Attendance List
Meeting with Consumers
on Midland
2/23/82

FREDERICK C. WILLIAMS* (PM ONLY) — ISHAM, LINCOLN & BEALE

Name	Organization
K. N. JABBOUR	NRC/DL/LB #4
R. GONZALES * (A.M. only)	NRC/DE/HGEB
Myron Fliegel * (Part Time AM)	NRC/DE/HGEB
DENNIS BUDZIK	CPCO
John Schaub	CPCo
S. S. AFIFI	Bechtel
FRANK RINALDI * (P.M.)	NRC/NRR/SEB
AL BOOS	Bechtel
TR Thiruvengadam	Consumers Power
L L TEUTEBERG	CONSUMERS/LIC. SECTION
W.C. PARIS, Jr.	Bechtel
G. DIRNBAUER	BECHTEL
Neal Swanberg	Bechtel
JOHN P MATRA JR * (P.M.)	NSWC
E.M. Burke	Mueser, Rutledge.
Karl B. Rankin	CPCo
Donald L. Chery Jr.* (A.M.)	NRC/HGEB
Alan S. Farnell	Isham, Lincoln + Beale (Chicago)
James K. Meisenheimer	CPCo Geotech Coord.
Ross B. Landsman	NRC/RIII
Hari Naraen Singh	U.S. Army Corps of Engineers, NCD, Chicago
Joseph D. Kane	NRC/NRR/DE/HGEB

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PDR FOIA
RICE84-96 PDR

MIDLAND SOILS FEB 24, 1982

ATTENDEE LIST

NAME	ORGANIZATION	TITLE
RON HERNAN	NRC/NRR/LICENSING	LPM(OL)
Pao Huang	NAVSWC	Consultant
FRANK RINALDI	NRC/NRR/SEB	STRUCTURAL E.
Joseph Kane	NRC/NRR/HGEB	Geotechnical Engr.
Hari Narain Singh	U.S. Army Corps of Engineers.	Geotechnical Engrs.
Fernando Villalta	Consumers Power Co.	Civil Eng.
Ross Landsman	NRC/RIII	Geotechnical Engr
KARL WIEDNER	BECHTEL	ENGR. MGR.
ROGER TEUTEBERG	CONSUMERS POWER Co.	LICENSING SECT.
Neal Swanberg	Bechtel	Asst. Proj. Eng.
T.R. Thiruvengadam	Consumers Power	Section Head - Civil Engg
Dennis A Zanner	Bechtel	Engineer.
John Schaub	CPCo	Project Management
DARL HOOD*	NRC/NRR/DL/LB4	PM
James Meisenheimer	CPCO	Geotech. Engr.
N. RAMANUJAM	CPCO	" "
Alan Jarnell	Isham, Lincoln + Beale	Chicago
JOHN P MATRA Jr	NSWC	CONSULTANT
Dennis Budzik	CPCo	

* Part time

CHRONOLOGICAL LIST OF EVENTS BEFORE AND AFTER ISSUANCE OF NRC STAFF ORDER MODIFYING CONSTRUCTION PERMITS

Date	Activity
October 5, 1977	Begin pouring the diesel generator building foundations to el 630'-6"
December 13, 1977	Begin pouring the diesel generator building walls to el 635'-0"
January 6, 1978	Diesel generator pedestal foundation (bay 4) is poured
January 25, 1978	Completed pouring the diesel genrator building foundations to el 630'-6" (see October 5, 1977)
February 14, 1978	Diesel generator pedestal foundation (bay 3) is poured
February 20, 1978	Completed pouring diesel generator building walls to el 635'-0" (see December 13, 1977)
March 8, 1978	Diesel generator pedestal foundation (bay 2) is poured
March 14, 1978	Begin pouring walls to el 654'-0"
March 23, 1978	Diesel generator pedestal foundation (bay 1) is poured
April 28, 1978	Completed pouring walls to el 654'-0"
July 10, 1978 - August 22, 1978	Placement of heating, ventilating, and air conditioning chamber slabs at el 656'-6"
August 22, 1978	NRC inspector at Midland jobsite is informed of unusual settlement of diesel generator building
August 23, 1978	Diesel generator building construction voluntarily halted (Reference: BEBC-2427)
August 25, 1978	Soil boring program initiated
September 7, 1978	Management Corrective Action Report 24 (MCAR) is issued
September 29, 1978	Interim Report 1 to MCAR 24 is forwarded to the NRC

Chronological List of Events (Continued)

Date	Activity
November 7, 1978	Interim Report 2 to MCAR 24 is forwarded to the NRC
November 16, 1978	Construction activities resume on the diesel generator building (Reference: BEBC-2547)
November 16, 1978	Isolate electrical duct bank from the diesel generator building in bay 3
November 18, 1978	Isolate electrical duct bank from the diesel generator building in bay 1
November 21, 1978	Isolate electrical duct bank from the diesel generator building in may 4
November 24, 1978	Isolate electrical duct bank from the diesel generator building in bay 2
December 12, 1978	Placed mezzanine floor to el 664-'0" in bay 4
December 19, 1978	Placed mezzanine floor to el 664'-0" in bay 3
December 20, 1978	Placed mezzanine floor to el 664'-0" in bay 1
December 21, 1978	NRC is informed (Howe 267-78) of decision to preload diesel generator building
December 28, 1978	Placed mezzanine floor to el 664'-0" in bay 2
January 5, 1979	Interim Report 3 to MCAR 24 is forwarded to the NRC
January 5, 1979	Commence pouring walls of building to el 678'-3" (see February 20, 1979)
January 12, 1979	End of pond fill
January 26, 1979	Beginning of surcharging (completed on April 6, 1979). Surcharge is placed in accordance with Specification 7220-C-81.
January 31, 1979	Condensate lines 20"-1HCD-169, 6"-1HCD-513, and 6"-2HCD-513 were cut loose on the south side of the turbine building. Horizontal movement of 3 to 4 inches to the west was observed (see October 22, 1979. Reference: field report)
February 1, 1979	Condensate line 20"-1HCD-169 was cut loose on the south side of the turbine building (see October 22, 1979. Reference: field reports)

Chronological List of Events (Continued)

Date	Activity
February 15, 1979	Preparatory work for installation of strain gage monitors in the turbine building wall started today. Strain gages are being installed in accordance with Specification 7220-C-83.
February 16, 1979	First crack mapping of diesel generator building is completed
February 20, 1979	Completed pouring walls to el 678'-3" (started on January 5, 1979)
February 23, 1979	Installation of strain gage monitors for "Q" line wall of turbine building is completed. Installation is in accordance with Specification 7220-C-83 (see February 15, 1979)
February 23, 1979	Interim Report 4 to MCAR 24 is forwarded to the NRC
March 5, 1979	All surcharge activities through Step III of Table I on Drawing 7220-C-1141(Q) have been completed. Surcharge placement is suspended until March 22, 1979, to observe effect of surcharge placed to date (surcharge approximate elevation is 644'-0")
March 8, 1979	Commence placing roof and parapet to el 681'-6" (completed on March 22, 1979)
March 21, 1979	NRC initiates 10 CFR 50.54(f) Requests Regarding Plant Fill
March 22, 1979	Placing of surcharge resumes in accordance with Step V of Drawing 7220-C-1141(Q) (see March 5, 1979. Reference: BEBC 2806). Roof and parapet completed i.e., last of diesel generator has been poured (See March 8, 1979)
April 6, 1979	Placement of surcharge is completed (began on January 26, 1979)
April 24, 1979	Applicant submits response to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f)
April 30, 1979	Interim Report 5 to MCAR 24 is forwarded to the NRC
May 31, 1979	Applicant submits Revision 1 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f)

Chronological List of Events (Continued)

Date	Activity
June 25, 1979	Interim Report 6 to MCAR 24 is forwarded to the NRC
July 9, 1979	Applicant submits Revision 2 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f)
August 15, 1979	Removal of surcharge commences
August 22, 1979	Construction activities resume on the diesel generator building
August 31, 1979	Removal of surcharge is complete
September 5, 1979	Interim Report 7 to MCAR 24 is forwarded to the NRC
September 13, 1979	Revision 3 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to NRC
October 22, 1979	Ann Arbor office allows field to reweld the condensate lines at the turbine building (see January 31 and February 1, 1979. Reference: BEBC-3344)
November 2, 1979	Interim Report 8 to MCAR 24 is forwarded to the NRC
November 13, 1979	Revision 4 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to NRC
December 6, 1979	NRC Staff issues Order Modifying the Construction Permits
December 1979	Crack mapping of diesel generator building is again performed
February 29, 1980	Revision 5 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to NRC
April 1, 1980	Revision 6 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to the NRC
May 5, 1980	Revision 7 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to the NRC

Chronological List of Events (Continued)

Date	Activity
August 1, 1980	North half of el 634'-0" slab is poured in bay 2
August 12, 1980	South half of el 634'-0" slab is poured in bay 2
August 15, 1980	Revision 8 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to the NRC
August 15, 1980	North half of el 634'-0" slab is poured in bay 1
August 22, 1980	South half of el 634'-0" slab is poured in bay 1
August 29, 1980	Begin grouting the gap between the diesel generator building footing and the mud mat (see September 11, 1980. Reference: REM C-2817)
September 11, 1980	Completed grouting of gap between building footing and mud mat (see August 29, 1980. Reference: REM C-2817)
September 14, 1980	Revision 9 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is forwarded to the NRC
October 8, 1980	North half of el 634'-0" slab is poured in bay 4
October 14, 1980	South half of el 634'-0" slab is poured in bay 4
October 16, 1980	North half of el 634'-0" slab is poured in bay 3
October 23, 1980	South half of el 634'-0" slab is poured in bay 3
October 31, 1980	Diesel generator has been installed in bay 1
November 13, 1980	Diesel generator has been installed in bay 2
November 21, 1980	Revision 10 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is submitted to the NRC

Chronological List of Events (Continued)

Date	Activity
December 15, 1980	Diesel generator has been installed in bay 3
February 5, 1981	Diesel generator has been installed in bay 4
February 27, 1981	Revision 11 of Responses to NRC Requests Regarding Plant Fill, 10 CFR 50.54(f) is submitted to the NRC
April 20-24, 1981	NRC performs Structural Technical Audit of Midland Nuclear Power Project
July 1981	Crack mapping of diesel generator building is again performed

9 months

628 bottom
started [illegible]'s Oct 77

~~635~~ finished el 635 Feb 78

TRY & JUSTIFY

654 first survey July 78

~~Sep 78~~

(1)

ducts cut loose Dec 78 Jan 79

667 meeting

SURCHARGE

(2)

68[illegible] 500 Δ March

Aug 79

(3)

40 YRS

Midland Plant Underpinning
Questions Regarding Service Water Pump Structure
Based on Submittals by Consumers Power
February 25, 1982

1. Design

1.1 How are spring constants selected for each loading condition? What values are being used?

1.2 How much differential settlement is assumed in design for long-term condition?

1.3 What maximum difference between load on each adjacent pier is acceptable to avoid breaking the shear keys?

1.4 For what out-of-plane forces has the underpinning wall been designed?

1.5 How is the shear load in the bolts estimated?

1.6 What are the existing maximum stresses and where do they occur?

1.7 Can SWPS support between corner piers? How much soil support is assumed.

2. Dewatering

2.1 Provide description of dewatering system. non Q

(drilling mud)

2.2 Provide location, depths, and types of piezometers for monitoring water levels.

2.3 How long in advance of first drift will dewatering be done? State that dewatering will be done well ahead of drift and excavation. procedure says do not ~~dont~~ excavate below water

3. Monitoring and Acceptance Criteria

3.1 Provide a table and plan that shows which cracks, pier loads, movements, and concrete stress charges will be monitored. State frequency of readings before, during and after underpinning. For critical stages of underpinning, for example when first drift passes under structure and during installation of piers 1, 2, 3, 4, 5, 6, state frequency. Select the critical measurements and detail how they will be used to control construction.

3.2 Provide a limiting criterion for each measurement.

3.3 How much time will pass between the measurement of a limiting reading and the action to prevent further distress. procedure

3.4 For each case state remedial actions that are intended if a limiting measurement is reached or exceeded. arresting plan

3.5 The use of 75% of predicted settlement as a criterion for judging whether settlement is occurring at a satisfactory rate is not applicable, since prediction may not be correct. This criterion should be deleted.

3.6 Jack loads and differential movements must be watched when sump is filled with water before jack removal.

4. <u>Bearing Stratum</u>

4.1 Who will accept the bearing stratum? only resident geotechnical engineers with approved qualifications

4.2 How will adequacy of alluvium as a bearing stratum be determined in situ. Why is lean concrete to be used under piers?

no lean concrete

4.3 What is maximum elevation difference of adjacent piers?

4.4 In one place it is stated that a penetiometer under 150 # load is to penetrate 1/2 in.,, and in another 3/4 in. Which is correct?

correct drawing incorrect

4.5 Is there prevalent gravel in the hard clay bearing stratum? yes Is the material stratified? yes

5. <u>Drift and Jacking</u>

5.1 Why is drift under the structure rather than alongside?

5.2 One pier should be load tested in detail to a value above the maximum expected hearing pressure.

5.3 Why is initial jacking load not equal to full final load? If full load is left in place as long as possible settlement will occur for a longer period before jack removal.

5.4 Why are piers 11 built after removal of jacks?

5.5 How often will loads on pier jacks be checked during underpinning.

6. Admi[illegible]ation

6.1 What is schedule of construction.

6.2 How much time will elapse between a critical measurement and remedial action.

6.3 Please provide _ flow chart showing expected sequence of activities.

Excavation of piers - non Q

Question of SWPS
6 Hexub

J Kane
Rec'd 2/15/82
from H Singh

SUBJECT: Review of the Technical Report on Underpinning of the Service Water Pump Structure submitted on ~~26 August~~, 1981.

PRELIMINARY

The Corps of Engineers has reviewed the subject technical report and has discussed its comments with applicant in a meeting on 17 September, 1981, held in NRC office, Bethesda, MD. Subsequent to this meeting, the applicant responded to the Corps' and NRC comments through its submission of 6 November, 1981. The Corps has reviewed the applicant response and has raised following questions:

Q1. (Pg. 9, Sect. 3.3, Para. 1) Please provide a section through the wall showing how the settlement dial indicators would be attached to the building, and their probes connected to the permanent bench mark.

Q2. (Pg. 9, Sect. 3.3, Para. 1) How will the settlement markers be monitored? Section 3.2, as stated in the paragraph, does not provide the details of monitoring of settlement markers.

Q3. (Pg. 9, Sect. 3.3, Para. 1, last sentence) From the last sentence of paragraph 1, it is apparent that some building movement will be allowed during the construction. Please provide details: how much building movement at the free end of the overhang you plan to permit, and what is your basis for choosing a particular value of permissible building movement.

Q4. (Pg. 9, Sect. 4) Please provide the details of bearing capacity analysis, shear strength parameters used, and resulting factor of safety for static and dynamic loadings. Since the soils are highly overconsolidated, bearing capacity analysis based on drained shear strength parameters are also required.

Q5. (Pg. 10, Sect. 5.0) Please explain variation in deformations of 0.2" over entire foundation, and how do you plan to incorporate the effects of these variation of deformations on the behavior of the structure. If the soil media under the foundation are to be represented by springs, please provide spring constants and the method used in their determination with the details of the analysis.

Q6. (Pg. 10, Sect. 6.1, Para 2) Section DD of Figure 5, Reference 1, does not show details at the top end of the rod. Please provide a sketch showing the instrumentations to be used at the top of the piers to measure deflections of the soils and the total top of the piers deflections.

Q7. (Pg. 10, Sect 6.1, Para 2) Please explain the statement made in the last sentence of paragraph 2. In our opinion, the difference between the soil deflection and total deflection at the top of pier will represent the behavior of the concrete in the pier rather than behavior of the supporting soil.

Q8. (Pg. 10, 11, Sect 6.1, Para 3) The predicted consolidation settlement is reported to be between 0.4 and 0.5 inches. We understand from Figure 4 that the above values of settlement include primary as well as secondary settlements. Please provide details how were the two components (Primary & Secondary) determined.

Q9. Figure 4 shows that initial jacking loadings on the underpinning walls are much less than the final jacking loadings. We understand that final jacking loadings correspond to the total load of the structure to be transmitted to the underpinning wall. Please provide basis for selecting lower initial jacking loadings and methods used in determining their values.

must be applied simultaneously

Q10. (Pg. 11, Sect. 6.1, Para 2) Since piers will be constructed sequentially, the initial jacking loadings must be applied ~~simultaneously~~ in same manner. Also, this is our understanding that 90 days time interval between initial and final jacking loadings must be counted from the date when initial jacking has been applied to the last pier (pier No. 12). Please provide your discussion on this aspect.

good inflection point

Q11. (Pg. 11, Sect. 6.1, Para 2) Please provide details to substantiate the statement, "At about 110 days, the curve will flatten so it will appear as a straight line on this semi-log plotting." It appears that only 20 days (110-90) have been allowed for primary consolidation to complete after application of final jacking loadings.

PUT IN PROCEDURE prove a st. line

Q12. (Pg. 11, Sect. 6.2) What is acceptable limits of settlement rate? How have you determined these limits?

Q13. (Pg. 12, Sect. 8.0) What is basis for selecting 2" of deflection at which soil indicate plastic behaviors. Also, provide basis for .01" settlement in 3 hours after 3 days of constant load, and .02" for interval 10 to 20 days under constant load.

water level

Q14. (Pg. 12, Sect. 9.0) Please provide plan showing the location of piezometer to be installed for monitoring ground water levels during construction.

Q15. (Pg. 13, Sect. 11) The soil spring constants have not yet been received.

Q16. (Pg. 8, Sect. 3.1.1) By constructing first pier #4 and pier #5, and then preloading them with initial jacking loads, the symmetrical application jacking loadings on the structure will be violated. Please explain what will be the consequences of unsymmetrically applied jacking loads?

Q17. (Pg. 8, Sect. 3.1.2) In our opinion, the measurement and the consideration of the loadings on pier Nos. 1, 2 and 3 alone would not provide sufficient information about structural problem encountered during construction. The construction of tunnels from piers 3's to piers 4's would transfer ~~no~~ additional load to piers 1, 2 and 3 which ultimately be transferred to the structure by cantiliver action because of increased settlement of tops of piers 1, 2 and 3, and as such, the stress recording device placed on top of the piers may not be able to record these additional loadings. Therefore, settlement of the piers must be monitor with utmost care and be used in determining the construction procedure.

Q18. (Pg. 8, Sect. 3.1.3) The allowable bearing intensity for foundation should be determined using the test results of samples for COE-16. Please revise and furnish the new values instead of 19.2 ksf bearing intensity and 1600 kips bearing load for each ~~pile~~ pier group.

Q19. (Pg. 2, Sect. 2.1.1.2, Para b) The long term shear strength parameter given in this paragraph is not consistent with Woodward-Clyde consultants' test data. As a matter of fact, no sample has been tested from zone of influence of the foundation.

Q20. (Pg. 3. Sect. 2.1.2.1) Since the jacking used during the final load transfer would only transfer the dead and the equipment load of the fill supported structure on the foundation media of the underpinning walls and those of structure originally founded on natural soil, it would not produce load on the foundation more than the total structure load of the overhang portion. Therefore, any load transfer caused by jacking should be considered as dead load and in loading combination in design of foundation and structure should be considered as dead load. Please explain why in one of the loading combination on page 3, it has been considered separately.

Q21. (Pg. 3, Sect. 2.1.2.2, Para c) Please explain the statement made in para. c. It is not known why the foundation of underpinning wall would not carry the dead load and live load?

Q22. (Pg. 4, Sect. 2.1.3.3) Does the bearing pressure 8.12 ksf include the effects of post tensioning the overhang structure? How did allowable bearing pressure of 16.7 ksf was determined?

Q23. (Pg. 4, Sect. 2.1.4.1) Why other loading combinations as used for lower foundation slab have not been verified in this case?

Q24. (Pg. 6, Sect. 2.1.6.2) Please explain why P_L has not been considered as dead load in loading combination used and shown in paragraph c.

Q25. (Pg. 6, Sect. 2.1.6.3) Please provide shear capacity of each 2" dia. rock anchors. Please, also provide the magnitude of horizontal shear at the interface of underpinning walls and the bottom of the existing foundation slab.

Q26. (Pg. 7, Sect. 2.1.7.1) Why appropriate load factor has not been used in design of underpinning walls?

J. Kane
HGEB

Subject: Design Issues to be Audited by HGEB at February 3-5, 1982 Audit in Ann Arbor, Michigan

License Condition No.	Review Issue	Documentation Anticipated to be Presented to HGEB	Design Audit Feb. 3-5, 1982
5a	Auxiliary Building Temporary Support System During Underpinning (EPA and Control Tower)	Plan and sectional views showing the locations in the structures and on the foundation bearing layer where temporary underpinning loads have resulted in the largest stresses. Drawings should indicate assumed exc. conditions at the various stages of construction.	Information was provided in Dasgupta presentation and handouts, but results are impacted by the requested sensitivity study on soil spring constant variations.
		Calculations that provide the magnitude of the above stresses.	Checked by SEB
		Calculations providing the factors of safety against bearing failure.	Provided in Dasgupta Presentation
5b	Auxiliary Building Temporary Support System During Underpinning (EPA & Control Tower)	Sketches showing deformation measuring instruments attached at top of pier at the selected locations.	Provided by Bob Adler. NRC needs to review
		Description of frequency of readings to be required.	Provided on drawing entitled "Instrumentation Matrix"
		Identification of the ALLOWABLE movements, strains or stresses at the selected monitoring locations and CALCULATIONS which are the basis for those allowable movements. What are crack monitoring plans?	Criteria given for FIVP piping. Tolerance criteria on movements is still required for both Phase II and Phase III instrumentation.
		Criteria to be followed for READJUSTING jacking load (?Settlement).	Criteria on jacking is controlled by both settlement and stress considerations CPC to provide drawings, procedures and criteria to NRC on Feb. 26, 1982.

License Condition No.	Review Issue	Documentation Anticipated to be Presented to HGEB	Design Audit Feb. 3-5, 1982
5b (continued)		This is ALLOWABLE movements. What valves (limiting) of movement or cracking or stress will require re-evaluation and stopping of underpinning? How established? Provide the time interval (maximum) between observing limiting movement or stress and time for action (re-evaluation or stopping).	Tolerance criteria will identify both an action level and a stopping level. CPC still needs to address crack propagation. NRC needs to review criteria on cracking provided in Auxil. Bldg. report and be prepared to discuss at Feb. 25, 1982.
5c	NRC Testimony (11/20/81) Attachment 21, Q.6	Previous discussions have resolved this issue.	Previously resolved.
5c	Attachment 21, Q.7	Provide explanation on how measured jacking load and pier settlement will be used in NAV-FAC DM-7, Fig. 11-9 to establish equivalent soil modulus.	By knowing the shape, embedment, deflection — Fig. 11-9 is used to establish coefficient which permits modulus to be computed. Issue is resolved.
5c	Attachment 21, Q.17	Provide CALCULATIONS which determined the magnitude of the test load for temporary support pier. What part of this load is due to Turbine Bldg. and what part is due to EPA? (Is this a location of large stress which has been covered in Lic. Cond. 5a?)	@ Pier W5, the Turbine Bldg load is 878^k. Total load is 2513^k (maximum).
5c	Attachment 21, Q.18	Does previous discussion under license condition 5b on ALLOWABLE movements cover Q.18?	Refer to status of 5b.
5c	Attachment 21, Q.19	Question has been adequately addressed including discussions at last audit of Jan. 18-20, 1982.	Previously Resolved.

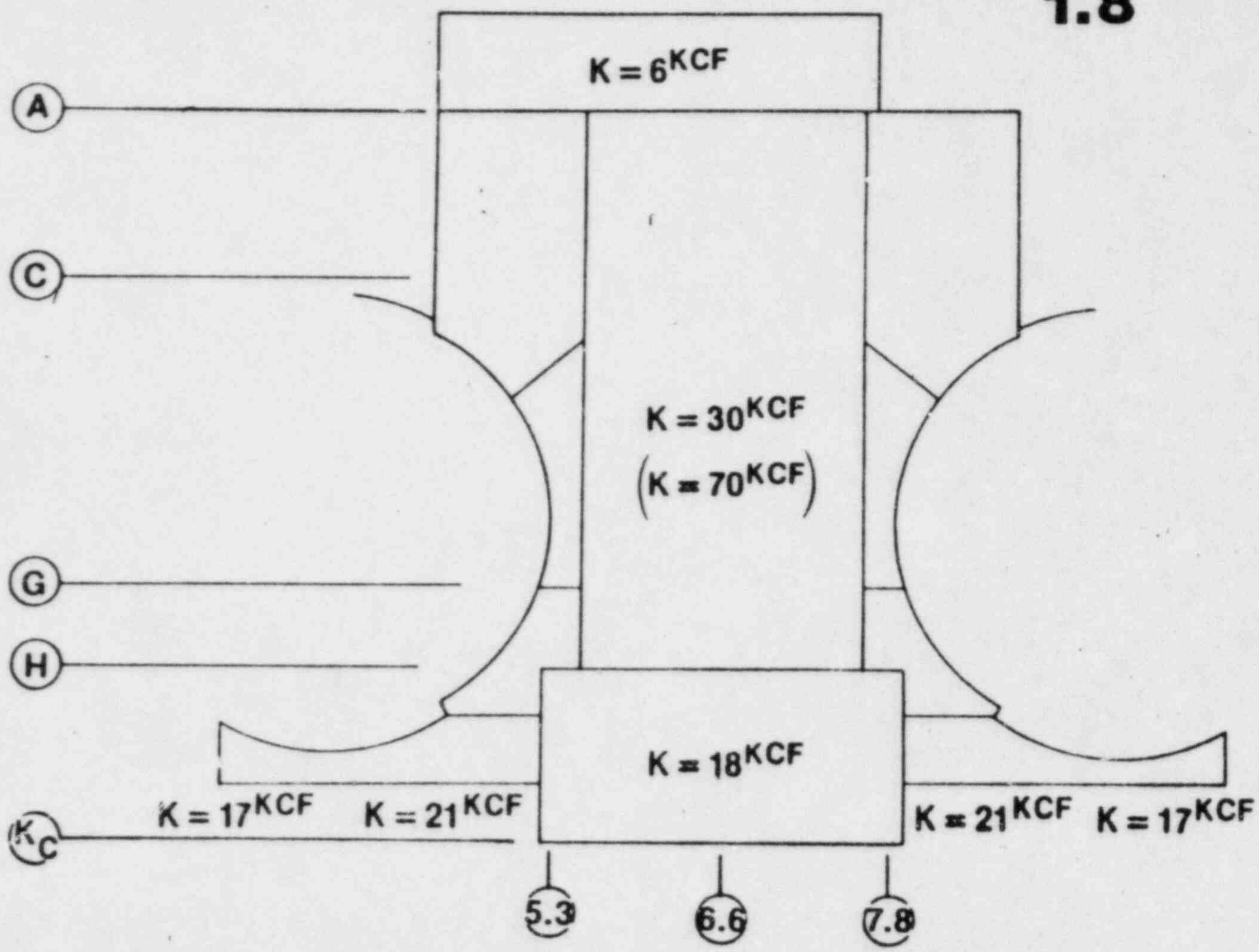
License Condition No.	Review Issue	Documentation Anticipated to be Presented to HGEB	Design Audit Feb. 3-5, 1982
5c	Attachment 21, Q.20	Previous discussions have resolved this issue.	Previously Resolved
5c	Attachment 21, Q.21	Describe what makes up the working load and calculations that establish it. Explain basis for 1.25 times the working load = Proof load. Provide calculations on resistance capacity of the EPA.	Working load = DL + Eqpt. loads + 25% LL + wt. block wall Proofload = Working load +25% working load Capacity of pier W8 is 4000 Kips
5c	Attachment 21, Q.22	Provide magnitude of jacking load for each control tower pier and method to establish it. Refer to CPC Auxil. Bldg testimony, Pg. 24. Describe criteria for monitoring jacking loads on Control Tower (if not covered in 5b). What method will be used to assurance maintenance of jacking loads on Control Tower? Request further discussion on load transfer beyond response to Q.22.	Jacking loads provided in Dasgupta presentation. Refer to previous response to license condition no. 5b for jacking criteria. Anticipate maximum & minimum loads will be provided by Feb. 26, 1982. Load transfer to final underpinning wall to be covered in May 1982 Audit.

CONSTRUCTION CONDITION

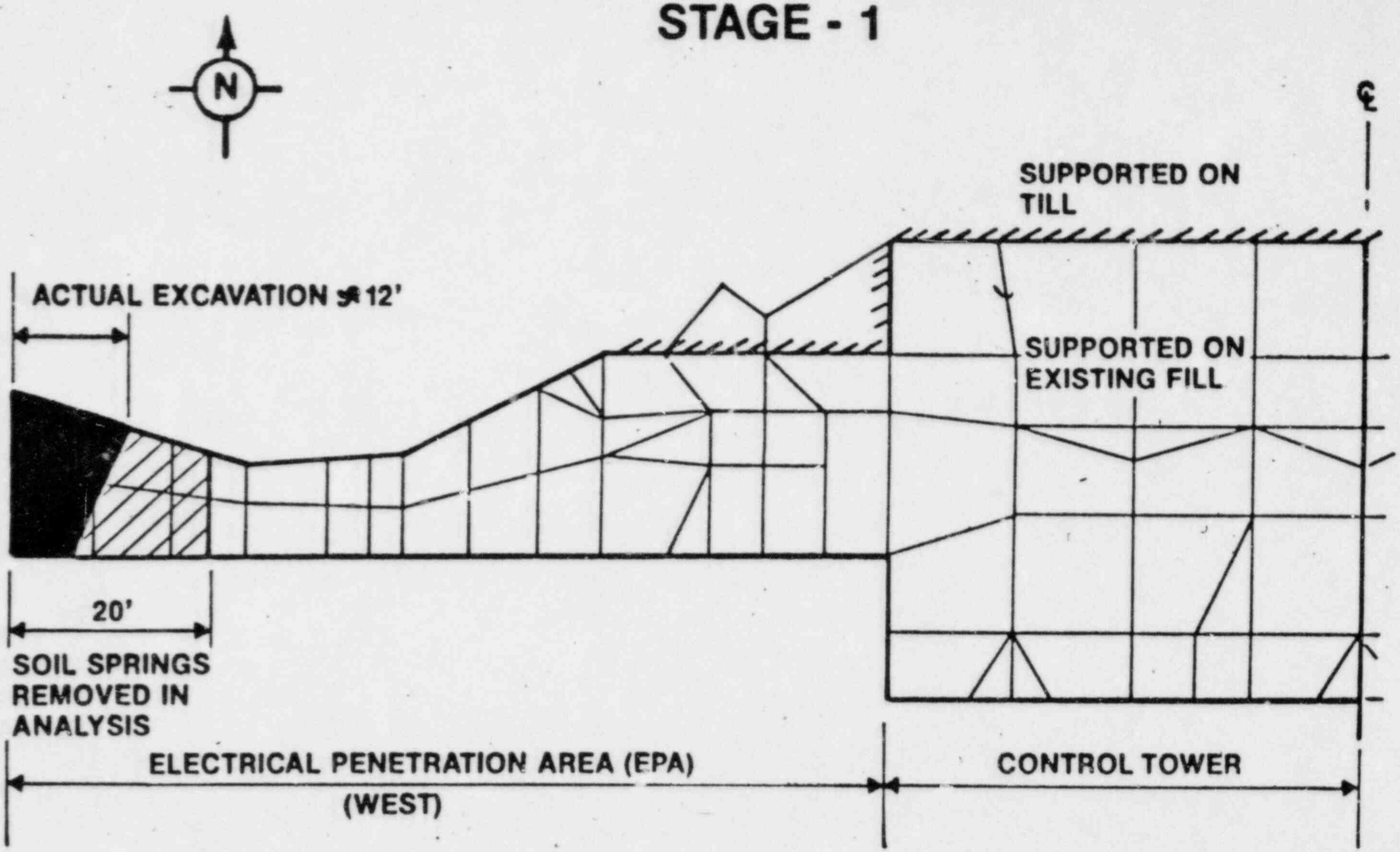
- PARAMETRIC STUDY
  - Effect of Soil Modulus Variation
- ALLOWABLE SETTLEMENTS
- ADMINISTRATIVE PLAN OF ACTION
- GAP BETWEEN TURBINE AND AUXILIARY BUILDINGS

EXISTING SOIL SPRINGS UNDER AUXILIARY BUILDING

(concrete modulus = $\frac{Ec}{1.8}$)



AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION SEQUENCE
STAGE - 1

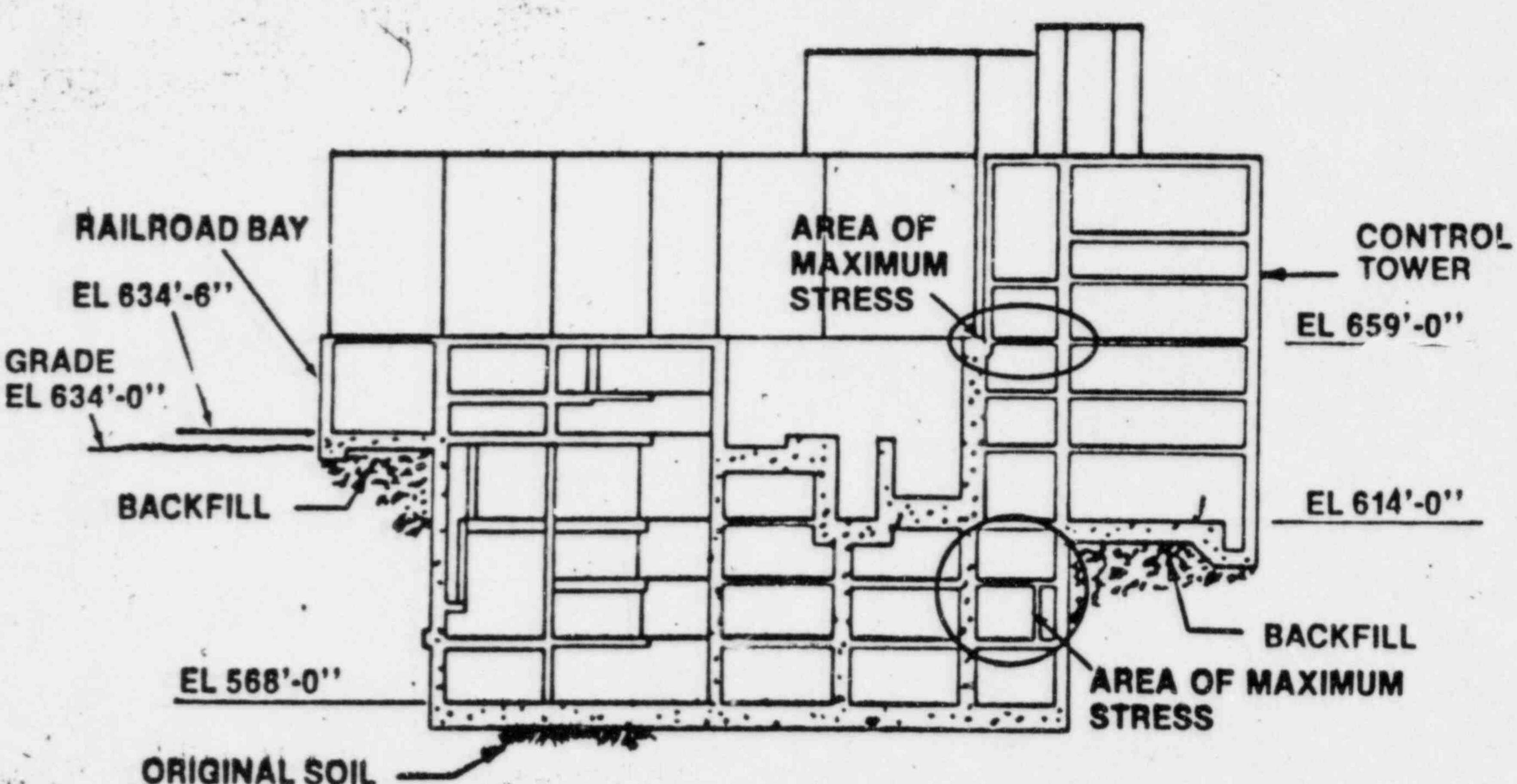


AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION SEQUENCE
STAGE 1

LOSS OF SUPPORT AT THE END OF EPA

	Estimated Support
SOIL MODULUS = 30^{KCF} (UNDER MAIN AUXILIARY BUILDING)	$1,240^{K}$
SOIL MODULUS = 70^{KCF} (UNDER MAIN AUXILIARY BUILDING)	844^{K}

AUXILIARY BUILDING UNDERPINNING
TYPICAL SECTION
LOCATION OF MAXIMUM STRESS
(Looking East)



AUXILIARY BUILDING UNDERPINNING
SOIL STIFFNESS VARIATION
(concrete modulus = $\frac{E_c}{1.8}$)

	Existing Stress	Stage 1 Excavation
SOIL MODULUS = 30^{KCF} (UNDER MAIN AUXILIARY BUILDING)	30^{K}/FT	37^{K}/FT
SOIL MODULUS = 70^{KCF} (UNDER MAIN AUXILIARY BUILDING)	43.4^{K}/FT	48^{K}/FT*
RATIO OF STRESS =	1.45	1.30

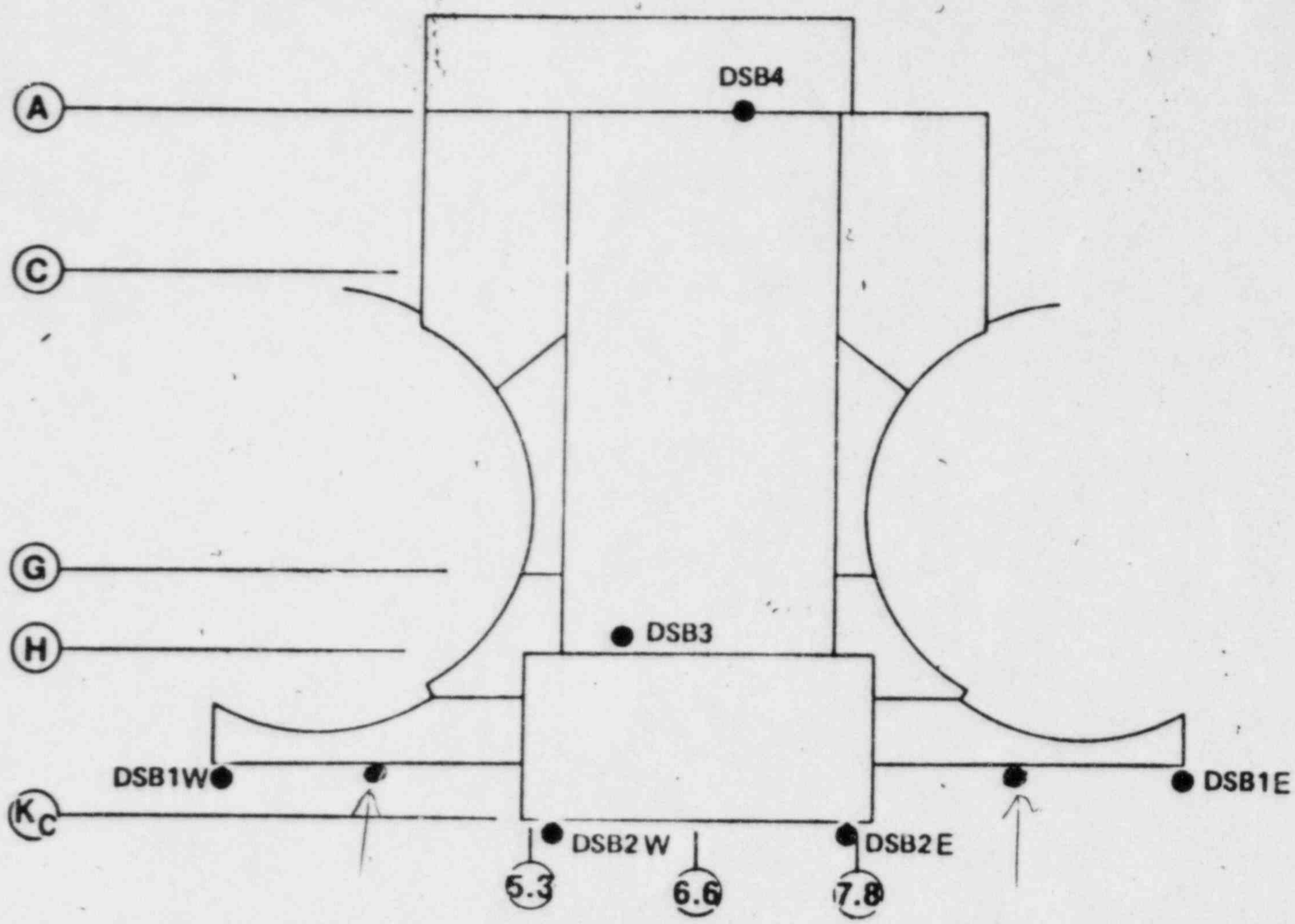
*Estimated value

AUXILIARY BUILDING UNDERPINNING

TYPES OF MONITORING

- BUILDING MOVEMENT
- CRACK MONITORING
- STRAIN MEASUREMENTS

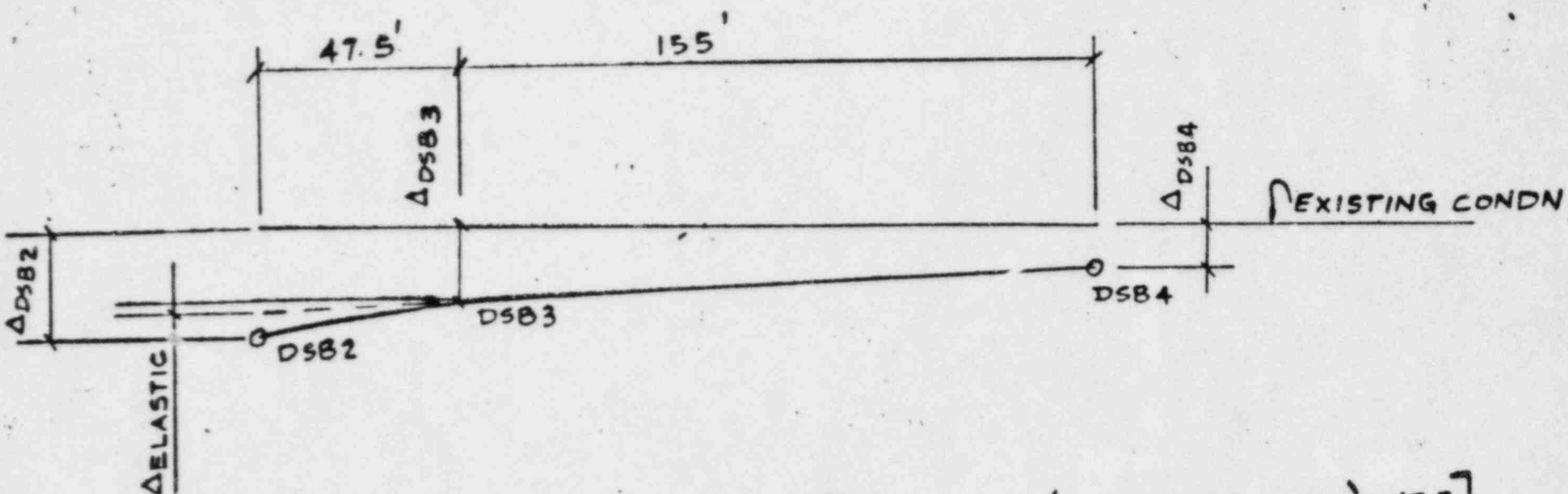
EXISTING SOIL SPRINGS UNDER AUXILIARY BUILDING



AUXILIARY BUILDING UNDERPINNING

CONSTRUCTION CONDITION

SETTLEMENT PREDICTION



$$\Delta_{ELASTIC} = \Delta_{DSB2} - \left[\Delta_{DSB3} + \frac{(\Delta_{DSB3} - \Delta_{DSB4}) \times 47.5}{155}\right]$$

AUXILIARY BUILDING UNDERPINNING
BUILDING MOVEMENTS
(W.R.T. DSB 3)

	DSB 1W	DSB 2W	DSB 1E	DSB 2E
PREDICTED	100 MLS	40 MLS	110 MLS	30 MLS
ALLOWABLE (preliminary)	130 MLS	110 MLS	130 MLS	110 MLS

0.1"

DIVISION OF RESPONSIBILITIES

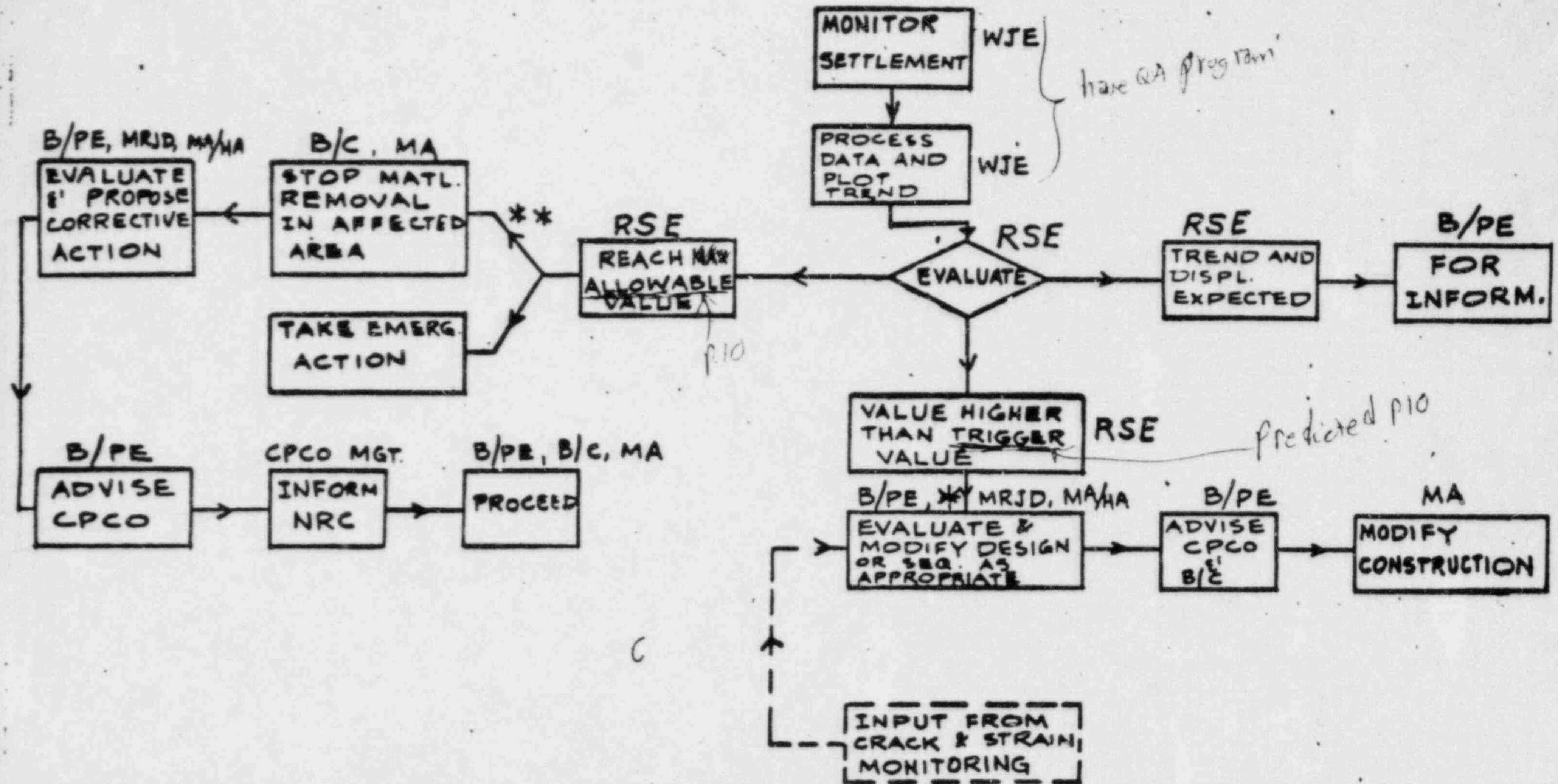
- WJE - READS DATA AND PLOTS
- RSE - INTERPRETS DATA-DETERMINES WHETHER
  (1) Routine
  (2) Non-routine but not serious
  (3) Serious
  (except in emergency)
- B/PE, MRJD, MA/HA - Determines necessity of corrective action of (2) and (3) and develops necessary detail if required
- B/C - IMPLEMENTS PROJECT ENGINEERING (B/PE) DIRECTIVE

DIVISION OF RESPONSIBILITIES (Cont'd)

- PCA/INDEPENDENT
  CONSULTANT - EVALUATES CRACKS
- MA - CONSTRUCTS PER INSTRUCTION FROM B/C
- C - REVIEWS B/PE ACTIONS. INFORMS NRC,

AUXILIARY BUILDING UNDERPINNING

ADMINISTRATIVE PLAN FOR ACTION
(BUILDING SETTLEMENT CRITERIA)



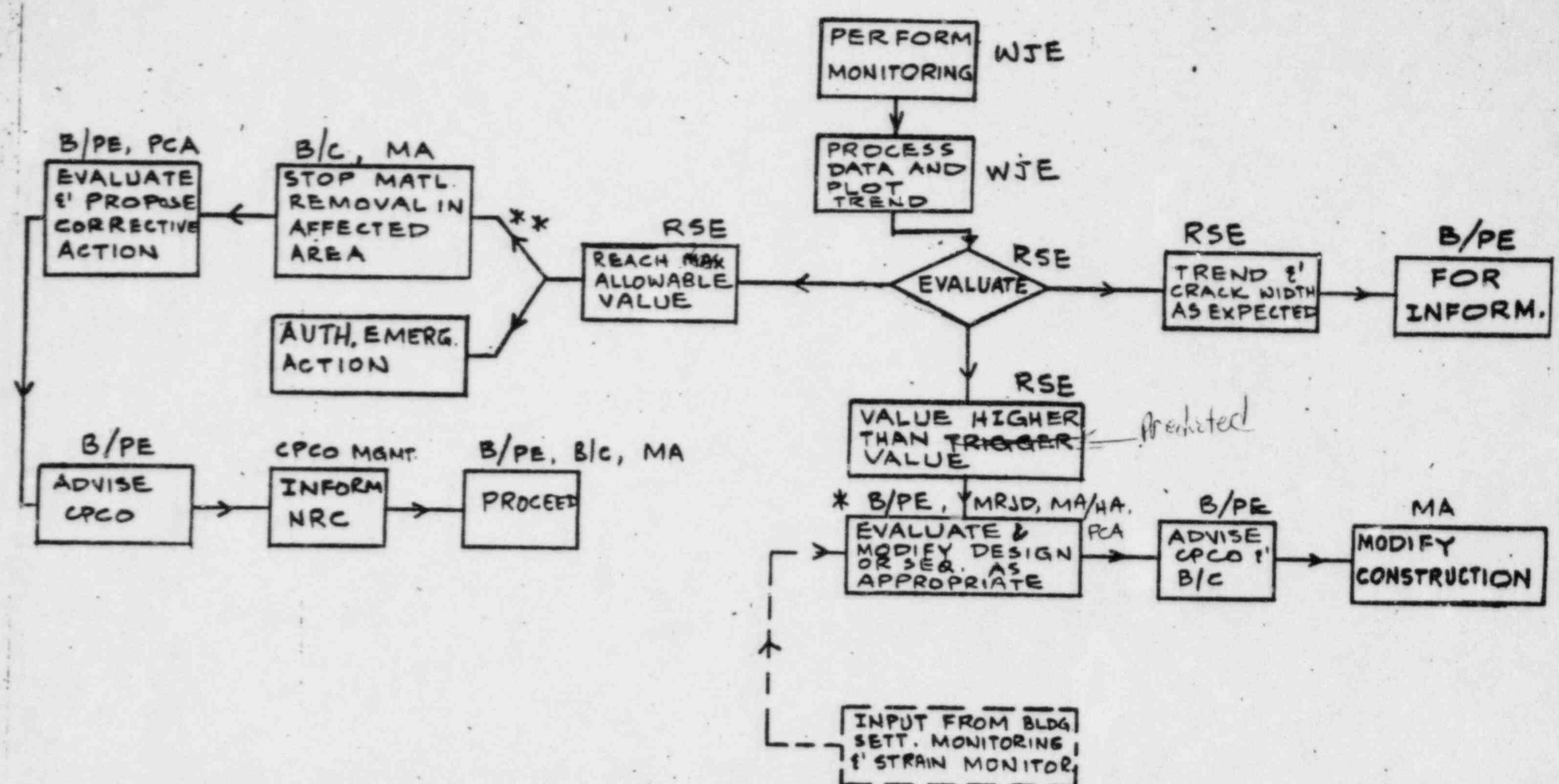
* NOTIFY B/PE WITHIN 24 HRS

** NOTIFY B/C (INCLUDING QC) AND MPQAD WITHIN SAME WORK SHIFT

RSE - resident soils engineer

AUXILIARY BUILDING UNDERPINNING

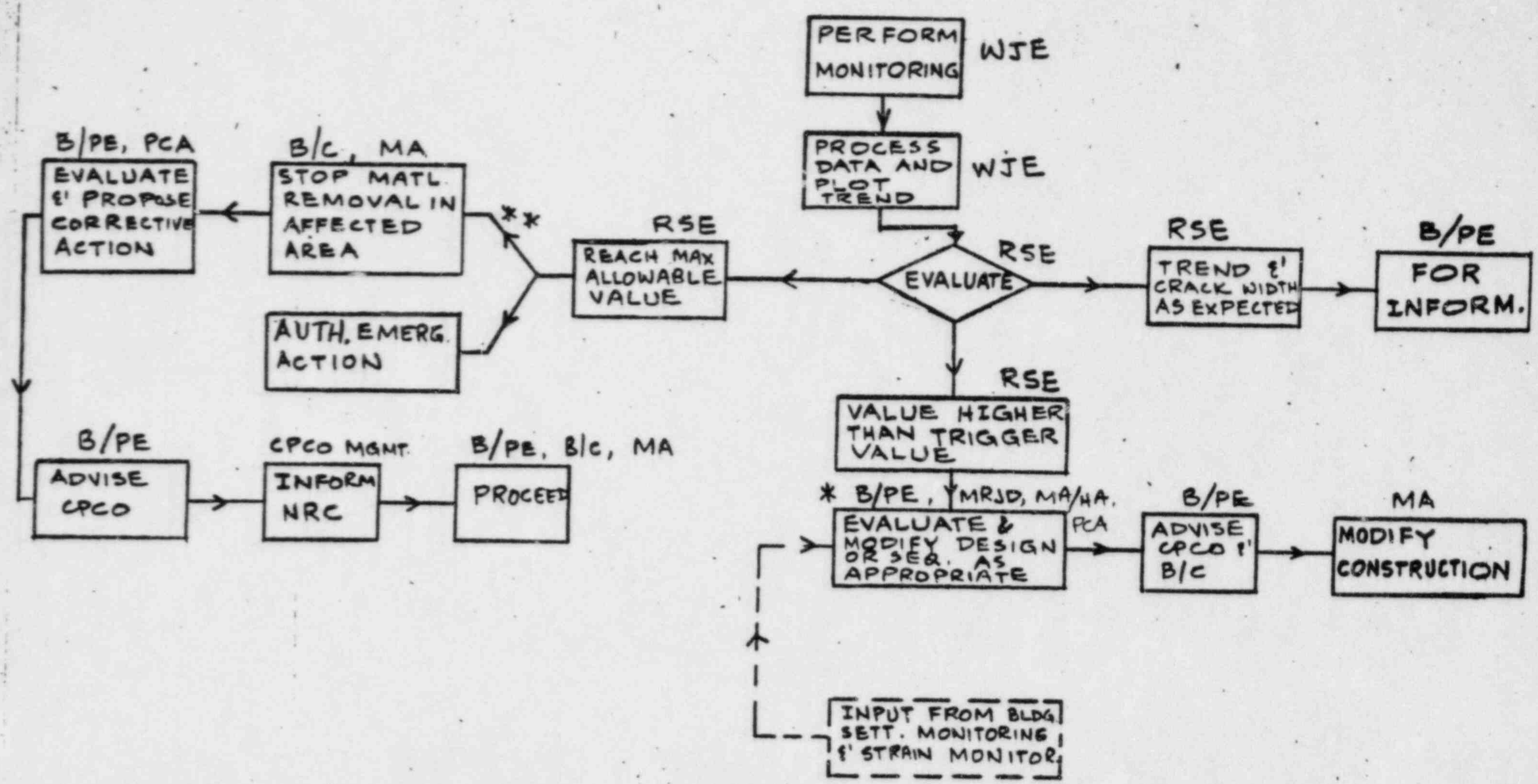
ADMINISTRATIVE PLAN FOR ACTION (BUILDING CRACK MONITORING)



* NOTIFY B/PE WITHIN 24 HRS

** NOTIFY B/C (INCLUDING QC) AND MPQAD WITHIN SAME WORKSHIFT

AUXILIARY BUILDING UNDERPINNING
ADMINISTRATIVE PLAN FOR ACTION (BUILDING CRACK MONITORING)

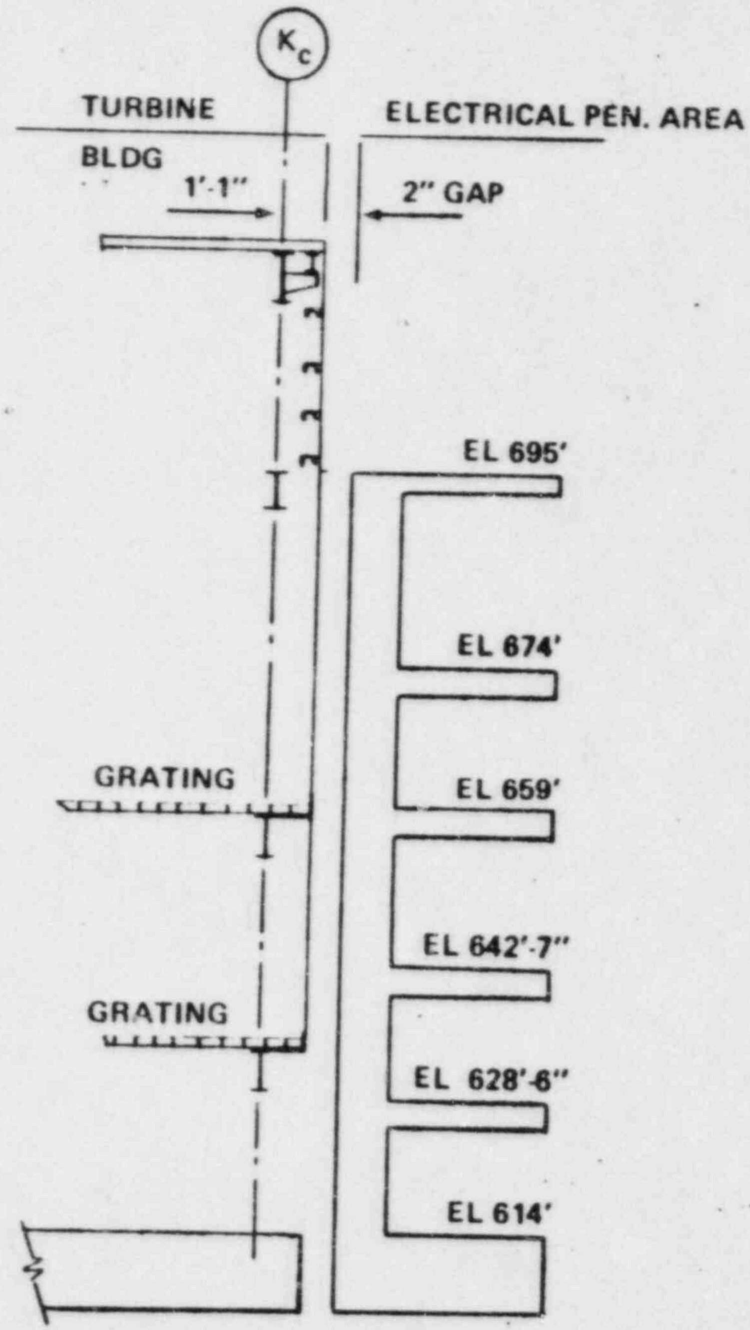


* NOTIFY B/PE WITHIN 24 HRS

** NOTIFY B/C (INCLUDING QC) AND MPQAD WITHIN SAME WORKSHIFT

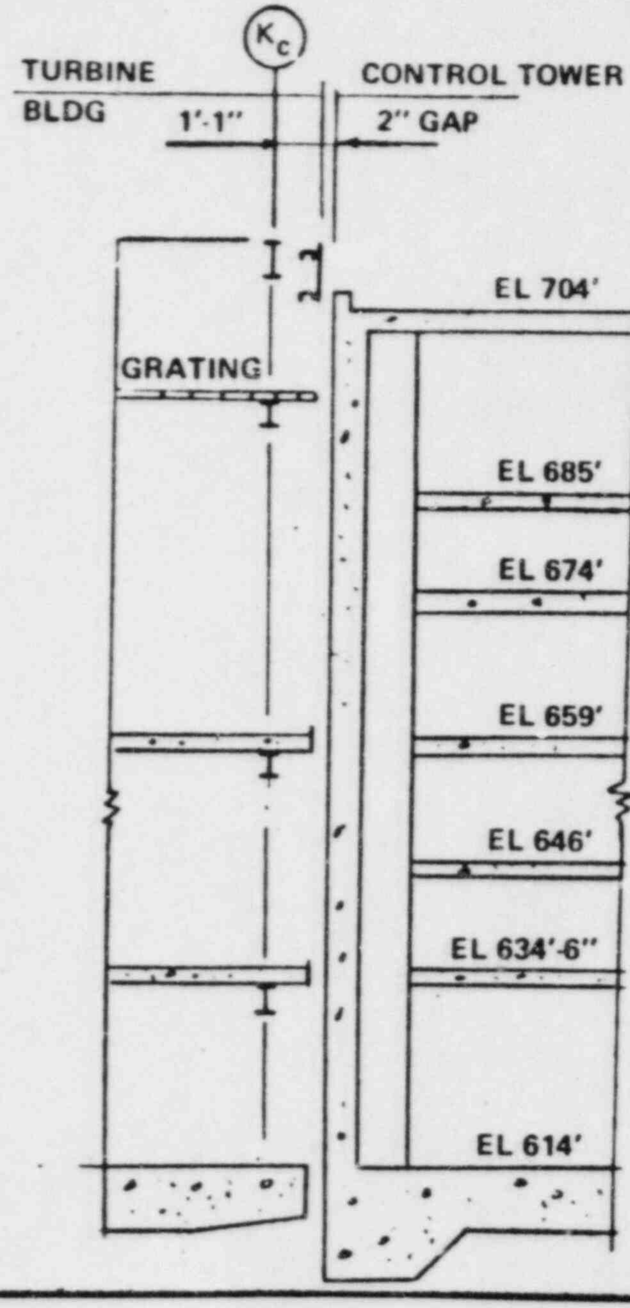
AUXILIARY BUILDING UNDERPINNING
SECTION THROUGH EPA

15



AUXILIARY BUILDING UNDERPINNING
SECTION THROUGH CONTROL TOWER

16



Landsman? 1. What is status of the completion of this effort

MEMORANDUM FOR: C. C. Williams, Chief, Plant Systems System

FROM: R. B. Landsman, Reactor Inspector

SUBJECT: INSPECTION PLAN FOR MIDLAND

2. Who is to take lead in ans. CP Regional addresses to NRC and Keppler

Must know today

Thx.
Cordell

Observation of work, specifications, design drawings, work procedures, QC inspection procedures and QA overinspection procedures will be reviewed for the following:

Dewatering wells.

Drawdown - Recharge test.

BWST surcharge program.

Benchmark installation.

Freeze-wall installation.

D.G. building crack repair.

BWST remedial fix.

Underground pipes.

Service water pump structure remedial fix.

Auxiliary building remedial fix.

Furthermore, each item listed is many faceted requiring approximately two months each. For example, the auxiliary building fix includes access shafts, a tunnel under the turbine building, concrete support piers, horizontal drifts under the control building, grillage support beams under the control building, mass excavation, permanent foundation and backfill.

Additionally, personnel qualifications for QA/QC will be reviewed taking approximately one week. A month is required for audits of laboratories, underpinning contractors and Bechtel Ann Arbor. Furthermore, around a month is needed for hearings and meetings.

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

February 16, 1982

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
EVALUATION REPORT FOR CONCRETE CRACKS
IN THE DIESEL GENERATOR BUILDING
FILE 0485.16 SERIAL 15978
ENCLOSURE: EVALUATION OF THE EFFECT ON STRUCTURAL STRENGTH
OF CRACKS IN THE WALLS OF THE DIESEL GENERATOR BUILDING.

On December 10, 1981 and January 11, 1982, meetings were held with the Staff and its consultants to discuss concrete cracks in the auxiliary building, the service water pump structure, the diesel generator buildings and the feedwater isolation valve pits. During the January 11, 1982 meeting, Consumers Power agreed to provide the NRC with an evaluation of the significance of concrete cracks relative to the design strength of the diesel generator buildings.

In response to this commitment, we are providing the enclosed report entitled "Evaluation of the Effect on Structural Strength of Cracks in the Walls of the Diesel Generator Building" by Dr. Mete A Sozen, Professor of Civil Engineering at the University of Illinois-Urbana. We also call your attention to Attachment 4 of the enclosed report which was contributed by Messrs. W G Corley and A E Fiorato of Construction Technology Laboratories, a Division of the Portland Cement Association. Both Dr. M A Sozen and Dr W G Corley are members of the American Concrete Institute (ACI) Committee 318 on standard building code. The enclosed report presents an evaluation of the significance of the cracks observed in the diesel generator building. The information, measurements and test data presented in the enclosed report lends further support to our conclusion that:

1. At an intermediate construction stage with the footing resting on the duct bank, normal horizontal tensile stresses in the walls would have caused the cracks near the duct banks, if those cracks had not occurred earlier in fresh concrete.

2. There is no evidence to indicate that the strength of the building is less than that assumed in its design.

oc0282-0026a100

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2pp.

Based upon the information contained in the enclosed report, we wish to emphasize that the function of the diesel generator building is well within the range of the experience which supports the theory and practice of reinforced concrete building construction. Therefore, there is no need to reanalyze the diesel generator building using a model to reflect the effects of tensile discontinuities implied by the concrete cracks.

JA Mooney

J A Mooney
Executive Manager
Midland Project Office

For J W Cook

JWC/RLT/mkh

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
GHarstead, Harstead Engineering, w/a
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, w/o
JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/a
WDPaton, Esq, w/o
SJPoulos, Geotechnical Engineering, w/a
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

oc0282-0026a100

ENCLOSURE

EVALUATION OF THE EFFECT ON STRUCTURAL STRENGTH OF CRACKS IN THE WALLS OF THE DIESEL GENERATOR BUILDING MIDLAND PLANT UNITS 1 AND 2 MIDLAND, MICHIGAN

A Report to
BECHTEL ASSOCIATES PROFESSIONAL CORPORATION
Ann Arbor, Michigan

by

Mete A. Sozen
503 W. Michigan
Urbana, IL 61801

11 February 1982

~~8202260258~~ 100 pp

CONTENTS

SUMMARY

This is a study of the effect on strength of the cracks on the walls of the Diesel Generator Building, a box-like reinforced concrete structure with overall dimensions of approximately 70 * 155 by 50 ft high. The exterior walls are 30-in. thick. Three 18-in. thick interior walls with their longitudinal axes in the short plan dimension of the building divide the building into four cells of approximately equal size (Fig. 1 and 2.).

In addition to typical volume-change cracking, some of the interior walls and the east exterior wall have been observed to contain systematic crack patterns (Fig. 6) near the locations of the duct banks (Fig. 4). The duct banks had provided unintended temporary supports for the walls in construction because of settlement of the fill on which the building is founded.

Stress conditions in an interior wall during an intermediate construction stage are analyzed. Residual crack widths and patterns are evaluated. Background information on cracking and strength of reinforced concrete structures is provided in Attachments 1 and 2. The study concludes that:

(1) At an intermediate construction stage, with the footing resting on the duct bank, normal horizontal tensile stresses in the walls would have caused the cracks near the duct banks, if those cracks had not occurred earlier in fresh concrete.

(2) Residual tensile stresses in wall reinforcement are likely to be less than 30 ksi and certainly in the linearly elastic range of the Grade 60 reinforcement.

(3) There is no evidence to indicate that the strength of the building is less than that assumed in its design.

It should be emphasized that the function of the Diesel Generator Building is well within the range of the experience which supports the theory and practice of reinforced concrete building construction. The existence of discontinuities in the concrete is a condition anticipated by ordinary methods of design for reinforced concrete structures. A crack in a concrete wall or beam is not comparable to a discontinuity in, for example, a steel plate girder. Continuity in tension of reinforced concrete structures is effected not by the concrete but by the reinforcing bars. Therefore, there is no need to reanalyze the building using a model to reflect the effects of tensile discontinuities implied by the cracks.

INTRODUCTION

The walls of the reinforced concrete structure to house the emergency diesel generators for the Midland Power Plant Units 1 and 2 have been observed to have developed cracks ranging in width up to a recorded maximum of 0.028 in. The object of this report is to study the widths and arrangement of the cracks to determine the conditions leading to cracking and the possible consequences of the existing cracks on the strength of the structure.

This report was written at the request of Bechtel Associates Professional Corporation, Ann Arbor, Michigan. In addition to a visit to inspect the Diesel Generator Building, the writer had access to information provided in the following Bechtel documents:

(1) Crack mapping sheet 1, February 1980.

(2) Drawing showing cracks surveyed in July 1981.

(3) Drawing SKC-616 showing progress of concrete casting for the Diesel Generator Building.

(4) Drawings C-1001 through C-1039 showing concrete outlines and reinforcement details.

(5) Response to NRC Question 14, containing a figure showing crack patterns in the walls of the Diesel Generator Building (dated 24 April 1979).

(6) Response to NRC Question 28, containing a figure differentiating cracks surveyed during December 1978 and cracks surveyed after September 1979 (dated February 1980).

(7) Response to NRC Question 40.

DIESEL GENERATOR BUILDING

The Diesel Generator Building is a stiff box-like structure covering an area of approximately 70 x 155 ft. Its plan and sections are shown in Fig. 1 and 2. Exterior walls are 30-in. thick. The interior space is divided into four cells of approximately equal size by three 18-in. thick interior walls running north-south. All interior and exterior walls are supported by continuous strip footings (10 by 2 ft 6 in. in cross section). The walls rise from an elevation of 628 (bottom of footing) to 680 (top of roof slab). The long exterior walls on north and south sides of the building have various openings as indicated in Fig. 1.

The design compressive strength for the concrete in the walls was 4000 psi. Uniformly spaced wall reinforcement is provided by Grade 60 No. 7 (interior wall) and No. 8 (exterior wall) bars at 12 in. each way near each face of wall. The uniform reinforcement ratios in both the horizontal and vertical directions are 0.56% for the interior and 0.44% for the exterior walls.

Because it houses the generators to provide power in an emergency, the Diesel Generator Building is classified as being in Seismic Category I. The building must maintain its integrity if subjected to an earthquake motion having an intensity equal to that of the motions postulated for the "safe shutdown earthquake (SSE)." It must also resist forces and missiles generated by tornados.

The building is founded on plant fill. Casting of the concrete structure was started in October 1977. Because the observed settlement of the building exceeded the estimated amount, construction was halted during August 1978. At the time the construction was stopped, walls had been completed to an elevation of approximately 662. Distribution of the settlement observations made indicated a slight "tilt" of the building, the southwest corner settling perceptibly more than the northeast corner. It was reported that the fill was settling away from the building under the footing of the east wall. These phenomena suggested that the duct banks (Fig. 3 and 4) had made contact with the footings of the interior walls and the east wall.

In November 1978 the duct banks were separated from the footings. Changes in settlement are illustrated schematically in Fig. 5. Construction was resumed in December 1978. To ameliorate future settlement of the fill, a surcharge (approximately 20 ft of sand) was placed to cover the construction site. The structure was completed in April 1979. Surcharge was removed in August 1979.

Figure 6 shows the cracks observed in December 1978 on the surfaces of the north-south walls up to an elevation of 664. A cursory review of the crack patterns suggests their compatibility with the settlement history of the structure. Cracks on the west wall, which did not have a duct bank below it, are of the type clearly attributable to ordinary volume-change effects of the concrete. On the other hand walls with duct banks beneath them have some cracks which imply a systematic stress pattern attributable to a support placed near the position of the duct banks. The cracks observed in the center wall provided the strongest indication of the pressence of such a support. The cracks shown in Fig. 6 are those which were measured to have widths of at least 0.01 in. Maximum crack width measured was reported to be 0.028 in. After the duct banks were separated from the footings there was observed a general reduction in width of the larger cracks near the duct banks.

Cracks on the north and south walls of the Diesel Generator Building were generally smaller in width. Their distribution indicates that they were caused primarily by volume-change tendencies of the concrete.

WALL STRESSES CAUSED BY TEMPORARY SUPPORT FROM THE DUCT BANKS

A schematic representation of the center wall is shown in Fig. 7. Soil reaction on the footings is represented by a series of springs. The effect of the duct bank, after it comes into contact with the bottom of the wall footing, is interpreted as a reaction provided by a very stiff spring.

Consider a particular stage during the construction of the wall. Concreting of portions A and B has been completed, in that order, within a few days of each other. Approximately two weeks later, after the concrete in portions A and B has hardened, lifts C and D are placed in succession. Because of the eccentricity of the reaction provided by the duct bank, the building is likely to tilt to the south as it settles. A limiting conditon is one in which the portion of the wall north of the duct spring is lifted off the springs representing soil reaction. Load-dependent stresses in the wall corresponding to this limiting condition may be estimated from an analysis of the stresses in a linearly elastic model of the "cantilevered" portion of the wall shown in Fig. 8.

The elevation and section shown in Fig. 8 represent the hardened concrete in portion A of the center wall up to elevation 650 (Fig. 7). It is assumed that the reaction of the duct bank may be concentrated as a line load at a point 22 ft from the inside face of the north wall, as shown in the lower left-hand corner of the wall elevation. The horizontal links represent the restraint of the rest of the wall to the south of the support.

The pressure of 12.5 psi on the upper surface of the wall represents the effect of the fresh concrete in lift C (Fig. 7).

The edge load represents part of the weight of the north wall. When included in the analysis, it was applied along the vertical edge uniformly except for a heavier concentration at the top to represent the weight of fresh concrete above that level.

Young's modulus of the concrete was assumed to be $4 * 10^6$ psi. Poisson's ratio was taken as zero. Density of reinforced concrete was set at 150 lb/cubic ft.

Internal stresses were analyzed for two conditions: (a) for zero edge load and (b) for a nominal distributed edge load of 200,000 lb. In both cases self-weight and pressure on top surface were included.

Horizontal stresses calculated on a vertical plane one foot away from the left face of the wall segment (fixed edge) considered are plotted in Fig. 9 for both solutions.

The tendency of both tensile stress distributions to increase near the effective neutral axis is due the "bursting" stresses caused by the concentrated reaction at the bottom flange.

The reason for showing two stress distributions in Fig. 9 is to emphasize the indeterminacy of the actual loading conditions on the wall. The edge load could be considerably higher than that assumed. The range of the calculated tensile stresses suggests that stresses of a magnitude to cause cracks in the hardened concrete would have existed in the wall in the vicinity of the duct bank at a time when the concrete in lifts C and D (Fig. 7) was fresh.

It is important to note that the analysis above demonstrates that cracking would have occurred after casting of lifts C and D but it does not preclude the appearance of cracks to accommodate settlement deformations before that stage in construction. In reference to Fig. 7, it will be appreciated that stress-related cracking depends on resistances and stiffnesses (strength and stiffness of the concrete as well as the stiffnesses of the duct banks and the supporting soil) which are all time-dependent. Complex as these combinations are, they are further complicated by construction events. To reconstruct the stress/strength interaction loading to cracking of the concrete is virtually impossible but also unnecessary. If no cracks had formed before the construction stage considered, calculations indicate that cracks would have formed then and consistently with the observations of settlements and crack patterns.

In relation to the observed phenomena, it is of interest to investigate the progress of a crack in the wall once it is initiated.

Figure 10 shows the reinforcement in the wall segment considered above. The relationship between crack height and resisting moment was determined assuming a direct tensile strength of $4 \sqrt{f'_c}$ for the 4000 psi concrete. Yield stress of all reinforcement was assumed to be 60,000 psi. Calculations were made with the bottom edge of the wall in compression.

The calculated relationship is plotted in Fig. 11. It illustrates two inherent features of crack development in a reinforced section subjected to flexure.

It is noted that after cracking occurs at a moment of approximately 8,500 kip-feet, there is a drop in resistance. Theoretically, the crack would penetrate almost to the flange (the footing) before the section redevelops a moment of comparable magnitude.

Even though the wall is adequately reinforced (ρ = 0.0056), the reinforcement is distributed over its height rather than being concentrated near the extreme fiber in tension. Consequently, the flexural crack penetrates deeply into the section before sufficient reinforcement force is mobilized to compensate for the loss of the tensile strength of the concrete.

It may also be noted from Fig. 11 that after the crack penetrates about 12 ft into the section, the slope of the curve becomes positive. Its progress is controlled after a penetration of approximately 17 ft. Equilibrium of internal forces and external effects is re-established.

The extent of the cracks observed especially in the center wall is quite consistent with the expected behavior of reinforced concrete sections subjected to flexure. It should, however, be remembered that the walls of the Diesel Generator Building are not likely to be subjected to flexural stresses of this magnitude in their normal function because the duct banks have been separated from the footings and because the building is now complete. The overall depth of the section is now over 50 ft rather than 22 ft as considered in the calculations.

RESIDUAL STRESSES

Figure 10 shows the trajectories of the cracks recorded in July 1981 on east face of the center wall. Cracks shown are those having widths of 0.01 in. or larger. East face of the center wall was chosen for study because it had more and wider cracks than the other walls.

The maximum crack width at the time of the July 1981 survey was reported to be 0.02 in.* This is less than the maximum of 0.028 in. observed earlier. The reduction in width is consistent with the result of the calculations in the previous section which supported the observation that bending stresses caused by the temporary concentrated support contributed to crack formation. Separation of the duct banks from the footings would cause the wall cracks in the vicinity of the duct banks to reduce in size. On the other hand, these cracks would not be expected to close completely because the concrete surfaces bounding the cracks are not likely to fit perfectly and because the foundation profile is not likely to have returned to precisely the shape it had before opening of the cracks.

It should also be remembered that crack-width measurements made at different times may differ. In addition to the natural scatter in

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*In a CTL survey made in February 1982 (Attachment 4) a maximum width of 0.025 in. was recorded on the center wall.

observation, changes in temperature and humidity may affect the size of the cracks within a short period of time. The small variation in maximum observed crack width from 0.028 to 0.02 in. is consistent with what would be anticipated, given the history of the building.

An estimate of the residual stress in the wall reinforcement may be obtained from the residual crack widths. A brief perspective of the information on the relationship between tensile reinforcement stress and crack width is provided in Attachment 1. Crack width estimates or measurements are used typically to make judgments about serviceability and/or durability of a reinforced concrete structure. For that task, the role of the crack-width estimate as an index value is relevant and useful. But the relationships which yield an estimate of the crack width as a function of steel stress, concrete cover and other variables are not typically used in reverse; to determine stress from width measurements. Used for that purpose, they may help provide information as to whether and to what extent yielding may have occurred at a given location. Any quantitative inference made on that basis must be treated as a very rough measure.

It was stated in the previous section that the cracks related to the support from the duct bank could have occurred before concrete in portions A and B (Fig. 7) hardened. In the following discussion, it will be assumed that cracks occurred in mature concrete and within a short period of time, thus leading to upper-bound estimates of residual stress in the reinforcement.

The simplest and most direct method for estimating steel stress from crack-width data is to use the data simply as a measure of bar extension.

Crack widths were measured at two levels on the east face of the center wall (Fig. 12). Widths measured at the upper level (elevation of approximately 645) are seen to add to a larger sum than those in the lower level. Considering the sum of crack widths at the outer level between two 0.02-in. cracks indicated by the letter B and assuming that the one crack not measured at that level had a width of 0.01 in. as measured at the lower level, the total extension is found to be approximately 1/8 in. The length, L_1, over which this extension is assumed to have taken place is approximately 150 in. The corresponding strain, ϵ_s, is approximately 0.008 and the related steel stress

$$f_s = \epsilon_s * E_s = (.125/150) * 29 * 10^3$$

$$= 24 \text{ ksi}$$

Considering the reliability of the crack width measurements and the probability of the very small cracks in this area not being reported, the plausible conclusion from this attempt is that the residual stress would be in the range 20 to 30 ksi if the crack occurred in mature concrete. The strong inference is that the reinforcement is in the linear (elastic) range of response.

To obtain another perspective of the residual steel stresses in relation to crack widths, it is instructive to attempt a calculation of the crack width using a predictor expression of the type described in Attachment 1. The conditions under which stress-related cracking is assumed to have occurred in the walls of the Diesel Generator Building are not typical of conditions in beams. Therefore, the predictor expression chosen is one developed by Holmberg and Lindgren (Attachment 3) from data obtained using wall elements in direct tension. Using the metre as a unit of length, Holmberg and Lindgren give the mean crack spacing, ℓ_c, for a wall (with all bars having the same diameter) as

$$\ell_c = 0.055 + 0.144\ (A_e/d_b)$$

where

A_e = is the "effective" concrete area around the bar

d_b = bar diameter

Holmberg and Lindgren tested wall segments with centrally located reinforcement in a specimen thickness representing twice the cover of the bars in the wall. Adopting their approach, for bars spaced at 12 in. with the distance from center of bar to near face of wall assumed to be 2.5 in.,

$$A_e = 12 * 5 = 60\ in^2 = 0.039\ m^2$$

resulting in

$$\ell_c = 0.055 + (0.144 * .039/0.022) = 0.31\ m\ \text{(approx. one ft)}$$

To obtain an estimate of the characteristic crack spacing, Holmberg and Lindgren multiply the mean calculated crack width by 1.4. To obtain the corresponding maximum crack width, a magnification factor (based on statistical data on crack width distribution) of 1.7 is used. For the walls of the Diesel Generator Building the maximum calculated crack width for a stress of 20 ksi would be

$$w_m = 1.7 * 1.4 * 0.31 * (20/29 * 10^3) = 0.5 * 10^{-3} \text{ m}$$

$$= \underline{0.02 \text{ in.}}$$

This result indicates that, on the basis of the experimental data obtained by Holmberg and Lindgren, the wall considered (for concrete cover and reinforcement amount specified) would be expected to develop a maximum crack width of approximately 0.02 in. for a bar stress of 20 ksi.

The calculations using the Holmberg-Lindgren expression confirm that a maximum residual crack width of 0.02 in. in the walls of the Diesel Generator Building implies a residual reinforcement stress of less than 30 ksi, well in the linear range of response of the Grade 60 reinforcement.

EFFECT OF EXISTING CRACKS ON WALL STRENGTH

Reinforced concrete structures are designed and built with the explicit assumption that concrete will crack. Appearance of cracks on structural components of a reinforced concrete building provide no cause for re-evaluation of the strength of the structure unless the cracks indicate general yielding or are related to imminent failure in shear or bond. Attachment 2 contains a discussion of the strength and behavior of "precracked" reinforced concrete structures, or elements with cracks which occurred before the application of a particular loading program.

Field observations and the analysis in this report indicate that cracks in the walls of the Diesel Generator Building were caused generally by ordinary volume change effects and locally, in some of the north-south walls, by tensile stresses resulting from temporary support of the duct banks.

Analysis of the stress conditions created by the temporary supports indicates that cracks could have occurred in fresh concrete during the setting of concrete in portions A and B of the center wall (Fig. 7) or, if it did not occur then, in mature concrete after the casting of lifts C and D. In either case, the cracks would be related primarily to bending deformation. There is no evidence, visual or analytical, to attribute the cracks to shear or bond-failure mechanisms.

All available evidence indicates that the residual stresses in the wall reinforcement are well within the linear (elastic) range of response of the material. Furthermore, residual reinforcement stresses associated with the existing cracks are on planes unlikely to be subjected to high normal tensile stresses under postulated design-load combinations.

The function of the Diesel Generator, unlike that of a containment vessel, is within the experience record which has led to the theory and practice of reinforced concrete construction. There is no reasonable cause for concern about the consequences of the cracks in question, except for protection of the steel from any unusual aggressive environment. Examples of the behavior of reinforced concrete elements subjected to axial load, bending, and shear after having been cracked as a result of other loading conditions are provided in Attachment 2.

Currently, there is no indication that the strength of the walls of the Diesel Generator Building is less than that assumed in the original design. Design methods for reinforced concrete structures have been based on the assumption that concrete does not provide resistance to normal tensile stresses. The presence of cracks in the walls of the Diesel Generator Building does not represent a condition which would require special procedures for modeling the existing structure.

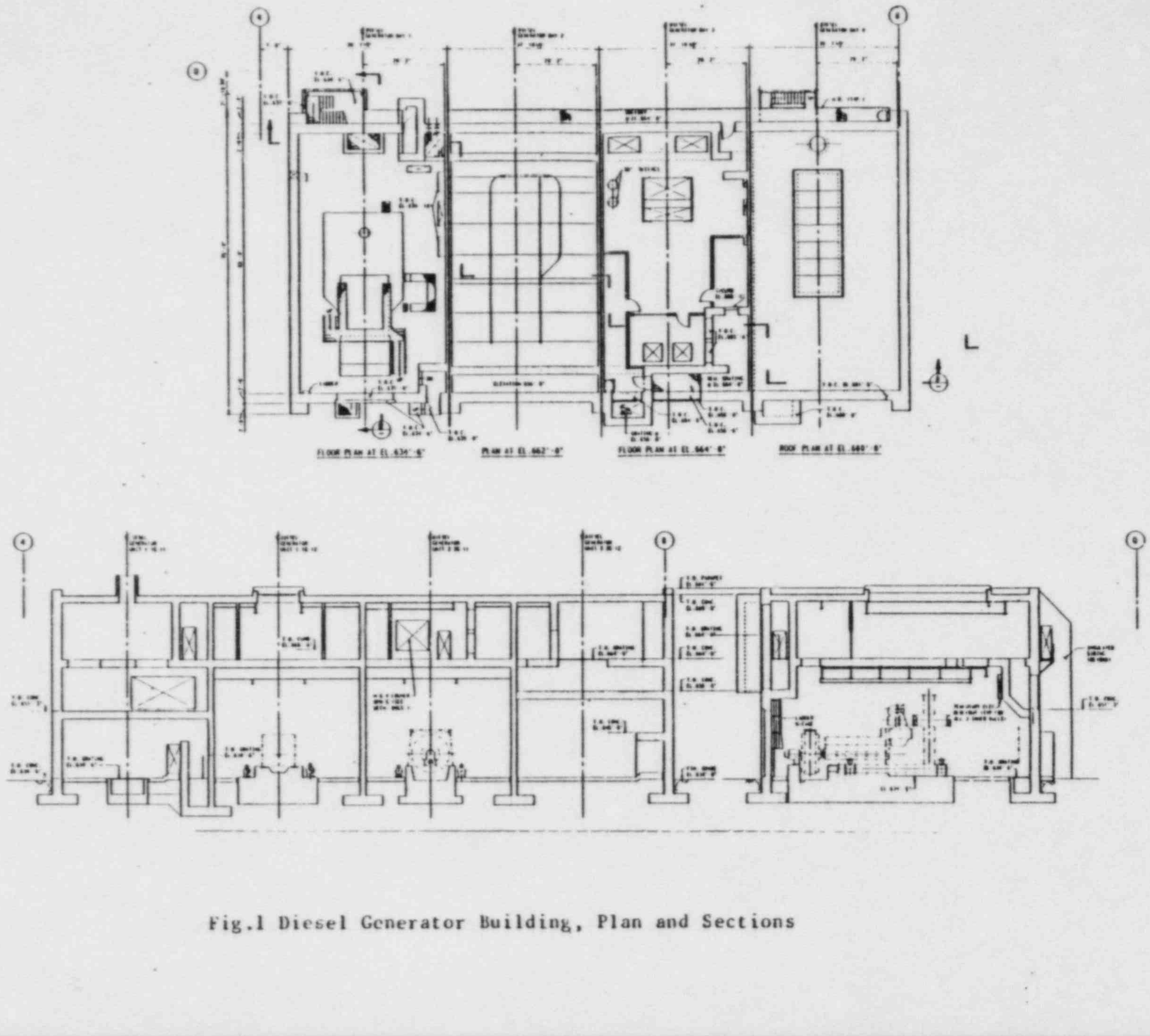

Fig.1 Diesel Generator Building, Plan and Sections

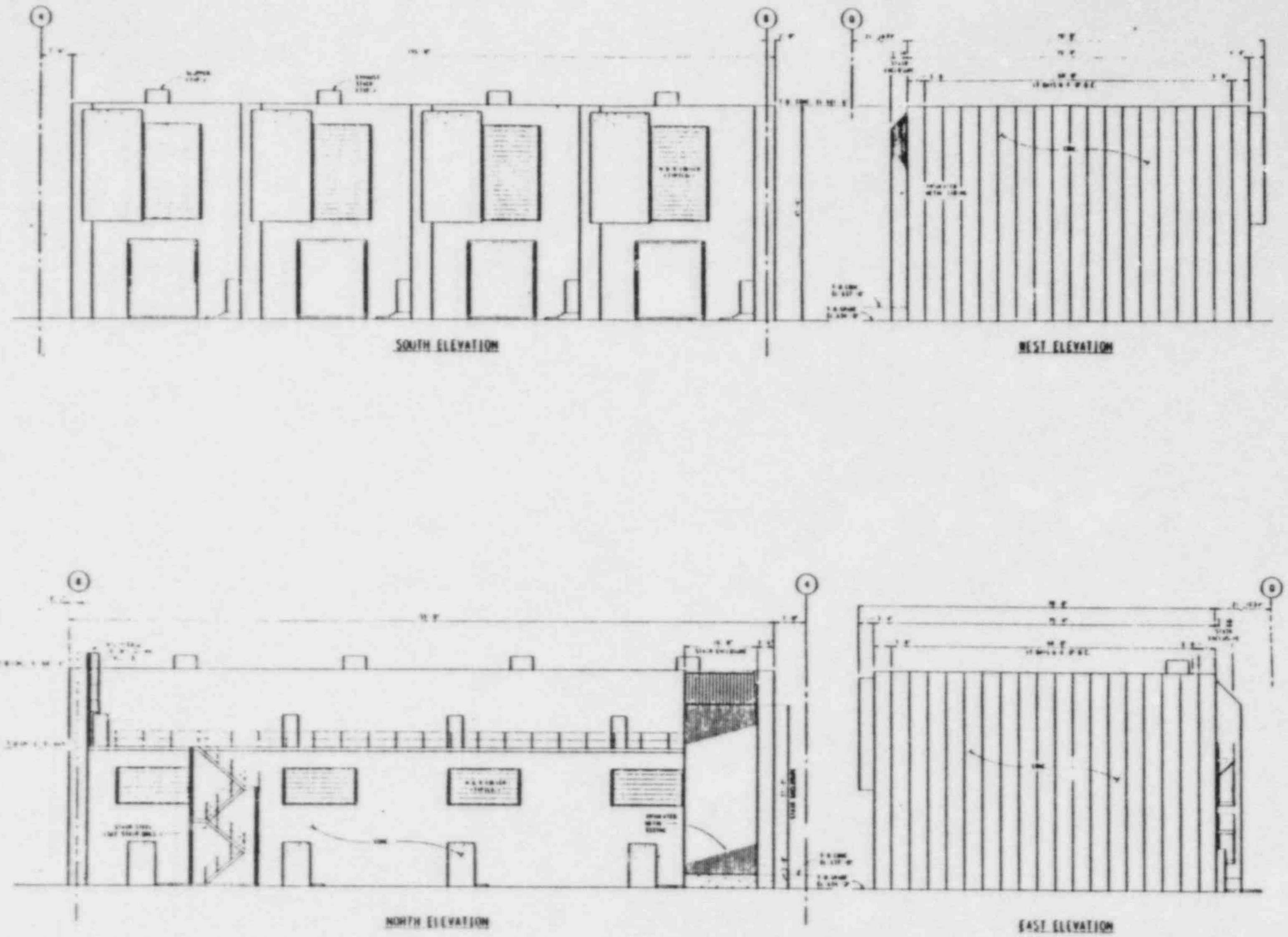

Fig.2 Diesel Generator Building, Elevations

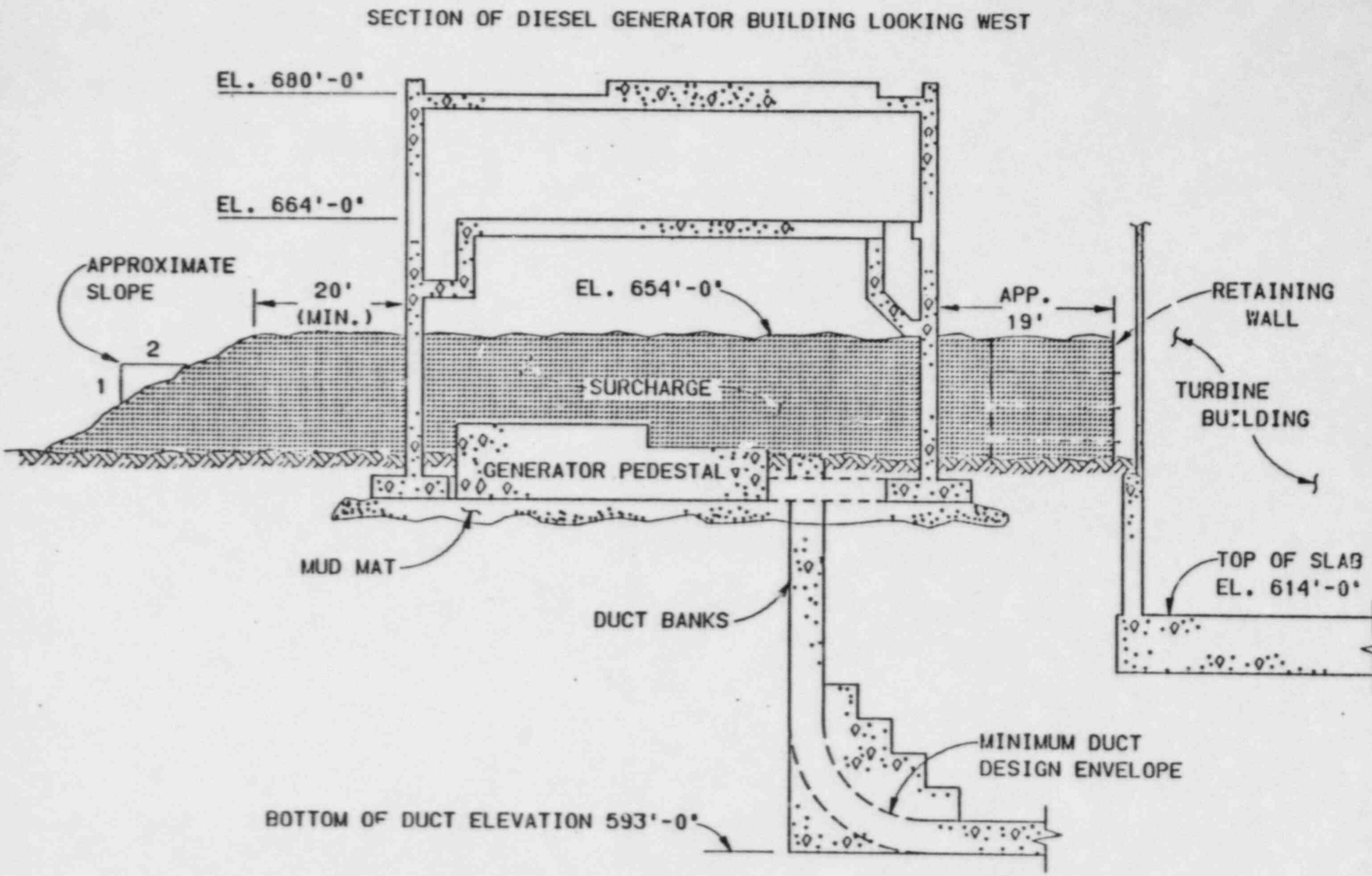

Fig.3 Duct Bank Elevation

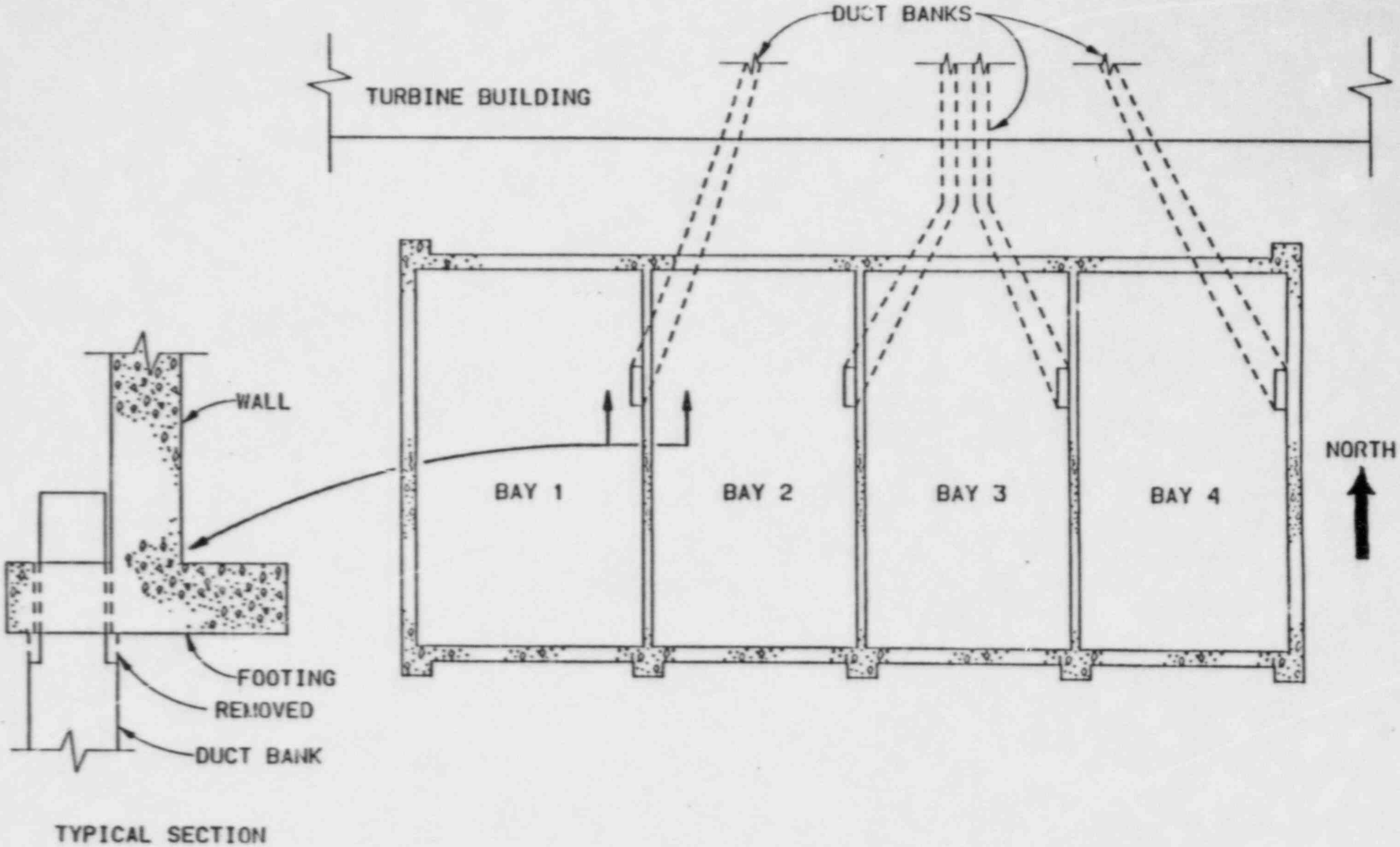

Fig.4 Duct Bank Layout

PLANE A NOMINAL REFERENCE PLANE

PLANE B SETTLEMENT PLOT AS OF 11/16/78
BEFORE CUTTING DUCT BANKS LOOSE

PLANE C SETTLEMENT PLOT AS OF 12/28/78
APPROX. A MONTH AFTER CUTTING DUCT BANKS

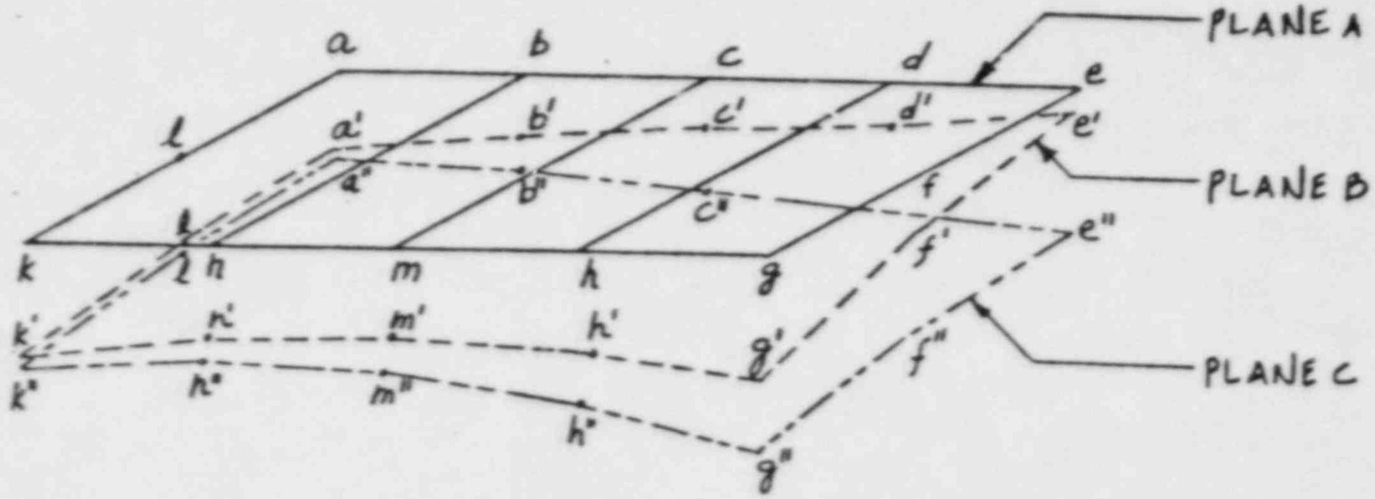

Fig.5 Trend of Measured Settlement

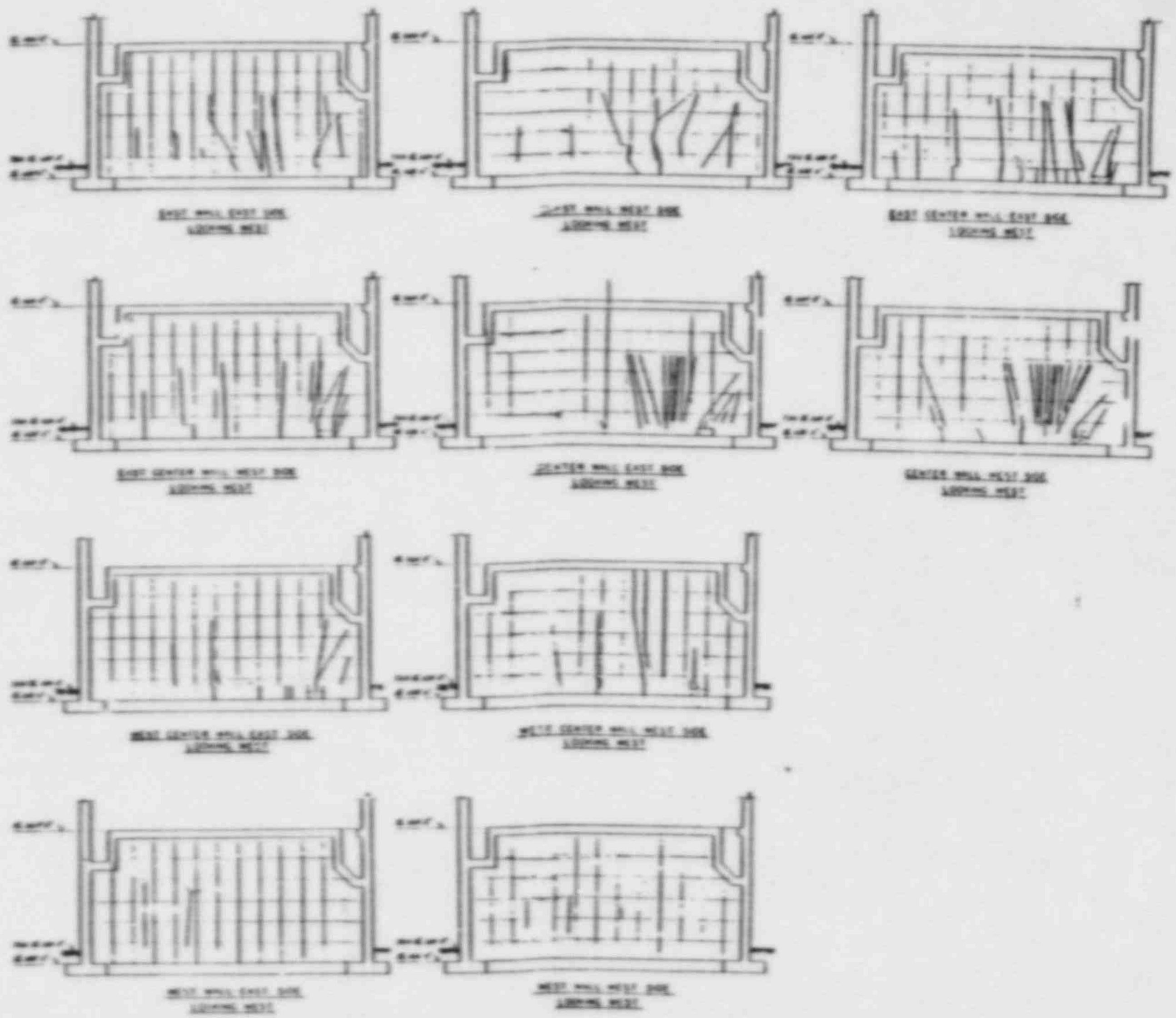

Fig.6 Cracks Mapped, December 1978

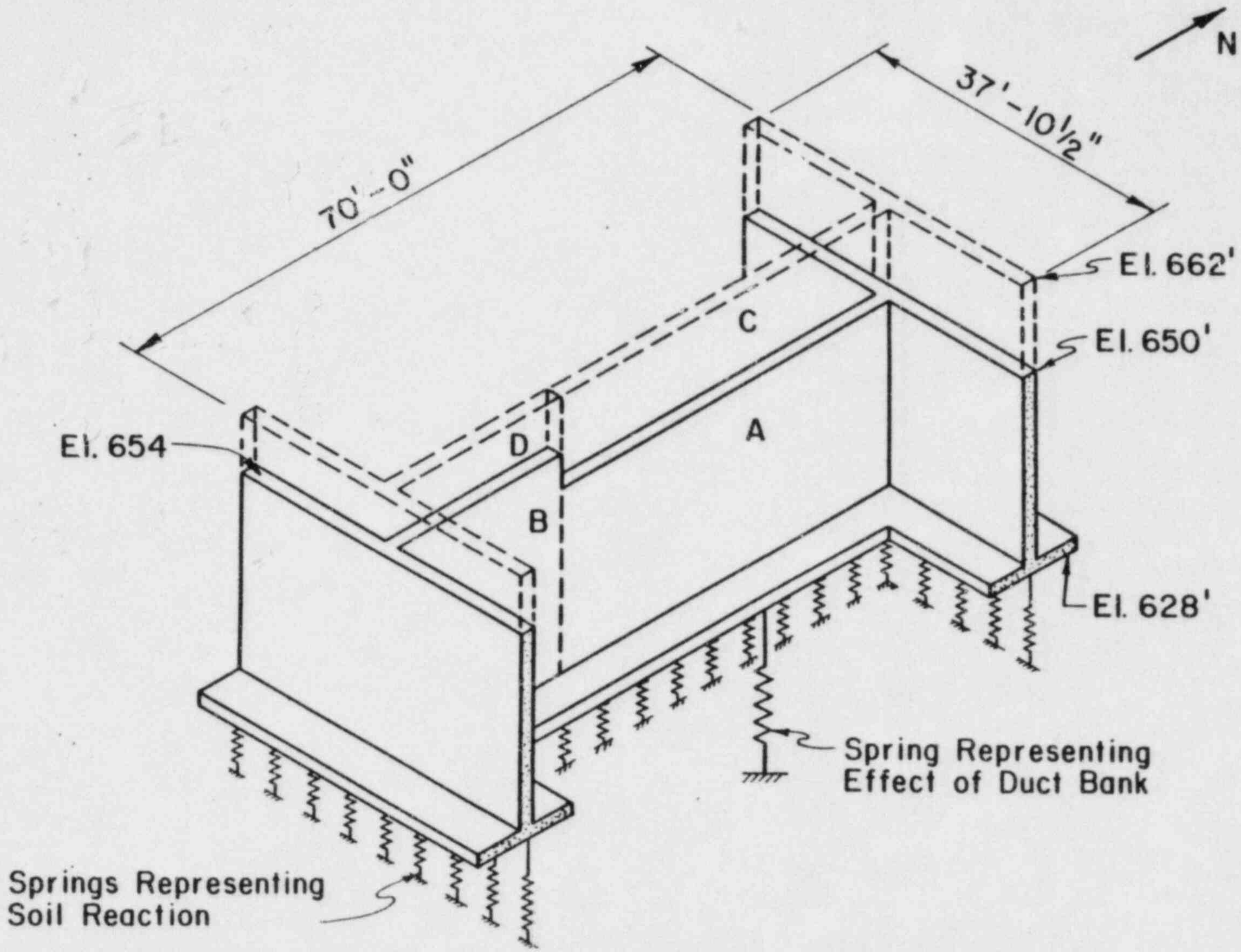

Fig.7 Center Wall

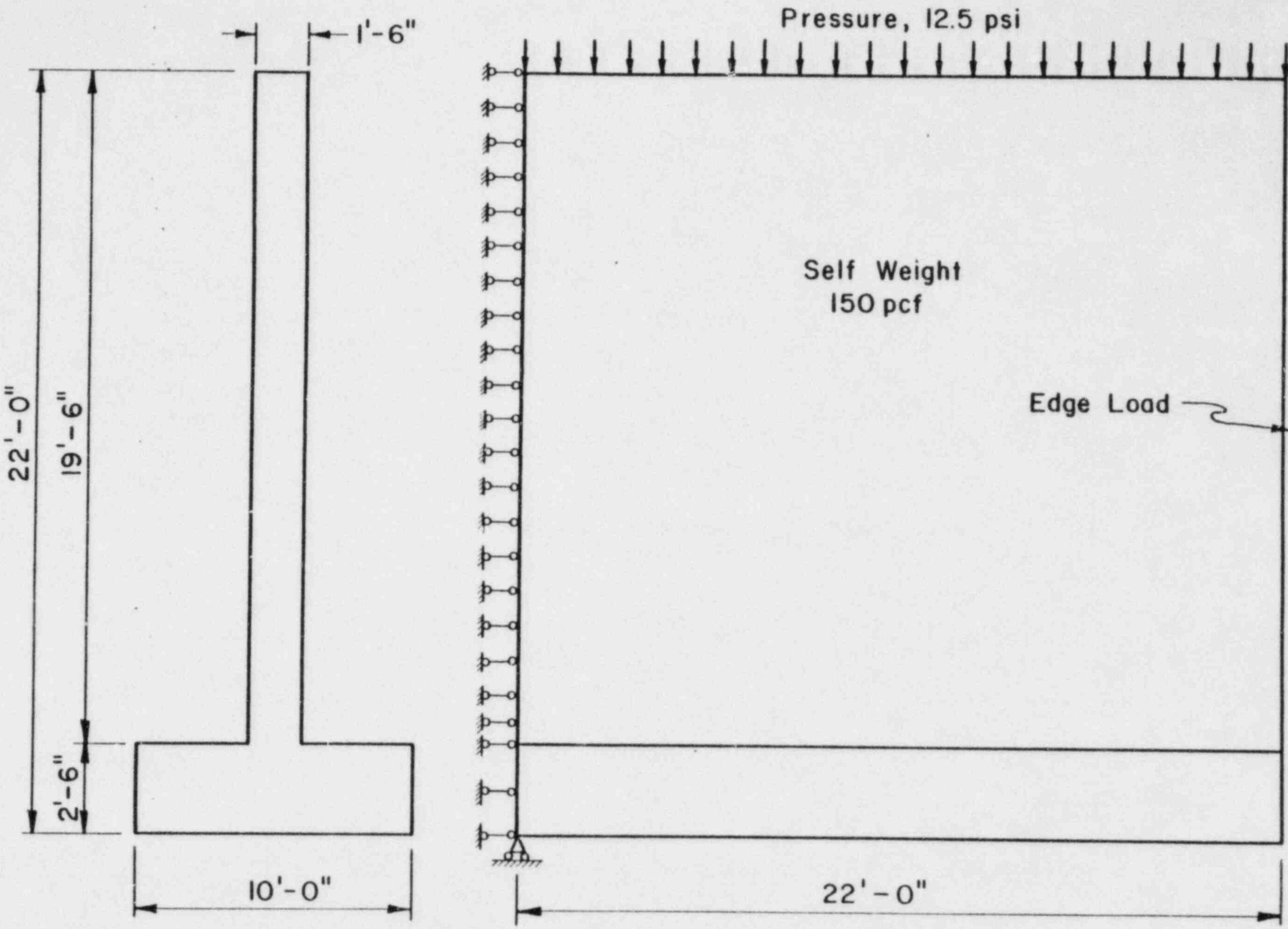

Fig.8 Portion of Center Wall to Elevation 656.0
from Assumed Reaction Point to North Wall

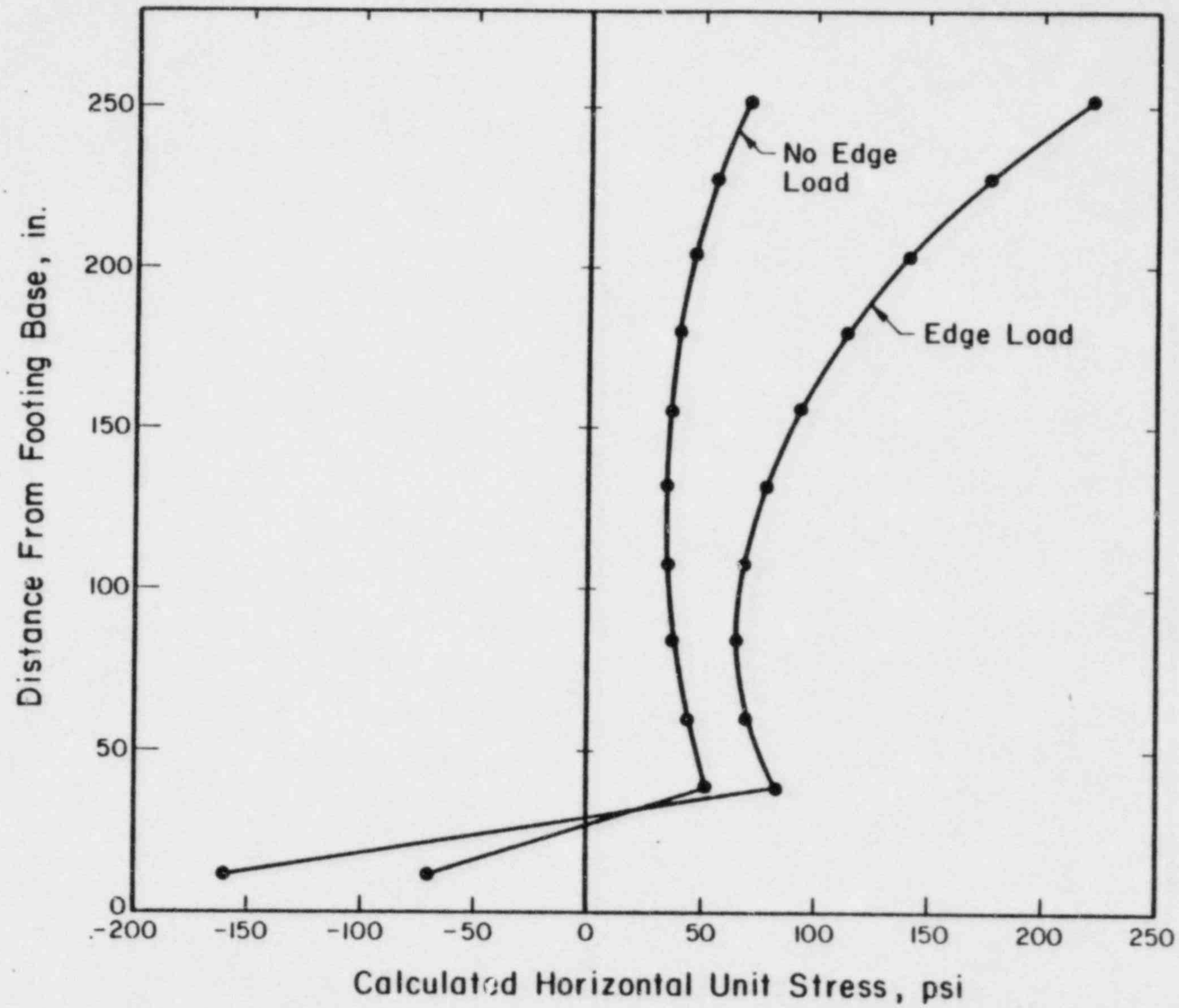

Fig.9 Distribution of Calculated Horizontal Normal Stress at 12-in. from "Fixed" Edge

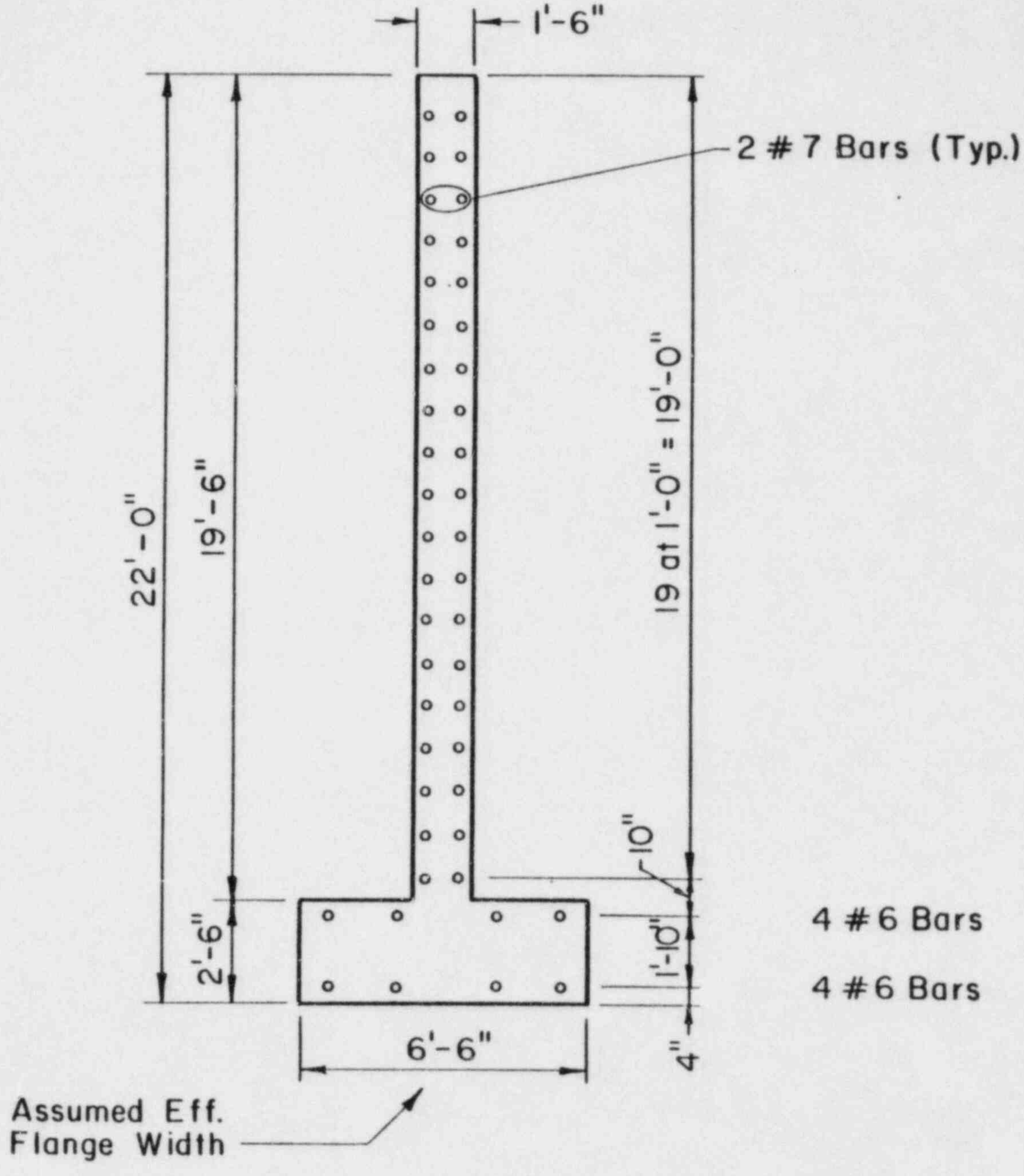

Fig. 10 Cross Section of Center Wall to Elevation 650.0

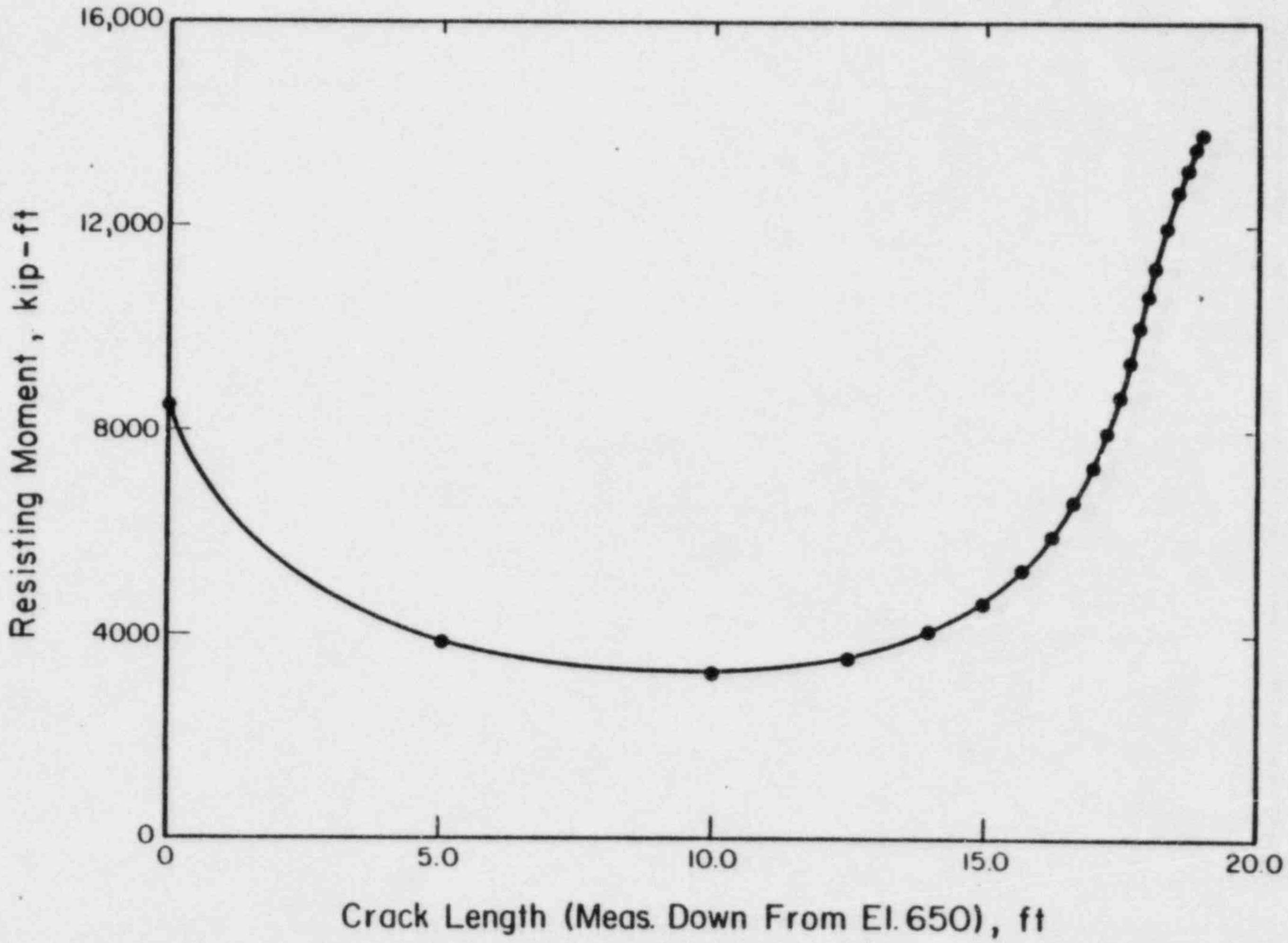

Fig. 11 Calculated Relationship Between Resisting Moment and Length of Crack (Measured from Elevation 650.0 down toward Wall Footing)

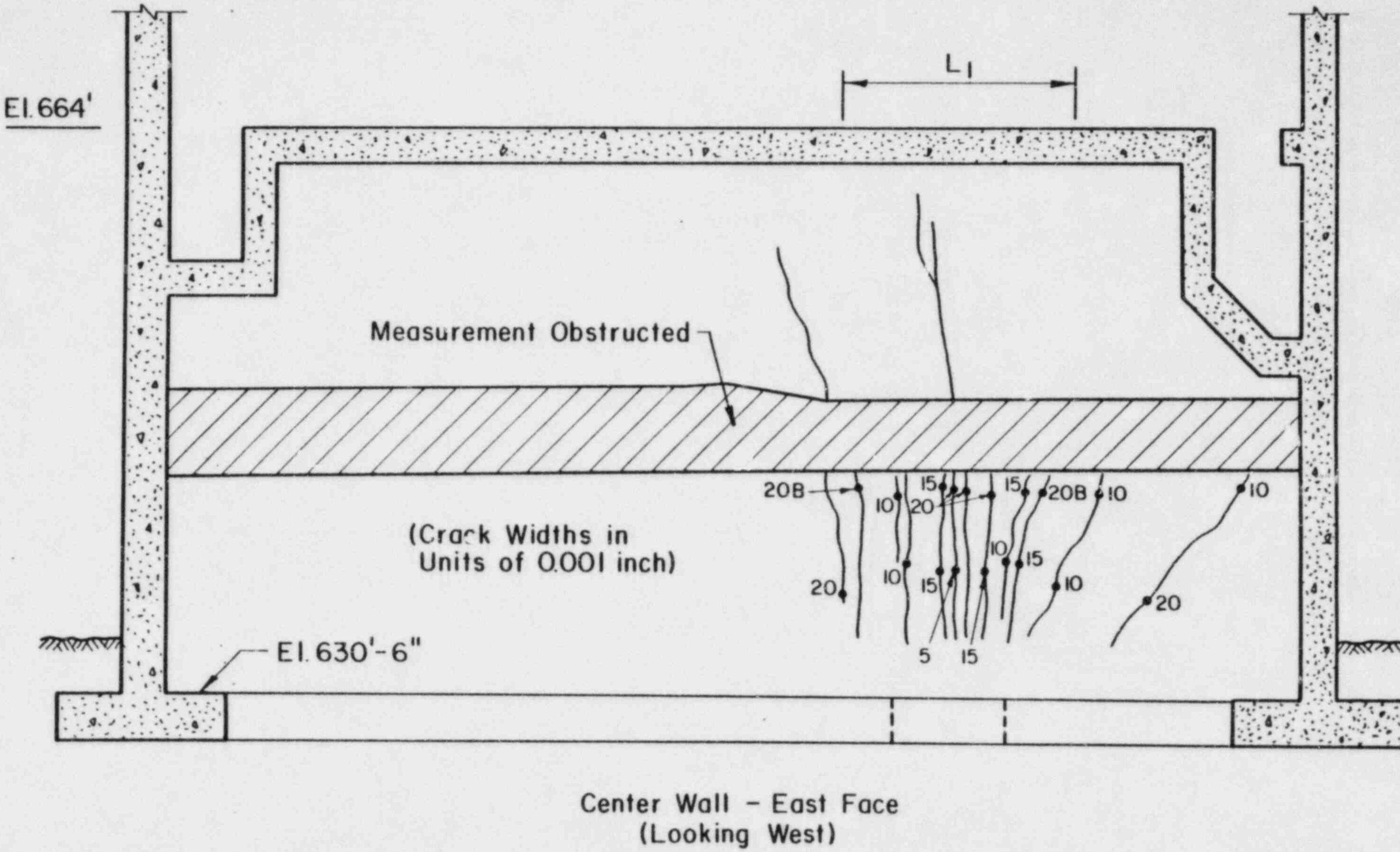

Fig.12 Crack-Width Measurements, July 1981

ATTACHMENT 1

CRACK DEVELOPMENT IN CONCRETE

Summary

This attachment has been prepared to provide a perspective of the development and use of predictor expressions for crack width. Derivations of the two common types of predictor expressions are described and a specific example of each type is used to calculate crack widths for a test beam.

Introduction

Tensile strength of concrete made with normal weight aggregate is approximately a tenth of its compressive strength. The low strength in tension is not compensated by a low Young's modulus. Initial modulus of concrete in tension is comparable to its modulus in compression. Furthermore, the limiting strain in tension is also low, approximately 0.0002. These properties combine to make concrete quite susceptible to cracking.

Cracking is not necessarily related to stresses generated by loads or externally imposed deformations. Much of the cracking in elements having low apparent stress levels is caused by time/temperature dependent volume changes or by chemical reactions causing local deformations (such as rusting of embedded reinforcement or expansion of aggregates). In general, cracks unrelated to load or imposed deformations are attributable to restraints on dimension change resulting from heating/cooling or expanding/shrinking.

Limiting the perspective to phenomena in one dimension only, a qualitative understanding of events leading to a crack as a result of volume change may be obtained with the help of Fig. 1.1.

The concrete prism ABCD is assumed to be perfectly insulated on faces AD and BC as well as on faces parallel to the plane of the paper so that there is no loss of heat and moisture on those faces. It is also assumed that there is no external restraint on any face of the prism.

At a given time after the concrete is cast, the unreinforced concrete prism ABCD may be expected to assume the shape described by the broken lines. The change in shape is the result of differential shrinkage (moisture content in regions closer to the free boundary is expected to diminish at a faster rate) or thermal gradient (assuming in this case an ambient temperature on the free boundary lower than that at longitudinal axis of prism, a typical state during setting of cement).

Considering the thin planar element PQRS, it is concluded from the free-body diagram in Fig. 1.1b that restraint forces along RS will result in a tensile force on edge QR.

Because it is produced by dimensional changes varying with time, the tensile stress on edge QR varies with time. The effective tensile stress, represented by the broken curve in Fig. 1.2, is the result of a complex interaction among variations with time of shrinkage, temperature, stiffness modulus, and creep, the last two also varying as a function of the stress level.

The solid curve in Fig. 1.2 represents increase in tensile strength with time. Ideally, when the two curves intersect, the crack occurs.

Even if the events are limited to the simple one-dimensional environment considered here, it may be inferred from the figure that exactly when the crack would form would be very difficult to predict because of the typical scatter-band widths of the two time functions in Fig. 1.1c. It should also be noted that, depending on the relative humidity and temperature on the free boundary, dimensional changes caused by shrinkage and thermal effects may reverse.

It is a statistically established truth that hardened concrete is likely to contain cracks especially at planes not having sustained compressive stress. The mechanism described in reference to Fig. 1.1 simply rationalizes in one dimension how cracking can occur without the necessity of stress generated by load or imposed deformation.

Relationship Between Crack Width and Reinforcement Stress

General concepts used to relate crack width to steel stress in terms of properties of the concrete section refer to the simplified model in Fig. 1.3: a concrete prism cast around a reinforcing bar. It is assumed that the crack occurs in mature concrete and as a result of tensile stress in the embedded bar.

If a sufficiently large tensile force is applied at both ends of the bar, the prism will crack ideally at equal intervals. The interval (crack spacing) is denoted by the notation ℓ_c.

The width of the crack at steel surface can then be calculated using the usual definition of strain.

$$w_o = \int_{\ell_c} (\varepsilon_{sx} - \varepsilon_{cx})\, dx \tag{1}$$

w_o = crack width

ε_{sx} = steel strain at point x

ε_{cx} = concrete strain at point x

If ε_{sx} is large compared with ε_{cx}, the variation of ε_{cx} may be neglected which also suggests that the crack width might as well be considered at the surface of the concrete, a more convenient location for measuring crack width. If the variation of steel stress over ℓ_c is small, the elongation may be written directly in terms of ε_{sm}, the mean steel strain without introducing intolerable error.

$$w_o = \varepsilon_{sm}\, \ell_c \tag{2}$$

As it would be expected, there is no controversy about the use of Eq. 2. However, there are different plausible approaches to organizing the variables in order to obtain the crack interval ℓ_c.

One of the popular approaches to determining ℓ_c from experimental data is very simple. In essence, it is patterned after the problem of stress trajectories in a "semi-infinite" solid subjected to a concentrated load on its boundary.

Consider the concrete cube in Fig. 1.4 with concentrated colinear tensile forces applied at both ends of a central steel bar fully bonded to the concrete. The distance from the loaded boundary at which there will be a surface crack depends on the dispersion rate of the stresses within the cube. From this idealization, the important variable determining crack spacing is seen to be the concrete cover, c. Thus, in evaluating experimental data, the basic equation may be set up as

$$\ell_c = \alpha c \tag{3}$$

α = constant to be determined experimentally

Another approach to the interpretation of crack-interval observations is illustrated in Fig. 1.5.

Figure 1.5a describes idealized conditions immediately before cracking. Bond between steel and concrete transfers the tensile force at a varying rate from the reinforcing bar to the concrete. At a point where the tensile strength of the concrete section is exceeded, the crack occurs. For the hypothetical example considered, this point has been selected to be at the middle of the prism length.

Figure 1.5b shows ideally the stress conditions after development of the first crack. According to the assumptions made, development of other cracks depends on whether bond is sufficient to transfer the force necessary to crack the section in approximately half the length available for transfer.

For a number, m, of bars of equal diameter, d_b, conditions leading to cracking according to this hypothesis may be expressed symbolically as shown below.

Tensile force transferred to concrete by bond over a length ℓ_c = tensile force necessary to crack concrete section.

$$m \, \pi \, d_b \cdot \int_{(\ell_c/2)} u \, dx = A_c \, f_t \tag{4}$$

where

m = number of bars

d_b = bar diameter

u = bond stress

A_c = area of concrete section

f_t = tensile strength of concrete

Introducing the definition of reinforcement ratio as

$$\rho = \frac{m \, \pi \, d_b^{\,2}}{4 \, A_c}$$

and assuming that bond stress is uniformly distributed along the length of the bar,

$$\ell_c = \frac{d_b}{\rho} \cdot \frac{f_t}{2u} \tag{5}$$

Assuming further that f_t and u vary similarly with concrete strength, the following equation may be used to evaluate observation of ℓ_c:

$$\ell_c = \beta \, \frac{d_b}{\rho} \tag{6}$$

Recognizing that the experimental constants α and β are dominant and that both mechanisms described may affect the physical phenomenon, Eq. 3 and 6 may be combined

$$\ell_c = \alpha c + \beta \frac{d_b}{\rho} \tag{7}$$

with the understanding that α and β are evaluated for the combined form.

Application of Predictor Expressions for Crack Width

To demonstrate the physical significance of predictor expressions for crack width, it is instructive to apply them to a case for which crack-width data are available.

Crack widths measured in the central constant-flexure span of a girder (G141) measuring 14.75 * 28-in. deep in section and spanning 30 ft were reported in Reference 1.1 The dimensions of the girder which was reinforced in tension with three Grade 60 No. 14 bars are shown in Fig. 1.6. Side cover for the reinforcement was 2-1/4 in.

Measured crack width distributions at various steel stresses from 10 to 42 ksi are illustrated in Fig. 1.7. Widths shown are those measured at the level of the reinforcement on the sides of girder G141.

It is to be noted that the number of cracks increased with steel stress as did the difference between minimum and maximum values. These are typical characteristics of crack-width distributions. They emphasize that a reference to or prediction of a crack width for a given structural element should never be treated as, say, a beam-depth measurement but always as an index to a distribution of measurements.

The arrows in the figure indicate magnitudes of the sum (mean plus two standard deviations). It is seen that this sum agreed quite consistently with the maximum width measured at each stress level.

A predictor expression based on the approach described by Eq. 3 is the one used in Reference 1.1. It is reproduced below as Eq. 8.

$$w_r = \frac{c\ f_s}{5} \tag{8}$$

w_r = reference crack width, defined as the sum of the mean crack width plus two standard deviations (effectively the maximum crack width), in 0.001 in.

c = concrete cover in in.

f_s = steel stress in ksi

Applying it to girder G141 with f_s = 14.0 ksi and c = 2.25 in.,

$w_r = (2.25 * 31)/5 = \underline{14 * 10^{-3}}$ in.

The European Concrete Committee (1.2) uses a predictor equation based on Eq. 7. It is reproduced below in its original units.

$$w_m = (1.5\ c + \frac{16\ d_b}{\rho_e})\ f_s * 10^{-7} \tag{9}$$

w_m = maximum crack width, in cm

c = concrete cover in cm

d_b = bar diameter in cm

ρ_e = percentage of reinforcement in the "tributary" area (area of concrete having its centroid coinciding with the centroid of the steel area)

f_s = steel stress in newtons/cm^2

Using Eq. 9 for G141, it is first necessary to evaluate ρ_e in percent:

$$\rho_e = [(3 * 2.25)/(6.2 * 14.75)] * 100$$

$$= 7.4\%$$

Substituting the relevant data in Eq. 9 for f_s = 31 ksi = 21000 N/cm^2,

$$c = 2.54 * 2.25 = 5.7 \text{ cm}$$

$$d_b = 4.3 \text{ cm}$$

$$w_m = [(1.5 * 5.7) + (16 * 4.3/714)] * 21000$$

$$= 0.038 \text{ cm}$$

$$= \underline{15 * 10^{-3}} \text{ in.}$$

Equations 8 and 9, based on different behavioral models give comparable results for the case considered. Considering that the two predictor expressions have been calibrated to approximately similar populations of data, it is not surprising that they lead to similar estimates of crack width. It is also noteworthy that both overestimate the measured crack width. There are two main reasons for the overestimate. Both expressions were calibrated to ignore the variation of steel strain between cracks. (Steel strain is assumed to be constant even though it reaches a lower value between cracks.) Expressions derived for general application tend to be conservative even in relation to the observed extremes and are likely to overestimate crack widths by varying margins in most cases.

The important feature of these predictor expressions is that, despite their differences, they emphasize that the quantitative relationship between maximum crack width and steel stress is not constant and that it depends on other variables.

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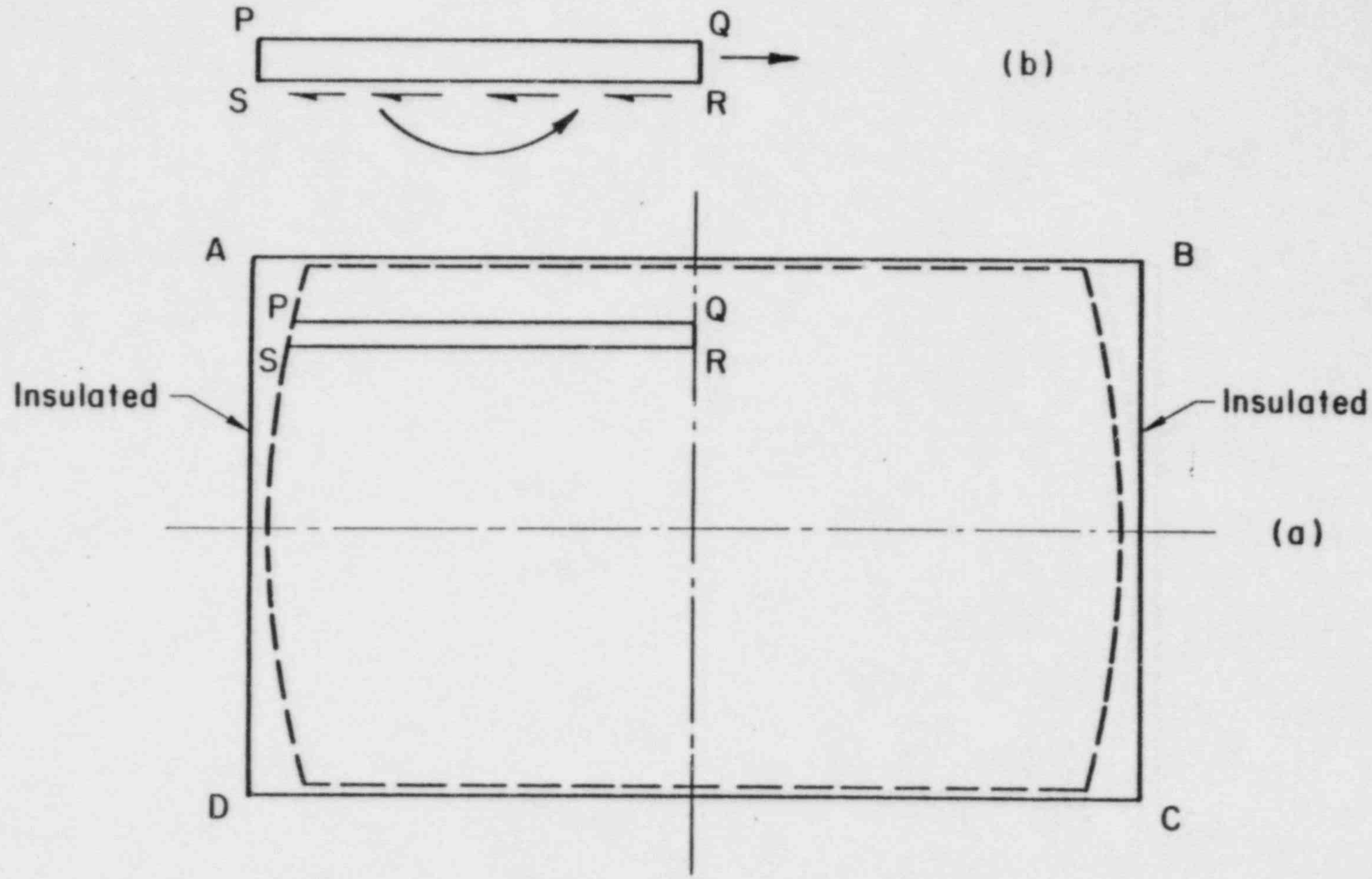

Fig.1.1 Idealized Representation of Volume Change in Concrete

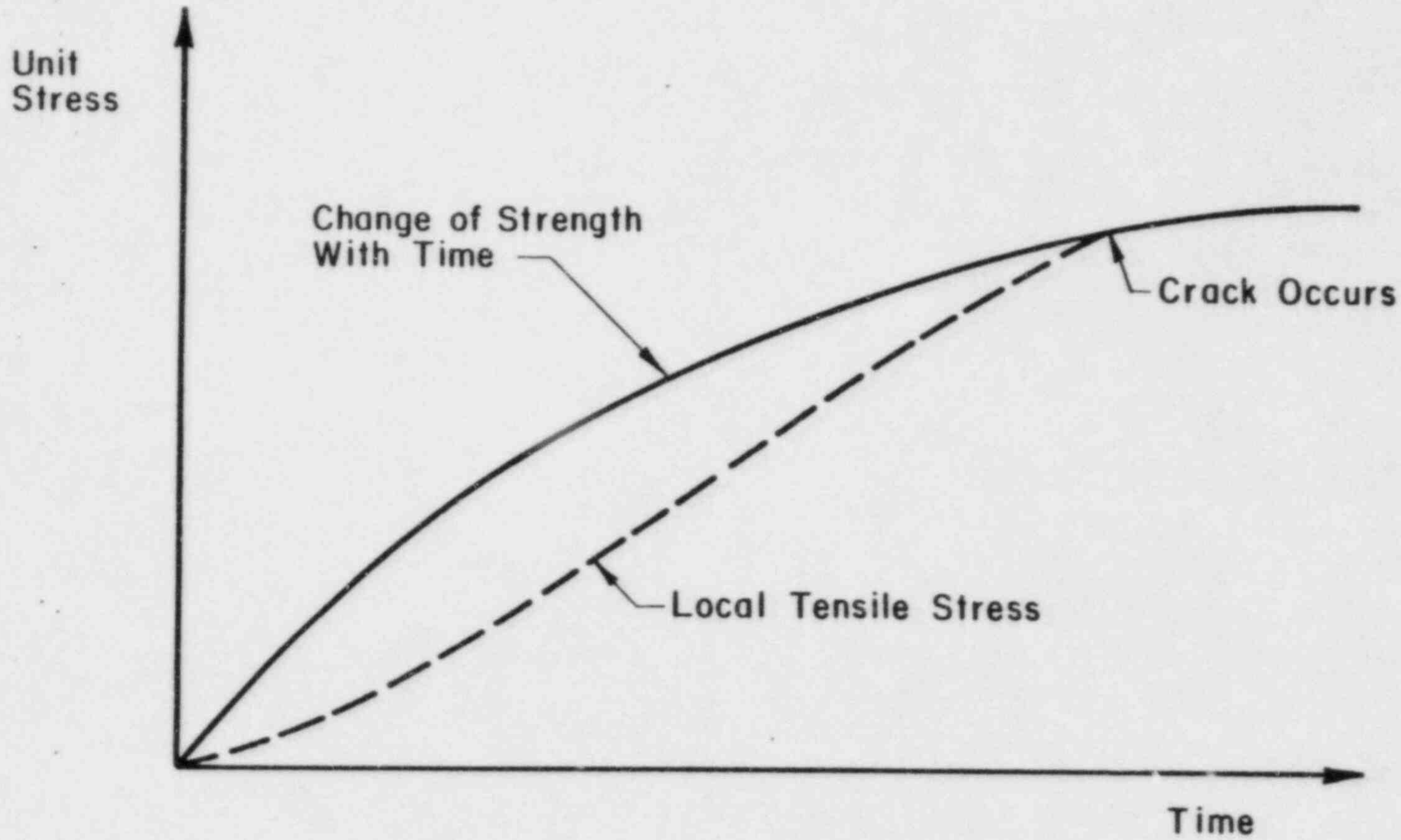

Fig.1.2 Variation of Concrete Strength and Internal Tensile Stress with Time

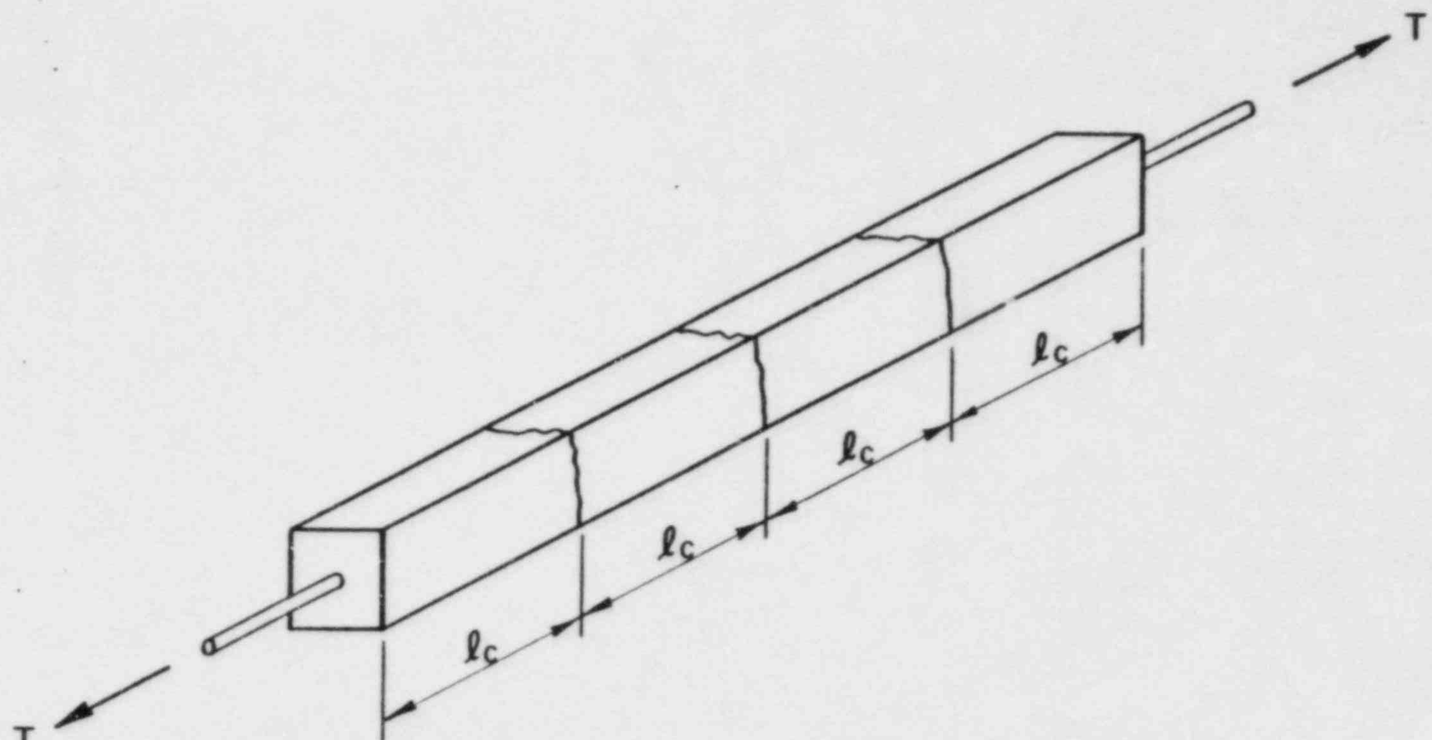

Fig.1.3 Reinforced Concrete Prism

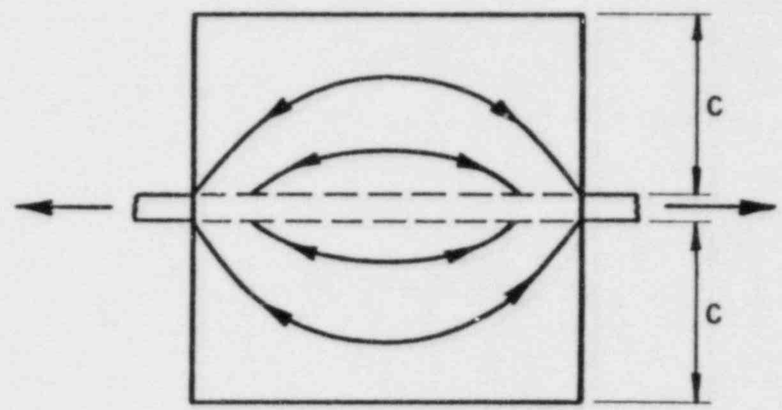

Fig.1.4 Stress Trajectories in Concrete

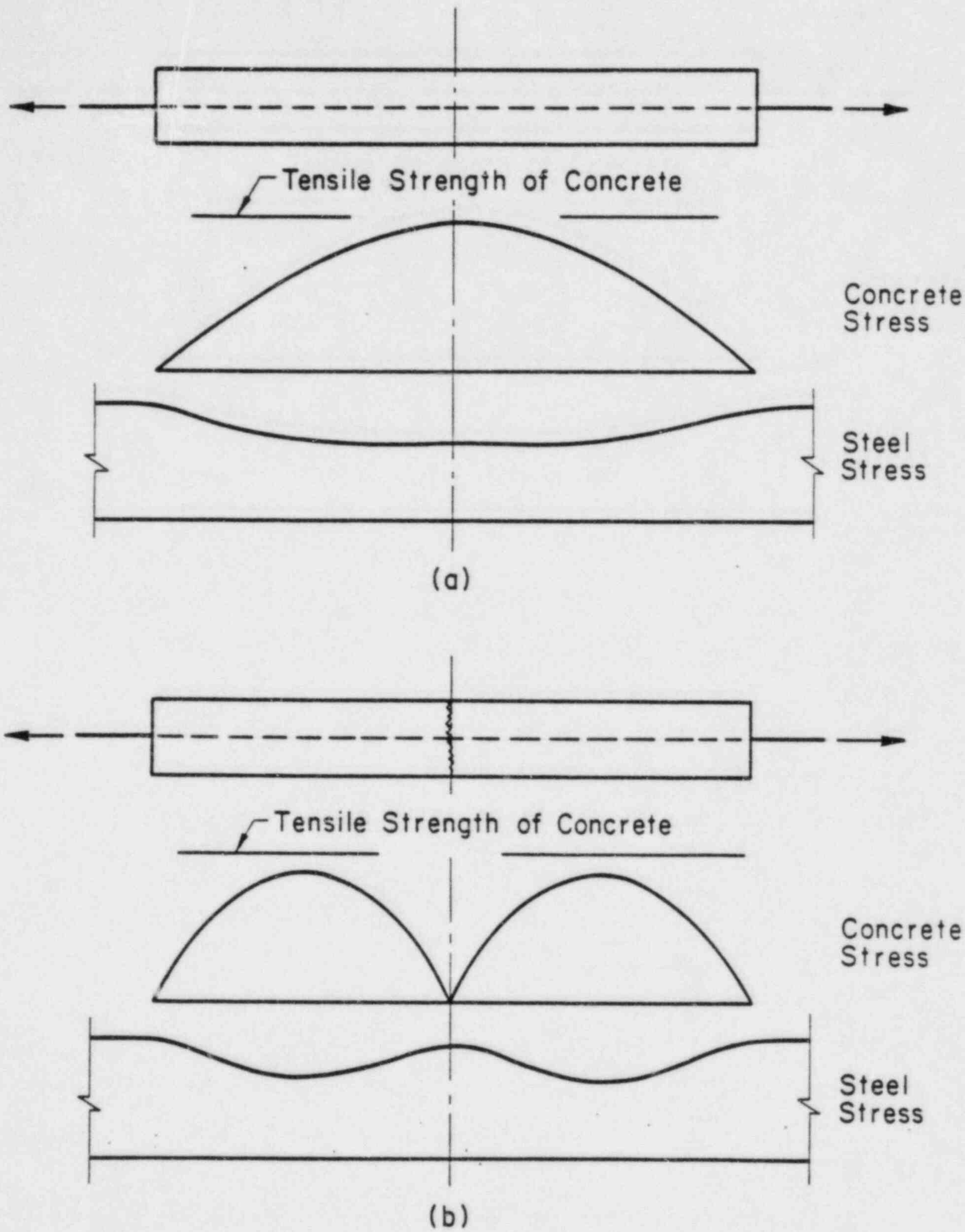

Fig.1.5 Tensile Stress Distributions

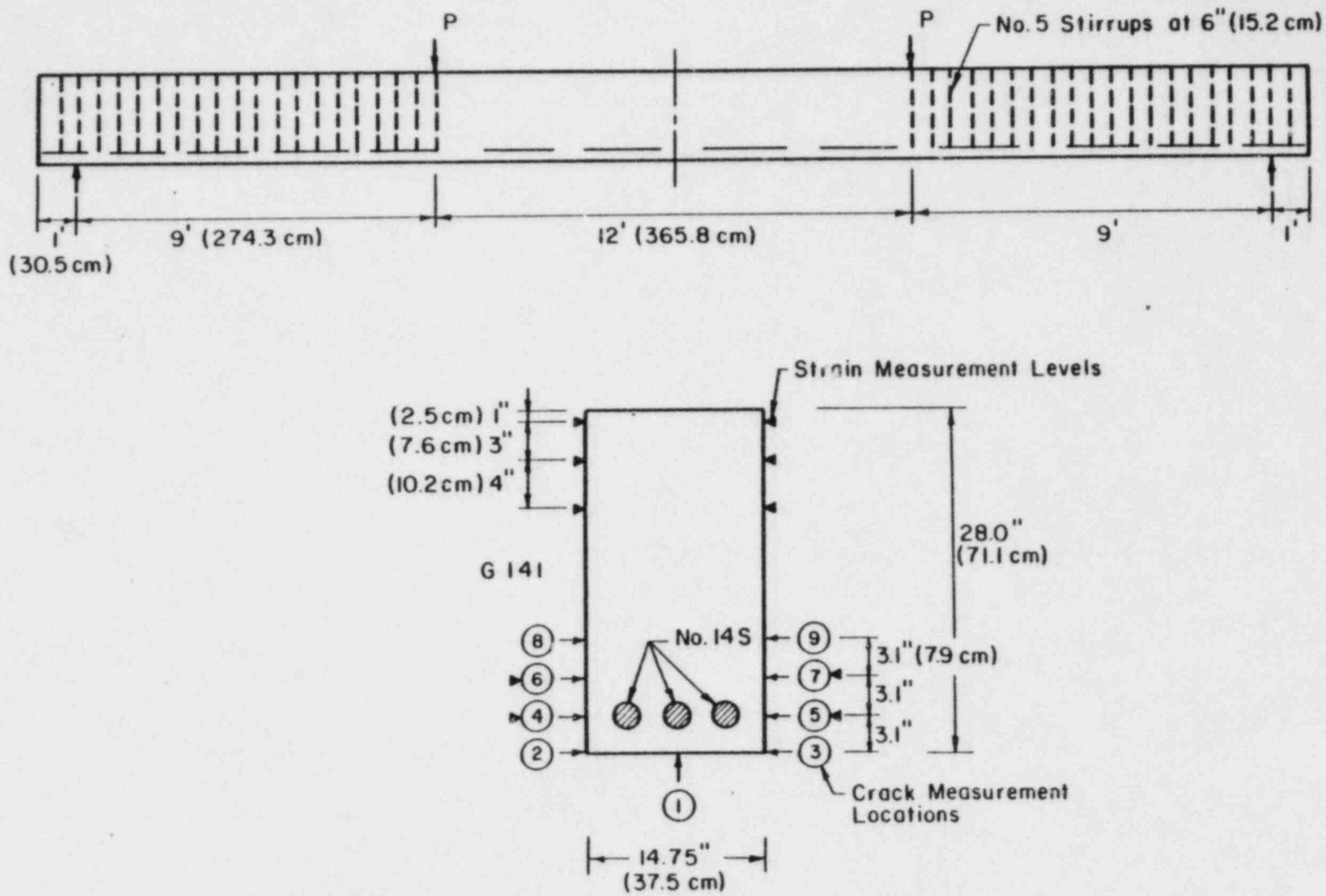

Fig.1.6 Test Girder G141

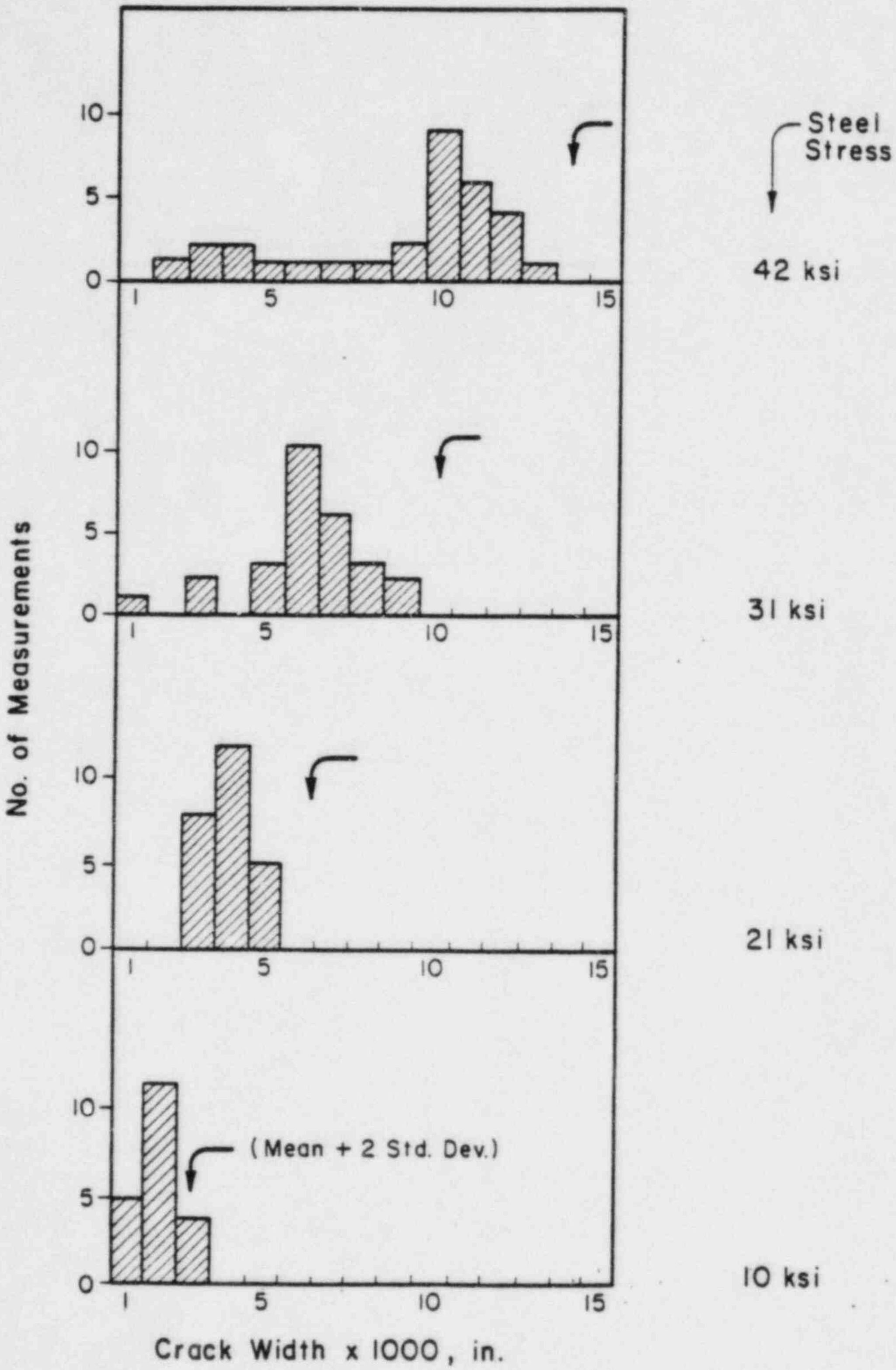

Fig.1.7 Measured Crack-Width Distribution in Girder G141
at Reinforcement Level

ATTACHMENT 2

EFFECT OF EXISTING CRACKS ON STRENGTH OF REINFORCED CONCRETE MEMBERS

Summary

Do existing cracks affect the strength of a reinforced concrete structure? Attachment 2 was prepared to provide information in answer to this question.

Referring specifically to the size and type of cracks in the Diesel Generator Building, the concern is whether such cracks would reduce the strength of the structure below the level of nominal strength assumed in design methods.

Examples of basic internal-resistance mechanisms are considered individually. Cracking in surrounding concrete certainly does not affect the strength of the reinforcement in tension. Test results from Richart and Brown (2.1) and Vecchio (2.4) are invoked to demonstrate that strengths in compression and shear are also insensitive to existing cracks.

Bending resistance may be considered as being made up of flanges working in essentially axial compression and tension. Evidence from the Richart and Brown (2.1) tests would suffice to conclude that flexural strength would be insensitive to existing cracks. Behavior of a beam subjected to cyclic loading (2.2) is shown to be consistent with this conclusion.

Cyclic loading data from a test of a box-like specimen with walls similar to the Diesel Generator Building are also discussed with the same

conclusion: existing cracks of the type observed in the Diesel Generator Building would not reduce the strength of the building below that assumed in its original design.

It is concluded that overwhelming evidence exists from laboratory experiments and experience with actual buildings to demonstrate that "precracks" of the type considered do not affect significantly the strength of a concrete structure which has been properly reinforced for the design load combinations.

Introduction

Reinforced concrete structures are often cracked before application of a load for which the structure has been proportioned. This note has been prepared to discuss the influence of such "precracks" on structural strength and behavior. Widths of cracks envisioned are assumed to be typically less than one quarter of an inch and never of a size that can lead to instability of a compressed reinforcing bar crossing the crack.

Initially strength of precracked reinforced concrete members subjected to four simple loading conditions are considered: (1) axial tension, (2) axial compression, (3) bending, and (4) shear. Discussions of behavior under these four "pure" loading conditions are followed by a description of the behavior of a box-like reinforced concrete specimen subjected to cyclic lateral loading.

Axial Tension

The condition of axial tension is considered not because it requires discussion but because it represents a fundamental case of loading and because it helps illustrate directly the basic premise of design in reinforced concrete.

A hypothetical case of a single reinforcing bar embedded along the longitudinal axis of a prism of concrete is considered in Fig. 2.1. Application of an axial tension on the bar will eventually cause cracking of the concrete at a number of sections as shown.

The basic premise of design in reinforced concrete is that all normal tensile forces are resisted entirely by reinforcement. If the element in Fig. 2.1 had been designed to carry a certain axial tensile force, all the force would have been assigned to the reinforcement. Consequently, whether these cracks form as the tensile force is applied or whether they had occurred earlier as a result of volume-change or stress effects is of no consequence to the proper functioning of this structural element. Cracking of the concrete would affect only the initial slope of the force-extension relationship.

Axial Compression

It is of interest to consider the strength of the same prism (with existing cracks) subjected to axial compression as shown ideally in Fig. 2.2. The prism is assumed to be loaded axially through stiff bearing plates so that the overall deformations in the concrete and the steel are the same.

Given that the existing cracks are not so wide as to lead to local instability of the bars or overall instability of the entire element, it can be inferred from a knowledge of the stress-strain properties of the materials involved that the reinforcement at the cracks will eventually be strained sufficiently to close the cracks. After that event, large compressive stresses will be developed in the concrete leading typically to failure initiated by spalling of the concrete. Whether this "reseating" process affects the strength of the concrete or of the reinforced concrete section can best be determined by experiment.

Several series of tests of reinforced concrete columns were reported by Richart and Brown (2.1) in the course of an experimental study which was to lead to the fundamental principles of reinforced concrete column design used today. One of these series, Series 3, was dedicated to the investigation of the effect of sustained loading on column strength. A group of tied and spirally reinforced columns, 5 ft long by 8-in. round (Fig. 2.3), were subjected to a sustained service load for approximately one year. A parallel group of columns were stored for the same period without any load. Changes in steel stress, calculated from measured strains, observed for the loaded and unloaded columns are illustrated in Fig. 2.4. The accumulated strain at the end of the observation period was approximately 0.008 in the loaded columns.

> "because of the arrangement of the time-loading rigs, it was necessary to release the loads and to remove the columns from the rigs before placing them in the testing machine. This release of load permitted a recovery of the large elastic strains in the steel and resulted in the formation of tension cracks in the concrete, generally 10 to 12 in. apart. The columns were tested at once, and strain measurements showed that when the applied load had reached the value of the one-year sustained load, the cracks had closed, and the steel and concrete strains corresponded closely with those measured under the spring [previous sustained] loading."

Richart and Brown did not report crack widths. The widths may be inferred to be approximately 0.01 in. from the strains indicated in Fig. 2.4 and the reported crack spacing. No cracks were observed in the columns without load.

Measured strengths of the columns with and without sustained loading are compared in Table 2.1 reproduced directly from Reference 2.1. The last column in the table indicates the ratios of the observed strengths of columns with sustained load (which had cracks) P_T to the observed strengths of comparable columns which had not been previously loaded (and which did not have cracks) P_N. The ratio is observed to vary from 0.86 to 1.15 with an overall mean value of 1.0 with a coefficient of variation of 6.2 percent. Richart and Brown concluded that, against the background of expected scatter in such test data, there was no significant difference between the strengths of the two groups of columns.

Bending

A simple and practical model to understand the flexural strength mechanism of a reinforced concrete section is provided by analogy to a structural steel wide-flange section with a thin web. Resisting moment is generated by a couple formed by tensile and compressive forces in the "flanges" of the section as shown schematically in Fig. 2.5. The tensile force is provided by the steel and the compressive force by a concrete-steel composite, quite similar to the idealized element in Fig. 2.2. From this interpretation of the flexural-strength mechanism and the information supplied above, it follows that existence of cracks perpendicular to the bars, whatever the cause, would not reduce the flexural strength of the section.

The same conclusion may be reached by recognizing that the flexural strength of reinforced concrete sections reinforced in tension only with typical amounts of reinforcement is insensitive to changes in concrete strength. Influence of the concrete strength on flexural capacity is even less if the section has compression reinforcement. Thus, any reduction in compressive strength because of local spalling during the reseating of the crack is likely to have negligible effect on flexural strength.

A common experimental demonstration of the trends discussed above is provided by response of reinforced concrete beams to load reversals.

Consider the measured relationship between force and mid-span deflection of a test beam reported in Reference 2.2. The first loading to over 10 kips would cause a pattern of cracks as shown ideally in Fig. 2.7d.

Return to zero load would leave a "residual" crack pattern as shown in Fig. 2.7e. Clearly, the concrete to work in compression when the load is increased in the opposite direction is cracked at zero load. But it is seen in Fig. 2.6 that the cracks do not prevent the beam from developing its strength in the opposite direction.

Shear

Vecchio (2.4) reported a series of 30 tests to investigate the force-deformation properties of reinforced concrete laminae subjected to in-plane forces. The results of this investigation permit a comparison of the strength of reinforced concrete laminae which have been cracked before shear loading with the strengths of laminae which had no visible cracks before loading. (The term "lamina" is used here for a slab to avoid association with "slab shear strength" which refers typically to out-of-plane forces.)

To approximate the conditions of a "pure" shear loading, Vecchio used the mechanism shown in Fig. 2.8 to apply reasonably uniform shear forces along the edges of a reinforced concrete lamina (Fig. 2.9) measuring 35 * 35 * 2-3/4 in. Reinforcement was provided by two layers of annealed welded wire fabric mats.

Twelve specimens, with properties listed in Table 2.2, failed in shear under "shear loading" before reinforcement in both directions had yielded. Of this group, only ten with concrete strength in the range 2300 to 3100 psi are considered here in order to be able to discuss the results directly, without normalizing the data to account for changes in concrete strength.

Measured unit shear strengths of the specimens loaded monotonically to failure are plotted using open circles against the product $\rho_t f_y$ in Fig. 2.10. (The term ρ_t refers to the lower of the reinforcement ratios in the two orthogonal directions.)

One specimen, PV 26, was cracked in biaxial tension before loading in shear. The cracks were obtained by applying forces equal to 60 percent of the calculated yield stress of the reinforcement simultaneously in each direction (of the reinforcement parallel to the edges of the specimen). Shear forces were applied after release of the tensile forces. As represented in Fig. 2.10 by a solid circle, this specimen developed a strength comparable to that of the monotonically loaded specimens.

Another specimen, PV 30, was also initially cracked in biaxial tension in the same manner as PV 26 was cracked. However, PV 30 was increased in 100-psi increments starting from 125 psi. At each stress level, the stress was cycled ten times. The maximum shear stress developed by PV 30 is also shown by a solid circle in Fig. 2.10. It is evident that the strength of PV 30 was not perceptibly affected by existence of initial cracks and by the stress reversals.

The observed results can be anticipated by interpreting the response of the lamina in terms of the simple "truss mechanism" illustrated in Fig. 2.11. The diagonal truss elements operate in a manner similar to the tension and compression elements shown in Fig. 2.1 and 2.2. The stiffness of the lamina would be expected to decrease because of cracks existing before load application, and it does. But given that the "precracks" do not affect strength in cases illustrated in Fig. 2.1 and 2.2, it follows that precracks would not change strength significantly in the case of a lamina subjected to shear forces.

Behavior Under Cyclic Loading of a Reinforced Concrete "Box"

The observed behavior of a stubby box-like reinforced concrete structure subjected to lateral-load reversals at the structural engineering laboratory of the University of Tokyo (2.5) is of interest for two reasons: (a) the specimen is a low-rise (stubby) reinforced concrete box with uniformly reinforced walls similar to the Diesel Generator Building and (b) the loading conditions in its walls combine the types of loadings considered individually in the preceding sections.

Plan and elevation of the specimen considered (B6) is shown in Fig. 2.12 which also describes the test rig. Plan dimensions, out-to-out of walls, of the specimen were 0.83 * 0.83 m (approx. 2.7 ft). Wall thickness was 0.08 m (approx. 3 in.). Lateral loads were applied at a level 0.8 m (approx. 2.6 ft) above the top of the base slab.

Concrete strength was reported to be 256 kg/cm^2 (3600 psi) at time of test. As shown in Fig. 2.13 walls were reinforced with 6-mm bars (corresponding approx. to No. 2 bars). Vertical and horizontal bars were spaced at 13.2 cm, except near the corners, resulting in a reinforcement ratio of 0.5 percent in the wall sections away from the corners. Yield stress of the reinforcement was 3910 kg/cm^2 (56 ksi).

Umemura, et al., calculated the maximum value of the applied lateral load to be 34.3 tons (75.7 kips) corresponding to the development of the calculated flexural capacity. The curve identified by the legend "e-function method" shows the calculated response of the specimen for a monotonically increasing lateral load.

The lateral load was applied alternately in opposite directions using the arrangement of hydraulic jacks shown in Fig. 2.12. The loading history is documented in Fig. 2.14.

Specimen B6 was loaded initially to 30 tons (66 kips). The load was then reduced to zero. At that time the walls parallel to the axes of the jacks would have been cracked as shown ideally by the sketch in the figure. The specimen may then be considered as one having "precracks" because the existing cracks were caused by a loading direction radically different from the one it is to sustain. As the load is applied in the reverse direction (negative values of load in Fig. 2.14), compressive stresses act on the crack planes while tensile stresses develop parallel to the crack planes. But the strength of the specimen is not reduced. This observation can be rationalized on the basis of the loading conditions described earlier. Flexural strength is developed primarily by forces on the "flange" walls which are subjected essentially to alternating axial and tensile forces. It was discussed that there should be no critical decay in axial compressive strength of the flange walls under the loading conditions considered. It can also be inferred from Vecchio's test results (2.4) that the "web" walls carrying the shear would not be affected critically by the existence of "precracks" at the beginning of loading in each direction in each cycle. Final states of cracks in the web and flange walls are illustrated in Fig. 2.15.

Concluding Discussion

Internal resistance mechanisms in reinforced concrete members may be described by combinations of three simple conditions: axial compression, axial tension, and shear. In fact, the last condition has to be treated independently only because the principal stress directions corresponding to the shear stress are not usually colinear with the directions of reinforcement.

It has been shown, by example where necessary, that existing cracks do not affect significantly the strength in tension, compression, and shear of properly reinforced concrete elements.

Overwhelming evidence from the field and from the laboratory indicates that reinforced concrete structures will develop their design strength even if they do have "precracks", provided the structure has been proportioned and detailed to resist the design load combinations. The examples discussed rationalize the experience.

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2.2 J. A. Blume, N. M. Newmark, and L. H. Corning, "Design of Multi Story Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, Skokie, 1961, p. 130.

2.3 M. A. Sozen, "Hysteresis in Structural Elements," Applied Mechanics in Earthquake Engineering, ASME Proc. of Winter Annual Meeting, 1974, pp. 63-98.

2.4 F. Vecchio, "The Response of Reinforced Concrete to In-Plane Shear and Normal Stresses," Ph.D. Thesis submitted to the School of Graduate Studies, University of Toronto, December 1981, 332 pp.

2.5 H. Umemura, H. Aoyama, and H. Noguchi, "Experimental Studies on Concrete Members," Faculty of Eng., Dept. of Architecture, University of Tokyo, December 1977, pp. 32-48.

TABLE 2.1

(Reproduced from Reference 2.1)

Strength of Columns of Series 3 After One Year Under Sustained Loading

Each value represents the test results from two columns. Column section, 50.2 for spiral columns, 53.4 for tied columns.

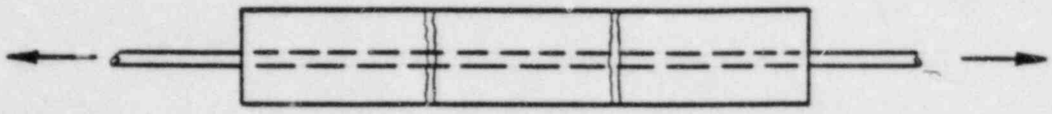
Nominal Design			Columns After One Year Under Sustained Loading		Columns After One Year Under No Loading		Ratio P_N/P_T
f_c' lb. per sq. in.	Percentage of Reinforcement		Ultimate Load, P_T		Ultimate Load, P_N		
	Vertical	Spiral	lb.	lb. per sq. in.	lb.	lb. per sq. in.	
Laboratory Air Storage							
2000	1.5	2	252 000	5015	244 500	4 865	0.97
	4	0	237 000	4435	245 500	4 595	1.04
	4	2	331 500	6595	323 700	6 440	0.98
	6	2	353 250	7030	355 700	7 075	1.01
3500	1.5	0	225 600	4220	....	.....	....
	4	0	302 500	5665	300 800	5 630	0.99
	4	2	403 500	8040	395 000	7 860	0.98
	6	0	304 000	5690	....	.....	....
	6	1.2	368 100	7325	....	.....	....
5000	1.5	2	385 200	7685	394 000	7 840	1.02
	4	0	369 700	6915	415 000	7 765	1.12
	4	2	500 200	9950	485 500	9 660	0.97
	6	2	499 700	9940	505 500	10 060	1.01
						Average	1.01
Moist Storage							
2000	4	0	222 000	4155	190 800	3 570	0.86
	4	1.2	277 200	5515	268 100	5 340	0.97
	4	2	321 200	6390	323 600	6 440	1.01
3500	4	0	261 700	4895	302 000	5 650	1.15
	4	1.2	334 000	6645	327 000	6 505	0.98
	4	2	398 500	7940	396 500	7 890	0.99
5000	4	0	351 500	6575	315 000	5 895	0.90
	4	1.2	431 400	8585	419 000	8 345	0.97
	4	2	483 000	9610	482 200	9 595	1.00
						Average	0.98
						Grand Average Value of Ratio P_N/P_T	1.00

TABLE 2 .2

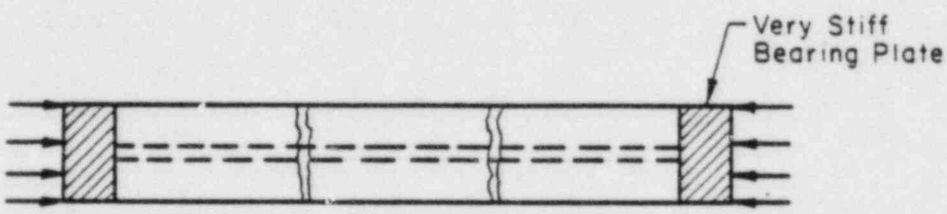
Properties and Test Results of Laminae Subjected to Shear and Failing before Yielding of Longitudinal Reinforcement

		Reinforcement				
		Long.		Transv.		Shear Stress
Mark	f'_c psi	ρ_ℓ %	f_y	ρ_t %	f_y	v_u psi
PV 9	1680	1.79	66.0	1.79	66.0	542
PV 10	2100	1.79	40.0	1.79	40.0	575
PV 12	2320	1.79	68.0	.45	39.0	454
PV 13	2640	1.79	36.0	0	0	292
PV 18	2830	1.79	62.5	.32	59.7	440
PV 19	2760	1.79	66.4	.71	43.4	573
PV 20	2840	1.79	66.7	.89	43.1	617
PV 21	2830	1.79	66.4	1.30	43.8	729
PV 22	2840	1.79	66.4	1.52	60.9	880
PV 26	3090	1.79	66.1	1.01	67.1	784
PV 27	2970	1.79	64.1	1.79	64.1	920
PV 30	2770	1.79	63.3	1.01	68.4	744

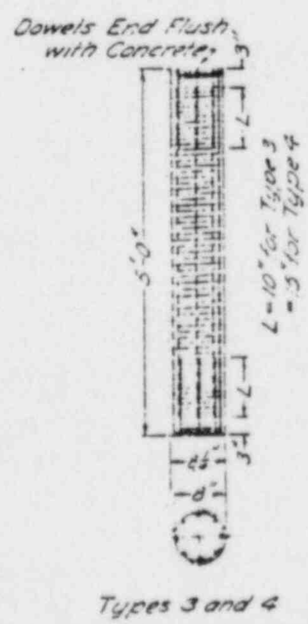
Note: Data from Reference 2.4



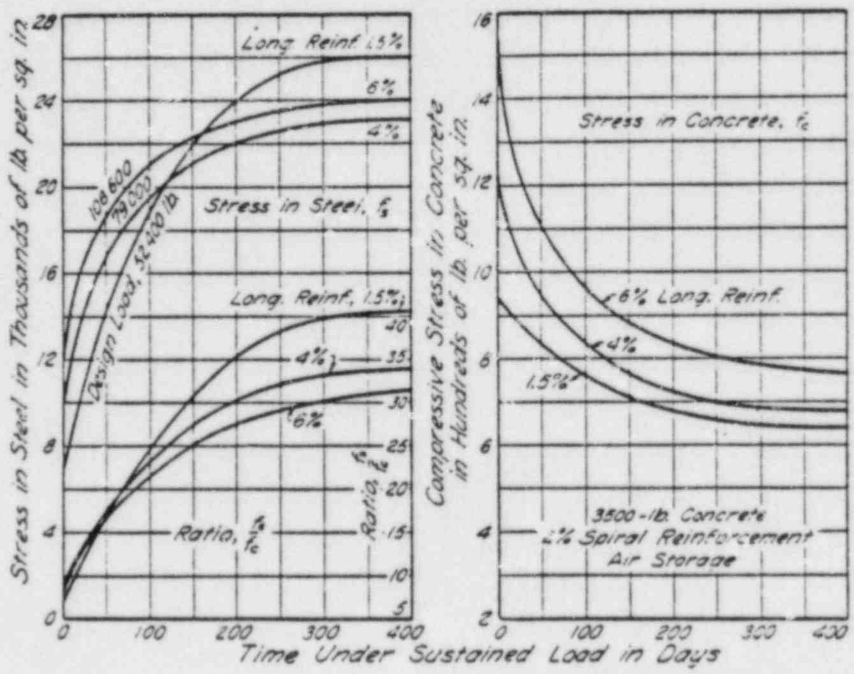
2.1 Reinforced Concrete Element Resisting Axial Tension



2.2 Reinforced Concrete Element Resisting Axial Compression

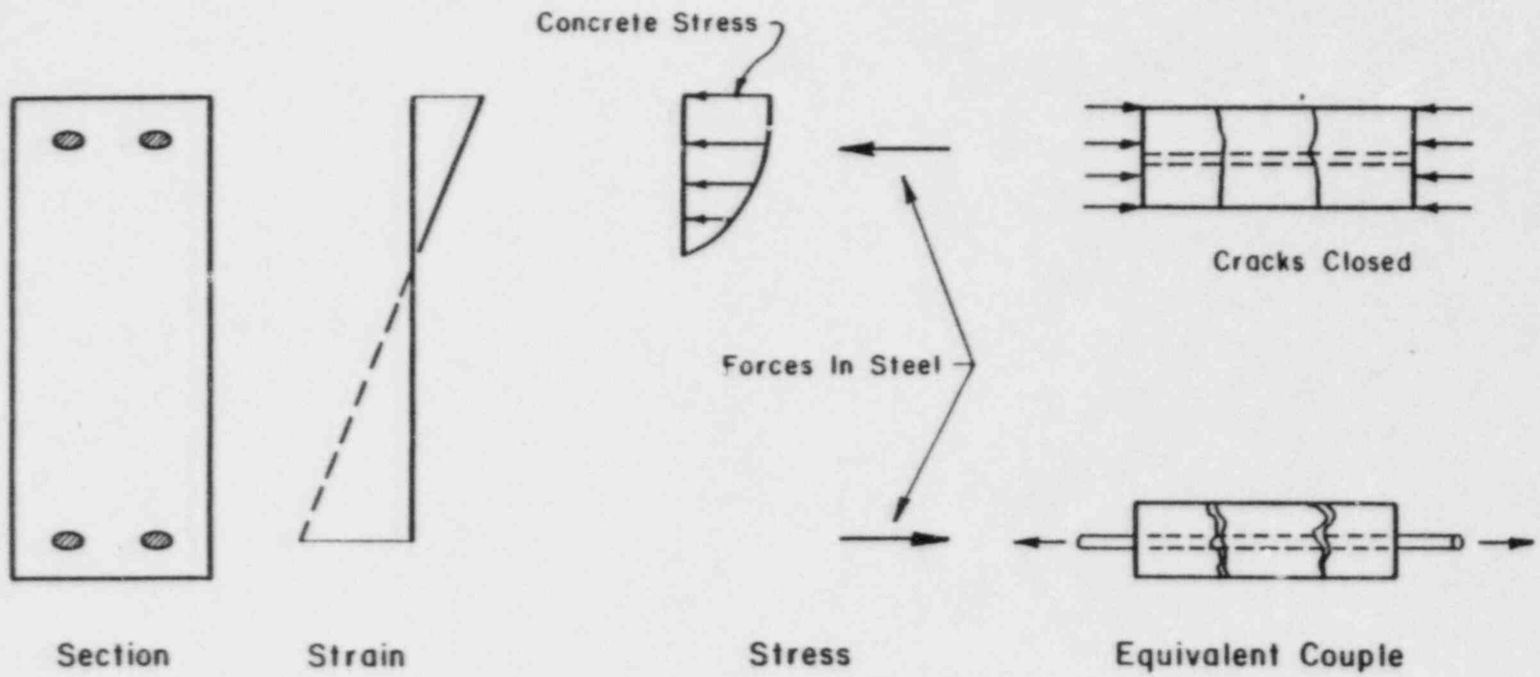


2.3 Reinforced Concrete Column Tested by Richart and Brown (2.1)

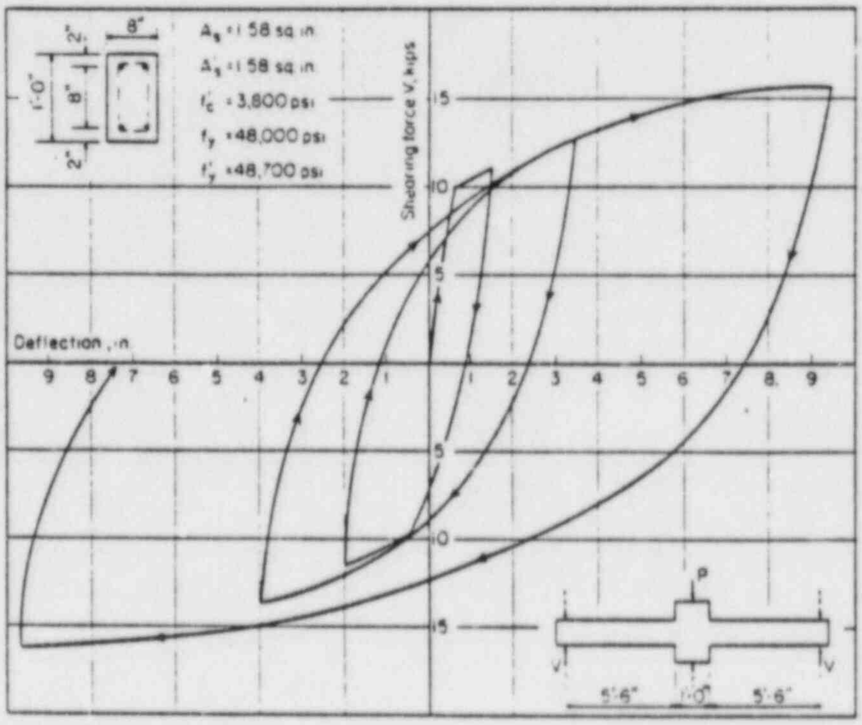


Redistribution of Stresses in Concrete and Steel During Sustained Loading

2.4 Stress Redistribution in Columns (Richart and Brown, 2.1)

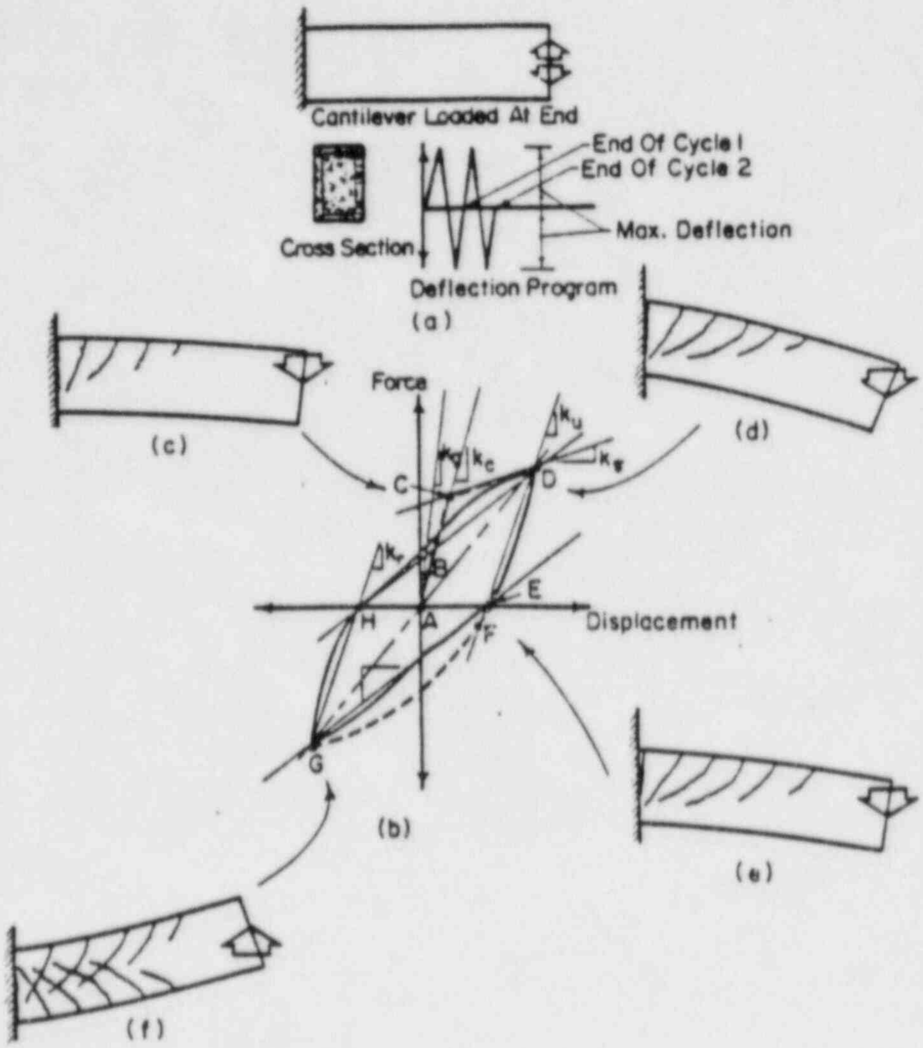


2.5 Idealized Mechanism for Flexural Strength

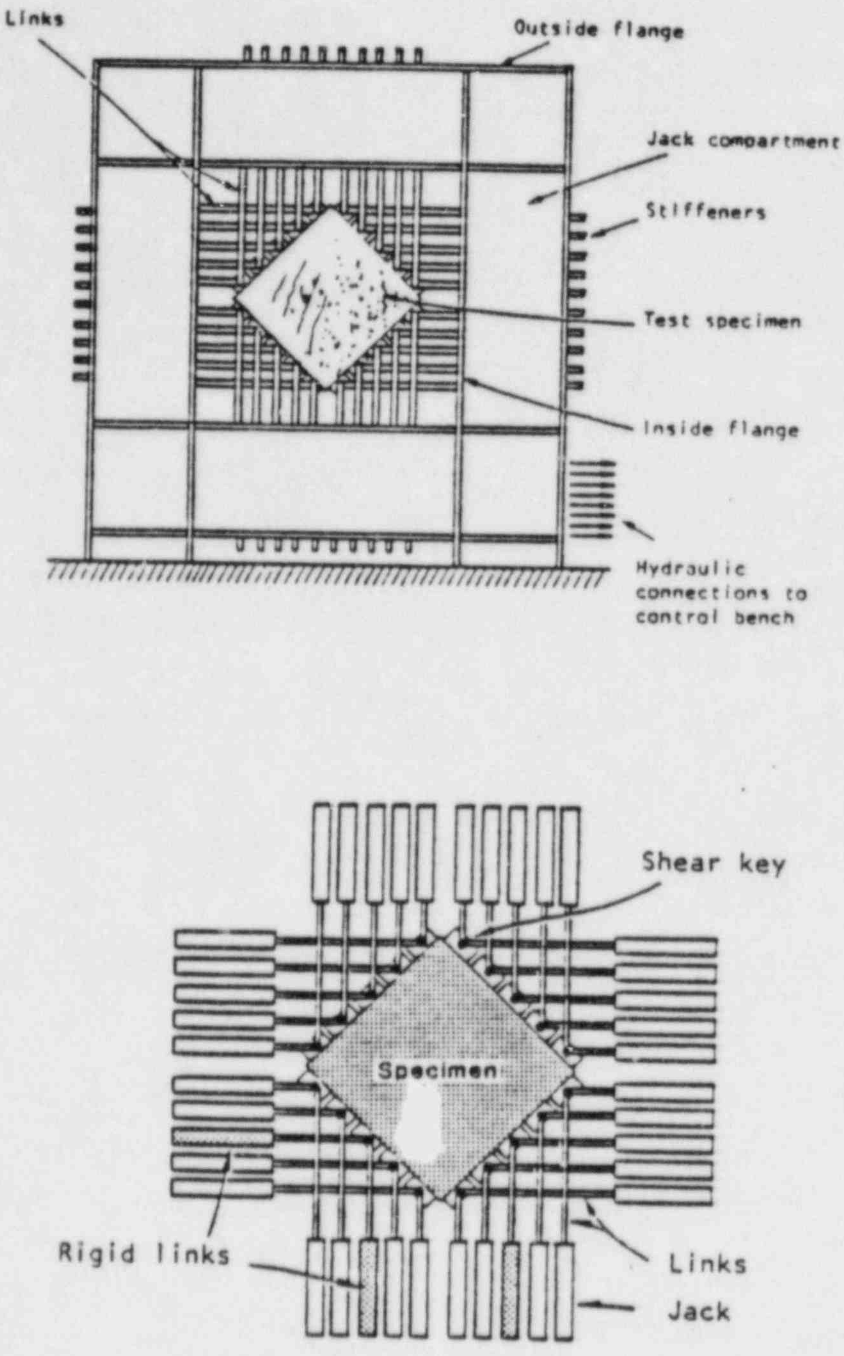


Measured load-deflection relationships for a beam-column connection subjected to reversals of load.

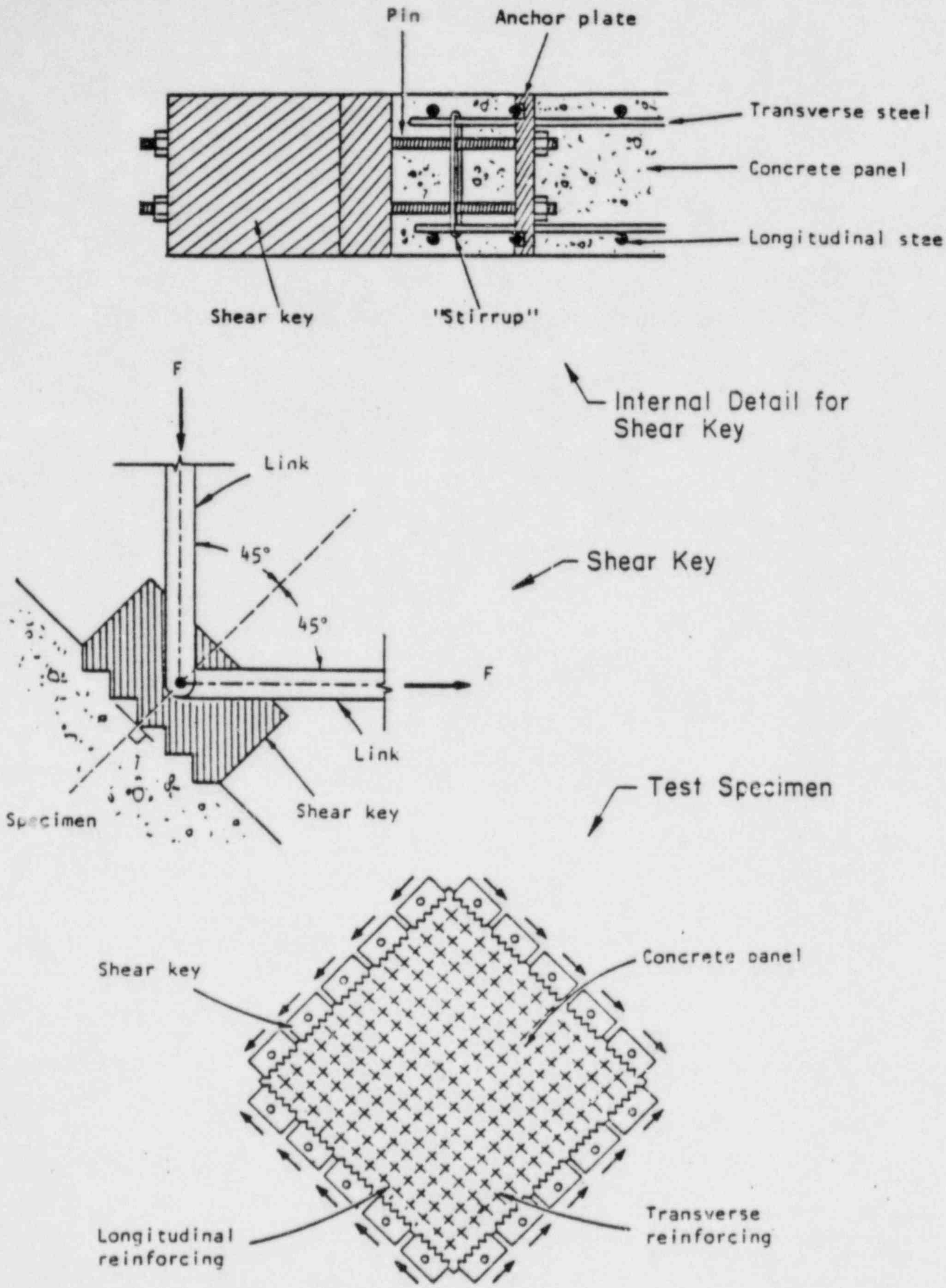
2.6 Load-Displacement Hysteresis (Blume, et al., 2.2)



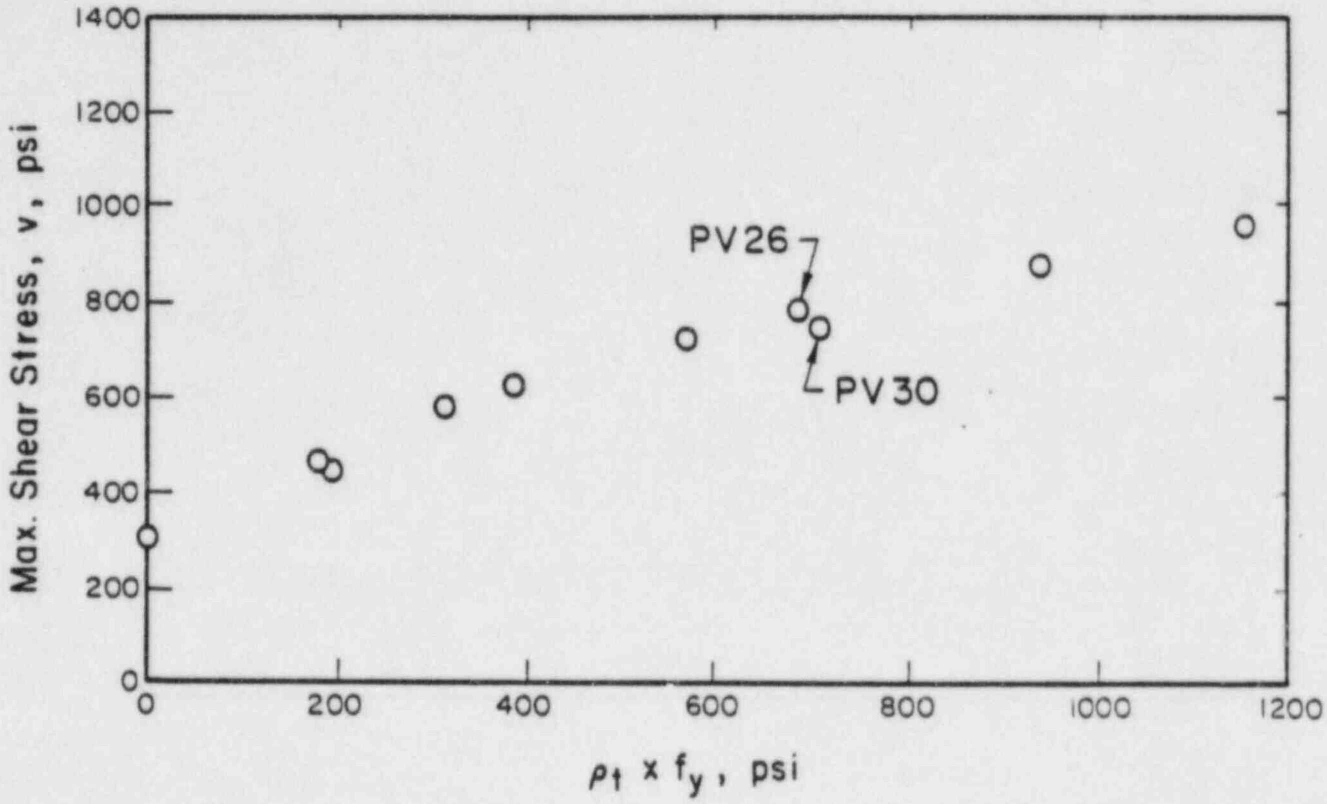
2.7 Cracks in Various Stages of a Load Cycle.



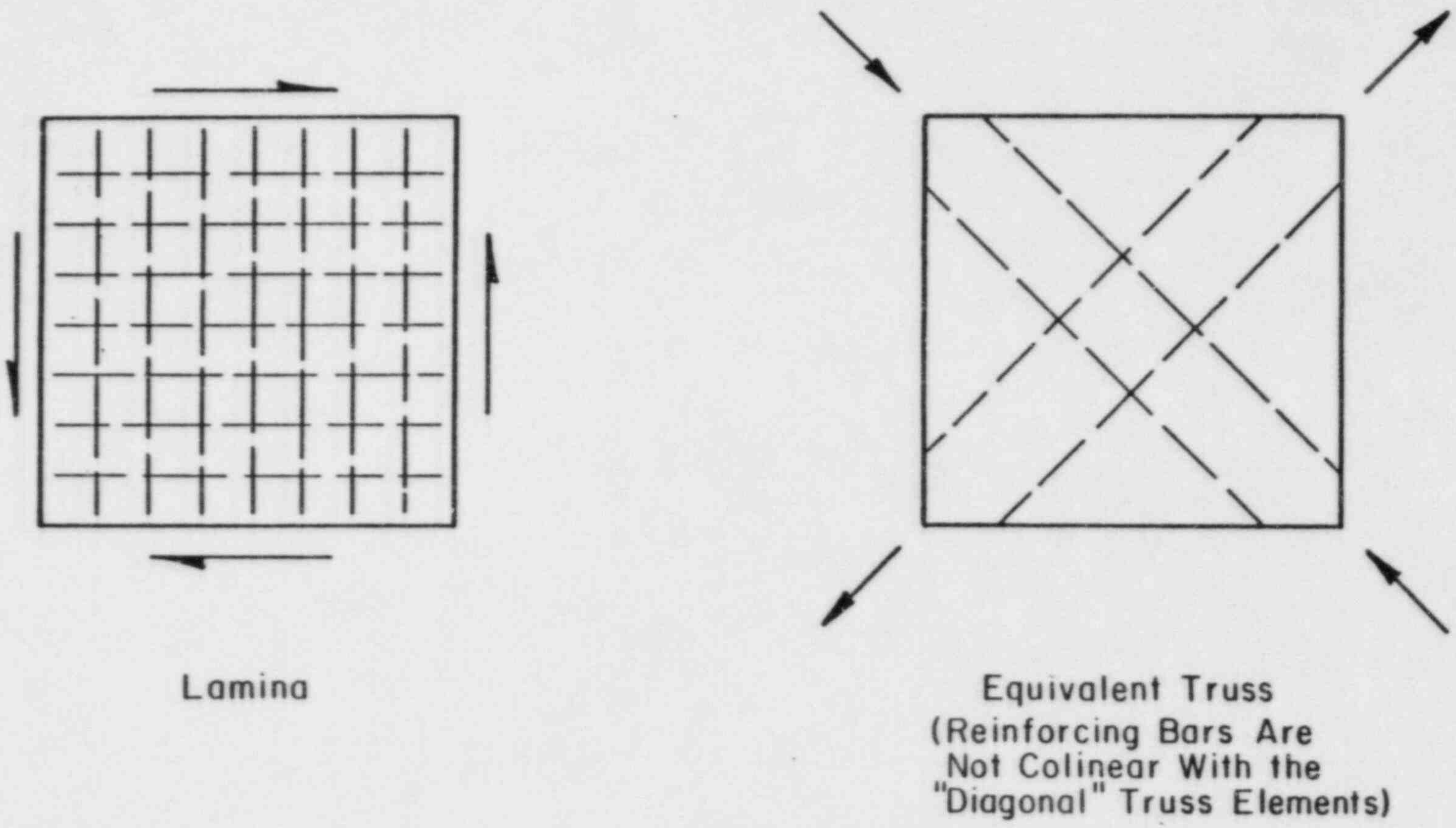
2.8 Loading Scheme Used by Vecchio (2.4)



2.9 Test Specimen and Boundary Loading Details (2.4)

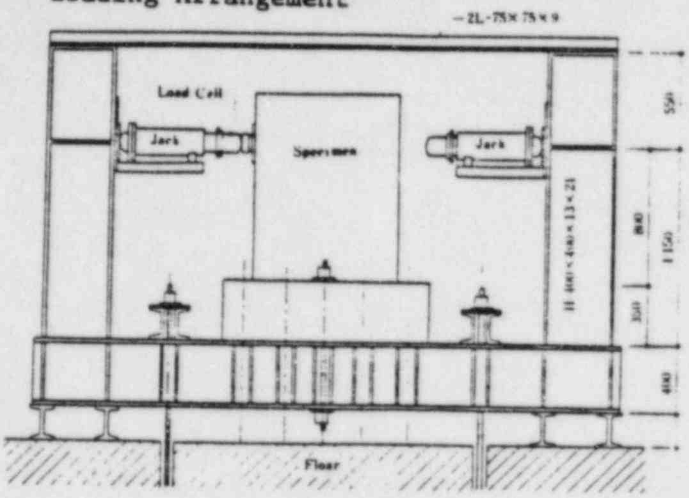


2.10 Variation of Measured Shear Strength (2.4)

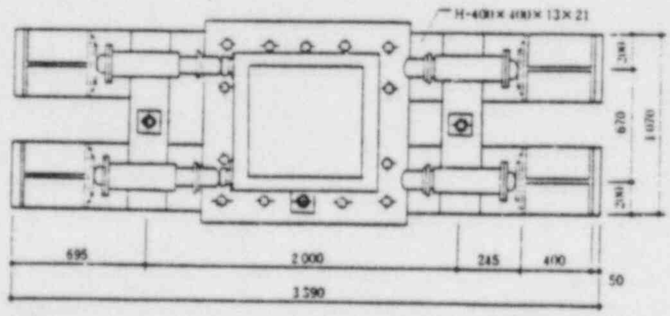


2.11 Idealized Mechanism for Resistance Mechanism of a Lamina Subjected to Shear Stress

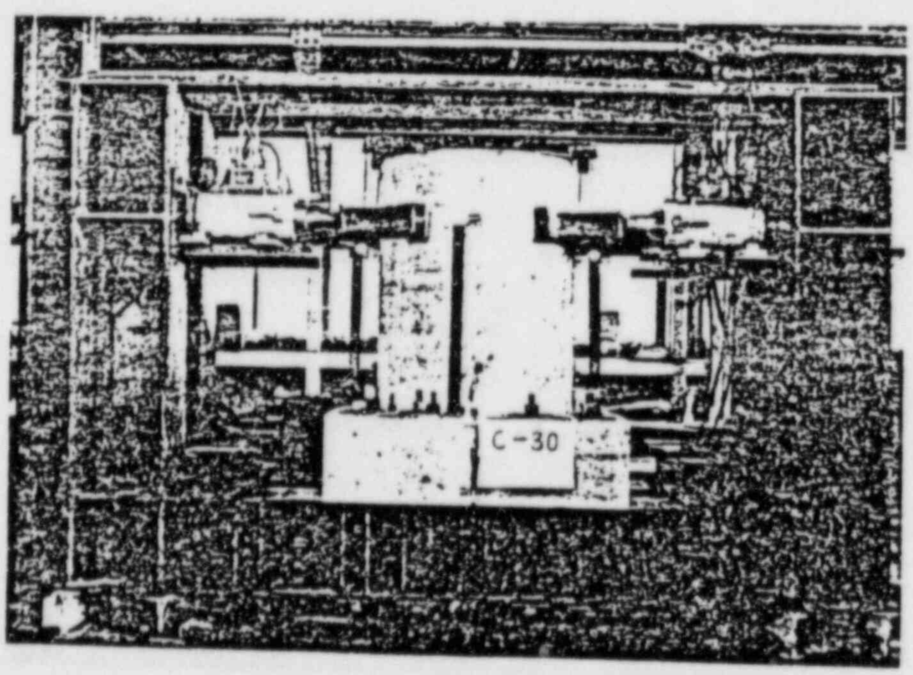
Loading Arrangement



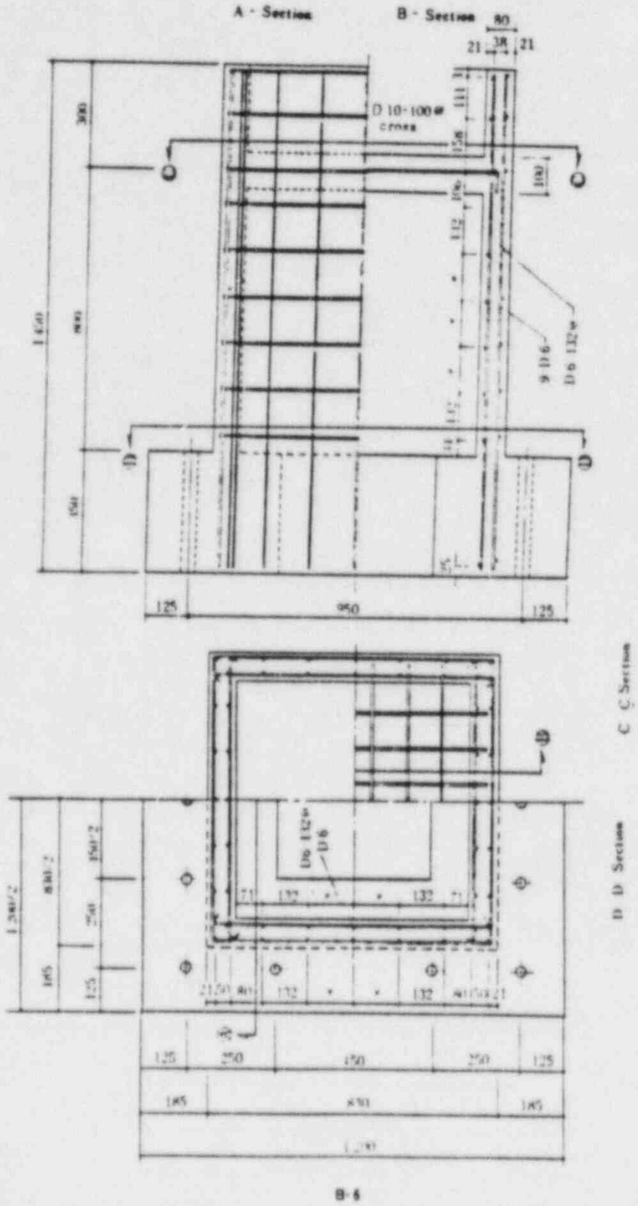
(a) Elevation



(b) Plan



2.12 Loading Rig for Reinforced Concrete "Box" DG Tested at University of Tokyo (2.5)



2.13 Dimensions and Reinforcement for Test Structure B6

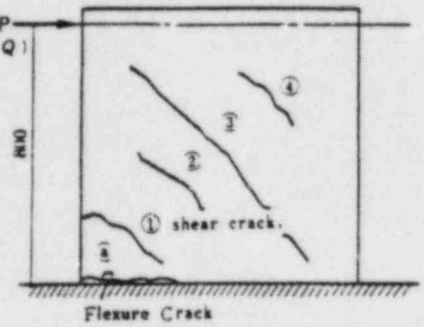
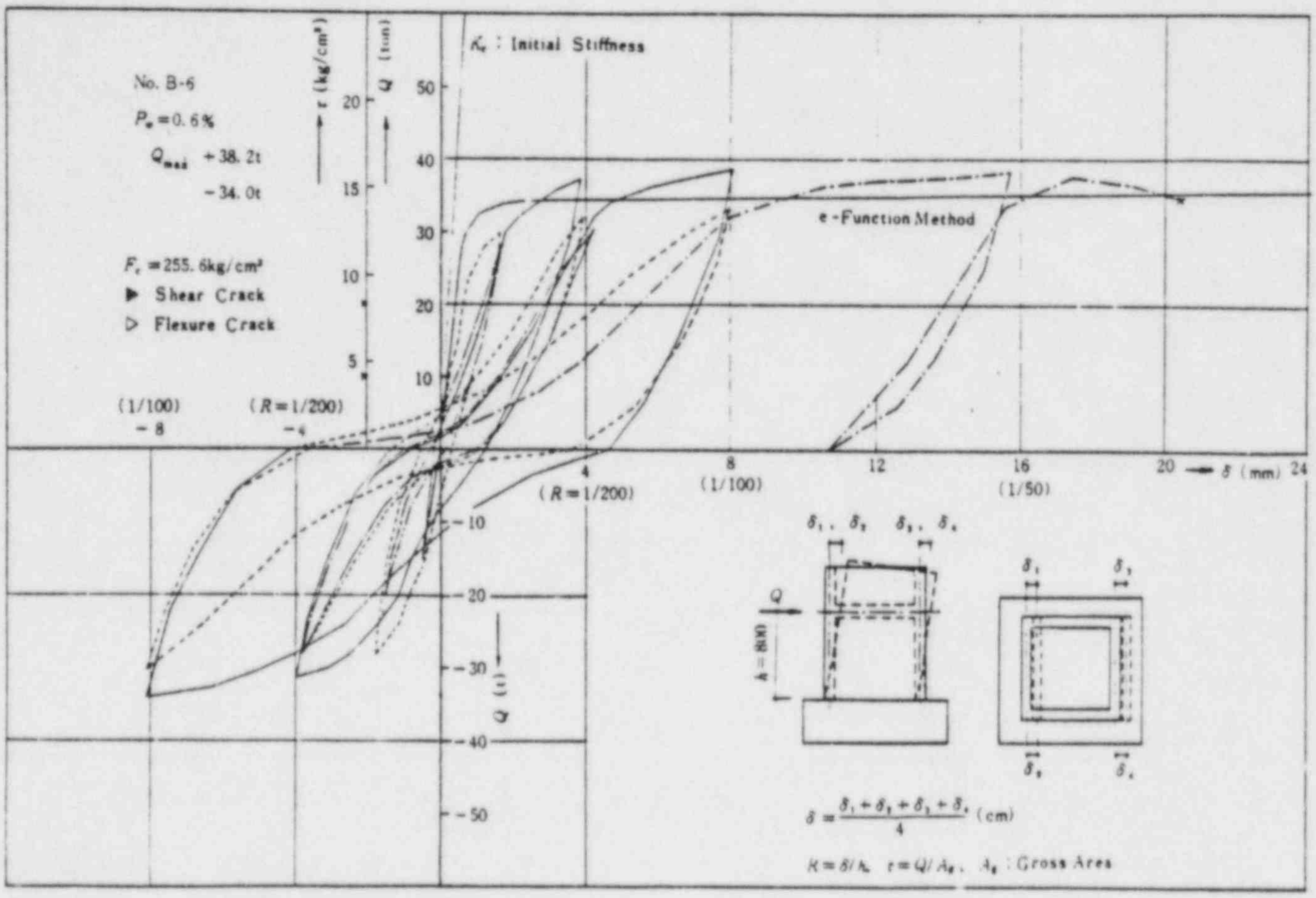
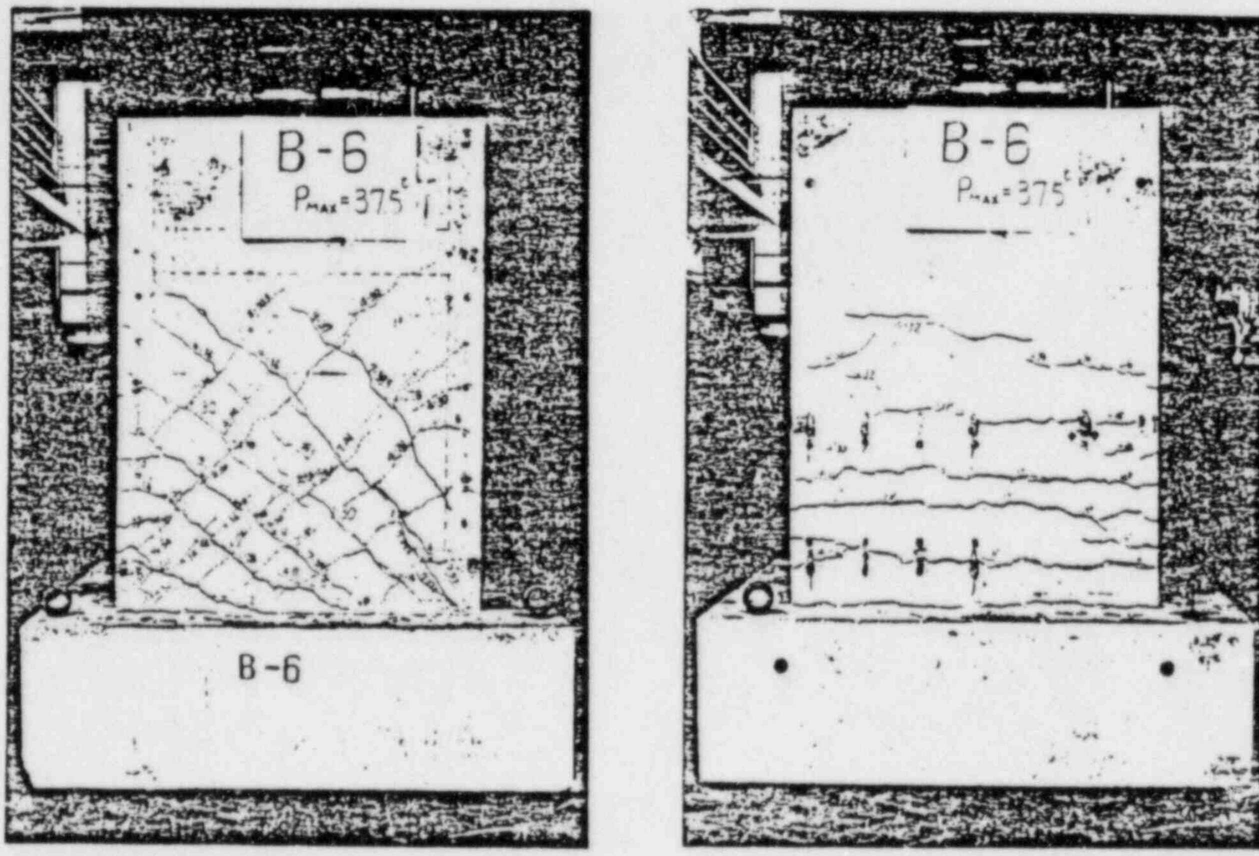


Fig-2.1

Load-Deflection Curves



2.14 Load-Displacement History for Test Structure B6



2.15 Crack Patterns for Test Structure B6

ATTACHMENT 3

CRACKS IN CONCRETE WALLS

Reprint of an article by

Ake Holmberg and Sten Lindgren

Cracks in concrete walls

Åke Holmberg & Sten Lindgren

National Swedish Building Research Summaries

D7:1972

Key words:

crack spacing, crack width, concrete walls, rigidity, imposed deformations

An earlier investigation, "Crack Spacing and Crack Widths Due to Normal Force or Bending Moment" (Document D2:1970), published by the same authors, has led to fundamental conclusions regarding the distribution and the width of stable cracks in concrete structures. The available data on crack spacings was summarized in a crack formula. The present report, however, rejects this formula and presents a new one which takes into account the full range of experimental material available, including material contained in this report. The new crack formula has a wider range of application, covering as it does walls having different types of reinforcement and also to a certain extent slabs with two-way reinforcement.

The present investigation was made on the walls shown in FIG. 1, which were 0.2 m in thickness. The undisturbed area of observation on each face of the wall was 1 × 3 m. Apart from a single exception, this investigation confirmed the earlier observations. The walls were strained in the test set-up shown in FIG. 2. They were restrained so as to remain plane in the cases where the tensile force was eccentric. The mean strain over a length of 3 m was increased in steps, and was maintained constant at the respective values, viz., 0.125 per mil, 0.2 per mil, 0.35 per mil, 0.65 per mil, and 1.25 per mil. The time interval between two consecutive steps was 1 day, but the last interval was almost 2 days.

As the field of application was extended, the previous formula for the calculation of the crack spacing was found to be unreliable. All the data that were now available have been analysed again, and this analysis resulted in a new formula, which is similar in principle to that deduced by *Efsen* and *Krenchel*. This new formula involves a slightly higher coefficient of variation than the previous formula in its range, but covers the totality of the test results under consideration. The new formula was furthermore verified by applying it to extensive results obtained by *Nawy, et al.*, from tests on two-way slabs. This verification indicated that the formula in question might also be applicable to judicious prediction of crack widths in such slabs.

The new formula is

$$s_{r,m} = 0.055 + 0.144 \frac{\phi A_e}{\sum \phi_1}$$

where

- $s_{r,m}$ — the final (smallest) mean crack spacing, in metres, reached at high values of the stress, σ_r, in the reinforcement in a crack
- ϕ — the diameter of a reinforcing bar
- ϕ_1 — the diameter of that reinforcing bar in a group which has the smallest concrete cover, and therefore produces a predominant effect on crack formation
- A_e — that maximum portion of the gross cross-sectional area of the concrete whose centre of gravity coincides with that of the reinforcement

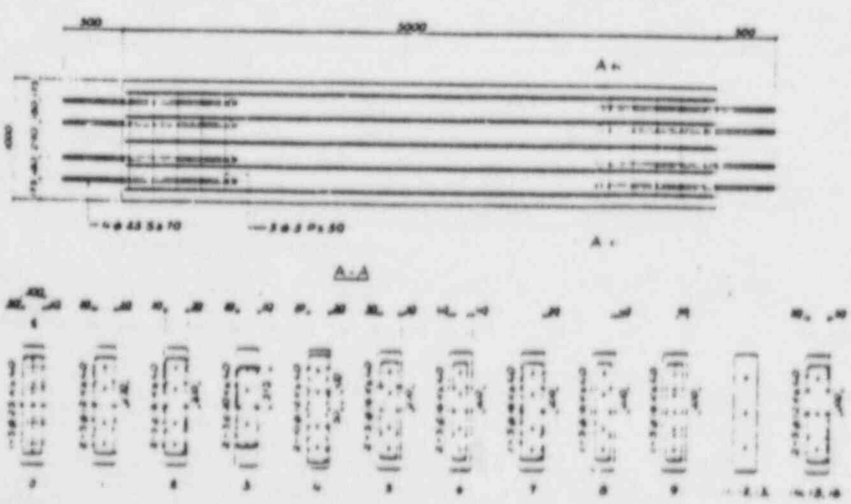


FIG. 1. Walls subjected to the tests, including Walls Nos. 11 to 16, which were used for shrinkage measurements. The gauge points on the concrete on the end surfaces of the latter walls are marked ◡. All dimensions are nominal, dimensions in mm. φ25 Ks40 = Type Kam steel ribbed bar, 25 mm in diameter, yield stress of reinforcement abt 400 MN/m². Ss70 = Type Plain, round steel bar, abt 700 MN/m² yield stress. Ps50 = Type Deformed steel bar, abt 500 MN/m² yield stress.

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Summary of:

Holmberg, Å & Lindgren, S. 1972. *Cracks in concrete walls.* (Statens institut för byggnadsforskning) Stockholm. Document D7:1972. 70 p., ill. 18 Sw.Kr.

The document is in English with Swedish and English summaries.

Distribution:

Svensk Byggtjänst
Box 1403, S-111 84 Stockholm
Sweden

in a crack.

The final crack width approaches as a limit the crack spacing multiplied by $\varepsilon_s = \sigma_s/E_s$, where E_s is the modulus of elasticity of the reinforcement. This statement holds good even if the action has not led to the final crack spacing. The variation in the crack width in a vertical direction, at right angles to the reinforcement, is considered to be a short-time effect.

The coefficient of variation in $s_{r,m}$ is 0.2, and hence a reasonable maximum value is 1.4 times the calculated value of $s_{r,m}$.

The maximum crack width in walls, etc., which are sufficiently long to contain the crack having a maximum width is about 1.7 times the calculated value of the crack width, and, for the magnification factor 1.7, the coefficient of variation is about 0.25.

If a wall is reinforced at one face only, and if A_e is smaller than the total cross-sectional area of the wall, then the crack formation on the face that is adjacent to the reinforcement will be in accordance with the above-mentioned formula. On the other hand, if the eccentricity of the reinforcement is great, then the crack development on the opposite face of the wall may be considered to be entirely uncontrolled. With a slight exaggeration, such a wall may be regarded as a reinforced wall that is contiguous to a non-reinforced wall, see FIG. 3. The formula for the calculation of the crack spacing and the above statement involve by implication certain practical recommendations for design.

The rigidity of the wall varies in such a way that it undergoes abrupt changes within extreme limits which are determined by the uncracked concrete and by the bare reinforcement in the crack, respectively. The present investigation affords a basis for estimating the actual limits of the rigidity at a defined stress.

FIG. 4 shows in terms of numerical values the decrease in the rigidity with increase in the mean strain, ε_m, expressed by the factor κ_2 in the relation

$$\sigma_s = E_s \varepsilon_m \kappa_2$$

where

- σ_s the stress in the reinforcement in a crack,
- E_s the modulus of elasticity of the reinforcement,
- ε_m the mean strain of the wall, deep beam, slab, etc.,
- σ_o the stress in the reinforcement at the instant of appearance of the first crack.

As may be seen from FIG. 4, the effect of the ratio of reinforcement, ρ, on the rigidity is slight. On the other hand, the effect of the tensile strength of the concrete, which is in itself difficult to determine and liable to vary, is by no means slight. If a system is highly statically indeterminate, then the design problem is to a certain extent transferred from statics to statistics.

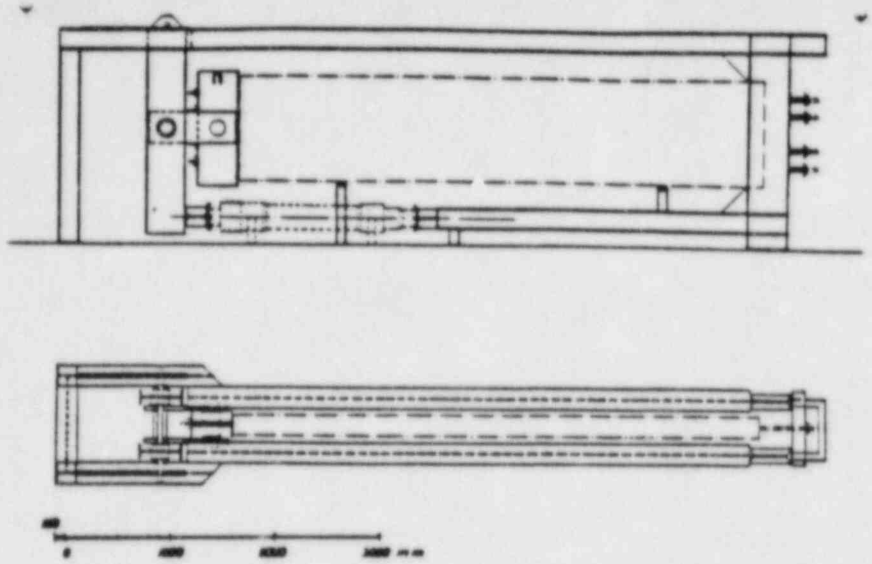

FIG. 2. Machine used for the tension tests on the walls, with controlled elongation.

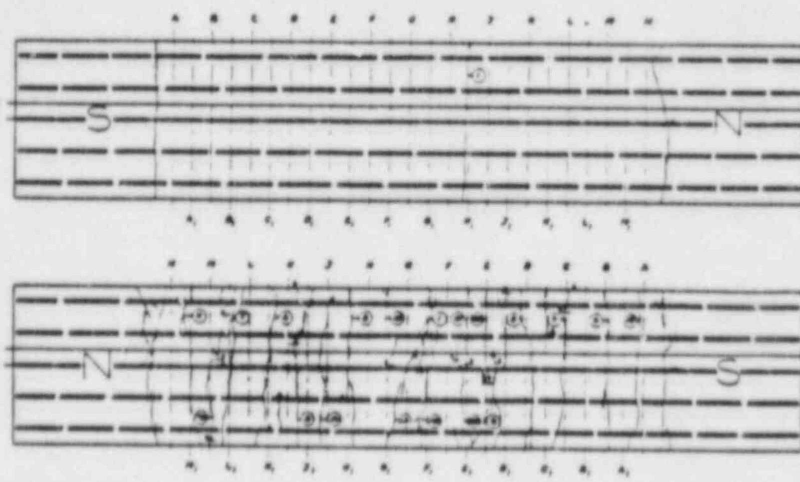

FIG. 3. Cracks in wall after completion of test.

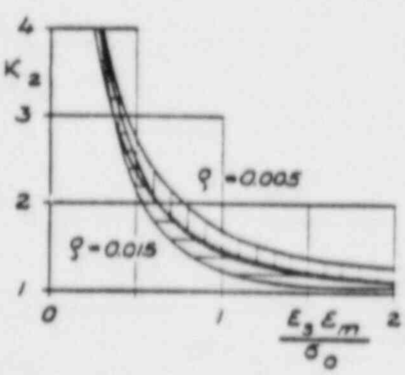

FIG. 4. Relation between κ_2 and $E_s\varepsilon_m/\sigma_o$. Extreme values at $\rho = 0.005$ and 0.015, respectively.

ATTACHMENT 4

EVALUATION OF CRACKING IN DIESEL GENERATOR BUILDING AT MIDLAND PLANT

TABLE OF CONTENTS

ATTACHMENT 4

EVALUATION OF CRACKING IN DIESEL GENERATOR BUILDING AT MIDLAND PLANT

by

W. G. Corley and A. E. Fiorato*

INTRODUCTION

This report presents an evaluation of the significance of cracks observed in the Diesel Generator Building located at Midland Nuclear Power Plant Units 1 and 2. Observed cracks in this structure are described. A program for future monitoring of structural integrity is described.

DESCRIPTION OF STRUCTURE

A site plan for the Midland Nuclear Power Plant is shown in Fig. 4.1. The Diesel Generator Building is located directly south of the Turbine Building. The building is a two-story reinforced concrete structure. It is partitioned into four bays by load-bearing reinforced concrete walls. Elevations, plans, and sections of the Diesel Generator Building are shown in Figs. 4.2 and 4.3.

Diesel generators housed in the building are used to provide power to attain safe shutdown of the plant in case of a design

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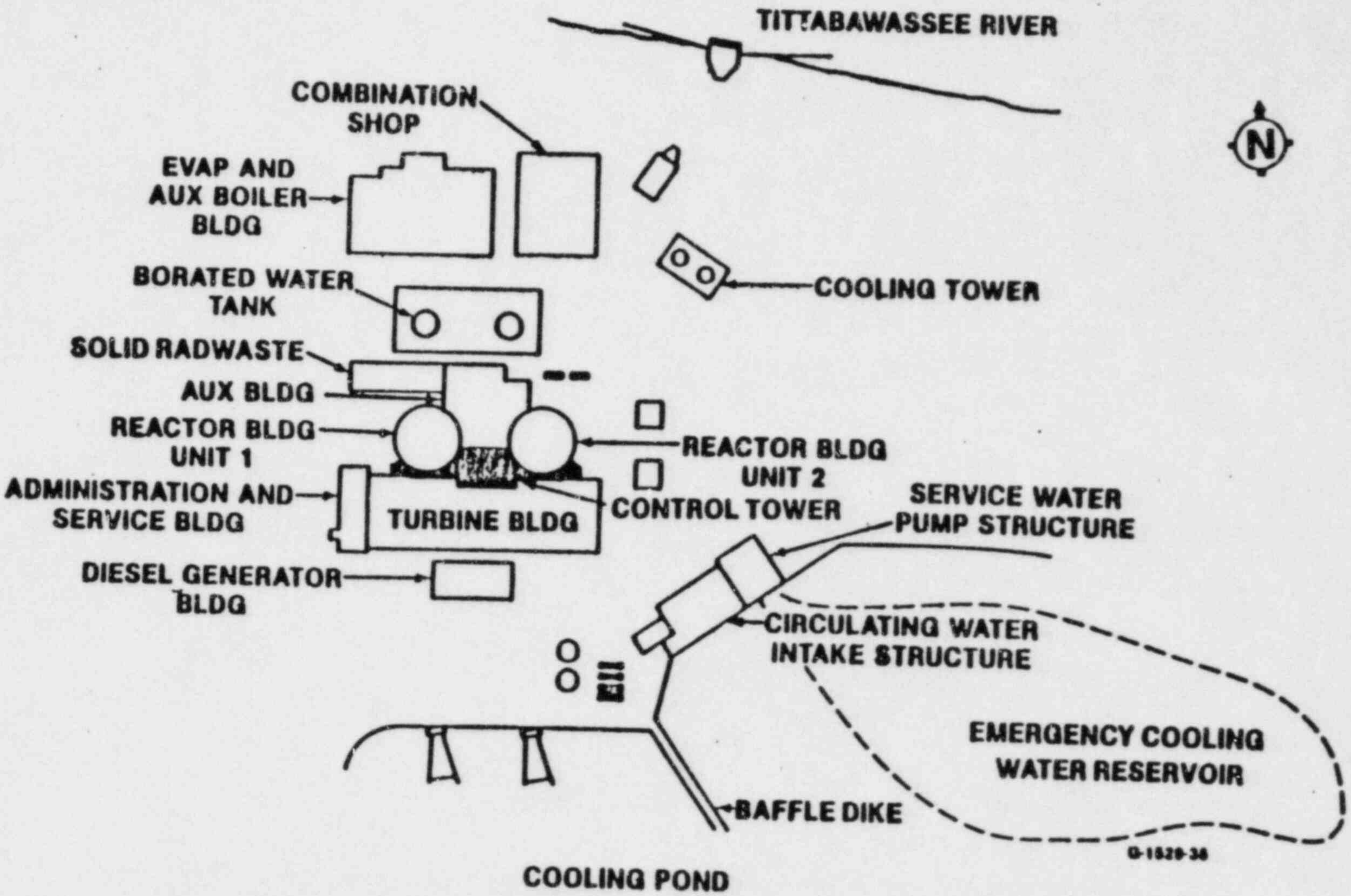


FIG. 4.1 SITE PLAN

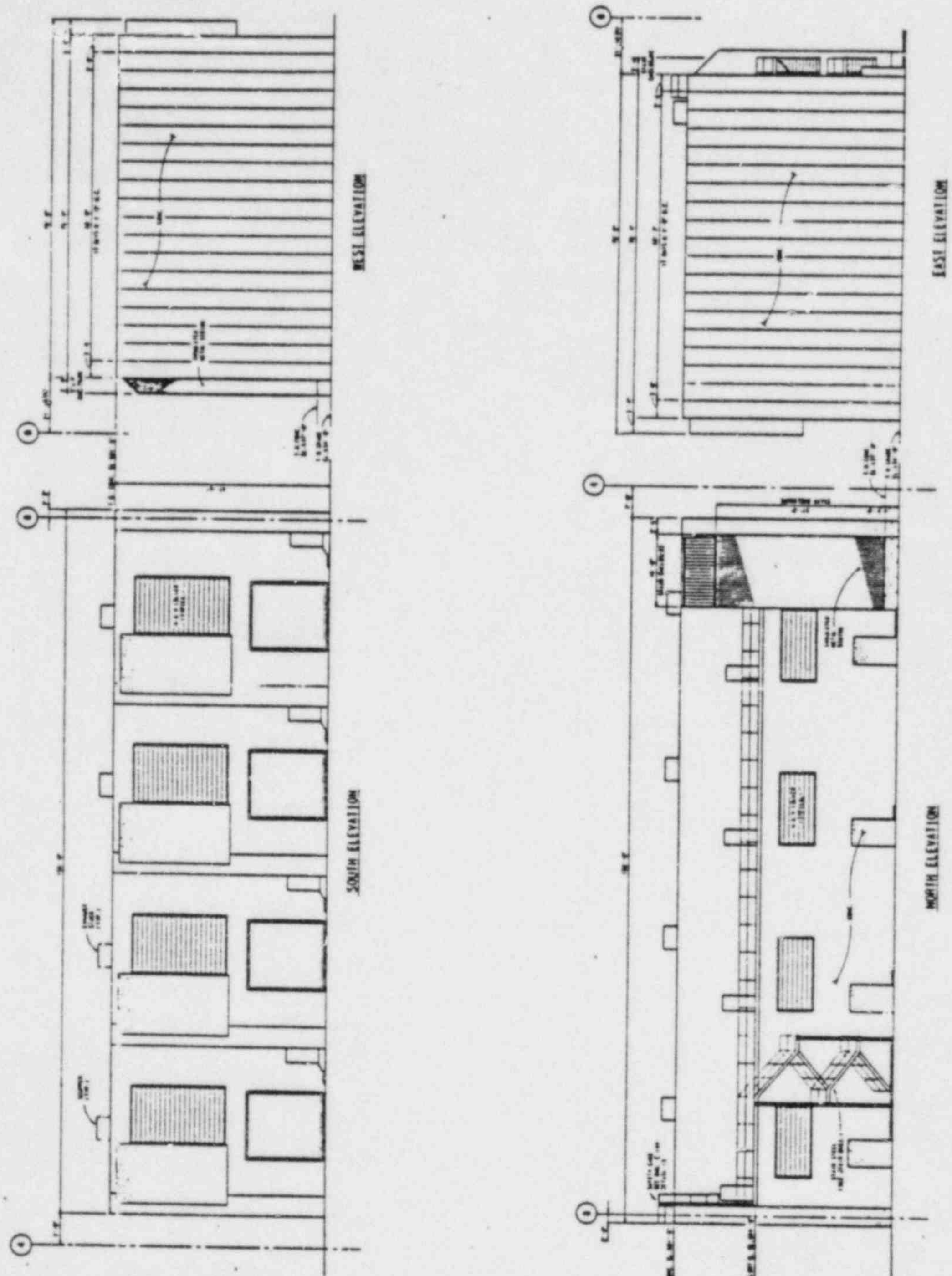


Fig. 4.2 Elevations of Diesel Generator Building

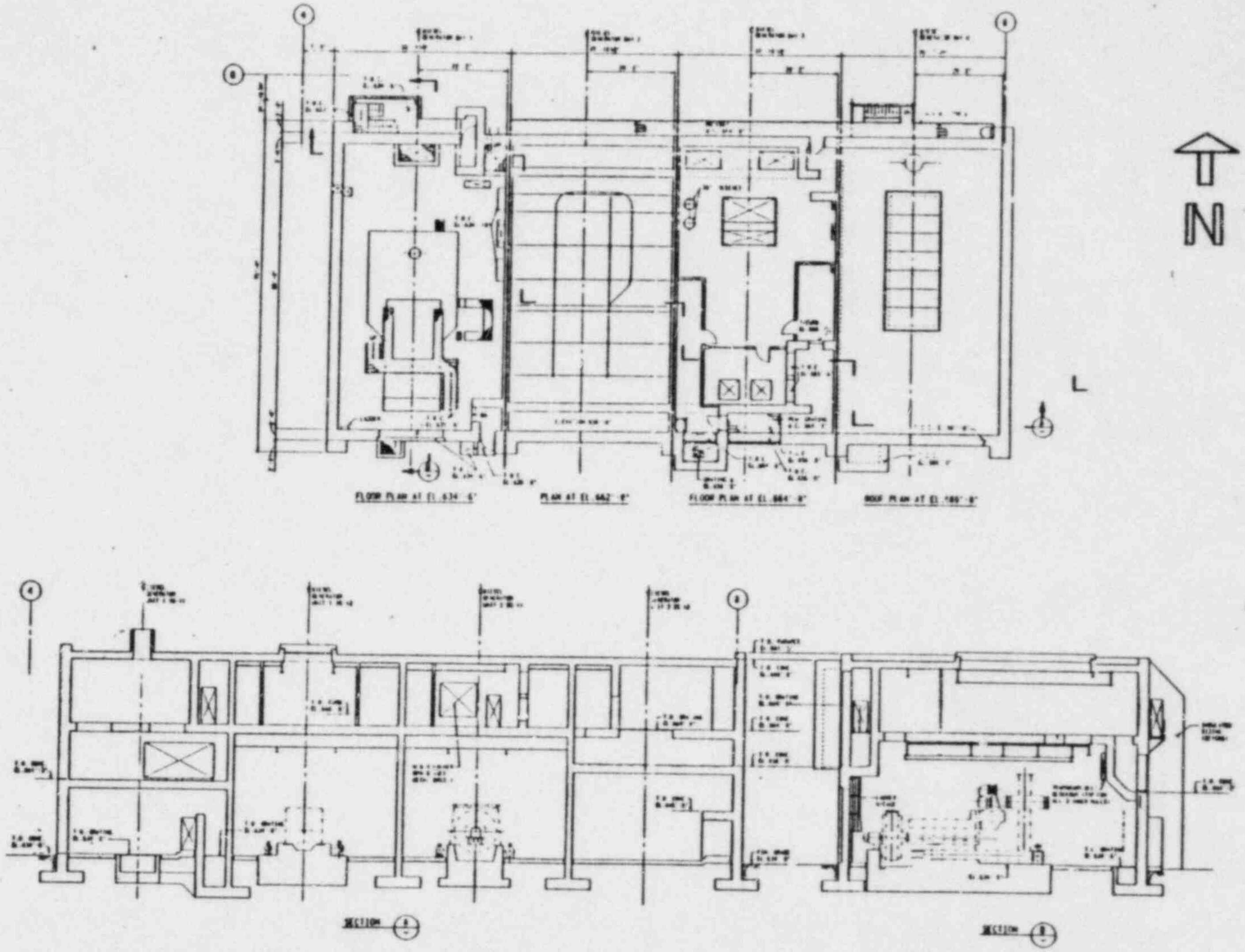


Fig. 4.3 Plans and Sections of Diesel Generator Building

basis accident, and to operate the plant in case of power outages. Because of its safety-related functions, the Diesel Generator Building is designed as a Seismic Category 1 structure. As such, it must maintain its structural integrity during and after a design basis accident, including a postulated safe shutdown earthquake.

As shown in the elevations in Fig. 4.2, overall length of the Diesel Generator Building is 155 ft. Overall width, excluding external enclosures, is 75 ft-4 in.

The basic layout of walls in the Diesel Generator Building is shown in Fig. 4.4. Table 4.1 contains details of selected walls designated in Fig. 4.4. Exterior walls of the structure running in the north-south and east-west directions are 2.5 ft thick. Primary vertical and horizontal reinforcement in these walls is No. 8 bars at 12 in. on centers at each face. Interior walls of the structure run in the north-south direction and are 1.5 ft thick. These walls contain No. 7 bars at 12 in. on center, each direction at each face.

Specified concrete strength for walls of the Diesel Generator Building is 4000 psi. Grade 60 reinforcement is used in the walls.

Table 4.2 contains a listing of Bechtel drawings that were used to obtain data on member dimensions, and on amounts and arrangement of reinforcement.

The Diesel Generator Building was founded on plant fill and constructed between the summer of 1977 and the spring of 1979. It has been reported that settlement of the Diesel Generator

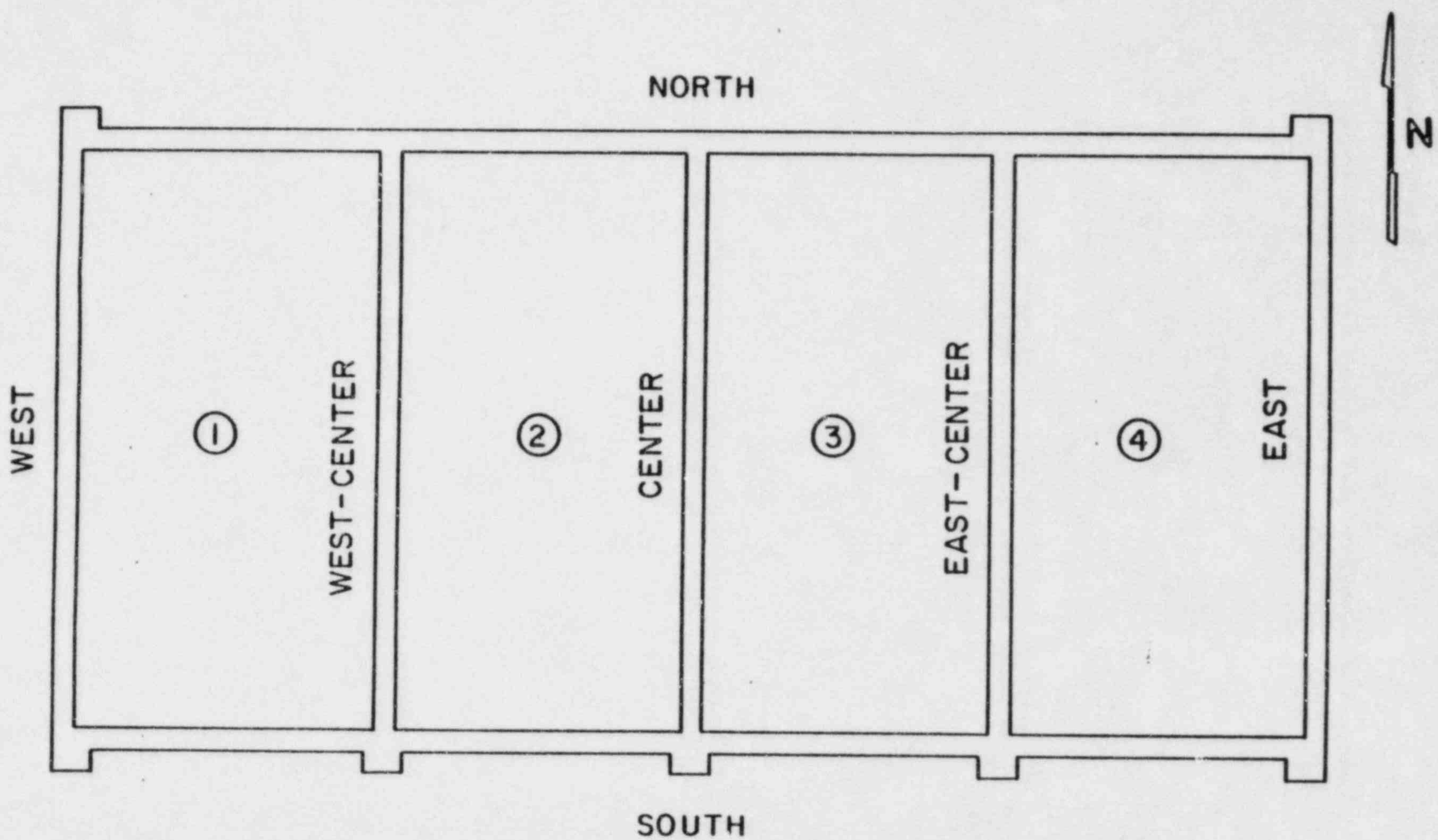


Fig. 4.4 Diesel Generator Building Wall Designations

TABLE 4.1 - DETAILS OF SELECTED WALLS IN DIESEL GENERATOR BUILDING

Wall Description	Wall Thickness, ft.	Primary Vertical Reinforcement*	Primary Horizontal Reinforcement*
North Wall	2.5	No. 8 @ 12"	No. 8 @ 12"
South Wall**	2.5	No. 8 @ 12"	No. 8 @ 12"
West Wall	2.5	No. 8 @ 12"	No. 8 @ 12"
West Center Wall	1.5	No. 7 @ 12"	No. 7 @ 12"
Center Wall	1.5	No. 7 @ 12"	No. 7 @ 12"
East Center Wall	1.5	No. 7 @ 12"	No. 7 @ 12"
East Wall	2.5	No. 8 @ 12"	No. 8 @ 12"

*Reinforcement each face
**Reinforcement layout varies because of numerous wall openings.

TABLE 4.2 - DIESEL GENERATOR BUILDING DRAWINGS

Bechtel Drawing No.	Revision No.	Date	Title
C-140	14	2/14/81	Project Civil Standards. Reinforced Concrete General Notes and Details Sheet No. 1
C-1001	12	10/28/81	Concrete Outlines - Plan at El. 634'-6" Sheet No. 1
C-1002	14	10/28/81	Concrete Outlines - Plan at El. 634'-6" Sheet No. 2
C-1003	9	6/26/80	Concrete Outlines - Plan at El. 664'-0" Sheet No. 1
C-1004	10	7/13/81	Concrete Outlines - Plan at El. 664'-0" Sheet No. 2
C-1005	4	1/31/80	Concrete Outlines - Roof Plan at El. 680'-0" Sheet No. 1
C-1006	3	2/28/79	Concrete Outlines - Roof Plan at El. 680'-0" Sheet No. 2
C-1007	6	3/22/79	Concrete Outlines - Longitudinal Section
C-1008	10	4/22/80	Concrete Outlines - Cross Section
C-1013	6	3/20/80	Reinforcing Details - Foundation Plan Sheet No. 1
C-1014	3	1/13/78	Reinforcing Details - Foundation Plan Sheet No. 2
C-1015	4	9/26/80	Reinforcing Details - Floor Plan at El. 634'-6" Sheet No. 1
C-1016	5	1/5/81	Reinforcing Details - Floor Plan at El. 634'-6" Sheet No. 2
C-1017	2	8/6/79	Reinforcing Details - Floor Plan at El. 664'-0" Sheet No. 1

TABLE 4.2 - DIESEL GENERATOR BUILDING DRAWINGS

(Continued)

Bechtel Drawing No.	Revision No.	Date	Title
C-1018	2	8/6/79	Reinforcing Details - Floor Plan at El. 664'-0" Sheet No. 2
C-1019	2	9/10/79	Reinforcing Details - Roof Plan at El. 680'-0" Sheet No. 1
C-1020	3	9/10/79	Reinforcing Details - Roof Plan at El. 680'-0" Sheet No. 2
C-1021	4	1/6/78	Reinforcing Details - Wall Elevation Sheet No. 1
C-1022	4	1/6/78	Reinforcing Details - Wall Elevation Sheet No. 2
C-1023	4	1/9/79	Reinforcing Details - Wall Elevation Sheet No. 3
C-1024	4	1/9/79	Reinforcing Details - Wall Elevation Sheet No. 4
C-1025	3	1/6/78	Reinforcing Details - Wall Elevation Sheet No. 5
C-1026	4	3/30/79	Reinforcing Details - Wall Elevation Sheet No. 6
C-1027	4	3/30/79	Reinforcing Details - Wall Elevation Sheet No. 7
C-1028	4	4/25/79	Reinforcing Details - Wall Elevation Sheet No. 8
C-1029	4	4/27/78	Reinforcing Details - Wall Elevation Sheet No. 9
C-1030	2	1/6/78	Reinforcing Details - Sections Sheet No. 1
C-1031	4	1/9/78	Reinforcing Details - Sections Sheet No. 2

TABLE 4.2 - DIESEL GENERATOR BUILDING DRAWINGS

(Continued)

Bechtel Drawing No.	Revision No.	Date	Title
C-1032	0	7/21/77	Reinforcing Details - Sections and Details Sheet No. 3
C-1033	0	7/21/77	Reinforcing Details - Sections and Details Sheet No. 4
C-1034	0	7/21/77	Reinforcing Details - Sections and Details Sheet No. 5
C-1035	1	4/27/78	Reinforcing Details - Sections and Details Sheet No. 6
C-1036	4	8/13/80	Reinforcing Details - Sections and Details Sheet No. 7

Building exceeded the estimated settlement value given in the Midland Plant Final Safety Analysis Report. It has also been reported that the excessive settlement was caused by plant fill having a different compaction from that assumed in design.

Footings of the north-south walls of the Diesel Generator Building are penetrated by electrical duct banks as shown in Figs. 4.5 and 4.6. It has been reported that when settlement of the buildings occurred, these duct banks were in contact with the footing. It is postulated that this support restrained vertical movement of the north-south walls. Contact between the duct banks and footings was eliminated in November 1978 by removing concrete at the duct bank-footing interface as illustrated in Figure 4.5.

EVALUATION OF CRACKING

During construction of the Diesel Generator Building, cracks were observed in the concrete walls. It has been hypothesized that these cracks are related to two factors. The first is the normal cracking that can occur from restrained volume changes in reinforced concrete. The second is cracking that can occur because of differential settlement such as that reported in the Diesel Generator Building. In this report, evaluation of cracking is based on crack mapping reported by Bechtel, and on overall visual observations of the building made by Construction Technology Laboratories (CTL) personnel.

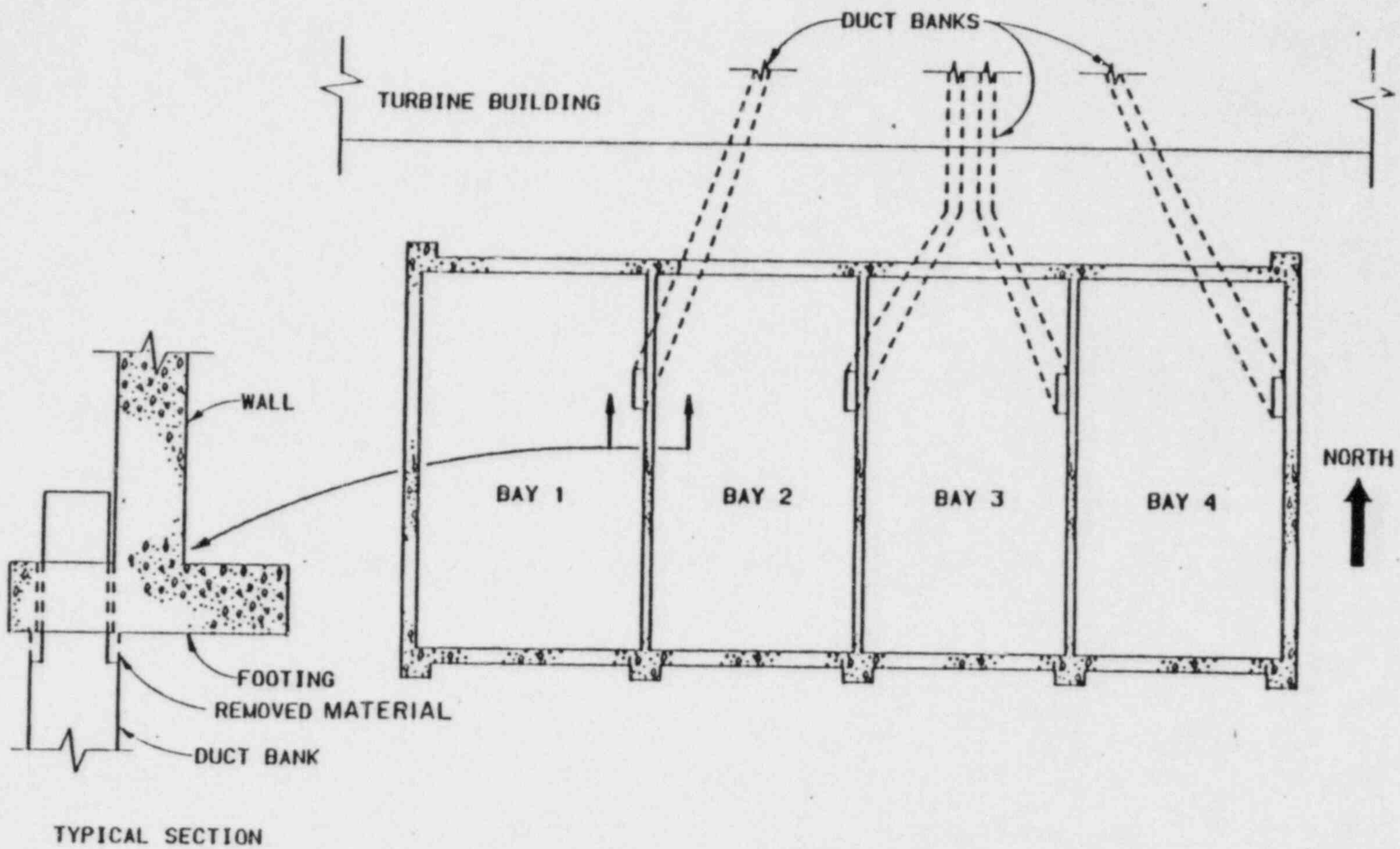


Fig. 4.5 Diesel Generator Building Duct Bank Layout

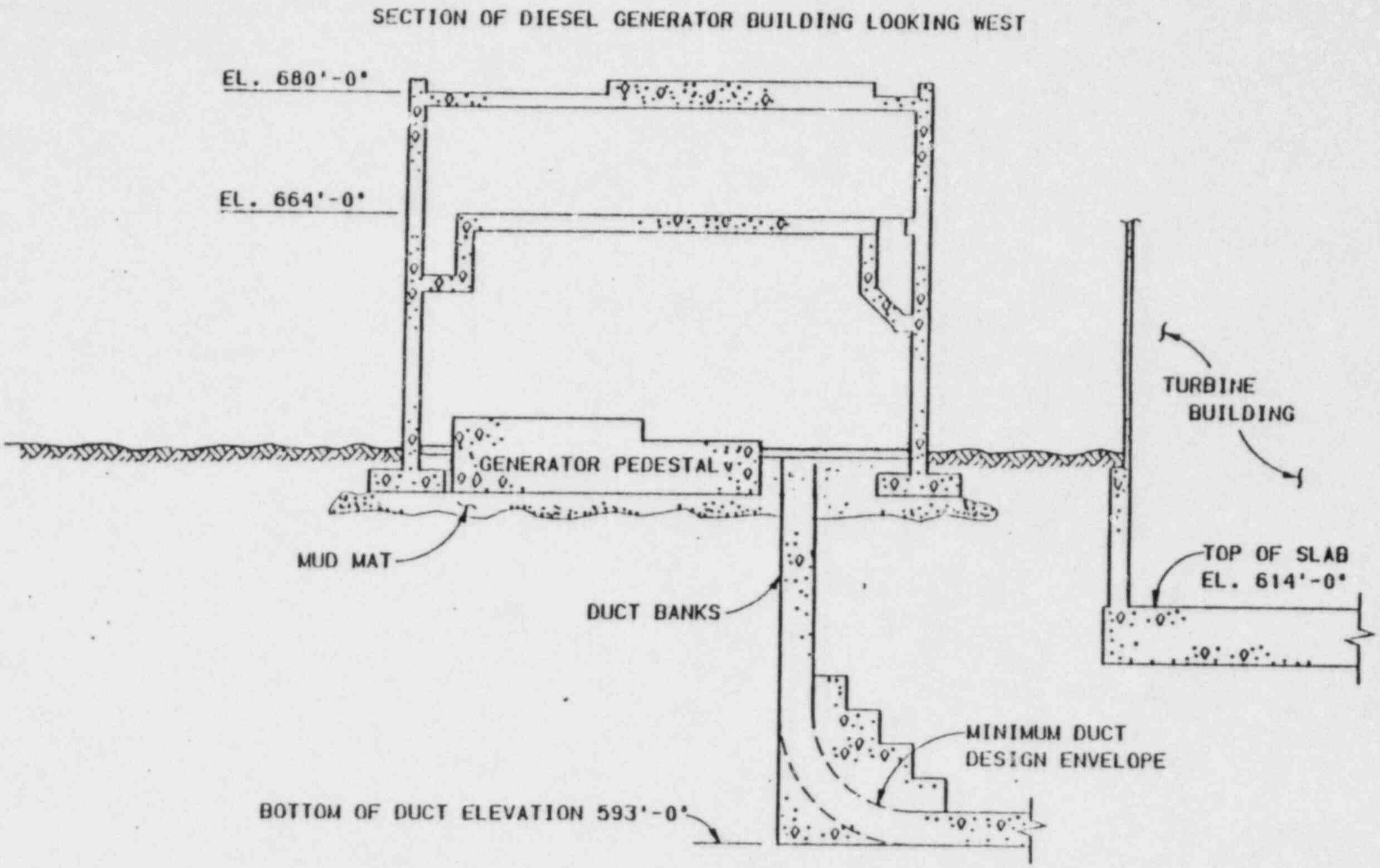


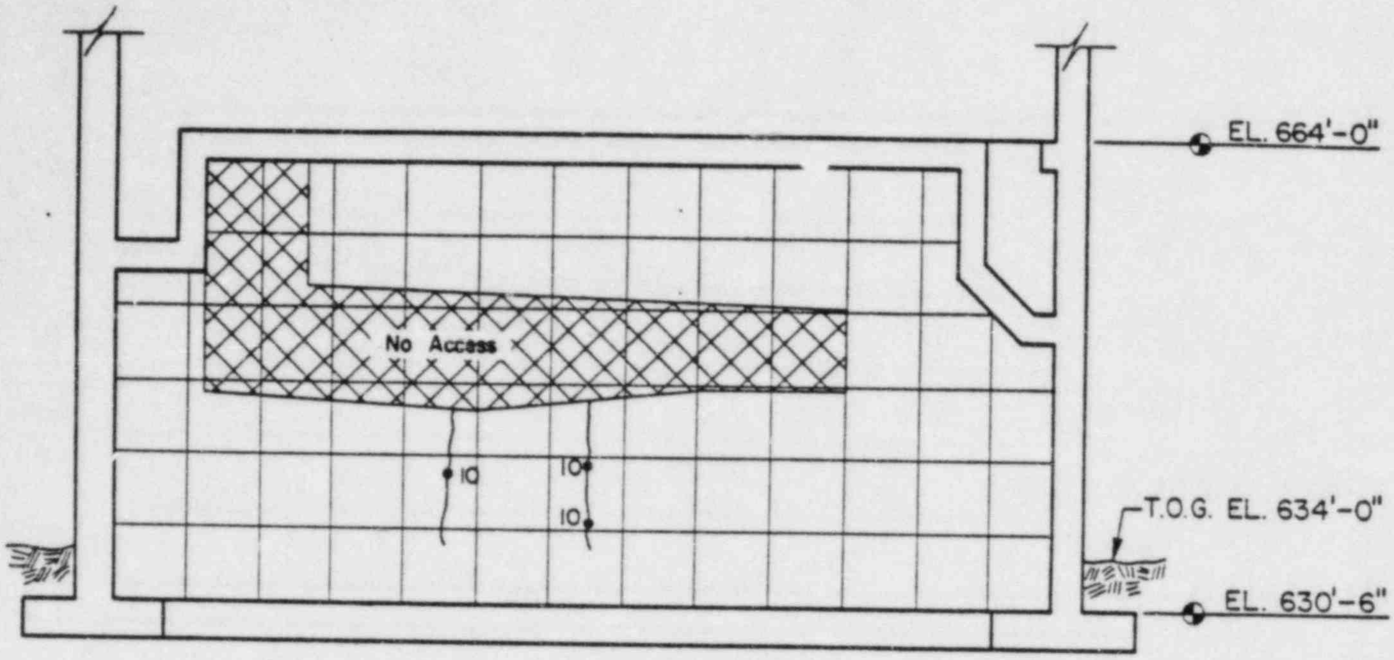
Fig. 4.6 Diesel Generator Building Duct Bank Elevation

Bechtel Crack Mapping

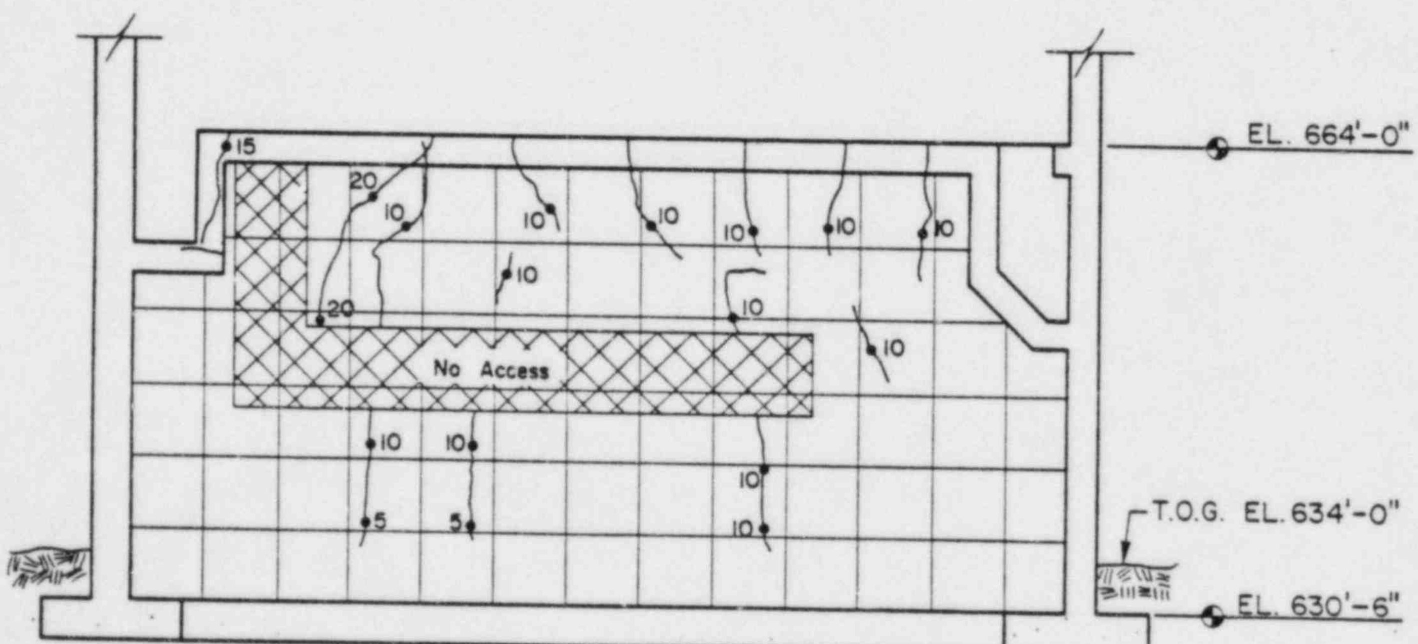
Cracks in walls of the Diesel Generator Building were mapped by Bechtel personnel at several stages of construction. Figures 4.7 through 4.11 show cracks observed in the north-south walls of the Diesel Generator Building between elevations 630 ft-6 in. and 664 ft-0 in. A key to wall designations is shown in Figure 4.4. In Figs. 4.7 through 4.11 only cracks with widths of 0.010 in. or greater are shown. Numbers show measured crack widths in thousandths to the nearest five thousandth. The drawings are based on cracks mapped in July 1981. Maximum reported crack width is 0.020 in. Cracking in the vicinity of duct banks is particularly evident in the center wall as shown in Fig 4.9.

Cracks observed in north-south walls of the Diesel Generator Building between elevations 664 ft-0 in. and 681 ft-6 in. are shown in Figs. 4.12 through 4.16. These figures are taken from Bechtel drawing SK-C-669. Cracks shown in this drawing were mapped in January 1980.

Figures 4.17 and 4.18 show cracking observed in the north wall of the Diesel Generator Building. These figures are taken from Bechtel drawing SK-C-659. The cracks were mapped in February 1980. Cracks in this wall were remapped by Bechtel personnel in July 1981. Results of the remapping are shown in Bechtel drawing SK-C-770, Revision A dated February 9, 1982. Although a few additional cracks with widths of 0.010 in. or greater were observed in July 1981, no significant differences in overall crack patterns were noted.



WEST WALL - EAST FACE



WEST WALL - WEST FACE

Fig. 4.7 Cracking in West Wall of Diesel Generator Building From Bechtel Drawing No. SK-C-759 (Crack Widths in 0.001" Increments)

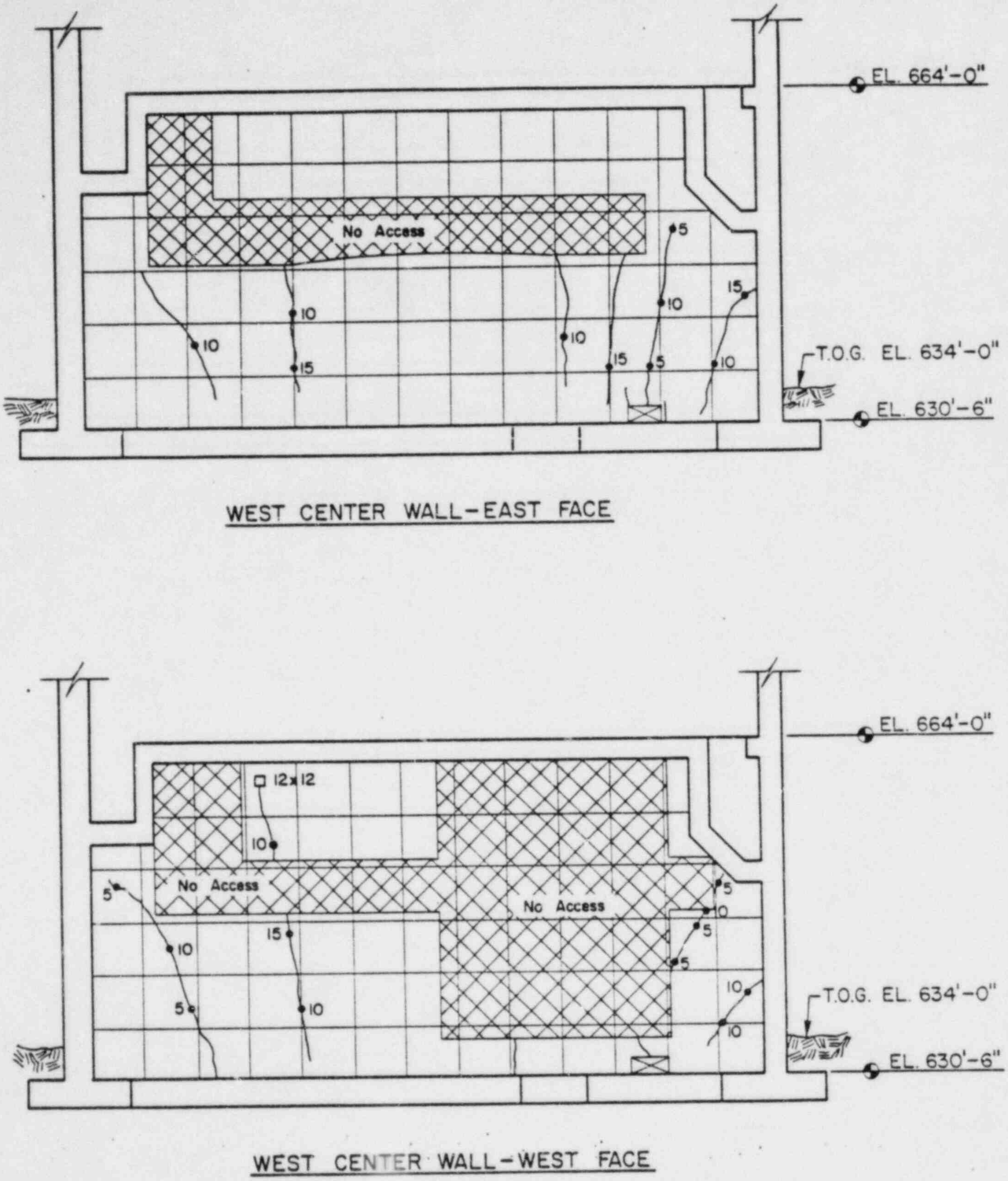
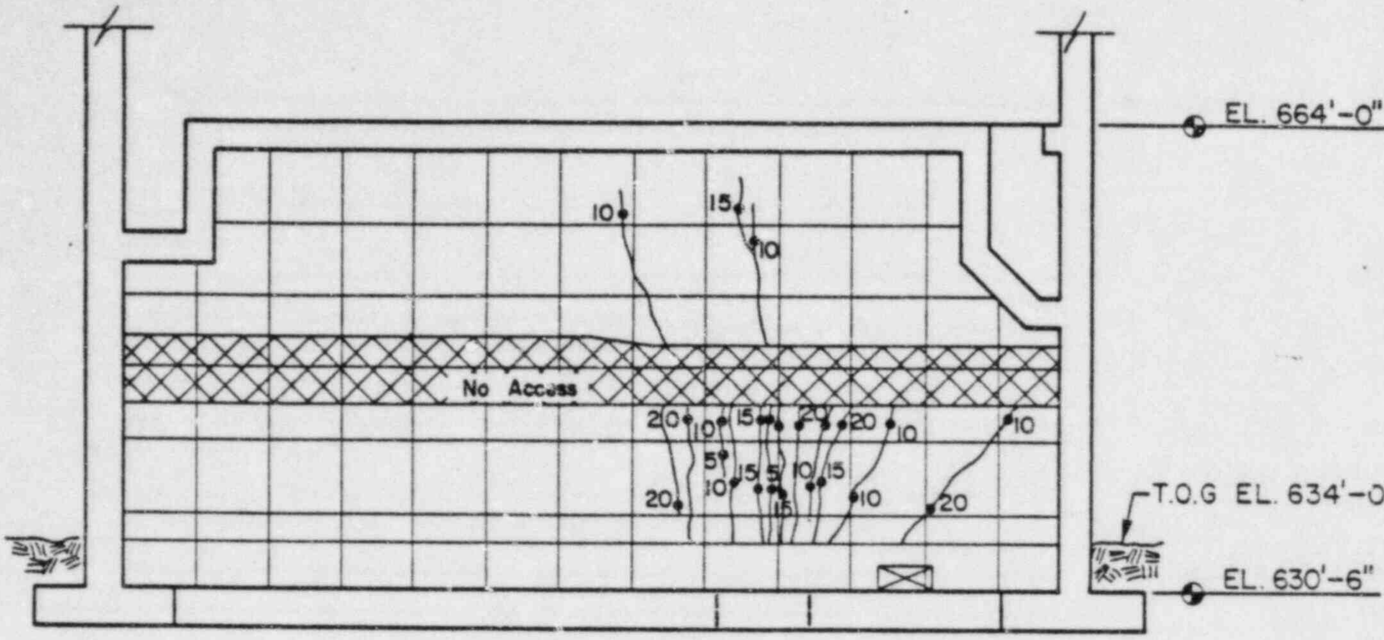
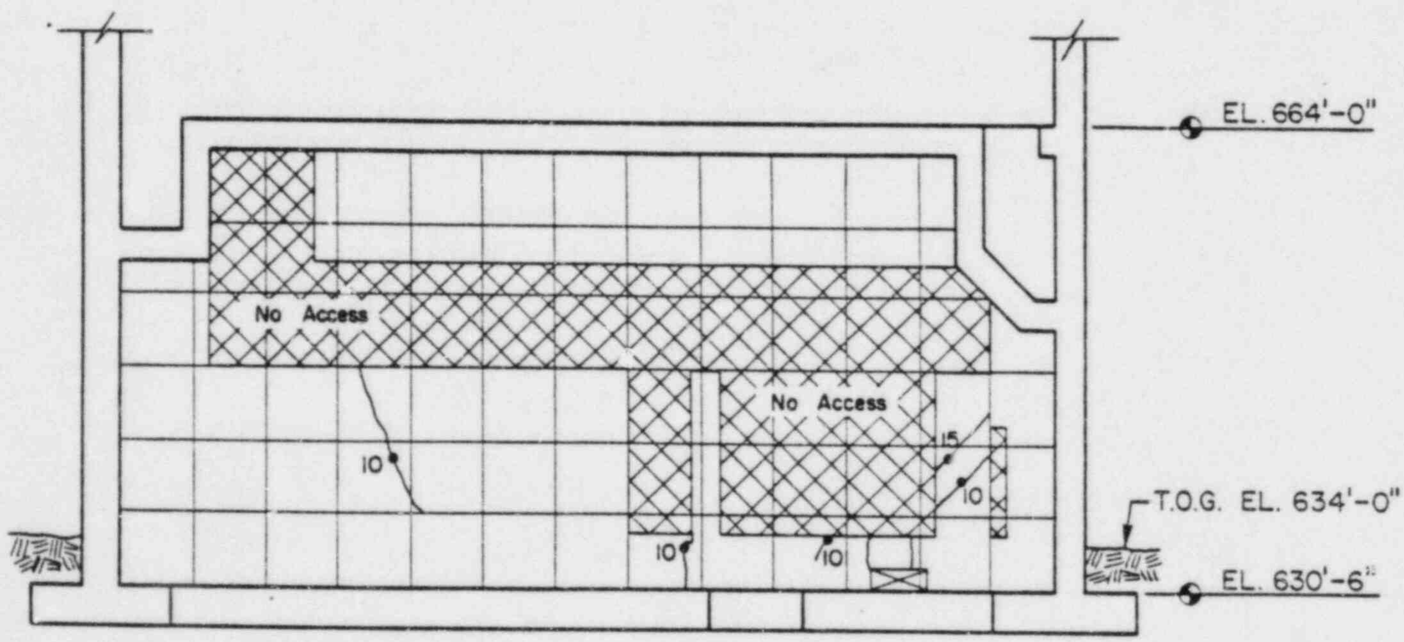


Fig. 4.8 Cracking in West Center Wall of Diesel Generator Building
From Bechtel Drawing No. SK-C-759
(Crack Widths in 0.001" Increments)

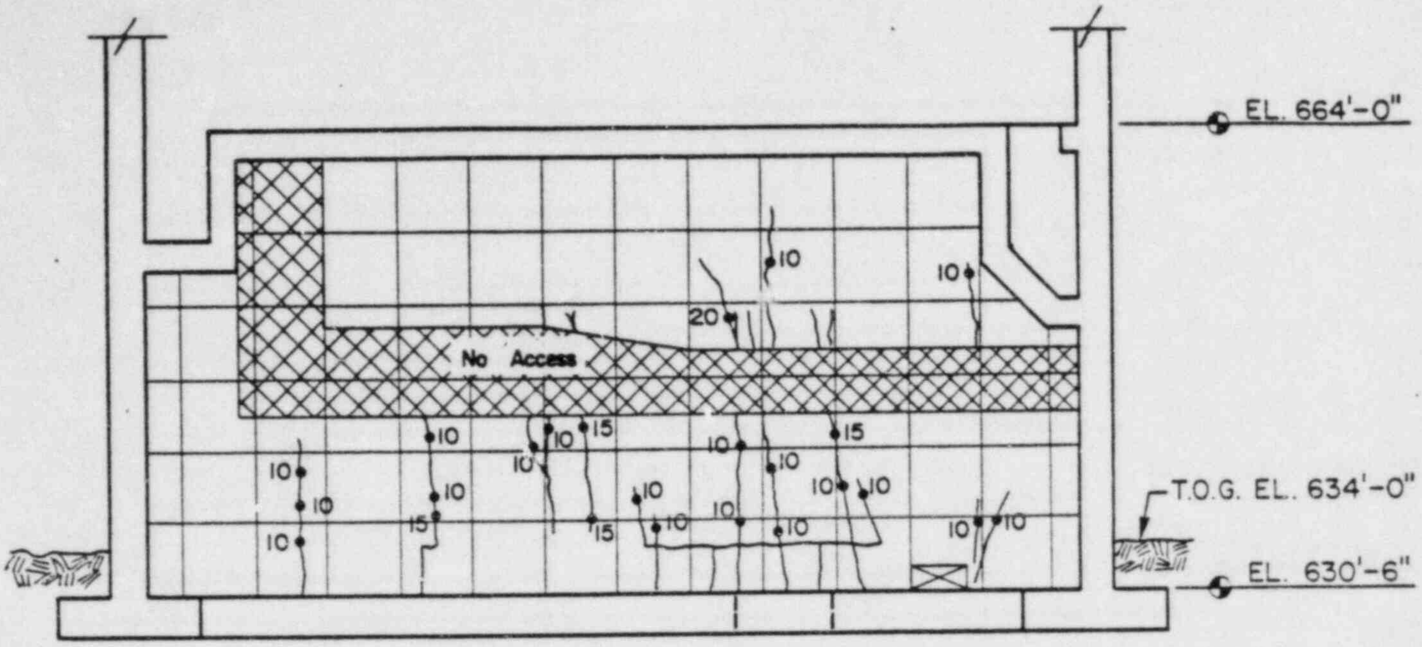


CENTER WALL - EAST FACE

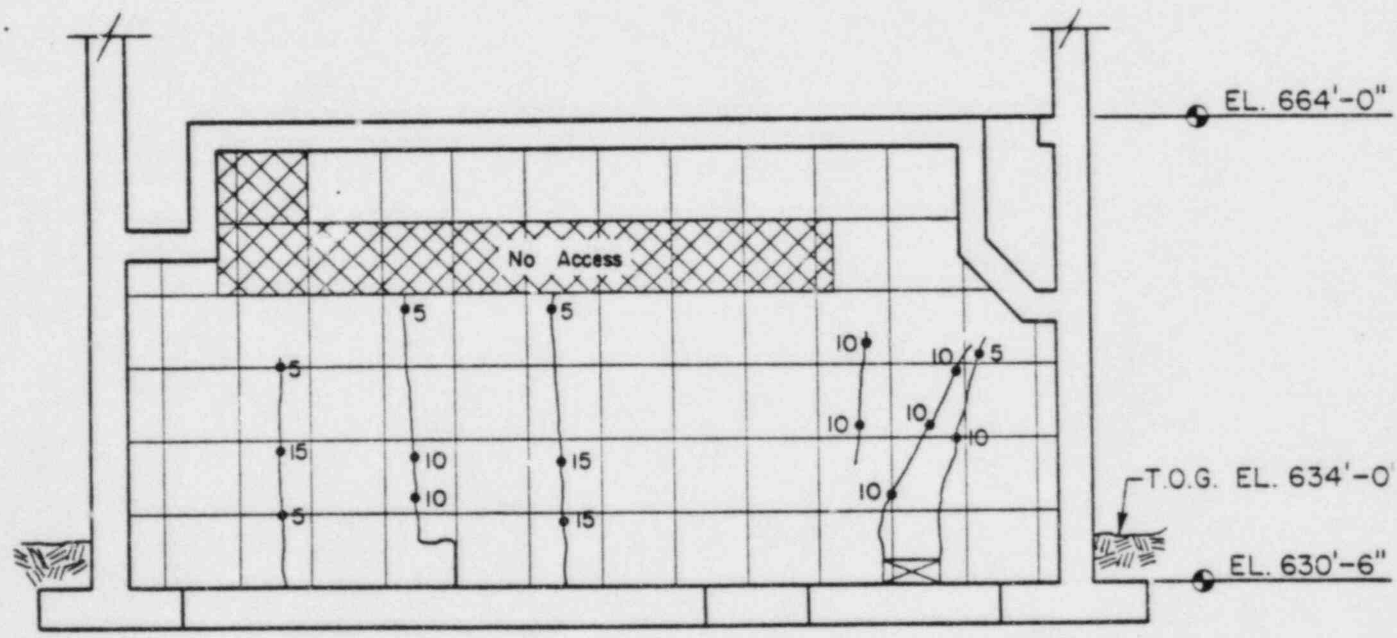


CENTER WALL - WEST FACE

Fig. 4.9 Cracking in Center Wall of Diesel Generator Building From Bechtel Drawing No. SK-C-759 (Crack Widths in 0.001" Increments)

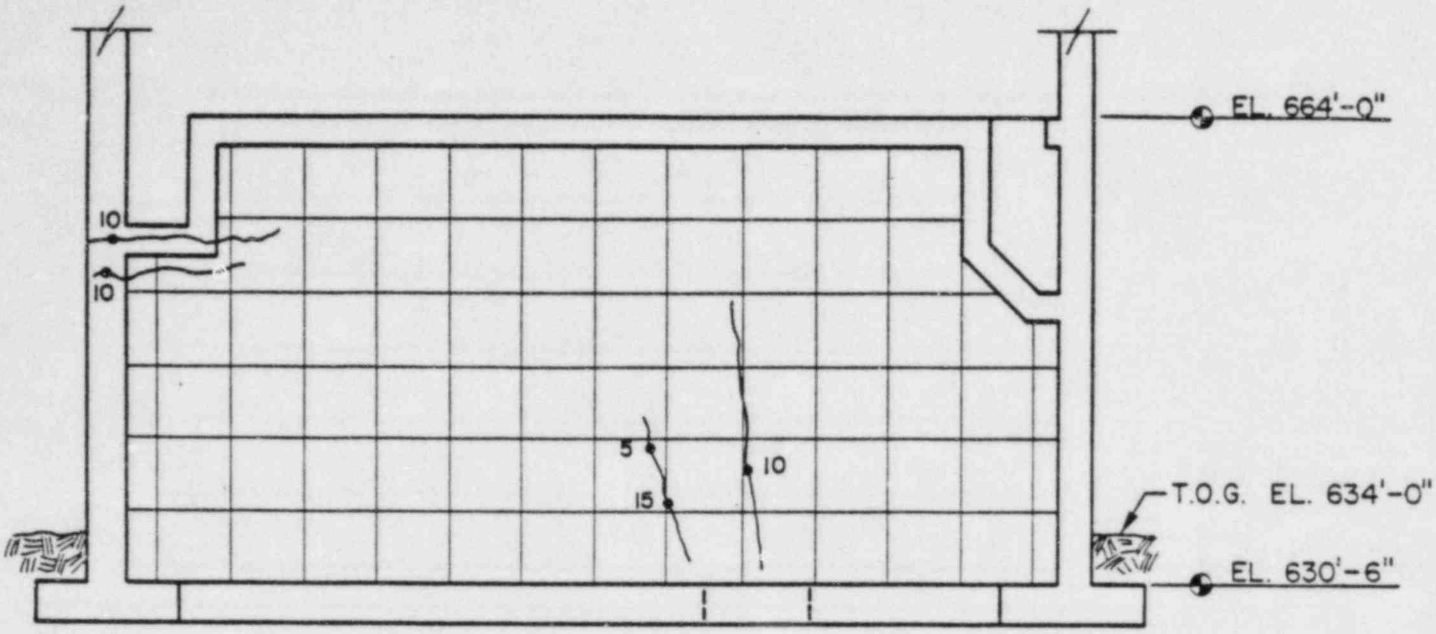


EAST CENTER WALL - EAST FACE

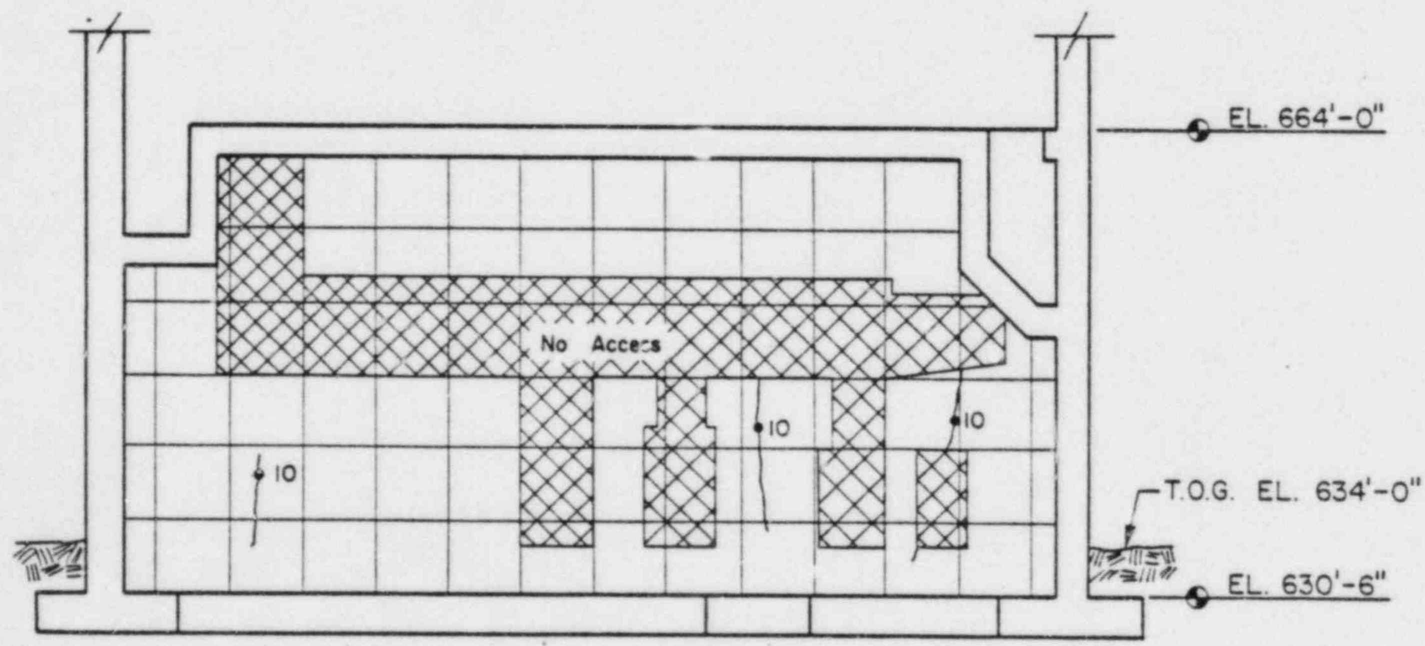


EAST CENTER WALL - WEST FACE

Fig. 4.10 Cracking in East Center Wall of Diesel Generator Building From Bechtel Drawing No. SK-C-759 (Crack Widths in 0.001" Increments)

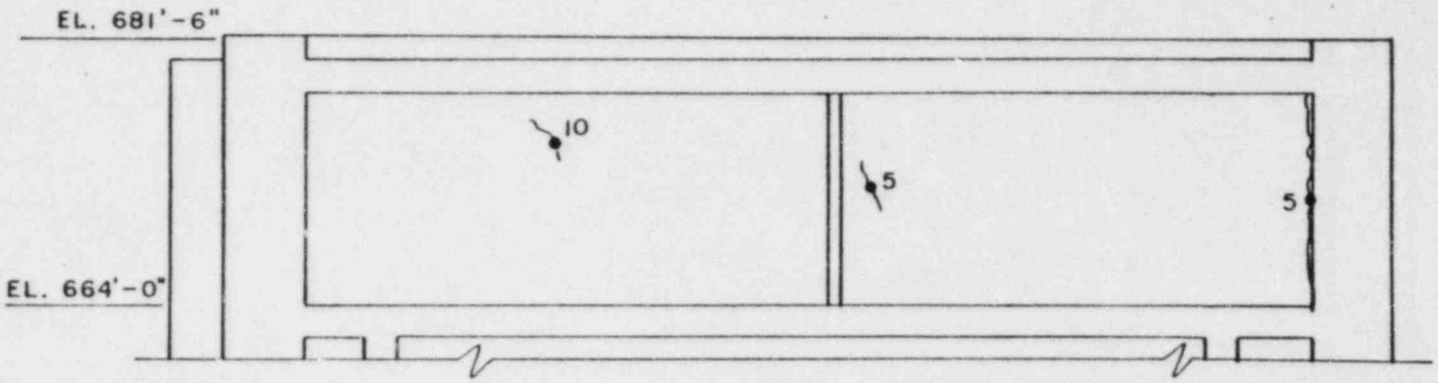


EAST WALL-EAST FACE

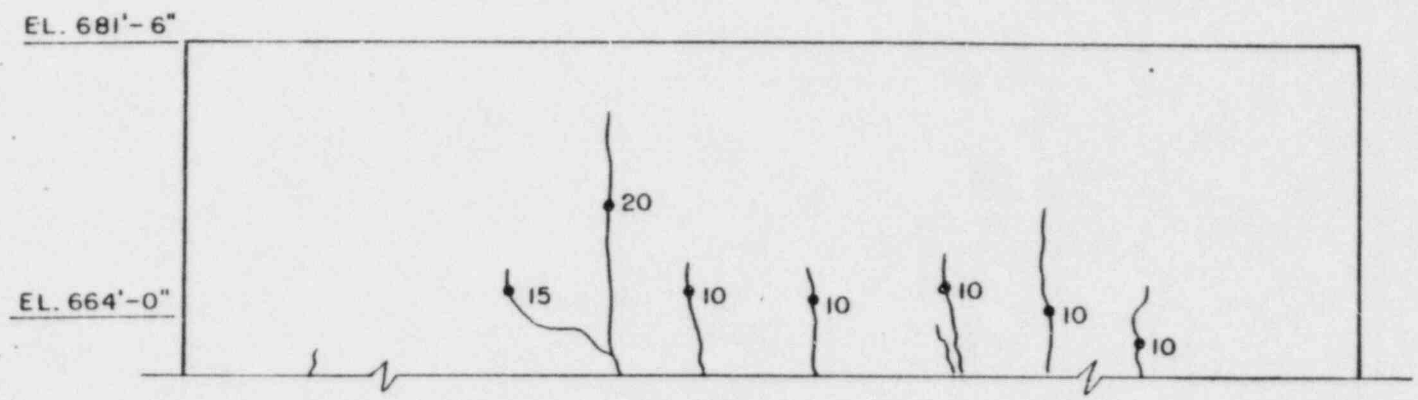


EAST WALL-WEST FACE

Fig. 4.11 Cracking in East Wall of Diesel Generator Building From Bechtel Drawing No. SK-C-759 (Crack Widths in 0.001" Increments)



WEST WALL - EAST FACE



WEST WALL - WEST FACE

Fig. 4.12 Cracking in West Wall of Diesel Generator Building
From Bechtel Drawing No. SK-C-669
(Crack Widths in 0.001" Increments)

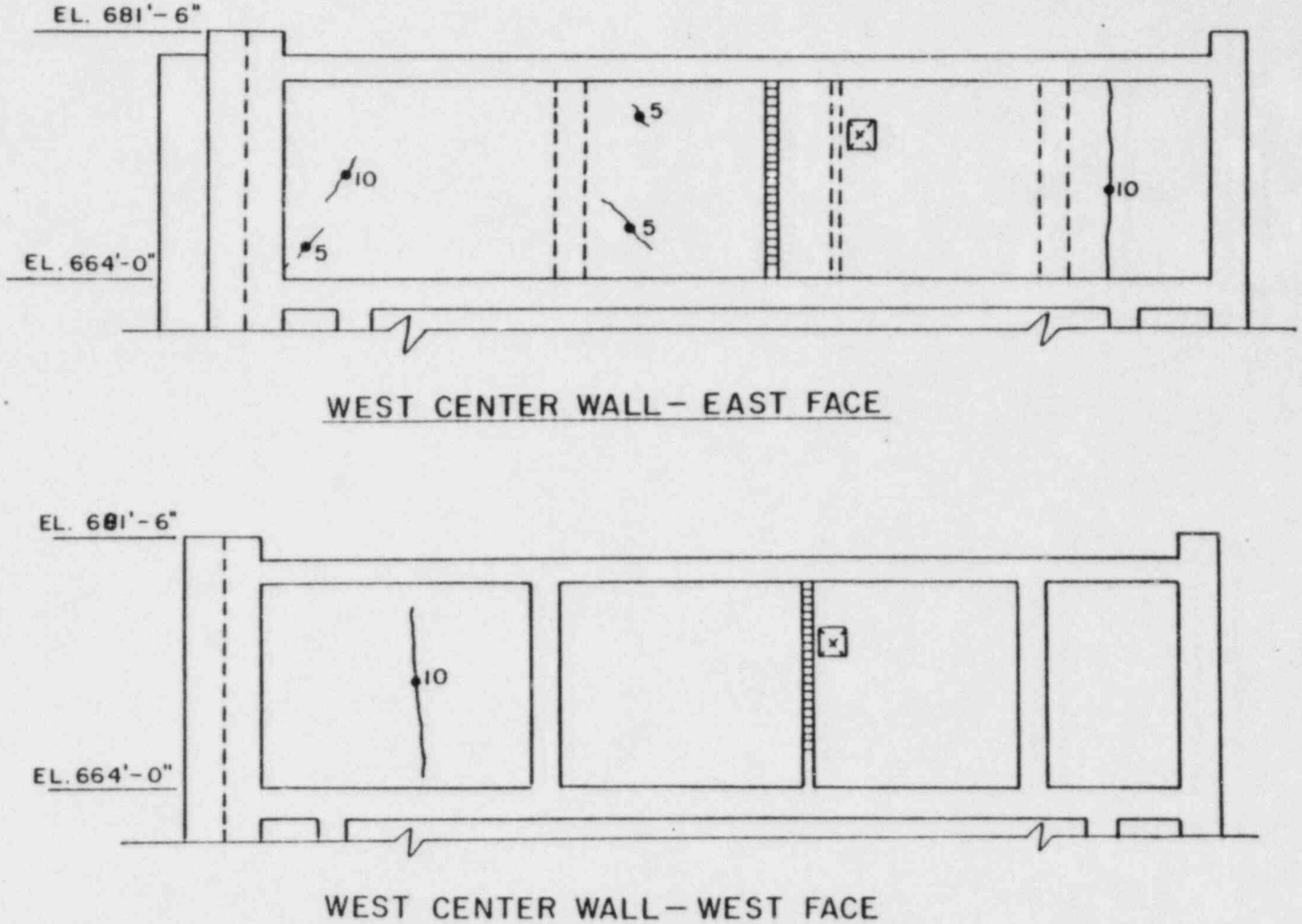
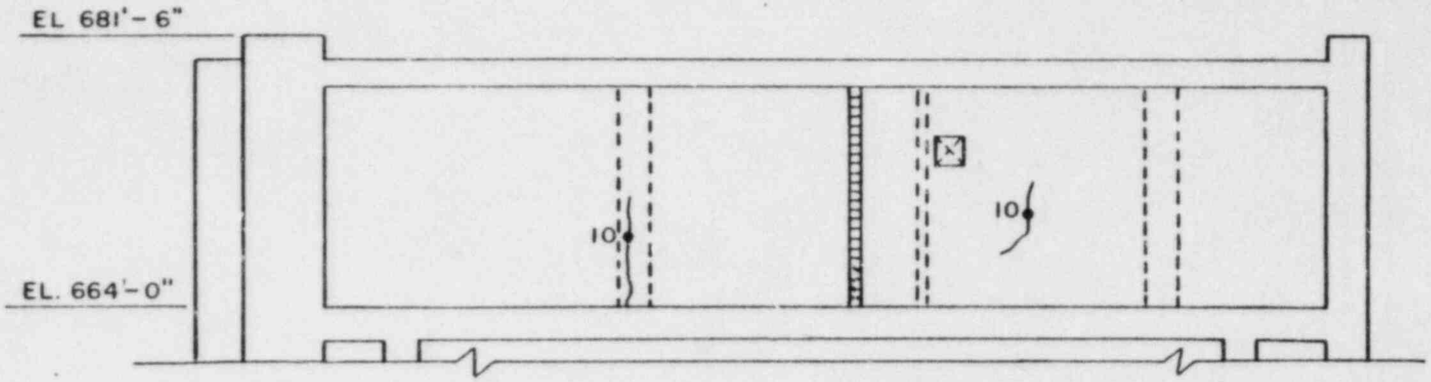
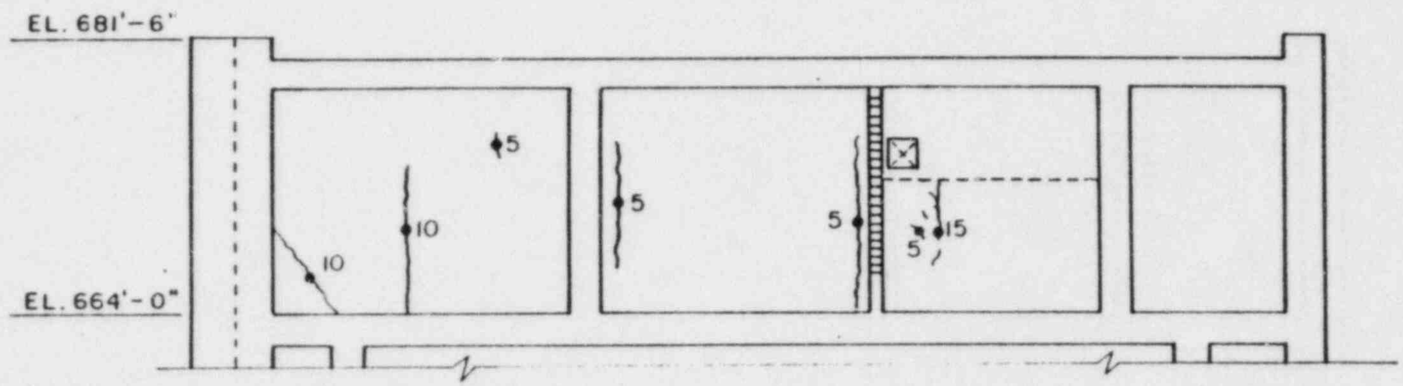


Fig. 4.13 Cracking in West Center Wall of Diesel Generator Building
From Bechtel Drawing No. SK-C-669
(Crack Widths in 0.001" Increments)

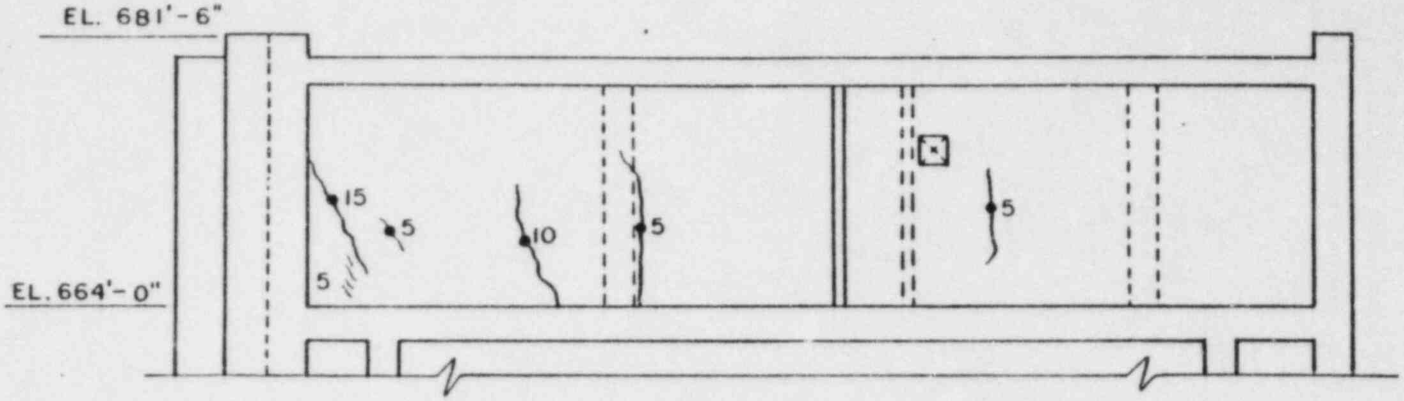


CENTER WALL-EAST FACE

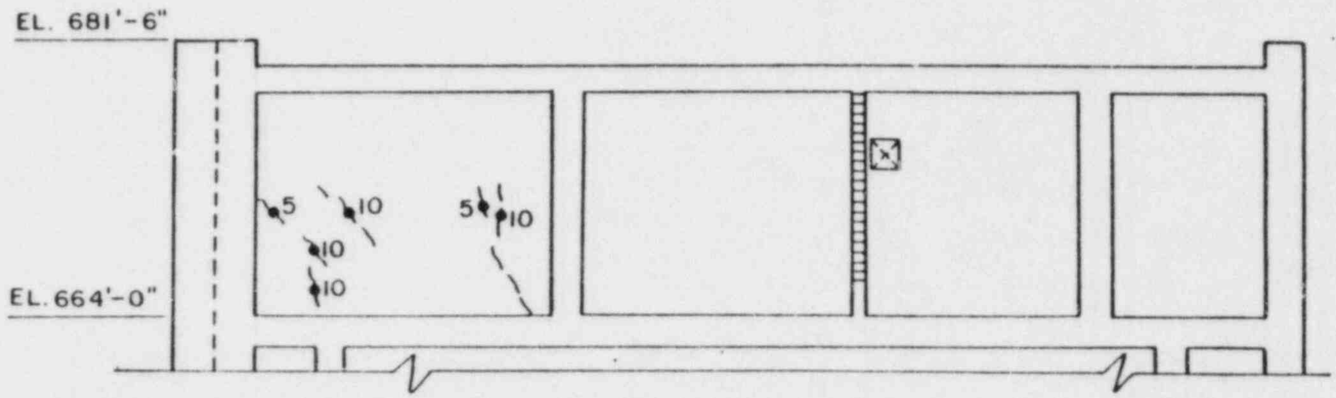


CENTER WALL - WEST FACE

Fig. 4.14 Cracking in Center Wall of Diesel Generator Building
From Bechtel Drawing No. SK-C-669
(Crack Widths in 0.001" Increments)

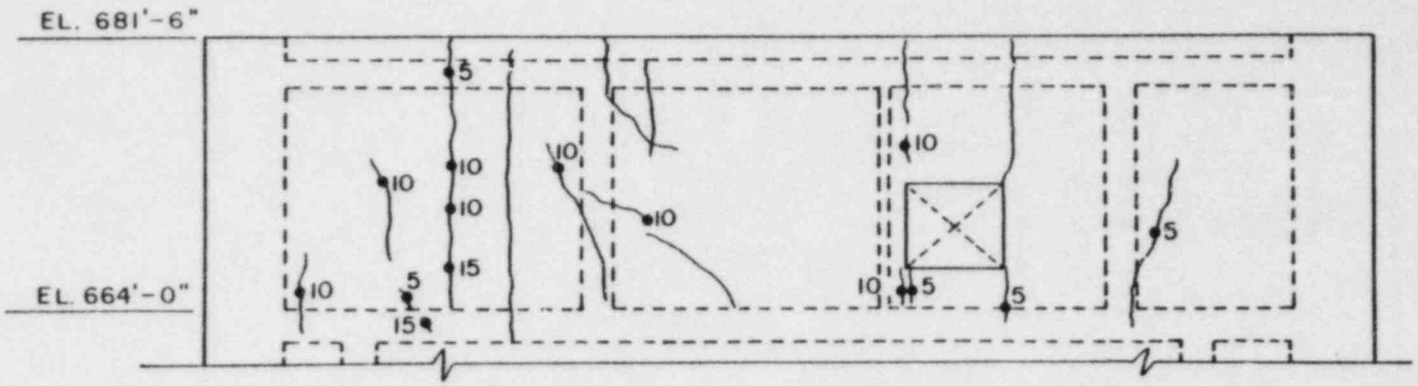


EAST CENTER WALL – EAST FACE

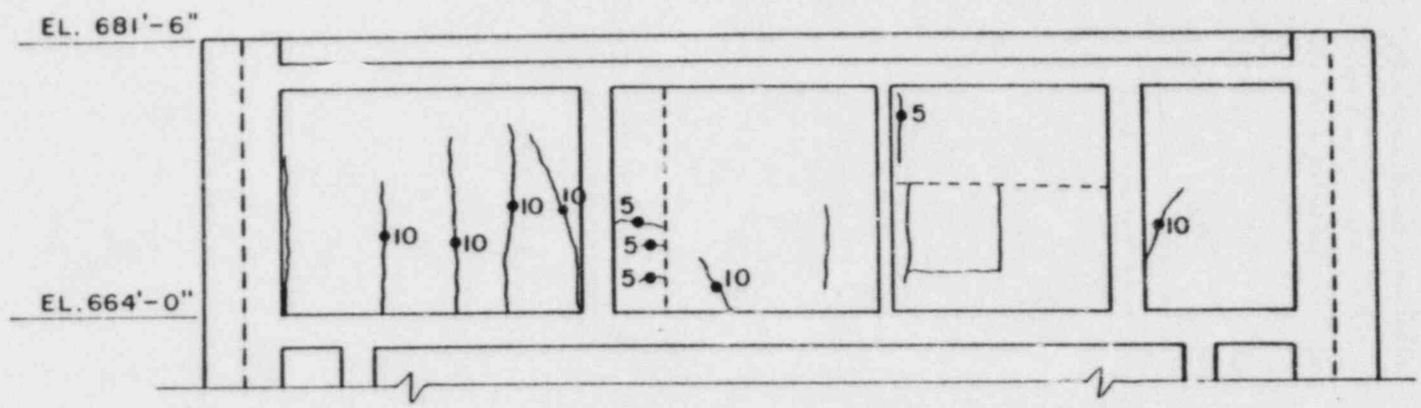


EAST CENTER WALL – WEST FACE

Fig. 4.15 Cracking in East Center Wall of Diesel Generator Building
From Bechtel Drawing No. SK-C-669
(Crack Widths in 0.001" Increments)



EAST WALL - EAST FACE



EAST WALL - WEST FACE

Fig. 4.16 Cracking in East Wall of Diesel Generator Building
From Bechtel Drawing No. SK-C-669
(Crack Widths in 0.001" Increments)

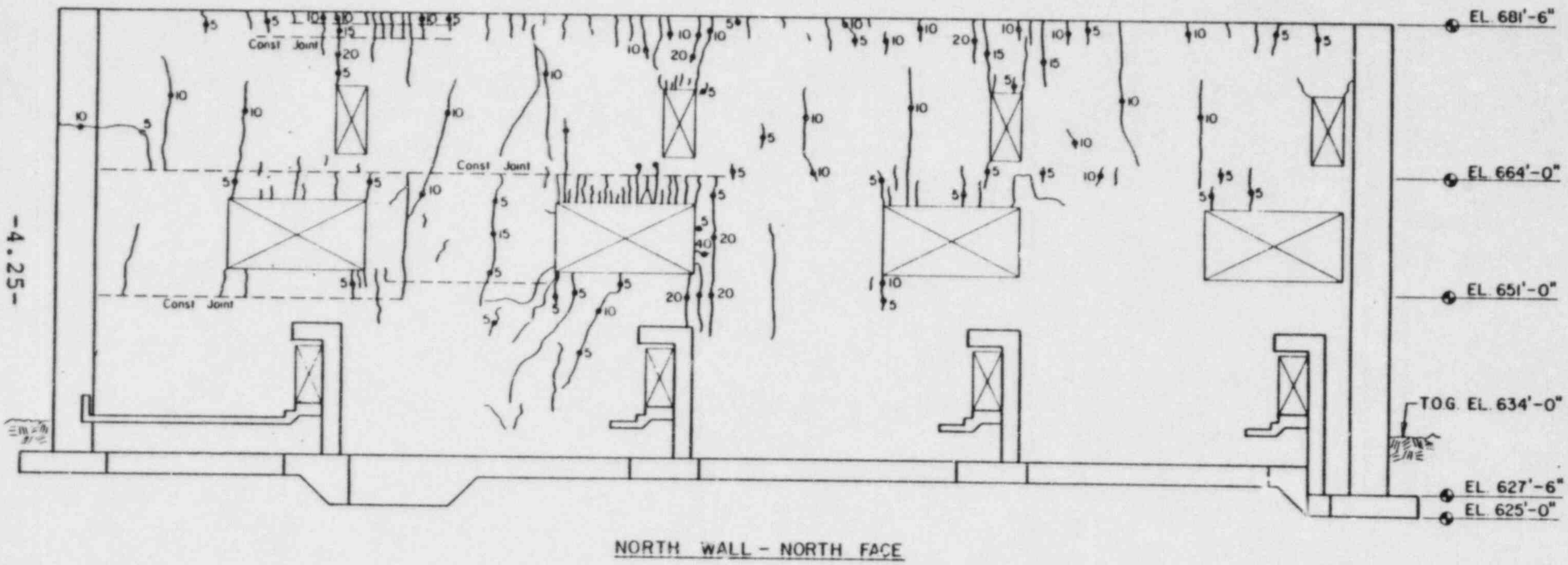
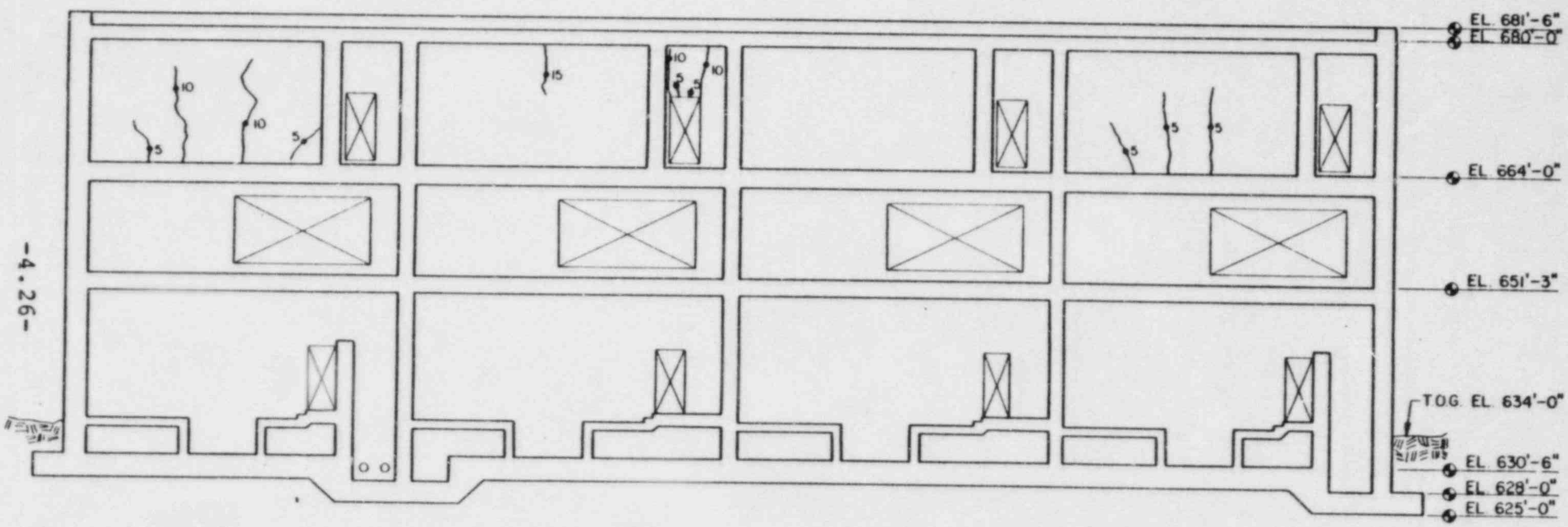


Fig. 4.17 Cracking in North Wall - North Face of Diesel Generator Building From Bechtel Drawing SK-C-659 (Crack Widths in 0.001" Increments)



NORTH WALL-SOUTH FACE

Fig. 4.18 Cracking in North Wall - South Face of Diesel Generator Building From Bechtel Drawing SK-C-659 (Crack Widths in 0.001" Increments)

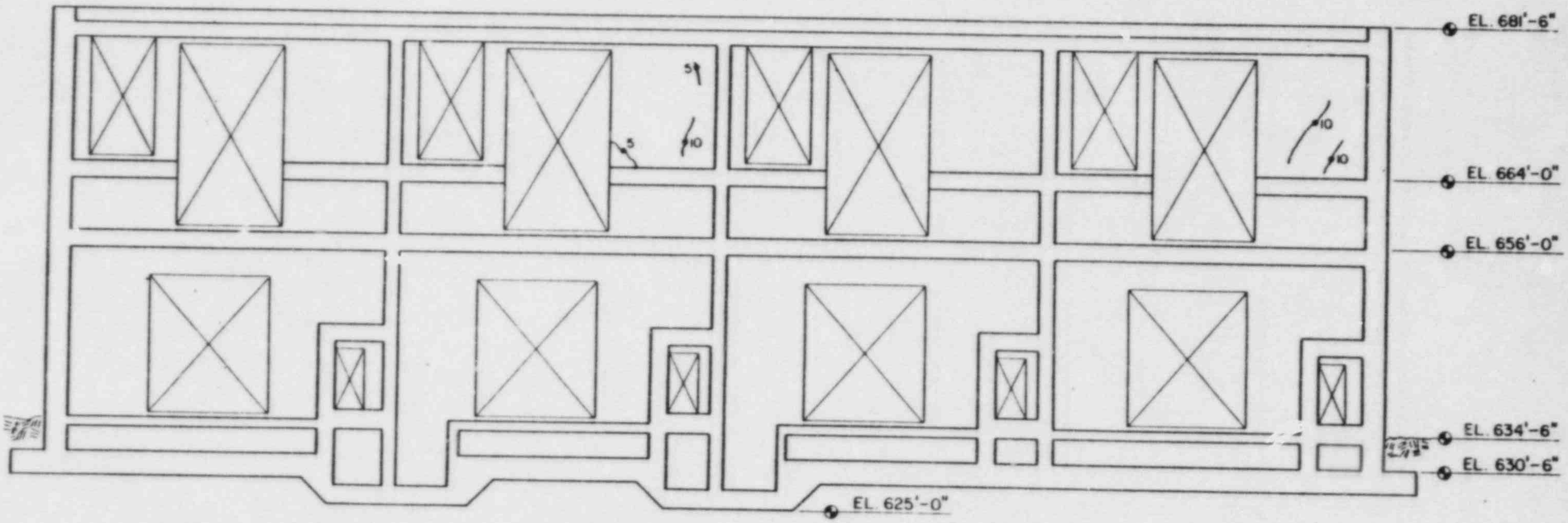
Figures 4.19 and 4.20 show cracks mapped in the south wall of the Diesel Generator Building. These figures were taken from Bechtel drawing Number SK-C-658.

Based on overall review of Bechtel drawings, it appears that many of the cracks shown are attributed to restrained volume changes that occur in concrete during curing and subsequent drying. However, the patterns observed in several north-south walls of the Diesel Generator Building indicate that cracks could have resulted from differential settlement of the walls between the duct banks and the north and south portions of the structure. It is possible that differential settlement was caused by extra support provided by the duct banks when they came in contact with the wall footings.

CTL Observations

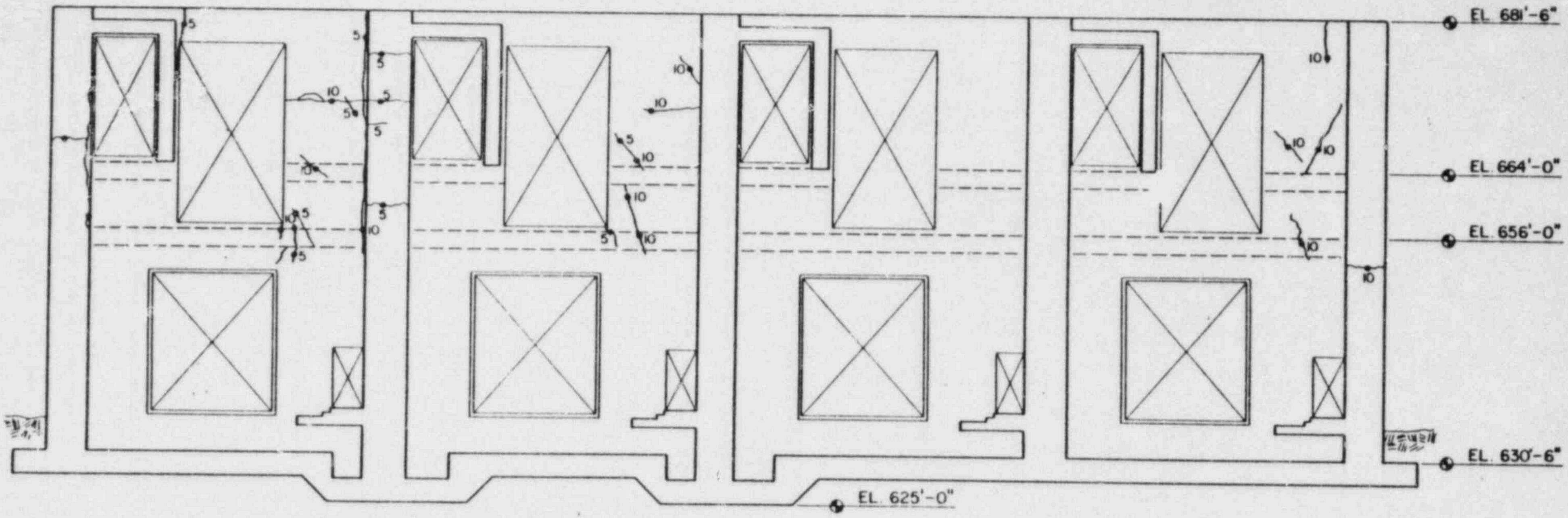
Visual observations of cracking in walls of the Diesel Generator Building were made by CTL personnel on January 12, 1982 and February 9, 1982. Construction Technology Laboratories personnel did not do detailed mapping of cracks. CTL inspections were made to obtain an overall impression of cracking in the structure and to correlate this impression with that obtained from review of Bechtel crack mapping drawings. In general, impressions obtained from the visual inspection at the site were consistent with those obtained from review of the Bechtel drawings.

Because the observed pattern of cracks in the center north-south wall of the Diesel Generator Building was most indicative of cracks caused by differential settlement, one face of this



SOUTH WALL - NORTH FACE

Fig. 4.19 Cracking in South Wall - North Face of Diesel Generator Building From Bechtel Drawing SK-C-658 (Crack Widths in 0.001" Increments)



SOUTH WALL-SOUTH FACE

Fig. 4.20 Cracking in South Wall - South Face of Diesel Generator Building From Bechtel Drawing SK-C-658 (Crack Widths in 0.001" Increments) wall was remapped by CTL personnel on February 9, 1982. Figure 4.21 shows cracks observed in the center wall on the east face. Maximum measured crack width was 0.025 in. The pattern of cracks at the electrical duct penetration is consistent with a pattern that could occur because of differential settlement about the duct. Development of settlement cracks is discussed by Dr. M. A. Sozen in the main body of this report.

RECOMMENDED PROGRAM FOR MONITORING STRUCTURAL INTEGRITY

It is recommended that future integrity of the Diesel Generator Building be monitored by periodic measurements of displacements of the structure and by periodic inspection of cracks.

Displacement Monitoring

Displacement measurements should be made periodically to monitor absolute and relative movement of walls of the Diesel Generator Building. Figure 4.22 shows approximate locations of recommended displacement measurement points. These measurements will confirm that current estimates of settlement limits are not exceeded and will provide a means to verify structural integrity. Measured displacements should be recorded as a function of time. The frequency of measurements will be selected in relation to the observed rate of displacement.

It is also recommended that the time history of displacements be submitted on a regular basis to qualified engineers familiar with reinforced concrete behavior and design. The qualified engineer will provide recommendations on whether wall

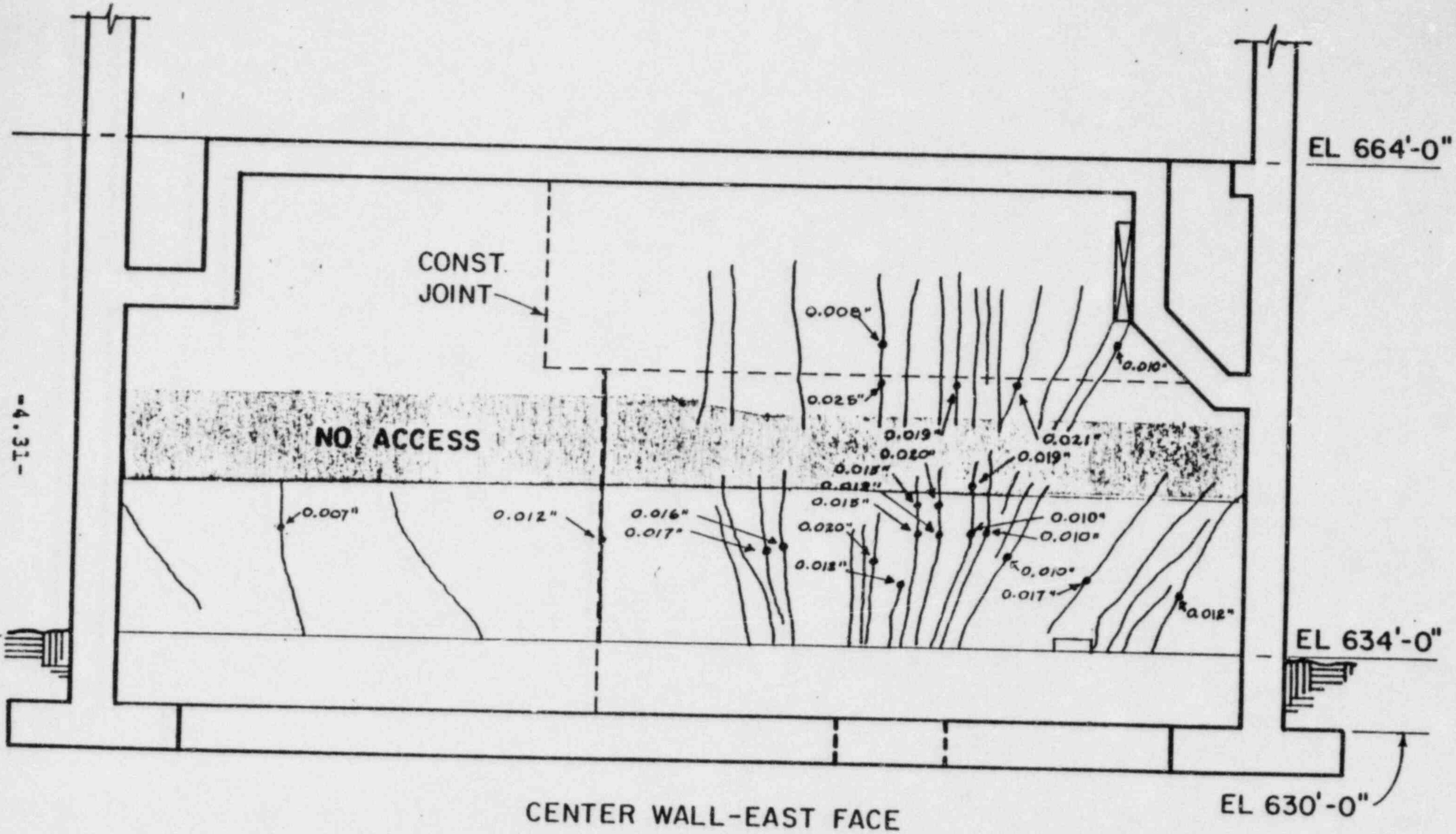


Fig. 4.21 Cracks Observed in Center Wall - East Face of Diesel Generator Building on February 9, 1982

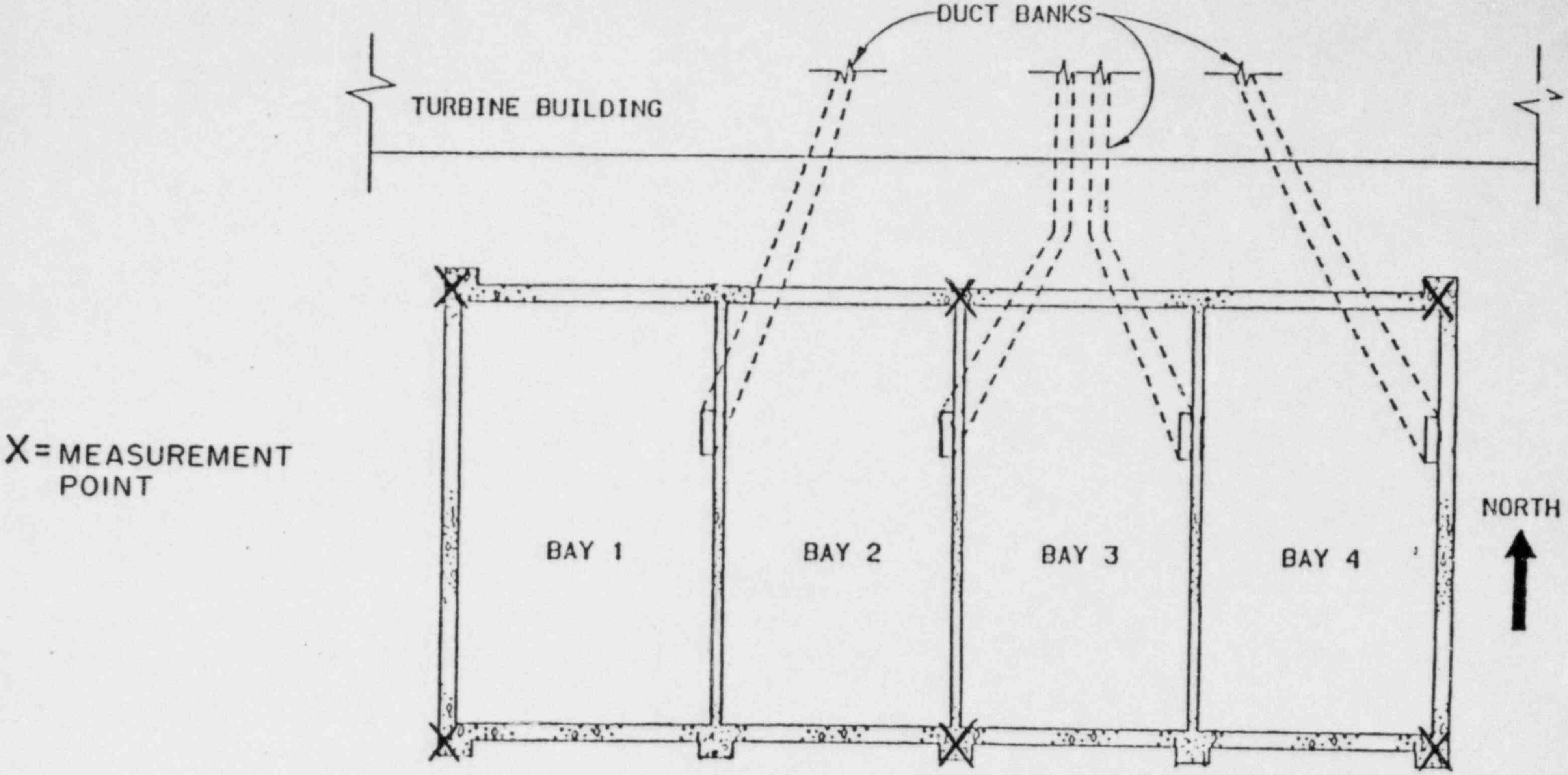


Fig. 4.22 Diesel Generator Building Showing Approximate Locations of Displacement Measurement Points

displacements are of significance with regard to structural integrity of the building.

Crack Monitoring

As a supplement to the displacement monitoring program, periodic visual inspections of the Diesel Generator Building should be made to determine if new cracking has developed or if existing cracks have changed in width or length. Crack inspections should be conducted by qualified personnel.

Because the Diesel Generator Building is not being underpinned, it is not anticipated that the crack monitoring program will be as rigorous as that for the Auxiliary Building. However, as a minimum, the following steps should be included. Initially a crack survey should be made for the entire structure. This will provide a base for future evaluation of changes in crack patterns or crack widths. All visible cracks should be marked and recorded. Selected cracks should be measured to obtain an estimate of maximum crack widths.

If displacement measurements indicate that building settlement exceeds the predicted values, cracks in the structure should be remapped. Within four weeks after observation of the cracks, an engineer familiar with reinforced concrete behavior and design should provide a written report that describes significance of observed cracks and recommendations for maintaining structural integrity of the building.

R. Landsman
RGN III

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

DEC 28 1981

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Docket Nos.: 50-329/330 OM, OL

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Raod
Jackson, Michigan 49201

Dear Mr. Cook:

Subject: Announcement of Geotechnical Engineers Inc. as NRC Staff Consultant for Underpinning of Auxiliary Building Area and Service Water Pump Structure

The NRC Staff's review of the geotechnical engineering aspects of the underpinning of the Auxiliary Building and Service Water Pump Structure for Midland Plant, Units 1 and 2 is being performed with the contractual assistance of:

Geotechnical Engineers Inc.
ATTN: Dr. Steve J. Poulos
1017 Main Street
Winchester, Massachusetts 01890

The principal investigator and Vice President of Geotechnical Engineers Inc., Dr. S. Poulos, is also being assisted by Mr. Reuben Samuels, Vice President of Crimmins Contracting Company in New York. This team adds extensive underpinning expertise to the NRC's geotechnical review of Midland and is in addition to our continuing contract with the U.S. Army Corps of Engineers. The NRC's technical coordinator for this additional contract will also be Mr. Joseph Kane.

We request that Geotechnical Engineers Inc. be added to your mailing service list for all technical documents, drawings or other correspondence dealing with the underpinning of the Midland Auxiliary Building, Feedwater Isolation Valve Pits, and the Service Water Pump Structure. We understand that Mr. Kane has made verbal requests regarding the transmittal of certain existing documents to Geotechnical Engineers Inc. and plans further discussions with your staff to this end.

We believe this addition will provide NRC with the increased expertise and experience needed to review Consumer's pending underpinning submittals

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in a timely and effective manner. Your prompt attention in forwarding information to our consultants is appreciated.

Sincerely,

Calvin W. Moon
for
Elinor G. Adensam, Chief
Licensing Branch #4
Division of Licensing

cc: See next page

MIDLAND

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Landsman

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/D		SLO	
✓ PI			
DE&TI			
DEP&OS		File	

DEC 3 1981

Docket Nos: 50-329
and 50-330

APPLICANT: CONSUMERS POWER COMPANY

FACILITY: Midland Plant, Units 1 and 2

SUBJECT: SUMMARY OF MEETING TO DISCUSS REMEDIAL PLANS FOR AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PIT FOUNDATIONS

On November 4, 1981, the NRC staff and their consultants met in Bethesda with Consumers Power Company (CPC) representatives and their consultants to discuss remedial plans for auxiliary building and feedwater isolation valve pit foundations. A list of attendees is attached as Enclosure 1 and the meeting agenda is attached as Enclosure 2. The following provides a summary of the meeting.

E. Adensam stated that the Midland project manager and his backup were not available, and therefore, K. Jabbour would coordinate the meeting. OELD stated that the hearing testimony for Midland should be in the mail by November 17, 1981. Discussion of the seismic model is scheduled for December 14 - 18, 1981. It is expected that, during the hearings, the NRC staff will inform the Licensing Board on areas of agreement between Consumers and the staff.

CPC stated that they started procurement for freeze wall hardware and access shaft. They invited the NRC staff to visit two work sites in Philadelphia and Louisiana where freeze wall technology is applied. A schedule of CPC work progress is provided as Enclosure 3.

Representatives of Mergentime and Ground Water Technology, Inc., discussed their plans for the Midland site, the freezing and grouting operations, and their experience in this area. They provided sketches of the access shaft, frozen earth membrane, proposed freeze wall locations, typical freeze element, and typical pressure and temperature monitor location. The sketches are attached as Enclosure 4. They also stated that there is no problem with frost heaving and committed to produce data on heaving.

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III

Following the presentation above, the attendees discussed the staff questions as stated in Enclosures 5 and 6. The NRC Structural Engineering Branch offered to provide their questions to Consumers on November 5, 1981. At the conclusion of the meeting, Consumers committed to provide written responses to the questions in Enclosure 5. These responses were provided in a letter from CPC to H. R. Denton dated November 16, 1981.

Kahtan N. Jabbour

Kahtan Jabbour, Project Manager
Licensing Branch No. 4
Division of Licensing

Enclosures:
As stated

cc: See next page

MIDLAND

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Mr. I. Charak, Manager
NRC Assistance Project
Argonne National Laboratory
9700 South Cass Avenue
Argonne, Illinois 60439

List of Attendees

November 4, 1981

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E. Adensam*
J. Kane
A. Hodgdon
W. Paton*
F. Rinaldi
G. Lear
F. Schauer*
M. Blume*

NRC Consultants

H. Singh
J. Matra

Consumers Power Company

K. Razdan
G. Keely
N. Ramanujam

Bechtel

B. Dhar
S. Afifi
N. Swanberg

Hanson Engineers

D. Bartlett

IL&B

F. Williams

Mergentime

C. Gould

Ground Water Tech. Inc.

D. Maishman

Mueser Rutledge

J. Gould

*Denotes part-time participation

Enclosure 2

November 4, 1981

Meeting with NRC Staff
on Midland Plant Auxiliary Building

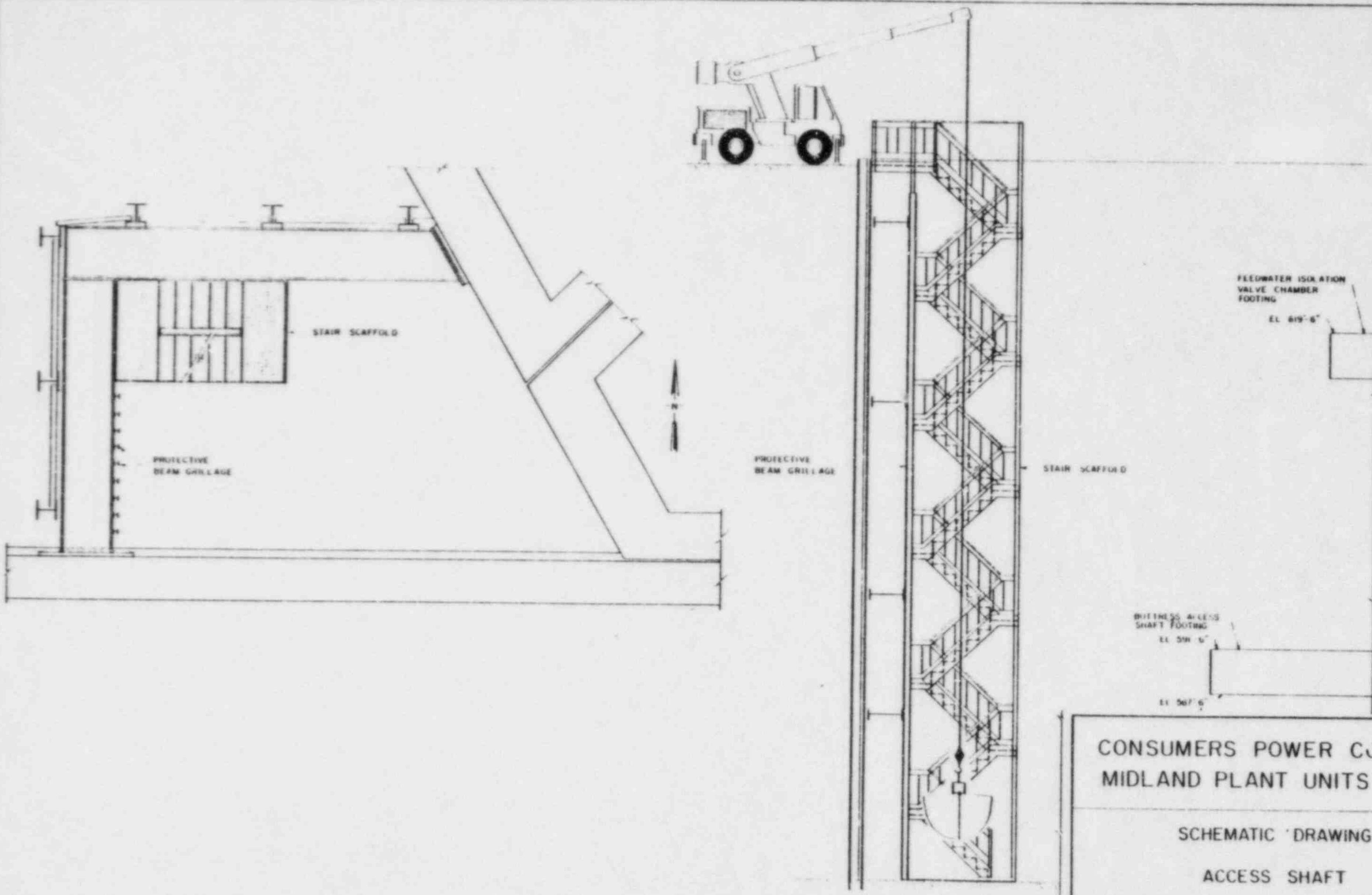
1. Access Shaft
2. Freeze Wall
3. Discussion of questions on 9/30/81 submittal on technical report.

Enclosure 3

MILESTONES FOR AUXILIARY BUILDING
AND FEEDWATER ISOLATION VALVE
PIT UNDERPINNING

ITEM	START DATE
1. Procurement of Freezewall Hardware	In Process
2. Award of subcontract for Underpinning (three phases of work)	12/15/81
3. Start installation of Freezewall (Phase 1 of subcontract)	12/29/81
4. Mobilize and start installation of access shaft to el. 609 feet (Phase 2 of subcontract)	1/15/82
5. Complete structural analysis for construction underpinning	1/1/82
6. Award of subcontract for Instrumentation (Design, furnish, install and monitor)	12/1/81
7. Start drifting for Underpinning (Phase 3 of subcontract)	2/15/82
8. Drill and develop additional 44 permanent wells	In Process
9. Start Recharge Test	11/25/81
10. Structural Acceptance Criteria for long term settlement	2/15/82

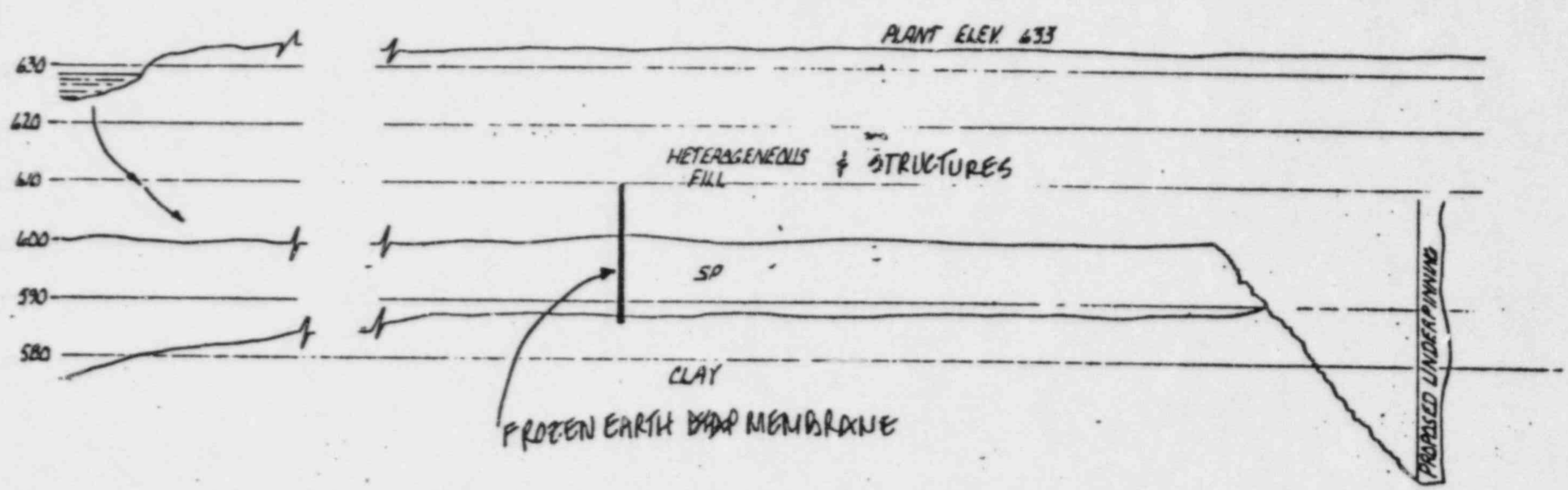
Enclosure 4

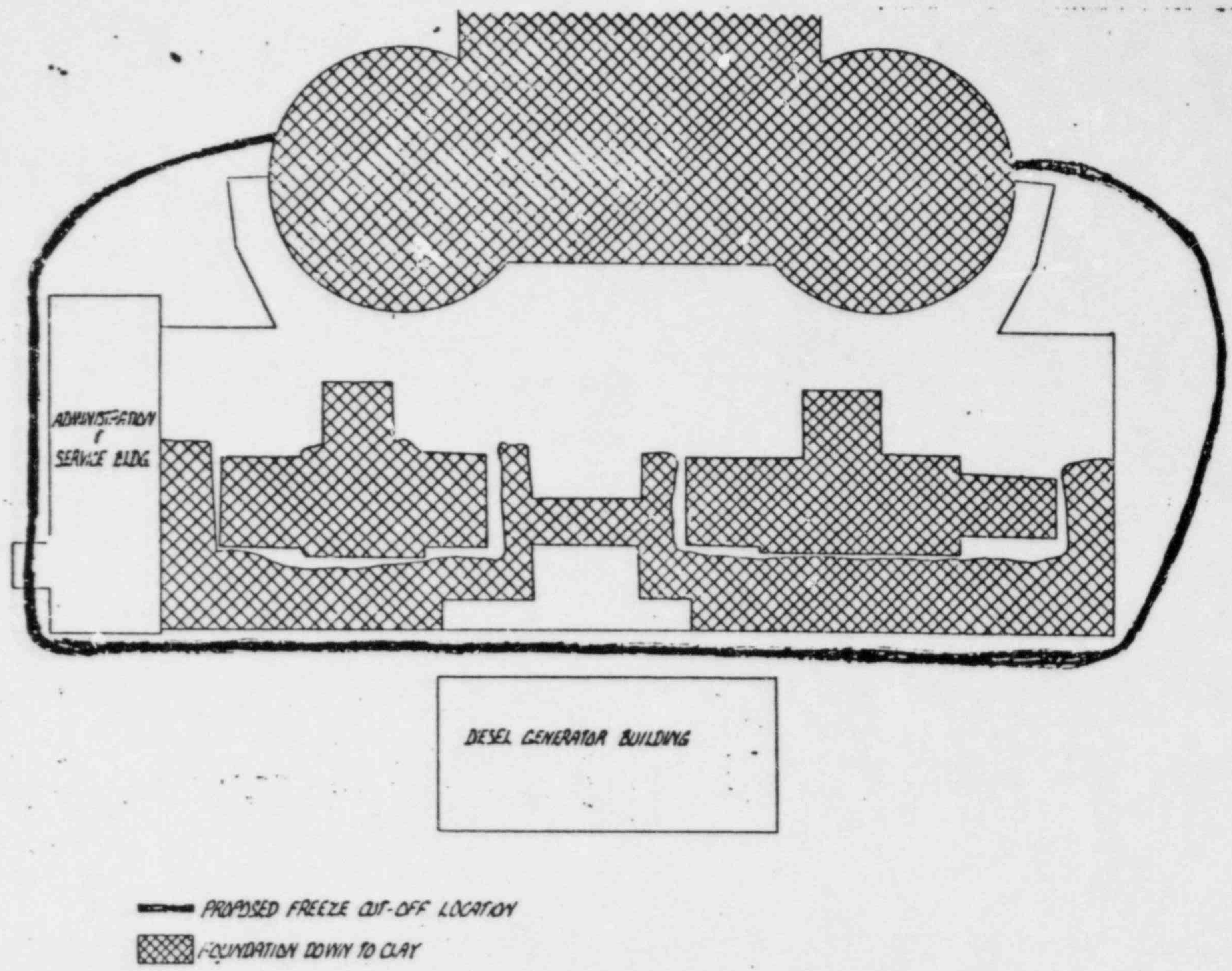


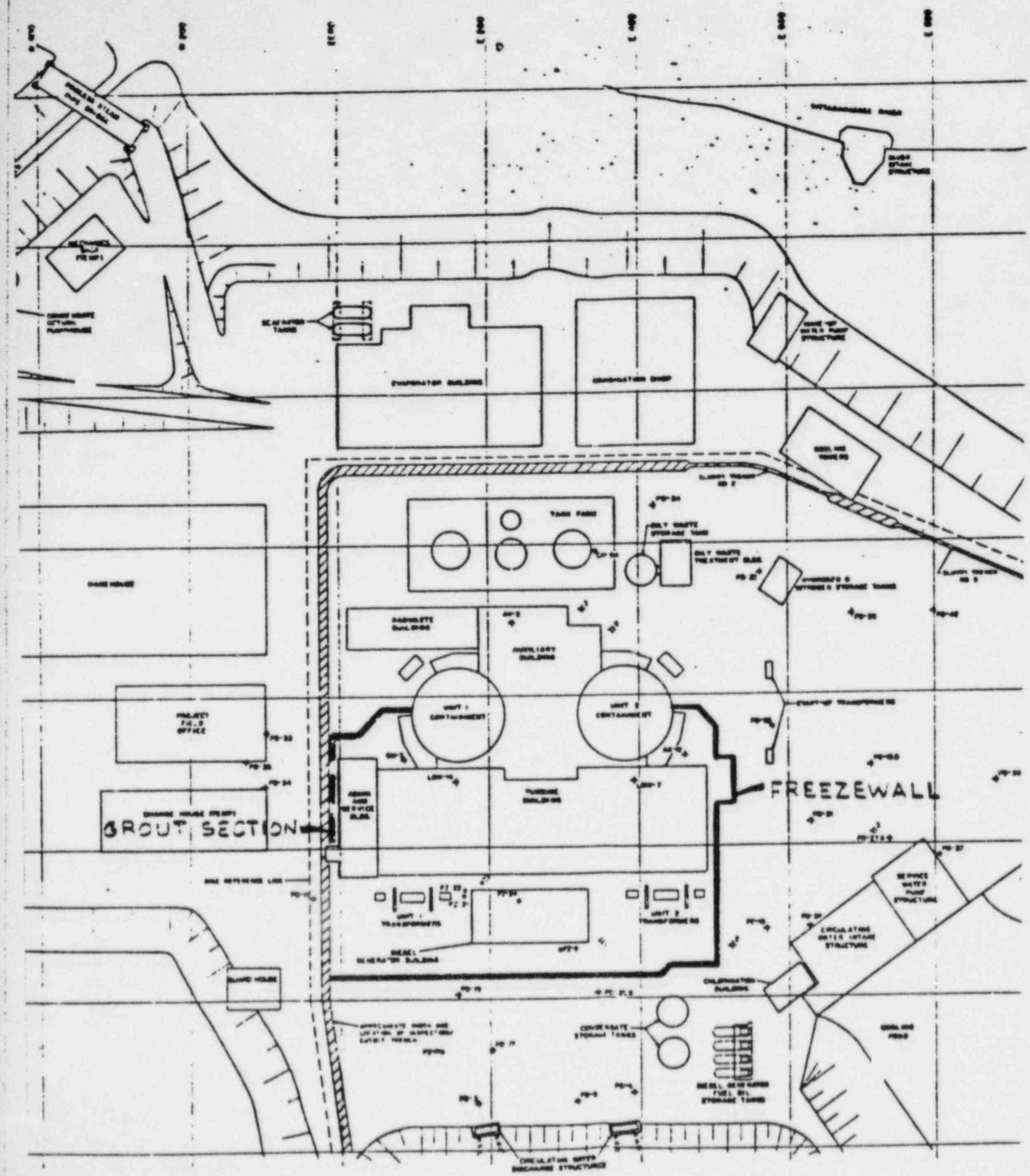
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

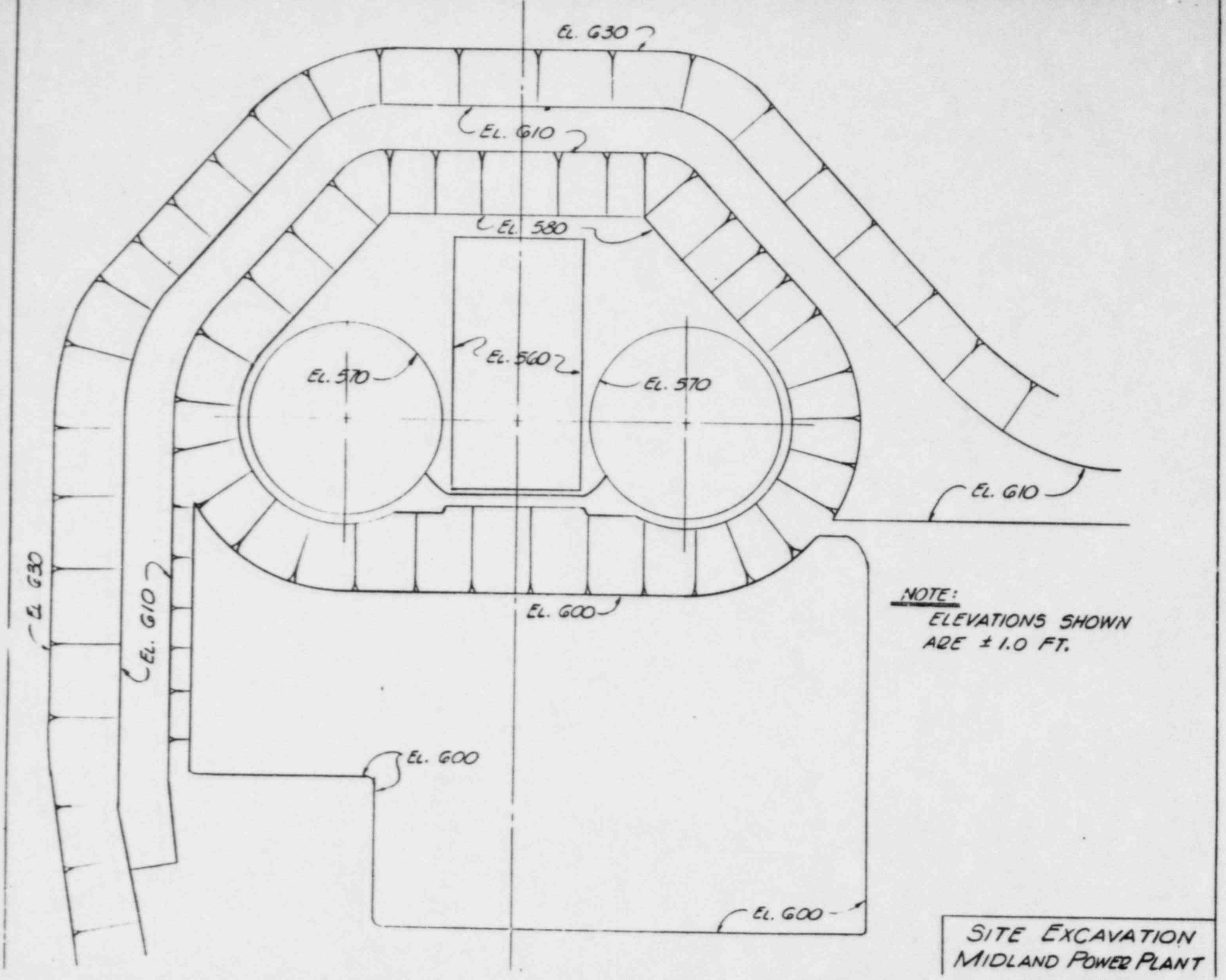
SCHEMATIC DRAWING

ACCESS SHAFT

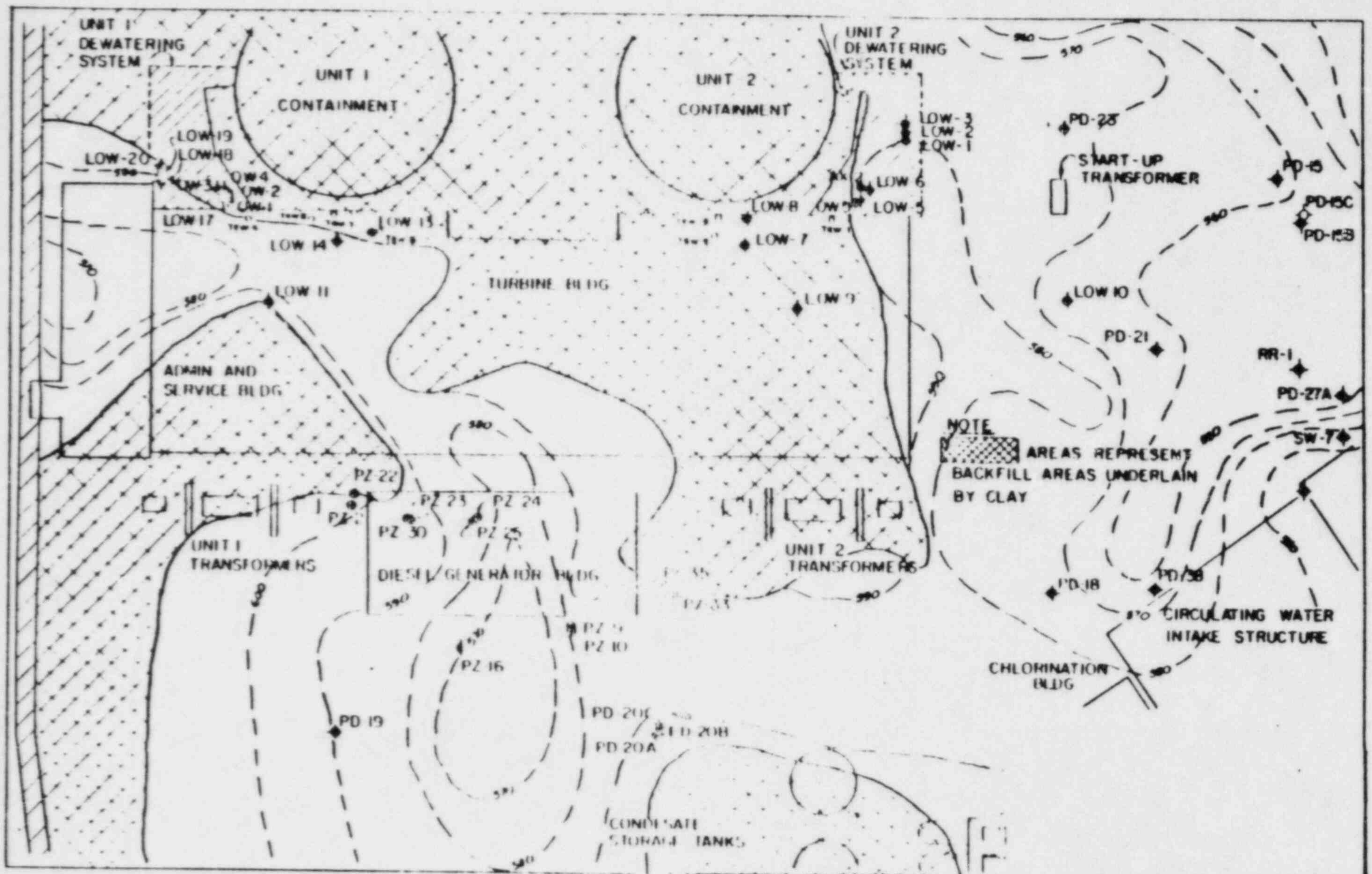

PLANT ELEV. 633
630
620
610
600
590
580
HETERAGENEOUS FILL & STRUCTURES
SP
CLAY
FROZEN EARTH MEMBRANE
PROPOSED UNDERPINNING


ADMINISTRATION & SERVICE BLDG.
DIESEL GENERATOR BUILDING
PROPOSED FREEZE CUT-OFF LOCATION
FOUNDATION DOWN TO CLAY

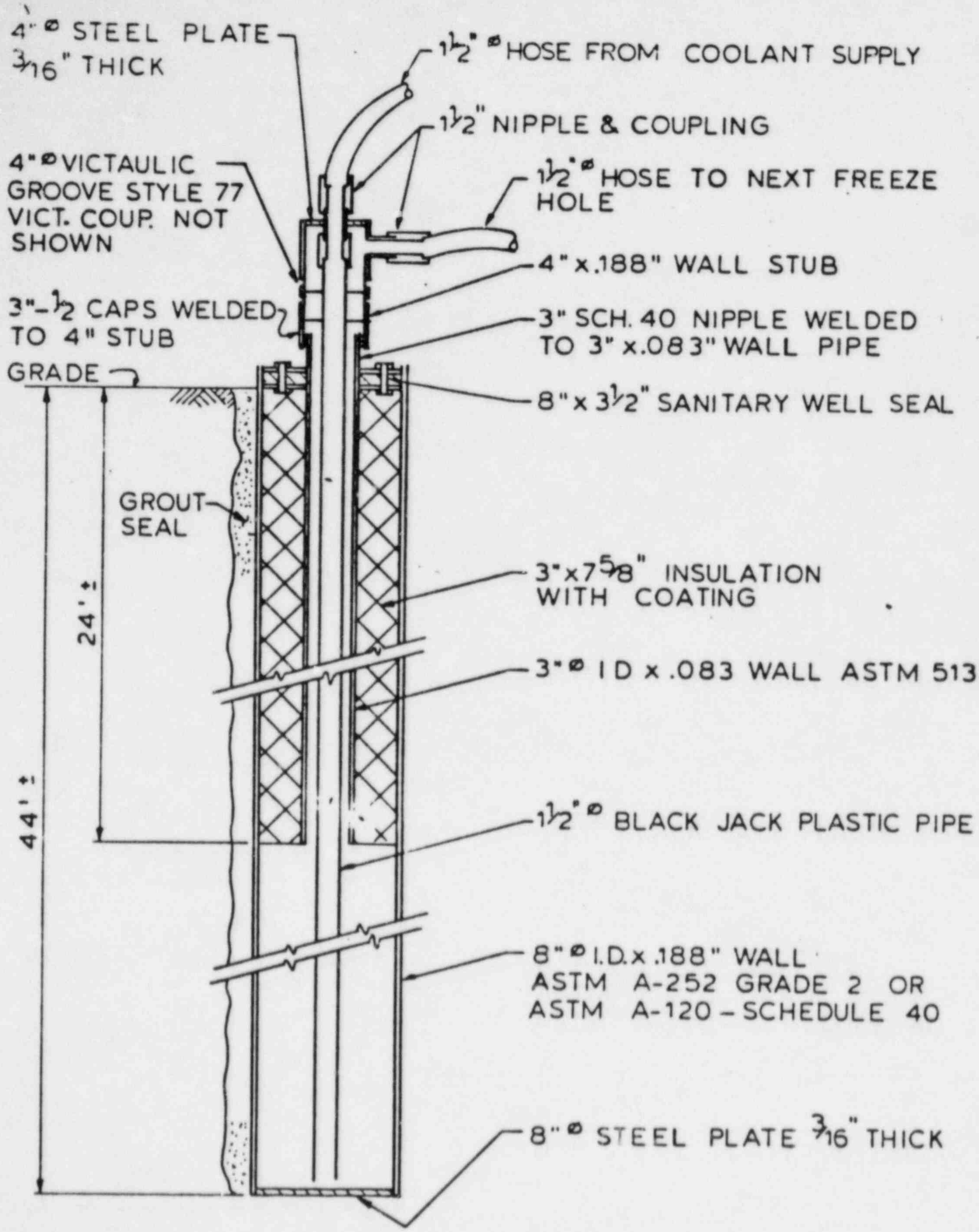

FREEZEWALL
GROUT SECTION



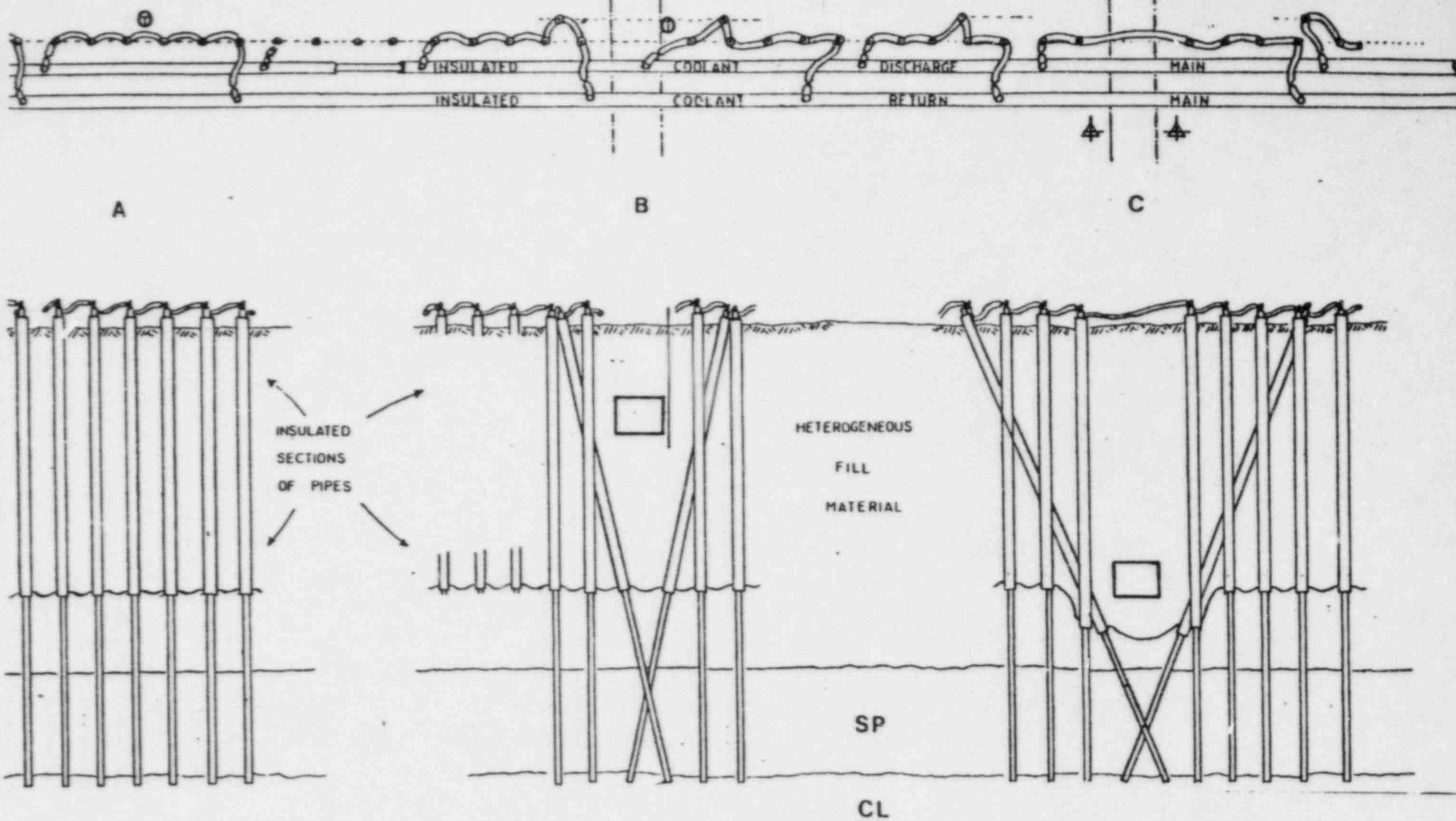
SITE EXCAVATION
MIDLAND POWER PLANT



UNIT 1 DEWATERING SYSTEM
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
UNIT 2 DEWATERING SYSTEM
LOW-3
LOW-2
LOW-1
PD-23
START-UP TRANSFORMER
PD-15
PD-15C
PD-15B
LOW-20
LOW-19
LOW-18
LOW 17
LOW 14
LOW 13
LOW-8
LOW-7
LOW 6
LOW-5
TURBINE BLDG
LOW 11
LOW 9
LOW 10
PD-21
RR-1
PD-27A
SW-7
ADMIN AND SERVICE BLDG
NOTE
AREAS REPRESENT BACKFILL AREAS UNDERLAIN BY CLAY
PZ-22
PZ 23
PZ 24
PZ 30
PZ 25
UNIT 1 TRANSFORMERS
DIESEL GENERATOR BLDG
UNIT 2 TRANSFORMERS
PD 18
PD 38
CIRCULATING WATER INTAKE STRUCTURE
PZ 9
PZ 10
PZ 16
CHLORINATION BLDG
PD 19
PD 20B
PD 20A
CONDENSATE STORAGE TANKS



DETAIL OF TYPICAL FREEZE ELEMENT



A - FREEZE ELEMENTS IN TYPICAL CUT-OFF WALL

B - WALL INSTALLED BELOW HIGH LEVEL UTILITY

C - LOW LEVEL UTILITY IN MONITORED 'WINDOW'

Ⓣ TEMPERATURE MONITOR

HIGH AND LOW LEVEL PIEZOMETERS

Enclosure 5

RECORD OF TELEPHONE CONVERSATIONS

Date: October 30, 1981 Project: Midland 50-330

Recorded by: Joseph D. Kane

Talked With:

CPCo	Bechtel	NRC	COE
D. Budzik	A. Boos	R. Landsman	H. Singh
G. Keeley	N. Swanberg	F. Rinaldi	
		D. Hood	
		J. Kane	

Route To: For Information

G. Lear
L. Heller
D. Hood
W. Paton
F. Rinaldi
R. Landsman, I&E, Region III
H. Singh, COE, Chicago
J. Kane

Main Subject of Call: Remedial Underpinning of Auxiliary Building and Feedwater Isolation Valve Pits

Items Discussed:

1. Enclosure 3 to CPCo September 30, 1981 submittal from J. W. Cook to H. R. Denton entitled "Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits". During the October 30, 1981 conference call CPCo was requested to respond to the following questions which had been developed in the COE/NRC review of Enclosure 3, relative to geotechnical engineering aspects in underpinning the Auxiliary Building.

   Q.1. (Pg. 2, Sect. 4, 2nd Para.) Please define "design jacking force," how established and the duration that it will be held?

   Q.2. (Pg. 2, Sect. 4, 3rd Para.) Discuss and provide detail of dowel connection. (Diameter, how distributed along wall, length of embedment, etc).

   Q.3. (Pg. 3, Sect. 5.1, last para) The agreed upon acceptance criteria for soil particle monitoring during dewatering requires 0.005 mm and not 0.05 mm. Correction by CPCo required.

Q.4. (Pg. 3, Sect. 5.1, Para. b) Installing the frozen cutoff membrane will cause expansion and possibly increase the soil voids. When ultimately unfrozen, what is the effect (e.g., further settlement) on safety related structures, conduits and piping. Provide discussion on the basic system of the frozen membrane [size and spacing of holes to be drilled, method for pumping brine into foundation layers, range of temperatures that are critical to wall stability which are to be monitored, decomissioning (e.g., grouting, etc)].

Q.5. (Pg. 3, Sect. 5.2) Clarify the procedure to be used in post tensioning the Electrical Penetration Area. Where will the buoyancy force be transmitted to the foundation and in what manner?

Q.6. (Pg. 4, Sect. 5.6, 2nd Para.) Please explain the meaning of "failure bearing capacity factors" and the basis for "the nine times the shear strength for the cone"?

Q.7. (Pg. 4, Sect. 5.b, 4th Para.) How will the equivalent soil modulus be determined? What is the depth that the measured settlement will be distributed over and what is the area to be used in determining the stress?

Q.8. (Pg. 4, Sect. 6) Presently, this paragraph implies that crack monitoring will not be performed on the existing structure. Please correct. Before remedial underpinning begins an accurate and up-to-date record of cracks should be developed for those safety related structures which could potentially be affected by the underpinning operations. This background record should be verified by I&E inspection and could serve as the basis for evaluating any changes in cracks due to underpinning operations.

Q.9. (Pg. 5, Sect 6.1.1 and 6.1.2) When will the acceptance criteria for the differential and absolute settlement be provided to the NRC?

Q.10. (Pg. 5, Sect. 6.2) Provide the basis for establishing the crack width of 0.03 inch. Appendix D should also address crack monitoring requirements during underpinning (frequency of reading, format for presenting observations, action levels etc).

Q.11. (Pg. 6, Sect. 7.2.1, last Para.) Provide discussion why the drained shear strength is not required to be considered in analyzing for adequate bearing capacity. Also in the last paragraph in Section 7.2.1, Pg. 7 indicate the basis for the 2 days and what would be required if the settlement rate does not reach a straight line trend in 2 days.

Q.12. (Pg. 7, Sect. 7.2.2) Where are the WCC controlled rebound-reload cycle soil test results? What is the corresponding stress level with a secant modulus of elasticity equal to 3500 KSF?

Q.13. (Pg. 8, Sect 7.2.3, 1st Para.) The estimates of settlement using the referenced NAVFAC DM-7 do not include secondary consolidation. What secondary consolidation would be indicated if the consolidation test results using the appropriate load increment were used? Compare this estimate with valves for permanent wall conditions "after jacking, long term". Please provide basis for the three estimated settlement valves for "Load transfer points for temporary load to reactor footing" at the bottom of pg. 8 and discuss any effects of this settlement on the reactor and pipe connections.

Q.14. (Pg A-1, Sect. 1, 2nd Par.) Please indicate how the soil spring constants were established for long term loads.

Q.15. (Pg C-2, last Par. and Pg. C-6, Par. B) What are the protective construction measures planned for the Turbine Building and Buttress Access Shafts and when will they be placed? Please provide discussion on the sequence of operations to complete the drift beneath the Turbine Building and show sectional views of this work with respect to the Turbine Building foundations and affected piping and conduits.

Q.16. (Pg C-3, Par. A.1.a) Please explain what is meant by minimizing the amount of concrete to be removed.

Q.17. (Pg. C-3, Par. A.1.c. and A.1.d) What is the magnitude of the load for testing the temporary support pier and how was it established and how will it be applied? Is the EPA foundation slab capable of supporting this load at this time?

Q.18. (Pg. C-4, Sect. A.1.f., 1st complete para.) Provide discussion on monitoring of the control tower behavior at this time. What criteria will be used to decide if preload should be stopped and support capacity should be added to the control tower?

Q.19. (Pg. C-4, Sect. A.2.) What are the reasons why the three temporary supports under the EPA should not be completed before the permanent support at the control tower is initiated?

Q.20. (Pg. C-4, Sect. A.3.a) Questions are raised as to whether the EPA structure can withstand the overhang condition which results if the initial temporary supports is assumed to fail. What is the basis and need for this extreme assumption? Is the EPA structure capable of withstanding this loading condition?

Q.21. (Pg. C-4, Sect A.3.b and A.3.c) The distinction between 3.b and 3.c is unclear. What is the magnitude of the load for testing and how established? Is there a problem with the EPA foundation slab providing a sufficient reaction load?

Q.22. (Pg. C-5, Sect. 14 and 15) It appears the operations described in these items are intended only for the wings and not the control tower. How is the load test and load transfer for the control tower to be completed. For the long term load test on the wings, what is the load magnitude and how was it established? What is the final sequence of operations in transferring the structure load to the permanent underpinning.

Q.23. (Pg. D-1, Sect 1.0, 2nd Par) Describe the procedure that relates allowable stresses and allowable strains with structure movements that are being monitored.

Q.24. (Pg D-2, Sect. 1, 3rd Par.) Please clarify the distinction between the first and second layer systems for detecting structure movement.

Q.25. (Pg D-2, Sect. 1, 4th, 6th, and 7th Para.) Please provide elevations and sectional views with typical details for the deep seated bench mark and the instrumentation for monitoring relative horizontal movement and absolute horizontal movement.

Q.26. (Pg. D.3, Sect. 2, 2nd Par.) Please clarify the explanation why the hydraulic pressure data cannot be used to measure load.

Q.27. (Pg. D-3, Sect. 2, 3rd Par.) Provide sectional view of set up for measuring difference in relative position. How does this procedure address the possibility of both the underpinning element and structure settling? Provide the basis for maintaining the jack/hydraulic system for 1 hour and for establishing the 0.01 inch movement.

Q.28. (Pg. D-4, Sect. 2, 4th Para.) When will the modeling and critical structural stresses and strains be determined and furnished to the NRC?

Q.29. (Pg D-5, Sect. 2, 2nd and 3rd Para.) Provide sketch and locations with typical details of instrumentation for measuring concrete stress, tell tale devices and predetermined points for monitoring vertical movement.

Q.30. (Pgs. D-5 and D-6, Sect. 3, Par. 3A.1, 3A.2, 3A.3) For the various types of monitoring described in these paragraphs provide an example of the forms to be used for plotting the recorded data. What are the predetermined levels of movements which would require adjustments and/or action by the onsite geotechnical engineer. Identify any specific instrumentation which would be continued to be read during plant operation and which eventually will be addressed by a Technical Specification.

2. Consumers was notified that the above questions do not contain the COE/NRC review comments on the laboratory test results for foundation soils beneath the Auxiliary Building. The COE/NRC comments on the test results will be furnished at a later date following CPCo submittal of the Part II lab test report which is expected to be submitted to the NRC the week of November 2, 1981.

3. Consumers indicated the questions asked in the conference call of October 30, 1981 would be addressed as far as possible in the upcoming meeting with NRC in Bethesda on November 4, 1981.

Staff Questions from 10/30/81 Telecon

1.	<u>Paragraph 4.0, page 2</u>	What is design jacking force; how established; how long held?
2.		What are details of dowels; dia., spacing, and embedment length?
3.	<u>Para 5.1, page 3</u>	Shouldn't 0.05 be <u>0.005</u>?
		What are consequences of settling of structure in region of freezewall when it is "thawed"?
		Basic description of system, e.g. layout, materials, temperatures, decommissioning.
4.	<u>Para 5.2, page 3</u>	Where will the buoyancy forces be transmitted to structure?
5.	<u>Para 5.6, page 4</u>	Define failure bearing capacity and how was value of 9 established.
		How will equivalent soils modulus be computed? At what depth will equivalent strain be calculated and what is corresponding stress at that level?
6.	<u>Para 6.0, page 4</u>	What is date for last auxiliary building crack mapping?
		What are the plans for crack monitoring during construction and will be establish a baseline?
		How are we going to monitor cracks in inaccessible areas?
	<u>Para 6.1.1, page 5</u>	When will the program for differential and absolute settlement of structures be established including acceptance criteria?
	<u>Para 6.1.2, page 5</u>	When will the program for monitoring underpinning during jacking be established including acceptance criteria?
	<u>Para 6.2, page 5</u>	Justify crack widths stated.
	<u>Para 7.2.1, page 6</u>	Justify why drained shear strengths were not used to determine bearing capacity.
	<u>Para 7.2.1, page 7</u>	What are the plans if rate doesn't reach a straight line after 2 days?

7. Para 7.2.2, page 7 — Where is cyclic testing reported?

   How was the modulus of 3500 ksf obtained?

8. Para 7.2.3, page 8 — What settlement is to be attributed to secondary consolidation (NAVAC reference is elastic; it does not cover effects of secondary consolidation)?

   How were settlements after jacking values given in table determined?

   How were settlement values during temporary loading on reactor buliding estimated?

   What is effect on reactor building and pipe connections?

9. Appendix A, Para 1.0, page A-1 — How were static long-term springs established?

10. Appendix C, Last para, page C-2 — What are protective construction details; where support placed; when installed?

    What about details of turbine building underpinning and its effect on buried Category I utilities in this area?

11. Page C-3 — Discuss turbine building underpinning.

12. Item 1-a — What is meant by "minimizing" concrete removal?

13. Item 1-c — Give details of load test (what is load; how arrived at; and how applied).

14. Item 1-d — Justify your statement about building performance as propped cantilever.

15. Page C-4, Item 1-f — What are we doing to monitor performance of control tower? What are the criteria and if a problem occurs, then what action is taken?

16. Item 2 — Rationale behind not completing all 3 needle beams on electrical penetration area before starting pit control tower area.

17. Item 3 — Can electrical penetration area support an assumed failure of the end beam?

    Give details of test load and relate it to the design load.

    What are differences between 3b and 3c?

18.	<u>Page C-5</u> <u>Item 4</u>	What is load test and load transfer program for control tower?
19.	<u>Item 14</u>	What is the load, how established, settlement acceptance criteria?
20.	<u>Appendix D</u> <u>Page D-1, 2nd para</u>	State program for correlating allowable strains and stresses.
21.	<u>Page D-2</u>	Discuss first layer and second layer movement monitoring. Give details of deep benchmark datum. Provide details of horizontal movement monitoring.
22.	<u>Page D-3, para 2.0</u>	Need better definition of hydraulic jacking program. Want sketch of setup for overall (building and underpinning) settling monitoring setup. What is basis for 1 hour and 0.01 inch?
23.	<u>Page D-4</u>	How will stress and strain be correlated?
24.	<u>Page D-5</u>	Give details on telltale setup and Carlson stress meters. Give details of settling monitoring points at end of electrical penetration area.
25.	<u>Para 3.0 (A)</u>	For each of 3A1, 2, 3, indicate: Data to be taken, what are predetermined allowable limits, how these limits are established, and action to be taken if these limits are reached. Which measurements will be included in technical specs?

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

December 3, 1981

PRINCIPAL STAFF			
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A/D		SLO	
DR&PI			
DE&TI			
DEP&OS		File	

Landsman

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
UNDERPINNING OF THE AUXILIARY BUILDING - CALCULATIONAL RESULTS
FILE 0485.16, B3.0.1 SERIAL 14899
REFERENCE: JWCOOK TO HRDENTON, SERIAL 14110, DATED SEPTEMBER 30, 1981
ENCLOSURE: ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

Attached to the above-referenced correspondence of September 30, 1981, we submitted a design report entitled, "Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits." We are providing as an enclosure to this correspondence twenty-five (25) copies of an addendum to the above-referenced technical report.

The purpose of the enclosed addendum is to supplement Section 7.5 of the above-referenced technical report and Appendix A of the same document. The enclosed addendum contains the following information:

1. Soil pressure data under the auxiliary building and the feedwater isolation valve pits underpinning area.

2. Load combinations used for preliminary design of the underpinning reinforcement walls and the connection joints of the underpinning walls to the auxiliary building.

3. Design forces and moments at the critical sections.

4. Reinforcement details provided in the underpinning walls.

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5. A summary of results from recent preliminary auxiliary building structural analyses which reflect the modified dynamic model of the structure, actual natural soils properties and the proposed underpinnings. These results identify certain areas within the structure which may require some modification in order to meet design requirements. As further analyses are completed, we will forward our proposed plans for any additional remedial actions to the Staff for their review and concurrence.

The material presented in this addendum is based on preliminary analyses of the permanent underpinning configuration. Detailed calculational checks will be performed as a part of the final analysis to verify the design adequacy. We are also currently performing analyses and design checks for the auxiliary building construction condition for various construction stages. The results of these detailed design checks for both the permanent underpinning configuration and the construction condition will be available to the NRC Staff for their audit in accordance with agreements reached at our November 17, 1981 meeting in Bethesda.

This addendum along with our previous submittals and discussions with the NRC Staff should adequately respond to the concerns identified by the Staff. We believe this information continues to support our conclusion that the design of the auxiliary building and feedwater isolation valve pit structures combined with the proposed underpinning remedial actions are adequate and appropriate for these structures.

James W. Cook

JWC/WJC/RLT/dsb

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, Esq, w/o
JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

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ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

DECEMBER 2, 1981

MIDLAND PLANT UNITS 1 AND 2
ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE
AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

CONTENTS

MIDLAND PLANT UNITS 1 AND 2
ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE
AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

1.0 INTRODUCTION

The purpose of this addendum is to supplement Section 7.5 of the Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits (Reference 1) with the following information:

a. Soil pressure data under the auxiliary building, feedwater isolation valve pits (FIVPs), and auxiliary building underpinning

b. Load combinations used for preliminary design of the underpinning reinforcement and the connection of the underpinning to the auxiliary building

c. Design forces and moments at the design sections

d. Reinforcement provided in the underpinning walls

e. Identification of the areas of potential overstress in the auxiliary building as indicated by the preliminary analysis

The material presented herein is based on preliminary analyses and design for the permanent underpinned configuration of the auxiliary building and the FIVPs. Detailed checking will be performed after final analysis to verify the design adequacy. The results of this detailed check will be provided later in an audit scheduled for May 17, 1982.

The results of the analysis for the construction condition with temporary support piers are not included. This analysis is in progress and the results will be provided later for the audit scheduled January 15, 1982.

2.0 SOIL PRESSURES

2.1 AUXILIARY BUILDING UNDERPINNING

Table 1 and Figure 1 show the magnitude and location of the net soil pressure under the main auxiliary building and underpinning under the control tower and the electrical penetration area. The soil pressures were computed for the following load combination considered to be critical for preliminary analysis.

$$D + L + R + E' + P_L$$

where

D = dead load

L = live load

R = pipe break load

E' = safe shutdown earthquake (SSE) loads corresponding to the ground acceleration given in the Midland FSAR Section 3.7

This load combination corresponds to the 19th load combination in Table 1 of Reference 1 without the thermal loads which are neglected in the preliminary design.

The allowable net bearing pressure is based on the allowable values submitted to the NRC in Subsection 7.2.1 of Reference 1 and Midland FSAR Section 2.5.

2.2 FEEDWATER ISOLATION VALVE PITS

The FIVPs will be supported on engineered sand backfill. A 3-foot thick concrete slab will be provided between the bottom of the pit and the top of the sand, as shown in Figure 2. The sand will be confined between the reactor building, electrical penetration area underpinning wall, turbine building underpinning, and buttress access shaft. The slab at the top of the engineered backfill will be jacked against the existing FIVP base slab. This jacking will minimize any future settlement due to compaction of the engineered backfill from the weight of the FIVP. After jacking, the space between the 3-foot slab and the bottom of the pit will be filled with concrete grout. The maximum bearing pressures on the engineered backfill are shown in Table 2.

The soil pressures (shown in Table 2) were computed for the following critical load combination considered in the preliminary analysis:

$$D + L + E' + P_L$$

This load combination corresponds to the 19th load combination in Table 1 of Reference 1 without the thermal loads which are neglected in the preliminary design.

3.0 UNDERPINNING WALL DESIGN

3.1 LOADS

The preliminary wall design is based on the following loads and load combinations:

a. $U = 1.4D + 1.7L + P_L$ (corresponds to the fifth case in Table 1 of Reference 1)

b. $U = D + L + R + 1.5E' + P_L$

For the above load combinations, the following loads have been considered:

a. Dead load - Includes soil pressure loads.

b. Jacking load applied as uniform load along the length of the underpinning

c. Live load

d. Seismic loads

e. Pipe break loads

3.2 UNDERPINNING BELOW THE ELECTRICAL PENETRATION AREA

The underpinning wall under the electrical penetration areas will carry the vertical loads which will be transferred to clay till at el 571'. The walls will also carry lateral loads due to seismic forces, soil pressure, and surcharge from the turbine building. These lateral loads will be resisted by the engineered sand backfill placed between the underpinning wall and the reactor building, as shown in Figure 4, and the friction between the concrete wall and the soil underneath (clay till). The net lateral loads in the second load combination exceed the available friction between the wall and soil. For this reason, an 11-foot wide, horizontal beam has been provided to resist the bending due to the net lateral loads (Figure 4).

The critical section for the wall is near column lines 5.3 and 7.8 (see Figure 3). The design forces are shown in Table 3 and reinforcement is presented in Figure 3.

3.3 UNDERPINNING BELOW THE CONTROL TOWER

The underpinning wall will be embedded in natural clay till between elevations 571 and 562, and will be restrained by a new slab at el 583'-6" to be constructed as shown in Figure 4. The space between el 571' and the slab at el 583'-6" will be backfilled with engineered granular material. Part of the lateral loads will be resisted by the clay till between elevations 571 and 562, and the balance will be transferred to the main building by the slab at el 583'-6".

The critical section for the wall is at column line 7.8. The location of the critical sections and reinforcement are presented in Figure 3. Design loads at the critical section are presented in Table 3.

4.0 STABILITY

The factors of safety against sliding and overturning are shown in Subsection 3.8.6.3.4 of the Midland FSAR (Reference 2). In the underpinned condition, the overall safety factors against sliding and overturning are expected to reduce or remain unchanged from the values shown in the Midland FSAR.

5.0 CONNECTION DETAIL

The connection of the underpinning to the auxiliary building will be designed to transfer shear and tension resulting from the seismic lateral loads and other concurrent loads. The design loads are presented in Table 3. The type and arrangement of dowels required for the connection are being finalized and will be provided during the structural audits.

At first, the dowels will be grouted only on one side, either at the building or the underpinning. The other side will be grouted only after jacking loads are applied and held. To achieve this for the horizontal dowels, the end portion of the underpinning wall will be poured after jacking loads are applied and held long enough for the till to be within secondary compression.

6.0 EXISTING STRUCTURE

Based on a preliminary analysis, the following areas between column lines G and H appear to be overstressed:

a. Slab at el 659'

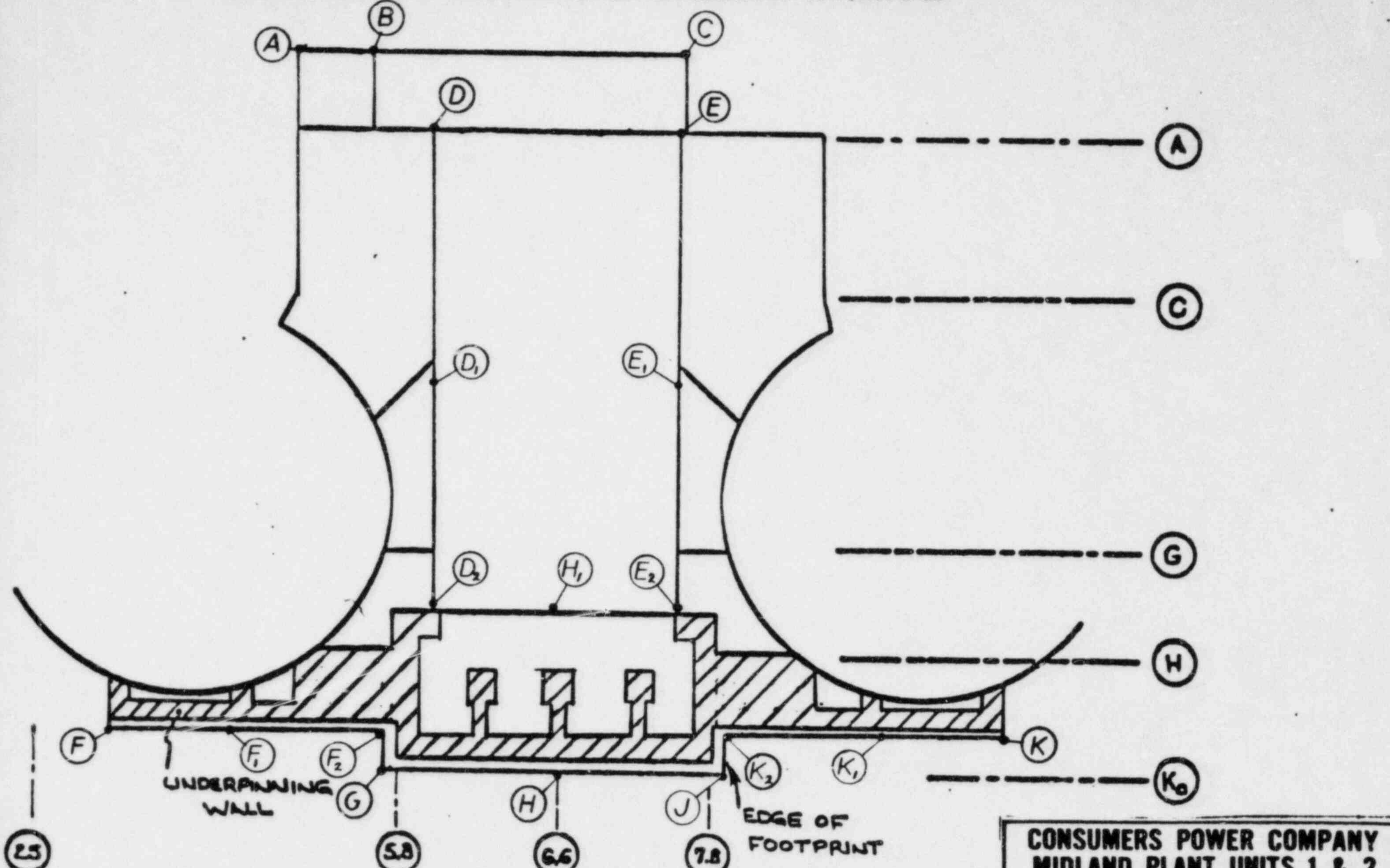
b. Shear walls on column lines 5.6 and 7.8 between elevations 564' and 614'

c. West staircase wall on column line 5.3 between elevations 646' and 685'

d. Walls on column lines 5.8 and 7.2 from elevations 659' to 699'

The above mentioned areas will be structurally upgraded to withstand all loads including 1.5 x E' if the more rigorous final analysis still indicates that these areas are overstressed.

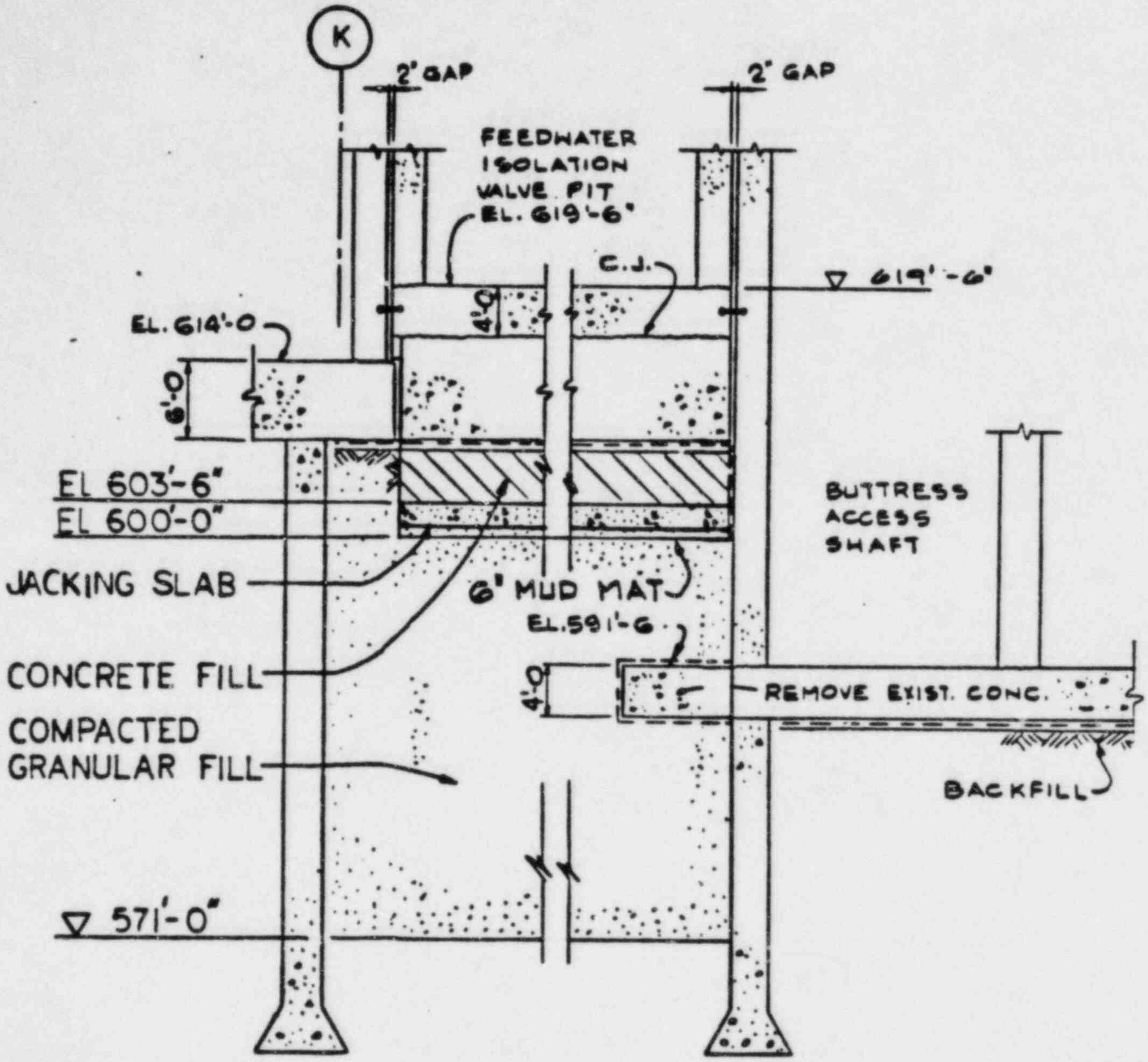
AUXILIARY BUILDING UNDERPINNING



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

SOIL PRESSURE DATA POINTS

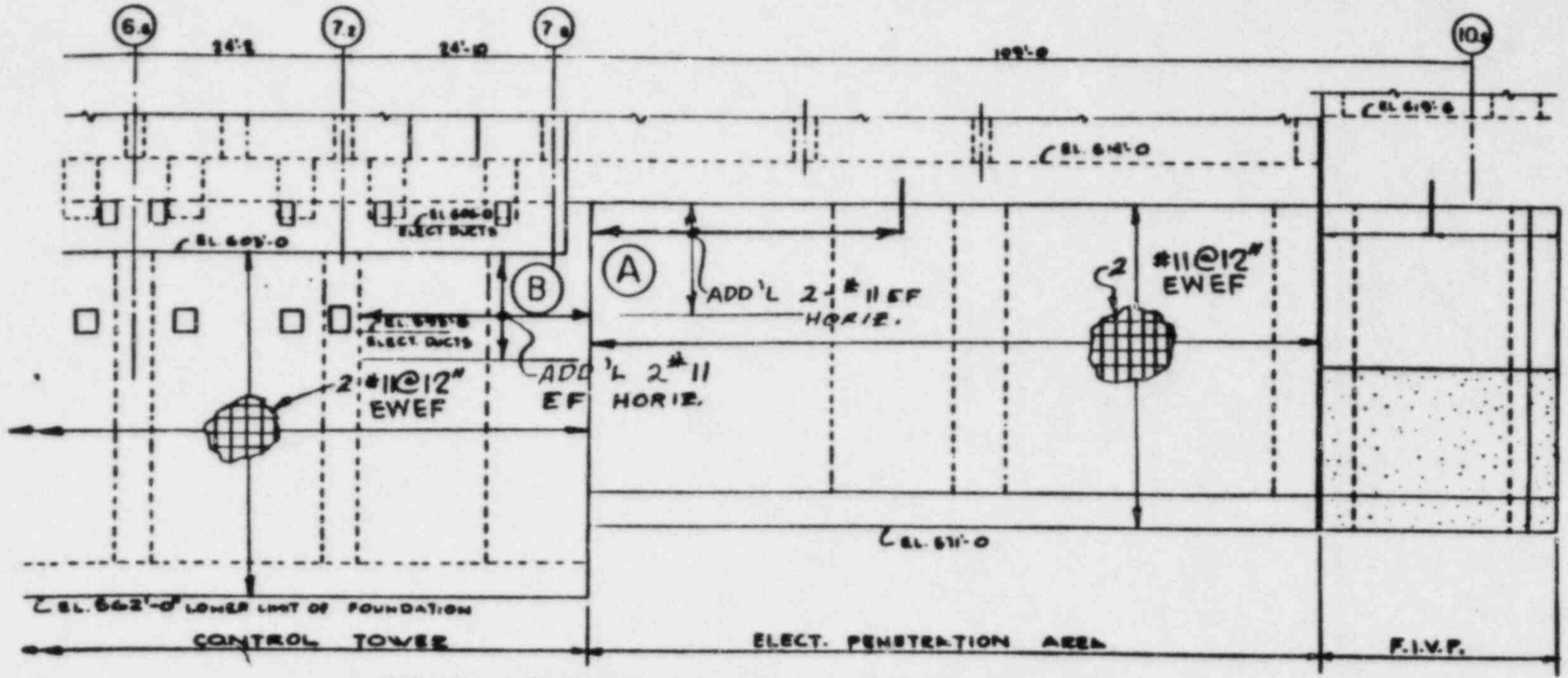
FIGURE 1



F.I.V.P. SUPPORT DETAIL

CONSUMERS POWER COMPANY **MIDLAND PLANT UNITS 1 & 2**
FEEDWATER ISOLATION VALVE PIT SUPPORT DETAIL
FIGURE 2

12-1-81



SOUTH ELEVATION OF UNDERPINNING WALL

CRITICAL SECTION FOR WALL DESIGN

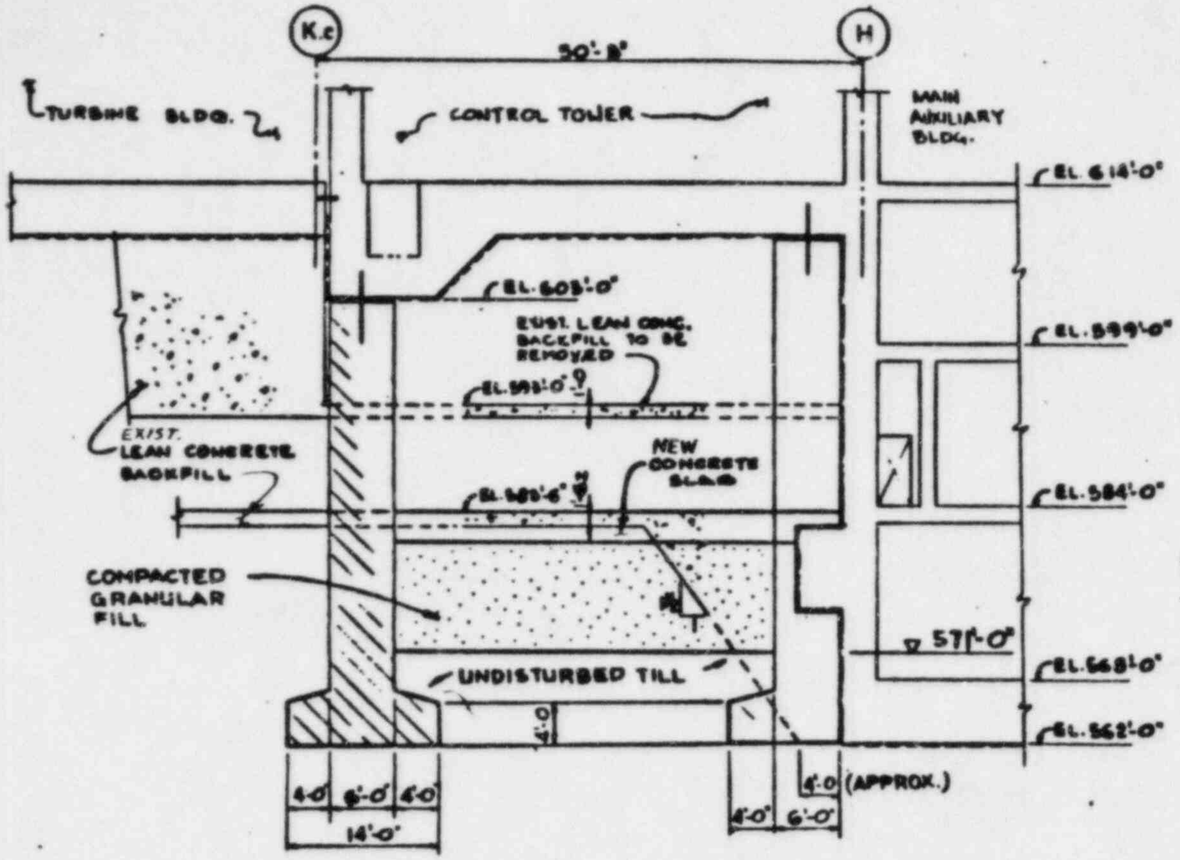
(A) ELECTRICAL PENETRATION WING

(B) CONTROL TOWER

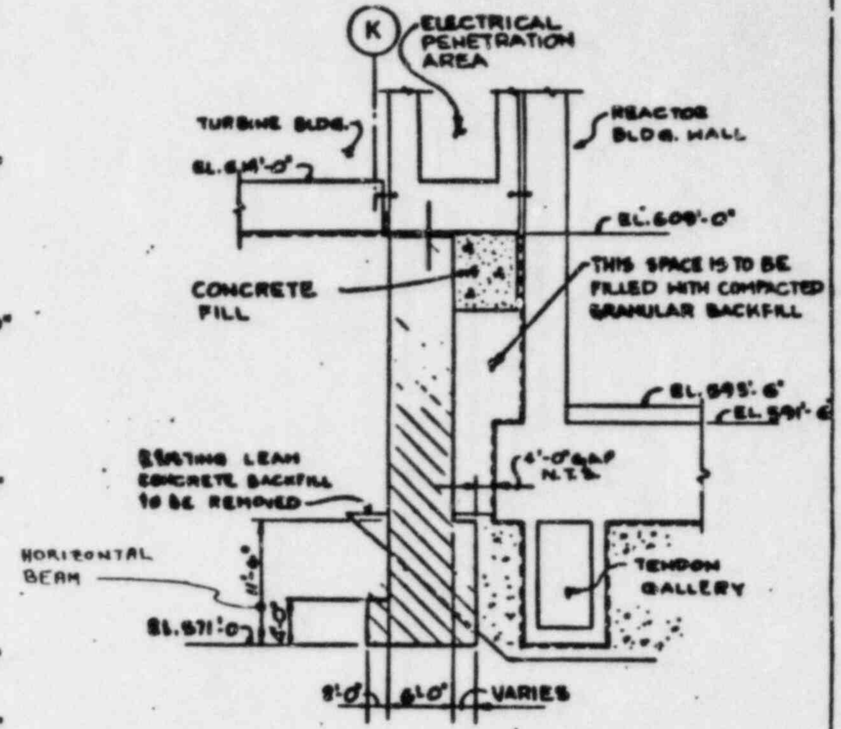
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

Auxilliary Building Underpinning
Wall Reinforcing Detail

Figure 3



WALL UNDER CONTROL TOWER



WALL UNDER ELECTRICAL PENETRATION AREA

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

UNDERPINNING WALL DETAILS

FIGURE 4

NET SOIL PRESSURE (KSF)					ULT. NET BEARING CAPACITY (KSF)
POINT (SEE FIG. 1)	EL.	D+L+R+E'+P_L CASE 1	D+L+R+E'+P_L CASE 2	D+L+R+P_L	
A	609'-0	-3.4	0.8	-1.3	30
B	609'-0	-2.4	-0.3	-1.3	30
C	630'-6"	1.2	-3.7	-1.2	15
D	562'-0	-7.1	-5.3	-6.2	44
E	562'-0	-7.9	-3.9	-5.9	44
D1	562'-0	-6.5	-2.3	-4.4	44
E1	562'-0	-2.9	-5.3	-4.1	44
D2	562'-0	-10.2	-3.0	-6.6	44
E2	562'-0	-5.8	-6.8	-6.3	44
F	571'-0	-18.2	1.6(-3.0)	-8.3	44
F1	571'-0	-15.3	-0.7	-8.0	44
F2	571'-0	-12.8	-2.8	-7.8	44
G	562'-0	-15.3	-4.7	-10.0	44
H	562'-0	-12.7	-7.3	-10.0	44
H1	562'-0	-7.6	-5.0	-6.3	44
J	562'-0	-9.9	-9.9	-9.9	44
K	571'-0	-2.5	-13.5	-8.0	44
K1	571'-0	-5.2	-10.4	-7.8	44
K2	571'-0	-7.5	-7.9	-7.7	44

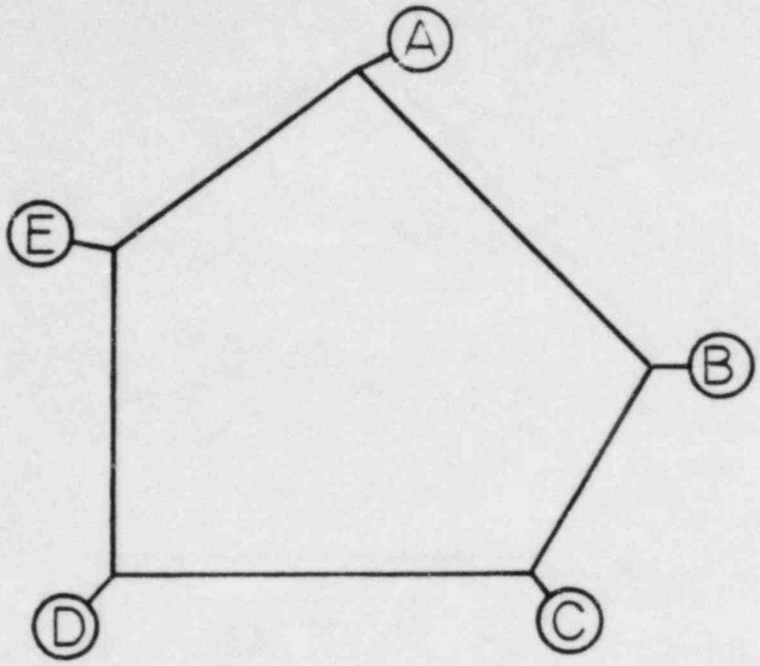
1. Case 1 corresponds to maximum compression @ PT. F
2. Case 2 corresponds to minimum compression @ PT. F
3. Gross soil pressure is given in parenthesis
4. Compression is negative

Note: Net pressure is total pressure minus the pressure due to the removed soil

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

AUX BLDG UNDERPINNING SOIL PRESSURE

TABLE-1



SOIL PRESSURE (KSF)

	D+L+E′		D+L
POINT	CASE 1	CASE 2	CASE 3
A	2.54	2.96	-3.07
B	-7.16	-6.52	-4.68
C	-10.83	-10.12	-5.27
D	-7.41	-6.78	-4.68
E	0.39	0.85	-3.40

1) CASE 1 CORRESPONDS TO MAX. COMPRESSION

2) CASE 2 CORRESPONDS TO MIN. COMPRESSION

3) COMPRESSION IS NEGATIVE

4) ULTIMATE BEARING CAPACITY = 25 KSF (ESTIMATED MINIMUM VALUE)

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

FIVP
SOIL PRESSURES

TABLE-2

UNDERPINNING WALLS

LOCATION (SEE FIG. 3)		LOAD COMB.	AXIAL K/FT	MOM'T K-FT/FT	MOM'T CAP. K-FT/FT	IN PLANE SHEAR K/FT	IN PLANE SHEAR CAP. K/FT
A	VERT. SECT.	1	358	-387	± 816	22.6	±278
A	HORIZ. SECT.	1	-48.5	-27.4	± 968	22.6	±318
B	VERT. SECT.	1	278	370	± 969	-29.8	±358
B	HORIZ. SECT.	1	-122.	30.1	±1100	-29.8	±318

INTERFACES (Load Comb. 2)

LOCATION	INTERFACE	AXIAL K/FT	SHEAR K/FT	SHEAR CAP. K/FT
A (FIG. 3)	HORIZ	15.7	117	*
E_2 (FIG. 1)	VERT	12.7	79.7	*

LOAD COMBINATIONS:

1. $U = 1.4D + 1.7L + P_L$
2. $U = D + L + R + 1.5E' + P_L$

NOTE: 1) THE CAPACITIES CORRESPOND TO THE EXISTING AXIAL LOADS.

2) +VE AXIAL LOAD IS TENSION

3) THE CRITICAL OUT OF PLANE SHEAR IN THE UNDERPINNING WALL IS 21.3k/ft WHILE THE CAPACITY IS 94k/ft

*THE TYPE AND SPACING OF DOWELS WILL BE FINALIZED

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

Aux. Bldg. Underpinning

Design Loads

Table 3

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

NOV 24 1981

PRINCIPAL STAFF			
DIR		FEIS	
D/D		PAO	
A/D		SLO	
RA&PI			
DE&TI			
DEP&OS		File	

Docket Nos: 50-329 OM, OL
and 50-330

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

Subject: Staff Concurrence for Construction of Access Shafts and Freeze Wall in Preparation for Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

During several meetings with the NRC staff, including more recently those on October 1 and November 4, 1981, members of Consumers Power Company (CPCo) and consultants have described the underpinning planned beneath the electrical penetration areas and the control tower portions of the auxiliary building and beneath the adjacent feedwater isolation valve pits for Midland Plant, Units 1 and 2. These discussions have included the fact that in order to prepare for implementing the underpinning scheme, vertical access shafts on the east and west ends of the auxiliary building and adjacent to each feedwater valve pit and the turbine building must first be constructed from plant grade (elevation 634 feet) down to elevation 609. In addition, a freezewall is necessary to augment the present construction dewatering scheme. The general locations of the access shafts and freezewall are shown on Enclosures 1 and 2. Your letters of October 28 and November 16, 1981 have responded to NRC requests for additional information and have requested staff concurrence to proceed with construction of the access shafts and freezewall.

Our review recognizes that the vertical portion of the access shaft will not undermine any existing structure. The shafts and the freezewall can be abandoned at any time and will be backfilled with concrete or soil upon completion of the underpinning activity. Accordingly, this activity does not represent an irreversible commitment. It also has no effect on any other remedial action that may be required as a result of the staff's continuing review of subsequent phases of the underpinning scheme or as a result of the staff's OL review or the OM-OL hearing. Our review further recognizes the commitment of your staff that Region III personnel will be notified prior to drilling near seismic Category I underground utilities and structures.

In view of the above, the NRC staff concurs with your plans to begin construction of the vertical access shaft down to elevation 609 and installation of the freeze wall hardware.

~~8112110044~~ cpp

NOV 30 1981

III

A later phase of your underpinning work is understood to involve excavation beneath the valve pit structures, and extending the access shaft deeper to permit excavation along the turbine building for eventual access beneath the auxiliary building. However, this later phase requires submittal of further information for staff review and approval and our above concurrence does not authorize excavation directly beneath any structure. Similarly, our review of the effects of operation of the freezewall involves submittal of additional information (e.g., potential heave and resettlement) and our above concurrence is limited to installation of the freezewall, and does not include its activation. The additional information associated with these later phases will be discussed by the staff during the OM-OL hearing session beginning December 1981.

Sincerely,

R Tedesco

Robert L. Tedesco, Assistant Director
for Licensing
Division of Licensing

Enclosure(s):
As stated

cc: See next page

MIDLAND

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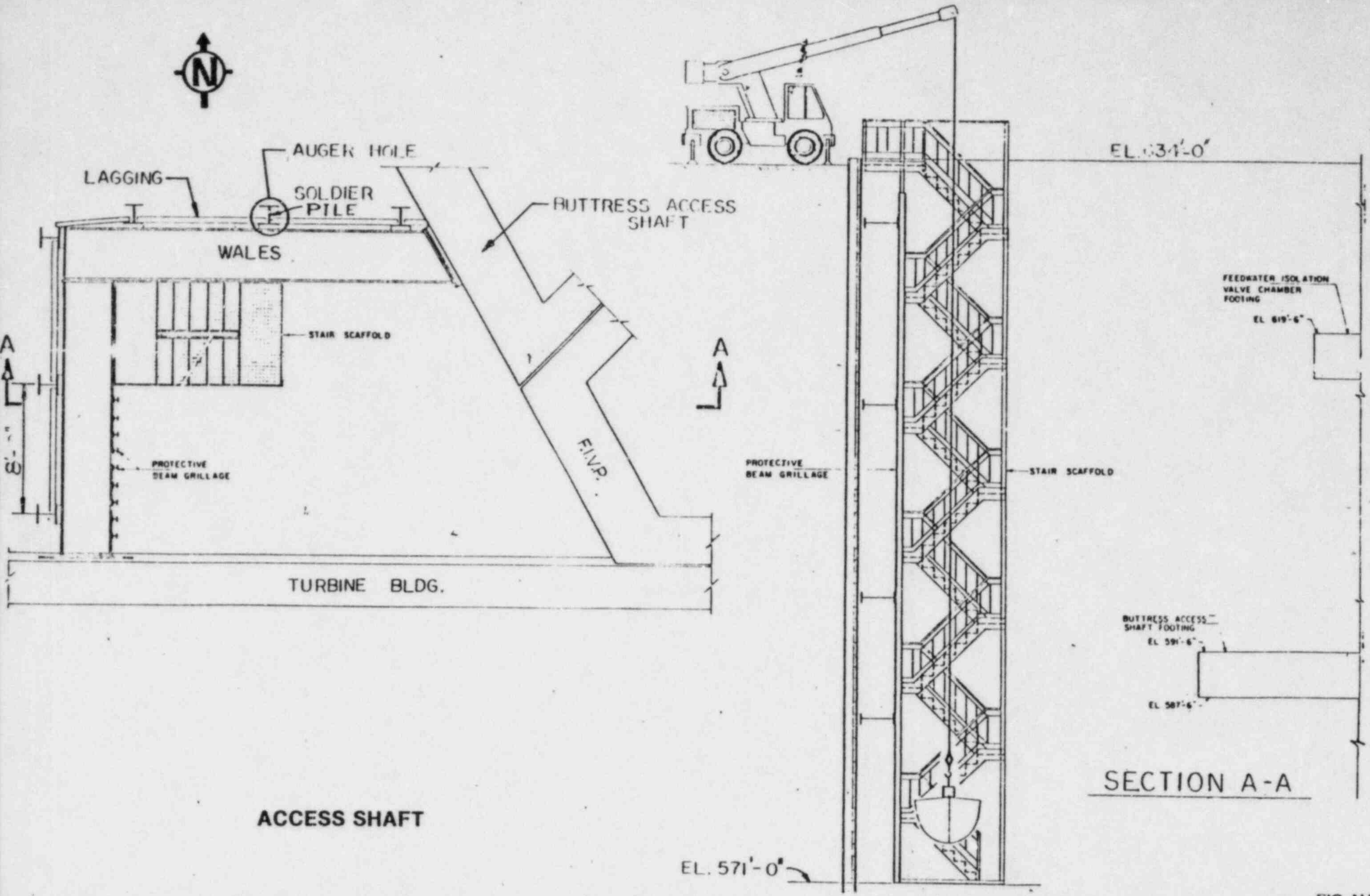

FIG AUX-29

ENCLOSURE 1

ENCLOSURE 2