C. H. GOULD
1165 Bardstown Trail
Ann Arbor, MI 48105
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020076

March 17, 1980

S.S. Afifi
Bechtel Professional Corp.
P.O. Box 1000
Ann Arbor, MI 48106

Dear Mr. Afifi:

In accordance with your request I have prepared and enclosed a synopsis of the presentation I made before Consumer's Power Company and the Nuclear Regulatory Commission representatives at Midland Michigan on February 28, 1980.

The material covered in the presentation deals with remedial measures for underpinning the Electrical Penetration Wings of the Control Building and the Isolation Valve Pits. The information conveyed to the attendee's of the meeting conforms to the content of the technical portions of Bechtel Associates Professional Corporation specification 7220-C-95(Q), Rev. 0, except with regard to dewatering and the structural tie-in between the wings and the isolation valve pit support.

The information presented on dewatering is based upon plans developed at a consultants meeting on October 30, 1979. According to that plan dewatering of the underpinning area will be initiated using eductors which have been installed in the turbine building; however, it is anticipated that some dewatering from inside the underpinning work space will probably be required. The information presented on the structural tie-in is developed from an analysis by Bechtel of the N-S seismic forces acting on the Electrical Penetration Wings when they are vertically supported on caissons. According to that analysis it will be necessary to use the mass concrete fill underpinning for the isolation valve pits to provide the stabilizing resistance.

Very truly yours,

C.H. Gould

CHG:jn
Enc.

REMEDIAL MEASURES FOR
THE ELECTRICAL PENETRATION WINGS OF THE
CONTROL BUILDING AND THE ISOLATION VALVE PITS
MIDLAND NUCLEAR POWER STATION, MIDLAND, MICHIGAN 7020673

This is a synopsis of the remedial measures which I recommend to be employed at the subject structures.

The subject structures have been constructed on top of approximately 30 feet of heterogenous soil fill overlying undisturbed glacial till.

Settlement of other similarly placed fill material at another location on the project prompted an investigation of the soil underlying the subject structures.

The investigation was performed by soil sampling and Standard Penetration Testing. An analysis of the resultant data indicates there are soil zones under the subject structures which have a questionable bearing capacity.

There has been no settlement or structural distress of the subject structures noted since their completion. The isolation valve pits are so rigid it is doubtful they would exhibit structural distress if settlement were to occur, but settlement could distress entering and exiting utility services. On the other hand, the wings which are integral with a central structure (Control Tower) are more structurally sensitive to settlement. Calculations show that the structural connection between the wings and control tower would crack if all the soil support were removed from under the wings. Other similar analyses would lead me to conclude that since there is no post-construction structural distress in the area of the Wing-Control Tower juncture the soils underlying the wings provide a minimum average bearing capacity of greater than 3 to 3.5 KSF.

The most positive remedy to the problem of questionable soil bearing capacity is to either remove the material and replace it with concrete or to structurally bypass the questionable fill with caissons founded in glacial till.

The decision as to which method to employ was predicated on a feasible construction procedure, the need to provide resistance for horizontal seismic forces, and probable cost. In this case, the use of caissons is the more feasible construction procedure because there is no wholesale removal of existing support and no need for temporary support. In addition, caissons require approximately one-third the bearing area compared to the soil removal procedure since caisson bearing capacity is determined by direct testing. Thusly, I would recommend caissons under the heavier structure (the wings at 8000 kips each), because of the relatively larger area of existing foundation support there is no need for temporary support and the area required to do the work is much smaller since the bearing capacity of the caissons are three times greater than the soil replacement method.

Conversely, the logical choice for permanently supporting the lighter (2000 kips each) valve pits is mass replacement of the questionable material with concrete while temporarily supporting the lighter structure, <u>given</u> the requirement for a concrete monolith sufficient to resist N-S horizontal seismic loads.

The amount of caisson capacity required for supporting the wings is based on the wing load being shared equally between caissons installed at the free and of the wing and the existing control tower foundation at the fixed end. The control tower foundation has already been preloaded in the amount necessary to support the wings when the wings are founded on soils of questionable value.

The plan of attack for performing the remdial work is as follows: 820673

1. Temporarily support the isolation valve pit by the use of needle beams spanning between the buttress access shaft and turbine building foundation wall at the ground surface.

2. Locally dewater the soils underlying the areas to be underpinned. The dewatering is to be performed in two stages.

   The first stage dewatering would be installed from inside the turbine building and from accessible areas outside the structures. The objective of stage 1 is the lowering of ground water to a minimum depth of ten feet below the bottom of the subject foundations. Excavation in the dry would then proceed to a maximum depth of 7 feet below the existing foundation.

   The second stage dewatering would then be installed from the excavated area under the foundation. The objective of the second stage is to dewater the fill to a depth that glacial till is encountered. The dewatering wells shall be packed to prevent piping and the discharge shall be monitored for fines.

3. At the completion of stage 1 dewatering. Excavate an access shaft adjacent to the isolation valve pits to a depth of approximately 7 feet below the bottom of these pits. The excavation would then proceed laterally as a drift until the excavation reaches the extreme edge of the electrical penetration area.

4. Install jacked caissons at this location utilizing the electrical penetration area foundation as the reaction. The jacked caisson method has been selected for the following reasons:

   a. It will be possible to jack through loose sands and soft clays without excavating material from within the caisson thus preventing loss of ground from under the electrical penetration area, turbine building and buttress access shaft.

   b. It is known that sizeable, but unreinforced, concrete mats were placed in the fill zone during construction. (The mats were used to support construction equipment.) It is anticipated that some of the caissons shall encounter the concrete. The caisson provides man-size working room to facilitate rapid demolition of the concrete.

   c. Likewise, the man-size working room of the caisson will permit direct excavation of highly compacted sands and/or clay as well as the glacial till (caissons penetrate the glacial till a minimum of 4 feet).

   d. The caisson provides access for direct visual inspection of the glacial till for the initial determination of bearing capacity (final bearing capacity is by load test).

5. Concrete the caisson and load test same.

   a. Load test one caisson under each electrical penetration area at 2.0 times design capacity.

   b. Load test each caisson individually at 1.5 times design capacity.

c. Load test all caissons as a group at 1.0 times design capacity or 1/4" of vertical structure movement, whichever occurs first.

d. Upon completion of any tests the caissons are to be left in a prestressed state to prevent any settlement.

6. Install support of excavation system along the turbine building foundation wall and connect it to the access shaft and the jacked caissons. The jacked caissons which were previously installed under the electrical penetration area will temporarily act as support of excavation for the excavation under the isolation valve pit. The containment structure and the buttress access shaft form the remainder of the excavation enclosure under the isolation pit.

   The support of excavation system along the turbine wall foundation will also act to:

   a. Support the temporary additional load imposed on the foundation wall by the needle beams which support the isolation valve pit at the surface.

   b. Support the turbine building vertical loads within the zone of influence of the excavation under the isolation valve pit.

7. Excavate all material from underneath the isolation valve pits to a depth at which undistrubed glacial till is encountered.

8. Fill the excavation under the isolation valve pit with lean concrete backfill to within 7 feet of the existing foundation.

9. Install steel dowels in the bottom of the wing base slab

concrete and in the top of the mass concrete placed under the isolation valve pit.

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10. Place steel reinforced concrete in the 7 foot high drift under the isolation valve pit and the access area used for installation of caissons underneath the wings. The reinforced concrete acts to structurally marry the wing to the mass concrete support of the valve pit. The structural connection is used to resist horizontal seismic forces developed in the wing.

11. Drypack or grout the remaining gap (3-6 inches) between the existing foundation and the structural concrete.

The design of the caisson is based upon a very conservative caisson tip pressure of 25 kips per square foot (KSF) for straight sided caissons. This provides a tip load intensity of approximately one-tenth that normally associated with jacked piling, and will bring the long term settlement into line with expected settlements of the balance of the auxillary building. The bearing strata pressure is limited to a maximum average of 20 KSF for straight sided caissons acting as a group. If the bottom of the jacked caissons are belled in the glacial fill, the design tip pressure is reduced to 17.7 KSF. The bearing strata pressure associated with belled caissons is not relevant. The steel shells for the jacked caissons are neglected in calculating the structural capacity of the caissons.

The bearing pressure on the glacial till below the isolation valve pit is only nominally increased by the substitution of concrete for earthen fill.

Bechtel Associates ofessional Corp.
77 East Eisenhower Parkway
Ann Arbor, MI 48106
Attn: L.H. Curtis

RE: Midland Plant Units 1 and 2
NRC Meeting of 2/28/80
Underpinning Presentation

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Dear Mr. Curtis:

Enclosed is a revised synopsis of the presentation I made before the NRC, their representatives and associates and Consumers Power Company.

I would like to point out that the synopsis as now revised includes material that was not presented at the February 28, 1980 meeting. It was necessary to expand the synopsis in order to clarify some concepts presented at the meeting.

Appendix A is included as background information only and is not directly related to the February presentation.

Yours truly,

Charles H. Gould

CHG:jn
Enc.

REMEDIAL MEASURES FOR
THE ELECTRICAL PENETRATION AREAS OF THE
AUXILIARY BUILDING WINGS AND THE ISOLATION VALVE PITS,
MIDLAND NUCLEAR POWER STATION, MIDLAND, MICHIGAN

This is a synopsis of the remedial measures which I recommend be employed at the subject structures.

The subject structures have been constructed on top of approximately 30 feet of heterogenous soil fill overlying undisturbed glacial till.

Settlement of other similarly placed fill material at another location on the project prompted an investigation of the soil underlying the subject structures.

The investigation was performed by soil sampling and Standard Penetration Testing. An analysis of the resultant data indicates there are soil zones under the subject structures which have questionable settlement characteristics.

There has been no settlement or structural distress of the subject structures noted since their completion. The isolation valve pits are so rigid it is doubtful they would exhibit structural distress if settlement were to occur, but settlement could distress entering and exiting utility services. On the other hand, the wings which are integrally connected to a central structure (Control Tower) are more structurally sensitive to differential settlement between the wings and control tower. Calculations show that the structural connection between the wings and control tower would crack if all the soil support were removed from under the wings. Other similar structural analyses indicate that the soil under the wings must provide a minimum average bearing capacity of greater than 3 KSF in order to prevent structural distress at the wings/control juncture. Since there is no post-construction structural distress in the area of the wings/control tower juncture, and since the expansion joint material between the wings and the contiguously adjacent reactor and turbine buildings offers no significant shear resistance, it would lead me to conclude that prior to raising the water table by filling the cooling pond that the soil under the wings provided a bearing resistance of 3 KSF without settlement in excess of that experienced by the soil under the control tower. The subsequent filling of the pond and raising of the ground water in the soil under the subject structure has the effect of reducing the 3 KSF lower boundary to 2 KSF.

The most positive remedy to the problem of questionable settlement characteristics is to either remove the material and replace it with concrete or to structurally bypass the questionable fill with caissons or piles founded in glacial till.

The decision as to which method to employ was predicated on the need to provide resistance for vertical static and seismic forces as well as horizontal seismic forces, a feasible construction procedure, and probable cost.

It is possible to accurately load test soil support for piles or caissons, whereas, soil overlain by large expanses of concrete is extremely difficult to test for capacity and/or settlement characteristics. Therefore, considerably more conservative soil bearing values must be employed if the soil replacement method is employed because loading testing is not practical. Since the original design soil bearing pressures for the wings was 5.5 KSF and the maximum untested soil bearing capacity for glacial till is 8 KSF, then the replacement method would require that approximately 67 percent of the soil must be replaced with concrete. However, by load testing caissons or piles it is possible to conservatively increase the soil bearing strata stress to 20 KSF in glacial till, provided the caisson or pile is adequately imbedded in the glacial till. Thus, in terms of the effective area to be undermined, the caisson or pile methods require only 27 percent of the structure to be undermined, provided that the concentration of bearing capacity does not result in distress to the structure. Therefore, in many respects the caisson or piles methods are two and one-half times more efficient than the soil removal/concrete replacement method.

The installation of caissons or piles is performed in small drifts (6 ft. x 6 ft. x 6 ft.) under the existing structure. The work is performed in such a manner that the total area of undermining (and its zone of influence) can be controlled so that it never exceeds 15 percent of the foundation area of a wing. Furthermore, once the first two caissons under one wing are installed, tested and stressed, they alone are capable of supporting that wing when they function as the prop in a propped-cantilever structure. Consequently, it is safe and feasible to perform the caisson method without elaborate and expensive temporary support. Consequently, I recommend that the heavier wing structures (800 kips each) be underpinned using the caisson method without temporary support.

Conversely, the logical choice for permanently supporting the lighter (2000 kips each) valve pits is mass replacement of the questionable material with concrete while temporarily supporting the lighter structure, <u>given</u> the requirement for a concrete monolith sufficient to resist N-S horizontal seismic loads.

The amount of caisson capacity required for supporting the wings is based on the wing load being shared between caissons installed at the free end of the wing and the existing control tower foundation at the fixed end. The control tower foundation soil has already been preloaded in the amount necessary to support the wings. A discussion of this condition is attached. 020673

The plan of attack for performing the remedial work is as follows:

1. Temporarily support the isolation valve pit by the use of needle beams spanning between the buttress access shaft and turbine building foundation wall at the ground surface.
2. Locally dewater the soils underlying the areas to be underpinned. The dewatering is to be performed in two stages.

   The first stage dewatering would be installed from inside the turbine building and from accessible areas outside the structures. The objective of stage 1 is the lowering of ground water to a minimum depth of ten feet below the bottom of the subject foundations. Excavation in the dry would then proceed to a maximum depth of 7 feet below the existing foundation.

   The second stage dewatering would then be installed from the excavated area under the foundation. The objective of the second stage is to dewater the fill to a depth that glacial till is encountered. The dewatering wells shall be packed to prevent piping and the discharge shall be monitored for fines.
3. At the completion of stage 1 dewatering. Excavate an access shaft adjacent to the isolation valve pits to a depth of approximately 7 feet below the bottom of these pits. The excavation would then proceed laterally as a drift until the excavation reaches the cantilever edge

of the electrical penetration area.

4. Install jacked caissons at this location utilizing the electrical penetration area foundation as the reaction. The jacked caisson method has been selected for the following reasons:

   a. It will be possible to jack through loose sands and soft clays without excavating material from within the caisson thus preventing loss of ground from under the electrical penetration area, turbine building and buttress access shaft.

   b. It is known that sizeable, but unreinforced, concrete mats were placed in the fill zone during construction. (The mats were used to support construction equipment.) It is anticipated that some of the caissons shall encounter the concrete. The caisson provides man-size working room to facilitate rapid demolition of the concrete.

   c. Likewise, the man-size working room of the caisson will permit direct excavation of highly compacted sands and/or clay as well as the glacial till (caissons penetrate the glacial till a minimum of 4 feet).

   d. The caisson provides access for direct visual inspection of the glacial till for the initial determination of bearing capacity (final bearing capacity is by load test).

5. Concrete the caisson and load test same.

   a. Load test one caisson under each electrical penetration area at 2.0 times design capacity.

b. Load test each caisson individually at 1.5 times design capacity. 020673

c. Load test all caissons under one wing simultaneously (as a group) at 1.0 times design capacity or a maximum 1/4" of vertical structure movement, whichever occurs first.

d. Upon completion of any tests the caissons are to be left in a prestressed state to prevent any settlement.

6. Perform work preparatory for starting the deep excavation to glacial till under the isolation valve pit. Install support of excavation system along the turbine building foundation wall and connect it to the access shaft and the jacked caissons. The jacked caissons which were previously installed under the electrical penetration area will temporarily act as support of excavation for the excavation under the isolation valve pit. The containment structure and the buttress access shaft form the remainder of the excavation enclosure under the isolation pit.

The support of excavation system along the turbine wall foundation will also act to:

a. Support the temporary additional load imposed on the foundation wall by the needle beams which support the isolation valve pit at the surface.

b. Support the turbine building vertical loads within the zone of influence of the excavation udner the isolation valve pit.

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7. Excavate all material from underneath the isolation valve pits to a depth at which undisturbed glacial till is encountered. Lag and brace excavation as required as excavation progresses.
8. Fill the excavation under the isolation valve pit with concrete backfill to within 7 feet of the existing foundation.
9. Install steel dowels in the bottom of the wing base slab concrete and in the top of mass concrete placed under the isolation valve pit.
10. Place concrete the 7 foot high drift under the isolation valve pit and access area used for installation of caissons under neath the wings. The reinforced concrete acts to structurally marry the wing to the mass concrete support of the valve pit. The structural connection is used to resist horizontal seismic forces developed in the wing.
11. Drypack or grout the remaining gap (3-6 inches between the existing foundation and the structural concrete).

The design of the caisson is based upon a very conservative caisson tip pressure of 25 kips per square foot (KSF) for straight sided caissons. This provides a tip load intensity of approximately one-tenth that normally associated with jacked piling, and will bring the long erm settlement into line with expected settlements of the balance of the auxiliary building. The bearing strata pressure is limited to a maximum average of 20 KSF for straight sided caissons acting as a group. If the bottom of the jacked caissons are belled in the glacial till, the design tip pressure is reduced to 17.7 KSF. The bearing strata pressure associated with belled caissons is not relevant. The steel shells for the jacked caissons are neglected in calculating the structural capacity of the caissons.

The bearing pressure on the glacial till below the isolation valve pit is only nominally increased by the substitution of concrete for earthen fill.

LOADING OF CONTROL TOWER SOILS

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The load/settlement characteristics of the fill material under the wings is different from the control tower. The difference lies in that the total depth of soil material fill under the control tower is less than under the wings and the soil is considerably more compact under the control tower.

The control tower and wings are structurally connected. Once the foundation slabs for the wings and control tower were placed, and as the walls and slabs were subsequently placed, the connection between the control tower and wings developed increasing moment capacity. The effect of this moment resistant connection was that the control tower assumed an increasing amount of the wing load as construction progressed. As the control tower and wings settled under the increasing load, the stiffer and shorter soil spring under the control tower prevented the soil reaction under the wings from developing at the same rate.

If the soil spring under the control tower were rigid, and the one under the wings were very soft, then the control tower would have assumed the total load of the wings. If this were so, then the control tower/wing connection would have cracked as determined by the finite element analysis of the structures. This connection has, in fact, not failed. As a result, it can be determined that for failure not to have occurred the wings soil reaction must be at least average 3 KSF in a dewatered state. (Construction was performed in a dewatered state.)

Calculations show that the dead load of the extraordinarily heavily reinforced torsion box of the wing which consisted of the foundation slab, first lift walls, and second floor slab would provide approximately a 2 KSF loading. Up to this point in construction, the wing structure was "flexible" and little wing load could have been transmitted to the control tower. Therefore, the soil under the wings was subjected to a minimum uniform load of 2 KSF. The remaining equivalent of 1 KSF required to prevent failure of the control/wing juncture was developed gradually and not necessarily uniformly as the moment resistant joint developed as the structures rose above the torsion box.

In view of the evidence from the post construction borings, the structural calculations, and the condition of the structures at the control tower/wing juncture, I conclude that the control tower foundation has already assumed an additional load over and above the original design load, approximately equal to the amount that it will be required to assume when the underpinning of the wings is completed.

APPENDIX A

Other Underpinning Methods Considered for the Wings 020673

Removal of unsuitable material must comtemplate removal of all of the backfill under the wings, because of the heterogenous nature of the soil and the random occurrance of a low STP blow values. There are basically two techniques for performing the work. One method is called "stealing". In this method little or no temporary support of the existing structure is employed. A discrete amount of soil is removed from under the foundation. The excavation of the discrete units is performed by sinking a hand excavated shaft approximately four feet square in plan area until glacial till is encountered, then filling the shaft with concrete. The concrete shaft can be load tested and pre-stressed by reacting against the existing structure; however, little confidence can be placed in the testing or stressing because of the frictional engagement between the shaft and the fill which will subsequently be removed. Since the distribution of the building loading or prestress load in the concrete shaft is unknown it must be assumed that settlement due to load transfer will occur in the order of magnitude expected for consolidation of the glacial till due to a load of approximately 6 KSF.

There are several other practical aspects which must be considered when "stealing". During the excavation of the shaft it is generally necessary to create "open" ground as the shaft excavation proceeds in order to install the shaft lining. If the soil exhibited standup time characteristics of greater than 30 minutes then little ground would be "lost", and the undistrubed soil outside the shaft would support the structure without significant settlement. In this case, the soil is a heterogeneous mixture of clay and sand in varying states of consolidation, 8206 7 3 some of which have no standup time or high squeeze rates. Since the soil is a random mixture of sand and clay it does not lend itself to consolidation grouting of the sands. Therefore, in order to control ground "loss" it would be necessary to use a box shield to control the loose sands and compressed air to control the squeeze in clays.

The other practical consideration is time. Because the access to the work area is limited to a single face approximately 20 feet wide which progressively narrows to approximately 10 feet, and since adjacent shafts cannot be excavated simultaneously, it is anticipated that progress would be tediously slow, at least to the point that fifty percent of the total wing area was resting on the underpinning. When one considers that settlement is time dependent and that the un-excavated material is overloaded and has questionable settlement characteristics, then the long duration of undermining to the fifty percent complete stage could induce settlement in addition to that caused by load transfer and ground loss.

The other technique for soil replacement under the wings is similar to the previous technique except the structure is temporarily supported in whole or in part during the undermining phase for installation of either small shafts as previously described or larger excavations with internal bracing. External (outside of the underpinning work space under the wings) temporary support must be supported on either the contiguously adjacent reactor building, the turbine building or a combination of both; or from the control tower. Since the wing essentially behaves as a rigid body, partial temporary support would be possible but dependent on the moment resistance capacity of the wing in the vicinity of the wing/control tower juncture. Since concrete acting in tension is critical in determining that moment capacity, there is an inherent high risk in utilizing partial external temporary support. In this case there is also a problem of coordinating deflection of the wings and the elongation, of its temporary support. In order to realize the moment resistance capacity of the wing/control tower joint it is necessary that the wing behave as a cantilever. In order for this behavior to occur, the free end of the cantilever must deflect. The free end is also where the partial temporary support will act to restrain deflection. Therefore, the temporary support must allow the free end of the wing to deflect at the same time that it assumes its share of the wing load.

If the partial temporary does not permit deflection of the free end of the wing then it will become over loaded. It is conceivable that hydraulic sensors and jacks could be installed to monitor this condition and permit deflection to occur, but in practice the questions of how much deflection to permit and how much load to hold are formidable. (How much deflection has already occurred?) Human error in operating the jacking system as well as failure of the hydraulic system are also a real possibility with disastrous consequences.

If total external temporary support is considered, then the shear/moment resistant connection at the wing/control tower juncture works in reverse; and one must consider that the control tower, as well as the wings must be temporarily supported or a sophisticated system for "lowering" the wings must be developed in the event the control tower settles during construction.

(Overall time constraints for the underpinning work would require that both wings be temporarily supported simultaneously.) The cost of approximately 25,000 kips of simultaneous temporary support capacity is staggering.

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If total external temporary support were designed as permanent support, (and underpinning of the wing were eliminated), it would have to be founded on an independent foundation, the reactor building or the control tower because it is supporting a Category I structure. An independent system would require a bridge over both wings and the control tower. The time and cost of such a scheme are prohibitive. If the wings were "hung" from the reactor building it would produce unacceptable permanent stresses in the reactor structure.

It might be possible to support the wings from the control tower by post-tensioned cables anchored at the "free" end of the wings, (like an inverted boot strap). Such a scheme would require the Control Tower foundation to assume a large additional load (a minimum of approximately 7200 kips based on 2400 square feet of wing area at 3 KSF). This would result in settlement of the Control Tower which is founded on fill. The Control Tower is integrally connected to the balance of the Auxiliary Building which is founded on glacial till. Thus differential settlement would occur and crack the Control Tower/Auxiliary Building juncture.