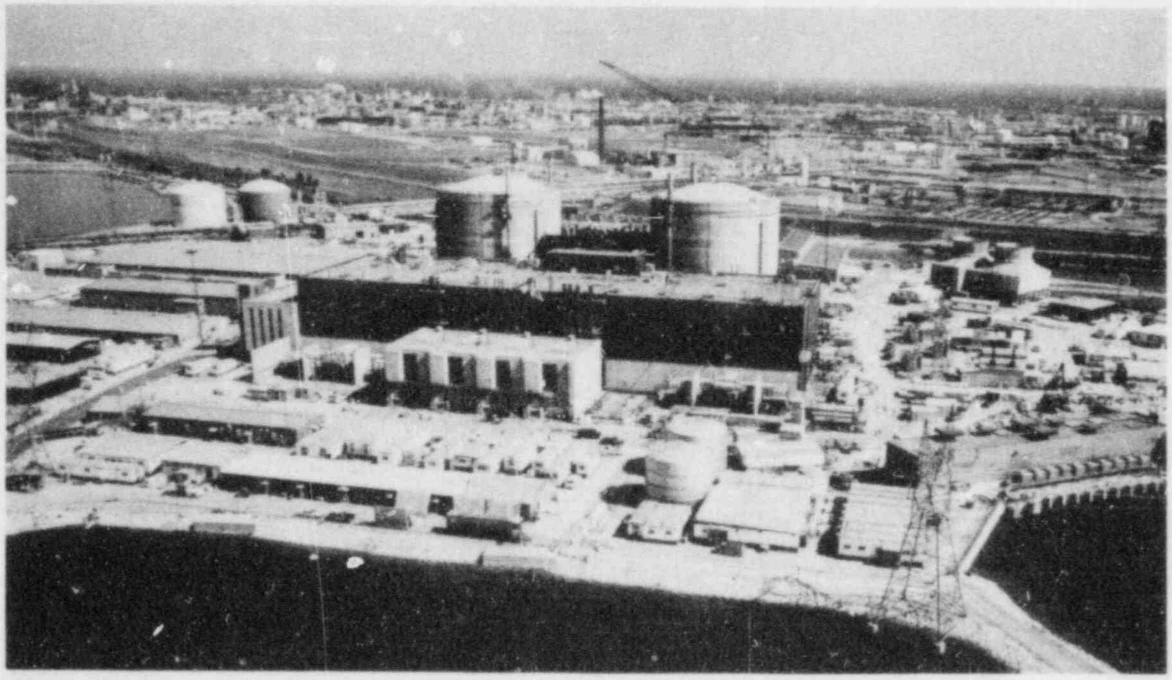
United States Nuclear Regulatory Commission
Docket Numbers 50-329 & 50-330

Midland Independent Design and Construction Verification Program

Structural Evaluation of the Diesel Generator Building



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January 4, 1983

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Re: Docket Nos. 50-329 OM, OL and 50-330 OM, OL
Midland Nuclear Plant - Units 1 and 2
Independent Design and Construction Verification (IDCV) Program
Structural Evaluation of the Diesel Generator Building -
Assessment of the Structural Performance Capability as
Potentially Affected by Settlement Induced Cracking

Gentlemen:

Attached is our recently completed engineering evaluation of the structural performance capability of the diesel generator building. This evaluation was undertaken in accordance with the defined scope of the IDCVP as part of our broader assessment of the quality of the design and constructed product of the Midland plant Standby Electric Power system. We are transmitting it to you because of its relevance to ongoing discussions concerning the potential effects of settlement induced cracking on the capability of the DGB to meet intended performance requirements over its service life.

We have concluded that the existing cracks, generally being of small size, are not indicative of a condition that would compromise the capability of the DGB in meeting its intended performance requirements. Furthermore, it is judged that significant future cracking is unanticipated and the DGB is expected to remain serviceable without further remedial action at this point in time. We have

reviewed Consumers Power Company's commitments to verify continued serviceability and have concluded that these are acceptable; however, we have offered certain recommendations for consideration that are intended to improve available information and reduce operational constraints.

Should you desire further articulation of the bases for our conclusions, we would welcome the opportunity for discussion.

Sincerely,

Howard A. Levin

Howard A. Levin
Project Manager
Midland IDCV Program

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ENGINEERING EVALUATION COVER SHEET

TITLE Structural Evaluation of the Diesel Generator Bldg.

CONT. ID. NO. 3201-001-031

NO. OF SHTS. 42

PROJECT: CONSUMERS POWER COMPANY MIDLAND IDCV

SUPERSEDES ENG. EVAL. NO.

REV. NO.	REVISION	ORIGINATOR	DATE	REVIEWED BY	DATE	APPROVED BY	DATE
0	Original	Clu	12/30/83	GES	12/30/83	[illegible]	1/4/84

☐ PRIMARY EVALUATION ☒ SUPPORTING EVALUATION

TOPIC NUMBER III.5-2, III.6-2, III.7-2

TOPIC TITLE Civil/Structural Design Considerations, Foundations, Concrete/Steel Design

METHOD/EXTENT OF REVIEW

1. Review of Midland project generated information including calculations, consultant reports, testimony, etc.
2. Independent calculations and evaluations by IDCVP Review Team.

PURPOSE

Evaluation of settlement induced cracking as it may potentially affect intended performance requirements and serviceability of the diesel generator building.

CONTENTS (SEE SECTION 2., PI-3201-001)

- ☒ ABSTRACT
- ☒ OVERVIEW OF REVIEW PROCESS
- ☒ BASES FOR SAMPLE SELECTION
- ☒ SOURCES OF INFORMATION/REFERENCES
- ☒ BACKGROUND DATA (INPUTS, ASSUMPTIONS, RELATED CALCULATIONS)
- ☒ ACCEPTANCE CRITERIA (CODES, STANDARDS, FSAR, NRC GUIDANCE, REGULATIONS)
- ☒ EVALUATION (DOCUMENTATION OF REVIEW AGAINST CHECK LIST, (ACCEPTANCE CRITERIA)
- ☒ CONCLUSIONS

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STRUCTURAL EVALUATION OF THE DIESEL GENERATOR BUILDING - ASSESSMENT OF THE STRUCTURAL PERFORMANCE CAPABILITY AND SERVICEABILITY AS POTENTIALLY AFFECTED BY SETTLEMENT INDUCED CRACKING

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1.0 ABSTRACT

An engineering evaluation has been completed to assess the structural performance capability and serviceability of the Midland plant diesel generator building (DGB) as potentially affected by settlement induced cracking. The evaluation was initiated by TERA Corporation as part of the Midland Independent Design and Construction Verification Program (IDCVP). The performance requirements for the DGB were identified and the acceptance criteria for meeting these requirements were reviewed. Information generated by the Midland project as well as independent calculations and evaluations by the IDCVP review team serve as input to the conclusions of the engineering evaluation. It was concluded that the existing cracks, generally being of small size, are not indicative of a condition that would compromise the capability of the DGB in meeting its intended performance requirements.

Furthermore, it was judged that significant future cracking is unanticipated and the DGB is expected to remain serviceable without further remedial action at this time. Consumers Power Company (CPC) commitments to verify continued serviceability were reviewed and found to be acceptable. Certain recommendations have been offered for consideration that are intended to improve available information and reduce operational constraints.

2.0 OVERVIEW OF REVIEW PROCESS

This engineering evaluation was undertaken as part of a broader assessment of the quality of the design and constructed product of the Midland plant Standby Electric Power (SEP) system. The specific scope of review documented herein includes a structural evaluation of the diesel generator building (DGB), the structure which houses four emergency diesel generators which are principal components of the SEP system. The main emphasis of the review is on the civil/structural design considerations for the DGB and how settlement induced cracking may potentially affect the intended performance requirements. Accordingly, this evaluation addresses the following topics within the Midland IDCVP:

- Topic III.5-2 - Civil/Structural Design Considerations
- Topic III.6-2 - Foundations, and
- Topic III.7-2 - Concrete/Steel Design;

therefore, representing partial fulfillment of the structural design review scope pertaining to SEP system. This evaluation has required input from other ongoing topic reviews such as:

- Topic III.1-2 - Seismic Design/Input to Equipment, and
- Topic III.2-2 - Wind and Tornado Design/Missile Protection;

however, these evaluations are documented under separate covers. The DGB construction/installation documentation reviews and the associated physical verification have not been completed and are not documented in this evaluation. Accordingly, should the results of these evaluations affect the conclusions drawn herein, the engineering evaluation will be appropriately revised.

The review concept includes a determination of the DGB performance requirements and important design inputs (i.e. engineering data and assumptions); an evaluation of their accuracy, consistency, and adequacy; and an evaluation of

the implementation of these commitments. Current licensing criteria are utilized as a baseline as well as consideration of various other regulatory criteria which evolved during the licensing process. Given the unique circumstances associated with the DGB design and construction processes, the IDCVP assessment used the intent of today's licensing criteria and corresponding margins of safety and reliability.

The review draws upon two principal sources of information; that generated by the Midland project (e.g. Bechtel calculations, consultant reports, testimony, etc.) and by the IDCVP review team (e.g. independent calculations and evaluations, etc.). Pertinent background data and references are documented in Section 3.0. Conclusions are reached through an integrated assessment of these data, discussions with Midland project personnel, as well as engineering judgement.

The following individuals made technical contributions to this engineering evaluation:

Dr. Jorma Arros	-	Structural Reviewer, Midland IDCVP and Senior Structural Engineer, TERA Corporation
Dr. William J. Hall	-	Member Senior Review Team, Midland IDCVP and Professor of Civil Engineering, University of Illinois
Professor Myle J. Holley	-	Consultant, Midland IDCVP, Professor of Civil Engineering Emeritus, Massachusetts Institute of Technology and President, Hansen, Holley and Biggs, Inc.
Mr. Howard Levin	-	Project Manager, Midland IDCVP and Manager, Engineering, TERA Corporation
Dr. Christian Mortgat	-	Lead Technical Reviewer, Standby Electric Power System Structural Review, Midland IDCVP and Principal Structural Engineer, TERA Corporation

The following chronology of external interactions transpired as part of this review.

Date	Activity
August 24, 1983	Review team members observe NRC task force meeting on structural rereview of DGB at Bechtel's Ann Arbor, Michigan offices.
November 17, 1983	Review team members inspect diesel-generator building.
November 18, 1983	Review team members discuss civil/structural design considerations for the DGB and collect information at Bechtel's Ann Arbor, Michigan offices.
December 12-16, 1983	Review team members review DGB finite element and seismic stick models at Bechtel's Ann Arbor, Michigan offices.

3.0 BACKGROUND DATA AND REFERENCES

The following table identifies references and sources of information that were selected for review and served as input to this engineering evaluation. The numbers in the left margin correspond to references made within the body of the engineering evaluation.

REFERENCES/SOURCES OF INFORMATION

TOPIC TITLE Civil/Structural Design Considerations, Foundations, Concrete/Structural Steel   TOPIC NO. III.5-2, III.6-2, III.7-2   PAGE 1 OF 3

ENGINEERING EVALUATION Structural Eval. of the Diesel Generator Bldg   CONT. ID. NO. 3201-001-031   REV 0   DATE 12/30/83

	ORIGINATING ORG./ AUTHOR	IDENTIFICATION/ NUMBER	REV.	DATE	TITLE	WHERE/HOW LOCATED	DOCUMENT TYPE
1.	Bechtel	File 0485.16/B13.3 Serial 22423	48	5/83	Final Safety Analysis Report	Ann Arbor	FSAR
2.	NRC	50-329/330	0	10/21 83	Report on the Review of the Diesel Generator Building - Midland	Docket	Report
3.	Bechtel	--	0	8/24/ 83	Midland Units 1 and 2 Diesel Gen. Bldg. Exec. Summary	Ann Arbor, 11/18/83 Meeting	
4.	Wiedner	testimony at pp. 10804-11007	0	9/8/ 82	Testimony of Karl Wiedner for the Midland Plant Diesel Gen. Bldg.	Docket	Testimony
5.	CPC	File 0485.16, B3.0.3, Serial 17228	0	6/1/ 82	Technical Report-Structural Stresses Induced by Differential Settlement of the Diesel Generator Bldg.	CPC, Jackson	Report
6.	CPC		3	9/79	Response to NRC regarding Plant Fill	Docket	Report
7.	ACI	ACI 318-77			Building Code Requirements for Reinforced Concrete	Library	Standard
8.	ACI	ACI 349-76			Code Requirements for Nuclear Safety Related Concrete Structures	Library	Standard
9.	TERA	PI-3201-009	3	7/15/ 83	Engineering Program Plan	IDCVP Proj. Files	Project Instruction
10.	Sozen	--	0	2/11/ 82	Eval. of the Effect on Structural Strength of Cracks in the Walls of the Diesel Generator Building	Transcript at 10950	Report
11.	Peck	Testimony at p. 10180	0	12/6/ 82	Testimony of Ralph Peck	Transcript at 10180	Report
12.	Corley, et. al.	--	0	4/19/ 82	Effects of Cracks on Serviceability of Structures at Midland Plant	Transcript at 11204	Report
13.	CPC	FSAR Ch. 16	45	9/82	Tech. Spec. 16.3/4.13 Settlement Monitoring	Ann Arbor	FSAR
14.	CPC	Exhibit 29R	0	--	DGB Areas for Crack Width Monitoring During Operation of the Plant	Ann Arbor, 11/18/83 Meeting	Partial/ Corres.
15.	Bechtel	DQ-52.0(Q)	2	9/9/ 83	Diesel Gen. Bldg. Reanalysis Using Revised Settlement Load Case	Ann Arbor	Calc

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REFERENCES/SOURCES OF INFORMATION

TOPIC TITLE Civil/Structural Design Considerations, Foundations, Concrete/Structural Steel TOPIC NO. III.5-2, III.6-2 III.7-2 PAGE 2 OF 3

ENGINEERING EVALUATION Structural Eval. of the Diesel Generator Bldg. CONT. ID. NO. 3201-001-031 REV 0 DATE 12/30/83

	ORIGINATING ORG./ AUTHOR	IDENTIFICATION/ NUMBER	REV.	DATE	TITLE	WHERE/HOW LOCATED	DOCUMENT TYPE
16.	Bechtel	DQ-52.1(Q)	1	8/2/82	DGB Settlement Analysis - Load Case 1A	Ann Arbor	Calc
17.	Bechtel	DQ-52.2(Q)	0	5/12/82	DGB Settlement Analysis - Load Case 1B	Ann Arbor	Calc
18.	Bechtel	DQ-52.3(Q)	1	9/28/83	DGB Surcharge Condition (2A)	Ann Arbor	Calc
19.	Bechtel	DQ-52.4(Q)	0	6/28/82	DGB Settlement for 40 yr Life (2B)	Ann Arbor	Calc
20.	Bechtel	DQ-52.6(Q)	1	9/7/83	DGB Analysis for Uniform Torsion	Ann Arbor	Calc
21.	Bechtel	DQ-52.7(Q)	1	9/7/83	DGB Anal. Imposing 40 yr displacements	Ann Arbor	Calc
22.	Bechtel	DQ-12(Q)	1	4/15/83	DGB Reanalysis (including cracking of concrete)	Ann Arbor	Calc
23.	Bechtel	DQ-52.0-C7(Q)	0	5/18/82	Optcon ACI-349 - Nonseismic Load Cases 7-18 - Diesel Gen. Bldg. Settlement Analysis (partial)	Ann Arbor	Calc
24.	Bechtel	DQ-52.0-C2(Q)	0	5/4/82	DGB Load Combination (partial)	Ann Arbor	Calc
25.	Bechtel	DQ-52.2-C5(Q)	0	5/12/82	DGB Settlement Analysis - Load Case 1B - Free Body Analysis of Trial #3 (partial)	Ann Arbor	Calc
26.	Bechtel	DQ-52.3-C7(Q)	0	9/28/83	DGB - Settlement Case 2A - Free Body Analysis on Best Fit (Surcharge) (Partial)	Ann Arbor	Calc
27.	Bechtel	DQ-52.4-C4(Q)	0	5/12/82	DGB Analysis - Free Body Analysis of Best Fit 40-Year Case	Ann Arbor	Calc
28.	Bechtel	DQ-23-C4(Q)	0	12/11/81	DGB Roller Support (FSAR Criteria)	Ann Arbor	Calc
29.	Bechtel	S-110	1	11/11/82	Static & Dynamic Spring Constant of DGB for Structural Stress Anal.	Ann Arbor	Calc
30.	Bechtel	S-175	3	2/22/82	Update of Settlement Prediction DGB - After Surcharge Removal	Ann Arbor	Calc
31.	Bechtel	S-238	0	7/15/82	Settlement of DGB Between 9/14/79 and 12/31/2021	Ann Arbor	Calc

REFERENCES/SOURCES OF INFORMATION

TOPIC TITLE Civil/Structural Design Considerations, Foundations, Concrete/Structural Steel TOPIC NO. III.5-2, III.6-2, III.7-2 PAGE 3 OF 3

ENGINEERING EVALUATION Structural Eval. of the Diesel Generator Bldg CONT. ID. NO. 3201-001-031 REV 0 DATE 12/30/83

	ORIGINATING ORG./ AUTHOR	IDENTIFICATION/ NUMBER	REV.	DATE	TITLE	WHERE/HOW LOCATED	DOCUMENT TYPE
32.	Bechtel	SQ-147(Q)	1	5/2/83	Seismic Analysis of DGB and DG Pedestal	Ann Arbor	Calc
33.	Bechtel	DQ-52.11(Q)	0	9/9/83	DGB: Reduction of Settlement Data (Partial)	Ann Arbor	Calc
34.	Bechtel	SK-C-2343-1/24	F	12/5 83	Settlement Data for DGB	Ann Arbor	Drawing
35.	CPC	File 0485.16 Serial 18371	0	8/2/82	Midland Concrete Wall Repair Program	Att. 2 to Testimony of Corley @ p.11206	Letter
36.	NRC	50-329/330	0	9/29/81	Trip Report - Midland DGB Structural Design Audit	Docket	Report
37.	Neville	Lib. of Congress No. 72-12066	1	1975	Properties of Concrete	Library	Text Book
38.	TERA	3201-003-007	0	12/30/83	Structural Analysis of Diesel Generator Building	IDCVP Proj. Files	Calc
39.	Bechtel	DQ-14(Q)	1	8/12/83	Response to NRC Question 26 Re: Diesel Generator Building	Ann Arbor	Calc
40.	Bechtel	DQ-23(Q)	1	10/11/83	DGB Reanalysis (in support of public hearing testimony)	Ann Arbor	Calc

4.0 ACCEPTANCE CRITERIA

4.1 LOAD COMBINATIONS

The loads and load combinations employed for the original design and analysis were provided in the FSAR subsection 3.8.6.3 (revision 0, dated November 1977). These original design criteria did not contain settlement effects. Four additional loading combinations were established and committed for consideration as a result of Question 15 of the NRC Requests Regarding Plant Fill of September 1979. These loading combinations combined differential settlement with long-term operating loads and either wind or the operating basis earthquake (OBE). As Wiedner (reference 4) and CPC (reference 5) point out these expressions are more stringent than the requirements of ACI 318 (reference 7), but less stringent than ACI 349 (reference 8). In the latter case the loading combinations combine differential settlement with extreme loads such as tornadoes and the safe shutdown earthquake (SSE). Subsequently, in response to Question 26 of the NRC Requests Regarding Plant Fill, a commitment was made to undertake a separate structural reanalysis of the DGB in accordance with ACI-349 as supplemented by NRC Regulatory Guide 1.142 for comparison purpose only.

The following loads were considered in the reanalysis:

(a) dead loads (D)

(b) effects of settlement combined with creep, shrinkage and temperature (T)

(c) live loads (L)

(d) wind loads (W)

(e) tornado loads (W')

(f) OBE loads (E)

(g) SSE loads (E')

(h) thermal effects (T_o)

It is to be noted that thermal effects appear twice by virtue of the manner in which the loading combinations were developed. The load combination established and committed to in response to NRC Requests Regarding Plant Fill, Question 15 are as follows:

a. $1.05\ D + 1.28\ L + 1.05\ T$

b. $1.4\ D + 1.4\ T$

c. $1.0\ D + 1.0\ L + 1.0\ W + 1.0\ T$

d. $1.0\ D + 1.0\ L + 1.0\ E + 1.0\ T$

A number of load cases appearing in the load combinations for Seismic Category I structures listed in FSAR Subsection 3.8.6.3 do not occur in the diesel generator building and other load combinations can be eliminated from the analysis after comparison with more severe loads or load equations (reference 5). As a result the remaining load combinations to be considered are:

e. $1.4\ D + 1.7\ L$

f. $1.25\ (D + L + W) + 1.0\ T_o$

g. $1.4\ (D + L + E) + 1.0\ T_o$

h. $0.9\ D + 1.25\ E + 1.0\ T_o$

i. $1.0\ (D + L + E') + 1.0\ T_o$

j. $1.0\ (D + L + W') + 1.0\ T_o$

4.2 ALLOWABLE MATERIAL LIMITS

In accordance with regulatory requirements, the maximum rebar tensile stress allowed in the diesel generator building rebar should not exceed $0.90\ f_y$ (where f_y equals yield strength) for computation of section capacities. Because the diesel generator building rebar has an f_y value of 60 ksi, the maximum allowable tensile rebar stress due to flexural and axial loads is 54.0 ksi. Accordingly, reinforced concrete section capacities for the diesel generator building were based on this

maximum allowable rebar stress value (54 ksi), a design concrete compressive strength of 4000 psi and a maximum allowable concrete compressive strain level of 0.003 in./in.

5.0 BASES FOR SAMPLE SELECTION

The diesel generator building (DGB) was selected for review because it serves an important support function in providing protection against external hazards for the diesel generators which are integral components of the Standby Electric Power (SEP) System. The DGB falls within the sample selection boundaries defined in the Engineering Program Plan (reference 9). Commitments were made in this reference to review civil/structural design considerations for the DGB including foundations and concrete/steel design. Based on programmatic commitments, emphasis is to be placed on structural performance and not detailed soil mechanics aspects which are not within the scope of the Midland Independent Design and Construction Verification Program (IDCVP).

This engineering evaluation addresses the potential effects of settlement induced cracking on the ability of the DGB to meet its intended performance requirements. Accordingly, verification of the Midland project treatment of the settlement/cracking issues which have affected several structures at the Midland site is addressed herein. While a structural review of the auxiliary building is also within the IDCVP scope as part of the Auxiliary Feedwater (AFW) system review, the specific settlement/cracking issue as it may affect the auxiliary building is not being treated directly by the IDCVP. Thus, this evaluation of the DGB represents the IDCVP sample addressing the settlement/cracking issues.

It is estimated that approximately one third of the project's calculations and evaluations addressing the structural design of the DGB were selected for review. Emphasis was placed on the selection of portions of the project's evaluations that address controlling design conditions (e.g. important load combinations producing the highest predicted stresses or strains, as appropriate). Principal project consultant reports were reviewed as well as other docketed information that documents CPC commitments to the NRC (see section 3.0).

6.0 ENGINEERING EVALUATION

6.1 BUILDING PERFORMANCE REQUIREMENTS

The diesel generator building (DGB) is a two story reinforced concrete box type building partitioned into four bays, each bay containing one diesel powered electric generator (see Figure 6-1). The purpose of the diesel generators is to supply standby electrical power to operate the Midland plant during power outages and to provide the necessary power to ensure safe shutdown of the plant in the event of a design basis event. Accordingly, the diesel generators and the DGB are classified as Seismic Category I, and as a result must maintain functionability during external events such as earthquakes and tornadoes.

The DGB provides protection for the diesel generators and associated supply and service lines, instruments and equipment, assuring ready availability of this supplementary power source. This protective function includes not only the normal sheltering of building contents from rain, snow, wind, and ice, but in addition, resistance to the effects of earthquakes and tornadoes including tornado generated missiles. It is these latter effects which are of principal structural interest, and which dictate a more massive type of construction than normally would be employed for shelter from the commonly considered weather extremes.

The DGB was founded on plant fill and constructed between the Fall of 1977 and the Spring of 1979. During that period it was discovered that the building was experiencing an unusual rate of unequal settlement, and duct banks had made contact with the footings which led to building distortion and reinforced concrete cracking. The details of the settlement monitoring, duct structural modifications, and surcharge consolidation program are described in detail in references 3 and 5.

6.2 ACCEPTANCE CRITERIA

In response to applied loadings (dead, live, earthquake-induced, wind, tornado, tornado missiles) and certain secondary effects such as settlement, local internal forces are developed throughout the structure. These local forces consist of in-plane forces, sometimes termed membrane forces, and out-of-plane forces, i.e., transverse shear forces, and bending moments. In design it is customary for the internal forces associated with a particular loading to be multiplied by a specified "load factor" and these load factored sets must be combined for the several specified loadings to obtain what may be called a local internal demand. This demand must not exceed the local "strength", i.e., capacity of the structure. The acceptance criteria consists of the following:

- Statements of the several different load combinations that must be satisfied, and the load factors to be applied to each of the loadings (dead, live, tornado, etc.) within that combination.

- Specific expressions, or procedures, for determining the local strength which must not be exceeded.

It may be noted that certain of the specified load combinations focus on serviceability of the structure. These do not include the infrequent extreme loadings, but incorporate relatively large load factors to assure a modest demand/capacity ratio for (unfactored) loadings experienced in normal operating conditions. For the combinations which include extreme and rare loadings, safety in the sense of protecting personnel and equipment, yet retaining functionability, is the primary consideration rather than serviceability. Thus crack widths, including those widths which may reflect yielding of the tension rebars, are not a consideration provided that they do not imply a reduction in the local strengths. Accordingly, such specified factored load combinations typically incorporate smaller specified load factors. In effect a larger demand/capacity ratio for these unfactored load combinations is acceptable for these rare conditions.

It should be noted that the specified expressions, or procedures, for determining the local internal strength do not typically include any direct limitation on rebar tensile strain, or on crack widths which accompany such strain, although there are indirect limitations for certain conditions. (Note that the limiting condition specified by various ACI codes (references 7 and 8) are related to maximum allowable concrete compressive strains where a value of 0.003 in./in. is specified). This strain reflects the fact that certain components of local strength are not sensitive to rebar strain but only to the tensile yield strength of the rebars. As an example, full development of the local out-of-plane bending strength of a slab, or beam, with a modest rebar ratio may imply tensile rebar strain into the yield range. Indeed this is specifically recognized by codes which specify that, for rebar strains in excess of the elastic strain at yield stress the stress must be assumed to be constant at the yield stress value. This approach often is overlooked because, for the majority of local conditions of interest it is computationally much more convenient to evaluate local sections on the assumption that the steel strains remain within the elastic range, and to compute rebar stresses associated with the particular factored load combination demand rather than to compute the local section strength, per se. In some cases this approach is slightly conservative, but often there is no difference whatever. However, the fact that there are circumstances, where small tensile rebar strains into the yield range occur, yet are acceptable, and do not degrade the required local strength, may be unrecognized because of the focus on elastic behavior inherent in the computation process. Margins of strength, as reflected in codes, are implicitly based on the ductile behavior of structural systems as just noted.

6.2.1 Structural Primary Loadings

The DGB must resist the following principal primary loadings:

- Gravity- induced dead and live loads
- Earthquake- induced loads
- Tornado- induced differential air pressure
- Tornado- borne missiles

Gravity- induced loads produce out-of-plane shear forces and bending moments in the floor and roof systems and in portions of the walls immediately adjacent thereto. These loads also produce in-plane forces in the walls and, of course, bending moments and shear forces in the strip footings.

Earthquake- induced loads produce in-plane forces in the walls which are substantial, and more modest in-plane forces in floor and roof slabs. They also produce out-of-plane shear forces in floor and roof slabs and walls.

Tornadic winds produce in-plane and out-of-plane forces in walls and roofs. Tornado- induced differential air pressures are the principal source of out-of-plane shear forces and bending moments in floor systems and walls, and they also produce in-plane forces.

Tornado- borne missiles produce highly localized out-of-plane loading of the walls. The capacity of the wall to resist such missiles is evaluated independently of all other loadings.

6.2.2 Secondary Loadings

Restrained non-load-induced volume changes (e,g., due to concrete shrinkage and or temperature strains) may produce internal forces. It has long been recognized that these forces rarely have any significant effect on the local strengths, and in most cases they are neglected. The reasons relate directly to the ductility of the tension rebars. If the local strength is mobilized, by an imposed set of local demand forces, it typically will be the same whether or not the forces associated with the non-load induced effects are included. The difference will be that the tensile rebar strain,including some yield strain, will be larger when these secondary forces are included. This yielding has the effect of decreasing, and sometimes completely eliminating, the local forces which were initialy introduced by the non-load effect. It is for this reason that the forces associated with such non-load induced effects often are termed "self-relieving" or secondary.

In the design of most reinforced concrete buildings the local internal forces arising from restrained shrinkage and thermal strains as well as that induced by settlement are not included in the application of the strength criteria. In the design of nuclear safety related concrete structures it is the accepted practice to account for through-the-wall thermal gradients, although shrinkage effects are not typically included. Even accounting for the thermal gradients is a conservative requirement the justification for which is at least debatable. However they were accounted for in the DGB design as required by the acceptance criteria. It may be noted that underlying codes, from which the acceptance criteria were developed, typically called for inclusion of these non-load-induced forces with the load-induced forces only where their structural effects may be significant. In the case of the DGB it may reasonably be debated whether such effects are indeed "significant", as envisioned by the code.

In the initial design of the DGB it would not reasonbly have been assumed that the forces associated with foundation settlement could be significant nor, that they should be included with the load-induced forces in the factored load combinations. Clearly, the building was designed for continuous support on what was intended to be a relatively homogeneous soil medium. Thus the designer could justifiably assume that there would be little if any redistribution of the upward soil reactions on the strip footings due to major point-to-point variations in local stiffness of the supporting medium. When the building was only partly completed it became evident that such stiffness variations did, in fact, exist i.e., a very stiff support at the location of footing contact with ducts, together with poorly consolidated soil (low in stiffness, and non-uniform) elsewhere. These conditions caused an extreme example of non-uniform settlement which did indeed induce internal forces sufficient to cause cracks in the walls of the then partially completed structure.

Upon noting that the settlement had led to interference between the foundation and buried ducts, the unintended footing-to-duct connections were physically disengaged and the unsatisfactory foundation condition was corrected by a surcharge loading procedure. It is to be noted (reference 36) that the surcharge loading procedure began on January 26, 1979, incrementally, and that

construction of the DGB continued thereafter. The final surcharge placement took place between March 22, 1979 and April 7, 1979, just as the roof and parapet construction was completed. The subsequently completed DGB structure has been in place, in its completed condition for more than four years with no indications of additional distress in any way comparable to that associated with the footing-to-duct contact and the poorly consolidated soil. It may be argued that the structure now is supported as was intended at the time of design, that the effects of any future differential settlement will not be significant, and that the effects of such cracking as developed in the partially completed structure also are not significant to local internal strengths relied upon to resist the forces associated with applied load combinations. From all this it would naturally follow that the internal forces induced by differential settlements need not necessarily be included with the load-induced forces in the combinations specified by the acceptance criteria. These arguments may be justified but, in fact, there is a licensing commitment to include the settlement-induced forces in the relevant load combinations.

Since the internal forces induced by a specific non-uniform settlement are self-relieving (as was described earlier, for thermally induced forces), why must they be included; i.e., when may their effects be "significant". In some structures the magnitude of possible future settlement may be uncertain, and there may be little or no prospect for monitoring of the settlement or the state of the structure during its service life. Accordingly, inclusion of settlement-induced forces in the design would be appropriate to limit the possible development of structural distress which would be costly to repair, or which in some special cases, like a containment structure, may affect functionability by creation of large liner strains. For other structures these forces might prudently be included to avoid excessive yield strains in the tension rebars (and the associated large crack widths) which might degrade the local internal strength under some set of the local internal forces associated with applied loads, particularly if no monitoring of the structure for such effects could be anticipated.

For the DGB structure the principal structural elements are relatively accessible, and a monitoring program is planned. Nevertheless it is required to

demonstrate by application of the relevant acceptance criteria, including the effects of differential settlement, that the local internal strengths are not presently degraded and are unlikely to be degraded by any probable future differential settlements. The acceptance criteria do not include any specification of the method by which the associated internal forces are to be determined. This is an important consideration in any effort to apply the acceptance criteria. There are essentialy three alternatives:

a) One may assume a magnitude and distribution of differential settlement and impose this displacement pattern upon the structure. In contrast to the situation at the design stage the analyst for the DGB has settlement measurements to consider in arriving at the postulated differential settlements to be used.

b) One may postulate one or more perturbations of the distribution of upward soil reactions associated with dead load which may be associated with differential settlement, and determine the local internal forces for each. It will be apparent that this approach produces the forces due to dead loads plus differential settlement. This is not an unreasonable approach, if sufficient attention is given to parametric variations, particularly if the analyst lacks data on differential settlement which he considers sufficiently precise to use directly in method (a).

c) One may postulate the local internal forces directly from the observed condition of an (existing) structure; i.e., the crack widths in the DGB. This is an option clearly not available at the time of design.

The method of imposed differential settlements may lead to unrealistically large internal forces unless the analysis can account for cracking, and time-dependent concrete properties. The cost-benefit of such an analysis may not be justified, particularly if other suitable options (b or c) exist.

The method of analyzing the dead load condition for several postulated distributions of soil reaction is suitable, but it may be difficult to choose sets of distributions which cover the possible differential settlements but which are not unjustifiably extreme.

For the DGB, which has been observed in its completed state for more than four years, inference of the internal local forces from the condition of the existing structure (c) seems to be the most attractive approach. It is the most direct. It is particularly attractive since any significant changes in the condition of the structure will be observable during its service life. Observations related to this approach follow.

6.3 EVALUATION OF BUILDING PERFORMANCE CAPABILITY

The performance capability of the structure is to be assessed in two steps: the first one considering the building in its present state and the other addressing its structural integrity and serviceability over the next 40 years. Inputs to the evaluation are keyed to a number of elements such as: available physical data, analytical studies, understanding of concrete behavior and engineering judgement.

6.3.1 Available Data

The most important data available to estimate the present state of stress in the DGB consists of:

1. Observations of the building as it exists today.
2. The record of the crack monitoring program.
3. The settlement history of the building.

The cracks have been surveyed on several occasions (Reference 3). The maximum crack width recorded during the monitoring program prior to isolation of the duct banks was 28 mils. After the isolation of the duct banks, the cracks decreased in size (testimony Peck and Weidner references 11 and 4 respectively) implying a stress decrease in the higher stressed areas. Presently the largest cracks are of the order of 20 mils. An evaluation of the existing cracks has been performed by two Bechtel consultants, Dr. Mete Sozen (reference 10) of the University of Illinois and Dr. W. Gene Corley (reference 12) of the Portland Cement Association.

The building settlements have been monitored at close intervals during the construction period and thereafter. Figure 6-2 presents the location of the settlement markers indicating where survey measurements are taken. The data spans over a period of 5 years with measurements taken approximately every other week. This large amount of data allows one to follow the settlement history through the stages of construction, duct bank isolation, surcharge period, dewatering, and up through the present. It also provides a means of assessing potential random and systematic errors in the measurements. The Midland project has concluded that significant errors exist in the measurements due to a variety of circumstances. A study of these data is presented in the following section.

6.3.2 Midland Project Evaluations

The Midland project followed two separate approaches to estimate the state of stress in the building:

- study of the cracking history
- study of the settlement history.

The future state of stress due to settlement was estimated based upon predicted settlements.

6.3.2.1 Evaluation of DGB Based on Observed Cracking

In its present condition the DGB has cracks which appear to be settlement-induced or settlement-intensified, generally arising during the early construction phases. Maximum present crack widths are reported to be about 20 mils, and Dr. Sozen (reference 10) has shown that the associated rebar stress as estimated in a region of numerous cracks, adjacent to a duct bank penetration of the center wall, may be judged to be between 20 and 30 ksi. We find his evaluation to be reasonable incorporating techniques that are state of the art, widely accepted and supported by laboratory tests. Dr. Sozen also has argued that the presence of initial cracks does not degrade the capacity of a reinforced concrete element

in any of the important structural modes; i.e., direct tension force, direct compression force, in-plane shear force, and out-of-plane bending. Again, we agree with Dr. Sozen that precracks of the width thus far evidenced in the walls of the Midland DGB would not significantly degrade capacities in the several modes developed by the principal loadings, and in their required factored combinations.

Dr. Sozen did not specifically address the possible influence of an initial rebar stress which is associated with a self-relieving internal force, that is, a force caused by foundation settlement. He does not indicate his opinion whether or not the self-relieving internal force implied by the initial rebar stress should be included with the internal forces due to applied loadings or can be neglected because it is self-relieving. It is our understanding that the Bechtel evaluations of the DGB for the effects of dead load plus foundation settlement did not utilize the initial rebar stress magnitude estimated by Dr. Sozen but rather computed it based on the settlement history of the building.

6.3.2.2 Evaluation of DGB Based on Settlement History

The settlement effects were modeled by Bechtel into the structure considering four distinct time periods. Measured or estimated settlement values corresponding to each of the time periods were used:

- Case IA: 3/28/78 to 8/15/78 (Structure partially completed to elevation 656.5') - A long hand calculation was used to determine the stresses due to early settlements. The structure was assumed fully cracked and the stresses in the reinforcing steel were assessed based upon local strains corresponding to an imposed differential settlement (reference 16).

- Case IB: 8/15/78 to 1/5/79 (Structure partially completed to elevation 662.'0.) - The duct banks were seperated from the structure which caused the north wall to settle rapidly. (reference 17)

- Case 2A: 1/5/79 to 8/3/79 (Structure in process of completion.)- Surcharge period. (reference 18)

- Case 2B: Forty year settlement composed of:

  - measured settlements from 8/3/79 to 12/31/81, and

  - predicted secondary consolidation settlement from 12/31/81 to 12/31/2025. (reference 19)

The last three analyses used a finite element model having stiffness corresponding to an uncracked condition. In these analyses the foundation stiffnesses have been varied, in an iterative process, to achieve final settlements approximating a set of target settlements. These target settlements were based upon a linear best fit through the measured settlement data. The analyses have been criticized (reference 2) because the analytically predicted settlements do not match variations in the measured settlements. It is appropriate to ask whether the iterated non-linear foundation stiffnesses are realistic since the target settlements were not the measured settlements but a linear best fit, essentially assuming rigid motion of the North and South walls. The best fit data were utilized in an attempt to deal with scatter in the measured data. Such scatter potentially due to either random or systematic errors was estimated to be of the order of plus or minus 0.125 inches.

In our opinion the described method of accounting for foundation stiffnesses utilizing the linear best fit data may not be satisfactory for correlation with observed cracking in relation to differential settlement. We concur that settlement measurements may not be of sufficient accuracy to permit a precision computation of settlement-induced internal forces. Furthermore, the marker locations are spaced at wider intervals than would be desirable as input to analyses of building strains. Nevertheless, the general level of stress implied by the magnitude of cracking is not in contradiction to that which may be derived from the measured settlement data, realistically accounting for flexibility including consideration of phenomena such as creep (see section

6.3.3 for a more detailed discussion). As discussed in Section 6.2.2, an exact determination of secondary stress levels is of lesser importance given the nature of the loading and the fact that capacity is not adversely affected.

In separate sensitivity studies Bechtel engineers considered among others, the two following cases:

- The zero spring condition analysis (reference 3) which investigated the structure's ability to span any soft soil condition. A zero soil spring value was used at the junction of the south wall and east center wall. Soil values were increased linearly back to their original value within a distance of approximately 15 feet from the zero spring. The stresses in the building underwent moderate increase in the area of the bridging. In our judgement this is a reasonable approach, but one may ask whether the size and locations of such postulated "soft" zones were bounding.

- The imposed 40 year settlement analysis (reference 2i) which forced the building to match the predicted settlement values at 10 points along the foundation. This analysis led to very large reaction forces at the points of imposed settlements, and some of these acted downward on the structure, i.e., implying tensions in the soil, which is not possible. Moreover, the analysis indicated very large rebar tensile stresses, where at several points a multiple of the yield strength was indicated. Of course the structure does not display the very wide cracks which would accompany such high stresses. For these reasons Bechtel engineers concluded that the settlement measurements cannot be an accurate representation of the actual settlement nonuniformities.

We have noted that the settlement data may not be an adequate basis for computing settlement effects. However, we believe the described analysis exaggerates the effects of the displacement input data which was questioned by the project. Our reasons are that the analysis assumed uncracked concrete and

used the short-term concrete modulus of elasticity. Appropriate reduction of the concrete modulus, to reflect creep under sustained loading, would have lead to reactions and internal forces perhaps 50 percent less than were obtained. Decreases in stiffness associated with concrete cracking could result in additional large reductions. An excellent discussion of the physical and engineering significance of creep is found in chapter 6 of reference 37.

Perhaps more important, rebar stresses appear to have been computed on the assumption that the local internal tensile forces developed in the uncracked concrete are unreduced by cracking, i.e., this unreduced force is imposed on the rebars. In our judgment this is not the best physical representation. The rebar stresses are expected to be more nearly indicated by the local strains in the concrete (uncracked) than by the forces in the concrete (uncracked). Thus, the rebar stresses are better approximated by the product of steel modulus and concrete strain (uncracked); i.e., by the product of modular ratio, n, (Youngs modulus of the steel/Youngs modulus of the concrete) and concrete stress.

$$f_s \cong n\, f_c$$

in contrast we believe that the following expression was used

$$f_s \cong \frac{1}{p} f_c$$

where p is the reinforcement ratio (rebar area/section area). This later expression greatly overestimates rebar stress. To illustrate, for p = 0.0043 and n = 8, the suggested approach gives rebar stress about 1/30 of the Bechtel computed value. While reality is likely in between, and the former expression is approximate, we believe that it is a closer representation of the existing situation.

6.3.3 IDCVP EVALUATIONS

In addition to reviewing the information generated by the project and the studies performed by others, the IDCVP concentrated attention on two major elements in the review process:

- Observations of the building and its present state of cracking, and
- The settlement history of the building.
  - Settlement data
  - Gross stress estimation

6.3.3.1 Building Inspection

A careful inspection of the building was performed together with a review of the crack mapping data. As it exists at present, many cracks of small size are evident in the building but there is no evidence to support that these cracks are indicative of a high state of stress in the building and degraded capacity. Past experience and laboratory tests indicate that concrete elements in a state of distress -particularly stiff shear walls of the type in the DGB - exhibit large deformations and cracks, much greater than present in the DGB. This would probably be accompanied by scabbing and other phenomena which are not apparent in the DGB.

Our conclusion from visual inspection of the building is that its state of stress is low and would not impair its performance and functionability. A body of relevant information developed in industry, university and government programs and structural experience supports this conclusion.

6.3.3.2 Settlement Data

A study of the settlement data recorded between 11/24/78 and 8/28/80 is presented in reference 5. We reproduced and expanded this analysis to include the most recent data (reference 38). The two time periods covered were from

5/12/78 to 9/14/79 (reference 33) and 9/14/79 to 8/23/83 (reference 34). Our goal was two fold: (1) assess the overall deformation of the building with time and (2) estimate the random error present in any one set of measurements. We studied the following data.

1. Cumulative settlement recorded overtime.

2. Incremental settlement between successive readings.

3. A measure of the curvature between any three consecutive markers along the foundation as it varies with time. The curvature d''_i at marker i is defined as:

   $$d''_i = 0.5\,(d_{i-1} + d_{i+1}) - d_i$$

   where d_i is the total settlement.

   The quantity d" equals zero when the three points are on a straight line; it remains constant in time if the three points move as a rigid body.

4. A measure of the deformation of the building with respect to its rigid body motion. The rigid body motion is "removed" by computing the vertical position of all markers with respect to the plane defined by three corner markers. This analysis was done both for each incremental reading and cumulatively.

An upper limit of the random error in any set of readings is given by the maximum difference of incremental settlement between any two markers from one reading time to the next. When the building has not experienced any settlement between two readings, this quantity is the random error; it bounds it otherwise. At the beginning of the record, this quantity is large where the building was undergoing large differential settlements and reading accuracy might have been reduced by marker transfer necessitated by the placement of surcharge. However, this quantity decreases rapidly and after June 1979 is never greater than 0.150". After the removal of the surcharge for the readings starting 9/19/79 which we will refer to as the recent readings, the random error is smaller than 0.125", 95 percent of the time which would give a random error of about ± 1/16 of an inch. This implies that a higher level of confidence can be given to the recent measurements.

Jumps in readings from one period to the next are sometimes large implying that the building would rapidly move up or down by a uniform amount. These jumps are attributed to systematic errors in locating the reference elevation.

Figure 6-3 shows the incremental settlement for 6 time periods between July 1978 and August 1979 for the south wall of the DGB. The first three measurements show large differential deformations and introduction of curvature in the wall. The latter ones show stabilization of differential settlements implying that the wall is still settling but as a rigid unit, introducing little additional in-plane bending. For more recent recordings the stabilizing trend is even more noticeable. Study of the foundation curvature variation and deformation of the building with respect to its rigid body motion point toward the same trend. This is supported by an evaluation discussed in reference 4, where it was noted that the settlements occurring during the time periods represented by lines c and d (reference 4, figure DGB-7), were those that are expected of a rigid body. In figure DGB-7, line c represents settlement during the surcharging period (1/79 - 8/79) and line d represents estimated settlement during the post-surcharge period (9/79 - 12/2025). The point here is that the early cracking occurred when the building was only partially completed. Upon completion, the five sided (four walls and a roof) structure is now responding as a stiffer, essentially rigid body as would be expected.

Hence during its construction stage, the building underwent substantial differential settlement that introduced in-plane curvature in the walls with resulting stress and cracking compounded with normal shrinkage cracking. As the building was completed and the concrete aged, its tended to behave more and more as a rigid unit, the whole foundation (or building) moving as a plane (or a unit). The recent data indicates that for the last four years the building has generally settled as a rigid body introducing relatively little additional distortion in the structure. We expect this behavior to persist in time.

One may speculate on the magnitude of the absolute settlements over the service life; however, these are of lesser structural concern to the building itself, and would only affect clearance to obstructions and connected items.

These latter elements can accommodate some degree of distortion and can be modified in the future if warranted.

6.3.3.3 Gross Stress Estimation

Even though we have noted that settlement data may not provide an acceptable basis for computing settlement effects, it is our opinion that if credit had been taken to account for:

- creep and stress relaxation in young concrete,
- reduced stiffness associated with the geometry of the uncompleted structure
- stiffness reduction due to cracking

the exact recorded settlement could have been imposed on the structure without generating stresses in gross contradiction to that observed via crack patterns in the DGB. This would have qualitative value to an overall understanding of building behavior.

In order to improve our understanding of building behavior and to generally qualify the influence of these effects, we modeled the north and south walls of the building using a simplified finite element model (reference 38). As a first order check of our partial model, we reproduced the 40 year imposed settlement analysis performed by Bechtel on the uncracked structure. We obtained stresses within 25 percent of Bechtel's which is reasonable considering the simplified model we used.

We imposed the recorded settlements on the incomplete wall for Case IA and IB and on the complete wall for Case 2B. For cracked concrete, the stresses were computed as described in Section 6.3.2.2.

The following approximate maximum values of stress were obtained:

LOADING	STEEL (ksi)
CASE IA	11.3
CASE IB	3.5
CASE 2A	4.6

This leads to a total stress of 19.4 ksi which is in good agreement with Dr. Sozen's independent analysis (see section 6.3.2.1 and reference 10).

We recognize that the above analysis represents a simplified approximation of the very complicated effects of creep and cracking but it provides a qualitative estimate of the state of stress of the building.

We believe the results of our analyses, properly interpreted are both useful and positive, specifically.

- When modified for the effect of concrete creep and concrete cracking the foundation reactions when combined with reactions due to dead load, would not imply a physically impossible state of tension stress in the soil.
- When the rebar tension stresses are properly determined, that is on the basis of strain in the uncracked concrete rather than on the basis of stress in the uncracked concrete, they are quite modest rather than unrealistically large.

6.3.4 IDCVP Assessment/Interpretation of Results

In our opinion the settlement-induced internal forces implied by the associated rebar stresses, as they presently exist in the Midland DGB will not degrade the capacities to resist the internal forces and moments caused by the factored load

combinations and therefore the DGB is expected to meet its intended performance requirements. There is reason to believe as supported by recent observations, that the completed building is settling as a rigid unit based upon the stabilized foundation properties. In this mode, the DGB capacity is not expected to be compromised over time. We believe that the settlement-induced, self-relieving, internal forces implied by the present crack widths and associated rebar stresses could safely be ignored in evaluating the building. However, licensing criteria include certain load combinations in which it is specifically required to include the settlement-induced internal forces. Based upon our knowledge of available margins associated with controlling load combinations, we believe that compliance with these criteria can generally be demonstrated, appropriately accounting for creep, relaxation and other phenomena; however, we do not endorse such an endeavor because of the secondary nature of the settlement induced loads and the fact that capacity is unaffected.

6.4 SERVICEABILITY, FUTURE CAPABILITY, AND MONITORING

The previous sections address the significance of settlement induced cracking on the performance capability of the DGB in its current condition. It is important that the DGB continue to meet specified performance requirements over its service life; hence, this section addresses serviceability of the DGB and any actions that may be necessary to identify and mitigate potential future conditions which could compromise the DGB performance.

6.4.1 Midland Project Evaluations and Commitments

The effects of cracks on the serviceability of Midland plant structures were addressed in reference 12. Three principal issues were evaluated:

- Freezing and thawing resistance,
- Chemical attack, and
- Corrosion of reinforcement

It was concluded in reference 12 that observed cracks are not expected to have a significant influence on the durability of the DGB. Accordingly, remedial measures such as epoxy injection were considered unnecessary to ensure long term performance capability. Nevertheless, CPC committed (reference 35) to repair existing cracks which are 20 mils and larger (up to a point in length where the crack remains 10 mils or larger) by epoxy injection and application of a concrete sealant to accessible surfaces.

A Technical Specification (TS) 16.3/4.13 (reference 13) has been proposed to monitor settlement over the service life of the DGB. The specification requires that the total settlement be measured (to nearest 0.01 foot) at least once every 90 days for the first year of operation. The frequency for subsequent years has been left for future determination. The total allowable settlement corresponding to predictions for the service life (12/31/81 thru 12/31/2025) has been specified at 12 markers. Engineering evaluations are required if total settlement reaches 80% of the allowable values (Alert Limit). Additionally, the inspection frequency is to be increased to once every 60 days if the 80% level has been reached.

If the DGB exceeds total allowable settlements, the plant must initiate actions to be in cold shutdown within 30 hours (Action Limit).

CPC has also committed to conduct a crack width monitoring program (reference 14) which includes individual crack width and cumulative crack width measurements at 3 locations over a 10 foot gage length. This program will be conducted once every year for the first five years of operation and at five year intervals thereafter. The following criteria apply:

	Alert Limit	Action Limit
single crack	50 mils	60 mils
cumulative cracks (over 10' gage length)	150 mils	200 mills

Identical actions as defined in T.S. 16.3/4.13 are required if these limits are reached.

6.4.2 IDCVP Assessment

We concur with the conclusions drawn in reference 12 relative to the influence of existing cracks on the performance capability of the DGB and its continued serviceability. While significant future cracking is unanticipated, it would only be in these circumstances that we would recommend remedial actions such as epoxy or sealant application to insure continued durability. Furthermore, should such procedures continue to be contemplated for purposes of potential increased protection, we urge that applications of any compounds not be made in such a manner as to mask surfaces so that cracks are not visually accessible. Notwithstanding the potential future inconvenience of removing compounds from selected surfaces, there is a potential that these compounds may influence behavior and modify surface expression of cracks, making future engineering evaluations more difficult.

We recommend that consideration be given to modifying T.S. 16.3/4.13. The following points summarize our evaluation and our recommendations.

- Visual inspection - The building should be examined visually twice a year in concert with an evaluation of settlement data to identify any unusual deviations in crack patterns and gross changes in dimensions. This may represent an additional commitment.

- Total allowable settlement - These limits should be based upon structural/mechanical performance requirements considering items such as the physical clearances to obstructions (e.g. duct banks) and permissible deflections for attached items (e.g. incoming fuel lines). Notwithstanding these considerations, absolute settlements and corresponding rigid body motion of the building is of minor concern to building performance capability other than as it might affect clearances to obstructions and connected items. The existing limits may trigger potentially unnecessary evaluations. A 90-day survey interval appears reasonable for the first year of operation. This approach may represent a redefinition of certain total allowable settlement limits.

- Differential settlement

  - Diesel Generator Building

    Forces induced by differential motion within the DGB are of interest, but generally only at a time at which crack width levels approach an order of magnitude greater than has been observed. Capacity is not expected to be degraded for settlement induced cracks with sizes up to this general level. Even at this point, the residual state of secondary stress in the DGB may be low due to factors discussed in Section 6.3; however, one must evaluate shear transfer mechanics across crack boundaries of dimensions of the same order as the fracture surface roughness. It is recommended for consideration that limits for differential motion between points within the DGB (discounting all rigid body components of motion) be specified such that these motions are correlated with potential future crack widths up to an order of magnitude greater than has been observed to date; thus providing functionally defined limits for differential movements. Remedial effort to protect external surfaces may be considered at approximately half these values. The program may include development of an initial set of data which would provide a baseline for potential future reference. Additional survey data would be collected in the future if indicated by the visual inspection program and absolute settlement measurement surveys. If adopted this approach may represent a redefiniton of allowable settlement limits and a restructuring of the proposed tech specs.

  - Diesel Generator Pedestals

    Although, relatively of lesser concern, at such a time as the diesel generators are run for an extended period, potential differential movement of the isolated diesel generator pedestals is of interest as such movement may affect connected lines. Accordingly, we endorse continued monitoring of pedestal settlement and comparison to functionally defined differential movements.

We conclude that the committed crack monitoring program will produce results which are of engineering interest but not necessarily of safety significance. Accordingly, we do not see a need to specify alert and action limits based upon

this program. We base this conclusion primarily on the limited number of locations to be monitored and the fact that appropriate locations are difficult to determine a priori, not knowing how the building will behave in the future. One could specify locations based upon predictions of future response, but if the building responds as predicted, this will be of less interest then if it does not, in which case alternate locations would be more desireable. This is related to our recommendation not to mask surfaces through application of new compounds.

In summary, we conclude that the performance characteristics of the DGB are not likely to be compromised over its service life. Various commitments have been made by CPC to verify continued serviceability. While we conclude that several of these commitments may not be totally necessary, we do not view that safety will be compromised by the specified actions. Certain improvements may be made which may produce valuable information and reduce operational constraints.

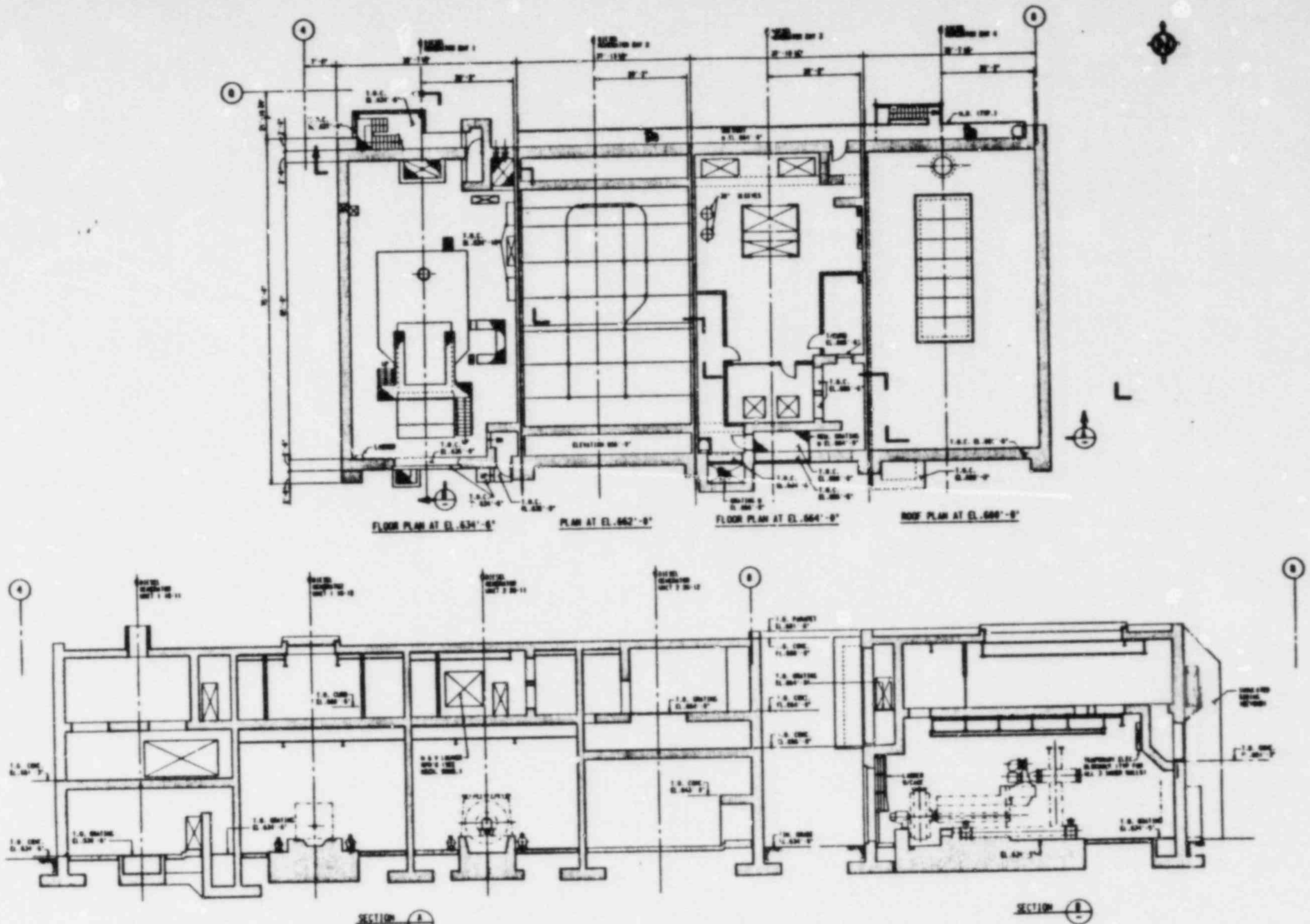

FIGURE 6-1

DIESEL GENERATOR BUILDING
PLAN VIEW AND SECTIONS

TERA CORPORATION

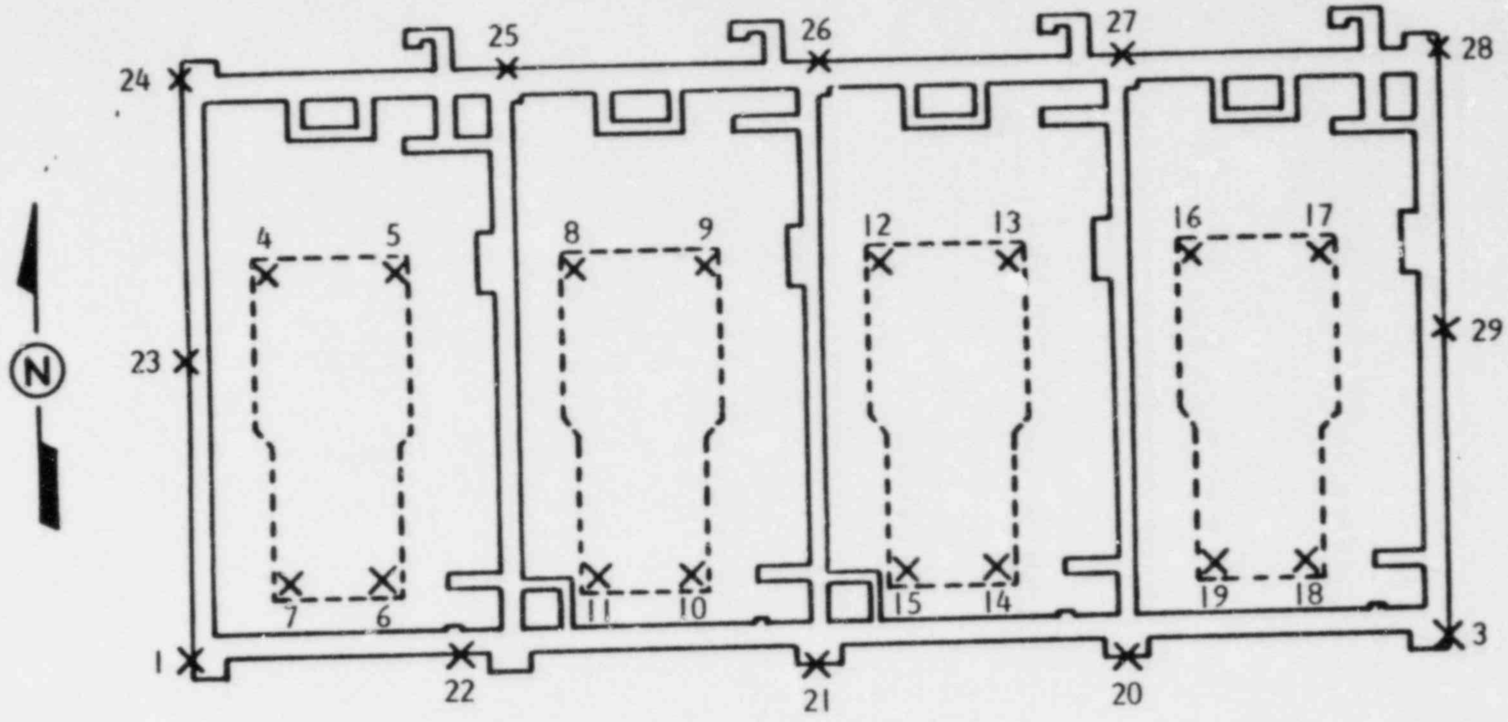

FIGURE 6-2

DIESEL GENERATOR BUILDING
SETTLEMENT MARKER LOCATIONS

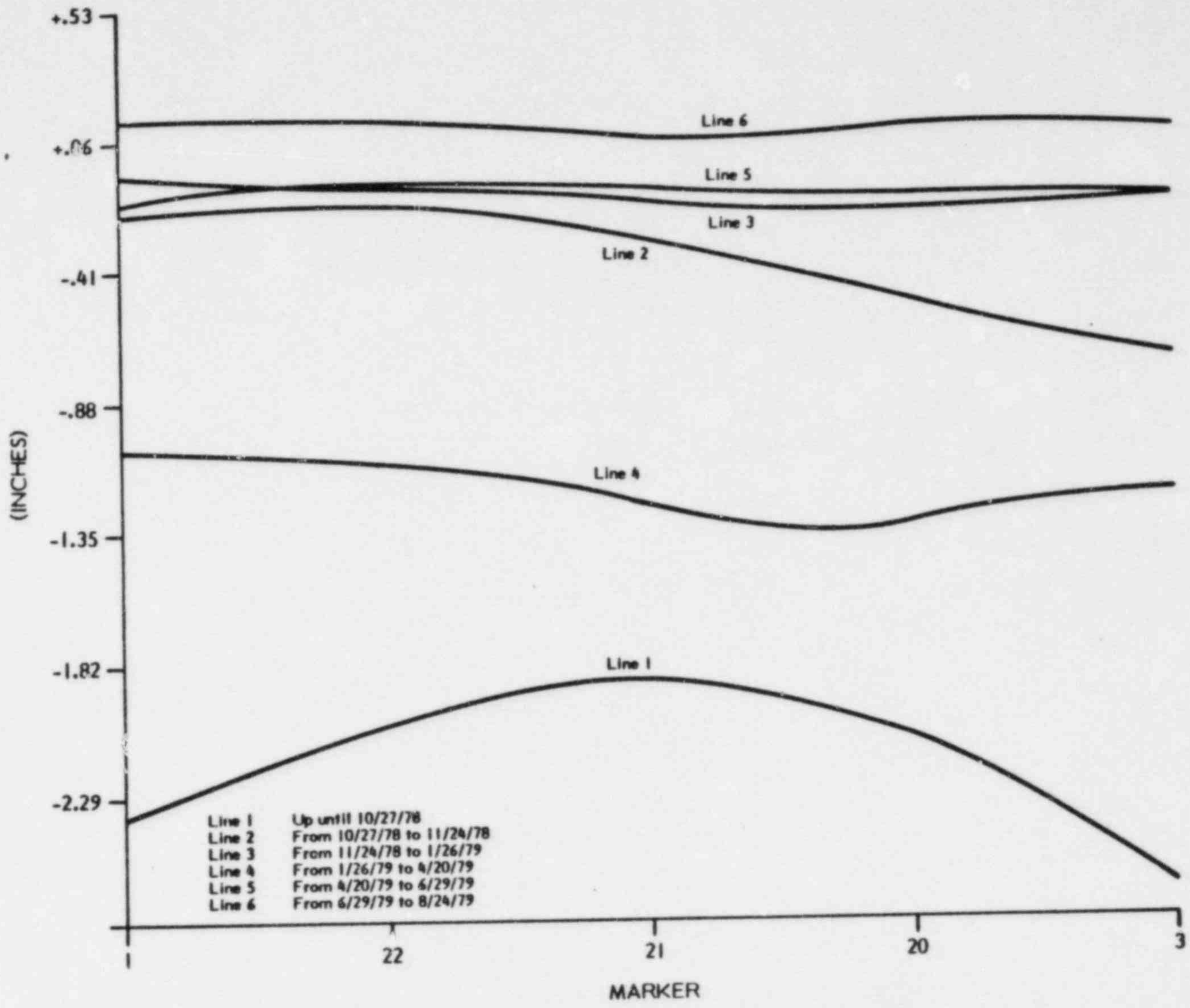

FIGURE 6-3

DIESEL GENERATOR BUILDING
SOUTH WALL
SETTLEMENT INCREMENT

TERA CORPORATION

7.0 CONCLUSIONS

As the diesel generator building exists today it is quite capable of performing its intended design functions. Many cracks of small size are evident in the existing building but there is no evidence to suggest that these cracks -- in spite of the various possible mechanisms of origin - generally of small size, would be indicative of a condition that would suggest the DGB is incapable of performing its function. It is our belief that in its present condition this building is fully functional in all respects. Although we believe it is improbable, if excessive localized differential settlement is observed, remedial corrective measures could be undertaken to improve serviceability.

The committed monitoring program clearly will reveal any potential distress. It is suggested that a comprehensive visual inspection of DGB be carried out biannually (twice a year) in concert with the settlement measurement program. In Section 6.4 we have offered certain recommendations for consideration that are intended to improve information collected and reduce operational constraints.