DOCKETED
USNRC

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

'82 OCT 25 A10:29

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

OF SECRETARY
DOCKETING & SERVICE
BRANCH

In the Matter of	)	
	)	Docket Nos. 50-329-OM
CONSUMERS POWER COMPANY	)	50-330-OM
	)	50-329-OL
(Midland Plant, Units 1 and 2)	)	50-330-OL

TESTIMONY OF DR. W. GENE CORLEY
CONCERNING CRACKING IN THE
SERVICE WATER PUMP STRUCTURE

My name is W. Gene Corley. I am a Divisional Director, Engineering Development Division, Construction Technology Laboratories, a Division of the Portland Cement Association. The Portland Cement Association (PCA) is a nonprofit Illinois corporation devoted to the improvement in uses of Portland Cement Concrete. PCA has been retained as a consultant to Consumers Power Company, and I am the representative of the PCA who is most familiar with the issues described in this testimony.

I am joint author, with Dr. Fiorato, of the attached report entitled "Evaluation of Cracking in Service Water Pump Structure at Midland Plant". This report was originally submitted to the NRC by letter from J. W. Cook to H. R. Denton dated March 2, 1982. I am not responsible for other portions of Applicant's Service Water Pump Structure testimony.

I have an M.S. and a Ph.D. in structural engineering from the University of Illinois and over twenty years of experience as a structural engineer, as described in more detail in my attached resume. My experience has included design, construction and testing of concrete structures. In addition, I have acted as a specialized consultant on many jobs where construction problems or structural damage have occurred. This specialized consulting work has included field inspections to evaluate earthquake damage, blast damage, damage caused by settlement, and other conditions relevant to questions raised by the NRC staff in their review of the Midland plant. My previous work has also included development of information on fatigue properties of reinforcing bars and nonferrous metals. I am a registered structural engineer

DS03

in the state of Illinois, and a registered professional engineer in three other states.

I am currently a member of the American Concrete Institute (ACI) Committee 318 on standard building code. In addition, I am a member of the ACI Technical Activities Committee, which has the responsibility for reviewing and approving all technical changes in all ACI codes and specifications, including ACI 318 and ACI 349.

I have personally visited the site and inspected the Service Water Pump Structure. In addition, I have inspected other structures at this site which have displayed concrete cracking. I believe that based on my education and work experience and this inspection of the Midland structures, I am qualified to testify as an expert concerning the matters described in this Service Water Pump Structure testimony.

I swear that the statements made in this Service Water Pump Structure testimony and the attached report are true and correct, to the best of my knowledge and belief.

Dr. W. Gene Corley

SUBSCRIBED ANS SWORN TO
before me this 14th day
of October, 1982.

Edna M. Ringquist
Notary Public

WILLIAM GENE CORLEY - Divisional Director, Engineering Development Division, Construction Technology Laboratories, a Division of the Portland Cement Association, Old Orchard Road, Skokie, Illinois 60077 (312) 966-6200

Education:

B.S. Civil Engineering, University of Illinois - 1958
M.S. Structural Engineering, University of Illinois - 1960
Ph.D. Structural Engineering, University of Illinois - 1961

Professional Experience:

Portland Cement Association -

1979 to present	- Divisional Director of Engineering Development Division
1974 to 1979	- Director of Engineering Development Department
1966 to 1974	- Manager of Structural Development Section
1964 to 1966	- Development Engineer in Structural Development Section

United States Army Corps of Engineers (1st Lt. U.S.A.)
Research and Development Coordinator for Military Bridging - 1961 to 1964

University of Illinois, Research Assistant - 1958 to 1961

Shelby County (Illinois) Department of Highways, Junior Engineer - 1958

Professional Affiliations and Registration:

American Society of Civil Engineers (ASCE) (Fellow)
- Member and Former Chairman, Structural Division Committee on Research
- Member, Committee on Limit Design
- Member and Former Secretary, Reinforced Concrete Research Council (RCRC)
- Member and Former Chairman, Committee on Concrete Bridge Design

National Society of Professional Engineers

Professional Affiliations and Registration: (Continued)

American Concrete Institute (Fellow)
- Member and Former Chairman, Committee on Bridge Design
- Member, Committee on Standard Building Code
- Member, Committee on Limit Design
- Former Member, Board Committee on International Activities
- Member, Technical Activities Committee
- Former Member, Committee on Deflections
- Former Member, Committee on Crossties

Building Seismic Safety Council (BSSC)
- Member, Board of Direction

Chicago Committee on High Rise Buildings - Vice Chairman

Earthquake Engineering Research Institute

International Association for Bridge and Structural Engineering

Prestressed Concrete Institute

Society of Sigma Xi

RILEM
- Member, Committee on Testing Structures In-Situ
- Member, Committee on Fatigue of Concrete

Post Tensioning Institute

- Member, Technical Activities Board

Transportation Research Board
- Member, Committee on Design of Concrete Superstructures

Registered Engineer - Virginia Reg. No. 3086

Registered Structural Engineer - Illinois Reg. No. 81-3459

Registered Professional Engineer - Washington Reg. No. 17224

Registered Professional Engineer - Mississippi Reg. No. 7666

Awards:

ACI Wason Medal for Research 1970
ACI Bloem Award 1978
PCI Martin Korn Award 1978
ASCE T.Y. Lin Award 1979

Publications: (Papers Published)

1. Corley, W. G., "Shear in Two-Way Slabs - ACI Approach," ACI-CEB-PCI-FIP Symposium, ACI Publication SP-59, CEB Bulletin 113, Copyright 1979, 346 p.

2. Russell, H. G., Oesterle, R. G., Fiorato, A. E., and Corley, W. G., "Applicability of Structural Wall Test Results to Seismic Design of Nuclear Facilities," Journal, Nuclear Engineering and Design, Vol. 50, October 1978, pp. 49-56.

3. Aristizabal-Ochoa, J. D., Oesterle, R. G., Fiorato, A. E., and Corley, W. G., "Cyclic Inelastic Behavior of Structural Walls," Proceedings, Sixth European Conference on Earthquake Engineering, Vol. 3, Tests on Structures and Structural Elements, Dubrovnik, Yugoslavia, September 1978, pp. 231-238.

4. Derecho, A. T., Iqbal, M., Ghosh, S. K., Fintel, M., and Corley, W. G., "Structural Walls in Earthquake-Resistant Buildings, Dynamic Analysis of Isolated Structural Walls-REPRESENTATIVE LOADING HISTORY." Report to the National Science Foundation (ASRA) under Grant No. ENV77-15333, August 1978.

5. Fiorato, A. E., Oesterle, R. G., and Corley, W. G., "Importance of Reinforcement Details in Earthquake-Resistant Structural Walls," Proceedings of a Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, Berkeley, June 1978, pp. 1430-1451.

6. Derecho, A. T., Iqbal, M., Fintel, M., and Corley, W. G., "Loading History for Use in Quasi-Static Simulated Earthquake Loading Tests," Proceedings of Symposium on Mathematical Modelling of Reinforced Concrete Structures Subjected to Wind and Earthquake Forces, Toronto, Canada, April 1978.

7. Iqbal, M., Derecho, A. T., and Corley, W. G., "Distribution of Inertial Forces Over the Heights of R.C. Structural Walls Subjected to Strong Ground Motion," Proceedings of Symposium on Mathematical Modelling of Reinforced Concrete Structures Subjected to Wind and Earthquake Forces, Toronto, Canada, April 1978.

8. Derecho, A. T., Iqbal, M., Ghosh, S. K., Fintel, M., and Corley, W. G., "Structural Walls in Earthquake-Resistant Buildings-Analytical Investigation, Dynamic Analysis of Isolated Structural Walls - REPRESENTATIVE LOADING HISTORY," Final Report to the National Science Foundation, RANN, Under Grant No. ENV77-1533, Portland Cement Association, April 1978.

9. Iqbal, M., Derecho, A. T., and Corley, W. G., "Ductility and Energy Dissipation in Earthquake-Resistant Reinforced Concrete Structural Walls," published in Symposium Proceedings of Symposium on Behavior of Building Systems and Building Compliments, Vanderbilt University, Nashville, Tennessee, March 1978.

10. Fiorato, A. E., Oesterle, R. G., Russell, H. G., and Corley, W. G., "Tests of Structural Walls Under Reversing Loads," Proceedings, Central American Conference on Earthquake Engineering, San Salvador, El Salvador, January 1978.

11. Shiu, K., Barney, G. B., Fiorato, A. E., and Corley, W. G., "Reversing Load Tests of Reinforced Concrete Coupling Beams," Proceedings, Central American Conference on Earthquake Engineering, San Salvador, El Salvador, January 1978, pp. 239-249.

12. Kaar, P. H., Fiorato, A. E., Carpenter, J. E., and Corley, W. G., "Limiting Strains of Concrete Confined by Rectangular Hoops," made into a PCA Research and Development Bulletin RD053.01D.

13. Russell, H. G. and Corley, W. G., "Time-Dependent Behavior of Columns in Water Tower Place," Douglas McHenry International Symposium on Concrete and Concrete Structures, ACI Symposium Volume SP-55; also PCA Research and Development Bulletin RD052.01B.

14. Corley, W. G., Hanson, J. M., and Helgason, Th., "Design of Reinforced Concrete for Fatigue," Journal of the Structural Division, ASCE, June 1978, pp. 921-932; also PCA Research and Development Bulletin RD059.01D.

15. Barney, G. B., Shiu, K. N., Rabbat, B. G., Fiorato, A. E., Russell, H. G., and Corley, W. G., "Earthquake Resistant Structural Walls - Tests of Coupling Beams," originally a report to NSF, January 1978. Available through National Technical Information Service-NBS.

16. Barney, G. B., Corley, W. G., Hanson, J. M., and Parmelee, R. A., "Behavior and Design of Prestressed Concrete Beams with Large Web Openings," Journal of the Prestressed Concrete Institute, Nov./Dec. 1977, pp. 32-61; also PCA Research and Development Bulletin RD054.01D.

17. Fiorato, A. E., and Corley, W. G., "Laboratory Tests of Earthquake-Resistant Structural Wall Systems and Elements," Proceedings of a Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, Berkeley, July 1977, pp. 1388-1429.

18. Barda, Felix, Hanson, John M., and Corley, W. Gene, "Shear Strength of Low-Rise Walls with Boundary Elements," ACI Special Publication SP-53, Reinforced Concrete Structures in Seismic Zones; also, PCA R&D Bulletin RD043.01D.

19. Corley, W. G., (Committee Chairman) ACI Committee 443, "Recommended Practice for Analysis and Design of Reinforced Concrete Bridge Structures," American Concrete Institute, Detroit, March 1977, 116 p.

20. Oesterle, R. G., Fiorato, A. E., and Corley, W. G., "Free Vibration Tests of Structural Walls," Proceedings, Sixth World Conference on Earthquake Engineering, India, January 1977.

21. Kaar, P. H., and Corley, W. G., "Properties of Confined Concrete for Design of Earthquake Resistant Structures," Proceedings, Sixth World Conference on Earthquake Engineering, India, January 1977.

22. Fiorato, A. E., Oesterle, R. G., and Corley, W. G., "Ductility of Structural Walls for Design of Earthquake Resistant Buildings," Proceedings, Sixth World Conference on Earthquake Engineering, India, January 1977.

23. Hanson, N. W., Russell, H. G., Corley, W. G., Schultz, D., and Fintel, M., "Tests of Cantilever Action in Damaged Large Panel Structures," Preliminary Report of the Tenth Congress of IABSE.

24. Oesterle, R. G., Fiorato, A. E., Johal, P., Carpenter, J. E., Russell, H. G., and Corley, W. G., "Earthquake Resistant Structural Walls-Tests of Isolated Walls." Report to the National Science Foundation, available through NTIS, November 1976.

25. Corley, W. G., (Committee Chairman) ACI Committee 443, "Prestressed Concrete Bridge Design," ACI Journal, Proceedings V. 73, No. 11, November 1976, pp. 597-612.

26. Corley, W. G., "Improved Seismic Design-Influence of Current Structural Concrete Research," Proceedings, Structural Engineers Association of California 1976 Convention, pp. 47-59, October 1976.

27. Kaar, P. H., Fiorato, A. E., Carpenter, J. E., and Corley, W. G., "Confined Concrete in Compression Zones of Structural Walls Designed to Resist Lateral Loads Due to Earthquakes," Proceedings, International Symposium on Earthquake Structural Engineering, University of Missouri-Rolla, St. Louis, 1976, pp. 1207-1218.

28. Fiorato, A. E., Oesterle, R. G., Kaar, P. H., Barney, G. B., Rabbat, B. G., Carpenter, J. E., Russell, H. G., and Corley, W. G., "Highlights of an Experimental Investigation of the Seismic Performance of Structural Walls," Proceedings, ASCE/EMD Specialty Conference Volume - Dynamic Response of Structures: Instrumentation, Testing Methods and System Identification, University of California, Los Angeles, 1976, pp. 308-317.

29. Carpenter, J. E., Hanson, J. M., Fiorato, A. E., Russell, H. G., Meinheit, D. F., Rosenthal, I., Corley, W. G., and Hognestad, E., "Design of Bent Caps for Concrete Box Girder Bridges," NCHRP Bulletin 163, Transportation Research Board, National Research Council, Washington, D. C. 1976, Part I, also PCA Research & Development Bulletin RD032.01E.

30. Helgason, Th., Hanson, J. M., Somes, N. F., Corley, W. G., and Hognestad, E., "Fatigue Strength of High Yield Reinforcing Bars," NCHRP Bulletin 164, Transportation Research Board, National Research Council, Washington, D. C. 1976, also PCA Research and Development Bulletin RD045.01D.

31. Corley, W. Gene, "Laboratory Tests of Shear Walls for Multi-Story Buildings," Proceedings, Fifth European Conference on Earthquake Engineering, Istanbul, Turkey, September 1975.

32. Helgason, Th., Russell, H. G., Corley, W. G., and Hognestad, E., "Time-Dependent Behavior of Columns in the World's Tallest Reinforced Concrete Building," Preliminary Reports, Behavior in Service of Concrete Structures, V. I., Liege, Belgium, June 1975, pp. 343-353.

33. Corley, W. G., "Put Openings in Your Beams," published in Concrete Construction, February 1975, pp. 47-49.

34. Kaar, P. H., Hanson, N. W., Corley, W. G., and Hognestad, E., "Bond Fatigue Tests of Pretensioned Concrete Crossties," 1974 FIP/PCI Congress, New York.

35. Barney, G. B., Corley, W. G., Hanson, J. M., and Parmelee, R. A., "Design of Prestressed Concrete Beams with Large Web Openings," 1974 FIP/PCI Congress, New York.

36. Corley, W. G., (Committee Chairman) ACI Committee 443, "Analysis and Design of Reinforced Concrete Bridge Structures," ACI Journal, Proceedings Vol. 71, No. 4, April 1974, pp. 171-200.

37. Corley, W. Gene, "Ductile Shear Walls in Multi-Story Buildings - Laboratory Tests," Proceedings, 42nd Annual Convention of SEAOC, October 1973.

38. Corley, W. G., (Committee Chairman) ACI Committee 443, "Preliminary Design and Proportioning of Reinforced Concrete Bridge Structures," ACI Journal, Proceedings Vol. 70, No. 5, May 1973, pp. 328-336.

39. Somes, Norman F., and Corley, W. Gene, "Circular Openings in Webs of Continuous Joists," ACI Symposium Volume SP-42; also PCA R&D Bulletin RD018.01B.

40. Corley, W. G., Carpenter, J. E., Russell, H. G., Hanson, N. W., Cardenas, A. E., Helgason, Th., Hanson, J. M., and Hognestad, E., "Construction and Testing of 1/10-Scale Micro-Concrete Model of New Potomac River Crossing, I-266," PCA Bulletin RD031.01E.

41. Hawkins, Neil M., and Corley, W. G., "Moment Transfer to Column in Slabs with Shearhead Reinforcement," ACI Special Publication SP-42, Shear in Reinforced Concrete; also, PCA R&D Bulletin RD037.01D.

42. Barda, F., Hanson, J. M., and Corley, W. G., "An Investigation of the Design and Repair of Low-Rise Sher Walls," Fifth World Conference on Earthquake Engineering, Rome 1973.

43. Corley, W. G., and Hanson, J. M., "Design of Earthquake-Resistant Walls," Fifth World Conference on Earthquake Engineering, Rome 1973.

44. Carpenter, J. E., Kaar, P. H., and Corley, W. G., "Design of Ductile Flat Plate Structures to Resist Earthquakes," Fifth World Conference on Earthquake Engineering, Rome 1973.

45. Hanson, J. M., Carpenter, J. E., and Corley, W. G., "Analysis and Design of Concrete Bridge Bents (NCHRP Project No. 12-10)," 58th Annual Meeting of AASHO, Phoenix, Arizona, November 1972.

46. Hanson, J. M., Corley, W. G., and Hognestad, E., "Evaluation of Structural Concrete Members Penetrated by Service Systems," Proceedings, RILEM-ASCE Symposium, Philadelphia, May 1972.

47. Cardenas, A. E., Hanson, J. M., Corley, W. G., and Hognestad, E., "Design Provision for Shear Walls," Journal of the American Concrete Institute, March 1973, No. 3, Proceedings Vol. 70; also PCA Bulletin RD028.01D.

48. Hawkins, N. W., and Corley, W. G., "Transfer of Unbalanced Moment and Shear from Flat Plates to Columns," Paper SP 30-7, ACI Special Publication SP-30, Detroit, 1971.

49. Corley, W. G., and Hognestad, E., "Tests of a 1/10-Scale Concrete Model to Aid Design of a Large Prestressed Bridge," Proceedings, 9th Congress of IABSE, Amsterdam, May 1972.

50. Corley, W. G., "Performance of Structures in 1971 Los Angeles Shock," Proceedings of the 47th Annual Meeting of the Concrete Reinforcing Steel Institute, CRSI, Chicago, Illinois.

51. Corley, W. G., et al, "Design Ultimate Load Tests of 1/10-Scale Micro-Concrete Model of New Potomac River Crossing, I-266," Journal of the Prestressed Concrete Institute, November-December 1971, pp. 70-84. Also PCA Bulletin RD-031.01E.

52. Magura, D. D., and Corley, W. G., "Tests to Destruction of a Multi-Panel Waffle Slab Structures - 1964-65 New York World's Fair," Journal of the American Concrete Institute, Digest Paper, September 1971.

53. Pfeifer, D. W., Magura, D. D., Russell, H. G., and Corley, W. G., "Time-Dependent Deformation in a 70-Story Structure," ACI Special Publication SP-27, Detroit, 1971.

54. Corley, W. G., "1969 Portland Cement Association Research on Shear Walls," Proceedings of the 1969 Annual Meeting of the Structural Engineers of California.

55. Corley, W. G., and Jirsa, J. O., "Equivalent Frame Analysis for Slab Design," Journal of the American Concrete Institute, November 1970.

56. Corley, W. G., "Effect of Research on the Future of Concrete Bridge Design," Proceedings of the Colorado State University Bridge Seminar, Ft. Collins, Colorado, 1969.

57. Kaar, P. H., Conner, H. W., and Corley, W. G., "Moment Redistribution in a Precast Concrete Rigid Frame," Journal of the Structural Division of ASCE, March 1970.

58. Corley, W. G., and Hanson, N. W., "Design of Beam-Column Joints for Seismic Resistant Reinforced Concrete Frames," Proceedings, Fourth World Conference on Earthquake Engineering, Santiago, Chile, 1969.

59. Magura, D. D., and Corley, W. G., "Tests to Destruction of a Multi-Panel Slab Structure - 1964-65 New York World's Fair," Vol. II, The Rathskeller Structure, Building Research Advisory Board, Publication 1721, 1969.

60. Magura, D. D., and Corley, W. G., "Techniques for Tests of a Multi-Panel Waffle Slab - 1964-65 New York World's Fair," Vol. II, The Rathskeller Structure, Building Research Advisory Board, Publication 1721, 1969.

61. Corley, W. G., and Hawkins, N. M., "Shearhead Reinforcement for Slabs," Journal of the American Concrete Institute, October 1968, pp. 811-824; PCA Development Department Bulletin D144.

62. Burton, K. T., Corley, W. G., and Hognestad, E., "Connections in Precast Concrete Structures - Effects of Restrained Creep and Shrinkage," Journal of the Prestressed Concrete Institute, Vol 12, No. 2, April 1967, pp. 18-37; PCA Development Department Bulletin D117.

63. Corley, W. G., "Rotational Capacity of Reinforced Concrete Beams," Journal of the Structural Division, ASCE, October 1966, pp. 121-146, PCA Development Department Bulletin D108.

64. Corley, W. G., and Sozen, M. A., "Time-Dependent Deflections of Reinforced Concrete Beams," Journal of the American Concrete Institute, March 1966, pp. 373-386.

65. Corley, W. G., "Dynamic Response of Military Bridges," Proceedings of the Army Conference on Dynamic Behavior of Materials and Structures, Springfield Armory, Springfield, Mass., September 1962, pp. 170-197.

66. Corley, W. G., and Sozen, M. A., Discussion: "Creep of Prestressed Concrete Beams," by W. S. Cottingham, P. G. Fluck, and G. W. Washa, Journal of the American Concrete Institute, September 1961, pp. 1787-1793.

67. Corley, W. G., Sozen, M. A., and Siess, C. P., "The Equivalent Frame Analysis for Reinforced Concrete Slabs," Structural Research Series No. 219, University of Illinois, Urbana, Illinois, June 1961.

68. Corley, W. G., Sozen, M. A., and Siess, C. P., "Time-Dependent Deflections of Prestressed Concrete Beams," Highway Research Board Bulletin 307, National Academy of Sciences - National Research Council, Washington, D. C., pp. 1-25.

69. Corley, W. G., Discussion: "The Apparent Modulus of Elasticity of Prestressed Concrete Beams under Different Stress Levels," by W. N. Lofroos and A. M. Ozell, Journal of the Prestressed Concrete Institute, June 1960, pp. 82-88.

70. Corley, W. G., "Bibliography on Time-Dependent Effects in Plain and Reinforced Concrete," Department of Civil Engineering, University of Illinois, Urbana, Illinois, December 1959.

71. Corley, W. G., Sozen, M. A., and Siess, C. P., "A Study of Time-Dependent Deflections of Prestressed Concrete Beams," Structural Research Series No. 184, University of Illinois, Urbana, Illinois, October 1959.

72. Oesterle, R. G., Aristizabal-Ochoa, J. D., Fiorato, A. E., Russell, H. G., and Corley, W. G., "Earthquake-Resistant Structural Walls - Tests of Isolated Walls - Phase II".

73. Corley, W. G., Colley, B. E., Hanna, A. N., Nussbaum, P. N., and Russell, H. G., "Prestressed Concrete in Transportation Systems" published in PCI Journal, 1980.

74. Barney, G. B., Shiu, K. N., Rabbat, B. G., Fiorato, A. E., Russell, H. G., and Corley, W. G., "Behavior of Coupling Beams Under Load Reversals," published as R & D Bulletin, RD068.01B, 1980.

75. Oesterle, R. G., Fiorato, A. E., Aristizabal-Ochoa, J. D., and Corley, W. G., "Hysteretic Response of Reinforced Concrete Structural Walls," published in ACI Special Symposium Volume, SP63, 1980.

76. Cardenas, A. E., Russell, H. G., and Corley, W. G., "Strength of Low-Rise Structural Walls," published in ACI Special Symposium Volume, SP63, 1980.

77. Corley, W. G., Fintel, M., Fiorato, A. E., and Derecho, A. T., "Earthquake Engineering Research at the Portland Cement Association - A Progress Report," published in Proceedings of Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, September 1980, Vol. 9, pp. 17-32.

78. Derecho, A. T., Iqbal, M., and Corley, W. G., "Determining Design Force Levels for Earthquake-Resistant Reinforced Concrete Structural Walls," published in Proceedings of Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, September 1980, Vol. 4, pp. 1-8.

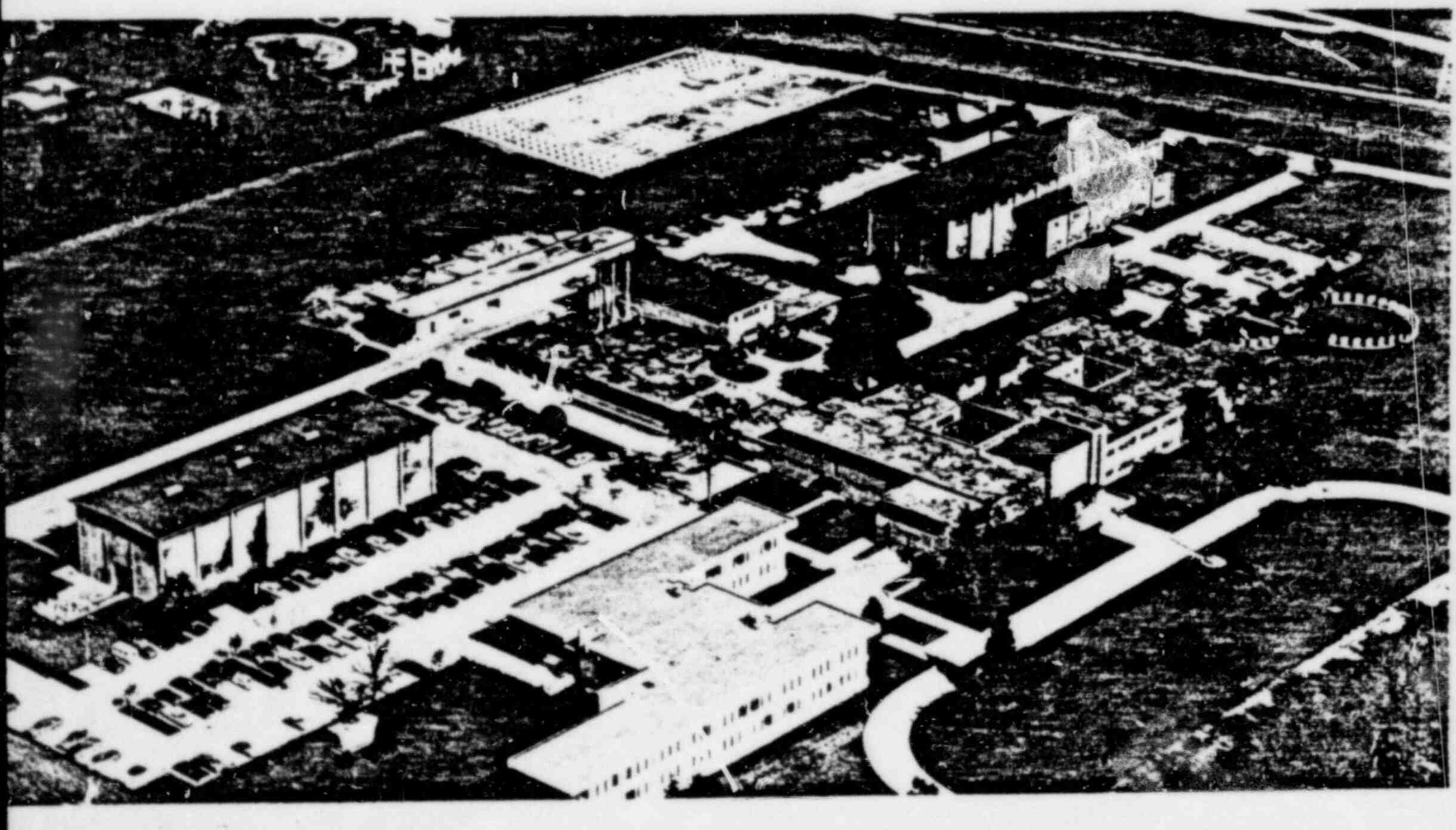
79. Aristizabal-Ochoa, J. D., Fiorato, A. E., and Corley, W. G., "Tension Lap Splices Under Severe Load Reversals," published in Proceedings of Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, September 1980, Vol. 7, pp. 55-62.

80. Oesterle, R. G., Fiorato, A. E. and Corley, W. G., "Reinforcement Details for Earthquake-Resistant Structural Walls," Concrete International: Design and Contruction 2 (12) 55-66, Dec. 1980. 12 refs.

construction technology laboratories

a Division of the PORTLAND CEMENT ASSOCIATION

EVALUATION OF CRACKING IN
SERVICE WATER PUMP STRUCTURE
AT MIDLAND PLANT



Report to

CONSUMERS POWER COMPANY
Jackson, Michigan

EVALUATION OF CRACKING IN SERVICE WATER PUMP STRUCTURE AT MIDLAND PLANT

by

W. G. Corley and A. E. Fiorato

Submitted by

CONSTRUCTION TECHNOLOGY LABORATORIES
A Division of the Portland Cement Association
5420 Old Orchard Road
Skokie, Illinois 60077

February 1982

TABLE OF CONTENTS

EVALUATION OF CRACKING IN SERVICE WATER PUMP STRUCTURE 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 Service Water Pump Structure located at Midland Nuclear Power Plant Units 1 and 2. Observed cracks in the structure are described and significance of the cracks with regard to future load carrying capacity is discussed. In addition, a program for monitoring structural integrity during implementation of remedial measures is described. Remedial measures include underpinning the north portion of the structure.

DESCRIPTION OF STRUCTURE

A site plan for the Midland Plant is shown in Fig. 1. The Service Water Pump Structure is located east of the turbine building adjacent to the emergency cooling water reservoir. The structure contains water-filled reservoirs and five pumps that provide water to cool equipment components during normal plant operation. These pumps also supply water to several

*Respectively, Divisional Director, Engineering Development Division, and Director, Construction Methods Department, Construction Technology Laboratories, A Division of the Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077

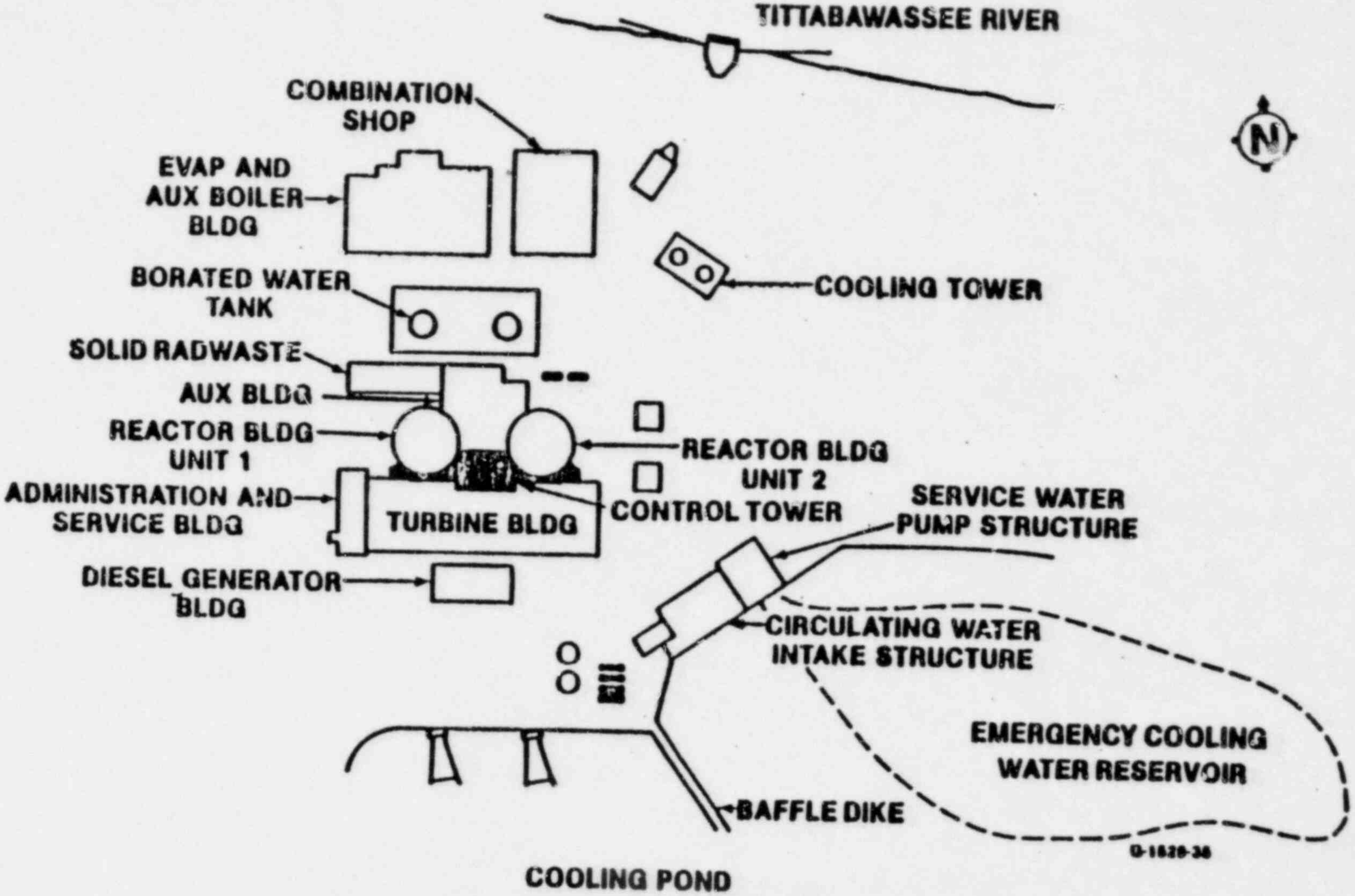


FIG. 1 SITE PLAN

safety-related cooling systems that are required to function during a design basis accident. Because of its safety-related function, the building is a Seismic Category 1 structure. As such it must maintain its integrity during and after a design basis accident including a postulated safe shutdown earthquake.

A section through the Service Water Pump Structure is shown in Fig. 2. The lower foundation slab of the structure at elevation 587.0 ft is 5-ft thick. This slab is founded on undisturbed natural material. A second foundation slab at elevation 617.0 ft is 3-ft thick. This slab is founded on backfill soil. Both slabs are locally thickened near sumps.

The south wall of the structure is adjacent to the cooling pond. Approximate high point elevation of the water level is 627.0 ft. Top of grade elevation on the north side of the structure is 634.0 ft. Roof elevation is 656.0 ft.

A plan of the Service Water Pump Structure at elevation 634.5 ft is shown in Fig. 3. The structure is rectangular in plan with overall dimensions of 86.0 by 106.0 ft at elevation 634.5 ft.

Figure 4 shows wall designations that will be used to describe the structure in this report. The east and west walls are main structural walls in the north-south direction. It should be noted that the center east wall extends from elevation 620.0 ft to the roof while the center west wall extends from elevation 634.5 ft to the roof.

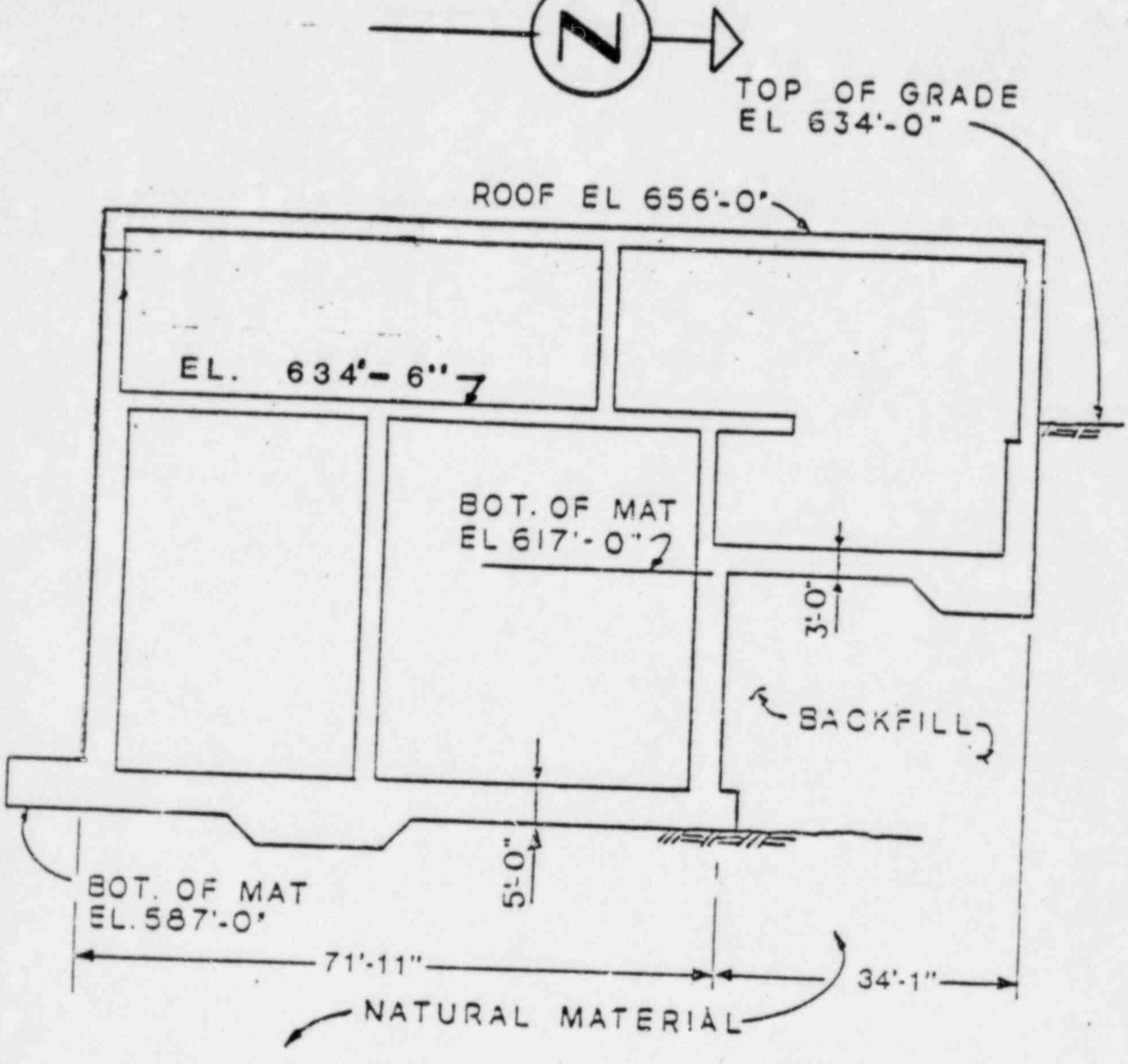


Fig. 2 Service Water Pump Structure Section

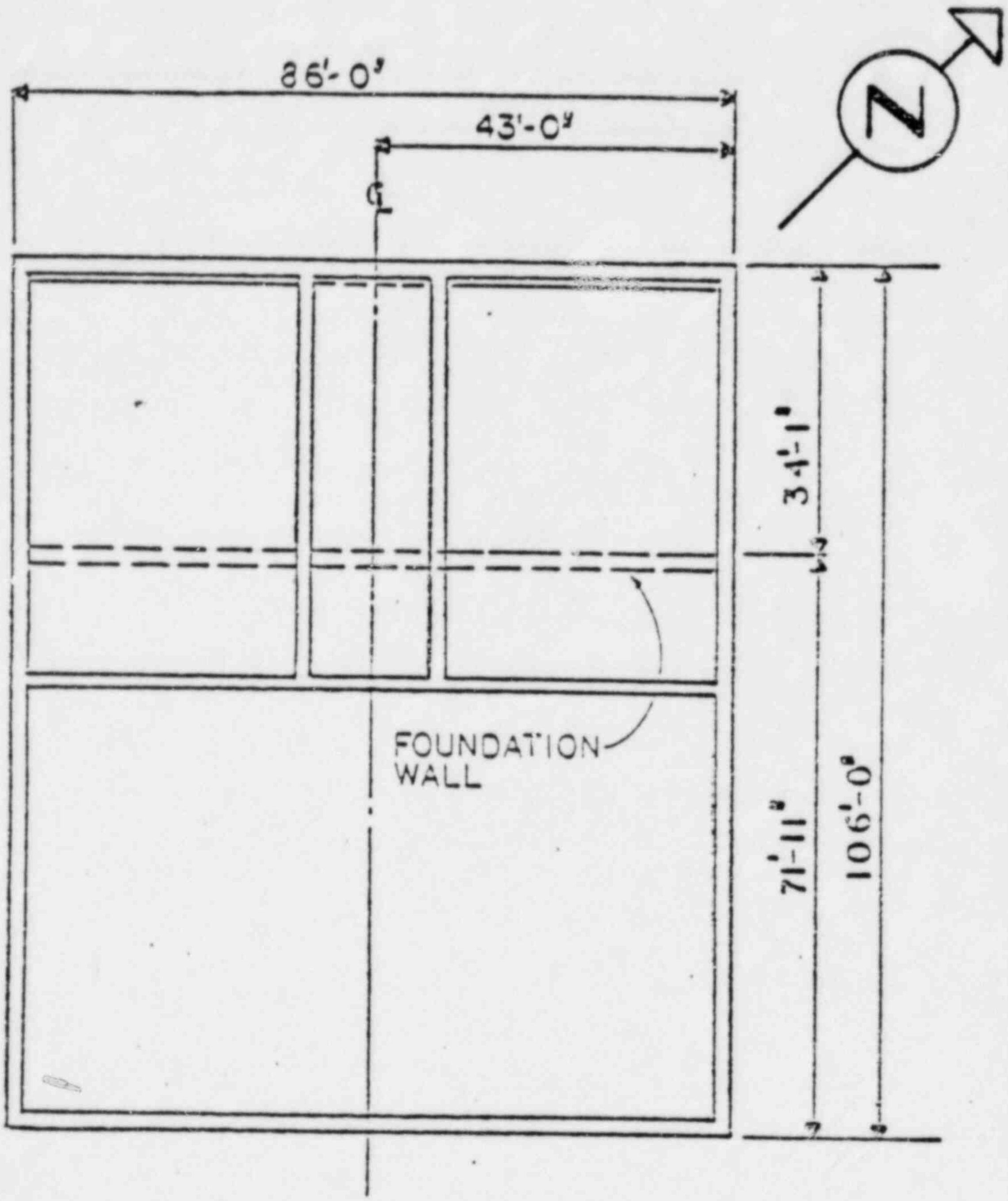


Fig. 3 Service Water Pump Structure Plan at Elevation 634'-6"

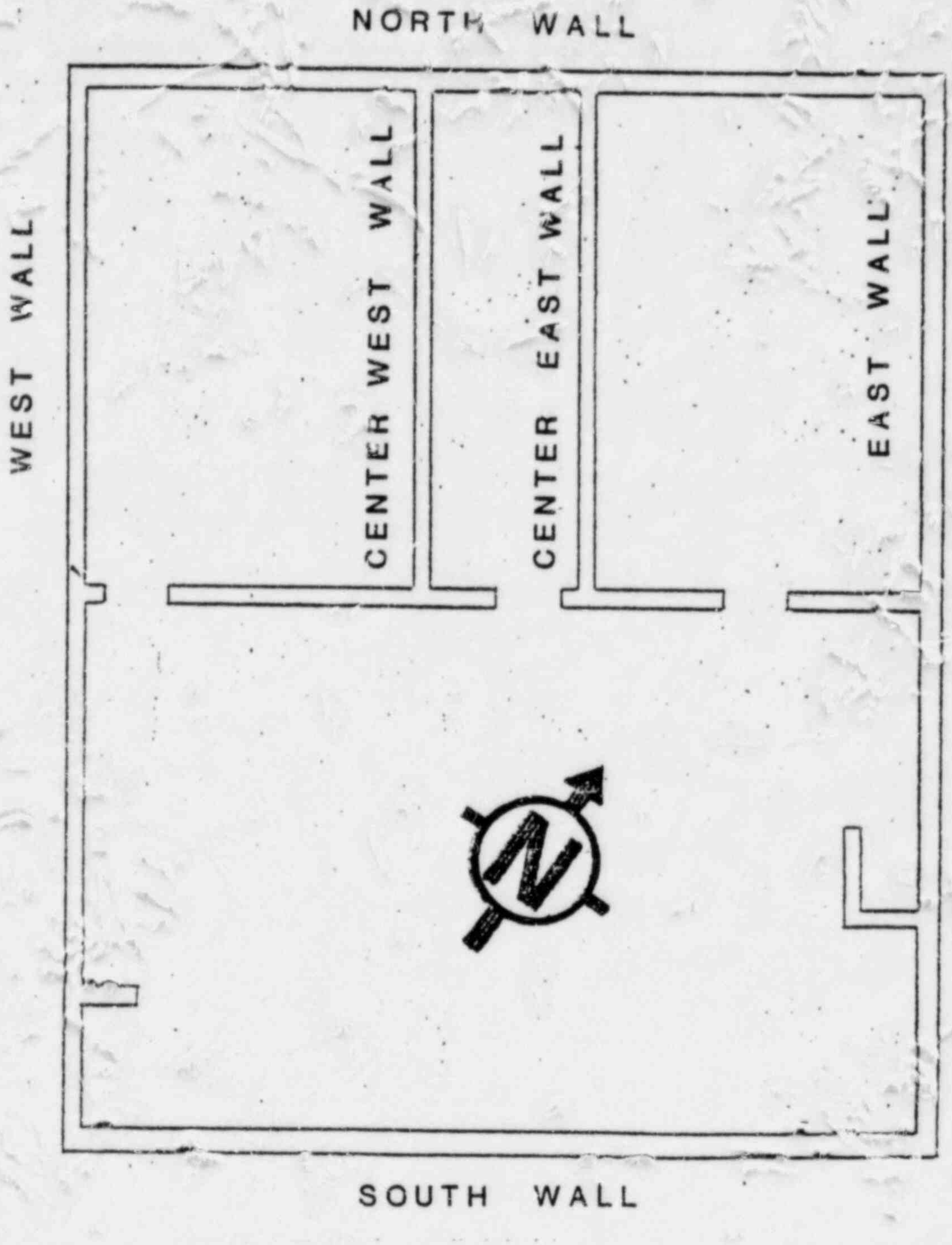


Fig. 4 Wall Designations for Service Water Pump Structure at Elevation 634'-6"

Exterior and interior walls of the Service Water Pump Structure are constructed of reinforced concrete. Figure 5 and Table 1 show details of selected walls in the structure. Table 2 contains a listing of drawings used to obtain data on member dimensions, and on amounts and arrangement of reinforcement.

The east, west, and south walls of the structure are 3.25-ft thick below elevation 634.5 ft. Above this elevation the walls are 2.0-ft thick. The north wall of the structure is 2.75-ft thick below elevation 632.75 ft. Above this level it is 2.0-ft thick. The center east and center west walls are 1.5-ft-thick.

The reinforcement ratio in exterior walls above elevation 634.5 ft is 0.0041 or greater. Additional reinforcement details are given in Table 1 and in drawings listed in Table 2.

Specified concrete strength for the Service Water Pump Structure is 4000 psi. Grade 60 reinforcement is used in the structure.

As indicated in Fig. 2, the north portion of the Service Water Pump Structure is founded on backfill. As a result of unsatisfactory backfill performance under other buildings at the Midland Plant, settlements occurring in the Service Water Pump Structure have been monitored. Although settlement measurements have not indicated the presence of significant differential movements in the building, the observation of cracks in walls of the structure has led to questions regarding future structural integrity. The following sections provide data on observed cracks in the structure and an evaluation of the significance of those cracks with regard to future load carrying capacity.

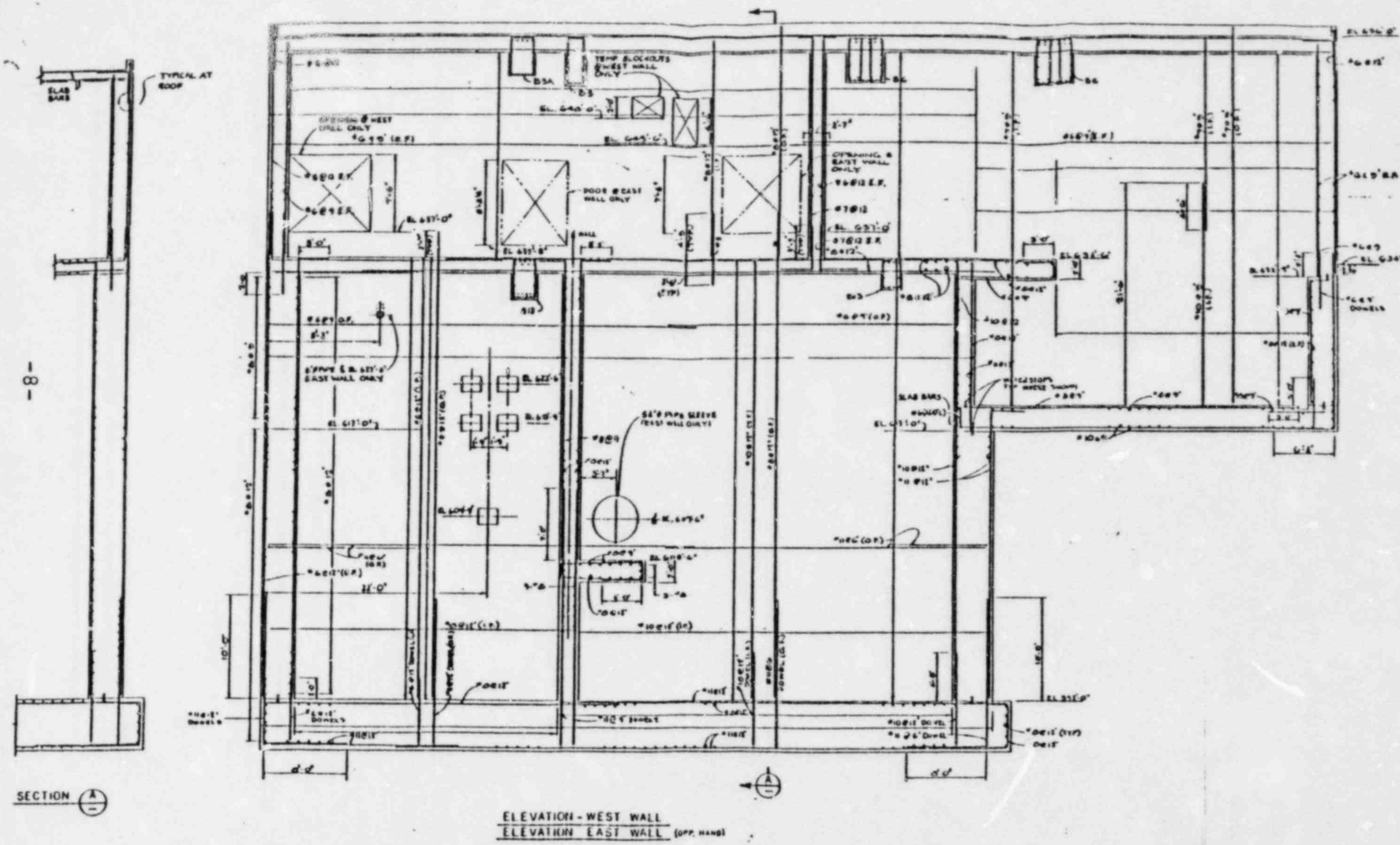


Fig. 5 Reinforcement in Service Water Pump Structure

TABLE 1 DETAILS OF SELECTED WALLS IN SERVICE WATER PUMP STRUCTURE

Wall Description	Wall Thickness, ft	Primary Vertical Reinforcement*	Primary Horizontal Reinforcement*
1. East and West Walls			
(a) North Portion Above El. 620'-0"	2.0	No. 7@ 9"(OF) No. 7@ 9"(IF) Above El.639'-3" No. 10@ 9"(IF) Below El.639'-3"	No. 6@ 9"(EF)
(b) South Portion Above El. 634'-6"	2.0	No. 8@12"(EF)	No. 6@ 9"(EF)
(c) North Portion Below El. 634'-6"	3.25	No. 8@12"(OF) No. 10@12"(IF)	No. 6@ 9"(OF) Above El.617'-0" No. 11@ 6"(OF) Below El.617'-0" No. 10@12"(IF)
(d) South Portion Below El. 634'-6"	3.25	No. 6@12"(EF)	No. 6@ 9"(OF) Above El.617'-0" No. 11@ 6"(OF) Below El.617'-0" No. 8@12"(IF)
2. Center East Wall			
(a) Above El. 634'-6"	1.5	No. 7@ 9"(EF)	No. 6@12"(EF)
(b) Below El. 634'-6"	1.5	No. 10@ 9"(EF)	No. 6@12"(EF)
3. Center West Wall	1.5	No. 6@18"(EF)	No. 6@18"(EF)
4. North Wall			
(a) Above El. 632'-9"	2.0	No. 6@ 9"(EF)	No. 6@ 9"(EF)
(b) Below El. 632'-9"	2.75	No. 11@ 9"(OF) No. 8@ 9"(IF)	No. 8@12"(EF)
5. South Wall			
(a) Above El. 634'-6"	2.0	No. 8@12"(EF)	No. 6@ 9"(EF)
(b) Below El. 634'-6"	3.25	No. 6@12"(EF)	No. 8@ 9"(OF) Above El. 617'-0" No. 8@12"(OF) Below El. 617'-0" No. 8@12"(IF)

* OF = Outside Face; IF = Inside Face; EF = Each Face

TABLE 2

SERVICE WATER PUMP STRUCTURE DRAWINGS

Bechtel Drawing No.	Revision No.	Date	Title
C-80	4	11/7/78	Reinforcing Slab El 634'-6"
C-86	6	12/8/79	Miscellaneous Steel and Concrete Details - Sheet 2
C-94	12	6/25/81	Concrete Floor Plans at El 592'-0" and El 634'-6"
C-95	3	9/20/78	Concrete Plan at El 656'-0" and Sections
C-96	8	1/8/80	Concrete Sections
C-97	5	1/8/80	Concrete Sections and Details, Sheet 1
C-99	7	5/19/80	Miscellaneous Steel and Concrete Details - Sheet 1
C-140	14	2/14/81	Reinforced Concrete General Notes and Details - Sheet 1

EVALUATION OF CRACKING

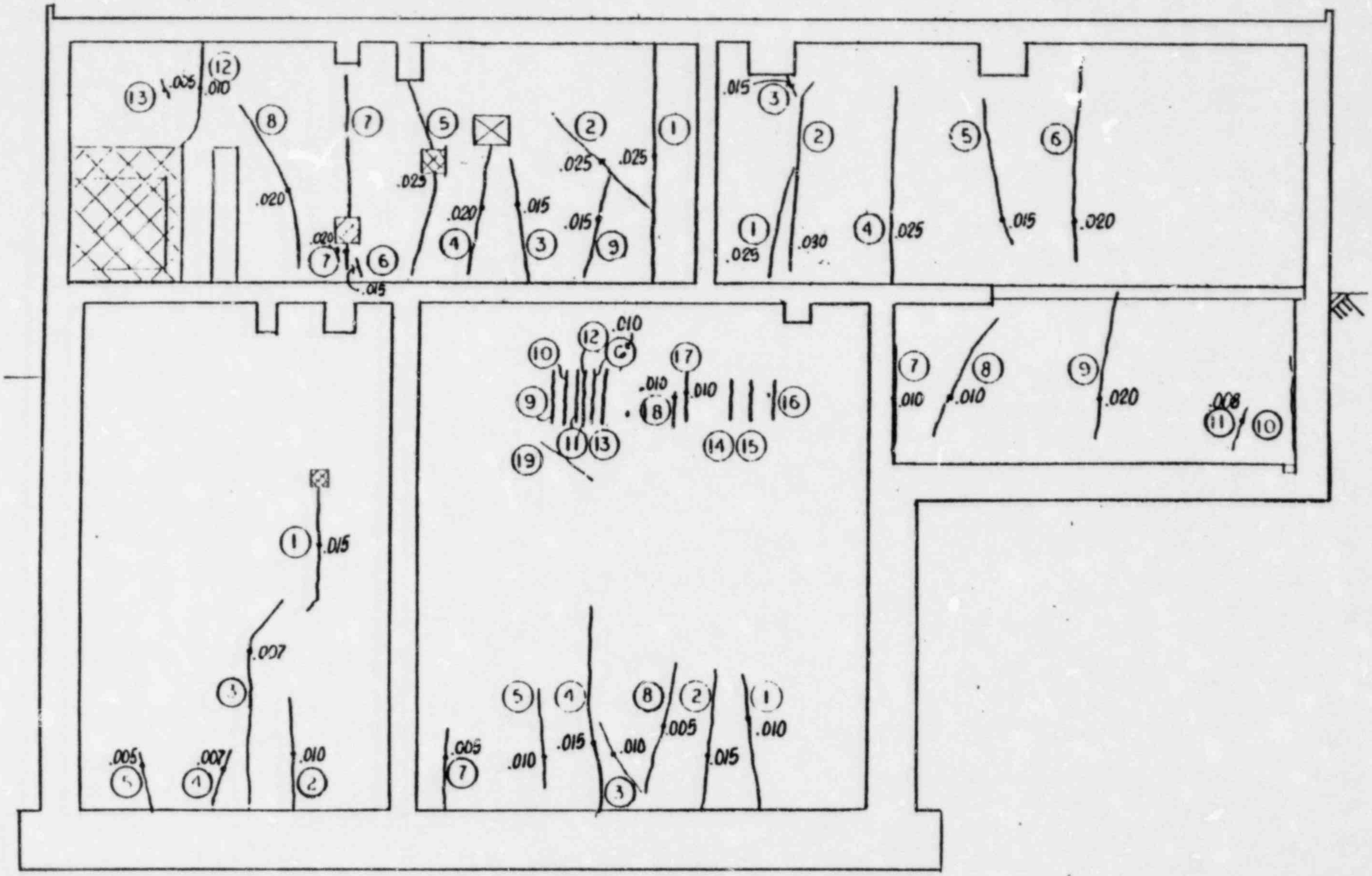
It has been hypothesized that cracking observed in the walls of the Service Water Pump Structure is related to two factors. The first is normal cracking that can occur from restrained volume changes in reinforced concrete. The second is cracking that could occur if the north portion of the structure, which is founded on fill, settles more than the south portion of the structure, which is founded on natural soil.

In this report, evaluation of cracking is based on review of crack mapping reported by Bechtel and on inspection of selected areas by personnel of the Construction Technology Laboratories (CTL). Areas inspected by CTL personnel were selected because of a need to obtain data that would clarify if cracking is related to hypothesized differential settlement in the structure. Therefore, primary emphasis was given to inspection of the east wall and the west wall. These walls were selected because they are the only load-carrying walls that continue through the entire north-south length of the structure.

Bechtel Crack Mapping

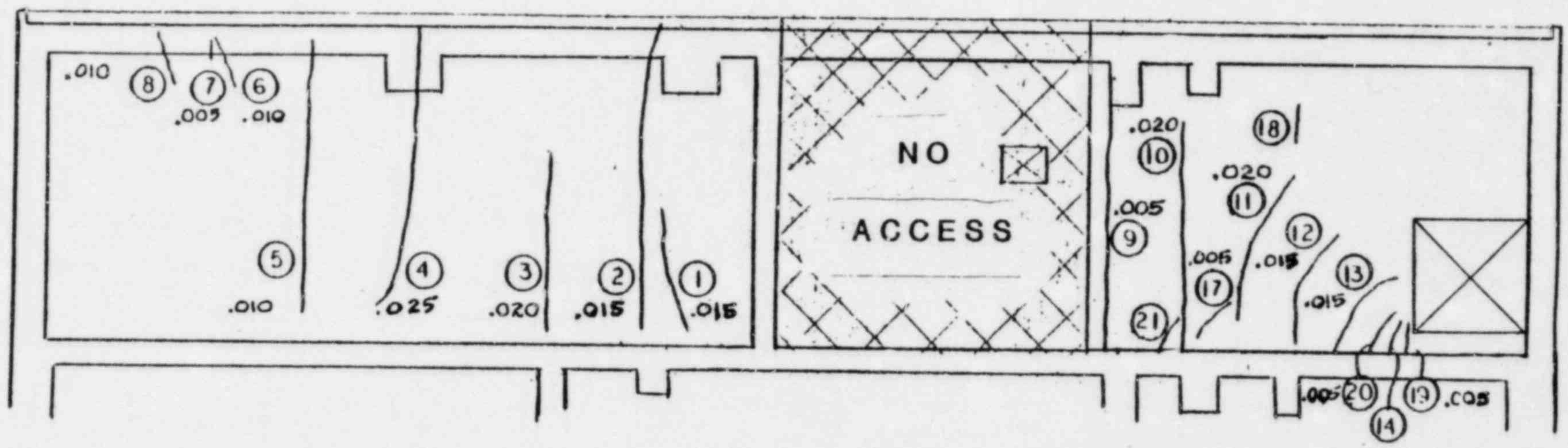
Cracks in walls of the Service Water Pump Structure were mapped by Bechtel personnel at several stages of construction.* Figure 6 shows cracking observed in west and east walls during a survey conducted in October and November 1981. A key to wall

*Crack widths shown in this report are from measurements made prior to installation of temporary post-tensioning which has subsequently been applied as part of the remedial measures to underpin the structure.



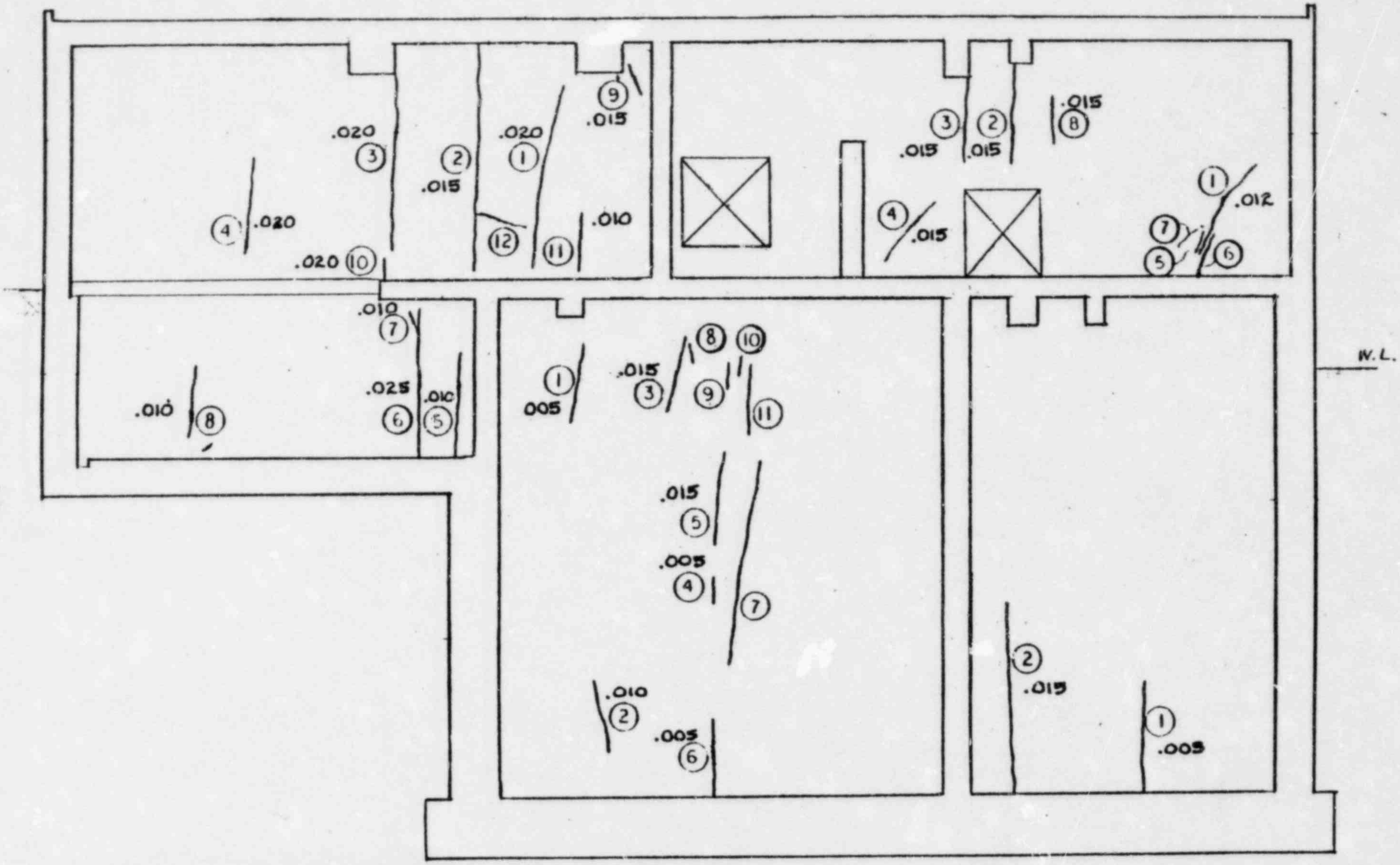
WEST WALL - EAST FACE

Fig. 6(a) Cracking in East Face of West Wall of Service Water Pump Structure from Bechtel Drawing C-2040 Revision A (Crack Widths in Inches)



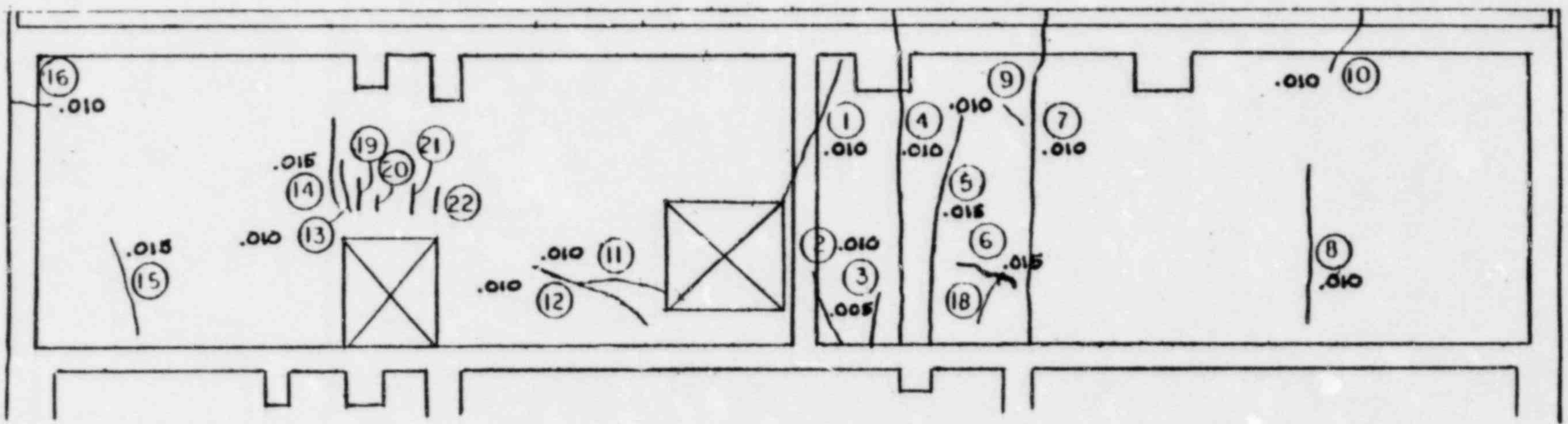
WEST WALL - WEST FACE

Fig. 6(b) Cracking in West Face of West Wall of Service Water Pump Structure from Bechtel Drawing C-2040 Revision A Crack Widths in Inches)



EAST WALL - WEST FACE

Fig. 6(c) Cracking in West Face of East Wall of Service Water Pump Structure from Bechtel Drawing C-2040 Revision A (Crack Widths in Inches)



EAST WALL - EAST FACE

Fig. 6(d) Cracking in East Face of East Wall of Service Water Pump Structure from Bechtel Drawing C-2040 Revision A (Crack Widths in Inches) designations is shown in Fig. 4. Numbers on the figure show crack widths in inches. Maximum reported crack widths are 0.030 in. in the west wall and 0.025 in. in the east wall of the structure. Cracks above elevation 634.5 ft run primarily vertically and are distributed over the entire horizontal length of the wall.

If cracks in the east and west walls were caused by settlement of the north end of the Service Water Pump Structure, a fan-shaped pattern radiating from the foundation wall 36.1 ft from the north end would be visible. No fan-shaped pattern is apparent. Rather, the cracks tend to be vertical at all locations along the walls.

Cracks observed in the center west and center east walls of the structure are shown in Fig. 7. These cracks were also mapped in October and November 1981. Maximum reported crack widths are 0.030 in the center west wall and 0.020 in the center east wall. The cracks are primarily vertical in direction and are distributed over the entire horizontal length of the center west and center east walls. The somewhat larger widths observed in the center west wall are consistent with the lower reinforcement ratio in this wall. Table 1 contains reinforcement details for both walls.

Figure 8 shows cracks observed in the south wall of the Service Water Pump Structure by Bechtel personnel in October and November 1981. Cracks on the south face of this wall were not mapped below the water line of the cooling pond. No cracks

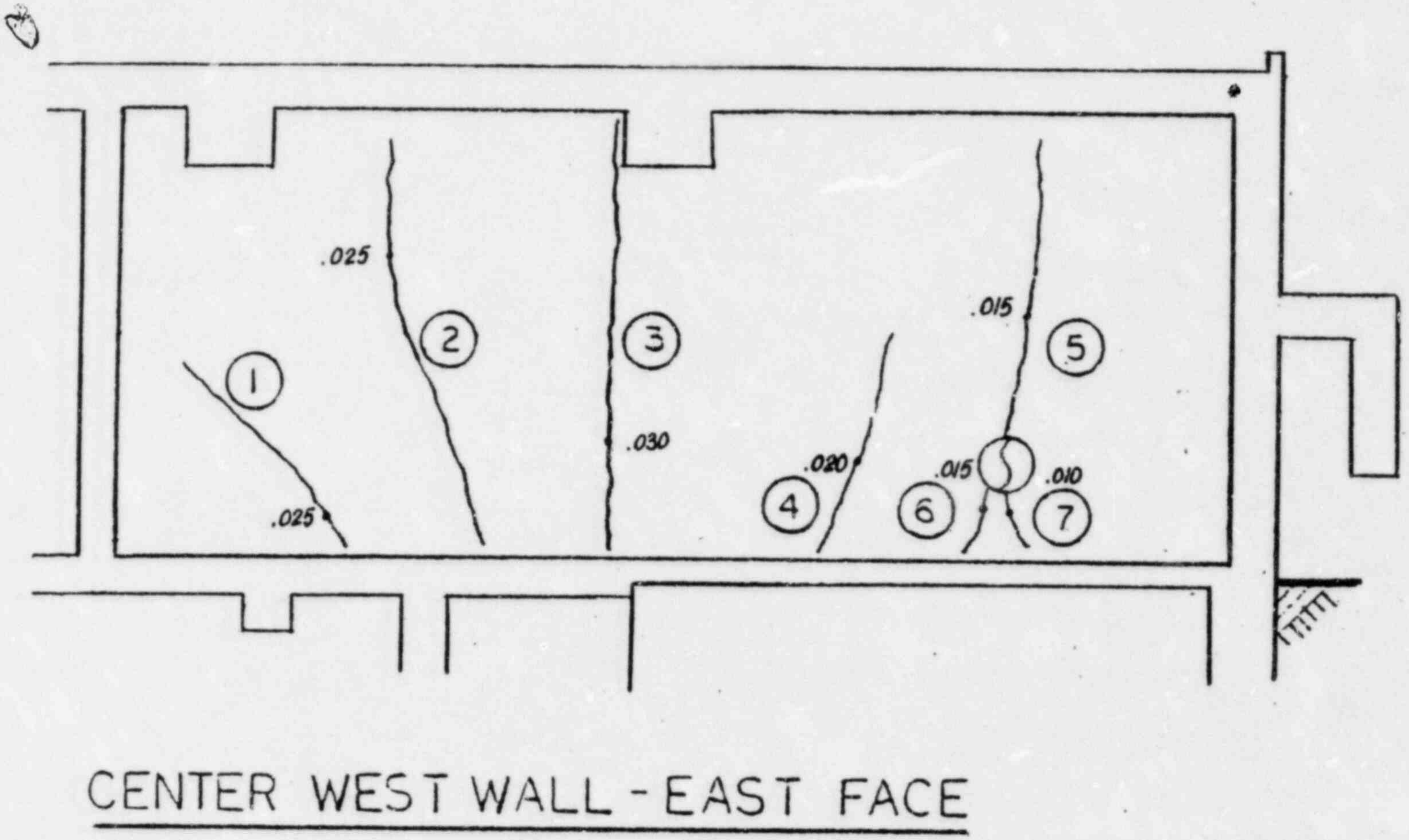
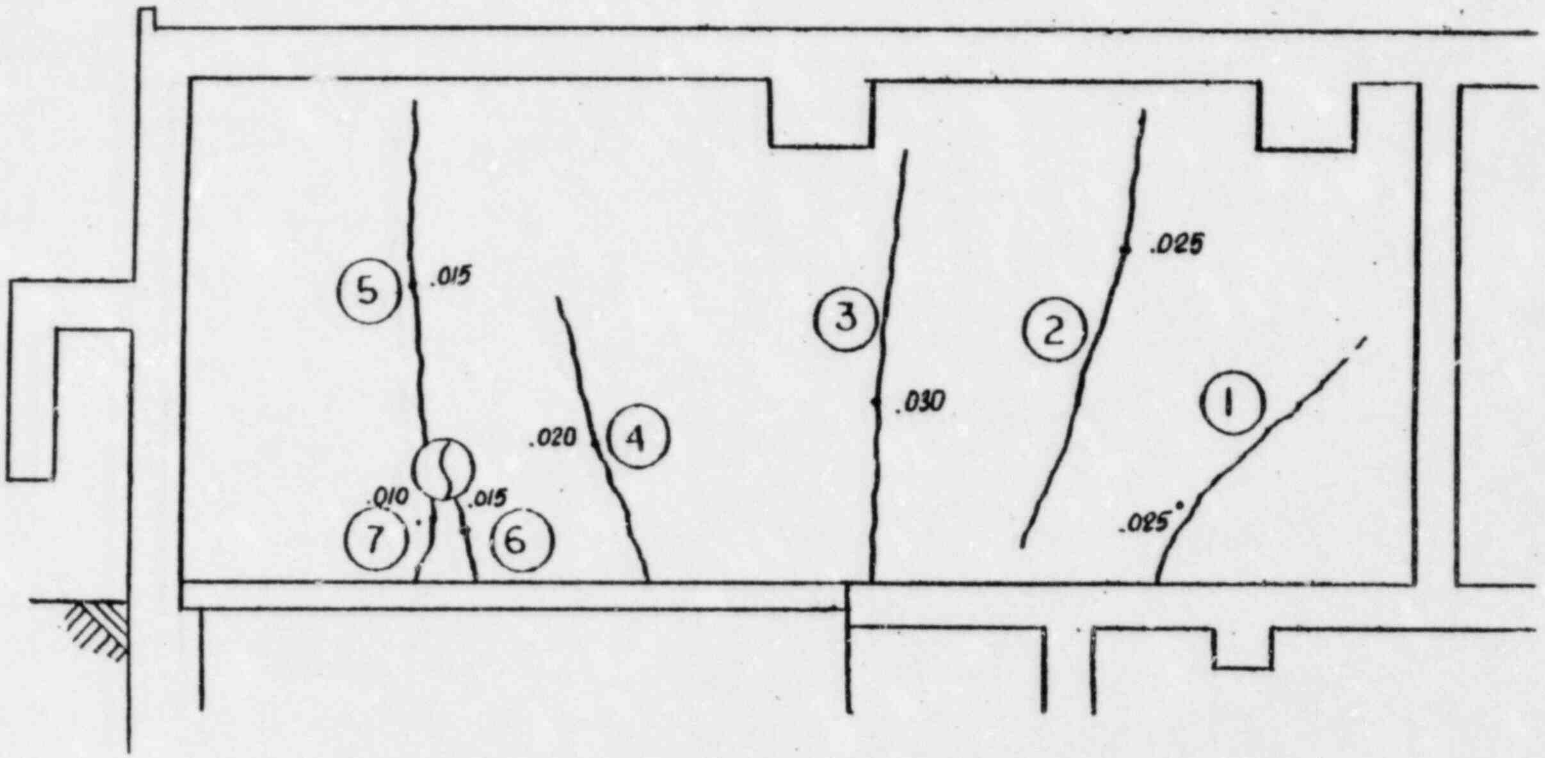
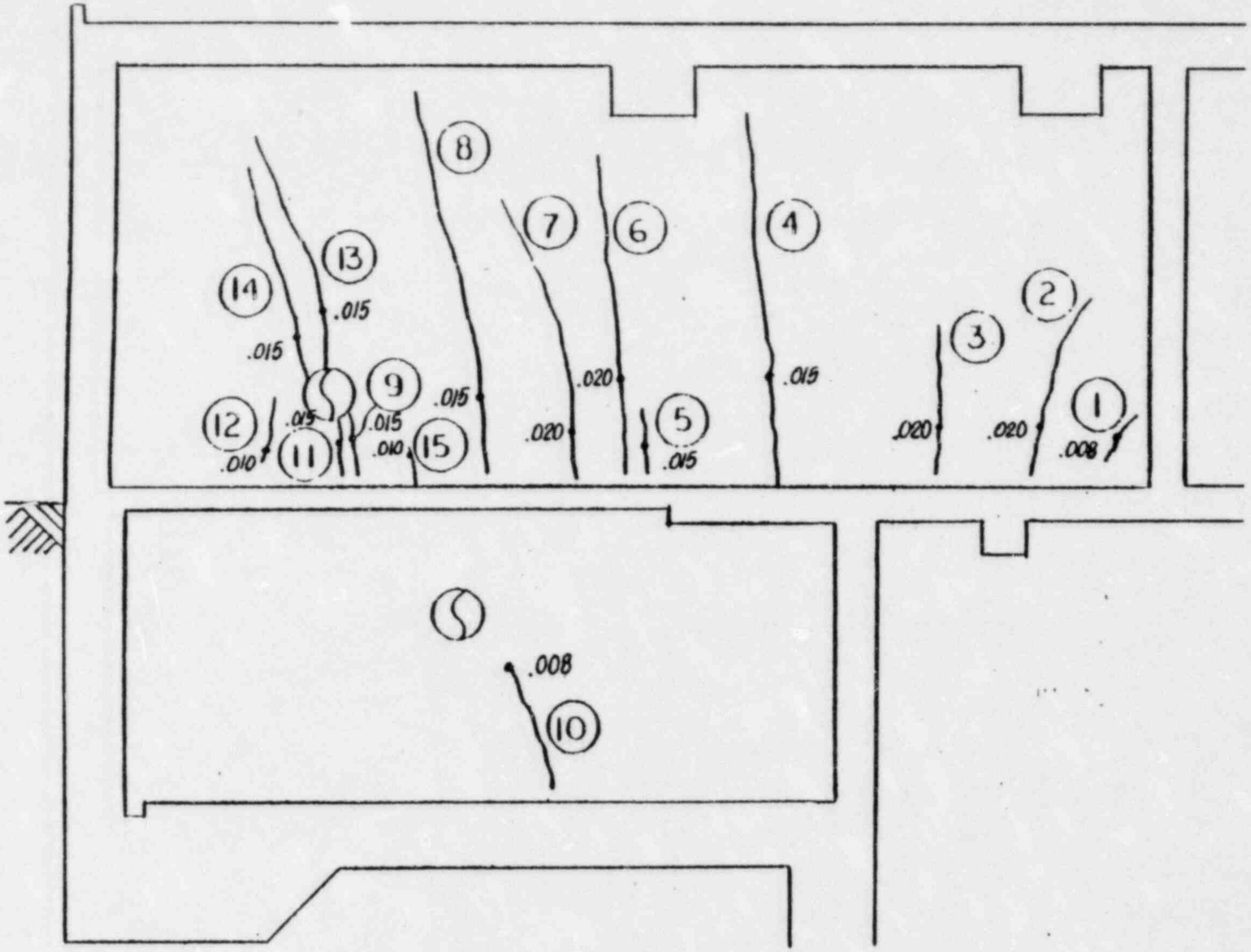


Fig. 7(a) Cracking in East Face of Center West Wall of Service Water Pump Structure from Bechtel Drawing C-2041 Revision A (Crack Widths in Inches)



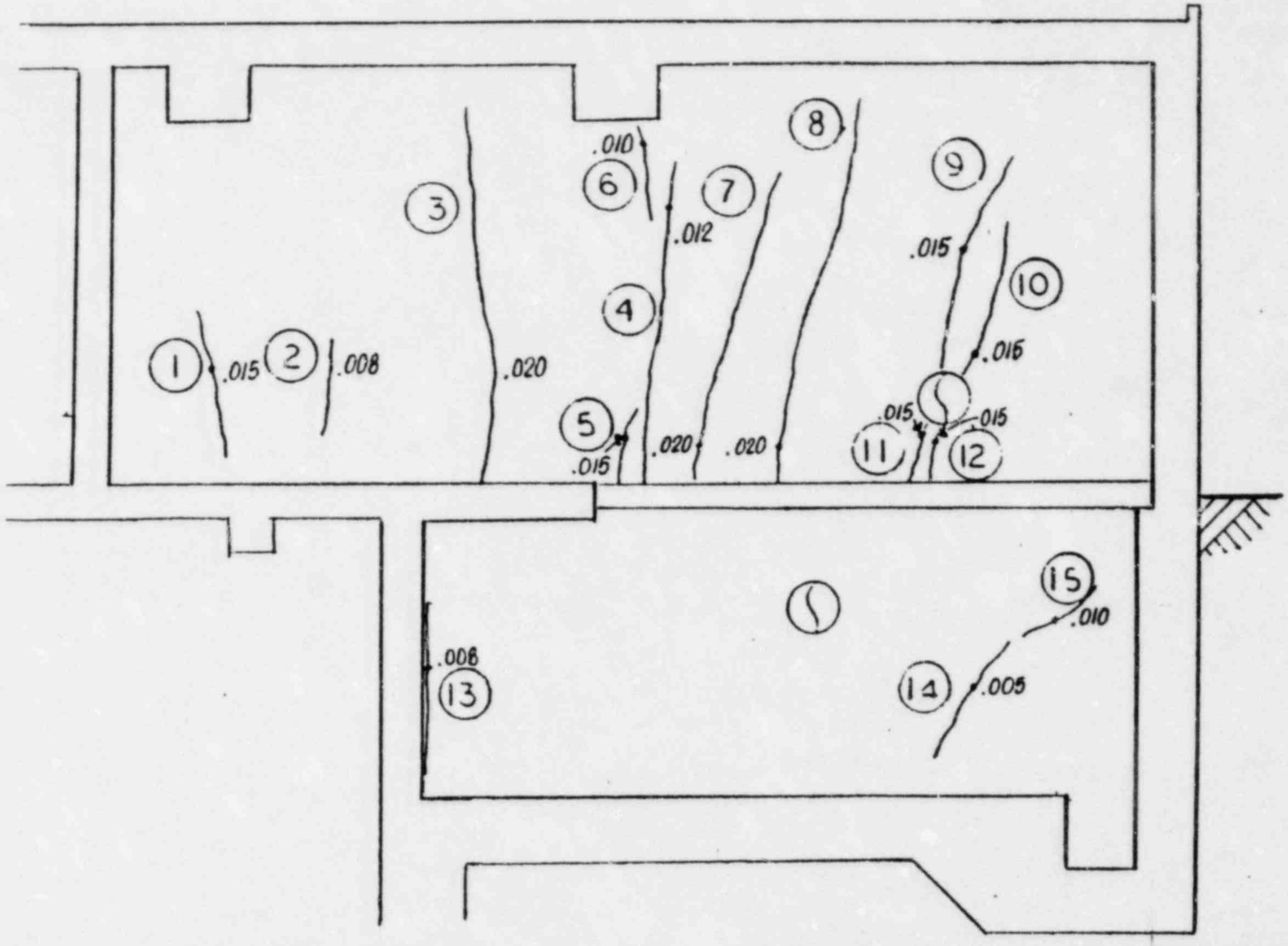
CENTER WEST WALL - WEST FACE

Fig. 7(b) Cracking in West Face of Center West Wall of Service Water Pump Structure from Bechtel Drawing C-2041 Revision A (Crack Widths in Inches)



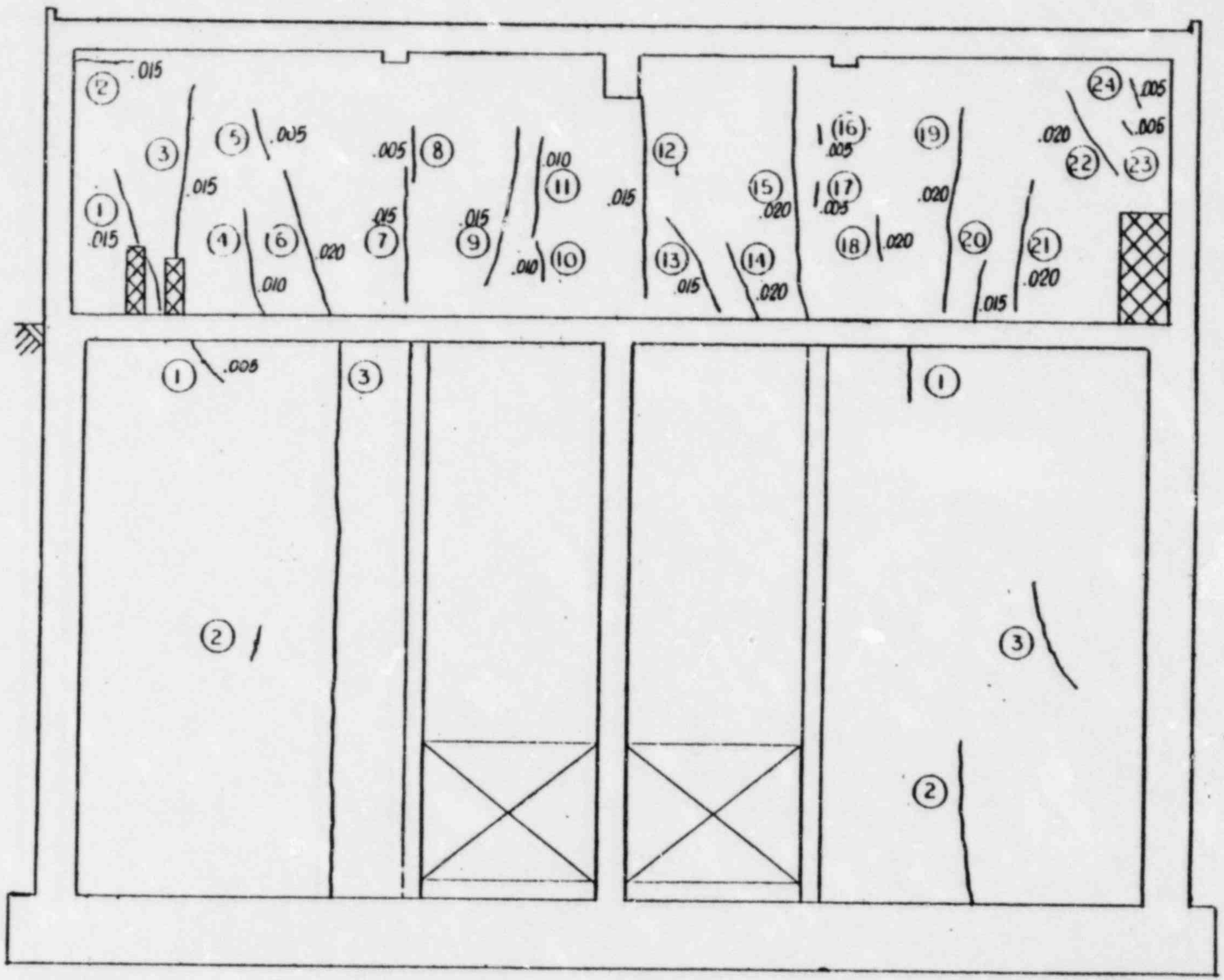
CENTER EAST WALL-WEST FACE

Fig. 7(c) Cracking in West Face of Center East Wall of Service Water Pump Structure from Bechtel Drawing C-2041 Revision A (Crack Widths in Inches)



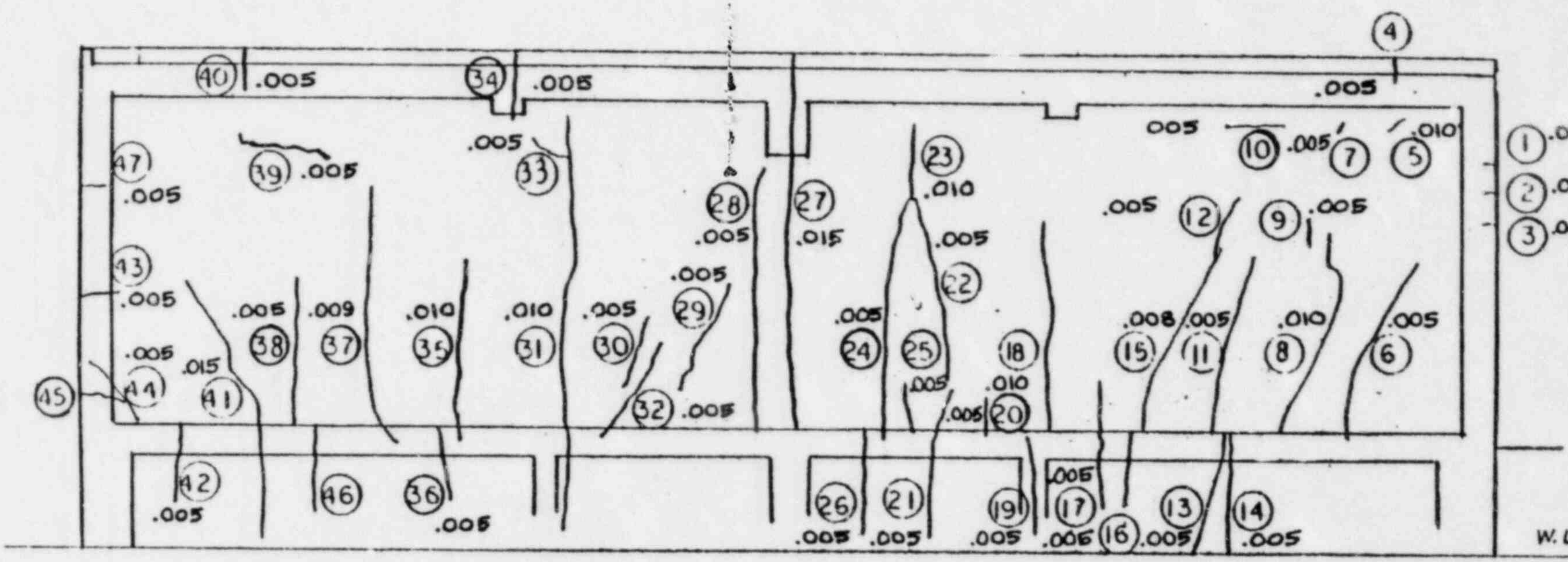
CENTER EAST WALL - EAST FACE

Fig. 7(d) Cracking In East Face of Center East Wall of Service Water Pump Structure from Bechtel Drawing C-2041 Revision A (Crack Widths in Inches)



SOUTH WALL - NORTH FACE

Fig. 8(a) Cracking in North Face of South Wall of Service Water Pump Structure from Bechtel Drawing C-2040 Revision A (Crack Widths in Inches)



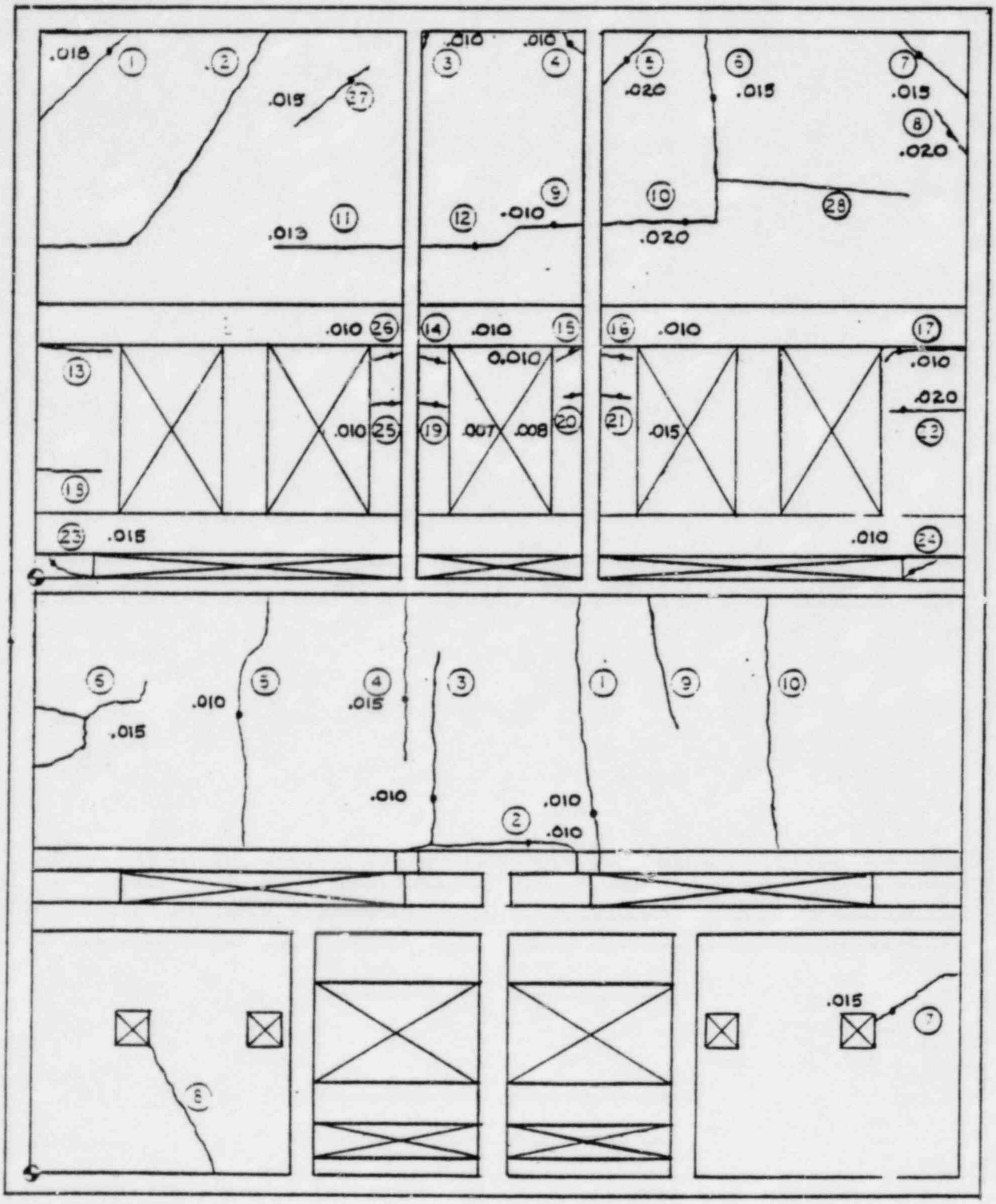
SOUTH WALL - SOUTH FACE

Fig. 8(b) Cracking in South Face of South Wall of Service Water Pump Structure from Bechtel Drawing C-2040 Revision A (Crack Widths in Inches) larger than 0.010 in. were recorded on the north face of the south wall below elevation 634.5 ft. Maximum observed crack width in the south wall was 0.020 in.

Figure 9 shows cracking in the roof of the Service Water Pump Structure as mapped by Bechtel personnel in October and November 1981. Maximum measured crack width reported for the underside of the roof is 0.020 in.

If the north end of the Service Water Pump Structure had settled more than the south end, and if such settlement was sufficient to induce cracking, it would be expected that a definite pattern of east-west cracks would show in the roof at the line where the north overhang starts. This location coincides with the series of five openings in the roof of the Service Water Pump Structure. These openings are covered by precast concrete panels. Some cracking in the cast-in-place concrete around the openings was recorded in the Bechtel survey. However, there is no clear pattern showing that these cracks are related to hypothesized building settlement. In fact, many of the cracks at this location occur at the intersection of members and at discontinuities in the roof.

Based on overall review of Bechtel drawings, it appears that cracks shown can be attributed primarily to restrained volume changes that occur in concrete after final set, and during subsequent curing and drying. The possibility that some cracks formed because of hypothesized differential settlement of the building cannot be completely excluded. However, the



ROOF SLAB UNDERSIDE PLAN

Fig. 9 Cracking in Underside of Roof Slab of Service Water Pump Structure from Bechtel Drawing C-2042 Revision A (Crack Widths in Inches) pattern and size of reported cracks does not entirely support the conclusion that settlement related cracking occurred.

CTL Observations

Visual observations of cracking in the Service Water Pump Structure were made by Dr. W. G. Corley of CTL on October 20, 1981 and December 3, 1981. At this time Dr. Corley did not do detailed mapping of cracks. Rather, the inspections were made to obtain an overall view of cracking in the structure and to correlate this with impressions obtained from review of Bechtel crack mapping drawings. In general, impressions obtained from visual inspections at the site were consistent with those obtained from review of the Bechtel drawings.

As noted earlier in this report, the east and west walls are the only continuous walls in the north-south direction of the building. Consequently, any cracks that could be attributed to hypothesized differential settlement would appear in these walls. For this reason, interior surfaces of the east and west walls were inspected in detail by CTL personnel on November 4 and 5, 1981. In addition, a general visual survey was made of selected walls, floor slabs, and roof slab surfaces. A general visual inspection of the exterior of the structure was also made. It should be noted that the top surface of the roof had been finished and the concrete slab surface was not visible.

General access to most areas inside the Service Water Pump Structure was difficult because of construction work in progress and because equipment already in place obstructed many areas. Inspection of wall areas was conducted from eye level at elevations 634.5 ft and 620.0 ft. Areas above eye level were inspected where access was available. General lighting in all areas of inspection was relatively poor. Therefore, primary light for inspection was provided by hand-held flashlights.

In addition to visual observations, widths of selected cracks were measured using a 50 power crack measuring microscope with a manufacturer's rated sensitivity of 0.001 in. Approximate crack locations were measured using commercial quality tape measures. These tapes provided accuracy of measurements well within that required to draw conclusions based on the results.

Figures 10 and 11 show cracks observed by CTL personnel in November 1981.* Cracks in the west and east walls of the Service Water Pump Structure are shown. The patterns of cracking are similar to those observed by Bechtel personnel. As shown in Fig. 10, maximum measured crack width in the west wall was 0.025 in. Maximum measured crack width in the east wall was 0.020 in.

Although cracks shown in Figs. 10 and 11 generally run in a vertical direction, there is no consistent trend of flexural cracking above the foundation wall where the north overhang intersects the remainder of the building. Larger measured widths in the vertical crack immediately above this foundation wall are attributed to the fact that there is a vertical construction joint at this location.

*Crack mapping undertaken by CTL personnel in November 1981 was done prior to installation of temporary post-tensioning which has subsequently been applied as part of the remedial measures to underpin the structure.

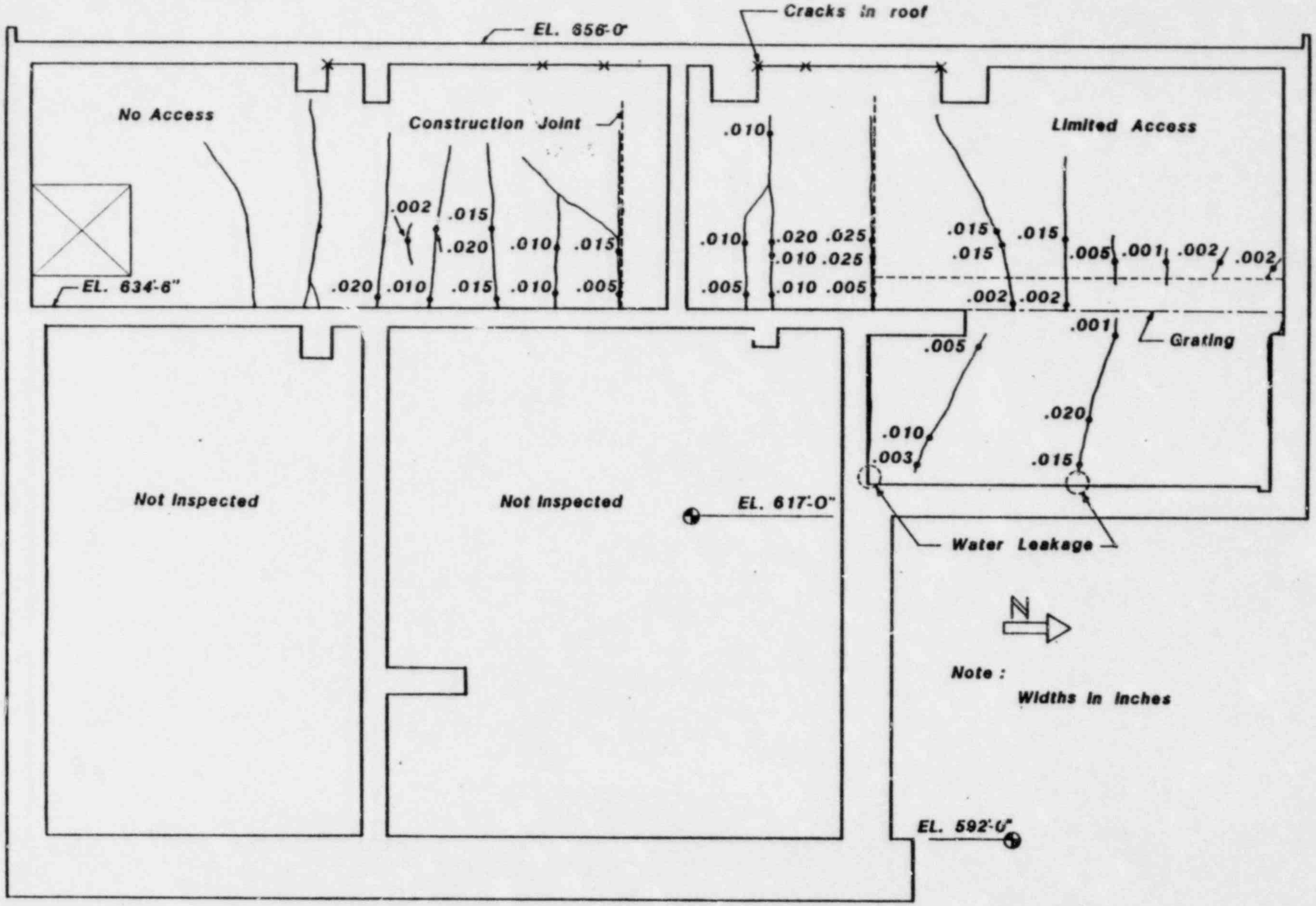


Fig. 10 Cracks Observed in West Wall of Service Water Pump Structure on November 5, 1981

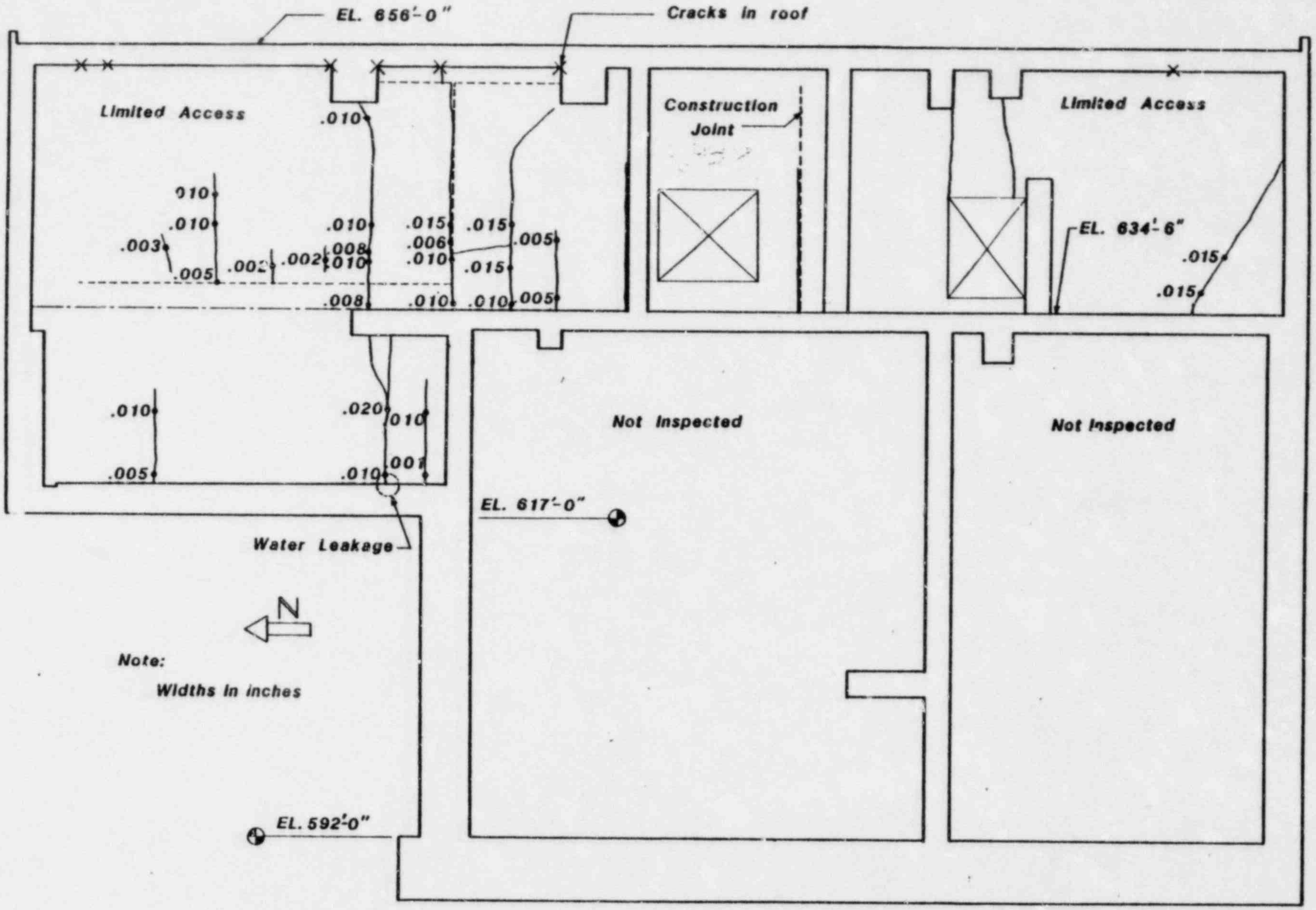


Fig. 11 Cracks Observed in East Wall of Service Water Pump Structure on November 4, 1981

The locations of vertical cracks in the east and west walls above elevation 634.5 ft are similar throughout the entire horizontal length of both walls. No fan-shapped pattern was seen radiating from the foundation wall where the overhanging north portion of the building intersects the remaining portion of the building. The hypothesis of concentrated bending at this intersection cannot be supported in the absence of the characteristic pattern of flexural cracking.

Based on crack data shown in Figs. 10 and 11, it cannot be concluded that differential settlement related cracking occurred in the Service Water Pump Structure. However, since the structure will be underpinned it is not necessary to make a more detailed analysis to determine the precise cause of observed cracks. Qualitatively, it appears that the cracking can be attributed to restrained volume changes caused by temperature and shrinkage of wall concrete combined with restraining effects of floor slabs.

SIGNIFICANCE OF CRACKS

Cracks observed in the Service Water Pump Structure by CTL personnel are primarily attributed to volume changes that occur in concrete during curing and subsequent drying. No evidence of structural distress was observed. Although the possibility of stress related cracking because of differential building settlement cannot be completely eliminated, crack patterns do not support the conclusion that this mechanism was a primary cause of cracking.

As a measure of significance of observed cracks relative to future integrity of the structure*, the tensile stress that uncracked concrete may be assumed to carry was compared to available tensile capacity provided by structural reinforcement crossing the cracks. This calculation was made for sections in the vicinity of cracks that had a measured width of 0.010 in. or greater. Available structural reinforcement was determined from Bechtel drawings listed in Table 2. It should be noted that this calculation is not intended to imply a change in design criteria for the walls. Rather it is a means of estimating membrane capacity.

Table 3 summarizes the comparison of "tensile capacity" for walls in which cracks larger than 0.010 in. were observed. In the calculation, concrete is assumed to carry a principle tensile stress of $4\sqrt{f'_c}$ where f'_c is specified concrete compressive strength. This assumption is consistent with Section 11.4.2.2 of the ACI Building Code.(1)** Resistance of reinforcement was calculated as $A_s f_y$ where

A_s = area of reinforcement and

f_y = specified yield stress of reinforcement.

If calculated resistance provided by reinforcement crossing the crack equals $4\sqrt{f'_c}$ there is sufficient reinforcement to carry the stress that may be attributed to concrete. As indicated in

* A general discussion of strength of cracked reinforced concrete members is given in Appendix A.

**Superscript numbers in parentheses refer to references listed at the end of this report.

TABLE 3 - AVAILABLE "TENSILE CAPACITY" AT SELECTED CRACK LOCATIONS IN PRIMARY LOAD BEARING WALLS.

Wall Designation	$4\sqrt{f'_c}\,A_g$ (kips)	$A_s f_y$ (kips) Vertical	$A_s f_y$ (kips) Horizontal
1. East and West Walls			
(a) North Portion Above El. 620'-0"	72.9	96.0	70.4
(b) South Portion Above El. 634'-6"	72.9	94.8	70.4
2. Center East Wall			
(a) Above El. 634'-6"	54.6	96.0	52.8
(b) Below El. 634'-6"	54.6	203.2	52.8
3. Center West Wall	54.6	35.2	35.2

Table 3 resistance provided by available horizontal reinforcement in upper levels of the east and west walls is only slightly less than the tensile stress assumed to be carried by the concrete. Resistance provided by the vertical reinforcement exceeds the tensile stress assumed to be carried by concrete.

Vertical and horizontal forces on the east and west walls would result in a principal stress direction that is inclined from vertical. Both vertical and horizontal reinforcement will provide tensile resistance across inclined cracks. A crack inclination of only 15 degrees from vertical would mobilize enough vertical reinforcement force to exceed the stress attributed to concrete tensile strength. Therefore, it is concluded that resistance provided by the reinforcement is sufficient.

The comparison of reinforcement resistance to assumed concrete tensile stress in the center east wall is similar to that observed for the east and west walls. Tensile resistance of the vertical reinforcement exceeds that attributed to the concrete while that in the horizontal reinforcement is slightly less. As was discussed for the east and west walls, it can be concluded that the reinforcement resistance is satisfactory.

Horizontal and vertical reinforcement resistance in the center west wall is approximately 65 percent of the tensile stress assumed to be carried by the concrete. It should be noted, however, that the center west wall extends only from elevation 634.5 ft to the roof. This wall is not a primary load resisting wall for the structure.

As an additional check on influence of cracking on the center west wall, an analysis was made to estimate two limits on capacity. First the wall was assumed to act as a horizontal cantilever with a concentrated vertical force at its exterior end. Based on sectional analysis, flexural capacity of the wall was calculated. This flexural capacity defines an upper limit on the amount of vertical shear that can be induced in the wall. For assumed conditions, it was estimated that the maximum nominal vertical shear stress which can be induced in the wall is $1.8\sqrt{f'_c}$. Available shear capacity would be in excess of $2.6\sqrt{f'_c}$ which is provided by the reinforcement. Thus, available shear capacity exceeds that needed to resist the potential maximum shear. It should be noted that assumptions for this calculation are very conservative in that underpinning support for the north end of the structure was neglected.

A second analysis was made to check shear capacity in the horizontal direction. A hypothetical horizontal force corresponding to a nominal shear stress of $2\sqrt{f'_c}$ was applied to the top of the center west wall.* The wall was assumed to be supported at its south end at elevation 634.5 ft. Based on building geometry, it was estimated that application of the hypothetical force would cause flexural distress or even uplift at the north end of the wall. Thus, shear could not limit wall capacity.

*Note that available shear capacity exceeds the nominal stress of $2.6\sqrt{f'_c}$ provided by the reinforcement.

Based on a conservative analysis of limits on capacity, it can be concluded that observed cracks do not affect shear strength of the center west wall.

RECOMMENDED PROGRAM FOR MONITORING STRUCTURAL INTEGRITY

As part of remedial measures to eliminate the possibility of unsatisfactory foundation conditions, the north portion of the Service Water Pump Structure will be underpinned as shown in Fig. 12. During underpinning operations, movement of the structure should be monitored. Monitoring operations should include periodic measurement of structure displacements and periodic visual inspection for cracking.

Displacement Monitoring

A time history of displacements of the Service Water Pump Structure should be maintained during underpinning operations. It is recommended that displacement readings be taken at selected construction milestones with a maximum interval of one week.

Displacement measurements should be made to monitor absolute movement and relative distortions of structural elements. Figure 13 shows approximate locations of recommended displacement measurement points. Designation of absolute and relative measurement points will be completed as part of the overall monitoring plan prior to start of underpinning operations.

Displacement measurements should be recorded as a function of time for the duration of underpinning operations. Significant construction milestones should be identified at appropriate time intervals. Prior to start of underpinning, limiting distortion criteria should be selected so that critical defor-

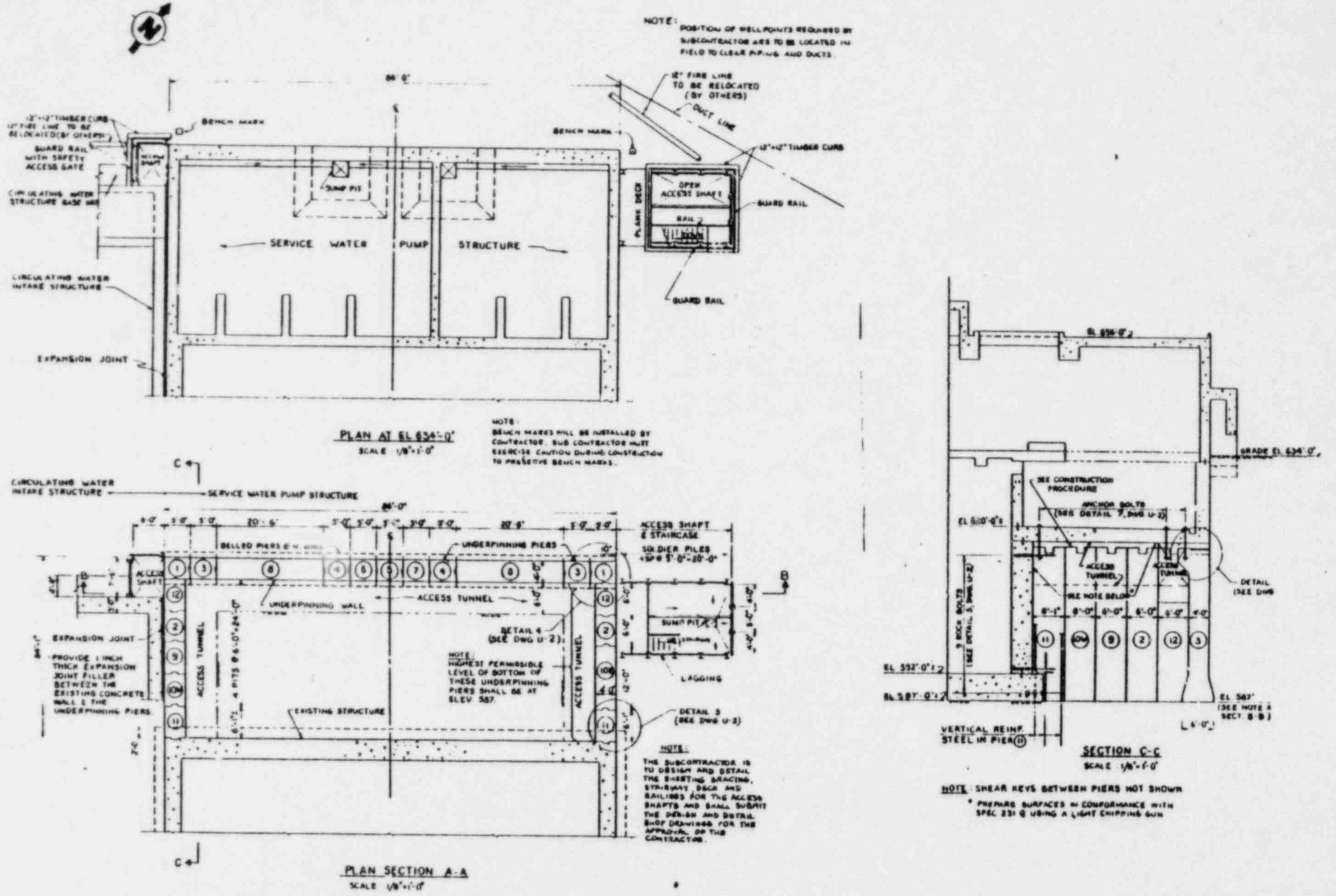


Fig. 12 Underpinning Plan for Service Water Pump Structure

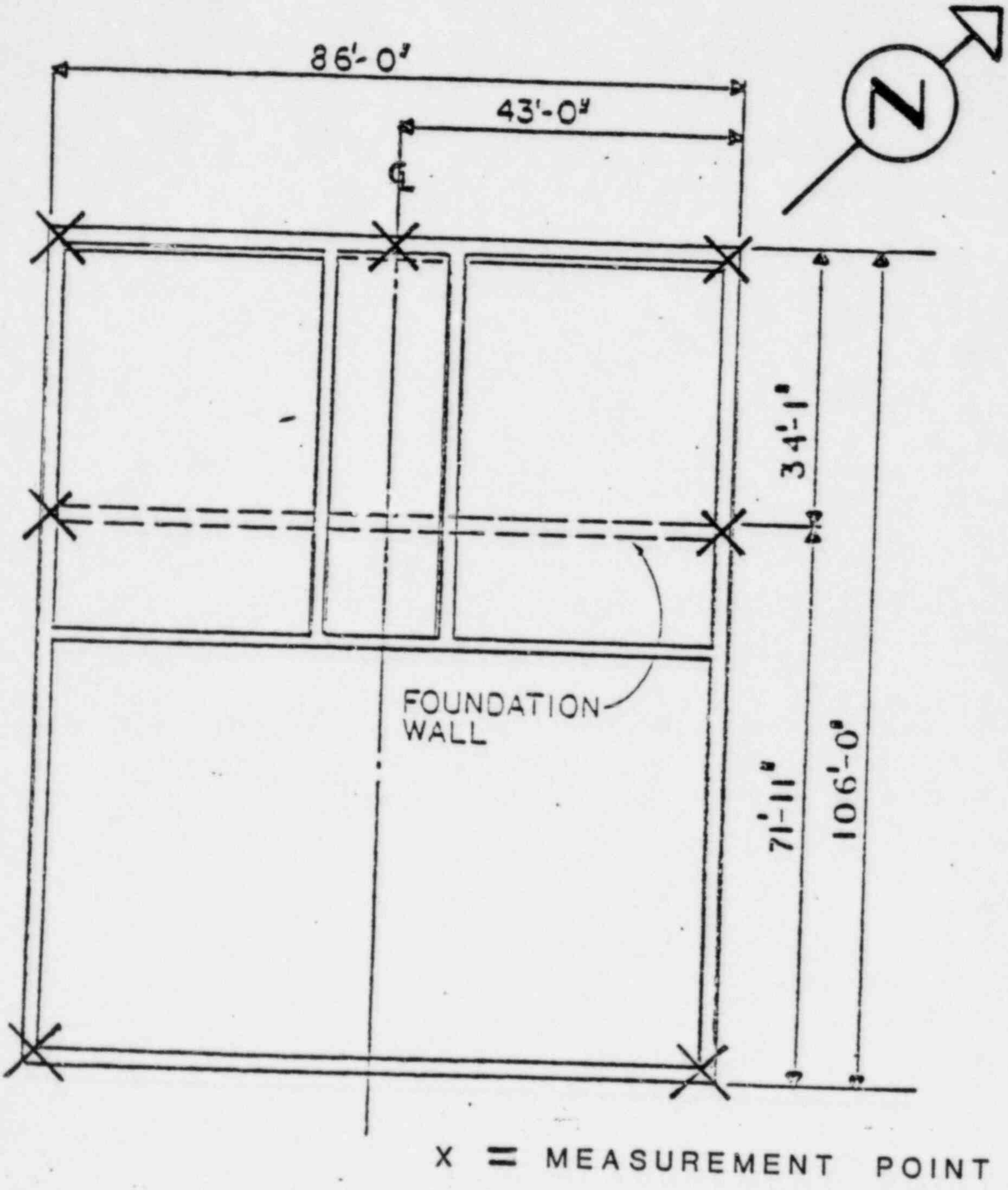


Fig. 13 Service Water Pump Structure Plan Showing Approximate Locations of Displacement Points

mation limits of the structure are not exceeded. In this way the measured displacements will provide a warning of impending structural distress. If displacement limits are reached, underpinning operations should be stopped until remedial measures are evaluated.

It is also recommended that the time history of displacements be submitted on a regular basis to a consultant familiar with reinforced concrete behavior and design. The consultant will provide recommendations on trends observed in the data. Prior to the start of underpinning operations and distortion monitoring, the consultant should review details of the monitoring plan.

Crack Monitoring

As a supplement to the displacement monitoring program periodic visual inspections of the Service Water Pump Structure 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 on a periodic basis by qualified personnel. In addition a consultant knowledgeable in reinforced concrete design and behavior should inspect the Service Water Pump Structure at selected construction milestones. Personnel who monitor cracking should be instructed in crack mapping techniques by the consultant prior to start of operations.

The following criteria should be used for evaluation of observed crack widths.

1. If a new crack develops that is wider than 0.010 in. a consultant should evaluate significance of the new

cracking. Within two hours after observation of the crack the consultant should provide a verbal report recommending whether underpinning operations should stop or continue. The verbal report should be confirmed with a written report within five days.

2. If any crack exceeds 0.030 in. in width a consultant should evaluate significance of the cracking. Within two hours after observation of the crack the consultant should provide a verbal report recommending whether underpinning operations should stop or continue. The verbal report should be confirmed with a written report within five days.

3. If development of yield strain in the reinforcement is inferred from any observed crack, underpinning operations should be stopped immediately. Individual criteria will be recommended by the consultant for the structure. If criteria are exceeded a consultant should evaluate significance of the cracking. Within two hours after observation of the crack the consultant should provide a verbal report recommending whether underpinning operations should resume. The verbal report should be confirmed by a written report within five days.

The following criteria should be used in evaluation of the significance of cracks that develop in the Service Water Pump Structure:

1. Geometry of member
2. Amount and distribution of reinforcement in the member
3. Material properties of the member
4. Function of the member
5. Magnitude and distribution of loads on the member
6. Construction technique
7. Sequence of construction
8. Crack location and distribution
9. Crack size
10. Interaction of multiple cracks

Basically, these criteria outline a procedure that requires the function and load carrying mechanism of the member or structure to first be defined. Then the influence of cracks on the path of load distribution is determined. In this way the cause of cracking is defined and the influence of cracking on future load carrying capacity of the structure can be evaluated.

In evaluating cracks in reinforced concrete structures it is not sufficient to base conclusions on a single criteria such as crack width. The overall crack pattern including location and direction of cracks, length and width of cracks, and interrelationship between multiple cracks must be considered. The pattern of cracking provides significant clues with regard to causes of cracks and their effects on future performance.

SUMMARY AND CONCLUSIONS

This report presents an evalution of the significance of cracks observed in the Service Water Pump Structure at Midland Nuclear Power Plant Units 1 and 2. Cracks observed in this structure by Bechtel personnel and by Construction Technology Laboratories personnel are attributed to restrained volume changes that occur during curing and drying of concrete. No indications of structural distress were observed during site visits. While occurrence of stress related cracking because of differential building settlement cannot be completely dismissed, it does not appear that such hypothesized settlements were a primary cause of cracks observed in the structure. Calculations based on section geometry and material properties indicate that structural reinforcement provided in primary load carrying walls at selected crack locations has sufficient capacity to offset the loss of tensile stress attributed to concrete.

A program for monitoring structural integrity of the Service Water Pump Structure during implementation of remedial measures to underpin the structure is described. It is recommended that measured displacements be used as the primary means of monitoring behavior of the structure. It is also recommended that periodic displacement measurements be supplemented with visual inspections to monitor cracking in the structure. Displacement and crack monitoring should be reviewed by a consultant knowledgeable in reinforced concrete behavior and design.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 381-77)," American Concrete Institute, Detroit, 1977.

APPENDIX A

STRENGTH OF CRACKED REINFORCED CONCRETE MEMBERS

APPENDIX A

TABLE OF CONTENTS

APPENDIX A

STRENGTH OF CRACKED REINFORCED CONCRETE MEMBERS

by

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

INTRODUCTION

Cracking is an inherent characteristic of reinforced concrete structures. The existence of cracks is not necessarily indicative of structural distress. The objective of this report is to clarify the relationship between cracking and strength of reinforced concrete members. The relationship will be demonstrated by examining the response of selected structural members that have been loaded to destruction in the laboratory. To provide a cross-section of data, results from tests on structural walls, beams, and containment elements will be considered.

TESTS OF STRUCTURAL WALLS

Reinforced concrete structural walls are commonly used as lateral load resisting elements in buildings. Both "low-rise" walls, which act as deep beams, and "high-rise" walls, which undergo significant flexural yielding, have been tested in the laboratory.

*Respectively, Manager, Construction Methods Section and Divisional Director, Engineering Development Division, Construction Technology Laboratories, a Division of the Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077.

Tests of "Low-Rise" Structural Walls

Figure 1 shows the test setup used to apply reversing loads to eight specimens representing "low-rise" structural walls with boundary elements.[(1)*]

Principal variables in this test program included amount of flexural reinforcement, amount of horizontal wall reinforcement, amount of vertical wall reinforcement, and height-to-horizontal length ratio of the wall. Flexural reinforcement was varied from 1.8 to 6.4% of the boundary element area. Horizontal and vertical wall reinforcement were varied from 0 to 0.5% of the wall area. Height-to-horizontal length ratio of the wall was varied from 1:4 to 1:1. The test program was designed to determine effects of load reversals. Data obtained also provided information on the relationship between cracking and strength.

Principal test results for the eight walls are shown in Table 1. For all specimens, except B5-4, the maximum nominal shear stress in the wall exceeded the stress at first observed shear cracking by a factor of at least 2.4. For Specimen B5-4, which contained no vertical reinforcement in the diaphram, the maximum nominal shear stress exceeded the stress at first shear cracking by a factor of 1.5. The ratio of maximum nominal shear force to first shear cracking even exceeded 2.5 for Specimen B4-3 which contained no horizontal reinforcment. For each of the "low-rise" walls tested, measured capacity exceeded

*The superscript numbers in parentheses refer to references listed at the end of this report. A copy of each reference is attached.

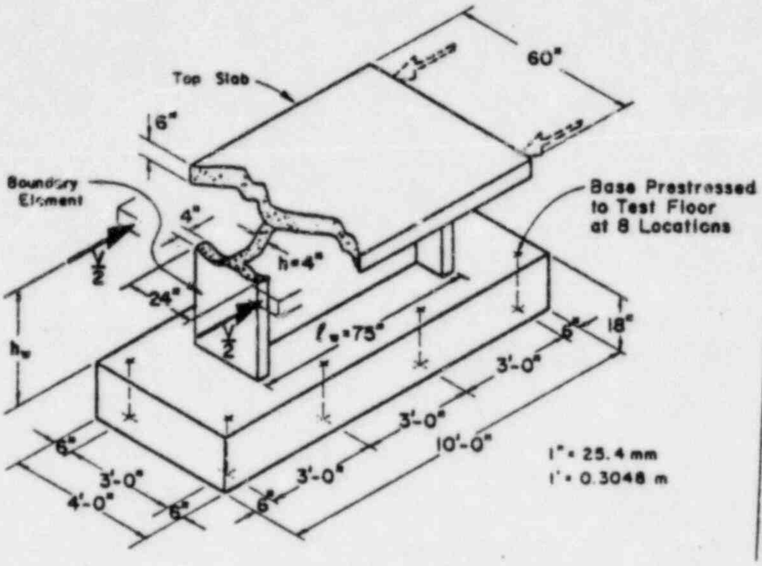


Fig. 1 - Setup for Tests of "Low Rise" Walls (1)

TABLE 1 - Principal Test Results (1)

Specimen	Variable(1)	First Shear Cracking: Shear stress v_{cr}, psi	$\frac{v_{cr}}{\sqrt{f'_c}}$	Deflection $\Delta \ell$ in.	$\frac{\Delta \ell}{h_w}$	Ultimate Load: Shear stress v_u, psi	$\frac{v_u}{\sqrt{f'_c}}$	Deflection $\Delta \ell$, in.	$\frac{\Delta \ell}{h_w}$	End of Test: Shear stress v_m, psi	$\frac{v_m}{\sqrt{f'_c}}$
B1-1	ρ = 1.8%(2)	420	6.5	0.027	0.00072	1,010	15.5	0.23	0.0061	280	4.4
B2-1	ρ = 6.4%(2)	240	4.9	0.016	0.00043	767	15.8	0.26	0.0069	270	5.5
B3-2	Control	330	5.2	not measured		881	14.1	0.21	0.0056	190	3.0
B3-2R	Repair	190	3.3	0.020	0.00053	676	11.5	0.49	0.0130	230	4.0
B4-3	$\rho_h = 0$	320	6.1	0.015	0.00040	810	15.4	0.20	0.0053	160	3.0
B5-4	$\rho_n = 0$	330	5.2	0.012	0.00032	538	8.3	0.20	0.0053	280	4.3
B6-4	ρ_n = 0.25%	280	5.0	0.013	0.00035	686	12.3	0.23	0.0061	190	3.5
B7-5	$h_w/\ell_w = 1/4$	330	5.4	0.006	0.00032	906	14.8	0.16	0.0085	350	5.7
B8-5	$h_w/\ell_w = 1$	200	3.5	0.027	0.00036	704	12.1	0.42	0.0056	150	2.6

(1) Except as indicated below, all specimens had the following characteristics: $h_w/\ell_w = 1/2$, ρ_h = 0.5%, ρ_n = 0.5%, ρ = 4.1%.

(2) Specimens subjected to static loading. All other specimens subjected to load reversals.

Note: 1 in. = 25.4 mm; 1,000 psi = 70.3 kg per square centimeter that calculated by American Concrete Institute Building Code Requirements for Reinforced Concrete.

Figure 2 shows crack patterns in the "low-rise" walls at the ultimate load levels listed in Table 1. The inclined cracks are indicative of shear stresses that predominate in short cantilever members. It is apparent that the presence of cracks does not necessarily indicate loss of structural capacity. Even with the extensive cracking shown in Fig. 2, the walls were carrying maximum applied loads. For a particular section geometry and applied loading, structural capacity is a function of the amount and distribution of reinforcement.

There was no evidence that reversing loads caused residual stresses that reduced strength of the walls. Additional data on these tests are given in Reference 1.

Tests of "High-Rise" Structural Walls

Tests reported in References 2, 3, and 4 were conducted to obtain data on strength and deformation capacity of structural walls subjected to significant numbers of inelastic load reversals. Effects of load history, section shape, vertical and horizontal reinforcement, confinement reinforcement, moment-to-shear ratio, axial compressive stress, and concrete strength were considered.

Figure 3 shows the setup used for tests of "high-rise" walls. The walls were tested as vertical cantilever members with forces applied through the top slab. The behavior of one of the test specimens is described in detail in the following

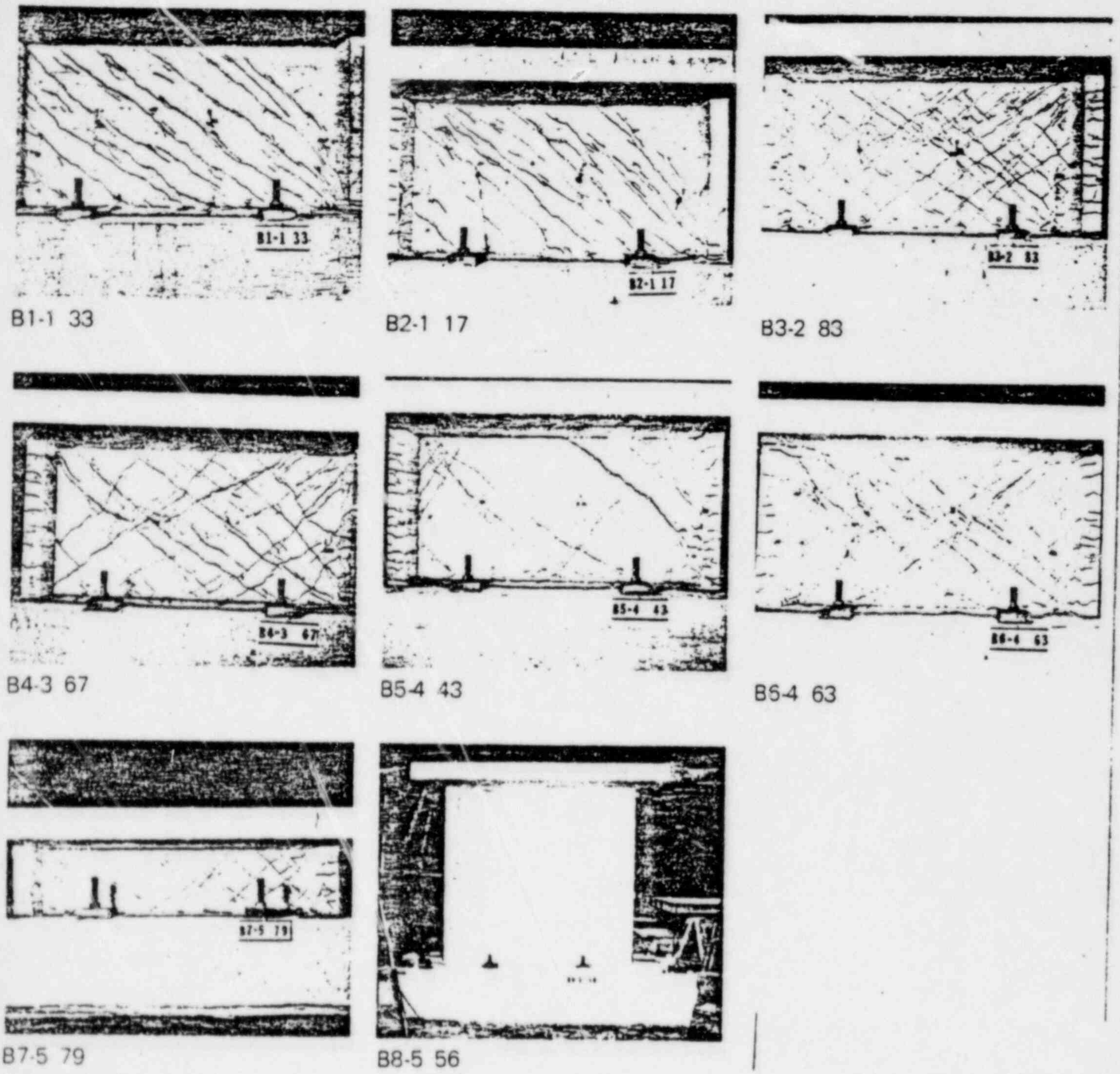


Fig. 2 "Low-Rise" Wall Test Specimens at Ultimate Load (1)

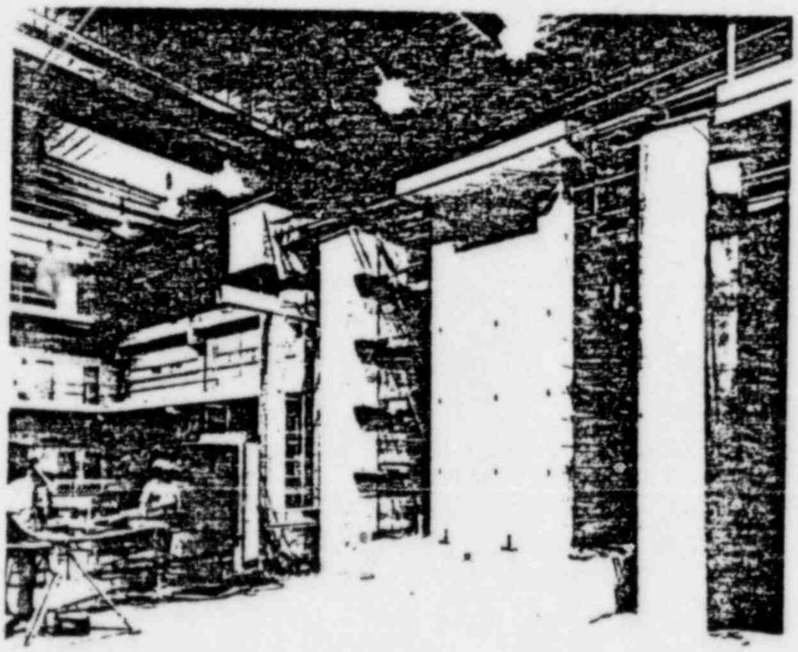


Fig. 3 Setup for Tests of "High-Rise" Walls

paragraphs. This behavior illustrates the influence of cracks that developed during the tests. Additional data on other specimens can be obtained in References 2, 3, and 4.

Figure 4 shows the measured load vs deflection relationship for Specimen B3. This was a barbell shaped specimen which represented a wall with column boundary elements at each end. As can be seen in Fig. 4, the wall was subjected to increasing levels of load reversals. The test consisted of 42 complete load cycles.

Initial cracking was observed in the fourth cycle at a load of 28 kips. First yielding in the vertical flexural reinforcement occurred in Cycle 10 at a load of 45 kips. Maximum measured crack widths were 0.012 in. in the tension boundary element and 0.025 in. across a diagonal crack in the web.

Figure 5 is a photograph of Specimen B3 at Load Stage 112. This load stage, which is marked on Fig. 4, represents a point in the test when the specimen was unloaded. There were no applied in-plane horizontal forces. Figure 5 shows the intersecting pattern of cracks in the lower six feet of the wall after the first 21 load cycles.

From Load Stage 112, loads were increased in a positive direction until Load Stage 117 was reached. Figure 6 shows the condition of the specimen at Load Stage 117. At Load Stage 117, maximum measured crack width in the tension boundary element was 0.07 in. and maximum measured crack width in the wall web was approximately 0.16 in. It should be noted that, at this load stage, the wall had been pushed to a lateral deflection of more than three times its yield deflection.

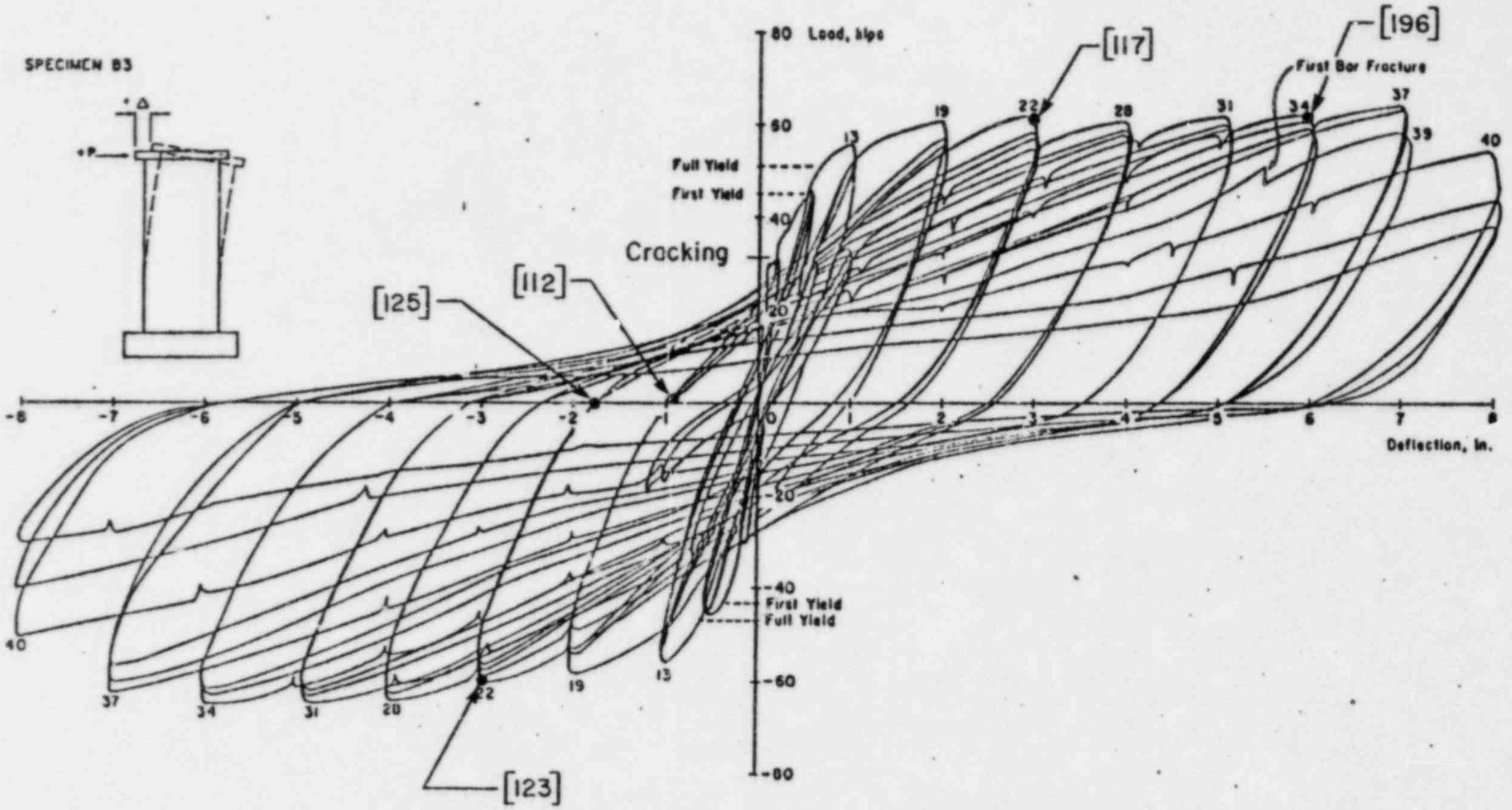


Fig. 4 Load-Deflection Relationship for Specimen B3

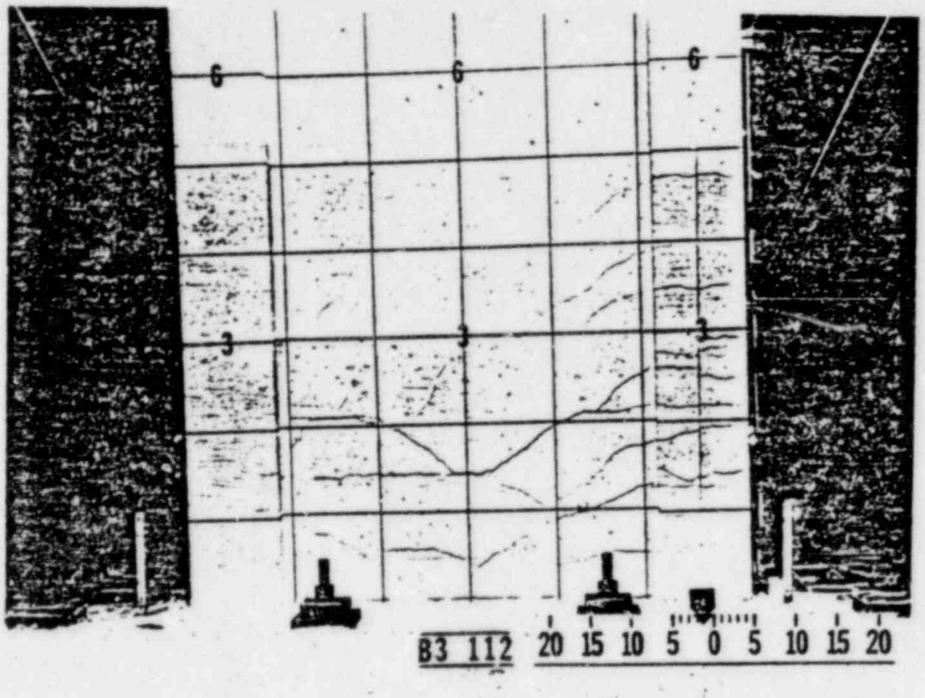


Fig. 5 Specimen B3 at Load Stage 112

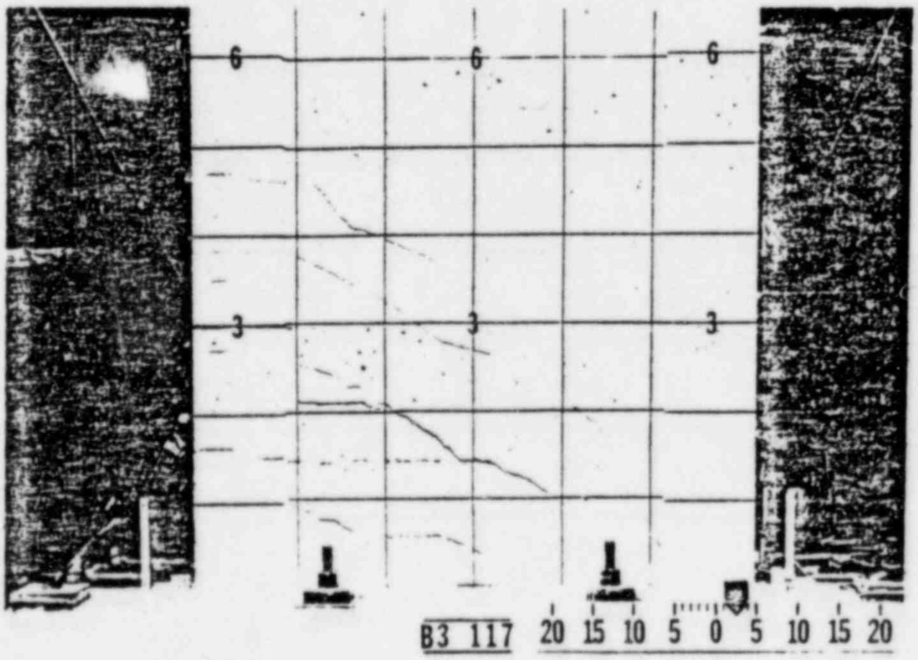


Fig. 6 Specimen B3 at Load Stage 117

After Load Stage 117 was reached, the wall was unloaded and pushed in the opposite direction until Load Stage 123 was reached. Figure 7 shows the condition of Specimen B3 at Load Stage 123. At this load stage, the maximum crack width measured in the tension column was approximately 0.07 in. and the maximum measured crack width in the wall web was 0.16 in. When the wall was again unloaded, to Load Stage 125, the crack pattern shown in Fig. 8 resulted. It is clearly evident from the behavior of Specimen B3 (and from other specimens tested) that the presence of cracks did not prevent the walls from maintaining their structural integrity and developing their nominal strength.

Figure 9 shows Specimen B3 at Load Stage 196. This load stage is also indicated in Fig. 4. The cracking pattern in Fig. 9 is indicative of severe distress in the member, yet at this stage the wall carried its maximum load which corresponded to approximately $3.1\sqrt{f'_c}$. For purposes of comparison, the design strength this member calculated in accordance with the American Concrete Institute Building Code is $2.3\sqrt{f'_c}$.

A question that occurs in evaluating cracked reinforced concrete structures is whether residual stresses associated with the occurrence of cracks influence strength of the member. It is evident from the behavior of Specimen B3 that internally balanced residual stresses, such as those existing when the specimen was unloaded, did not influence strength.

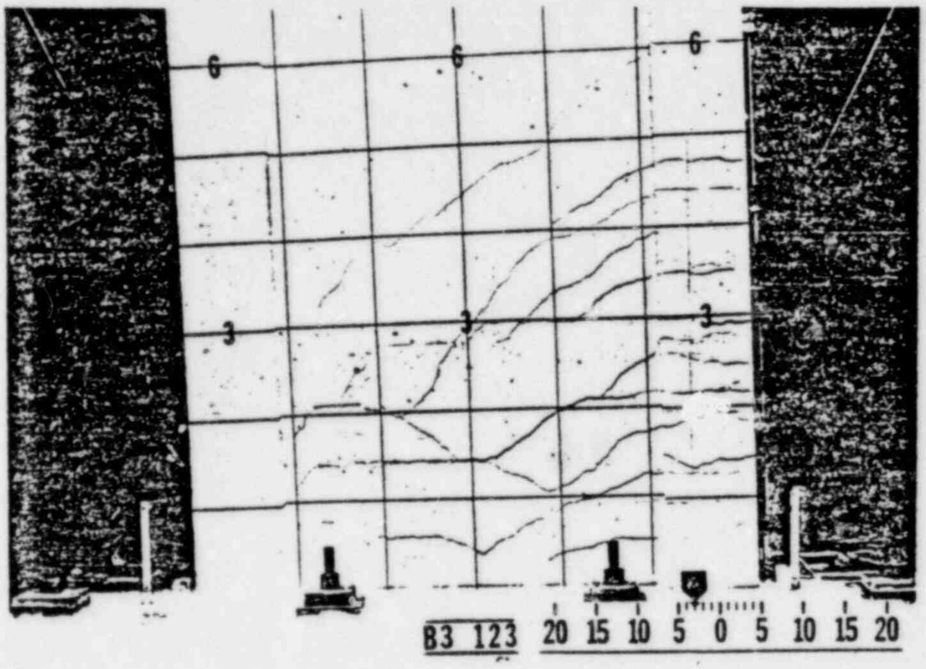


Fig. 7 Specimen B3 at Load Stage 123

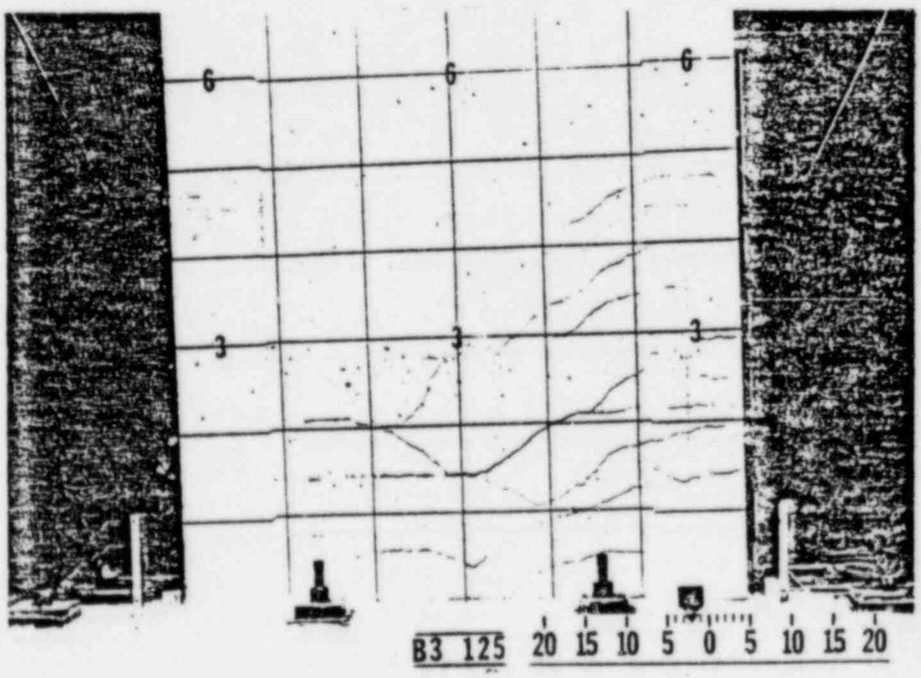


Fig. 8 Specimen B3 at Load Stage 125

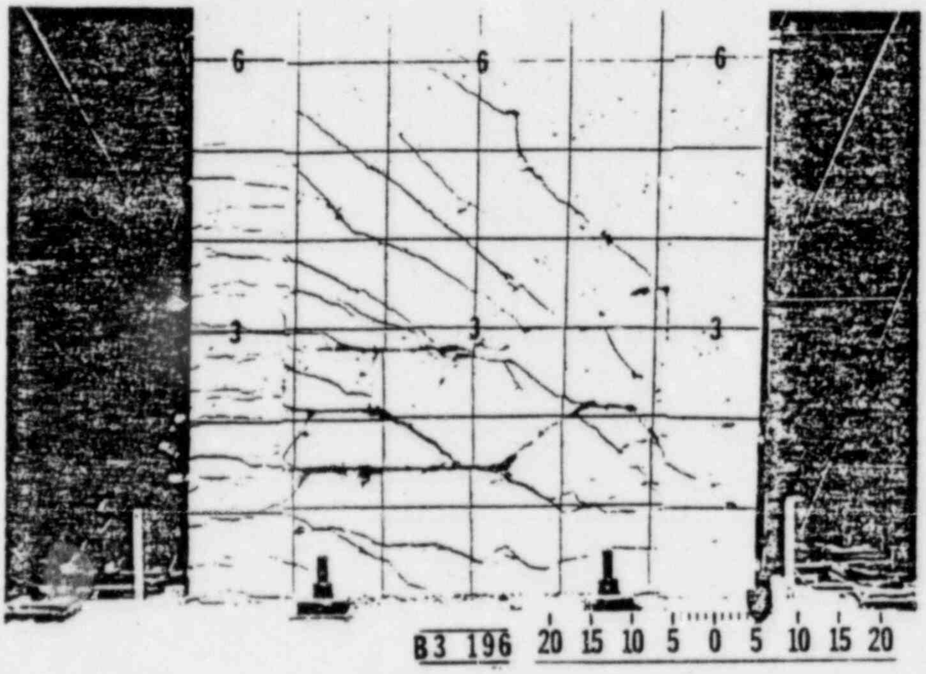


Fig. 9 Specimen B3 at Load Stage 196

TESTS OF BEAMS

Background data on strength of cracked reinforced concrete members can also be obtained from tests on reinforced concrete beams. Data from tests reported by Scribner and Wight are shown in Figs. 10 and 11.[5]

Figure 10 shows the load vs displacement curve for a reinforced concrete beam element that contained positive and negative steel. The beam was subjected to increasing levels of fully reversed load cycles. Yielding occurred in the first load cycle as indicated in Fig. 10.

Figure 11 illustrates crack patterns that developed during the first inelastic loading and during subsequent load reversals. As increasing numbers of load cycles were applied, the entire beam moment at the face of the column was carried by a force couple between the top and bottom layers of longitudinal steel. Thus, applied moments were primarily resisted by the positive and negative longitudinal reinforcement.

Under load reversals a complete crack plane, labeled A-B-C in Fig. 11, formed through the beam. This crack plane did not prevent the beam from transferring load. During the final stages of the test, increasing numbers of inelastic load reversals caused concrete near the face of the column to abrade and eventually disintegrate. This resulted in a "slip plane" along the beam at the face of the column. The significance of such a slip plane is related to the number of inelastic load reversals and the level of shear stress on the beam. The existence of

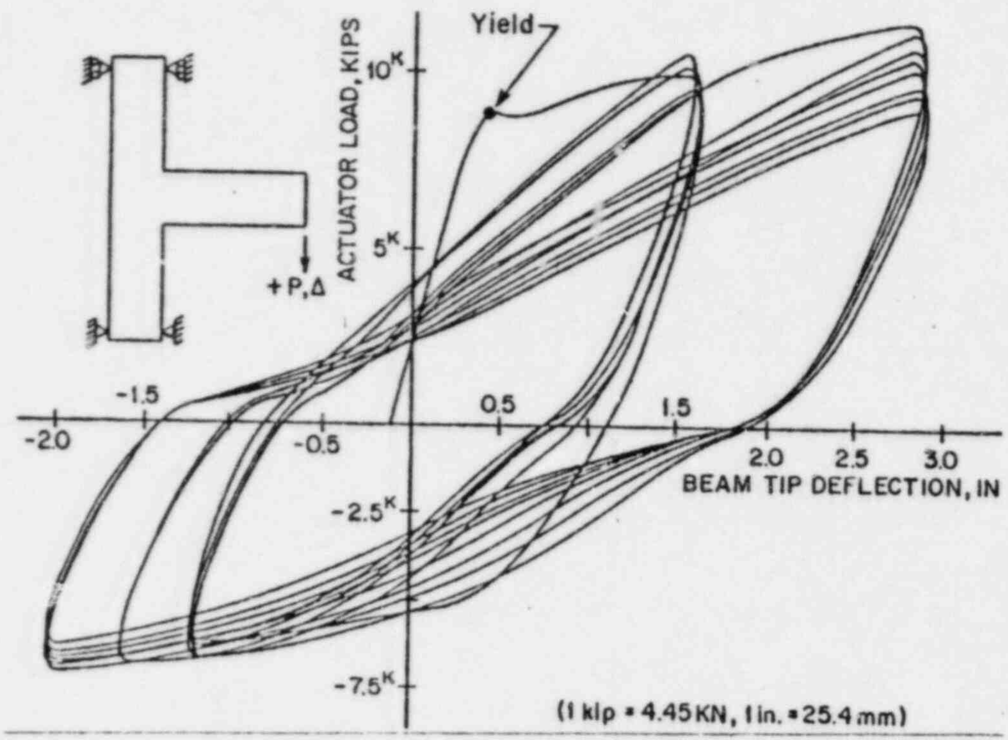


Fig. 10 Load vs Displacement Curve - Specimen 1 (After Ref. 5)

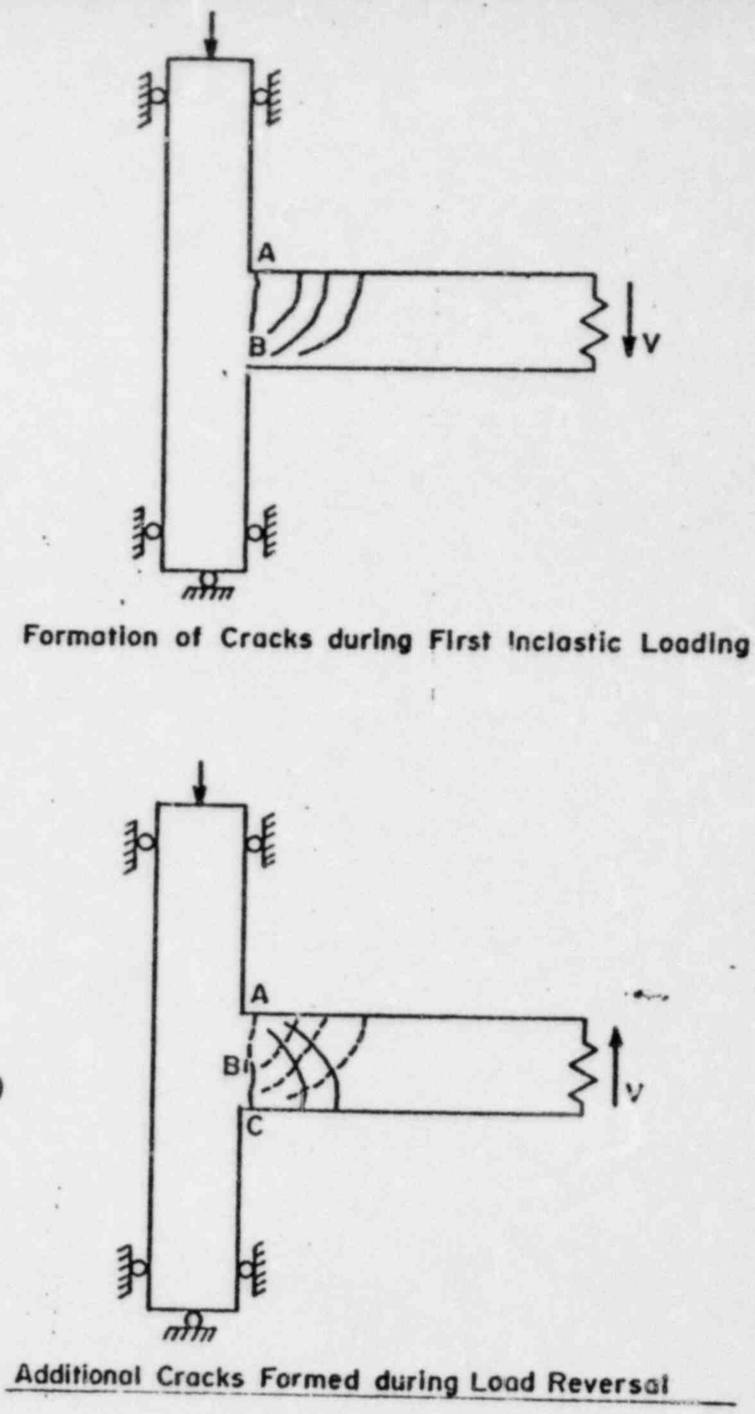


Fig. 11 Crack Pattern (After Ref. 5)

the crack plane did not become significant until repeated numbers of inelastic cycles were applied.

Additional data on beam tests can be obtained from References 6 and 7. In addition, tests of beam-column joints reported in Reference 8 also provide useful information.

Results shown in Fig. 10 indicate that beams can transfer flexural and shear loads even with the presence of cracks through their entire depth. Tests conducted at the University of Washington have shown that the effectiveness of web reinforcement in resisting shear in reinforced concrete beams is not affected by axial force in the beam.[9] These tests were conducted on beams subjected to combined axial tension, bending, and shear. Results indicated that effectiveness of web reinforcement is not reduced by the presence of axial tension. In the tests, applied axial load was sufficient to cause cracking prior to the application of transverse load. For all beams with web reinforcement, measured load capacity of the precracked beams exceeded values calculated in accordance with the American Concrete Institute Building Code.

TESTS OF CONTAINMENT ELEMENTS

Another series of tests that can be used to demonstrate the strength of cracked reinforced concrete members is reported in an experimental program to investigate shear transfer in cracked containments without diagonal reinforcement.[10] The test setup was designed and constructed to simulate boundary conditions of a wall element of a pressurized containment subjected to tangential shear stresses. Forces on an element in a containment wall are illustrated in Fig. 12. Figures 13 and 14 show the test setup used for the experiments. The experimental program included monotonic and reversing load tests on large-scale specimens subjected to biaxial tension and shear. Specimens were 5-ft square and 2-ft thick with No. 14 and No. 18 reinforcement.

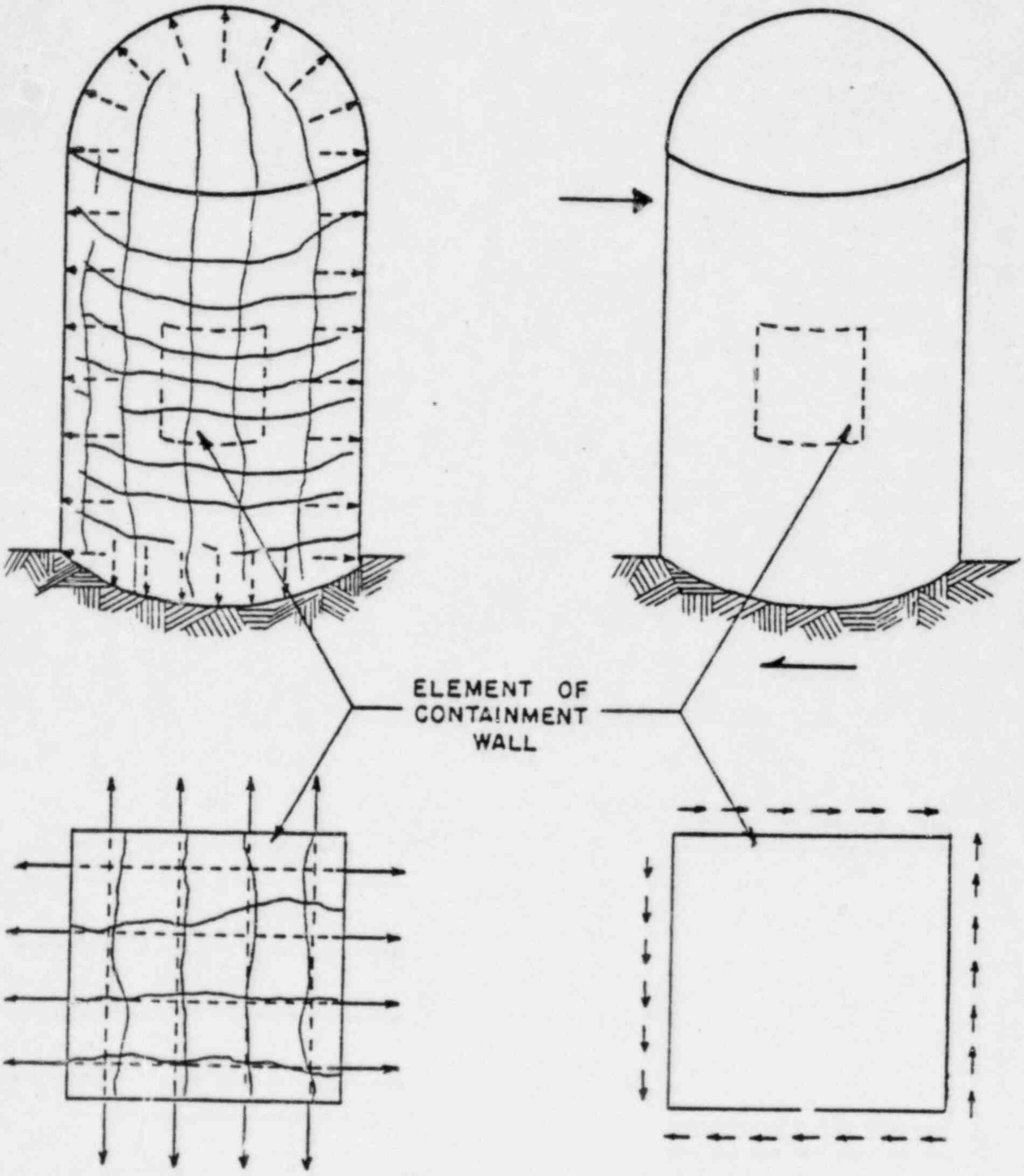
This discussion includes a description of one of the test specimens. Additional data are available in Reference 10.

Figure 15 shows the crack pattern observed in Specimen MB1 after reinforcement in the element was loaded to obtain a tension stress of 54 ksi in the steel. This stress corresponds to 90% of the yield stress of the reinforcement. Crack width measurements made on the specimen after biaxial tension was applied indicated a maximum width of approximately 0.036 in.

Figures 16 and 17 show the crack pattern and nominal shear stress vs shear distortion relationship for Specimen MB1. Shear forces were applied while constant biaxial tension was maintained. It is evident from Fig. 17 that the reinforced concrete element was capable of transferring shear forces even though it was traversed by biaxial tension cracks through the complete thickness.

SUMMARY AND CONCLUSIONS

Test data presented in this report demonstrate that cracks in an adequately reinforced concrete member do not prevent the member from developing its expected strength. Adequate reinforcement for the test specimens was determined in accordance with current code provisions. Data presented also indicate the



a) Biaxial Tension Due to Internal Pressurization

b) Shear Forces Due to Lateral Load

Fig. 12 Forces on Element in Containment Wall (10)

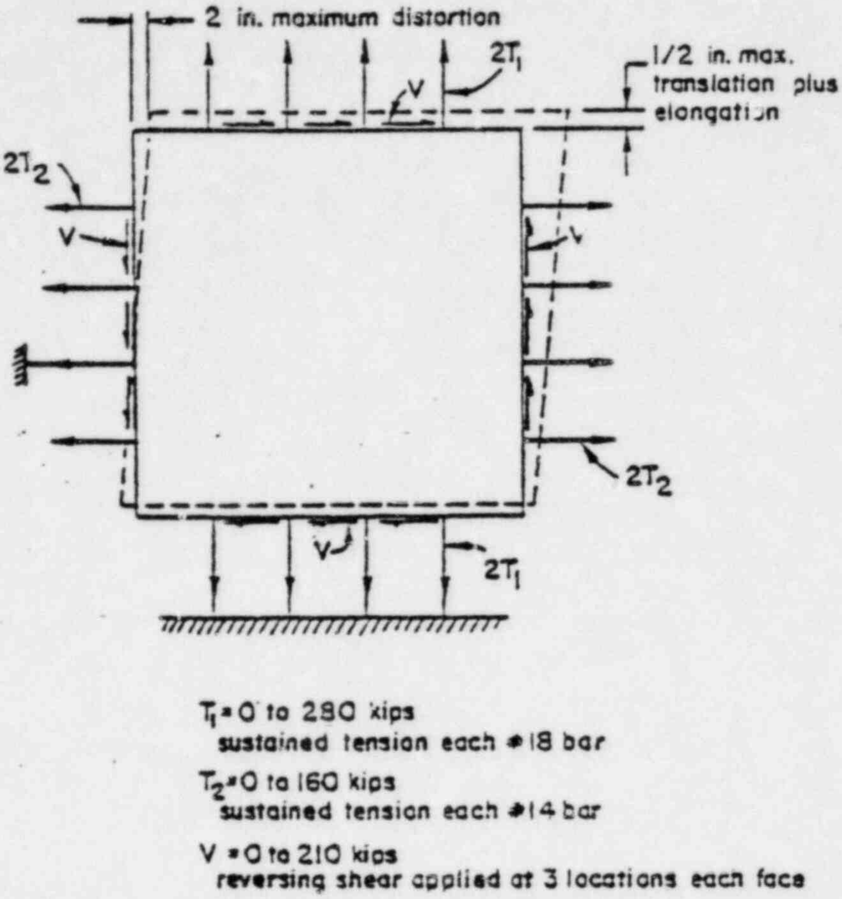
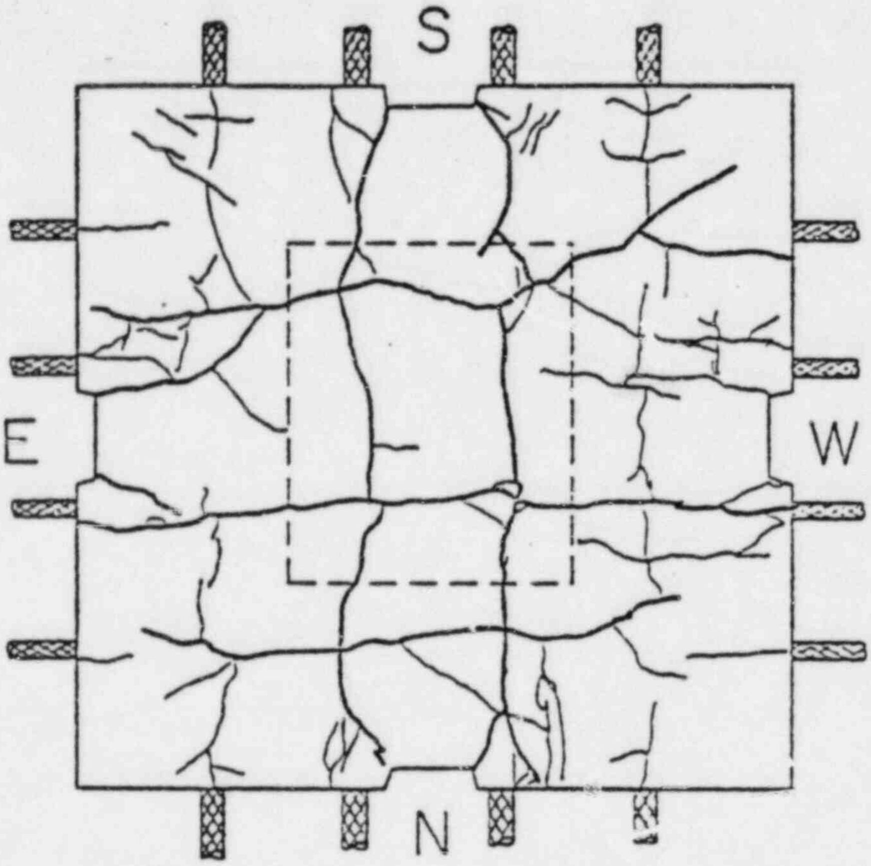


Fig. 13 Loading System Capabilities (10)

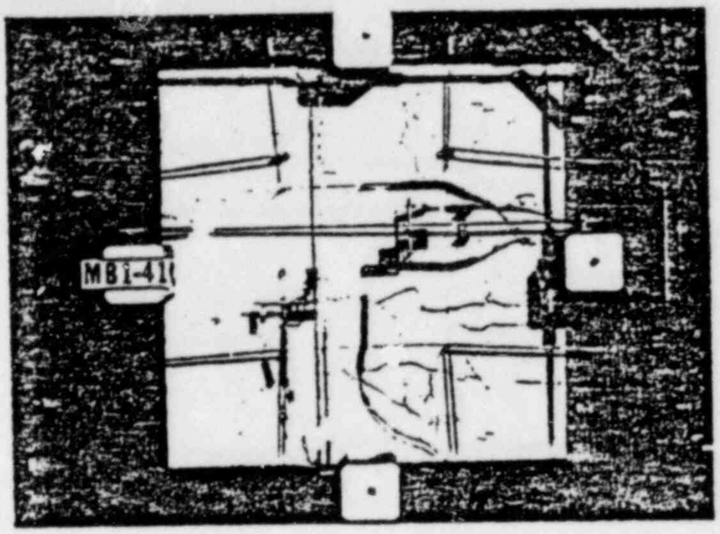
Fig. 14 Test Setup for Containment Element (10)



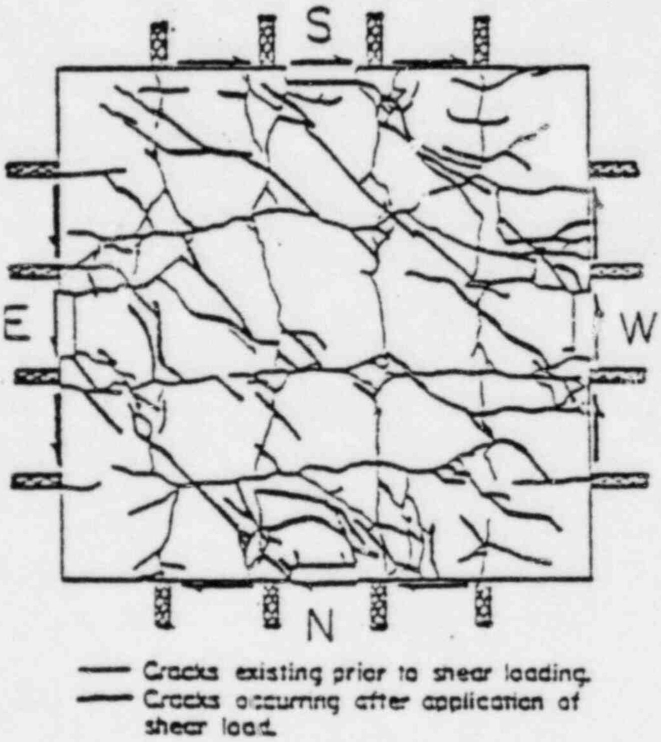
Cracks considered to Penetrate the Full Thickness of the Specimen.

Cracks considered to Penetrate only the Cover Layer.

Fig. 15 Crack Pattern After Biaxial Tension of 54 ksi in Containment Element Specimen MB1 (10)



a) Just Prior to Loss of Shear Capacity



b) Crack Pattern Just Prior to Maximum Shear Load

Fig. 16 Crack Pattern in Specimen MB1 (10)

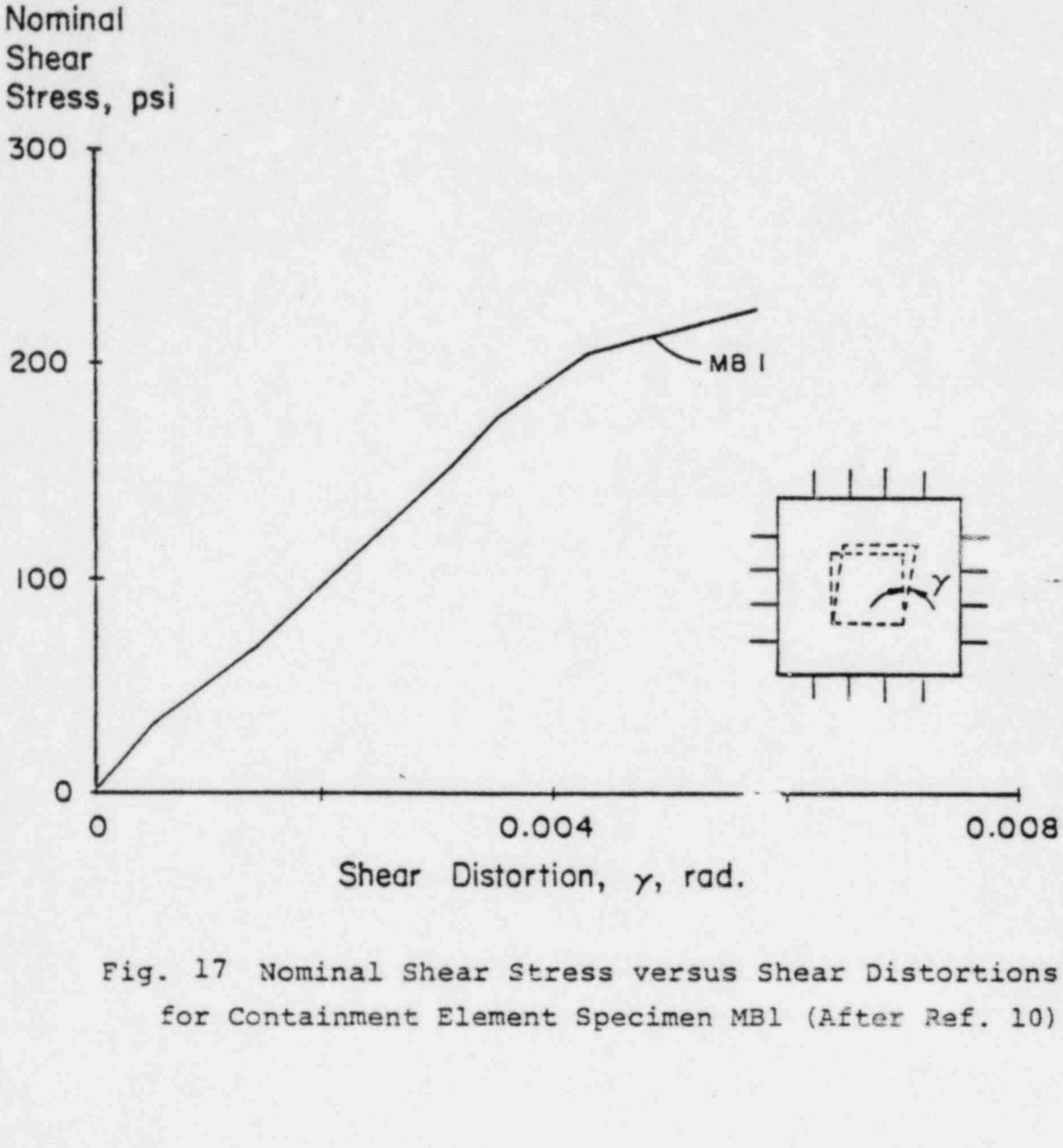


Fig. 17 Nominal Shear Stress versus Shear Distortions for Containment Element Specimen MB1 (After Ref. 10)

level or severity of cracking associated with severe stress in reinforced concrete members. Obviously the presence of cracks in a reinforced concrete structure cannot be summarily dismissed as insignificant. The pattern of cracking and crack widths should be evaluated to determine their significance. However, the mere presence of a crack does not necessarily indicate that the integrity of the structure is in jeopardy, or that its load-carrying capacity has been reduced.

REFERENCES

1. Barda, F., Hanson, J.M., and Corley, W.G., "Shear Strength of Low-Rise Walls with Boundary Elements," Special Publication SP-53, Reinforced Concrete Structures in Seismic Zones, American Concrete Institute, Detroit, 1977, 496 pp.

2. Corley, W.G., Fiorato, A.E., and Oesterle, R.G., "Structural Walls," Special Publication, C.P. Siess Symposium, American Concrete Institute, Detroit, 1979 (to be published).

3. Oesterle, R.G., Fiorato, A.E., and Corley, W.G., "Reinforcement Details for Earthquake-Resistant Structural Walls," Concrete International, December 1980, pp. 55-66.

4. Oesterle, R.G, Fiorato, A.E., and Corley, W.G., "Effects of Reinforcement Details on Seismic Performance of Walls," Proceedings of a Conference on Earthquakes and Earthquake Engineering: The Eastern United States, Vol. 2, Ann Arbor Science Publishers, Inc., 1981, pp. 685-707.

5. Scribner, C.F. and Wight, J.K., "A Method for Delaying Shear Strength Decay of RC Beams," Proceedings of a Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, Vol. 3, University of California, Berkeley, June 1978, pp. 1215-1241.

6. Wight, J.K. and Sozen, M.A., "Strength Decay of RC Columns Under Shear Reversals," Journal of the Structural Division, ASCE, May 1975, pp. 1053-1065.

7. Brown, R.H. and Jirsa, J.O., "Reinforced Concrete Beams Under Load Reversals," Journal of the American Concrete Institute, Vol. 68, No. 5, May 1971, pp. 380-390.

8. Hanson, N.W. and Conner, H.W., "Tests of Reinforced Concrete Beam-Column Joints Under Simulated Seismic Loading," Research and Development Bulletin RD012, Portland Cement Association, 1972, 12 pp.

9. Haddadin, M.J., Hong, S.T., and Mattock, A.H., "Stirrup Effectiveness in Reinforced Concrete Beams with Axial Force," Journal of the Structural Division, ASCE, September 1971, pp. 2277-2297.

10. Oesterle, R.G. and Russell, H.G., "Shear Transfer in Large Scale Reinforced Concrete Containment Elements," Report No. 1, NUREG/CR-1374, Construction Technology Laboratories, a Division of the Portland Cement Association, prepared for U.S. Nuclear Regulatory Commission, Washington, D.C., April 1980.

ATTACHMENT