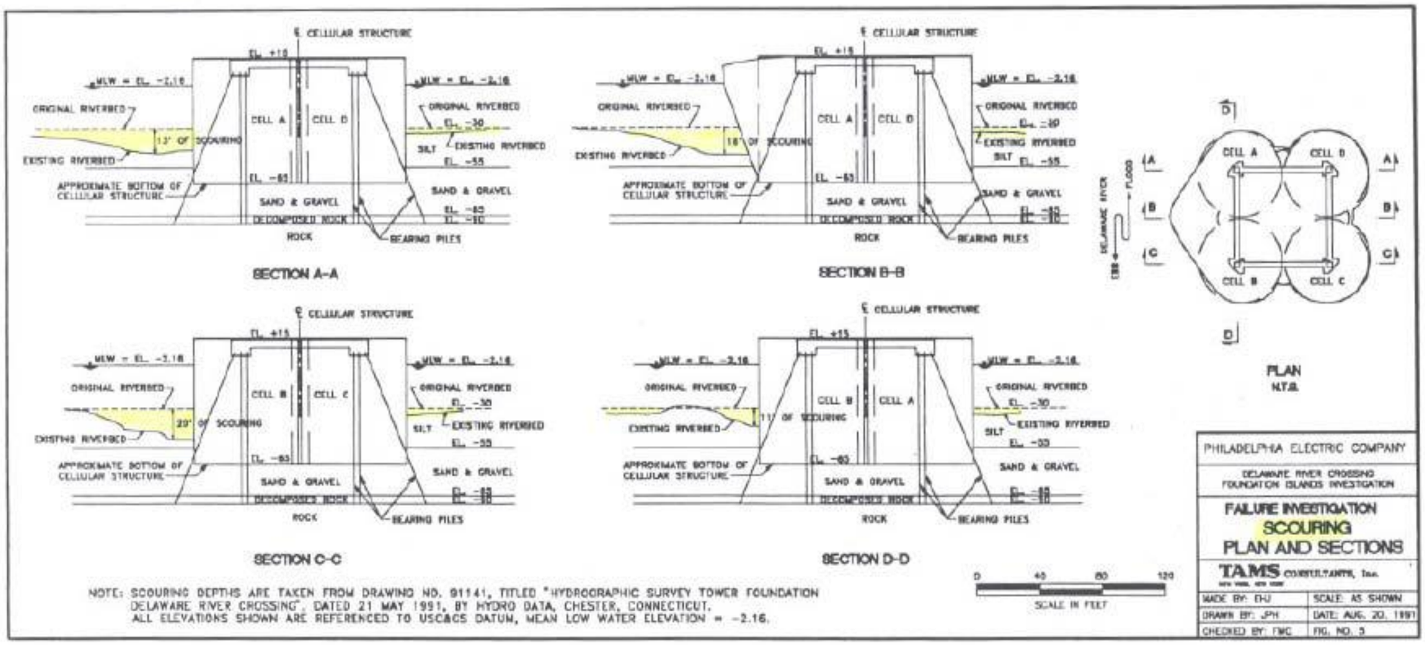

Fig. 7: Depth of Scour

REVIEW OF EXISTING INFORMATION

Original Construction Drawings

Existing drawings were obtained from PECO and US government agencies. The drawings were used to prepare a geologic profile of the New Jersey Island (Fig. 4) and to define the original configuration of the cells (Fig. 2).

Archival Documents

Documents received from PECO's archives included the original specifications for the sheetpiles, H-piles, fill, concrete and corrosion protection features and the pile driving records for the steel H-piles on the Pennsylvania Island. The design calculations, sheetpile driving records and H-pile driving records for the New Jersey Island were no longer available or could not be located. Longtime PECO personnel indicated that additional fill had been placed on the islands in the post-construction period, but no records of the work could be located.

Oral Recollections

AECOM spoke with PECO personnel who were involved with the construction and maintenance of the two islands. At the time of the interviews in 1991, the events they recalled had happened as much as thirty years ago and no written documentation of their original observations was available.

The following is a summary of relevant information drawn from the oral recollections: There is some uncertainty about the actual slope of the batter H-piles. The construction drawings contain a note indicating that the slope was changed, yet a slope of 1:2.5 is shown, which would intersect with the lower portion of the sheetpiling. Additional fill had been placed on the New Jersey Island several times since the original construction was completed. Fill had also been added periodically to the Pennsylvania Island, but the rate of loss of fill and the size of the sinkholes have been less than on the New Jersey Island. The foundation island area was predredged on the New Jersey side. Prior to construction soft sediments were re-deposited in the dredged area during high river stages. The area was not re-dredged, but excavation of the silt from the interior of the cells using a clamshell bucket may have been attempted. It is assumed that this was not effective and that soft compressible soils were left in place. This would be consistent with the need to periodically refill sinkholes at the surface. About three years after construction, divers determined that some of the sheetpiles in the connecting arc were not embedded and may not have been interlocked at all depths. At some locations where short sheetpiles should have been butt welded, the sheetpiles did not abut. Also some sheetpiles in Cell B were not embedded and appeared to be bearing on random rock outcrops.

FINDINGS OF CONDITION SURVEY

A condition survey was performed by AECOM and Lane-Robinson Associates (LRA), the diving subconsultant. Corrosion Probe, Inc. and Hydro Data, Inc. were subconsultants to LRA. The original components of the condition survey were an underwater inspection of the islands, surface observations and evaluation of the cathodic protection system. Additional investigations included hydrographic surveys of the perimeter of both islands, a land survey of the post-failure configuration of the New Jersey Island and a program of metallurgical testing of steel coupons taken from the failed cells. Surface and underwater investigations and the metallurgical report are described later. The condition survey was conducted under the continuous direction of AECOM.

Divers Survey

LRA performed an underwater inspection of the island structures and inspected the exposed surface of the sheetpiles. The principal findings of the diver's inspection included the general condition of the sheetpiles and H-piles, detection of scour, observations related to the failure of the New Jersey Island and the reason why the Pennsylvania Island was experiencing sinkholes.

General Condition of Sheetpiles

Generally, the intact sheetpiles exhibited little loss of section and were in good condition. However, a band of severely pitted steel encircles each island at the low water splash zone. The pitted band extends from approximate El. -1.0 to -4.0 ft USC&G (US Coast and Geodetic Datum). The pits are as much as 1 1/4-inches in diameter, and up to 0.420 inches deep, as measured with a pit gauge. The deeper pits can be penetrated with a sharp hammer. The interlocks in this zone are severely corroded; in some cases the outer knuckle is completely corroded.

Condition of H-Piles

The loss of fill on the New Jersey Island exposed between 1 and 2 ft of the H-Piles below the pile caps. The exposed portions appeared to be in good condition and the protective coating appeared intact, except where it was torn away by the movement of the fill. The flange of one H-pile was bent approximately 1 inch out of plane along a 4-inch section. This may have been the result of the pile driving operation that is, hard driving to rock.

Detection of Scour

The divers performed a pneumo-fathometer survey around the perimeter of the islands at the mudline. The results of this survey were consistent with the precision hydrographic survey, and indicated that about 20 ft of scour had occurred on the channel side of both islands since original borings were conducted in 1959. Divers observed several 1 to 6-ft vertical drops in the mudline profile around both foundation islands. These appeared to the divers to be at locations where current velocity increased.

Additional Observations

New Jersey Island. Additional observations that may have a bearing on the failure of the New Jersey Island are summarized below. The numbering system used to identify specific sheetpiles that are discussed in this section is shown in Fig. 3.

- The failure appears essentially symmetrical when viewed in plan. Ruptured sheetpiles in Cells A and B are interior section MP-101 sheetpiles located under the North-South grade beam. Torch-cut holes, presumably for handling, were found on several sheetpiles near the failure zone between EI. +1.0 and +2.0 ft USC&GS datum.

- Cell A has 95 exterior section MP-102 sheetpiles. Other cells on both islands have 97 exterior section MP-102 sheetpiles, as called for in the construction drawings. The number of interior sheetpiles in the cells was not determined as the tops of the sheetpiles were covered with fill.

- Sheetpile 5 of the AB connecting arc is not embedded, and is only 47 1/8 inches in length. The tip elevation of this sheetpile is at EI.-32.7 ft. Based on borings made in 1959 the mudline was at approximate EI. -30.0 ft in this area. Therefore this sheetpile had insufficient embedment at the time of construction. The divers felt two sheetpiles that were present inside the connecting arc behind the short sheetpile. However, the two sheetpiles are not connected to the cell and they would not compensate for the lack of embedment of the short sheet pile. Sheetpile 4 of Cell A has a short length of sheetpile driven in front of it.

- A large mound of clay and gravel is located in front of the AB closure arc. The mound is highest in front of sheetpiles 4 and 5, where the greatest lateral movement of the cells occurred. The divers estimated that the crest of the mound is approximately 12 ft above the adjacent scoured bottom of the riverbed.

- The sheetpiles on the New Jersey Island were to be spliced by butt welding to attain the required length. In Cell A, exterior sheetpile 95 and interior sheetpile 3N did not show evidence of ever having been welded, and there was a 1 inch gap between the two lengths comprising sheetpile 95.

- There were no weep holes in the structure to allow drainage of the cells.

- Angle plates of both connecting tees of the AB closure arc were torn (Fig. 11).

As a result of the partial collapse, the two badly damaged connecting tees which connected Cell A and Cell B were distorted and appeared to be gradually tearing apart, making the structure of doubtful value. If the two tee connections were to completely separate, the large radius arc formed by failed cells A and B would be lost, and the cell fill would be free to move into the river. It would be more difficult to salvage and repair the island. We did not want to lose what we had and there was a good chance that we could. To arrest the progressive worsening of the tee connections, our expert consultant, Mr. E. Paul Swatek, recommended that we immediately carry out the modest temporary repairs described below:

- In order to equalize the water inside the cells with the level of the river, burn weep holes in the sheetpiles of all four Cells A, B, C, and D above the water line on the more or less vertical portions of the sheetpiles. After burning of these drain holes, they should be rodded to develop a stream of water. To do this take a welding rod and churn it around in the hole. The rodding would dislodge any large round stone which might plug the hole, and develop a crude filter behind the sheetpile. This may have to be done several times before a good weep is developed.

- Weld horizontal steel straps, 4" x 1/2" at 8" centers vertically, across the tee-pile splits - all the way across both splits. Form to fit sheetpile cell radius. Weld these straps at 8 inch centers from the top of split down to low water, using a low hydrogen welding rod because of sheetpile chemistry.

Pennsylvania Island. A sinkhole located in the NE cell of the Pennsylvania Island developed prior to the preliminary site inspection. The sinkhole was adjacent to the exterior sheetpiles and semicircular in shape, approximately 13'-3" long, 7'-8" at its widest point and 8'-4" at its deepest point. The sinkhole spanned the 13th, and 23rd sheetpiles of Cell D. During the underwater inspection of the area corresponding to the surface sinkhole, the divers observed that the tips of six sheetpiles (Nos. 16, 18, 20, 22, 24 and 26) were 4 inches to 9 inches above the mudline. Active loss of fill was occurring in several openings whereas the presence of cobbles and gravel in other openings inhibited the loss of fill.

Hydrographic Surveys

Hydrographic surveys were carried out in order to investigate general river bottom con conditions in the vicinity of the two islands and to document scour near the structures. Depth soundings taken by the Corps of Engineers in 1954 and riverbed elevations from the 1959 borings were compared with 1991 soundings. Soundings were not available for the period immediately before or after construction. Based on the comparison, approximately 20 ft of scour had occurred on the channel side of both islands.

The maximum depth of scour in the vicinity of the New Jersey Island is on the channel side of Cell B. The riverbed elevation at the deepest point is at approximately El. -49.9 ft. The riverbed elevation on the channel side of the island in 1959 was approximately El. -30.0 ft. On the Pennsylvania Island the deepest scour is also on the channel side, at approximately El. -39.3 ft, compared to El. -20.5 ft, in 1954.

Evaluation of Cathodic Protection

Corrosion Probe, Inc. performed ultrasonic thickness testing of the sheetpiles and evaluated the condition of the cathodic protection system. The cathodic protection system on the New Jersey Island was not operational. An overload trip feature probably deactivated the system when the cables from the H-pile groups to the anodes in the two failed cells were ruptured due to the large displacements of the cell fill. Potential measurements indicate that all structures on the New Jersey Island, including the sheetpiles, H-piles, pile caps and electrical grounds were electrically continuous. This appears to corroborate calculations and oral recollections which indicated that the batter H-piles on that island intersected the sheetpile cells, as shown on Fig. 2. The cathodic protection system on the Pennsylvania Island was operational and in service.

Metallurgical Analysis and Failure Mechanism

Steel coupons from the failed cells were torch cut from sheetpiles in the failure zone and analyzed in PECo's Metallurgy Laboratory. The principal conclusions of the metallurgical analysis regarding the failure mechanism are paraphrased below:

- Fracture of the interlock of the ruptured panel in Cell B started beneath the soil surface and progressed upward in a fast, brittle manner. This was evidenced by the characteristic herringbone failure pattern observed on the rupture surface.

- Fracture of the interlock of the ruptured panel in Cell A initiated approximately 78 inches below the top of the sheetpile and progressed upward in a manner similar to the crack in Cell B.

- The web of the sheetpile adjoining ruptured panel had a large crack. This section exhibited a considerable amount of deformation due to twisting and tearing. The initiation site of this crack was below the soil level and therefore not identified.

- All of the cracks that were observed, including a small axial tear emanating from the bottom edge of a lifting hole, indicate a high tensile hoop stress being applied to the damaged cells.

- The brittle nature of the cracks in the interlocks indicates either a high energy induced mode and/or cracking occurring at a temperature below the ductile-to-brittle transition temperature.

- There was evidence that corrosion damage had degraded the sheetpiles sufficiently to influence the observed fractures.

- Material analysis included spectrochemical analysis, tensile testing, impact testing, hardness testing and metallography. The samples were determined to be carbon steel and their mechanical properties were consistent with that expected for this material.

Survey Monitoring

PECO personnel surveyed the New Jersey Island to record the present configuration of the island. A plan of the New Jersey Island before and after failure based on the survey data is presented in Fig. 3.

Soils Laboratory Analyses

Sieve analyses were performed on samples retrieved from the upper 8 ft of the failure surface. Below this elevation, debris from the slide covers the existing soil profile. The sieve analyses confirm visual observations that the upper 4½ ft of soil is granular material composed of gravelly sands, silty sands, and sand and gravel. The granular material is underlain by a 1 foot thick layer of silty clay. The underlying fill is gravel and sand. It is not known whether the soil samples are from fill that was placed at the time of construction or from fill that was added later. Atterberg limits were performed on riverbed samples. The New Jersey sample was obtained from the mound of soil in front of the failed cells. This sample is highly plastic clay, with a liquid limit of 103% and plasticity index of 67%. The Pennsylvania sample was obtained from the channel side of the riverbed. This sample is also highly plastic clay, with a liquid limit of 99% and a plasticity index of 62%.

Analysis of Water Sample

A water sample obtained from within the failed area was analyzed by ion chromatography and atomic absorption spectroscopy in PECO's Chemistry Branch. The pH of the sample was 6.39, and chlorides were 39.3 ppm. This indicated that the river environment is not chemically aggressive, as evidenced by the good condition of the sheetpiles outside of the low water splash zone.

FAILURE ANALYSIS

In this section a hypothesis is developed which attempts to explain the probable cause(s) of failure of the New Jersey Island and describes the sequence of events that led to the structural collapse of Cells A and B.

Failure Hypothesis

The failure hypothesis was developed from findings of site visits, condition surveys, metallurgical testing, engineering analysis and reasonable engineering judgment. A number of contributing factors suggest that a progression of events occurred over time, which eventually culminated in the failure of two of the four circular cells comprising the New Jersey Island. Conclusions regarding the initiation of fracture at the toe or lower reach of the sheetpiling are necessarily inferred, because the failed condition of the structure below the existing mudline is not observable without the expense and risk of extracting the sheetpiles.

Contributing Factors and Sequence of Events

The following paragraphs describe the several factors that contributed to the failure. The role of each of these factors in leading to the eventual failure mechanism constitutes a failure hypothesis. The inferred sequence of events leading to failure is depicted in Fig. 8.

1. Scour and Loss of Sheetpile Embedment. First, there was scour, probably over a long term. Scour is the process of soil erosion in which soil particles are lifted, moved and transported by the force of flowing water. Scour can be a gradual process or it can occur rapidly, depending on the velocity of water flow and the type and properties of the soil being eroded. Washing out or undermining of pier foundations due to scour of riverbeds is a common cause of bridge failures (Jumikis, 1971).

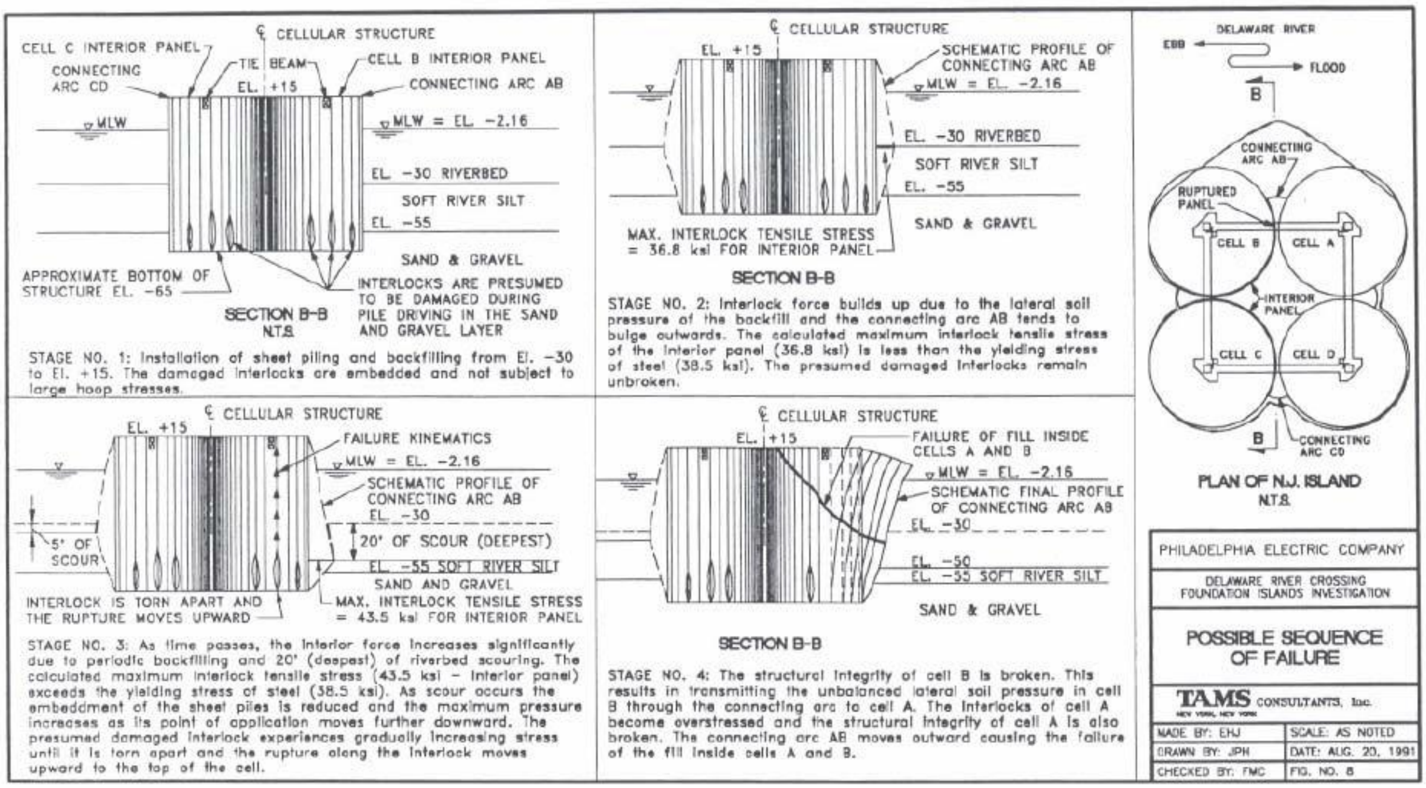

Fig. 8: Possible Sequence of Failure

As mentioned, a hydrographic survey was conducted around both islands as part of the condition survey. The hydrographic survey disclosed substantial depth of scour around the channel side of the New Jersey Island. Scour reached a depth 20 ft in the vicinity of Cell B. Near Cell A the maximum depth of scour reached about 13 ft. However, the scoured depth before failure may have been even greater than these values. The divers observed a mound of displaced river bottom soils in front of the failed, sloping sheeting, a consequence of soil displacement when the cells underwent a large movement toward the channel. The eroded depth prior to failure was obscured by the material displaced by the movement of the cellular structure. Nevertheless, the depth of scour reported in the hydrographic survey was substantial, and is believed to be the initiating factor of the failure. Scour deepened the water along the channel side of Cell B which lowered the elevation of passive resistance on the outside of the cell and increased the net internal pressure, adding a significant amount of hoop tension at the interlocks. With the loss of sheet pile embedment due to scour there is an increase in internal earth pressure and a corresponding increase in circumferential tensile stress in the webs andat the interlocks of the sheetpiles. Without occurrence of scour the failure would not likely have occurred, in spite of any damage to the sheetpiles that might have occurred during construction.

2. Damage of Sheetpiles during Installation. Driving records for the installation of the sheetpiling were not available. Therefore, the actual driving resistance encountered is not known. However, in light of findings of other projects with similar subsurface conditions, it is reasonable to assume that the sheetpiles suffered some degree of damage during installation.

The soil profile in Fig. 4 shows that the sheetpiles were driven through the soft riverbed silt into an underlying sand and gravel layer. This layer is medium dense to dense and could have caused substantial resistance to penetration of the driven sheetpiles. In overcoming this resistance the sheetpiles, being long flexible members, were vulnerable to damage by tearing of the webs or deformation of the interlocks.

The vulnerability of sheetpiles to damage during driving is supported by Jahren (1990) as follows: "For cell structures, many failures are due to construction problems, such as rough driving that damages interlocks of sheet piles", by Bowles (1968): "To achieve a cell which is stable against bursting it is necessary that the sheetpiling be driven so that continuity of the interlocks is maintained. Small stones in the driving zone may wedge in the interlock so that the interlock joint can be damaged or the adjacent [sheet] pile may be driven out of position", and by Koerner (1984): "splitting of the web during driving is not uncommon, particularly when obstructions or dense granular soils are being penetrated." Initially, sheetpiles and interlocks which may have been damaged at or near the toe during installation were well embedded below the river bottom. Under this condition the internal earth pressure and resulting interlock stresses were small and of no consequence to the behavior of the cellular structure. However, with the progression of scouring of the riverbed, the embedment of damaged sheetpiles and interlocks was gradually reduced and the circumferential tensile stresses increased accordingly.

3. Soft Sediments and Periodic Backfilling of Cells. As mentioned under oral recollections, during the original construction attempts were made to dredge the soft river bottom sediments, but were apparently abandoned when new deposits occurred following a spring freshette. Hence, it is believed that the cellular structure was constructed through the soft sediments and it appears that attempts to excavate the soft silty sediments from within the cells was not successful, prior to filling the cells with granular backfill. The presence of the soft clay in the cell had the following undesirable effects on the structure:

- Soft clays trapped at riverbed elevation and deep in Cell B added to hoop tension. The intention was to remove these soft plastic river bottom sediments before placing the cell fill. The considered opinion is that they were not. They are revealed in borings taken after the collapse. The excess pressures transmitted by these trapped plastic clays in Cell B produced interlock values in the neighborhood of 16 kips per inch, which is at or above the ultimate value of the M-l0l interlock.

- Long term consolidation under the weight of the backfill led to compression, subsidence and surface settlement. This is consistent with the history of sinkholes and depressions that have required backfilling to bring the ground level in the cells back to design grade. Of course, each time backfill is placed to correct the depressions, the added weight induces still more settlement and the need for subsequent additional backfill, which increases the stress even further. The periodic backfilling increased the overburden pressures in the cell and likewise the internal lateral pressure exerted outwardly on the sheetpiles. This in turn increases the circumferential hoop stresses.

- The location of the soft soils is down low in the cells and coincident with the zone where the maximum pressure arises. The soft sediments having a low shear strength ($\varphi' = 20^{\circ}$), even after some improvement due to the long term effect of consolidation, gives rise to a relatively high coefficient of lateral earth pressure ($K = 0.8$) such that 80% of the vertical overburden pressure is exerted laterally on the walls. See Fig. 7. Had the sediments been replaced with a granular material the lateral pressure and corresponding hoop stresses would be reduced, typically to around 50 to 60 % of the vertical stress.

- Sheetpile steel has a low Charpy impact resistance, especially at lowered temperatures. At some time during the previous winter or winters a defect such as noted cracks at a pulling hole or a notch at the top of a burned off sheetpile could enlarge, and lengthen the crack. This would create a stress raiser for hoop tension stresses.

- The failed sheetpile was at an interior location in cell B near a tee. The interlock and web stresses in tees and the sheets adjacent thereto have an increased indeterminate stress from the connecting arc in addition to the other sheets in the cell.

4. Differential Water Pressure. The ground surface of each foundation island is exposed to climatological elements. Neither island is paved and during a heavy storm they readily admit rainwater, which could result in full saturation of the backfill. Because the cell is relatively watertight, and the backfill material is not entirely free - draining, as disclosed by the gradation curves from the grain size sieve analyses, rainwater could accumulate until the cells are completely saturated. There were no weep holes in the outside sheetpiles above water. Encrustation over time made a more or less watertight vessel of the cell. High storm tides with waves overtopping the sheetpiles would fill the cell. During the tidal cycle at low tide, approximately 800 psf would be added to the internal pressure, increasing interlock tension by yet another significant amount.

Full saturation would be more likely if the storm occurred during high tide. Because of tidal lag, the differential head between the saturated ground line inside the cells. (El. 15) and the mean low water of the river outside the structure (El. -2.16) would be about 17 ft or a differential water pressure of about 1,060 psf. This hydrostatic pressure could cause almost 3 kips per inch circumferential tensile stress within the cells below the water level of the river.

5. Increase of Internal Earth Pressure and Hoop Tensile Stress. As scour proceeded, the embedment of the cell was reduced on the channel side. This reduction in embedment served to increase the unsupported height of the structure, which then resulted in greater lateral earth pressure acting on the sheeting. Furthermore, this earth pressure is related to the square of the unsupported height. Not only would the pressure increase but with the loss of embedment the location of the maximum earth pressure descended to a point lower in the cell, nearer to the locations where possible driving damage to the sheeting existed.

6. Excessive Circumferential Stress and Interlock Failures. As a result of the above mentioned contributing

circumstances, the circumferential tensile stresses increased to a level sufficient to overcome the strength of the interior sheetpiles in Cells A and B. A recent reliability study of sheetpile cellular structures found that bursting is the most likely failure mode for cell structures that are designed according to the present state of the art (Jahren, 1990).

The manufacturer's guaranteed ultimate strength of the interlocks for the sheeting used in this structure is 16 kips per inch. At this value the interlocks are expected to overcome the contact friction, letting the joined sheetpiles separate from each other. The recommended allowable force is 8 or 9 kips per inch, maximum (Lacroix, Esrig and Lusher, 1970). A stress analysis of Cell B shown in Fig. 9 was conducted to estimate the maximum hoop stress in the cell, which occurs on the interior sheets at or near the connecting arc (TVA, 1957), the depth to the maximum stress was assumed to correspond to the depth of scour outside Cell B. This is based on Maitland and Schroeder (1979) who recommend a plane of fixity concept to estimate the location of maximum lateral earth pressure. The weaker the soil in which a cell is embedded, the greater the depth to the plane of fixity. Therefore, the maximum interlock tension should be calculated at a lower level for weak soils compared to strong soils, and may even occur at or below the dredgeline.

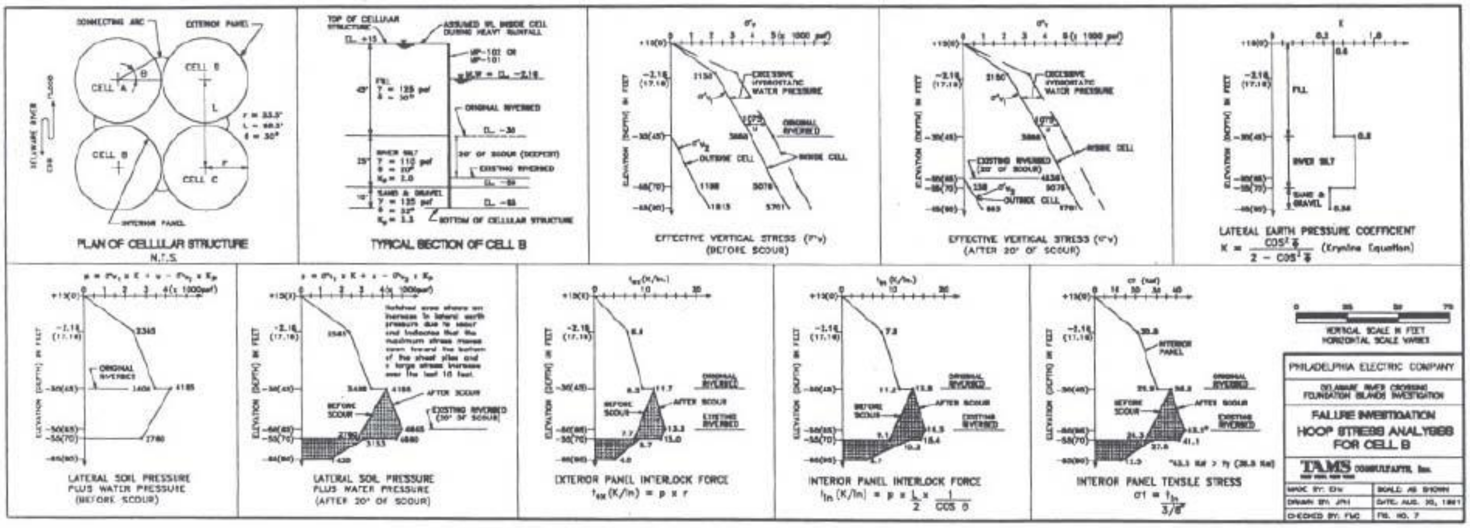

Fig. 9: Hoop Stress Analysis of Cell B

The analysis arrived at an estimated circumferential tension of 16.3 kips per inch under the conditions of scour, saturation of the backfill above MLW, and the presence of soft sediments inside the cell. This value exceeds the allowable force (8 kips per inch) as well as the ultimate interlock strength of 16 kips per inch. The corresponding stress is 43.5 ksi, which exceeds the yield point of 38.5 ksi. The analysis demonstrates that the tension was sufficiently high to exceed the ultimate strength for separation of interlocks and the yield point of the webs on the interior sheetpiles.

The cells were constructed in 1959 using MP-101 sheetpiles (web thickness = 3/8 inch) for the interior members and MP-102 (web thickness = 1/2 inch) for the exterior members of the cells. This may have been done to afford the exterior cells a longer life against corrosion. In contrast, TVA identified the interior sheeting of cells as being more highly stressed than the exterior as a result of the added pulling effect on the main cells by the connecting arcs. However, the TVA document containing this information was published in December 1957 and may not have been widely disseminated by 1959. Had the larger wall thickness also been employed on the interior members, perhaps the failure would not have occurred, although continued scour, if not discovered, would have increased tension stresses in the future, possibly sufficient to cause failure. Although the computed values are based on the maximum depth of scour (about 20 ft), and therefore may be a slight overestimate, it should be clear that the interior of the cells, and to a lesser degree the connecting arcs and exterior members, were experiencing a condition of substantial distress, sufficient to initiate shear tearing of preexisting damaged webs and/or brittle fracture of the interlocks. This was consistent with the physical evidence, above the mudline.

The major principal stress in a cell is circumferential tension (or hoop stress) and tends to pull the interlocks apart. However, cellular structures frequently exhibit a non-ductile or brittle mode of failure at stresses far below the yield strength. Brittle fractures are usually associated with flaws (i.e. damaged sheetpiles, torch-cut hole, etc.), are often sudden, and usually occur without warning. The absence of gross plastic deformation distinguishes brittle fractures that occur below the yield point from ductile failure. Low temperatures can cause a normally ductile material to behave in a brittle manner. Since the failure was noted sometime between December, 1990 and April 1991, low temperature could have been another influencing factor in initiating a brittle mode of failure.

In the majority of interlock failures the contact friction between the interlocking fingers and thumbs holding adjacent sheetpiles together is overcome by circumferential stress and separate entirely one sheetpile from the other. However, this mode of failure was not evident at the New Jersey Island. Instead the failure mechanism consisted of the steel fracturing vertically through the narrowest dimension (root) of an interlocking thumb, leaving it behind and inside the thumb and finger of the other interlock. That is, the steel fractured before the interlocks could separate (Fig. 10).

Fig. 10: Brittle Fracture through Interlock

The fracture occurred along a vertical plane. Chevron or herringbone markings pointed downward, an indication that the fracture began below the mudline and proceeded upward unabated.

Summary of Failure Mode

Considering the high calculated stresses, the potential for damage to the webs (tearing) and interlocks during installation, as well as other random local stress raisers observed by the divers, such as torch-cut holes and perhaps unseen welds below ground, the brittle fracture failure mechanism at or below yield point is a consequence compatible with the several contributing factors and the interactive scenario described herein.

With the maximum depth of scour in front of Cell B, a large pulling force was exerted on the connecting arc AB, which in turn pulled on the interior sheetpiles until Cell B was breached in its lower reaches. The lack of redundancy in this type of structure permitted the crack to propagate upward, as evidenced by the downward pointing chevron pattern on the exposed fracture surface. As Cell B was then free to lurch toward the channel, it pulled on Cell A through connecting arc AB. This additional stress, added to an already severely stressed Cell A (13 ft of scour), was sufficient to initiate a fracture of Cell A several feet below the top of the sheeting, which at this location had already been cut down several feet below grade to permit construction of the tie beam.

As the two cells failed behind the connecting arc they lurched outward trying to individually open up, but were restrained by their mutual connecting arc, as evidenced by the distortion and slight pulling apart of the T-connection from the top of the sheeting to a point several feet down (Fig. 11). This latter damage is a result of the collapse, not the cause.

Fig. 11: Connecting Arc A-B in Distress

Other possible causes of failure such as bearing capacity, corrosion and vessel impact were considered, but were ruled out. The origin of the split was not examined because it was located at a deep elevation in the cell and could not be easily recovered. Therefore, consultant Paul Swatek indicated the possibility that a ductile necked down section of interlocking thumbs might have initiated failure, say due to steel with slag inclusions, at a point of high tensile stress. But having said that, he reported that he had seen and heard of many brittle fractures of sheetpiles, including webs, fingers, and thumbs. The steel in sheetpiles is subject to brittle fracture at reduced stress and this failure was one of that kind. The following is an excerpt from E.P. Swatek's report (1991):

> *There was no single event or blow which caused the failure, rather it was the accumulation of scour, resulting overloads and weaknesses which finally produced the failure. It has been shown that the interlock stress was in the order of 16 kips per inch, near or above the ultimate strength of the interlock and/or the web. Although sheetpile interlocks are tested and guaranteed to a value of 16 kips per inch, the guarantee could be meaningless. [In E.P. Swatek's experience he had knowledge of tests on some sheetpiles delivered from the mills that found values well below the guarantee.] That is why a factor of safety of two is used, resulting in an allowable design stress of 8 kips per inch.*

Two interlocks failed. The first was an interior M-10l near a tee connector in cell B. The failure was a classic brittle fracture which left telltale chevrons in the fracture surface, indicating it progressed upward from a point of origin deep in the cell. This is consistent with the suspected high failure tensile stress alluded to.

The second failure was in cell A in a position more or less symmetrical to that in cell B. The second fracture was not a clean brittle cleavage as in the first, but showed signs of distortion & tearing. This suggests the following scenario. The first failure in cell B resulted in movement and distortion of the fragments of cell B. Collapse of Cell B was rapid and put an overload in cell A sufficient to cause the second rupture. That this second rupture showed tearing and distortion of Cell A sheetpiles places these events subsequent to the failure in Cell B. The first failure (Cell B) was rather explosive and instantaneous. The second failure (Cell A) must have followed shortly thereafter.

There is another weakness in the sheetpile cells that is seldom given attention in the design. This is the web stress in the net area of the row of rivet holes of the tee connector. Assuming an interlock stress of 15,700 kips per inch at the time of failure, the stress in the net area of the web of the tee connector would be approximately 60,000 psi. This elevated stress plus the flexure stress in the web of the tee connector are reason enough to suggest that it is not inconceivable to imagine the origin of failure in the tee connector with the split crossing over several sheets to the observed above-water failure location.

ALTERNATIVE METHODS OF REHABILITATION

Several concepts were explored for restoring the foundation islands to a safe long-term condition. The restoration plan included remediation of the failed New Jersey Island, the Pennsylvania Island sinkhole, and protection of the severely pitted zone around both islands. Placing a large diameter cell or steel structure encircling the entire island was not practical from design or economy. Other alternatives were deemed not viable. For the sake of brevity, only the recommended alternatives will be discussed.

New Jersey Island

The failed area had to be backfilled because of the hazard it presented to public safety. This island is located in a public waterway and was accessible to the public from two stationary ladders. Members of the public could climb onto the island, and through their own actions, injure themselves for example, by falling from the exposed grade beam. Unless the structure was stabilized the possibility of movements of the cells could also present a safety hazard to boaters who might be near the structure at the time of additional failure. The integrity of the structure could be jeopardized since the cathodic protection system was no longer providing corrosion protection to the H-piles supporting the tower, and these piles were now partially exposed to the atmosphere. Future fill movements could damage the coating on the piles and expose a longer portion of the H-piles to the atmosphere.

Since Cells A and B were no longer closed systems, much of the tension in the sheetpiles along the failed portion was taken by soil friction in the intact portion of the cells and possibly through the connecting arcs of cells C and D. Backfilling would create additional instability which could lead to additional fill movements in cells A and B and possibly progressive failure of all cells.

Therefore, it was agreed that the sinkhole in the failed area would be backfilled with lightweight granular material, the cathodic protection system would be repaired, and a buttress of crushed stone armored with riprap would be placed around the island to stabilize the failed structure in-place and provide a protection blanket against scour. These measures would prevent further movement and potential loss of the fill into the Delaware River.

Use of lightweight fill would minimize lateral loads on the buttress. A buttress of considerable width might be required. However, the structure is located outside of the navigation channel defined by the Corps of Engineers, and did not have an impact on the navigation channel.

This alternative is relatively simple to construct because it does not require a structural connection to be made between the existing cells and the rock buttress. This alternative would be built from the water and the existing structure would not interfere with construction.

Periodically, bathymetric surveys would be performed to detect scour and the cathodic protection system would be tested.

Initially, it was thought that the soft sediments in the existing riverbed would not provide an adequate foundation for the buttress and would have to be removed by dredging to allow the buttress to rest on underlying bedrock. But dredging could result in destabilizing the state of quasi-equilibrium the cells had reached. Dredging would also require disposal of the sediments. Alternatively, cyclopean riprap was considered in order to displace the soft soil, but it was rejected because it was questionable whether the large rocks could actually displace the soft sediments.

Fortunately, in-house laboratory testing for soil shear strength and compressibility indicated that the soft sediments were somewhat more favorable than previously thought. Slope stability studies using the laboratory strength values confirmed that in lieu of excavating the varved clay to bedrock, a large perimeter buttress could safely rest on the varved clays. This design modification resulted in a $3 million savings and reduced permitting and construction time.

To avoid additional differential surcharging and displacement of the island backfill outwardly against the circular ring of sheetpiles, placing the fill would be sequenced such that the elevation of the buttress was always a few feet higher than the backfill placed inside the failed cells. Once the buttress was fully in-place, the backfill could be topped - out.

In placing the buttress, care had to be taken to avoid increasing the surcharge and hoop tension in the failed island sheets that had been repaired with the steel straps. The straps could take some hoop load but were considered only a stop-gap measure - an attempt to maintain what we had. In no way were the straps a permanent solution. The full and permanent confinement/stability of the island awaited placement of the buttress.

Thus, a rock buttress was recommended to stabilize the failure by placing a crushed stone buttress around the failed island to prevent further movement and potential loss of the fill into the Delaware River. The overburden would be removed to allow the buttress to rest on underlying bedrock. However, in-house laboratory testing for soil strength and compressibility followed by slope stability studies confirmed that in lieu of excavating the silt/clay to bedrock, the large perimeter buttress could safely rest on the silt/clay. This is to be preferred over dredging out the clays and disposing of a large volume of spoil.

Paul Swatek recommended that the minimum berm around the undamaged side of the New Jersey Island be 15 ft thick and 30 ft wide at the top, and be of the same crushed rock and riprap protection. Thus, with the exception for a small inlet for a boat to access the island, there would be a buttress completely around the New Jersey Island.

However, the toe of the buttress at the level of the riverbed would pass very closely to the buried location of two 20-inch and one 6-inch diameter natural gas pipelines. Finite element analysis indicated a potential for vertical and horizontal movements of about 1 to 2 inches. It was expected that the flexibility of the pipelines would tolerate the predicted movements. Nevertheless, a geotechnical instrumentation program, comprised of settlement plates, inclinometers and piezometers, was implemented to monitor vertical and lateral movements and subsurface pore pressures in the vicinity of the proposed toe of slope, in order to determine whether the actual movements of the pipelines would exceed the predicted values. And they did not.

<u>Pennsylvania Island</u>

A sinkhole on the Pennsylvania Island was actively losing fill. Additional loss of fill could cause a redistribution of stresses in the affected sheetpile cell and lead to instability of the cell. Accordingly, temporary remedial work was performed in this area. A small berm comprised of grout-filled burlap bags was constructed underwater by LRA. Steel plates (1/4 inch thick, 14 inches wide and 20 inches long) were inserted in front of each of the six sheetpiles that were not embedded. Grout-filled burlap bags were placed in an interlocking and overlapping pattern to form the berm. About 2 cubic yards of grout were placed. Sharpened 18- inch long, No. 4 reinforcing bars were driven through the layered bags (30 bags total) to pin them down. This grout berm is considered a temporary solution which is subject to scour.

The sinkhole would be filled with 3" to 6" stone. This way, if future scour ever exposes the tips of the unembedded sheets, the stone will tend to choke off further loss. A berm against this cell at the sinkhole can be placed 10 ft thick with a top that is 15 ft wide out from the cell. This berm should be of coarse stone, protected with riprap.

Because scour had played a major role in the failure of the New Jersey Island, and the condition survey indicated that approximately 20 ft of scour occurred on the channel side of both islands, it would be prudent to provide permanent scour protection to the Pennsylvania Island to maintain adequate embedment of the sheetpiles. The long term performance of the structure could be jeopardized unless repairs were made.

Therefore, a buttress of crushed stone armored with riprap would be placed locally in the vicinity of the sinkhole to stabilize the structure and provide protection against scour. A riprap scour protection blanket would be placed and the sinkhole would be backfilled. Periodic bathymetric surveys would be performed to detect scour. This alternative is relatively simple to construct because the sinkhole is still localized and the cell walls are still vertical. The buttress did not impact navigation because the Pennsylvania Island is well outside of the navigation channel. Construction of scour protection would lower the risk of a cell failure similar to that experienced on the New Jersey Island. Construction, if done concurrently with the restoration of the New Jersey Island, would involve lower mobilization costs than if it is performed separately.

Based on the above evaluations a large island-encompassing rock buttress was recommended for the New Jersey Island (Fig. 12) and a small localized rock buttress for the Pennsylvania Island. The buttress is shown under construction in Figs. 13, 14 and 15.

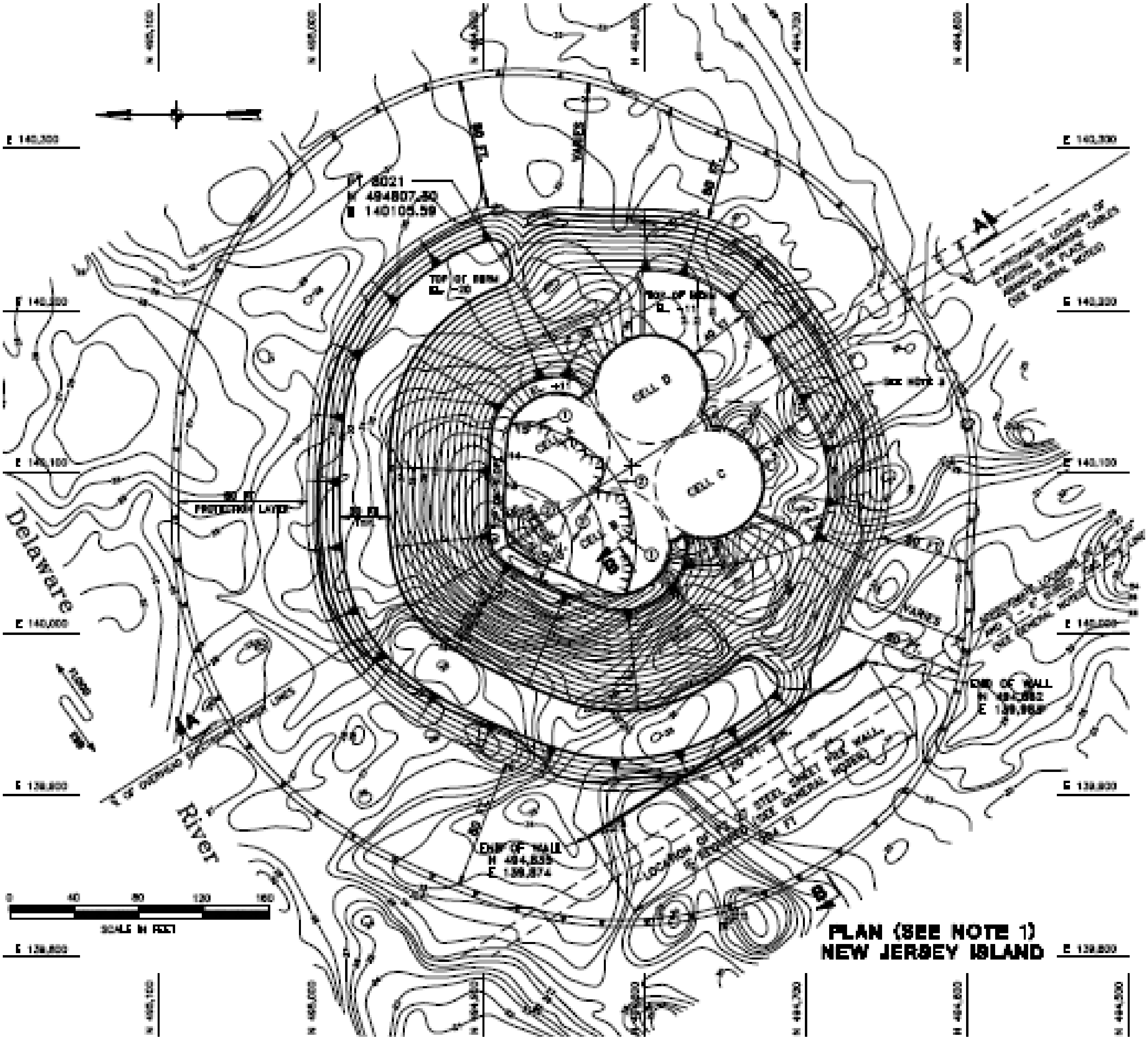

Fig. 12: Proposed Buttress of Crushed Stone and Riprap

It is likely that storm tides also inundated the Pennsylvania Island, causing internal water head and an increase in the interlock stress. Interlock stress computations revealed that without drainage the interlock stress was a possible 14.5 kips per inch, with a factor of safety of 1.1, whereas with effective drainage these figures come down to 10.5 kips per inch wth a factor of safety of 1.5. We try for a factor of safety of 2.0. Also, the stress in the net area along the row of rivet holes at 3 inch centers in the main sheetpile web of the tees on the Pennsylvania Island reduced from a possible 54,600 psi to 38,600 psi, still a high figure. In figuring maximum interlock values of the repaired island it is recommended that the envelope of the maximum 10 ft of the values be averaged over the 10 ft to recognize redistribution of stresses.

Fig. 13: Buttress Topping-Out

Fig. 14: Buttress of Crushed Stone, Riprap and Geotextile

Fig. 15: Failed Area with Lightweight Backfill, Geotextile, Crushed Stone and Riprap

<u>Repair of the Severely Pitted Tidal Splash Zone</u>

For both islands, the initial reaction to seeing the severely pitted steel sheetpiles in the splash zone was to restore the full sheetpile section and/or protect the area from further corrosion. Measures were considered to restore the steel sections to full structural thickness, such as encasing the pitted zone with welded steel plates or by forming a 4 ft concrete belt doweled into the existing cells. Divers attempted to apply protective coatings to create a moisture barrier to inhibit corrosion of the sheetpiles, but found this very difficult to achieve. However, calculations indicated an internal lateral pressure of only 900 psf at this elevation. Assuming a loss of one-third of the 3/8" thickness of the web of the sheetpile due to pitting, the web stress computed to only 13,300 psi, sufficiently low that it was agreed that the concrete ring need not be implemented. It was decided to specify a 4-foot epoxy coating around the tidal zone. Even if the steel in the tidal zone was eventually penetrated, the buttress would be there to confine it.

MONITORING

Since the repairs to the New Jersey Island were carried out effectively, given periodic inspection and maintenance, the island is expected to be stable for a long time.

At the Pennsylvania Island, the original 60-foot length of sheetpiles was set too short. Scour on both islands over the years was about the same, namely 20 ft. The sinkholes at the Pennsylvania Island indicated that scour was undermining the sheetpile perimeter of the island. Future scour may attack at some point other than where the repairs were made. Therefore, annual scour surveys should be conducted to monitor scour. If scour increases, repairs may have to be undertaken in the form of a low permanent berm around the island. New sinkholes would also warn of scour.

Drainage through the weep holes should be kept permanent. It would be well to observe this drainage and make sure it is maintained. Periodic rodding of the holes may be required. Drainage is a necessity for both islands to keep interlock stresses within reason.

CLOSURE

The collapse of Cells A and B of the New Jersey Island due to scour was not a unique occurrence. Scour is a common geo-hazard around structures in waterways. Even well planned scour protection may be subject to undermining. The occurrence of scour may be silent, and requires proactive diligent monitoring, including periodic underwater inspection and hydrographic measurements.

The findings of this investigation have implications for the successful performance of other cellular structures. It is recommend that this case-history be disseminated and that a program of inspection, assessment, rating, maintenance, and effective early repair of the condition of similar structures subject to scour be established for the purpose of preventing future failures.

The following quotation was a warning from White and Prentis (1950), which is as true today as it was then, ten years before these Delaware River transmission structures were constructed, and makes a fitting end to this paper:

> *An inherent weakness of the cellular type of cofferdam is that if even one pile or interlock fails the cell is lost.*

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REFERENCES

Bowles, J.E. [1968]. *Foundation Analysis and Design*, McGraw-Hill Book Co., Inc., New York, NY.

Clemente, Jr., F.M. and Fox, I. [August, 1991]. Consultant Report to PECO: *Probable Causes(s) of Failure and Conceptual Restoration Schemes, Engineering and Inspection Services, Foundation Islands, Delaware River Crossing Structures*, TAMS Consultants, Inc., New York, NY, pp. 1-53. Not published.

Jahren, C.T. [1990]. "Reliability Comparison for Sheet-pile Cellular Structures", ASCE, Journal of Performance of Constructed Facilities, Vol. 4, No. 4, pp. 216-235.

Jumikis, A.R. [1971]. *Foundation Engineering*, International Textbook Company, pp. 702-703.

Koerner, R.E. [1984]. *Construction and Geotechnical Methods in Foundation Engineering,* McGraw-Hill Book Company, Inc., ISBN: 0070352453

Lacroix, Y. et al.[1970]. "Design, Construction, and Performance of Cellular Cofferdams". Specialty Conference Proceedings, "Lateral Stresses in the Ground and Earth Retaining Structures", Cornell University, Ithaca, NY, ASCE, JSMFD, pp. 271-328.

Maitland, J.K. and Schroeder, W.L. [1979]. "Model Study of Circular Sheetpile Cells". ASCE, Journal of the Geotechnical Engineering Division, Vol. 105, No.7, pp. 805-821.

Swatek, Jr., E.P. [1970]. "Summary - Cellular Structure Design and Installation". *Proceedings of the Conference on Design and Installation of Pile Foundations and Cellular Structures*, Lehigh University, Bethlehem, Pennsylvania, Envo Publishing Co., Inc., Lehigh Valley, Pennsylvania, pp. 413-423.

Swatek, Jr., E.P. [1992]. Consultant's Report: "Failure of Protection Cells", pp. 1-23. Not published.

White, L., and Prentis, E. A. [1950]. *Cofferdams*, Columbia University Press, New York, NY.