Berkovitz
OGC

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

March 1, 1984

MEMORANDUM FOR: James G. Keppler, Regional Administrator

FROM: Stephen H. Lewis, Regional Counsel

SUBJECT: DOW LITIGATION (MIDLAND)

On February 29, 1984 Carol Rice of Kirkland & Ellis, representing Dow Chemical, called my office to speak with Dan Berkovitz (OGC) who was in Chicago on a deposition. I took the following information from Ms. Rice, which I have communicated to OGC by telephone.

Dow, at NRC's request, has identified the following NRC employees who they want to interview:

Wayne Shafer, RIII
Isa Yin, RIII
Bill Lovelace, RM
Ron Cook, RIII
Ron Gardner, RIII
Ross Landsman, RIII
Bob Warnick, RIII
Darl Hood, NRR
Joe Kane, NRR
Gene Gallagher, OPE
George Maxwell, RII

Additionally, she indicated they wanted to interview Gerry Phillip, a former Region III investigator.

OGC had requested this listing of NRC personnel Dow wished to interview because of concern that Dow would otherwise just "feel their way along" and NRC would end up with a great drain on its resources. This list and additional information provided by Ms. Rice confirms that there could be a substantial drain on NRC (and particularly RIII) resources to accommodate Dow's request.

The additional information is that Dow has already identified certain NRC personnel (she only identified you) who Dow already knows they want to depose. She further indicated that they might want to interview Jay Harrison, Bruce Burgess, Chuck Norelius, and Lee Spessard.

I indicated to Ms. Rice that Region III was concerned about the substantial time commitment that would be involved in making all of the identified personnel available for interviews, particularly if the interviews led to a request from Consumers that they be interviewed and eventual depositions of some or all of these individuals. I suggested (I had previously discussed this with OGC) that as a preliminary step Dow submit a list of questions they wanted to pose and NRC could then determine who we would consider making available. She stated that Dow was reluctant to limit the areas it wanted to explore and was also reluctant to put any questions in writing (I gather because they do not want to run the risk of having to produce the questions upon a discovery request).

I indicated that I would pass along this information to OGC. She stated she would contact Mr. Berkovitz on March 2.

OGC will be representing NRC personnel in connection with any depositions in this Federal District Court litigation. I will coordinate with them, so that they are aware of Region III's position. Prior to that, I will schedule a meeting with you to discuss this matter further.

Steve

Stephen H. Lewis
Regional Counsel

cc: D. Berkovitz, OGC
J. Lieberman, ELD
R. Hartfield, RM
G. Lear, SGEB, DE
E. Adensam, LB, DOL
J. Zerbe, OPE
R. Lewis, RII
R. Warnick, RIII
C. Norelius, RIII
R. Spessard, RIII
A. Bert Davis, RIII

E. Gallagher

J-1013

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R. Spessard, RIII
A. Bert Davis, RIII

— Midland —

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

OCT 21 1983

MEMORANDUM FOR: James P. Knight, Assistant Director
for Components and Structures Engineering
Division of Engineering

FROM: Pao-Tsin Kuo, Section Leader
Structural Engineering Section B
Structural and Geotechnical Engineering Branch
Division of Engineering, ONRR

SUBJECT: REPORT ON THE REVIEW OF THE DIESEL GENERATOR BUILDING AT MIDLAND

References:

1. Memo from R. F. Wanick, Region III to D. G. Eisenhut NRR/DE, "Evaluation of Dr. Landsman's Concerns Regarding the Diesel Generator Building at Midland," dated July 21, 1983.

2. Memo from R. H. Vollmer, DE to D. G. Eisenhut, DL "Evaluation of Dr. Landsman's Concerns Regarding Diesel Generator Building at Midland," dated July 21, 1983.

Pursuant to Reference 2 above, a task group, consisting of three members of the Structural Engineering staff and a consultant team of Brookhaven National Laboratory, was formed to re-evaluate the structural design and construction adequacy of the Midland Diesel Generator Building (DGB). The group, headed by P. T. Kuo, reviewed the design review documents and the construction reports; physically inspected the building; interviewed concerned individuals, including Dr. Landsman; and prepared a final report on the adequacy of the Midland NPP Diesel Generator Building. The final report on the adequacy of the Midland DGB is enclosed.

The task group's conclusions and recommendations are summarized as follows:

1. The settlement data indicate that the fill under the DGB is well into the secondary consolidation phase so that large additional settlements are not anticipated;

2. It is judged that there is reasonable assurance that the structural integrity of the DGB will be maintained and its functional requirement fulfilled. However, it is difficult to show that the stresses in the DGB can meet the criteria of the FSAR. The stresses due to settlement were either underestimated or overestimated by the Applicant's previous analyses;

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3. The most reasonable estimate of stresses due to settlement is based on the crack width data. However, the calculations that have been done in this area need to be completely documented;

4. There is evidence that the number of cracks in the DGB is continuing to grow. It is essential that a more accurate and reliable crack monitoring program be established; and

5. The monitoring program should specify an upset crack width level that would reflect a sufficient stress margin available to resist critical load combinations. The monitoring program should mandate structural repairs if the Alert Limit (in crack width) were exceeded.

Pao-Tsin Kuo, Section Leader
Structural Engineering Section B
Structural and Geotechnical
Engineering Branch
Division of Engineering

Enclosure:
As stated

cc: H. Denton
D. Eisenhut
R. Vollmer
G. Lear
E. Adensam
D. Hood
N. Romney
C. Tan
R. Landsman, R III
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LIST OF NONCONFORMANCE REPORTS
& AUDIT REPORTS @ Midland

REPORT ON THE REVIEW OF THE

DIESEL GENERATOR BUILDING AT MIDLAND

OCTOBER, 1983

BY

Dr. Chen P. Tan
Mr. Norman D. Romney
Dr. Pao-Tsin Kuo, Task Group Leader

Structural Engineering Section B
Structural and Geotechnical Engineering Branch
Division of Engineering, ONRR

Assisted By:

Professor Charles Miller
Professor Carl Costantino
Dr. A. J. Philippacopoulos
Dr. Morris Reich

Brookhaven National Laboratory

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1. INTRODUCTION

The Diesel Generator Building (DGB) at the Midland Nuclear Power Plant (NPP) is a reinforced concrete structure which has undergone excessive unequal settlement since its construction. The concrete walls of the DGB have been more extensively cracked than usually expected of such a concrete structure. On the basis of review and evaluation of the Applicant's (Consumer Power Co.) various analytical studies, remedial measures taken, and the commitments made and of the staff's own assessments, the original structural engineering staff reviewer came to the conclusion that the DGB was acceptable. However, an NRC regional inspector disagrees with the conclusion as to the acceptability of the DGB and has expressed his concerns in a hearing before a Congressional Government Oversight Committee.

In the wake of this controversy, the Division of Engineering (DE) formed an independent Task Group to re-review the structural adequacy of the DGB. The Task Group consists of three members from the structural engineering staff and a consultant team from Brookhaven National Laboratory. The consultant team provides expertise in both structural and geotechnical engineering. The charter of the group and its composition, the names of the Staff, and its consultants involved are included in Appendix I to this report. The Charter of this Task Group has three elements that are interwoven and do not lend themselves to neat separation. The Task Group was charged:

(1) to re-evaluate the structural design and construction adequacy of the DGB as accepted by the structural engineering staff reviewer

(2) to assess the concerns as indicated by comments from other NRC personnel, and

(3) to make recommendations to resolve any lingering concerns.

It is acknowledged that the Task Group has had outstanding cooperation from the Applicant, the structural engineering staff reviewer and its consultants, the geotechnical engineering staff reviewer and its consultant, and NRC Region III Inspector, in either group's on-site inspection, interviews, or design audit in Applicant's A/E office. It is this cooperation that enables the Task Group to assemble all the necessary information and facts in a short period of time. The chronology of the group's various activities and persons contacted are presented in Appendix II to this report.

An independent report written by Brookhaven National Laboratory is included in Appendix III of this report.

2. DESCRIPTION OF THE DGB AND ITS PROBLEMS

The DGB is a two-story, box-type reinforced-concrete (RC) structure with three cross walls that divide the structure into four cells, each of which contains a diesel generator unit. The building is supported on continuous RC footings 10' - 0" wide and 2' - 6" thick founded at plant elevation 628' and resting on a fill that extends down to approximately elevation 603'. The building has exterior wall thickness of 30", roof slab and interior wall thickness of 18". Plan dimensions of DGB are 155' x 70 with a total internal height of approximately 44'. Each diesel generator rests on a 6'-6" thick, RC pedestal that is not structurally connected to the building foundation. Figure 1 shows the general layout of the DGB.

The DGB as implied by its name is a building which houses the diesel generators and is classified as a seismic Category I structure. As such it is designed against the effects of extreme environmental conditions such as seismic load and tornado wind load. The latter includes a wind pressure, a differential pressure and tornado missile impact. The use of thick exterior walls and roof slab is basically a result of the consideration of the effects of the tornado missile impact load.

When the building was approximately 60% complete, unusual settlement and cracking of concrete walls were observed. The building was settling due to the consolidation of the underlying fill while it was partially supported along the north portion by four electrical duct banks acting as vertical piers resting on natural soil below the fill. A soil boring program to determine the quality of the backfill under the foundation discovered that the fill was uncontrolled and improperly compacted. The fill consisted of both cohesive soil, granular soil and lean concrete. The fill ranged from very soft to very stiff for cohesive soil and from very loose to dense for granular soil. At the time of the soil exploration, the groundwater level was observed to be ranging from elev. 616' to 622' and the cooling pond, located about 275 feet south of the building, had a water level at approximately elev. 622'.

In view of the condition of the DGB as described above, it was apparent that corrective measures must be taken to relieve the DGB from its distress. The remedial actions taken by the Applicant can be summarized as follows:

(A) Separate the DGB from the duct banks - The duct banks entering the DGB were isolated from the building, thus relieving the building from the effects of the rigid supports.

(B) Surcharge the DGB and the surrounding area - The purpose of the surcharge was to accelerate the settlement and consolidate the fill material so that future settlement under the operating loads would be within tolerable limits.

(C) Install a permanent dewatering system - The purpose of the permanent dewatering system is to maintain water level below elev. 610' in the area of DGB, thus minimizing the potential of liquefaction of the loose sands contained in the fill.

The effects of the remedial measures taken can be observed from the amount of settlement which the DGB has gone through as indicated in Figure 2 and also from the crack sizes and crack patterns of the walls as shown in Figure 3. Details of both settlement and cracking issues are discussed in the following sections.

3. SETTLEMENT AND CRACKING ISSUES

As a result of the remedial actions taken by the Applicant, it appears that the settlement of the DGB has mostly stabilized. However the fact still remains that the building has undergone unusual settlement and its walls have experienced extensive cracking. It has given rise to the concern of the DGB's structural capability to fulfill the function of protecting the safety-related equipment located therein as originally designed. In order to alleviate this concern and to assure that the structural integrity is preserved, the Applicant undertook a number of structural re-analyses using the FSAR criteria and the ACI 349 criteria and taking the settlement and cracking into consideration. On the basis of the results of the re-analyses, the Applicant concluded as follows:

(a) The settlements during early stages of construction and during the surcharge did not cause any unusual distress or significant loss of structural strength. As a result of surcharging, future settlement can be conservatively predicted and will not be excessive. The installation of the permanent dewatering system has eliminated any potential for liquefaction of the sand backfill below the DGB during a seismic event.

(b) Cracking of the walls during construction and surcharging has not impaired the ultimate strength of the structure.

(c) The building will be re-evaluated for its structural adequacy when the allowable limit for the cracking width is exceeded under the established monitoring program, thus insuring its safety function.

The structural engineering staff reviewer and its consultants with findings of their own independent assessments in essence concurred with the Applicant's conclusions. However, the geotechnical engineering staff reviewer and its consultant together with the Region III inspector disagreed.

A major point of contention was that the Applicant's analyses linearized the unequal settlements and thus the effect of unequal settlements has not properly been considered. The Region III inspector also contended that, because actual cracking of the concrete walls was not considered in the Applicant's analyses, the rebar stresses as calculated by the Applicant were not representative of the stress for the loading combinations considered.

In what follows the Task Group shall present its major observations of the analyses performed by the Applicant and by the consultants to the structural engineering staff, the issues raised, and its assessment of the Applicant's conclusion on the DGB structural integrity.

4. STRUCTURAL RE-ANALYSES

In the preceding section, it is indicated that the Applicant has made a number of structural re-analyses and used the results of the re-analyses to justify the DGB structural adequacy, and that there have been concerns expressed as to the appropriateness of the re-analyses. The essential elements of the applicant's re-analyses are succinctly summarized.

Settlement Analyses

Settlement of the DGB is time-dependent and load-dependent, but a complete and accurate settlement history does not exist. On the basis of the availability of the measured or estimated settlement values at various stages of construction, four cases of settlement analyses were performed by the Applicant as listed in Table 1, with the corresponding settlement values shown in Figure 2. With the exception of Case 1A which was analyzed by long hand computation and by idealizing the partially completed DGB as a series of individual beams, the other three cases were analyzed by computer through the discretion of the DGB into a number of finite elements as exemplified in Figure 4. Case 1A was accomplished by passing deflection curve through any three measured neighboring settlement points and selecting the one with the largest curvature for moment computation, and eventually, stress determination. This calculation indicated that the measured displacements would result in a maximum rebar stress of 11 ksi. For the other three settlement cases, individual finite-element models were used. For settlement Case 1B, the finite-element model represents the structure as built to el. 662 f 0 in.

For settlement Cases 2A and 2B, the finite-element model represents a fully completed structure. For Cases 1B, 2A, and 2B, springs were typically calculated at each nodal point along the foundation by dividing the structural load represented at the selected point by the measured or predicted settlement at that point. The finite-element analysis of each case then involved several iterations in which the soil springs were varied until the deflected shape of the DGB, as calculated by the model, approximated the "best fit" settlements. The resulting deflections of the DGB from these analyses as shown in Figures 5 and 6 are not in conformance with the measured values and are almost linearly related. The magnitude of stresses would depend on the final cycle of iteration selected and would bear no relationship to the actual stresses resulting from settlement. Other analyses performed by the Applicant consisted of (1) using zero and near zero soil springs to simulate the soft soil condition, and (2) considering the DGB to be simply supported. The purpose of these analyses was to study if the DGB has the capability of bridging voids and soft spots in the soil.

In an attempt to provide more insight into the problem the consultant to the structural engineering staff was requested to make an independent analysis by using the measured settlement values at 12 locations as input. It was found that the DGB should have cracked extensively and yielded to failure. However, the cracking condition as exhibited by the DGB does not bear out the conclusion of the analysis. It was, therefore, concluded by the staff's consultant that the DGB did not experience the settlement as measured and that the analysis did not reflect the actual settlement history of the DGB.

Cracking Analysis

Cracks in reinforced concrete (RC) members may be caused by the conditions of hardening or curing of the concrete (its shrinkage) or by excessive stresses in the materials (induced by too heavy loads, settlement of the footings and/or changes in temperature). Cracks due to excessive stresses appear most frequent in the tension zones and are seldom encountered in the compression zone of concrete members. Cracks in the RC walls of the DGB are caused by a combination of shrinkage, unequal settlement and temperature changes.

Drying shrinkage and thermal contraction cause shallow cracks at surface.As soon as the cracks are formed the tensile strain is relieved. In the case of cracks due to unequal settlement the tensile strain is to be resisted by the reinforcing steel. The purpose of the cracking analysis is to determine the rebar stresses from the measured crack width. First, the Applicant made an analysis of a single through crack in a subsection of the east wall of the DGB by using the Automatic Dynamic Incremental Non-linear Analysis (ADINA) computer program. The purpose of this analysis was to evaluate the ultimate capacity of a concrete section containing a single crack. As such, the results of the analysis are of only limited value in assessing the effects of the cracks. As a further attempt to resolve the concerns on cracking, the Applicant sought the opinion of Professor M. A. Sozen of the University of Illinois. On the basis of the crack patterns and crack-size, Prof. Sozen estimated the stresses in the rebar across the cracks to be in the range of 20 to 30 ksi.

The structural engineering staff reviewer also made his own assessment by combining the rebar stresses estimated from crack widths with stresses resulting from the Applicant's analyses for other operating loads. It showed that the resultant stress was within the acceptance criteria (Tr. 11086).

In order to assure the structural integrity of the DGB, the Applicant has proposed a crack monitoring and evaluation program to be used during the life of the DGB, in addition to an initial repair program. Specific acceptance criteria (i.e. alert limits and action limits) for crack width and crack width increases have been specified by the structural engineering staff reviewer and agreed to by the Applicant.

5. VIEWS ON THE ISSUES RAISED

The four concerns as raised by Region III inspector, Dr. R. B. Landsman, are directly quoted from his memorandum to R. F. Warnick, Director, Chief of Special Cases of NRC Region III, dated July 19, 1983, as follows.

I. Concern:

"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."

Comment:

The first part of this concern is that the cracks have not been considered in the Applicant's analyses. As indicated in previous discussion, cracks in the walls of the DGB are due to a combination of shrinkage, unequal settlement and temperature changes. Ordinary drying shrinkage and temperature change cracks are generally surface cracks. As soon as the cracks are formed, the tensile strain is relieved. Cracks due to differential settlement are generally through cracks across the wall thickness and, therefore,reduce the stiffness of the structural members. Structural engineers involved in reinforced concrete design are well aware of this fact. In order to take cracking of structural members into consideration, structural engineers first assume these members are uncracked and perform the structural analyses to obtain the moments, shears and axial forces required for the design of member sections. In designing the members concrete is then assumed to be cracked and does not take tension. Such a procedure of analysis and design is a standard practice and is, in fact, recommended by the ACI 318-77 code.

The second part of this concern is that the actually measured settlements have not been used in the Applicant's analyses. From the settlement data available it is obvious that settlement was continuing with the progress of construction with the maximum attained after the removal of the duct bank restraints and at the end of surcharging. In the early stages of construction the components such as the continuous strip footings, and wall portions forming the lower part of the DGB were most likely very flexible, and deflected in conformance with the settlement without creating any excessive stresses in the as-built portion of the DGB. There might be cracks in some of the components of this portion of the DGB due to shrinkage and/or displacement of the green concrete as a result of settlement. In order to adequately consider effects of settlement over the period of time during different phases of construction, the analytical models would have to be different for different phases of construction and to be meaningful there should be settlement measurements corresponding to each phase. However, there are no such detailed settlement measurements available, especially for the early stages of construction. The settlement measurements which are available correspond to those in the later stages of DGB construction, that is, when the as-built portions of the DGB are relatively rigid. The Applicant performed three separate finite element analyses for which measured and/or predicted settlement values are available. The measured and/or predicted settlement values are used as data points in linearizing the settlement. The differences between the measured/predicted settlement values and the resulting linearized values have been discounted as survey inaccuracies. This is basically equivalent to assuming that the north and south walls underwent rigid body motions. The computed stresses from this model are due to racking only. The stresses obtained in the process of linearizing the settlements, therefore, do not represent the actual settlement stresses.

The use of survey inaccuracies to discount the differences between the measured/predicted settlements and the linearized values is not convincing in view of the fact that all the settlements have not occurred after the completion of the DGB construction.

The third part of this concern is that the time dependent effect has not been considered in the Applicant's analyses. The Applicant has considered the four stages of construction, therefore the time factor has been taken into consideration but in a very gross manner. As indicated in the preceding comment in order to assess accurately the stresses in the walls of the DGB, detailed information on wall cracks (time-dependent) and on settlement values (also time-dependent) would be required for each step in the construction. There is no detailed information on either the cracks or the settlement values to cover the whole time span of construction. Basically this portion of the concern is inherent in the above two portions of the concern.

The fourth portion of the concern is that the structural engineering staff reviewer has taken no position with respect to the Applicant's analysis. From the preceding comments it is obvious that the adequacy of the Applicant's settlement analysis is questionable and it cannot be relied on to reach any conclusion. The structural engineering staff reviewer took a practical approach by ignoring the analysis, and resorted to the solution through crack analysis.

II. Concern:

"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 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."

Comment:

The first portion of concern is that the structural engineering staff reviewer disregarded the results of an analysis done by its consultants on the basis of the actual settlement values. This portion of the concern is in essence the same as the first concern. It is indicated in the comment on the first concern that the settlement was continuing with the progress of construction. When the strip footing concrete was placed, settlement started. Since the footing is a comparatively thin slab, it would likely deform with the settlement without creating excessive stresses. With the build-up of the walls, settlement increases and rigidity also increases. When the intermediate floor slab and the roof slab were completed, the complete structure became a very rigid structure and any settlement should be nearly linear unless there were weak sections across the building. To analyze the completed DGB on the basis of the settlement values which were accumulated during the construction and after its completion would result in exceedingly high stresses which are not representative of the actual values.

The second portion of this concern is that the staff has not used plastic analysis. It is suggested, that in order to conform to the measured settlement values a plastic analysis should be made to allow redistribution of loads in the structure. This observation is valid providing that rebar in the walls and slabs of the DGB have undergone yielding and plastic hinges have formed. It is the judgment of this Task

Group that, without the knowledge of accurate geometry of the DGB at the various phases of settlement, a non-linear model accounting for plastic effects would not be meaningful.

The third portion of this concern is the staff's official position that the results of the analysis by the staff's consultants on the basis of actual settlement measurements cannot be relied upon to determine if the DGB meets regulatory requirements. From the preceding comments, one cannot accurately calculate the stresses in the completed DGB without settlement data from the initial phase of construction. Given the unavailability of the data necessary to complete the input to the analysis by the staff's consultant, the previously stated staff position is reasonable.

III. Concern:

"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."

Comment:

This concern is related to the use of crack analysis to accept the DGB. Contrary to the concern expressed there are computational tools available to relate crack width to rebar stresses, but in effecting the analyses one still has to make some major simplifying assumptions which requires the judgment of the analyst. The results of such analyses in most likelihood will not be exactly the same as what actually exists. In the case of DGB the estimation of rebar stresses from the sizes of cracks is admittedly an approximation. However, it is the judgment of the Task Group that this is the only practical approach available to evaluate the DGB rebar stresses.

In evaluating the rebar stresses estimated from crack widths the following, as a minimum, needs to be considered and documented by the Applicant: whether or not the cracks are through the wall thickness; the sizes and locations of the cracks; whether or not the cracks are growing in width and/or length; whether or not the number of cracks are increasing; and whether the estimated rebar stresses due to settlement are less than the allowable values after accounting for load combinations is made.

IV. Concern:

"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."

Comment:

This concern questions the staff's acceptance of the DGB on the basis of a crack monitoring program which is not well defined in crack size criteria and in corrective action. The DGB is designed for combinations of dead, live, tornado and earthquake loads, and therefore it is expected to be able to resist these loads and their loading combinations with adequate margins of safety as designed. However, as a result of settlement which was not considered in the original design, the margins of safety have been reduced to some extent and there is some uncertainty as to its capability to resist the design loads. The purpose of monitoring the cracks is to insure that if there is any change in the condition of the structure it will be observed and appropriate actions can be taken, if necessary. The structural engineering staff reviewer has specified and the Applicant has agreed to the crack size criteria and the corrective action to be taken when the allowed sizes are exceeded. The Task Group is of the opinion that, while the approach is reasonable, details of the program should be further examined and improved. It should also be noted that the crack monitoring program should be in complement with a settlement monitoring program, since any assessment based on either of the two monitoring programs alone may be misleading.

6. AN ASSESSMENT OF THE DGB

Before assessing the structural adequacy of the DGB, let us examine general characteristics of structures in their capability to adapt to the settlement of the foundation soil. Structures may be classified as highly flexible, practically flexible, highly rigid and practically rigid on the basis of their deformability with respect to the settlement of the foundation soil.

Highly flexible structures follow the displacement of the foundation soil surface at all points. An example of such a structure is an earth embankment. Non-uniform (differential) settlements do not give rise to any complications in the deformation of such a structure.

Highly rigid structures either have a uniform settlement when subjected to a symmetrical load with symmetrical distribution of the soil compliance, or else tilt without bending. As an example of this are grain elevators, factory chimneys (smoke stacks), blast furnaces, etc. These structures level out the settlements, i.e., they perform in conjunction with the soil bearing material. It is because of re-distribution of the pressure by the structure that differential settlement effect of the supporting material diminishes.

Practically rigid structures, which include most buildings and many engineering structures (multispan trestles and bridges with continuous structural members, reservoirs, storage tanks, etc.), cannot closely follow the foundation soil deformations at all points and, because of differential settlement, are subject to bending. Such structures level out only in part the non-uniform settlements of the foundation soil surface. This results in the development of additional forces in the supporting members of the structures, which are usually disregarded in the course of their designing. Hence the possible development of cracks in such members.

Practically flexible structures largely follow the displacements of the soil surface, i.e., they bend (such as low single-story buildings), but over short sections they are capable of levelling out to a certain extent the differential settlement. This results in the emergence of usually insignificant additional forces in the supporting members. In the event of highly non-uniform settlements these forces can cause the development of cracks and fractures.

On the basis of above classification and because of the box-type construction with heavy reinforced concrete walls and slabs, the completed DGB can be considered as a highly rigid structure. However, in the process of construction, the as-built portions of the DGB at different stages of construction can be considered to vary from highly flexible, practically flexible, practically rigid to highly rigid. It is believed that most of the settlement and settlement cracks appeared at the various stages of construction. However, the cracks have not been carefully studied and mapped at each stage of construction so that a reasonable correlation of the cracks with all the causes can be established. Only the cracks which were mapped in January 1980 have been identified as shrinkage and/or settlement cracks. Most of the cracks which have been identified to be due to unequal settlement are the cracks in the cross-walls, the movement of which was restrained by the duct banks.

The DGB design, as indicated by Applicant's analyses, is controlled by the tornado wind. Under such a load, especially the postulated internal pressure, the full strength of the walls will be mobilized, and there will be a redistribution of the load, if there exist localized high stress areas. This will also be true if the seismic loads are considered. One can make such judgments on the basis of the observation that the DGB is a highly redundant structure. The structural elements are not columns and beams. They are heavy reinforced concrete walls and slabs. With necessary repair work to be done and with adequate monitoring programs, there is reasonable assurance that the structural integrity of the DGB will be maintained and its functional requirement will be fulfilled.

7. CONCLUSIONS AND RECOMMENDATION

Most of our conclusions have been expressed in our comments to the concerns they may be summarized as follows:

1. Analyses of the DGB either by linearizing the settlements or by applying the settlements as measured render unrealistic results. The stresses due to settlement are either underestimated or overestimated. A realistic analysis would be one which simulates the stage-by-stage construction of the DGB, and uses the actual and more detailed settlement measurements at each stage. However, such settlement history for the DGB does not exist. For this reason, the Task Group believes that a rigorous analysis to compute rebar stresses is unattainable.

2. The estimation of rebar stresses from the crack width is admittedly an approximation. The estimated stresses of 20 to 30 ksi appear to be reasonable. However to be convincing a detailed procedure of crack analysis should be documented and provided.

3. Inconsistences in the documentation of the settlement history needs to be resolved. For example, the Midland Units 1 and 2 Executive Summary dated August, 1983 states that for the July 1978 period, the maximum settlements recorded were 3.5 inches while Figure ES-14 of the same document indicates a maximum of 1.99 inches for the same period.

4. The current monitoring program is inadequate to deduce future distress. Thus, an adequate monitoring program for both settlement and cracks should be developed and implemented to assure that the structural integrity of the DGB should be maintained during the life of the plant.

5. On the basis of the overall evaluation, it is nevertheless felt that the DGB in its current state can fulfill its functional requirement .

6. It is recommended that a repair program be developed and implemented.

TABLE 1

DIESEL GENERATOR BUILDING

SETTLEMENT CASES

CASE	TIME PERIOD	PERIOD	PORTION OF BLDG COMPLETE
1A	3/78 - 8/78	PRE-SURCHARGE	WALLS TO ELEV 654'
1B	8/78 - 1/79	PRE-SURCHARGE	WALLS TO ELEV 662' (BELOW MEZZANINE SLAB)
2A	1/79 - 8/79	SURCHARGE	COMPLETE BUILDING
2B	9/79 - 12/2025	40 YEAR	COMPLETE BUILDING

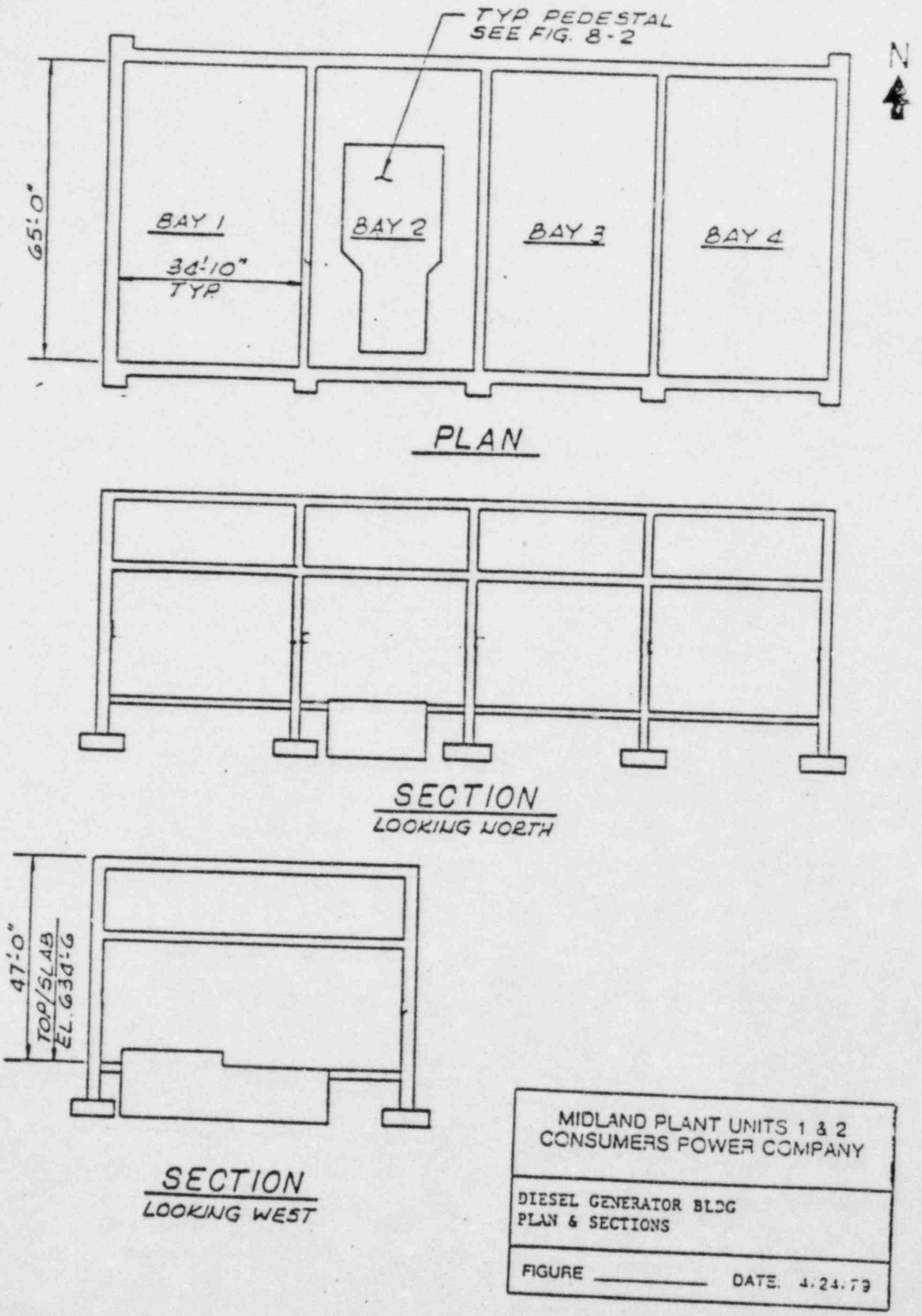
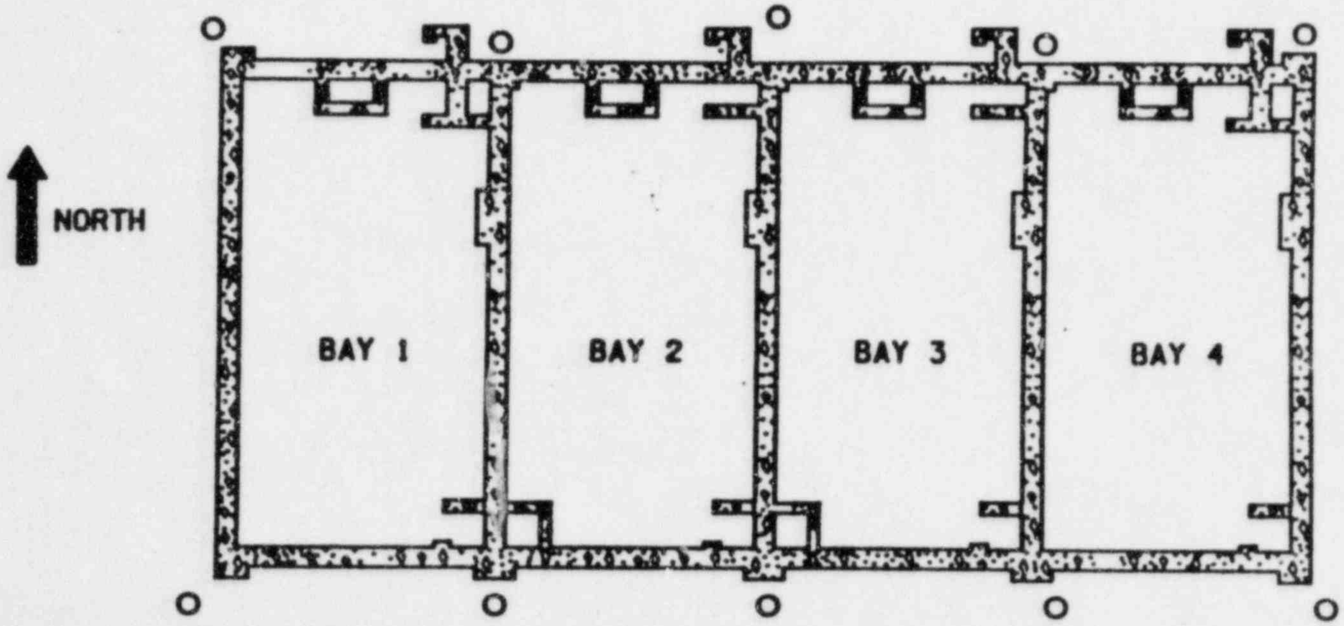

TYP PEDESTAL
SEE FIG. 8-2
N
65'-0"
BAY 1
BAY 2
BAY 3
BAY 4
34'-10"
TYP.
PLAN
SECTION
LOOKING NORTH
47'-0"
TOP/SLAB
EL. 634'-6
SECTION
LOOKING WEST
MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY
DIESEL GENERATOR BLDG
PLAN & SECTIONS
FIGURE
DATE. 4.24.79

FIGURE 1

4

LINE A	1.19	1.02	0.90	0.85	0.76
LINE B	0.77	1.09	1.54	1.98	2.41
LINE C	1.50	1.51	1.78	1.86	1.91
LINE D	1.33	1.15	1.19	1.18	1.29
TOTAL	4.79	4.77	5.41	5.87	6.37



LINE A	1.67	1.42	1.28	1.44	1.99
LINE B	1.14	1.12	1.46	1.92	2.21
LINE C	3.00	2.92	3.16	3.37	3.24
LINE D	1.62	1.67	1.69	1.98	1.89
TOTAL	7.43	7.13	7.59	8.71	9.33

LEGEND

O —— DIESEL GENERATOR BUILDING SETTLEMENT MARKER

SETTLEMENT IN INCHES FOR

PRE-SURCHARGE PERIOD (3/78-8/78)............LINE A
PRE-SURCHARGE PERIOD (8/78-1/79)............LINE B
SURCHARGE PERIOD (1/79-8/79) ....LINE C
POST SURCHARGE PERIOD (9/79-12/2025) ....LINE D
ASSUMING SURCHARGE REMAINS IN PLACE

FIGURE 2

DIESEL GENERATOR BUILDING
EXECUTIVE SUMMARY

SUMMARY OF ACTUAL AND ESTIMATED SETTLEMENTS

FIGURE ES-14

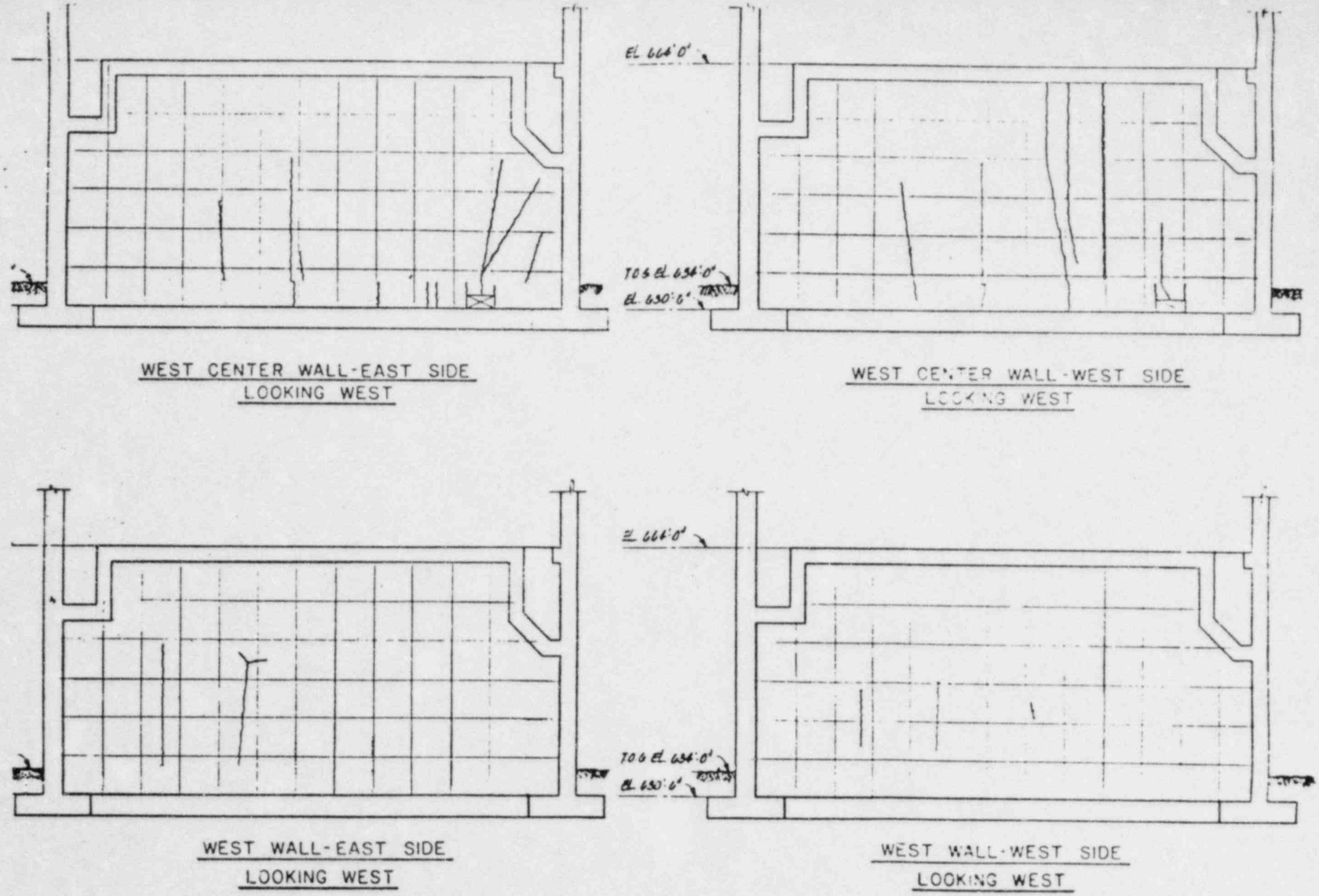
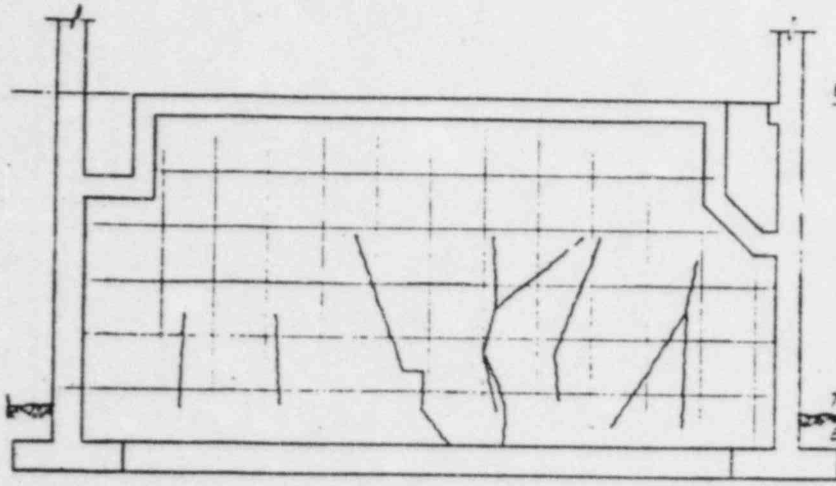
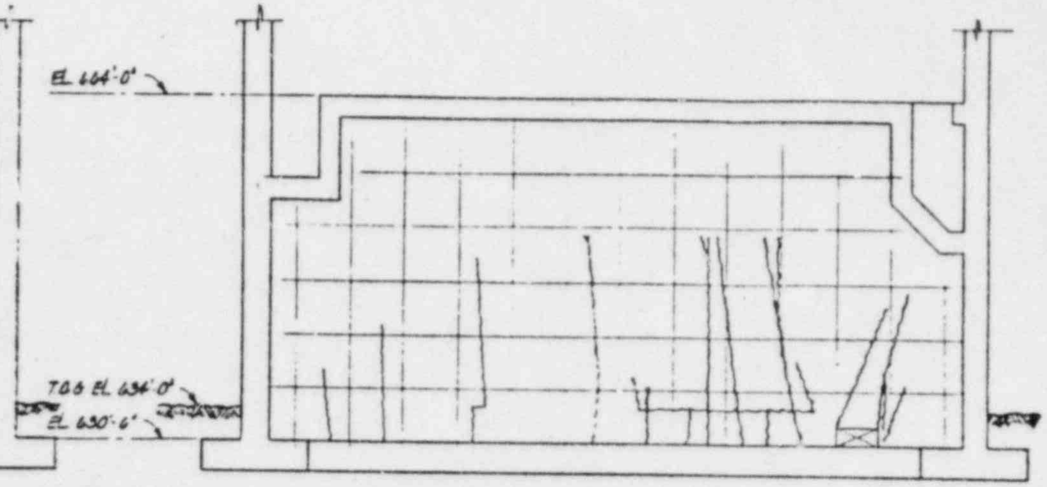

EL 664'0"
T.O.S EL 634'0"
EL 630'0"
WEST CENTER WALL-EAST SIDE
LOOKING WEST
WEST CENTER WALL-WEST SIDE
LOOKING WEST
EL 664'0"
T.O.S EL 634'0"
EL 630'0"
WEST WALL-EAST SIDE
LOOKING WEST
WEST WALL-WEST SIDE
LOOKING WEST

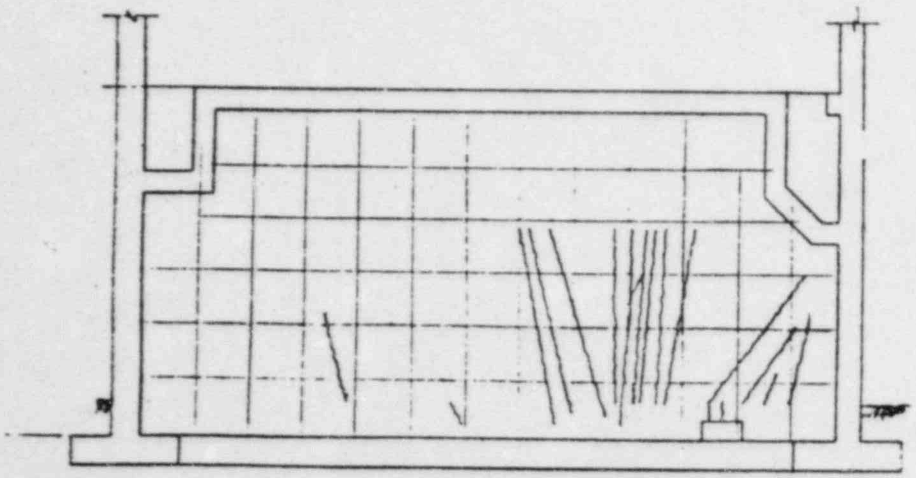
FIGURE 3 - A - 1



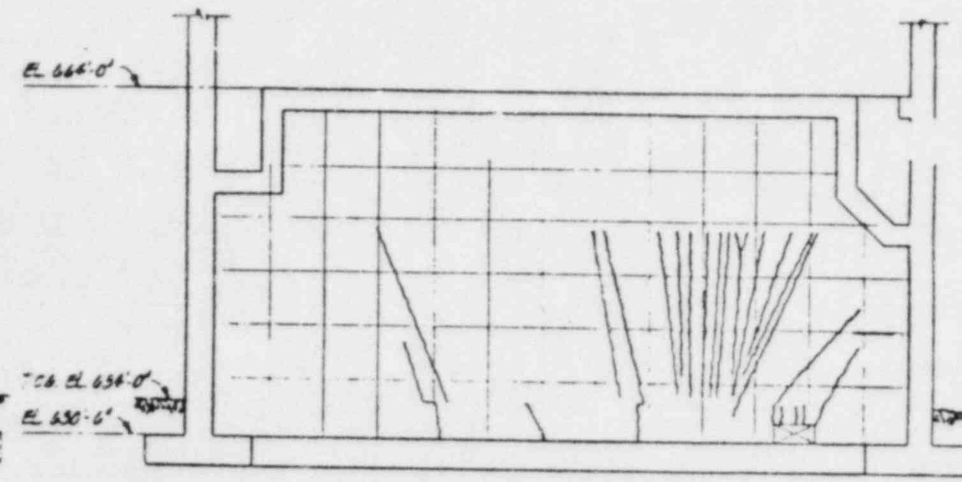
EAST WALL-WEST SIDE
LOOKING WEST



EAST CENTER WALL-EAST SIDE
LOOKING WEST

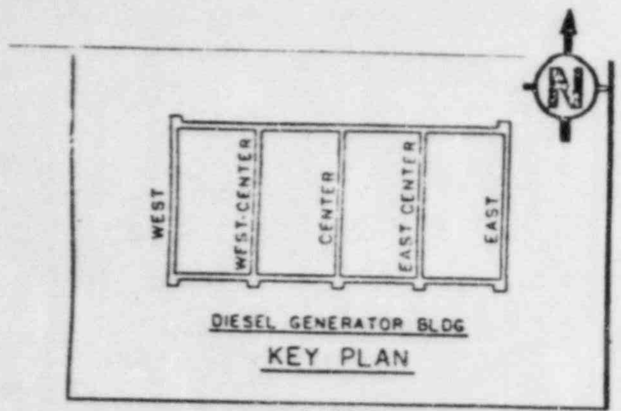


CENTER WALL-EAST SIDE
LOOKING WEST



CENTER WALL-WEST SIDE
LOOKING WEST

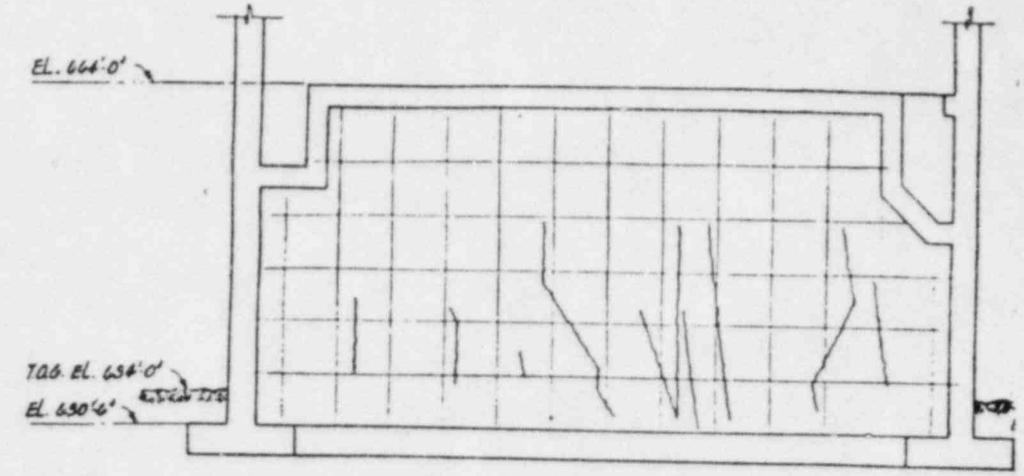
FIGURE 3 A - 2



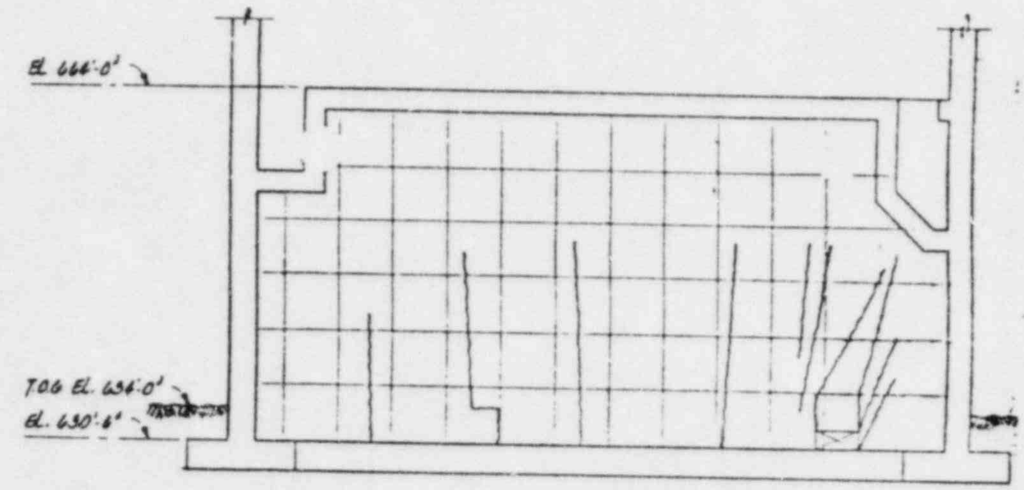
KEY PLAN

NOTES

1. CRACK SHOWN WERE MAPPED PRIOR TO PLACING THE PRELOAD. (12-13-78 TO 12-18-78).
2. NORTH & SOUTH WERE NOT MAPPED SINCE NO SIGNIFICANT CRACKS WERE OBSERVED.
3. IN GENERAL, ALL CRACKS WERE HAIRLINE WITH SOME CRACKS WITH A THICKNESS OF 28 MILS AS OF 2-2-79.

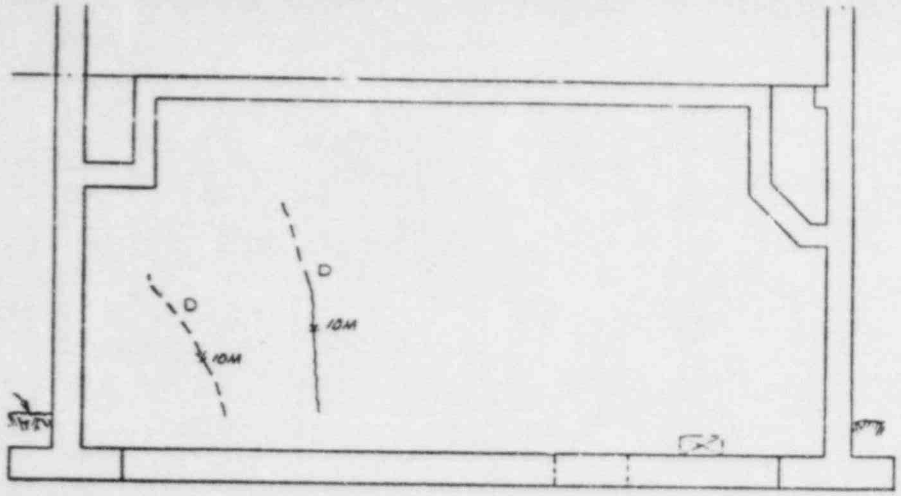


EAST WALL-EAST SIDE
LOOKING WEST

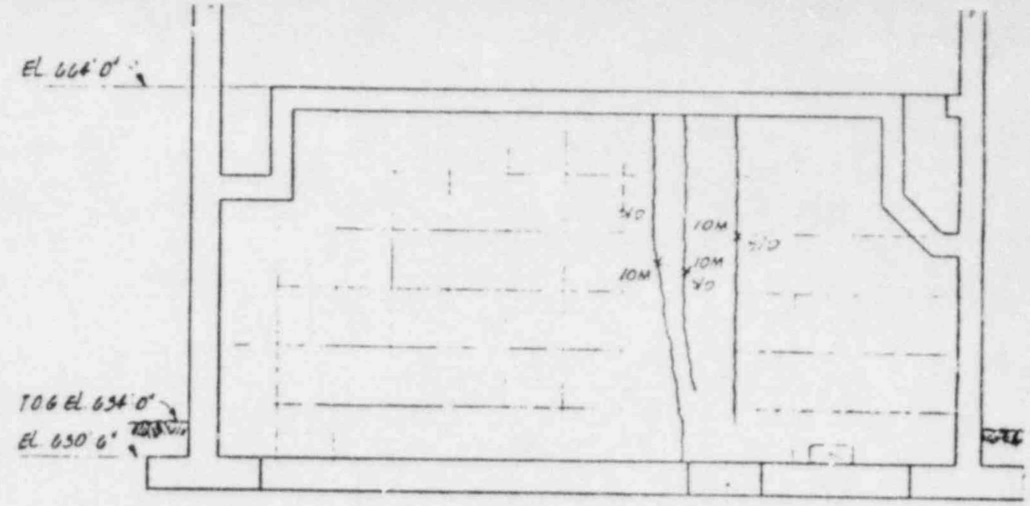


EAST CENTER WALL-WEST SIDE
LOOKING WEST

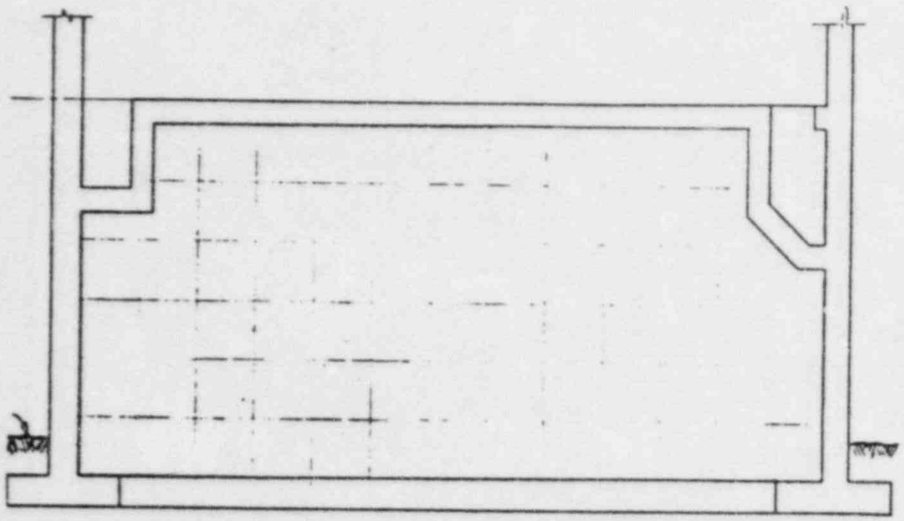
FIGURE 3 A - 3



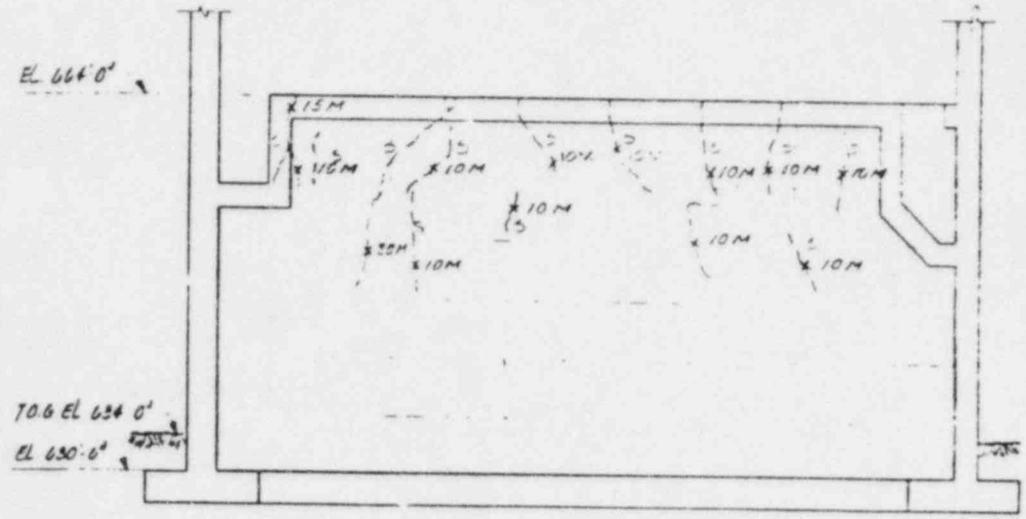
WEST CENTER WALL-EAST SIDE
LOOKING WEST



WEST CENTER WALL-WEST SIDE
LOOKING WEST

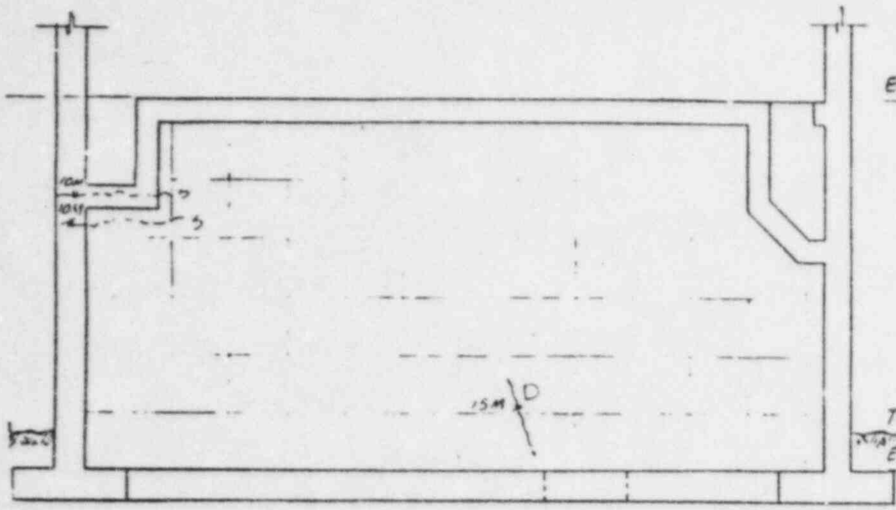


WEST WALL-EAST SIDE
LOOKING WEST



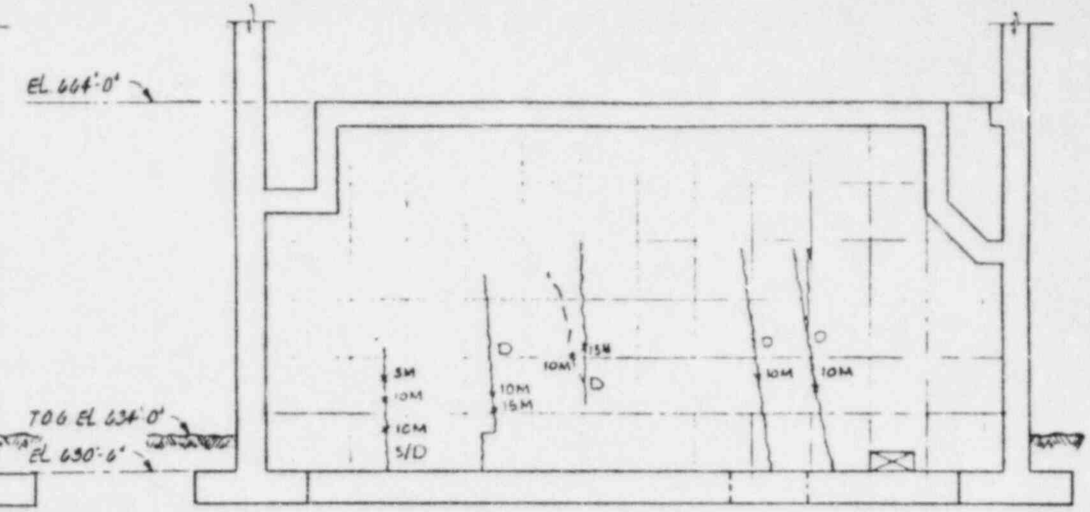
WEST WALL-WEST SIDE
LOOKING WEST

FIGURE 3 B - 1



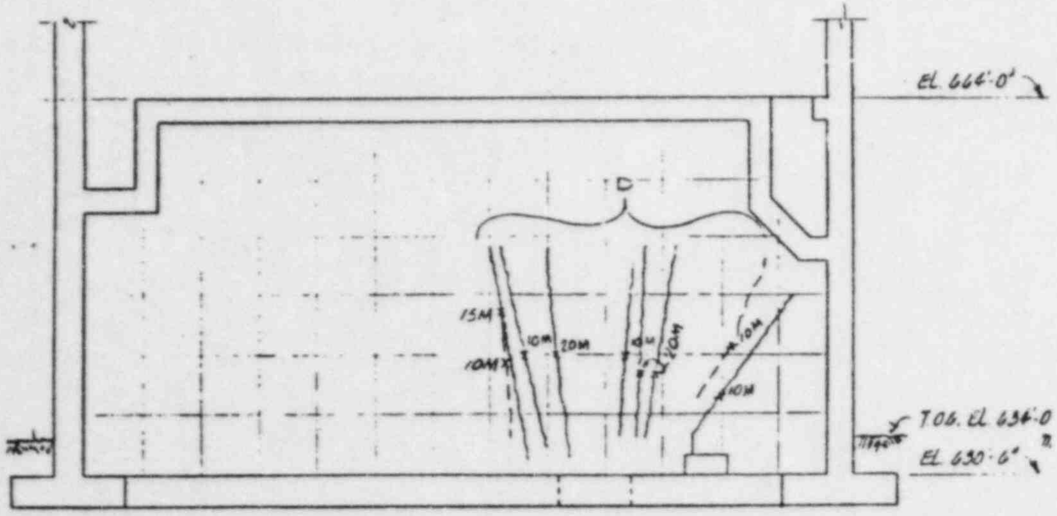
EAST WALL - EAST SIDE

LOOKING WEST



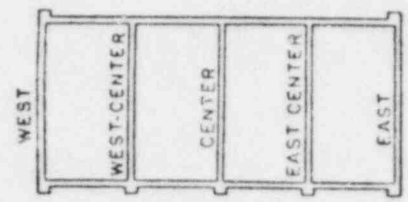
EAST CENTER WALL - EAST SIDE

LOOKING WEST



CENTER WALL - EAST SIDE

LOOKING WEST



DIESEL GENERATOR BLDG

KEY PLAN

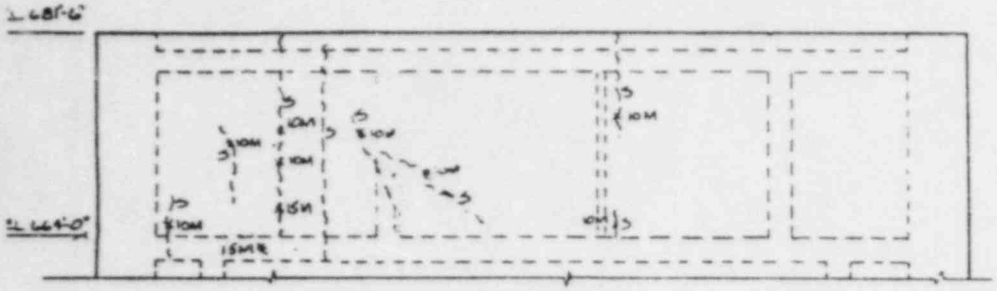
NOTES

1. IN GENERAL ALL CRACKS MAPPED IN DEC 1978 WERE HAIRLINE. SOME CRACKS HAD A THICKNESS OF 28 MILS AS OF 7-7-79.

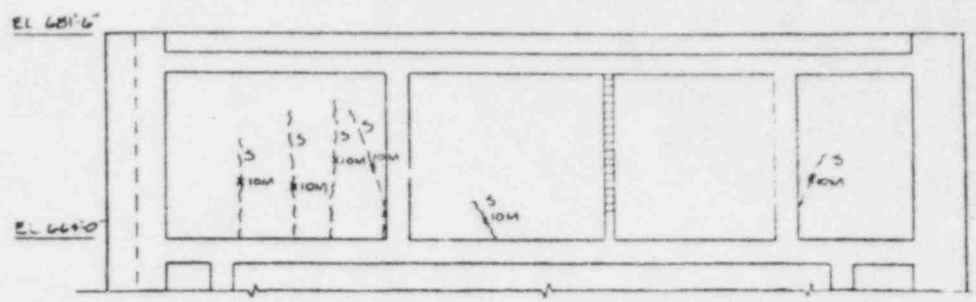
2. FOR CRACK MAPPING OF WALLS FROM EL. 664'-0" TO EL 681'-6" SEE FIG. 3B-3

3. CRACKS LESS THAN 10 MILLS IN SIZE ARE NOT SHOWN.

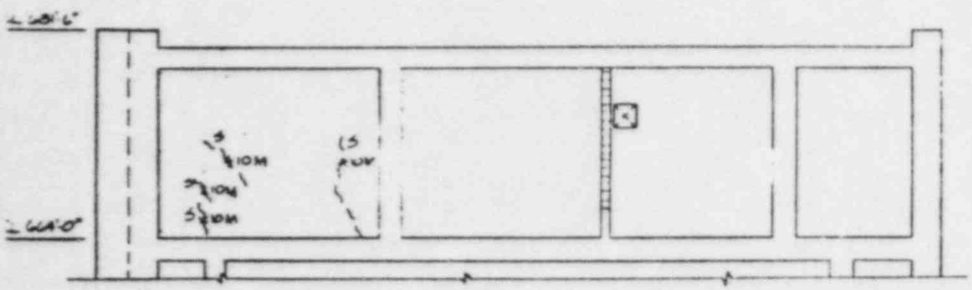
FIGURE 3 B - 2



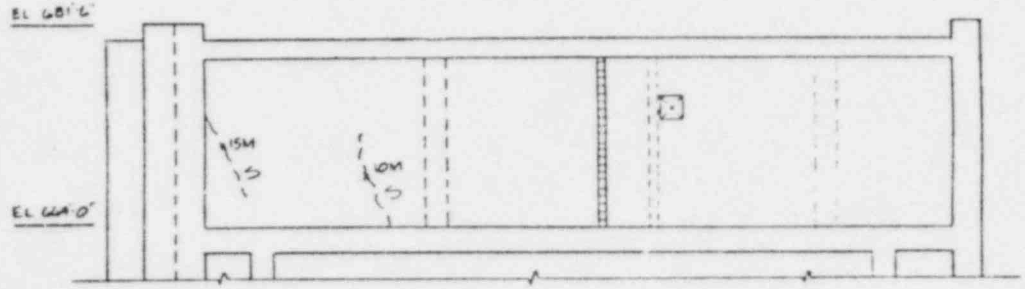
EAST WALL - EAST SIDE
LOOKING WEST



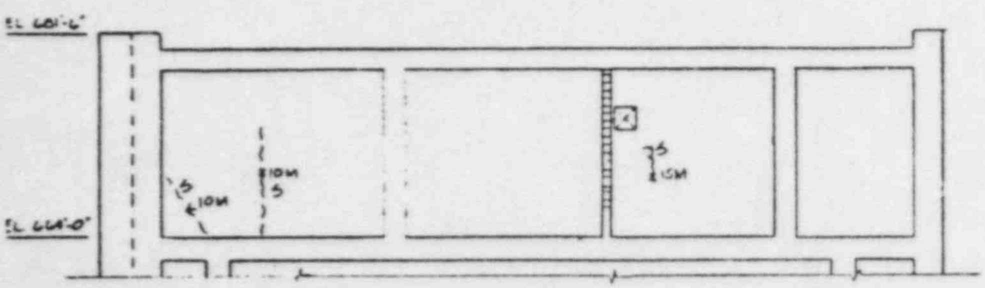
EAST WALL-WEST SIDE
LOOKING WEST



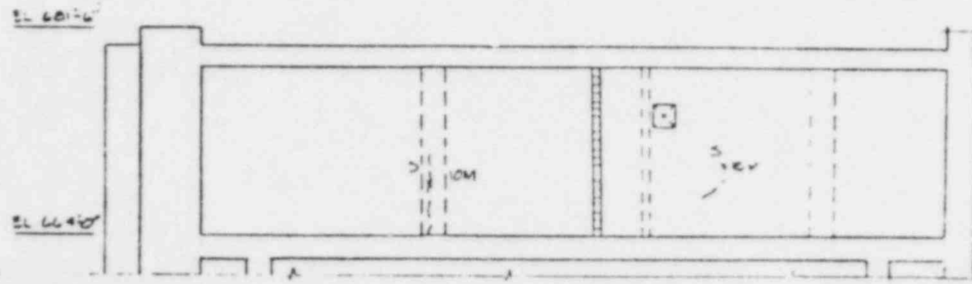
EAST CENTER WALL-WEST SIDE
LOOKING WEST



EAST CENTER WALL-EAST SIDE
LOOKING WEST

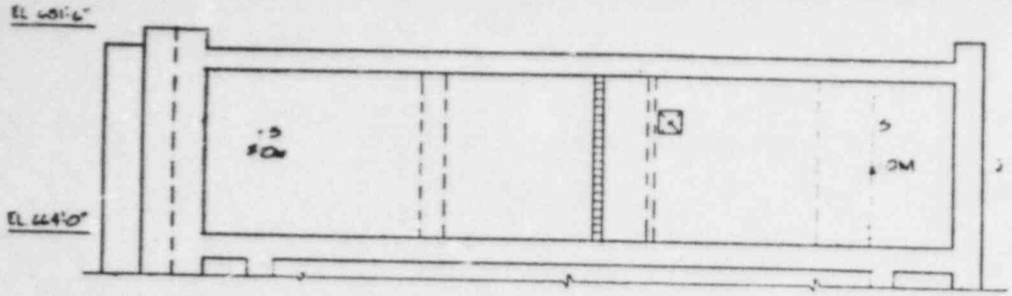


CENTER WALL-WEST SIDE
LOOKING WEST

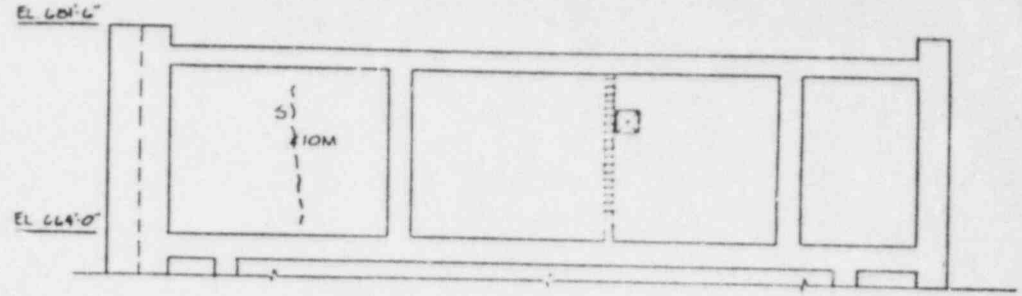


CENTER WALL-EAST SIDE
LOOKING WEST

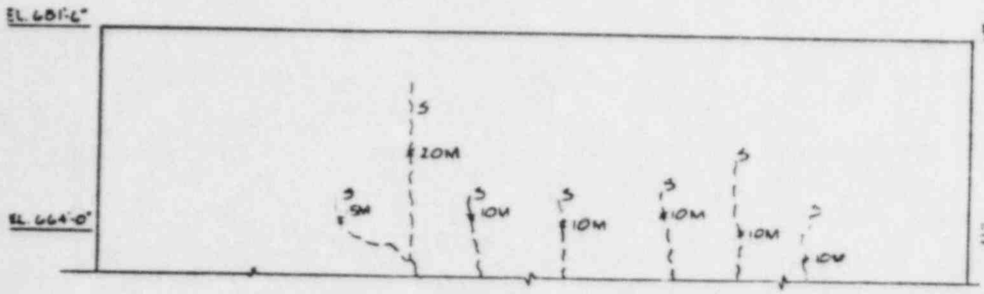
FIGURE 3 B - 3



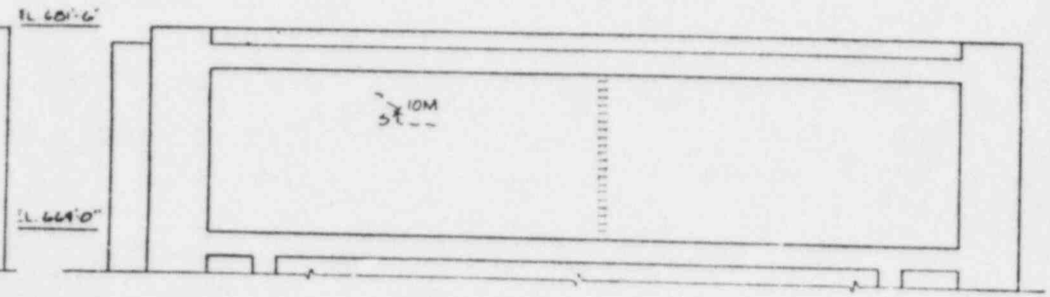
WEST CENTER WALL - EAST SIDE
LOOKING WEST



WEST CENTER WALL - WEST SIDE
LOOKING WEST



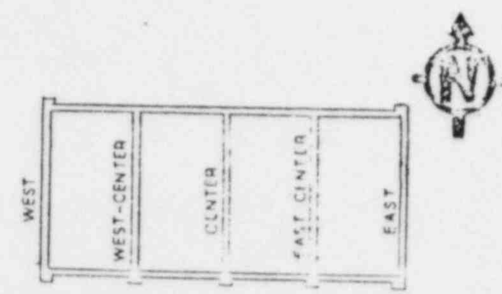
WEST WALL - WEST SIDE
LOOKING WEST



WEST WALL - EAST SIDE
LOOKING WEST

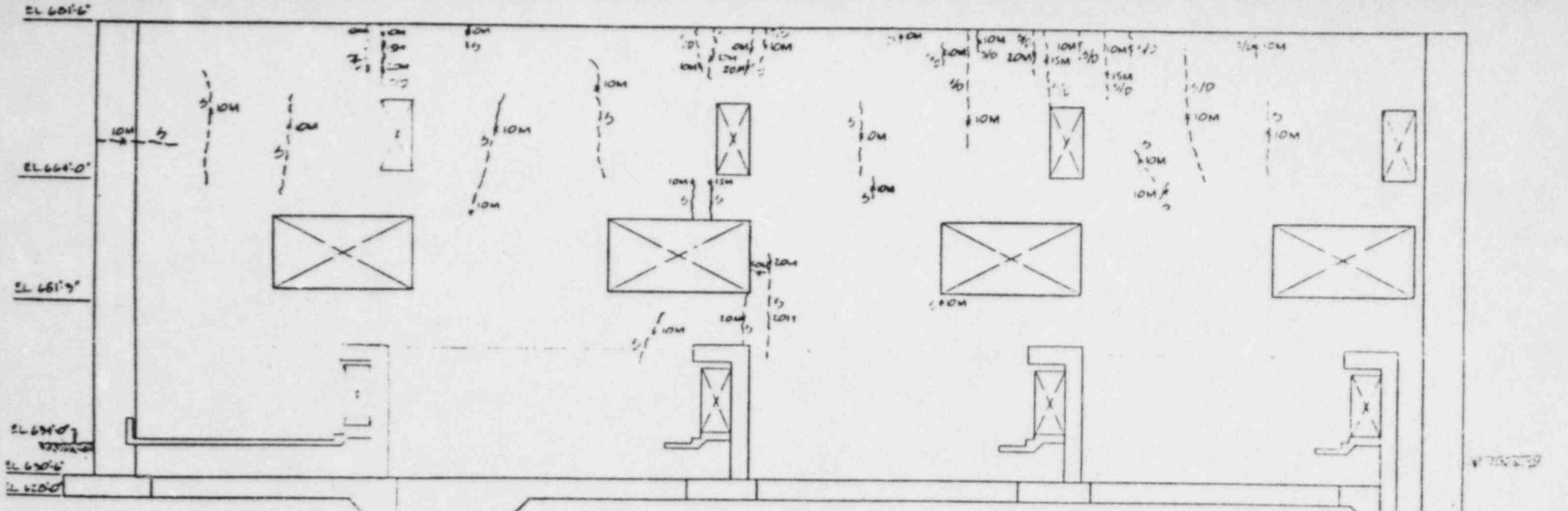
NOTES

1. CRACKS SHOWN WERE MAPPED ON JAN 1980.
2. SEE FIG 28-2 FOR TYPICAL CONSTRUCTION SEQUENCE.
3. SEE FIG 28-2 FOR CRACK MAPPING OF WALLS FROM ELEV. ON 630'-6" TO 664'-0".
4. SEE FIG 28-2 FOR ADDITIONAL NOTES AND LEGEND



DIESEL GENERATOR BLDG
KEY PLAN

FIGURE 3 3 - 4



NORTH WALL NORTH SIDE

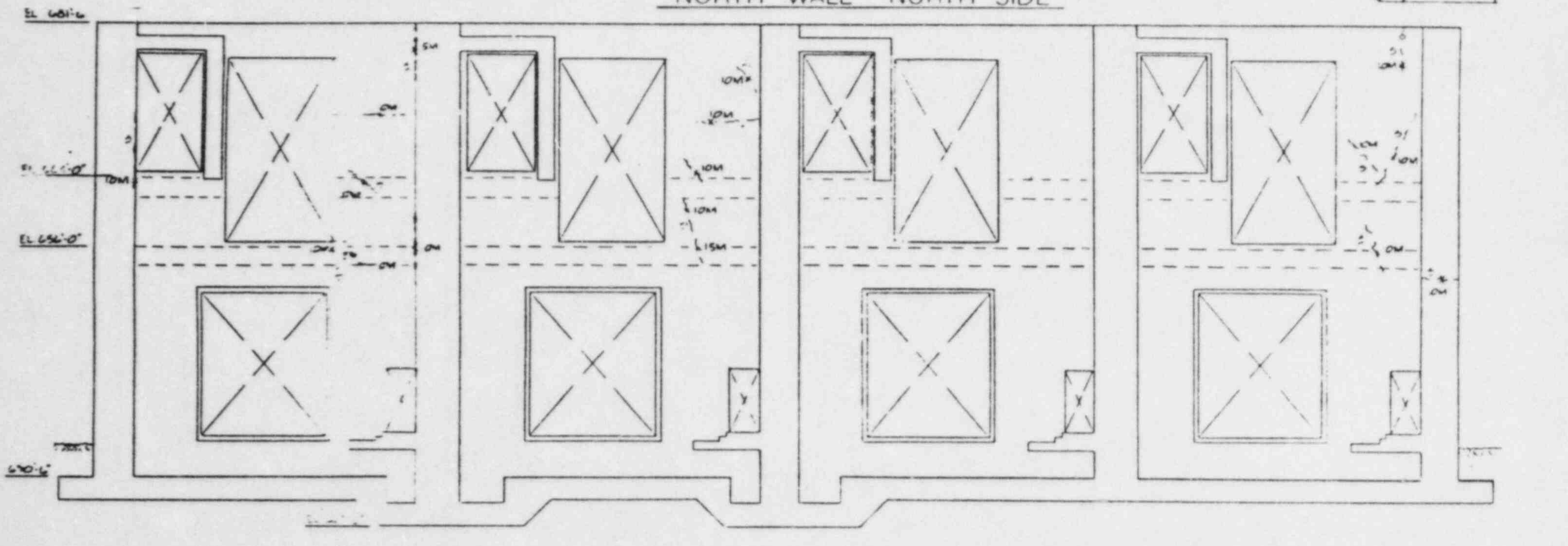
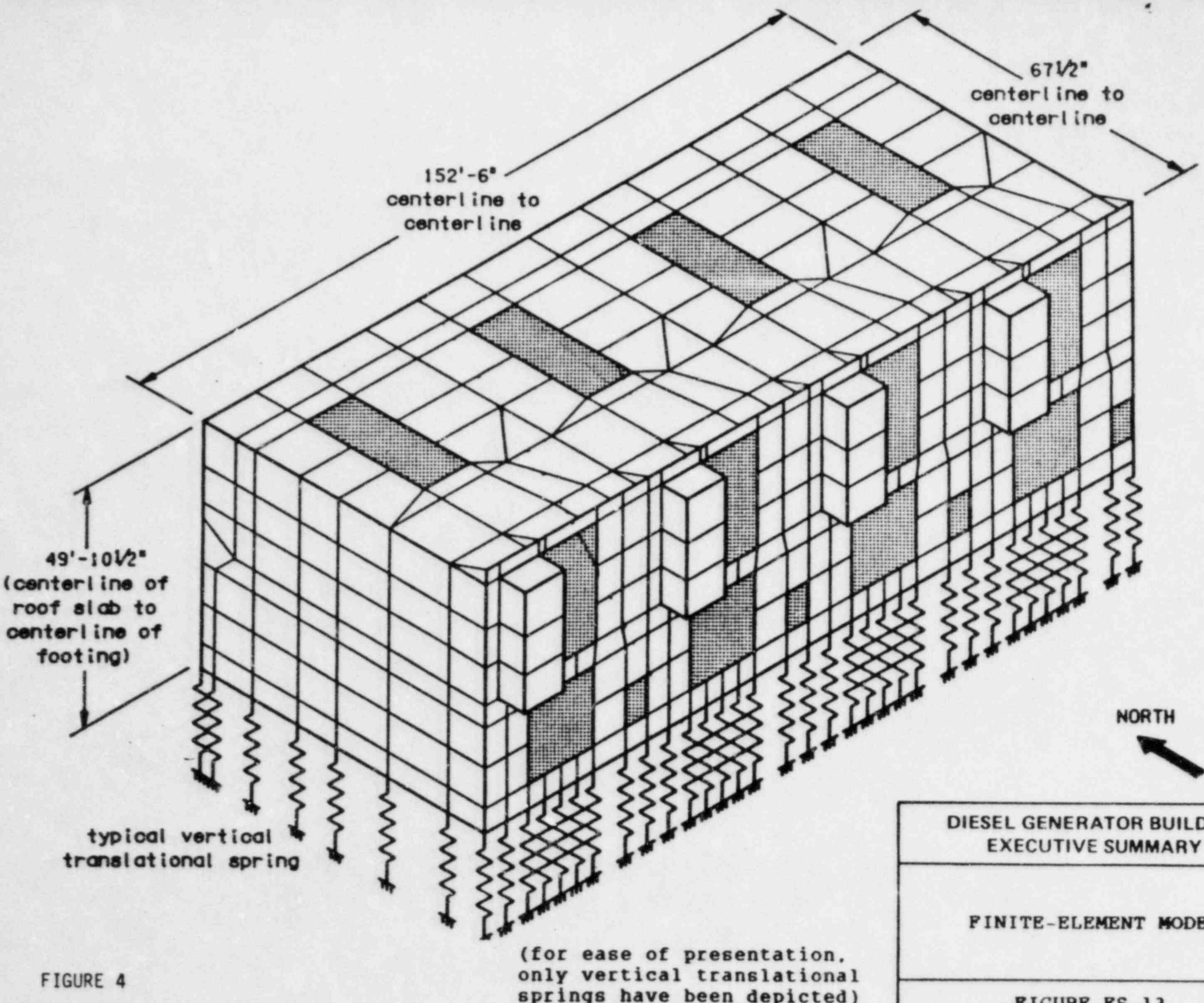


FIGURE 3 B - 5

SOUTH WALL SOUTH SIDE

LOOKING NORTH



DIESEL GENERATOR BUILDING EXECUTIVE SUMMARY
FINITE-ELEMENT MODEL
FIGURE ES-13

FIGURE 4

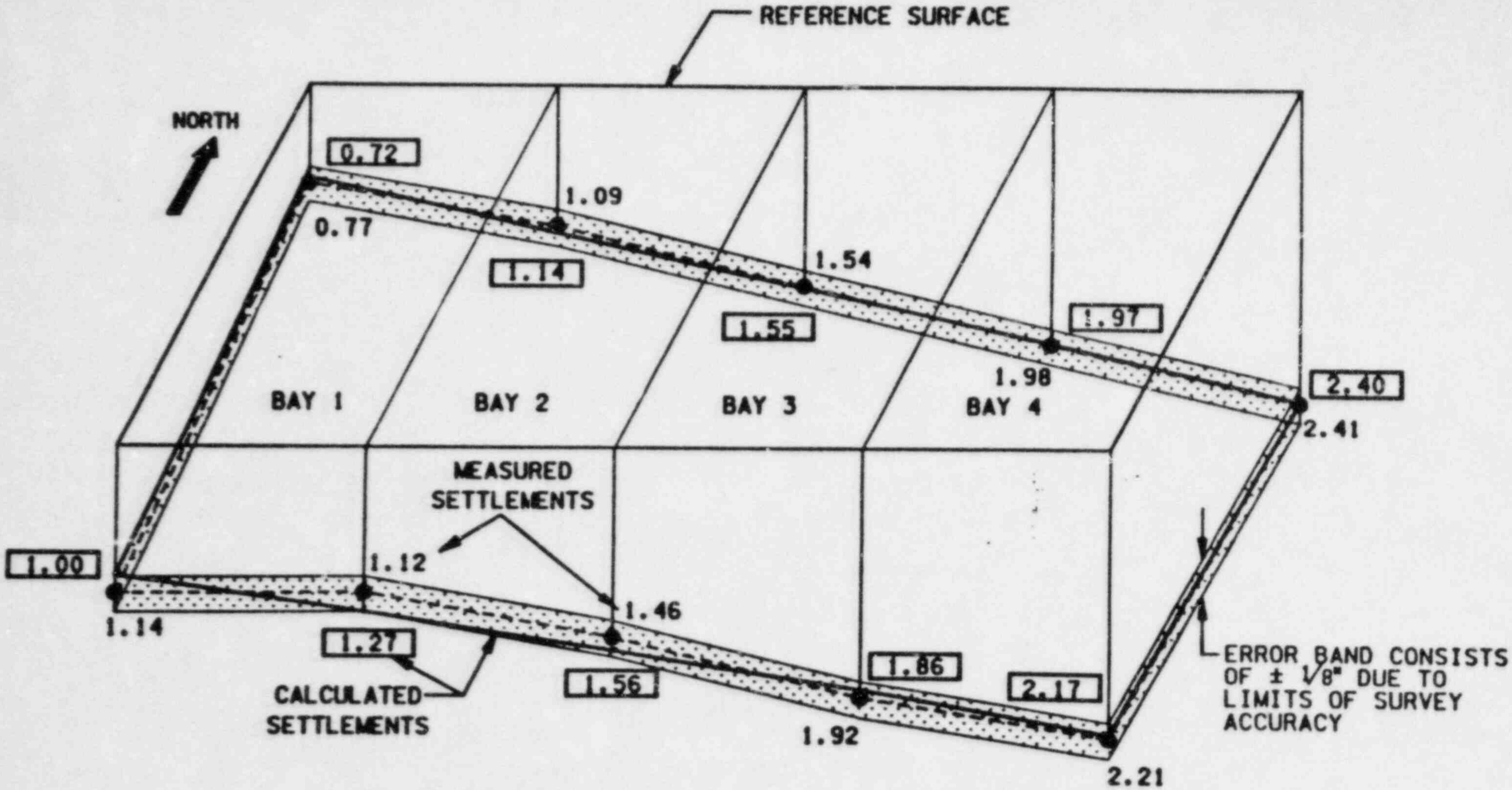


FIGURE 5

DIESEL GENERATOR BUILDING EXECUTIVE SUMMARY
COMPARISON OF SETTLEMENT VALUES PRE-SURCHARGE PERIOD AUGUST 1978 - JANUARY 1979
FIGURE ES-15

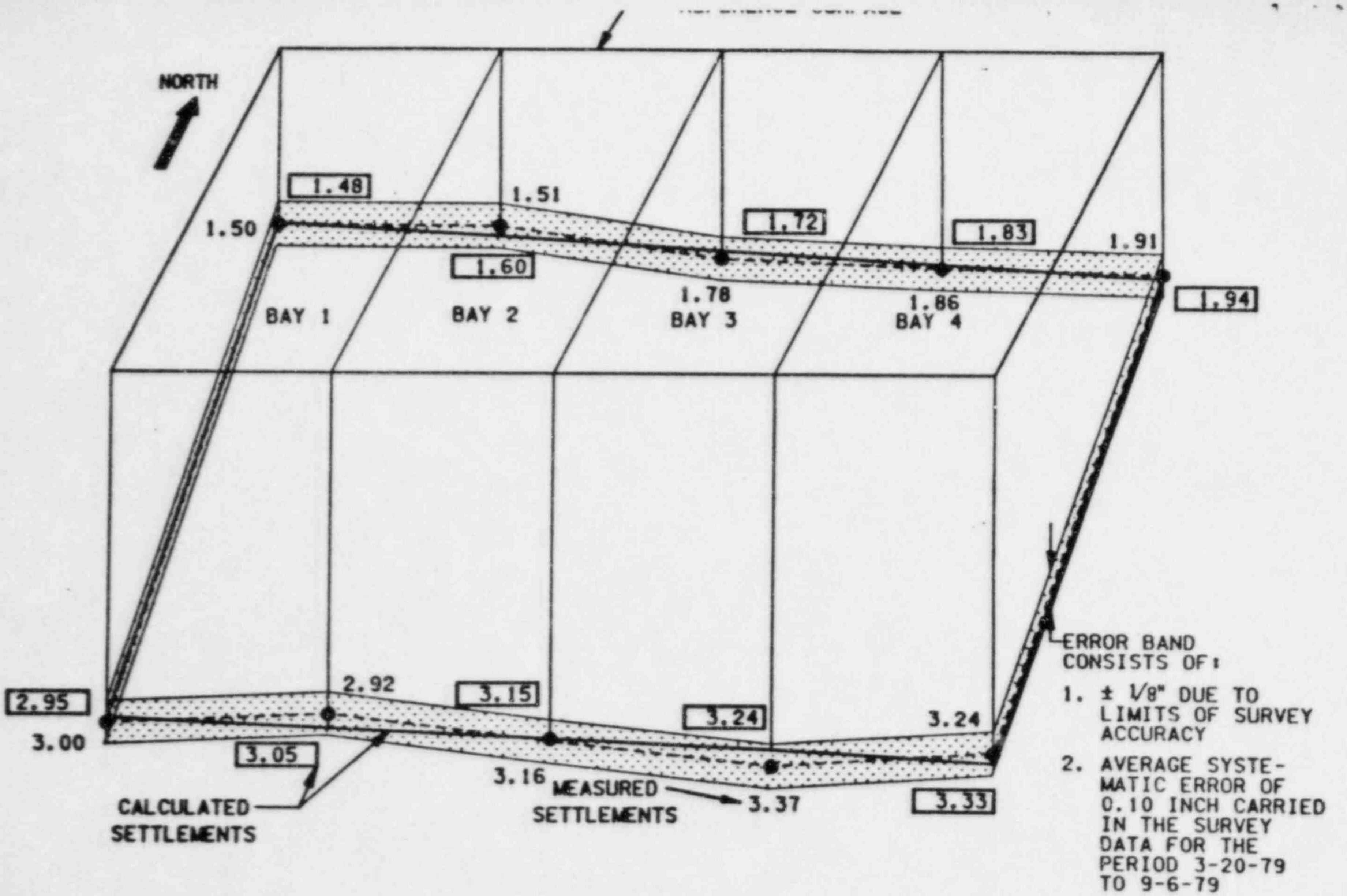
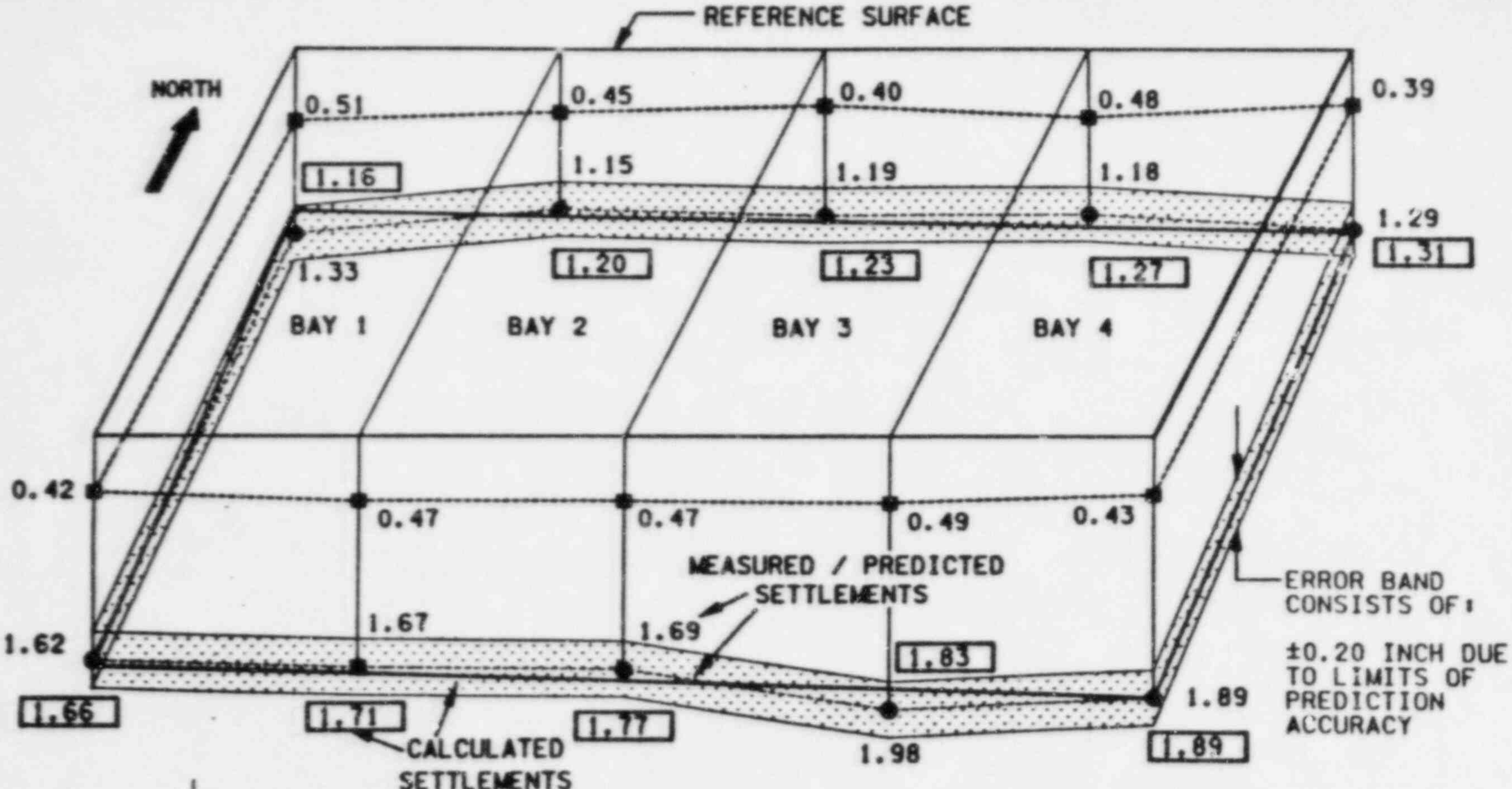


FIGURE 6

DIESEL GENERATOR BUILDING
EXECUTIVE SUMMARY

COMPARISON OF SETTLEMENT VALUES
SURCHARGE PERIOD
JANUARY 1979 - AUGUST 1979

FIGURE ES-16



ACTUAL MEASURED SETTLEMENT FROM SEPT.14, 1979 TO DEC.31, 1981. THESE INCLUDE EFFECT OF DEWATERING TO APPROXIMATELY EL. 595', AND REPRESENT MOVEMENT OF THE STRUCTURE DUE TO SETTLEMENT OF THE FILL AND NATURAL SOIL BELOW.

ACTUAL MEASURED SETTLEMENTS FROM SEPT.14, 1979 TO DEC.31, 1981 PLUS ESTIMATED SECONDARY COMPRESSION SETTLEMENT FROM DEC.31, 1981 TO DEC.31, 2025 ASSUMING SURCHARGE REMAINS IN PLACE.

FIGURE 7

DIESEL GENERATOR BUILDING
EXECUTIVE SUMMARY

COMPARISON OF SETTLEMENT VALUES
POST-SURCHARGE PERIOD
SEPTEMBER 1979 - DECEMBER 2025

FIGURE ES-17

APPENDIX I

COMPOSITION OF TASK GROUP

NRC Staff:	
Task Group Leader	Dr. Pao-Tsin Kuo, Section Leader Structural Engineering Section B Structural and Geotechnical Engineering Branch
	Dr. Chen P.Tan, Structural Engineer Structural Engineering Section B Structural and Geotechnical Engineering Branch
	Mr. Norman D. Romney, Structural Engineer Structural Engineering Section B Structural and Geotechnical Engineering Branch
NRC Consultants:	Dr. A. J. Philippacopoulos, Associate Scientist Structural Analysis Division Brookhaven National Laboratory (BNL)
	Dr. Charles A. Miller, Senior Consultant Structural Analysis Division Brookhaven National Laboratory
	Dr. Carl J. Costantino, Senior Consultant Structural Analysis Division Brookhaven National Laboratory

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

AUG 8 1983

MEMORANDUM FOR: C. P. Tan
Norman Romney
Structural Engineering Section B
Structural and Geotechnical Engineering Branch, DE

THRU: George Lear, Chief
Structural and Geotechnical Engineering Branch, DE GL 8/5/83

FROM: P. T. Kuo, Structural Engineering Section B Leader
Structural and Geotechnical Engineering Branch, DE

SUBJECT: EVALUATION OF LANDSMAN'S CONCERNS REGARDING DIESEL GENERATOR BUILDING AT MIDLAND

Reference: Memorandum from R. H. Vollmer to D. G. Eisenhut, dated July 21, 1983

Per the enclosed memo from R. H. Vollmer to D. Eisenhut, a task group to re-evaluate the structural design and construction adequacy of the Midland Diesel Generator Building has been formed and I have been designated as the leader of the group. You are assigned as members of this group. The mission of the group is described in the enclosure.

P. T. Kuo
Structural Engineering Section B Leader
Structural and Geotechnical
Engineering Branch, DE

Enclosure: As stated

cc: w/o enclosure
R. H. Vollmer
J. P. Knight
G. Lear

ENCLOSURE

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

JUL 2 1 1983

MEMORANDUM FOR: Darrell G. Eisenhut, Director
Division of Licensing

FROM: Richard H. Vollmer, Director
Division of Engineering

SUBJECT: EVALUATION OF LANDSMAN'S CONCERNS REGARDING DIESEL GENERATOR BUILDING AT MIDLAND

Responding to your memorandum, subject as above dated June 27, 1983, J. Knight, Assistant Director for Components & Structures Engineering, has formed a task group to re-evaluate the structural design and construction adequacy of the Midland Diesel Generator Building. The group, headed by Dr. P. T. Kuo, will review the design review documents and the construction reports; physically inspect the building; search out and interview concerned individuals, including Mr. Landsman; and prepare a final report on the adequacy of the Midland NPP Diesel Generator Building. The particulars of the groups' composition and charter are developed in more detail in the attached document. Note that we intend to use a consultant in a capacity to critique our findings on Mr. Landsman's concerns. The consultant's views will be provided in our report.

Richard H. Vollmer, Director
Division of Engineering

cc: H. Denton
J. Knight
J. Keppler
T. Novak
E. Adensam
G. Lear
P. Kuo
F. Rinaldi
D. Hood

IMPLEMENTATION CONCEPT

REVIEW OF THE MIDLAND NPP

DIESEL GENERATOR BUILDING

1. MISSION

A review will be conducted as to the structural adequacy of the Midland NPP diesel generator building. All information available from NRC regional inspectors in this matter will be obtained and the impact of that information will be fully considered in the review.

2. BACKGROUND

The NRC structural engineering staff (headquarters) has reviewed the Midland NPP diesel generator building's engineering design and construction and has indicated that the building is structurally adequate to resist its design loads. However, during hearings before a NRC Congressional Oversite Committee, the structural adequacy of the Midland NPP diesel generator building was questioned by an NRC employee, Mr. Ross Landsman, a Region III site inspector for the Midland project. It is considered prudent that a review be undertaken by a technical group to assure that Mr. Landsman's concerns are fully heard and carefully evaluated so that the adequacy of the diesel generator building may be further assured.

3. ORGANIZATION

The review group is composed of four technical members -

a group leader, two team members from the structural review staff and a structural consultant. The consultant will be asked to provide his critique of Landsman's concerns and our findings directly into the final report.

4. SUPPORT

The NRC structural review staff will provide the background technical studies, reports, and other review materials that formed the basis for their review and technical conclusions. The NRC project staff for the Midland NPP will provide general administrative arrangements to facilitate the review. Region III will provide a complete listing of Mr. Landsman's concerns.

5. SCOPE OF EFFORT

The efforts of the review group may include but will not be limited to 1) review of all pertinent technical materials, 2) on-site inspection of the diesel generator building, 3) on-site interviews with all inspection personnel that have information to contribute and 4) preparation of a technical report summarizing their activities, considerations and findings. The report will include, as a separate attachment, the opinion of the consultant group member.

6. TIMING

Review activities should be completed NLT 30 working days after receipt of a written statement of Mr. Landsman's concerns and the final report will be due to the Director, DE NLT 15 working days after completion of the review.

7. DESIRED PRODUCT

The desired final report of the review is a report that discusses each of Mr. Landsman's concerns, as well as any other concerns that might be offered during the review, and provide a basis for acceptance or rejection of each concern. A technical review of the adequacy of the diesel generator building should then be presented that is reflective of the groups' final recommendations in this matter in light of new information furnished by Mr. Landsman and others.

APPENDIX II

SUMMARY OF MEETINGS

August Meeting with Applicant and Site Visit

On August 24, 1983 members of the Task Group met with Bechtel and Consumers Power Co. staff in the Bechtel, Ann Arbor, Michigan offices. At this meeting, presentations were made by the applicant and their consultants to provide background on the history of the DGB construction original design philosophy and the analyses done to demonstrate the adequacy of the structure following settlement.

On the evening of August 24 and during the morning of August 25, 1983 the members of the Task Group visited the Midland site to observe the DGB. The Task Group members observed the cracks in the DGB and held discussions with construction personnel to determine the sequence of concrete placement during construction of the DGB. At the site crack maps of the DGB were provided by the Applicant.

Task Group Interviews With Original Reviewers

On September 8, 1983 the Task Group met individually with the original NRC staff reviewers responsible for the Geotechnical and Structural Engineering evaluation of the Midland DGB. The persons interviewed were: Dr. Harry Singh of the U.S. Army Corps of Engineers, Chicago (geotechnical engineering consultant); Mr. Joseph Kane of the Geotechnical Engineering Section, SGEB; Dr. Lyman Heller, Geotechnical Engineering Section Leader, SGEB; Mr. Frank Rinaldi, Structural Engineering Section B, SGEB, Mr. John Matra, Naval Surface Weapons Center, (structural engineering consultant); and Dr. Gunnar Harstead, Harstead Associates (structural engineering consultant. The purpose of the interviews was to gain an understanding and/or clarification of the concerns each reviewer had regarding the Midland DGB.

Dr. Harry Singh was retained by the Geotechnical Engineering Section after discovery of the soils problems existing at the Midland site. Dr. Singh was concerned that the structural analysis of the DGB did not take into account the settlement data as measured. Dr. Singh was concerned with the appropriateness of using crack widths to evaluate rebar stress due to settlement; although he did recommend that the cracks should be monitored as a measure of the DGB's structural adequacy. Generally, Dr. Singh expressed his opinion that the cracks in the DGB were much more extensive than one sees in normal concrete work. Dr. Singh is of the opinion that the DGB is in secondary settlement and that future long term settlement would be about 1-1/4 inches over 30-40 years.

The primary concern of Mr. Joseph Kane involved the Applicant's assumption of a straight line, rigid body motion in the structural evaluation of the effects of settlement on the DGB. Mr. Kane was of the opinion that the settlement values measured by the applicant are appropriate to use in the structural analysis because the building did settle as the soil conditions would have indicated (i.e., nonuniform). Furthermore, Mr. Kane was not concerned about the accuracy of the settlement data because they are the best data available from the Applicant and were more appropriate to use than to assume straight line settlement. With regard to the structural analyses using actual settlement data, Mr. Kane observed 70-80% of the cracks to be in areas where the analyses indicated areas of high stress. Mr. Kane has documented his concerns in memos dated August 2, 1983 and are included in Attachments 1 and 2.

Dr. Lyman Heller met with the Task Group to express his concurrence with the concerns expressed by Mr. Kane. Dr. Heller also offered an explanation as to why cracks were observed in areas where the analyses of the DGB indicated low stresses. The explanation offered was that the settlement of the concrete forms (i.e., yielding) during the pour created discontinuities in the finished concrete which served as preferred paths for the development of cracks.

Dr. Gunnar Harstead, Mr. John Matra and Mr. Frank Rinaldi were interviewed together. Mr. Rinaldi, Mr. Matra and Dr. Harstead maintained that use of the measured settlements would be inappropriate given the accuracy between survey measurements of + or - 1/8". Such inaccuracies in the survey data would result in unrealistic concrete stresses. Mr. Matra discussed the finite element models he prepared and executed for various stages of construction using the settlement measurements as inputs.

He indicated that there was not sufficient settlement data points to make a reasonable stress analysis. To obtain the required input, Mr. Matra stated that he linearly interpolated between the measured settlement data points. As expected there was extremely high stress in areas where no cracks in concrete were observed. Both Dr. Harstead and Mr. Matra mentioned that stresses depended on higher order derivatives. These higher order derivatives cannot be determined accurately from the five measured data points. Mr. Rinaldi indicated the most appropriate method of estimating rebar stresses due to settlement was to estimate stresses from crack widths. This method produced rebar stresses of about 5 ksi which when added to the stresses from the controlling load cases was less than the 54 ksi allowable. Mr. Rinaldi described the crack monitoring program the Applicant agreed to (0.05 /10' as alert limit and 0.06" or 0.020"/10' as action limit). Finally, Mr. Rinaldi and Mr. Matra indicated that the controlling load case for the DGB was tornado depressurization which assumed the DGB to be unvented which is conservative considering the building is vented. Mr. Rinaldi documented his response to Landsman's concerns in a memo in Attachment 3.

Task Group Audit of Design Calculation

The Task Group visited the Bechtel, Ann Arbor, Michigan offices on September 12 and 13, 1983. The purpose of the visit was to conduct an audit of the structural design calculations of the Midland DGB.

On Monday, September 12, 1983 the NRC Task Group reviewed the following DGB calculations:

- concrete/rebar stresses using settlement data by Karl Wiedner;
- straight line (rigid body) settlement by Karl Wiedner;
- concrete/rebar stresses assuming the DGB is supported at four points;
- stress totals from all load combinations;
- finite element modal for DGB.

On Tuesday, September 13, 1983, the NRC Task Group discussed with Dr. Mete Sozen the calculations he did on rebar stresses estimated from concrete crack widths. Dr. Sozen had made calculations estimating rebar stresses from crack widths for the center cross wall only. A call was made to Mr. Rinaldi in Bethesda to verify how he made his calculations on the other walls. Mr. Rinaldi indicated he did the same type of analysis using Dr. Sozen's approach for other walls. However, Mr. Rinaldi did not document the details of his analysis.

Landsman Interview

The Task Group interviewed Dr. Landsman on September 13, 1983 for about 3 hours. Dr. Landsman discussed each of his concerns at length. During the interview, potential resolution of the problem of the DGB cracks was discussed. DR. Landsman agreed that stresses determined from analysis of crack widths would be acceptable, provided that:

(1) these calculations were sufficiently documented; and
(2) an acceptable crack monitoring program was specified and implemented.

A copy of Dr. Landsman's memo of July 19, 1983 documenting his concerns on the Midland Diesel Generator Building is included as Appendix IV.

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 planned 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

Summary of Sept. 8, 1981 Meeting on Seismic Input Parameters   Midland Plant
USNRC 12/3/81

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

Final Results of Soil Boring and Testing Program for Perimeter and Baffle Dike Areas 7/27/81
Docket Nos. 50-329 50-330
Consumers Power Company

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

Transcript of Proceedings USA/NRC 1/9/81
Deposition of Pao C. Huang

Transcript of Proceedings USA/NRC Docket Nos. 50-329 OM, OL 50-330 OM,OL
Deposition of John P. Matra 1/7/81

USA/NRC Before the Atomic Safety and Licensing Board Docket Nos. 50-329 OM-OL
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 9 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)

ATTACHMENT 1

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

MEMORANDUM FOR: George Lear, Chief
Structural and Geotechnical Engineering Branch
Division of Engineering

THRU: Lyman Heller, Leader
Geotechnical Engineering Section
Structural and Geotechnical Engineering Branch
Division of Engineering

LWH

FROM: Joseph Kane, Senior Geotechnical Engineer
Geotechnical Engineering Section
Structural and Geotechnical Engineering Branch
Division of Engineering

SUBJECT: REVIEW OF REGION III REACTOR INSPECTOR'S CONCERNS REGARDING THE DIESEL GENERATOR BUILDING AT MIDLAND

In response to your verbal request of July 27, 1983 I am providing my comments on the July 19, 1983 memorandum prepared by R. B. Landsman on his concerns for the Diesel Generator Building. Since many of the concerns covered in the July 19, 1983 memorandum had previously been expressed in the ASLB hearing sessions of December 6-10, 1983, I have attempted to identify the specific transcript pages where these issues were discussed. Hopefully this listing of transcript pages will permit the interested reviewer in recognizing and evaluating the similarities and differences with both my previously expressed views and those of GES Consultant, the U.S. Army Corps of Engineers, and those views now provided by Dr. Landsman.

Joseph D. Kane

Joseph D. Kane, Senior Geotechnical Engineer
Geotechnical Engineering Section
Structural and Geotechnical
Engineering Branch
Division of Engineering

Enclosure:
As stated

cc: See page 2

cc: w/enclosure
R. Vollmer
J. Knight
G. Lear
P. Kuo
L. Heller
E. Adensam
T. Sullivan
D. Hood
F. Rinaldi
H. Singh, COE
R. Landsman, Region III
J. Harrison, Region III
W. Paton, OELD
J. Kane

Review Comments of

Joseph Kane

Diesel Generator Building Concerns

at Midland

Reference - July 19, 1983 Memorandum, From R. B. Landsman thru J. J. Harrison to R. F. Warnick, Subject: Diesel Generator Building Concerns at Midland.

1. First Concern - The problems and limitations inherent in the finite element analysis completed by CPC because of the effects of cracks and CPC interpretation of settlement data.

   Comment: To the best of my understanding and recollection the statements expressed in this first concern are accurate. I am in agreement with these statements except for the sentence "It is this time dependent effect that was also not used in their model." It is not clear to me what is intended by "time dependent effect". If it means the effect of cracking that resulted because of settlements, then I would agree with the statement. If it implies that time dependent settlements were not considered, then I believe the statement is in error.

   Pertinent Transcript Pages - December 10, 19[illegible], Pages 11173 to 11203.

2. Second Concern - Problems with analysis performed by NRC Consultant, the U.S. Naval Surface Weapons Center, and statement that this analysis gave unacceptable results.

Comment: In my opinion it was very unfortunate that the study by NSWC was not provided to the NRC Staff who are affected by the study results in sufficient time to permit a full internal NRC review with opportunity for calm and deliberate discussions on its contents before this document was introduced by the Applicant into evidence before the ASLB. I personally have serious problems and questions with the NSWC report. I have not pursued my concerns with the NSWC report for two reasons. First, I was under the impression that all review issues related to the DGB had been fully addressed at the December 6 through 10, 1982 ASLB Hearing session and secondly, my understanding of the procedure used by NRC Structural Engineering Section to arrive at its conclusion as to the magnitude of the stresses induced by settlement (the crack analysis approach) does not rely on the results or conclusions of the NSWC study.

With respect to Dr. Landsman's stated second concern, I essentially am in agreement with his statements except I do not understand what is meant by the words "and this portion of the SSER should be stricken" which appears in the second sentence.

3. Third Concern - Crack analysis approach used by the Staff is not normal engineering practice.

   Comment: In response to examination questions from both OELD and ASLB, both Mr. Singh and I gave our views on the crack analysis approach. An important conclusion reached by Dr. Landsman, which is different from my position, is that the Staff's crack analysis to determine rebar stresses is unacceptable. I believe a review of the transcript records will clearly show that I did not make this conclusion on unacceptability because I feel it is outside my area of responsibility and expertise.

   Pertinent Transcript Pages - December 10, 1982, Pages 11187 to 11201.

4. Fourth Concern - Problems with relying on the crack monitoring program to evaluate stresses during the service life of the DGB.

   Comment: The hearing transcripts will show that neither H. Singh or myself was questioned on the acceptability of the crack monitoring program for the Diesel Generator Building. The discussions that did occur in the hearings were provided by CPC consultants and NRC Structural Engineering Section. It is my impression that technical specification details still need to be resolved with the Applicant on the crack monitoring program for the DGB. Some of the details to be resolved would include the actual method to be used in measuring the cracks and the requirements for jointly coordinating and evaluating both settlement and crack readings. I share the same concern as Mr. Landsman on the "lack of formulated corrective action to be taken when the allowed crack sizes are exceeded." In addition to Mr. Landsman's concern I have problems with the following aspects of the crack monitoring program which were worked out by NRC Structural Engineering Section and the Applicant.

a. The criteria on crack widths permitted under both the alert and action limits (December 10, 1982 transcript, page 11069) are not sufficiently restrictive to prevent potential sections of the DGB from experiencing cracks where tensile stresses in the reinforcing steel would be well above the allowable stress.

b. It is not clear what is intended by the wording "summation of the increase in all the crack widths...." as it pertains to both the alert and action limits. Are the crack widths identified in transcript page 11069 to be the increases that are permitted? Increase over what existing width and date?

c. A crack monitoring program may elect to select certain wall sections for more careful measurement of cracks but it should not fail to require reasonable surveillance on other portions of the structure. My understanding of the agreed upon monitoring program for the DGB is that it is limited to localized areas on the faces of three selected walls.

d. The decision to require crack monitoring at a frequency of once in five years after yearly monitoring for the first five years should not be made at this time. The decision to significantly increase the required monitoring interval should be withheld until the initial data and trends are known and evaluated.

ATTACHMENT 2

[illegible] 10/2/83
[illegible]
[illegible]

Comments on J. Matra's Study - Structural Reanalysis of DGB

The time frames for the phases of construction (e.g. pre-surcharge, during surcharge, etc.) have been selected for the convenience of major construction phases or events and to more accurately estimate the DGB's stiffness at these specific times when the effects of settlement are evaluated.

It is not clear why total settlements (Figs. 29, 34, 36, 38, 40 and 42) are being used to compute max. stresses and moments. It is my understanding that computed stresses and moments are only appropriate for the various time frames where the specific settlement increment for that time frame has been used. The comments provided in Tables 2, 3, 4, and 5 should not be comparing stresses and moments based on total settlements when checking for areas of cracking. Need to clarify this with NSWC and reexamine computed stresses and moments with available crack mapping. In several of the walls (See Table notes) there does appear to be correlation of cracks with high stress areas. Discuss w/ NSWC.

Tables 3 and 4 provide results of NSWC on various floor and roof elevations. Since crack maps for floors and roof are not provided in the NSWC, is it intended to check study results of stresses and moments against existing cracks by a site inspection or request for additional mapping?

Tables 5 and 6 when addressing the settlements on Figs 4 and 43 in causing high stresses and moments should recognize the settlements are predicted to the year 2025. The major portion of these settlements have yet to occur, therefore, a check for cracking due to these settlements can not be made at this time.

(over)

The elements in the F.E. analysis appear to be approx. 20' in length. What effect does this 20' length have on the results of the analysis (location of high stresses and strains) recognizing that it is assumed the strain is constant over this length. Could check by using smaller elements, e.g. 5' lengths.

Explanation for "out of plane" moment?

Explanation on how allowable axial load and allowable out of plane moment were established (e.g. Table 2) for the 30" wide wall that has reinforcing of # 8 bars, 12" O.C. in both H & V directions

Following discussions with NSWC, is there a need to set up site inspection to check areas of high stress and moment with visually observed areas of cracking

ATTACHMENT 3

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

MEMORANDUM FOR: P. T. Kuo, Section Leader
Structural Engineering Section B
Structural and Geotechnical Engineering Branch
Division of Engineering

FROM: Frank Rinaldi, Structural Engineer
Structural Engineering Section B
Structural and Geotechnical Engineering Branch
Division of Engineering

SUBJECT: R. LANDSMAN'S CONCERNS ON INTEGRITY OF DIESEL GENERATOR BUILDING AT MIDLAND SITE

Enclosed please find the initial response to R. Landsman's concerns on the integrity of the Diesel Generator Building at the Midland site, as prepared during a working meeting on July 28, 1983, by myself and our consultants, John Matra and Gunnar Harstead.

Frank Rinaldi

Frank Rinaldi, Structural Engineer
Structural Engineering Section B
Structural and Geotechnical
Engineering Branch
Division of Engineering

Enclosure: As stated

cc: H. Denton
D. Eisenhut
R. DeYoung
E. Christenburg
C. Bechhoefer
R. Vollmer
R. Warnick
J. Knight
G. Lear
J. Kane
R. Landsman
J. Matra
G. Harstead
F. Rinaldi

REPLY TO R. B. LANDSMAN'S CONCERNS ON THE STRUCTURAL INTEGRITY OF THE DIESEL GENERATOR BUILDING FOR MIDLAND NUCLEAR POWER PLANT

INTRODUCTION:

The structural engineering staff and their consultants have reviewed and evaluated the structural adequacy of the Diesel Generator Building (DGB) to determine the functionality of the DGB and compliance of the design to the structural engineering requirements of NRC for the licensing of a nuclear power plant.

The Midland Nuclear Power Plant (NPP) has had a number of technical reviewers throughout the licensing period, Construction Permit (CP) and Operating License (OL) stages.

This report concentrates on the period following the determination by Consumer Power Co. (CPCo) that the fill material under the DGB did not meet the design specifications and that remedial actions were necessary. The applicant, under advice of their consultants, surcharged the structure with approximately 30 feet of sand and implemented a permanent dewatering program to correct the poor soil conditions under the DGB. In addition, electrical ducts were discovered to be supported by a competent foundation and were structurally connected to the base of the DGB. This condition imposed new loads on the structure in addition to all other design loads (Dead Loads, Live Loads, Tornado Loads, Earthquake Loads, Temperature Loads), and the abnormal differential settlement loads. Considerable cracks developed as a result of these additional loads. In order to eliminate this condition, the duct banks were released, therby removing one of the abnormal loads.

The DGB is a reinforced concrete structure with three crosswalls that divide the structure into four cells. Each cell contains a 6 ft.-6 inch-thick concrete pedestal to support a diesel generator unit. The building is supported on continuous footings that are founded at el. 628 ft. and rest on backfill that extends down to approximately el. 603 ft. This rectangular boxlike structure covers an area of approximately 70 ft. by 155 ft. The exterior walls are 30 in. thick, and the interior walls are 18 in. thick. The foundations of the exterior and interior walls of the DGB consist of continuous reinforced concrete footings, 10 ft. wide and 2 ft. 6 inch thick, with their base at el. 628 ft. The walls rise from an elevation of 628 ft. (bottom of footing) to el. 690 ft. (top of roof slab).

Sections 3.8.3.4 and 3.8.3.5 of Supplement No. 2 to the Midland NPP Safety Evaluation Report summarize the NRC structural staff and consultants evaluation of the DGB. This document was modified during the (ASLB) hearing of December 10, 1982, by the additional written testimony of Frank Rinaldi, Franz Schauer, John Matra, and Gunnar Harstead and all oral correction introduced by the same witnesses. The adequacy of the DGB is based upon many analyses, reviews, and monitoring requirements which address normal loads, settlement loads and postulated environmental loads. Due to the fact that available measured and predicted settlement data is not sufficiently refined to calculate structural component's stress by the use of a finite element analyses, the following quotations summarize the structural staff position for acceptance of the DGB:

(a) The NRC Staff believes the actual measured settlement values are the best characterization of settlement at the Midland site.

(b) The NRC Staff has not fully relied on these settlement values in any analyses to ascertain the acceptability of the DGB to withstand its design load over the lifetime of the plant. Instead, the Staff has looked at the current condition of the structure to estimate stresses due to settlement. To these it added stresses due to other design loads which are not presently on the structure but which have to be considered. The staff relied on Applicant's finite element analysis only for the latter stresses.

(c) The NRC Staff finds the DGB to be structurally acceptable.

(d) The NRC Staff is requiring a program of surveillance of the structure and for its foundation to ensure the continued safety of the structure.

(e) The NRC Staff takes no postion with respect to the acceptability of Applicant's finite element analysis of the DGB (as applicable to settlement effects).

(f) The NRC Staff's acceptance of the DGB is subject to the outcome of Seismic Margin Review.

Summary of Landsman's Concerns:

The concerns documented by R. Landsman regarding the DGB by his memorandum to R. F. Warnick, Director, Office of Special Cases, Region III, dated July 19, 1983, transmitted to D. G. Eisenhut, Director, Division of Licensing, NRR, by memorandum dated July 21, 1983, were received by the undersigned on July 27, 1983. This memorandum identifies, in general, concerns previously discussed by the staff during internal meetings and at the ASLB December 1982 hearings related to the DGB. The undersigned fail to understand why R. Landsman has not chosen to participate more fully during these meetings, or why he had not documented his concerns during the review process. The concerns identified in his July 19, 1983 memorandum in some cases are not clear, do not give specific reference to transcripts and other official documents, and in some cases, references to various statements are not fully correct. We will first summarize our understanding of his concerns and then address them in the following order:

FIRST CONCERN: Claim of inadequacy of the Finite Element (FE) Analysis performed by the applicant for the DGB as applies to the following:

(a) Effect of cracks on stiffness of DGB
(b) Validity of straight line settlement data
(c) Time dependency effects of settlements
(d) Corley statement on cracks and time dependency effects of settlement
(e) Staff's official position on FE analyses as stated by F. Schauer.

SECOND CONCERN:

(a) Claim that the analyses performed by NRC staff consultant (NSWC) is not properly documented in the SSER #2 based on their testimony at ASLB hearing.

(b) Claim that different analyses (Plastic) should have been used.

(c) Claim that F. Rinaldi stated that the staff cannot rely on the results of the NSWC analyses using actual settlement values.

THIRD CONCERN: Claim that the crack evaluation used to determine the stress in the reinforcing steel is not an adequate practical engineering approach.

FOURTH CONCERN: Claim that the crack monitoring program accepted by the staff to evaluate the rebar stresses during the service life of the building is not adequate.

SUMMARY: Recommendation for new remedial structural fixes required to ensure structural integrity and provide adequate margins of safety.

<u>Reply to Landsman's Concern:</u>

FIRST CONCERN

Part (a) In the design of reinforced concrete structures, the composite of concrete and rebars is modelled as homogeneous material with the concrete expected to crack under tensile loads. It is acceptable to assume concrete sections as uncracked for calculational purposes. The assumption of uncracked concrete neglects both the expected cracks and the stiffness of reinforcing bars which are compensating effects in the calculation of stiffness. Also, a reduced stiffness would reduce moments and forces due to settlement, therefore, reducing some conservatism from the structural analyses.

In conclusion, we find the design practice of neglecting the cracks in an analysis of the reinforced concrete structure is acceptable. Note that extensive crack evaluation efforts have been carried out by the applicant and their consultants and by the staff and our consultants, to determine the effects of cracks on the structure.

Part (b) The direct use of settlement data can give results which can be used to develop indications of the state of stress in the structure. The applicant used what they considered the best practical approach to determine the effects of the measured displacements on the structure, based on the available number of measured points and on the accuracy of the measurements.

The DGB is a stiff structure. The characterization of the boundary conditions used in the analyses should be consistent with that of a stiff structure; namely, linear. Also, settlement data has an inaccuracy inherent in the readings. The applicant's engineers claimed to have an accuracy no better than 1/8". Bending moments are proportional to the second derivative of displacement with respect to length and shear is proportional to the third derivative of displacement with respect to length. A mathematical error analysis shows that the accuracy diminishes with subsequent differentiation. Therefore, the accuracy of the moments and shears will be unreliable if the raw settlement data is used. Structural engineering judgment must be exercised in the formulation of the models and in the evaluation of the results.

The applicant performed many of the analyses to represent various stages of construction, including a completed model, a 40-year life-model and a model using no soil support in an area where we could not rely on the competence of the soil.

Attempts to directly use the raw settlement data resulted in anomalies such as tension in the soil and moments and forces in the structure that cannot be justified by prudent engineering judgment, analyses, and observations of the structure.

In conclusion we state that the use of the straight line or other representation using the available settlement data cannot produce credible results.Therefore, the staff did develop a conservative estimate of the state of stress of the structure based on the crack-evaluation and added these results to the stress levels for the environmental loads as per code requirements. However, we like to point out that several loads (DL, LL, T) were added twice. Also, the controlling load combination is the one with the tornado load. The applicant did not account for venting of the structure in their analysis, but the drawings and site visits indicated that considerable venting is provided. We like to point out that these two factors add a great deal of conservatism to the results. In addition, the effects of future settlement was considered in the applicant analysis, but the staff will rely on the monitoring program.

Part (c) The fact that settlement took place over a period of time was accounted for in the applicant's and in NSWC's analyses. Settlements that took place prior to the completion of construction has less effect on the final stresses in the structure, for the following reasons:

a. The partially constructed structure is less stiff and, therefore, moments and forces were minimized

b. reinforced concrete that had not yet been installed could not be subjected to stresses resulting from previous settlement. We, therefore, find that the time dependent effect was used to our satisfaction.

Part (d) We recommend contacting W. G. Corley and request his direct comments to R. Landsman's in First Concern Part (d).

Part (e) F. Schauer did make the statement identified by R. Landsman during the ASLB hearing of December 10, 1982 (p. 11149). However, we suggest that R. Landsman read the cross-examination by the ASLB on page 11150 of the December 10, 1983 hearing to fully understand the staff position as stated by F. Schauer.

The answers provided on that page of the transcripts states that one cannot fully rely on all of the analyses, and that engineering judgment needs to be exercized.

Second Concern

Part (a) The summary report of the NSWC analyses was entered into evidence at the ASLB, December 10, 1982, hearing. It was discussed in detail by J. Matra and commented on by F. Rinaldi, G. Harstead, and F. Schauer. In summary, that report stated the following points:

1. The behavior of this structure as shown by the results of the analyses is inconsistent with respect to the actual observations in the structure as far as crack locations. (Not for duct bank impingement consideration).

2. Analyses of the partial structure, including duct impingement, resulted in very high stresses in the walls at the duct banks. With these stresses over twenty times yield, a great possibility of cracks in these areas existed. A comparison between the crack mapping survey at this time of construction (3/78 to 1/79) and the analyses are in good agreement as far as the location of structural cracks in the area of the duct banks are concerned. However, the analyses show that other areas of the DGB walls still have high stresses and in probability should also be cracked. But no cracks were observed in these areas.

3. In all cases where the duct banks have been released, the measured or predicted settlement values imposed on the analytical models resulted in very high stresses in areas where no cracks now exist. Thus, indicating that these settlement values as such were not seen by this structure.

4. Imposing the measured settlement values on a partially completed model, and then considering these values as part of the total settlement values for the completed structure, without considering the following effects:

   (a) redistribution of loads once yield is reached,

   (b) the relaxation effects,

   (c) the accuracy of the measured data, and

   (d) the location of the measured settlement value relative to the footings where the actual displaccd values were input are discussed, but not actually input into the analysis,

   can and does lead to large errors. Thus, this structure will never undergo the differential settlements as predicted nor the patterns of settlement indicated in the measured and or predicted settlements.

Also, as indicated in the reply to First Concern Part (b), the results indicate tension in the soil and moments and forces in the structure that cannot be accounted for using sound engineering practice.

The analyses indicated that the direct use of the limited number of actual measured settlement data in the engineering analyses cannot be used without proper structural engineering judgment. The analyses were used in selecting a crack monitoring point for the ervice life of the DGB (a location of high stress as per these analyses, but having no major cracks was selected).

Part (b) The elastic analyses performed by the applicant give correct and conservative indications of stress for non-settlement loads. This is concluded after having reviewed the structural model, the analyses and the results. If an elastic analysis shows a region of high bending moment such that reinforcing bar stresses exceed their yield stress, the section may then be considered plastic; i.e., increasing rotation will not increase moments or stresses. However, there is no indication of yielding rebars or spalling of concrete which would indicate that a portion of the structure has become plastic. In fact, the formation of plastic sections in a structure mitigates the secondary stress effects of conditions such as differential settlement. To state that "supposed areas of high stress, where cracks are not located, may not exist due to redistribution of loads," is inconsistent with the mechanism of redistribution of stresses.

Part (c) The claim that F. Rinaldi stated, "that the actual settlement values could not be relied upon to determine if the DGB meets the regulatory requirements" is not complete. The additional testimony clearly states that the applicant's analyses using linear settlement data were not fully relied upon in our evaluation. This is stated on pages 11084 - 11087 of the ASLB hearing transcripts, dated December 10, 1982. The staff performed an additional crack evaluation as stated in our written testimony presented on the pages following page 11086 of the above mentioned ASLB hearings. All stress levels were below code allowable. Therefore, we found the concrete cracking levels in the DGB, as reported by the applicant, acceptable. The proposed crack monitoring will provide controls over potential future crack-patterns.

Third Concern

The evaluation of cracks as performed by the Staff is not a structural analysis, but rather a method of estimating upper bound stresses in the rebars of an existing reinforced concrete structure. These values were used as conservative values for stress due to differential settlement, shrinkage and other secondary effects. These stresses were conservatively added to total stresses developed by the applicant.

The structural analyses of the DGB were performed by the applicant considering all load combinations as documented in their report, "Structural Stresses Induced by Differential Settlement of the DGB."

The results are documented in the additional written testimony. See transcripts for the ASLB hearing of December 10, 1982.

The DGB is not a complex structure, instead, it is a simple box-like structure. Also, all reinforced concrete structures have cracks and we disagree with the statement that "there is no practical method available today to analyze a complex structure with cracks in it." Note that the applicant's structural consultants and our structural staff and their consultants have performed several evaluations of the DGB without finding any unresolved concerns.

Fourth Concern

The DGB was not accepted by the staff soley by relying on a crack monitoring program. On the contrary, the acceptance was based upon reviews of the analyses and designs prepared by the applicant as well as independent calculations. Furthermore, the stresses caused by settlements are secondary stresses. Secondary stresses are defined as those stresses which can exist in a structural material which do not impair that capability of the structural material to carry primary stresses, provided the secondary stresses do not cause rupture or gross distortions of the structural material. From a variety of evaluations, the indications are that the stresses in the reinforcing bars are well below yield and far from rupture. The compressive stresses in the concrete are very low. There are no indications of gross distortions of the structure. Therefore, the cracks that have occurred merely indicate that the reinforcing bars will carry imposed tensile forces while imposed compressive forces will cause the cracks to close. While there are no expectations of rupture or gross distortions in the future, a crack monitoring program has been established to provide engineers with information to assess the condition of the structure, as a prudent measure.

The criteria for the monitoring program is identified as ASLB exhibit #29. It contains specific requirements for Alert and Action levels for the monitoring of single and collective crack widths.

Reply to Summary:

It is surprising that, with all of the data and information available on the subject of DGB there still exists such a misunderstanding. Beyond this response we would respectfully direct R. Landsman to evaluate all of the information currently available in the field of structural analysis and specifically to that available in the docket of the Midland project.

It is our conclusion that all analyses, designs, crack mapping and evaluations and the monitoring program are adequate to establish the structural integrity of the DGB. Only unexpected results during the monitoring program would necessitate a reassessment of the DGB.

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