APPENDIX III

Review of Diesel Generator Building at Midland Plant

by

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1.0 INTRODUCTION

This report describes a study undertaken by Brookhaven National Laboratory (BNL) to evaluate the extent to which settlement cracks observed in the Diesel Generator Building (DGB) at the Midland Nuclear Power Plant impact on the ability of the building to satisfy design requirements. Dr. R.B Landsman, of Region III, has raised questions regarding this safety issue (Ref. 1). The specific objective of this study is to assess the significance of his comments and to prepare a written response.

This objective was achieved by reviewing the existing pertinent work (published reports, testimony and analytical studies), and by interviewing key personnel so that a correct interpretation of the work performed could be made. Additional calculations were specifically omitted from the scope of this study. All of the conclusions drawn in this report are based on an assessment of calculations and studies performed by others.

The study described herein was carried out during the period of August through September 1983. On August 4, a meeting was held at NRC to discuss the problem and to obtain some of the pertinent literature. Some of this literature was carried back to BNL while other documents were mailed to NRC during the following week. Appendix A contains a listing of all reports used during the program. On August 24, a meeting was held at Bechtel Corporation offices in Ann Arbor, Michigan. Presentations were made by Bechtel and Consumers Power staff summarizing the work performed by project personnel to demonstrate the adequacy of the DGB. Their consultant's (Dr. M. Sozen of the University of Illinois and Dr. G. Corley of Construction Technology Laboratories) also discussed their work. An inspection of the DGB was held on the evening of August 24 and during the morning of August 25. At this inspection, the cracks were observed although no new detailed crack maps were made. Discussions were held with construction personnel to determine the sequence of concrete placement.

Further interviews were held at NRC on September 8. Individual interviews were held with Dr. Harry Singh (soils consultant for NRC from the Army Corps of Engineers), Joseph Kane (NRC staff), and Lyman Heller (NRC staff).

A combined interview was also conducted with Frank Rinaldi (NRC staff), John Matra (structural consultant for NRC from Naval Special Weapons Center), and Dr. Gunnar Haarstead (structural consultant for NRC). The purpose of these interviews was to explore the role each played in the design and analysis of the DGB and to learn of their concerns regarding the adequacy of the DGB.

An audit of the DGB calculations by the task group was held at Bechtel's Ann Arbor offices on September 12 and 13. Dr. Sozen was present on September 13. The following items were reviewed in detail during this audit: numerical models used by Bechtel to calculate stresses in the DGB due to settlement; the magnitude of stresses due to the various load cases; the method of determining stresses from crack data; the accuracy of the survey methods used to monitor settlments; and the concrete pour data. A meeting was held with Dr. Landsman of Region III on September 13, at which time his specific concerns raised in Ref. 1 were discussed.

This report is organized as follows. An evaluation of the literature is presented in Section 2 of the report. Section 3 contains BNL's assessment of the adequacy of the DGB, while specific responses to Dr. Landsman's concerns are given in Section 4. Conclusions are listed in Section 5.

2.0 EVALUATION OF PERTINENT WORK

The material on the DGB which was reviewed during the course of this study is divided into six categories; namely, historical description of the structure and its settelment behavior; developed crack patterns; structural analyses to evaluate settelment stresses; treatment of other loads and stresses; and survey data. The material in each category is described and evaluated in this section of the report.

2.1 History of Structure

The DGB is a reinforced concrete shear wall building consisting of five cross walls connecting a north and south wall. The interior walls are 18" thick while the exterior walls are 30" thick. The structure is 155' by 70' in

plan and is 51' high with an intermediate floor slab located 35' above the foundation. Wall footings are located under each of the walls, the footings being 10' wide and 30" deep. The building is founded on about 30' of various fills overlying the natural glacial till.

The fill was placed from 1975 through 1977 with construction of the DGB begun in October 1977. Concrete was placed in 6 lifts as follows:

October	1977	- to Elev. 630.5	(foundation)
December	1977	- to Elev. 635.0	
March	1978	- to Elev. 654.0	
August	1978	- to Elev. 662.0	
December	1978	- to Elev. 664.0	
February	1979	- to Elev. 678.3	

Within each lift the pours were generally made from east to west. Construction joints occur in the middle of the cross walls and at the west end of each bay for the north and south walls.

Large settlements and cracks in the concrete were noticed while the lift going to Elev. 662 was being poured. Construction was halted while the problem was being studied. It was concluded that the large settlement was due to poor compaction of the fill material. This settlement caused the structure to "hang up" on the duct banks which penetrate the footings on the cross walls. The duct banks were cut loose from the DGB foundation in November 1978 and construction of the building restarted. In January 1979, 20' of sand surcharge was placed on the site to consolidate the fill. This remained in place until August 1979. In September 1980, a permanent dewatering system was installed to maintain the water table below Elev. 610.

2.2 Settlement History

The DGB is founded on approximately 30' of fill material, underlain by a very stiff glacial till about 190 feet thick. A dense sand layer about 140' thick lies below the till, which is in turn underlain by bedrock. The

majority of the fill was placed at the site between 1975 and 1977, with actual foundation construction completed by January 1978. During July 1978, settlements of the order of 3.5 inches (Ref. 7) were noted which were greater than the original 40 year predicted settlements. Apparently consolidation of the fill was taking place as structural dead loads were applied. In addition, the four electrical duct banks under the structural crosswalls were acting as hard points to the foundation since they were in turn being supported by the stiff natural soils below the fill. This caused rotation of the building about the duct banks.

Construction was halted during August 1978, a soil boring program undertaken to determine the problem with the fill and Drs. R.B. Peck and A.J. Hendron retained to advise on the remedial action. The exploratory program consisted of 32 borings (with no undisturbed sampling) and 14 Dutch cone penetrometers. These confirmed that the fill had been improperly placed (in an extremely variable density state) and consisted of varying amounts of cohesive as well as granular backfill. Lean concrete was also encountered in the backfill. The thickness of silty clay backfill was found to be greater under the south-east side of the building leading to the generally larger settlements on this side.

A surcharge program was implemented to attempt to consolidate the fill more uniformly. In addition, the duct banks were cut loose from the foundation in November 1978 to eliminate the foundation hard points. Surcharging began in January 1979 and remained in place until August 1979, when it was determined that primary consolidation had been completed. Instrumentation (primarily settlement plates and Borros anchors) placed in the fill was used to arrive at this conclusion. It should be noted that the consolidation test results, obtained from undisturbed samples taken after completion of the surcharge program, did not confirm this conclusion. Data was sufficiently scattered to indicate that the fill may not be uniformly consolidated. Unfortunately, the boring program conducted after the surcharge program was completed, did not include cone penetrometer soundings for comparison with the readings taken before the surcharge was applied.

At the completion of the surcharge program, it was decided that since loose sands still existed in the fill, a permanent dewatering system would be installed to preclude the potential for soil liquefaction during a seismic event. This dewatering caused additional settlements to be developed at the site, but apparently these were related to deep seated consolidation of the natural soils under the fill, and would be more uniform than the settlements caused by the fill consolidation.

It is questionable whether the piezometer data was of any significance in analyzing the excess pore pressure condition developed in the fill during the consolidation process. The readings indicate generally very low pore pressures, about 1/20 the magnitude of the applied surcharge pressures. It is not clear in fact whether the fill was ever fully saturated at the time of the surcharge program.

Peak settlements anticipated at the end of 2025 (actual settlements to date plus secondary settlements from now till then) are specified in Ref. 7 to vary from 4.79 inches (under the NW corner) to 9.33 inches (under the SE corner). However, it should be mentioned that the exact settlement history at the various settlement markers at the DGB is open to question. For example, it is mentioned in Ref. 7 that the maximum settlements in August 1978 were about 3.5 inches. Yet the data used in the stress analyses for the presurcharge period (Figures ES-14 of Ref. 7) indicates peak settlements of only 1.99 inches. It was stated at one of the Bechtel presentations that prior to cutting the duct banks loose from the footing, footings along the North wall actually lifted off from the soil, with the DGB rotating about the duct banks. There is no indication of this behavior in any of the settlement data used in the computations. Ref. 8 lists the settlement increment from 8/79 to 12/2025 to be 2.36 inches under the SE corner of the building. For the same period Ref. 7 lists this data as 1.89 inches. Thus some inconsistencies appear to exist in the various documents.

2.3 Crack Patterns

After it was determined that settlement was a problem, Bechtel initiated a program to monitor cracks in the structure. In general cracks were visually observed and an optical comparator used to determine crack width. Crack widths greater than 10 mils were of specific interest as this corresponds to reinforcing stresses of about 10 ksi. Crack maps were prepared based on surveys conducted during December 1978, September 1979, February 1980 and July 1981. Dr. Corely observed the cracking in January 1982 (Ref. 6) and confirmed that the general pattern of cracks agreed with the July 1981 Bechtel crack maps. He prepared a detailed crack map for the center interior wall. A comparison of this center wall map (Fig. 4.21 of Ref. 6) with that prepared by Bechtel in July 1981 (Fig. 4.17) indicates that more cracking had occurred although the widths of the cracks appear to be about the same.

Cracks were observed during the BNL inspection of the plant on August 25, 1983 and some photographs taken. In general the pattern of cracks appears to be similar to the previously mapped cracks. However cracks, which had not been shown on any of the Bechtel cracks maps, were noted in both the north and south walls. These additional cracks are in the lower level (up to Elev. 664) and run at 45 degree angles to the horizontal up to the cross walls.

The first crack maps prepared from the December 1978 survey indicate vertical cracks in the cross walls which begin near the bottom of the wall and run up to Elev. 664 (this was the top of the concrete pour at the time the settlement problem was first noticed). The pattern of cracking is more severe in the east side of the building. This crack pattern is compatible with the model that assumes the cracks result from flexural stresses caused by the building "hanging up on the duct banks". No crack maps were prepared for the north or south walls.

Th second set of crack maps were prepared from the September 1979 survey. In general, many of the cracks which occurred in the east wall prior to placing the surcharge do not appear on these maps. The east center and center walls show the same type of crack patterns as shown on the first crack maps except for the appearance of additional cracks. These maps also show cracks

in the upper level of the building These cracks occur near the south side of the building in the cross walls. The cracks tend to be vertical with some inclination of the cracks near the south wall. Some cracks are indicated in these maps for the south wall. Primary cracking occurs in the east side of the wall and are concentrated in the upper portion of the wall. The north wall is shown to be more severely cracked than the south wall and contains mostly vertical cracks in the upper part of the wall. The cracks appear to be centered about the three interior walls.

The third set of crack maps were prepared from the July 1981 survey. These maps indicate the same type of cracking as before although the cross wall now contain more cracking near the north side of the building than was evident before. The west wall contains many more cracks than were shown previously. These cracks run from the Elev. 664 level down to the base of the structure.

It appears that many of the cracks which have occurred may be attributed to the building resting on the duct banks. Other cracks have occurred, however, which were most likely caused by differential settlement of the wall footings. Comparison of successive crack observations generally indicates that more cracks are occurring, but that the maximum size of the cracks is still about 20 mils.

2.4 Structural Analyses

The various analyses which have been used to evaluate stresses in the DGB are discussed in this section. The first analysis described is the method used by Bechtel to estimate stresses due to settlement for use in its load combination study. This analysis makes use of the straight line approximations to the profiles of the settlements of the north and south walls. The second and third analyses described are the Bechtel and Matra studies, which attempt to use the actual measured settlements to estimate settlement stresses. These analyses, though different in detail, lead to the similar conclusion that the settlement measurements were (and continue to be) in significant error. The fourth analysis describes a cruder model which attempts to approximate an upper bound to settlement stresses by looking at

the crack measurements. The first three analyses are based on detailed finite element models, while the fourth is based on crack patterns and crack widths.

2.4.1 Bechtel's Computation of Settlement Stresses (Ref. 2)

Since the building settlements occurred when the structure was in various stages of construction, the settlement stresses were evaluated for four different time periods. The first period spans from the beginning of construction through August 1978 at which time construction was halted. The second time period extends from August 1978 to January 1979 during which the duct banks were cut loose from the structure and construction resumed. The third time period extends from January 1979 to August 1979 during which time the surcharge was placed. The last time period extends to the year 2025 and includes measured settlements from August 1979 to December 1981 as well as the predicted settlements over the forty year life of the structure.

The actual measured settlements were used to calculate stresses for the first period. Stresses were calculated in each of the walls by determining the arc of a circle which fit any three adjacent measured displacements. The radius of the arc was then used to find the resulting bending moment in the wall, and the moment used to calculate stress. The maximum stress in each of the walls was assumed to exist over the entire wall. The stress in the south wall was 11.3 ksi; the east wall 6.6 ksi; and all other walls 2 ksi.

The increments in stress which occurred during each of the other three time periods were evaluated using a finite element model of the DGB. This model was constructed and run on the Bechtel version of SAP (BSAP). The building was defined with 853 nodal points. Plate elements were used to model the walls, and beam elements used for the footings. Eighty-four (84) boundary elements were used to model the vertical soil stiffness (equivalent to the coefficient of subgrade reaction). An iterative process was then used to determine the stiffness of these boundary elements. A best fit straight line was first fit through the measured settlements for the north wall and another straight line fit to the data for the south wall. It was shown that the measured displacements departure from the best fit straight lines is within the tolerance of the survey data. Dead load reactions were next estimated at

each of the 84 boundary elements. The stiffness of any soil element was then determined as the ratio of the dead load reaction to the displacement of the best fit straight line. The BSAP program was run and the reaction found at each of these boundary elements. A new stiffness was then calculated as the ratio of the reaction to the displacement of the best fit straight line. This process was continued for several iterations.

It is our opinion that this model will yield unconservative estimates of stresses. If the iteration process were successfully completed, the deformation of the north and south walls will be straight lines. The only stresses that would be computed would then occur due to racking of the structure caused by the difference in the north and south wall straight lnes. It should be clear that if a best fit plane could be passed through all the settlement points under both the north and south walls, no stresses would be computed anywhere in the building. The stresses computed by this approach are a function of which iterative cycle is used to define to soil spring parameters, and bears no resemblance to the actual soil conditions at the site. There is no reason to expect that the soil stiffness should vary from point to point as shown by the analyses. We therefore conclude that this approach to compute settlement stresses is inappropriate.

2.4.2 Bechtel's Analysis Using Measured Settlements (Ref. 3)

This analysis was performed using the same finite element model described above. This time however, the known survey displacement data was input to the program at the ten (10) wall intersection points. The settlements used were the displacement increments measured for the fourth time period described above. At the remaining 74 boundary element points, the structure was allowed to deform as required to maintain equilibrium (forces equal zero). It was found that computed stresses were very high in those elements adjacent to the wall intersection, but fall off rapidly away from these points. This indicates that the analysis overly penalizes the structure by imposing large concentrated forces at the wall intersections. In fact, at some points, the soil is required to pull the structure downward to match these known displacements.

A modified analysis was performed by Bechtel at the suggestion of the task group. Rather than input only the ten known displacements, a smoothed curve was generated which matched the known settlement data, but eliminated the sharp profile changes developed in the analysis described above. A best fit polynomial was passed through both the north and south wall settlements, and displacements computed at all boundary element points of the finite element model. Comparative plots of wall profiles indicate that this approach would still yield high stresses.

2.4.3 Matra's Analysis Using Measured Settlements (Ref. 4)

The analysis performed by Matra is similar in intent to that described above. Differences between the two are as follows. First, this finite element analysis was performed for all four time periods described in Section 2.4.1. Three separate finite element models were used to define the DGB at various stages of construction. For each problem analyzed, the known settlement data at the wall intersection points was input to the models. The report does not specifically state what input was used at the remaining boundary element points between the wall intersection. However, at the interview, Matra stated that a linear displacement profile was assumed between these points. The stress results of the analyses are similar to those described above for the Bechtel study, with similar conclusions reached. In fact, it can be anticipated that the Matra stress calculations would be even higher than the corresponding Bechtel results due to the linear assumption between data points. If in fact this was done, the conclusions reached in that report would be of little value since such high bending stresses would be generated at these discontinuities.

2.4.4 Estimation of Stresses from Crack Data (Ref. 5)

Sozen considered the problem of predicting reinforcement stresses from a knowledge of the crack patterns. He observed that the usual problem is to predict crack width based upon a given reinforcement stress. When these methods are applied to the DGB center wall, a 20 ksi steel stress is consistent with a crack width of 20 mils. He also adds the crack widths for a series of cracks in the center wall and equates this to the total elongation

in the reinforcement. Using an estimated gage length over which this elongation occurred he obtains an estimated stress of 24 ksi, and indicates a probable range of 20-30 ksi considering the uncertainties of the method. (This was presented by Sozen at the August 24 meeting). It is likely that these stress values would be reduced with time. A major cause of cracking was the hard points provided by the duct banks. When these were cut free, one would expect the stresses induced by the uneven support to be relieved. Creep in the concrete would also tend to relieve the settlement-induced stresses.

Rinaldi (pg. 11086 of the testimony) reported at the interview of September 8, that he calculated stresses using Sozen's method in each of the 5 cross walls, as well as the north and south walls. He then added these stresses to the maximum stress reported in each of the walls by Bechtel. The resultant maximum reinforcement stress was found to be less than 54 ksi (the allowable limit). It was noted that the Bechtel stresses already included settlement stresses (to an unknown degree however) from the analyses described in 2.4.1. The crack-based estimates of settlement stresses were added to the maximum of the Bechtel stresses without regard to where they occurred. While this is a conservative approach, there is no documentation of the computations. It should be noted that there would be some question in the application of this method on those walls where relatively few cracks occurred.

2.5 Stress Totals

The finite element model described in 2.4.1 was used to calculate wall forces from all loadings except for the seismic loading. A lumped mass model was used to determine forces resulting from the seismic loading. These forces were then combined according to the load combinations required in ACI 318 and ACI 349. Critical elements were then identified in each of the walls and Bechtel's program OPTCON used to evaluate reinforcement stresses. OPTCON determines the reinforcement stress resulting from out-of-plane bending moment plus in-plane shear loading. The shear capacity of the concrete is deducted from the total shear load with the difference assumed to be carried by the reinforcement. The following are peak reinforcement stresses reported by Bechtel for the critical load cases: north wall - 22 ksi; south wall - 34 ksi; west wall - 29 ksi; east wall - 23 ksi; and interior walls - 20 ksi. The allowable steel streess is 54 ksi.

2.6 Survey Data

Bechtel reports that the accuracy of the survey data describing the DGB settlements is 1/8" until the surcharge was removed and 1/16" since that time. Standard survey techniques and equipment were used.

3.0 ASSESSMENT OF THE DIESEL GENERATOR BUILDING

The DGB has undergone very large settlements which have undoubtedly caused serious structural distress. This distress is manifested in the cracks which have occurred in the building. The purpose of this section of the report is to give an opinion as to (1) whether the building is structurally sound and (2) whether the building still meets the criteria as stated in the FSAR.

An important issue is whether the major part of the settlement has occurred. The settlement data indicate that settlements are well into the secondary consolidation phase so that large additional settlements would not be anticipated. This leads to confidence that predictions of the adequacy of the structure based on settlements which have taken place to date should hold for the life of the structure. Certainly, settlements should be monitored and the problem reconsidered should more than the anticpated additional settlements occur. Relative settlements of points on the structure of .005" are significant. The accuracy of the settlement measurements should be refined to reflect this requirement.

While significant cracking has occurred in the structure, it would appear that there is little evidence to indicate that the structure is unsound. The structure is very massive and is not subjected to large loadings. Even the tornado and seismic loadings do not introduce large stresses and usually these stresses occur at locations that are not critical locations for the settlement stresses.

It is difficult to show that the stresses in the DGB meet the criteria of the FSAR. Bechtel's straight line analysis (see 2.4.1) is based on the claim that the settlement survey data is not sufficiently accurate to calculate

structural stresses. The adjustment they make to account for this inaccuracy gives results that are likely unconservative. If conservative assumptions are made then the calculated stresses are too large to satisfy the criteria and not consistent with the crack patterns observed in the structure (see 2.4.2). It is doubtful whether any analysis could now be developed which would provide more realistic estimates of settlement stresses with the required degree of confidence.

The most likely source for obtaining reasonable estimates of settlement stresses are the crack studies (see 2.4.4). However, these studies must be documented much more completely than has been done to date. It is imperative that significantly better methods be used to monitor crack growth than is currently being considered. Whitemore strain gages should be used extensively. Plugs are attached to the concrete on a 2" gage. An instrument is then used to measure the distance between the plugs. Accuracies of .0001" is routine. Such gages would give a good picture of the overall behavior of the cracks. It should be noted that the repair of cracks would not interfere with the use of these instruments. No special "windows" need to be maintained during the crack repair program. This program of crack monitoring is also important because there is some indication that cracks in the DGB have not stabilized and that the number of cracks may in fact be increasing.

4.0 RESPONSE TO CONCERNS OF R.B. LANDSMAN

The Region III inspector has raised four concerns (Ref. 1) regarding the adequacy of the DGB. Each of these is addressed in the following.

Concern 1: FINITE ELEMENT ANALYSIS

The first concern deals with the Bechtel finite element models (see 2.4.1 and 2.4.2) of the DGB used to evaluate stresses due to settlement. There are four objections made to the models.

Concern is raised with regard to the use of uncracked section properties while the concrete is known to be cracked. All concrete structures are

cracked and it is standard practice (specifically permitted in the ACI code) to determine forces in concrete structures based on gross section properties (i.e., neglect the cracks in the concrete and the reinforcement). If cracked section properties were used then the stresses calculated by Bechtel (2.4.1) would have been smaller. Therefore neglecting cracks in this analysis is a conservative approximation. On the other hand, the analysis reported in 2.4.2 was used to show that the measured settlements result in stresses which are so high that much more severe cracking would be expected than was observed. It was then argued that the measured values must be in error. If cracked sections were assumed for this analysis the calculated stresses would have been smaller, but probably still not consistent with the observed crack patterns.

The straight line representation of the settlements along the north and south wall for the analysis reported in 2.4.1 is said to be in error. As indicated in that section of this report, it is our opinion that this analysis will result in unconservative predictions of stresses due to settlements. As such, it is considered to be an inappropriate analysis.

The third part of this concern raises questions regarding the time effects of the settlements. Bechtel does calculate stresses for different phases of the settlement. The structure was changing during the significant settlement period. Construction was still in progress during the largest settlements. Therefore the structural geometry changed as did the concrete properties (while maturing). The Bechtel models did not account for these changes. This would have been conservative for the calculation of stresses, but would result in lower stresses in the analyses performed using the measured settlements as input.

The fourth objection deals with the claim that the NRC staff did not approve of the Bechtel analysis. It appears that this is the case and the intention of the staff was to use settlement stress data based on an analysis of the cracks rather than the finite element analyses.

Concern 2: RELIABILITY OF MEASURED SETTLEMENT VALUES

The analyses reported in 2.4.2 and 2.4.3 were used to show that stresses computed from structural models subjected to the measured settlements are very high and would indicate cracking in the structure where no cracks are observed. The objection is raised that a linear model was used and that a non-linear model accounting for plastic effects would result in a redistribution of stresses and the same conclusion may not apply. This observation is true, but by itself would not change the conclusions drawn from these analyses.

As stated above, however, there are other factors which when coupled with this objection may result in a different conclusion. The other important factors are: the assumed shape of the settlement between the measured points; and the differing geometry of the DGB when the various phases of settlement occurred.

Concern 3: STRESSES DETERMINED FROM CRACK SIZES

If the finite element analyses are not reliable then one alternative approach is to find settlement stresses from a study of the crack sizes. The objection raised is that this approach is not consistent with normal engineering practice and that there are no equations available to evaluate stresses from crack data when the stress fields are as complex as occur in the DGB. It is true that this would not be standard practice, but "non-standard" analyses may be used provided they are sufficiently documented and shown to give results that are conservative.

An approach that could predict approximate settlement stresses in the DGB could probably be used to demonstrate its adequacy. This is true for two reasons. First, stresses in the structure due to other loadings are rather low and there is a large reserve for settlement stresses. Second, if large settlement stresses and local yielding of the reinforcement occurs, the resulting deformations of the structure will reduce the settlement induced loadings.

The documentation of the crack analyses used to determine stresses is not sufficient. There is no calculation on record which calculates stresses in all of the walls using this method. There is also no written justification showing that the method may be used for structures like the DGB.

Concern 4: CRACK MONITORING

This concern deals with the lack of a good crack monitoring system and specification of action to be taken if the cracks exceed certain limits. As stated in Section 3.0, it is our opinion that the planned crack monitoring system is not adequate. More reliable gages (e.g., Whitemore Strain Gages) should be placed in areas where cracking is now evident. These gages can be used even after crack repairs are made.

Two limits are now defined in the current crack monitoring program. If the crack width reaches .05" (Action Limit) a meeting will be held to evaluate what steps to take when the cracks reach the next limit. The next upset limit is set at .06" (Alert Limit). It is our opinion that the form of this plan is adequate, but that the specific threshold numbers must be based on a resolution of the current settlement stresses. A safety margin must be left for the other potential loading events, such as tornado or seismic loads, with the remaining allowable stress allocated to future potential settelments.

Once this limit was reached the only solution would be to make a structural repair. The exact form of this repair would depend on the location and extent of the crack which exceeded the limit. The [illegible] response could not specify the nature of the repair, but could indicate that an exceedance of the Alert Limit would result in a structural repair rather than performing additional analyses.

5.0 CONCLUSIONS

Based on the review of the studies performed to demonstrate the adequacy of the DGB, the following conclusions are drawn:

1. The settlement data indicates that primary consolidation of the fill is completed. However, it is recommended that the anomolies in the documentation of the settlement history be resolved. (See last paragraph of Section 2.2).

2. It is unlikely that a satisfactory stress analysis can be performed based on the measured settlement data. It is recommended that settlement stresses be estimated from the crack width data. The existing work that has been done in this area must be completely documented.

3. It appears that the number of cracks in the DGB are continuing to increase. It is essential that a better crack monitoring program be established as outlined in Section 3.0.

4. The upset crack width levels specified in the crack monitoring program should be chosen so that a sufficient stress margin is available to resist the critical load combinations.

5. If the Alert Limit (in crack width) were exceeded, specific structural repairs should be mandated.

6. While significant cracking has occurred in the DGB, it is our opinion that the structure will continue to fulfill its functional requirement. This conclusion is based on the fact that stresses induced in the structure by all other extreme loadings are small.

REFERENCES

1. Memorandum for R.F. Warnick through J.J. Harrison from R.B. Landsman, Subject Diesel Generator Building Concerns at Midland, dated July 19, 1983.

2. Bechtel Calculation No. DQ-52.0 (Q), Rev. 2.

3. Bechtel Calculation No. DQ-52.7 (Q) - Finite Element Calculation of Settlement Stresses Using Actual Displacements.

4. Structural Reanalysis of Diesel Generator Building Utilizing Actual Measured Deflections as Load Input, by John Matra, Naval Surface Weapons Center.

5. Evaluation of the Effect on Structural Strength of Cracks in the Walls of the Diesel Generator Building Midland Plant Units 1 and 2, by Mete Sosen, February 11, 1982.

6. Effects of Cracks on Serviceability of Structures at Midland Plant, by W.G. Corely, A.E. Fiorato, and D.C. Stark, April 19, 1982.

7. Executive Summary, Diesel Generator Building, Midland Plants Units 1 and 2, August 1983.

8. Letter from CPCo to NRR dated October 21, 1981; Enclosure 1, Tech. Report, Structural Stresses Induced by Differential Settlement of the DGB.

APPENDIX A: SOURCE MATERIAL FOR STUDY

Site Specific Response Spectra — Midland Plant Units 1 & 2
Addendum to Part 1
Response Spectra--Orginal Ground Surface
Jan 81 Weston Geophysical Corp

Site Specific Response Spectra — Midland Plant Units 1 & 2 Part II
Response Spectra Applicable for the top of fill material at the plant site
April 81 Weston Geophysical Corp

Site Specific Response Spectra — Midland Plant Units 1 & 2 Part III
Seismic Hazard Analysis
Feb 81 Weston Geophysical Corp

Soil Boring and Testing Program — Midland Plant Units 1 & 2
Test Results Foundation Soils
Auxiliary Building
Woodward-Clyde Consultants Aug 81
Docket Nos. 50-329,50-330

Test Results Perimeter and Baffle Dike Areas Soil Boring and Testing Program
Volume II Supporting Data July 81
Docket Nos. 50-329,50-330

Test Results Perimeter and Baffle Dike Areas Soil Boring and Testing Program
Volume I
Woodward-Clyde Consultants July 81
Docket Nos. 50-329,50,330

Estimates of Maximum Past Consolidation Pressure of Cohesive Fill Materials
Diesel Generator Building
July 81 Woodward-Clyde Consultants
Docket Nos. 50-329,50-330

USA/NRC Before The Atomic Safety and Licensing Board 12/7/82
testimony of; Frank Rinaldi
John Matra
Gunnar Harstead
with respect to the Structural Adequacy of The Diesel Generator Building at Midland

Official Transcript Proceedings Before NRC Atomic Safety and Licensing Board
DKT/CASE No. 50-329,50-330 OL & OM
12/10/82 pages 11008 through 11228

Evaluation Report for Concrete Cracks in the Diesel Generator Building
Consumers Power Company 2/16/82

Evaluation of the Effect on Structural Strength of Cracks in the Walls of the Diesel Generator Building Mete A. Sozen 2/11/82

Relationship of Observed Concrete Crack Widths and Spacing to Reinforcement Residual Stresses Consumers Power Company 6/14/82

Observed Cracks in Walls of Midland Plant Structures 6/14/82
Corley and Fiorato
Portland Cement Association

Safety Evaluation Report related to the operation of Midland Plant
Docket Nos. 50-329 and 50-330
Consumers Power Company
USNRC 5/82

Effects of Cracks on Serviceability of Concrete Structures and Repair of Cracks
Consumers Power Company 4/30/82

Effects of Cracks on Serviceability of Structures at Midland Plant
Corley, Fiorato, Stark
Portland Cement Association

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USA/NRC Before the Atomic Safety and Licensing Board 50-329,50-330
testimony of Jeffrey K. Kimball 9/29/81

NRC Atomic Safety and Licensing Board 50-329 OM,OL 50-330 OM,OL
witnesses; Johnson
Burke
Corley
Sozen
Gould

NRC Before the Atomic Safety and Licensing Board (no date)
NRC staff testimony of Joseph Kane
on Stamiris Contention 4.B
Docket Nos. 50-329 OM,OL 50-330 OM,OL

Safety Evaluation Report related to the operation of Midland Plant October 82
Docket Nos. 50-329 50-330
USNRC NUREG-0793 Supplement No. 2

Safety Evaluation Report related to the operation of Midland Plant June 82
Docket Nos. 50-329 50-330
USNRC NUREG-0793 Supplement No. 1

NRC Atomic Safety and Licensing Board 9/29/81
Applicant's Brief on Compatibility of Site Specific Response Spectra Approach with 10 CRF part 100 Appendix A

Safety Evaluation Report related to the operation of Midland Plant May 82
Docket Nos. 50-329 50-330
NUREG-0793

Response to the NRC Staff request for Settlement Related Analyses for the Diesel Generator Building 6/1/82
Consumers

Technical Report Structural Stresses Induced by Differential Settlement of the Diesel Generator Building
Consumers Power Company

Test Results of Soil Boring and Testing Program for Diesel Generator Building
Docket Nos. 50-329 50-330 7/31/81
Consumers Power Company

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Docket Nos. 50-329 50-330
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Witnesses; Hood 12/3/81
Kane
Singh
Rinaldi

NRC Atomic Safety and Licensing Board Docket Nos. 50-329 OM,OL 50-330 OM,OL
Witnesses; Kennedy 2/17/82
Campbell Rinaldi
Kane Matra
Hood
Singh

CSE Input to the Midland SER Supplement Aug. 82
Geotechnical, structural, mechanical and hydrologic inputs for the Midland Ser Supplement

Transcript of Proceedings USA/NRC 1/6/81
Deposition of Frank Rinaldi

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Deposition of Pao C. Huang

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50-330 OM-OL

NRC Staff Brief in Support of the use of a Site Specific Response Spectra to comply with the Requirements if 10 CFR Part 100, Appendix A 9/29/81

USA/NRC Before the Atomic Safety and Licensing Board Docket Nos. 50-329 OM-OL
50-330 OM-OL

Testimony of Dr. Paul F. Hadala with Respect to the Study of Amplication of Earthquake Induced Ground Motions and the Stability of the Cooling Pond Dike Slopes Under Earthquake Loading 9/29/81

USA/NRC Before the Atomic Safety and Licensing Board Docket Nos. 50-329 OM,OL
50-330 OM,OL

Witnesses; Boos
Hendron
Hanson

Testimony of Ralph B. Peck before the Atomic Safety and Licensing Board, in the the matter of Consumers Power Company (Midland Plant, Units 1 and 2), Docket Nos. 50-329 OM, 50-330 OM, 50-329 OL, 50-330 OL, notarized Nov. 3, 1982.

Letter from CPCo to H.R. Denton dated June 14, 1982 with Enclosure "Response to the NRC Staff Request for Additional Information Required for Completion of Staff Review of Soils Remedial Workd dated June 14, 1982.

Summary of August 17, 1982 Meeting on Soils-Related Construction Release, dated September 7, 1982, by Darl Hood.

"Structural Reanalysis of Diesel Generator Building Utilizing Actual Measured Deflections as Input", by John Matra.

Letter from CPCo to H.R. Denton dated October 21, 1981 with Enclosures:
"Structural Stresses Induced by Differential Settlement of DGB",
"Subgrade Modulus & Spring Constant Values for DGB Structural Analysis",
"Bearing Capacity Evaluation of DGB Foundation"
"Logterm Monitoring of Settlement for DGB",
"Relative Density and Shakedown Settelment of Sand under DGB",
"Estimates fo Relative Density of granular Fill Materials, DGB",
"Review and Control of Facility Chagnes to DGB",
"DGB Bearing Pressaure due to Equipment and Commodities",

Report form Woodward-Clyde to CPCo dated June 10, 1981, "Preliminary Test Results, Soil Boring & Testing Program, Perimeter and Baffle Dike Areas",

"Seismic Margin Review, Midland Energy Center Project": Volumne 1, Methodology and Criteria, dated February 1983, Volume V, Diesel Generator Building, dated July 1983, prepared for CPCo by Structural Mechanics Associates.

Applicant's Propsed Findings of Facts and Conclusions of Law on Remedial Soils Issue

Docket Nos. 50-329-OM
50-330-OM
50-329-OL
50-330-OL

Testimony of Karl Weidner for the Midland Plant Diesel Generator Building September 8, 1982

Docket Nos. 50-329-OL
50-330-OL
50-329-OM
50-330-OM

Find Report on the ADINA Concrete Cracking Analysis for the Diesel Generator Building by Gygna Energy Services, September 16, 1981

APPENDIX IV

ENCLOSURE

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

JUL 1 8 1983

MEMORANDUM FOR: R. F. Warnick, Director, Office of Special Cases

THRU: J. J. Harrison, Chief, Section 2, Midland

FROM: R. B. Landsman, Reactor Inspector

SUBJECT: DIESEL GENERATOR BUILDING CONCERNS AT MIDLAND

At the recent hearing before Congressman Udall's subcommittee, I expressed my concern regarding the structural adequacy of the diesel generator building because of numerous structural cracks that have occurred throughout the building over the years. I also expressed the same concern during the recent ASLB hearings. Mr. Eisenhut has requested me to document the basis of my concerns about the building so an independent review group can analyze them.

My first concern deals with the finite element analysis that Consumers Power Company (CPCo) used to show that the building is structurally sound. Their model of the building assumed a very rigid structure without any cracks. The building has numerous cracks, reducing the rigidity of the structure. The effects of these cracks have not been taken into account in the analysis. CPCo's interpretation of the settlement data as a straight line approximation always stems from their position that the building is too rigid to deform as indicated by actual settlement readings. The settlement of the building occurred over a period of time during different phases of construction. It is this time dependent effect that was also not used in their model. Even CPCo expert Dr. Corely testified at the ASLB hearings that the analysis should have "taken into account cracking and time dependent effects" in order to give correct results. Finally, the staff's official position, as stated by Dr. Schauer, on CPCo's analysis was, "The staff takes no position with regard to that analysis."

My second concern deals with the acceptance of the diesel generator building in the SSER #2 which was subject to the results of an analysis to be performed by the NRC consultants using the actual settlement values. The consultants testified at the ASLB hearing that this analysis gave unacceptable results and this portion of the SSER should be stricken. They are basing their unacceptable results and comments on their finding of

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very high stresses obtained in areas where no cracks exist. Therefore, the actual settlement values are not accurate enough (are in error) to be used in an analysis. The consultants, as well as CPCo, ran a linear analysis (structure always in the elastic range) instead of a plastic analysis which would allow a redistribution of loads in the structure. Therefore, supposed areas of high stress, where cracks are not located, may not exist due to redistribution of loads. Finally, the staff's official position, as stated by Mr. Rinaldi, on this analysis as performed by the consultants, was that the actual settlement values could not be relied upon to determine if the diesel generator building meets regulatory requirements.

My third concern deals with the fact that we are not following normal engineering practice in accepting the building by using a crack analysis approach because there is no practical method available today to analyze a complex structure with cracks in it. The basis of this concern is that there are no formulas available that can estimate stresses in a complex stress field like those which exist in this building. Thus, the evaluation of the structure based on the staff's crack analysis using empirical unproven formulas to determine the rebar stresses is unacceptable.

My fourth concern deals with the staff accepting the building by relying on a crack monitoring program to evaluate the stresses during the service life of the building. If cracks exceed certain levels, recommendations will be made for maintaining the structural integrity of the building. The basis for my concern deals with the lack of crack size criteria and the lack of formulated corrective action to be taken when the allowed crack sizes are exceeded.

These concerns which I have just enumerated are also shared by members of Mr. Vollmer's engineering staff, as well as their consultant. These concerns were documented in the ASLB hearing transcripts of December 10, 1982, prior to my ever expressing my concerns before the ASLB hearing or Congressman Udall's subcommittee.

In summary, since it is impossible to analyze this severely cracked structure to the total staff's approval, I recommend some remedial structural fixes be undertaken to ensure the structural integrity of the building to provide an adequate margin of safety.

Ross B. Landsman

Ross B. Landsman
Reactor Inspector

cc: DMB/Document Control Desk (RIDS)